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ANSI/SDI AISI S100-2024

North American Specification for the Design of Cold-Formed Steel Structural Members

Approved American National Standard ANSI

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2024 Edition

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First Printing, December 2024

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Florence, South Carolina 29503

PREFACE

(This Preface is not part of the ANSI/SDI AISI S100-2024, *North American Specification for the Design of Cold-Formed Steel Structural Members*, but is included for informational purposes only.)

This Standard is a revision of ANSI/AISI S100-2016(R2020)w/S3-2022.

This Standard has been developed as a consensus document for the design of cold-formed steel members and structures. The intention is to provide criteria for routine use and not to provide specific criteria for infrequently encountered problems, which occur in the full range of structural design. The Symbols and Appendices to this Standard are an integral part of the Standard. A non-mandatory Commentary has been prepared to provide background for the Standard provisions and the user is encouraged to consult it. Additionally, non-mandatory User Notes may be interspersed throughout the Standard to provide concise and practical guidance in the application of the provisions. The user is cautioned that professional judgment must be exercised when data or recommendations in the Standard are applied, as described more fully in the disclaimer notice preceding this Preface.

SYMBOLS

Symbol	Definition	Section
A	Full, unreduced *cross-sectional area* of member	A1.3, E3.3, I6.2.3, I6.2.4, 2.3.1, 3.2
A_{av}	Active area subject to shear (parallel to force)	J6.3
A_{avg}	Weighted average of *cross-sectional area*	2.3.1
A_b	$b_1t + A_s$, for bearing stiffener at interior support or under concentrated load, and $b_2t + A_s$, for bearing stiffeners at end support	G7.1
A_b	Gross *cross-sectional area* of bolt	J3.4
A_c	$18t^2 + A_s$, for bearing stiffener at interior support or under concentrated load, and $10t^2 + A_s$, for bearing stiffeners at end support	G7.1
A_e	*Effective area* at *stress* F_n	A1.3, E3.1, E3.3, I1.2.3,
A_e	*Effective area* of bearing stiffener	G7.1
A_e	Effective *net area* subject to tension	J6.2
A_f	*Cross-sectional area* of *flange* stiffener	2.3.3, 2.3.3.1
A_g	*Gross area* of cross-section	A1.3, C1.1.1.3, D2, E2, E3.1, E4, J6.2, 2.1, 2.3.1, 2.3.1.1, 2.3.2.1, 2.3.3.1
A_g	*Gross area* of element including stiffeners	1.4.1
A_{gv}	*Gross area* subject to shear	J6.3
A_h	Area of circular hole or slotted hole	G3
A_{net}	*Net area* of cross-section	A1.3, D3, E3.1, E3.2, 2.3.1, 2.3.2.1
A_o	Reduced area due to *local buckling*	E3.3
A_{nv}	*Net area* subject to shear (parallel to force)	J6.1, J6.3
A_p	Gross *cross-sectional area* of roof panel per unit width	I6.4.1
A_{pt}	*Net area* subject to tension perpendicular to force	J6.3
A_s	*Cross-sectional area* of bearing stiffener	G7.1
A_s	*Gross area* of stiffener	1.4.1, 1.4.1.1, 1.4.1.2
A_{st}	Active area in the staggered plane	J6.3
A_t	Net tensile area	M4
A_w	Area of *web*	G2.3, 2.1
$A_{web,gross}$	*Web* surface area along the member length	2.3.3.3
$A_{web,net}$	*Web* surface area along member length subtracting the hole areas	2.3.3.3
a	Longitudinal distance between centerline of braces	C2.2.1
a	Shear panel length of unreinforced *web* element, or Clear distance between shear stiffeners of reinforced *web* elements	G2.3, G4.1

SYMBOLS

Symbol	Definition	Section
a	Distance between stiffeners for stiffened *webs* or twice the distance from the end of the section to the center of the hole for transversely unstiffened webs	G3
a	Spacing of intermediate fasteners or welds carrying shear between sections	I1.2.2.1
a	Fastener distance from outside *web* edge	I6.2.3
a	Longitudinal distance between centerline of bracing	C2.2.1
a	Major diameter of the tapered *PAF* head	J5, J5.2.3
a	Coefficient defined in Table 4.2.3.1.1-1	4.2.3.1.1
a_d	Variable	3.4
a_ℓ	A variable calculated per Eq. 3.3-4	3.3
a_0, a_1, a_2	Coefficients	G3
B_a	*Available bimoment strength* [*factored resistance*] determined in accordance with Section G8.1	H4
B_c	Term for determining tensile *yield stress* of corners	A3.3.2
B_{crf}	Bimoment at critical elastic *buckling* of the *flange* determined using rational elastic *buckling* analysis in accordance with Appendix 2, including the influence of holes if applicable	G8.1
B_{crw}	Bimoment at critical elastic *buckling* of the *web* determined using rational elastic *buckling* analysis in accordance with Appendix 2, including the influence of holes if applicable	G8.1
B_1	Multiplier to account for *P-δ effects*	C1.1.1.1, C1.2.1.1
B_2	Multiplier to account for *P-Δ effects*	C1.2.1.1
B_n	*Nominal bimoment strength* [*factored resistance*]	G8.1
B_p	Plastic bimoment	G8.1
B_y	First yield bimoment	G8.1
$\overline{B}$	*Required bimoment strength* [bimoment due to *factored loads*], determined as required in Section C1, in accordance with *ASD*, *LRFD*, or *LSD load combinations*, taken as positive	H4, I6.1.3
b	*Flat width* of element with edge stiffeners (disregard intermediate stiffeners)	B4.1
b	*Effective design width*	B4.3, 1.1, 1.1.1, 1.1.4, 1.2.1, 1.2.2, 1.3
b	*Flange* width	I6.2.3, I6.2.4, I6.4.1
b	Centerline dimension of *flange*	2.3.3

SYMBOLS

Symbol	Definition	Section
b	Coefficient defined in Table 4.2.3.1.1-1	4.2.3.1.1
b_d	*Effective width* for deflection calculation	1.1, 1.1.1, 1.2.1, 1.2.2, 1.3, 1.4.1.1, 1.4.1.2, 1.4.2
b_d	Variable	3.4
b_e	*Effective width* of elements, located at centroid of element including stiffeners	1.4.1
b_e	*Effective width*, b, determined in accordance with Section 1.1, with f_1 substituted for f and with k determined as given in Section 1.1.2	1.1.2, 1.4.2
b_f	Overall *flange* width	G3
b_f	Out-to-out width of *flange* not connected	J6.2
b_o	Out-to-out width of element with edge stiffeners (disregard intermediate stiffeners)	B4.1
b_o	Out-to-out width of compression *flange* as defined in Figure 1.1.2-2	1.1.2
b_o	Overall width of unstiffened element as defined in Figure 1.2.2-3	1.2.2
b_o	Total *flat width* of stiffened element	1.4.1, 1.4.1.1, 1.4.1.2
b_o	Total *flat width* of edge-stiffened element	1.4.2
b_ℓ	A variable calculated per Eq. 3.3-5	3.3
b_p	Largest sub-element *flat width*	1.4.1, 1.4.1.1, 1.4.1.2
b_{st}	Stiffener flat width perpendicular to the *web*	G4.1
b_w	Out-to-out width of *web* connected	J6.2
b_1, b_2	Portions of e*ffective width*	1.1.2, 1.1.3, 1.3
b_1, b_2	*Effective widths* of bearing stiffeners	G7.1
b_1	Out-to-out width of angle leg not connected	J6.2
b_2	Out-to-out width of angle leg connected	J6.2
C	For compression members, ratio of total corner *cross-sectional area* to total *cross-sectional area* of full section; for flexural members, ratio of total corner *cross-sectional area* of controlling *flange* to full *cross-sectional area* of controlling *flange*	A3.3.2
C	Coefficient	G5
C	Bearing factor	J3.3.1
C_b	Bending coefficient dependent on moment gradient	2.3.1, 2.3.1.2.1, 2.3.1.2.2, 2.3.1.2.3, 2.3.1.4
C_c	Correlation coefficient	K2.1.1
C_f	Constant from Table M1-1	M1, M3, M4
C_h	*Web* slenderness coefficient	G5
C_m	Coefficient assuming no lateral translation of frame	C1.2.1.1

SYMBOLS

Symbol	Definition	Section
C_N	Bearing length coefficient	G5
C_P	Correction factor	B4.2, K2.1.1
C_R	Inside bend radius coefficient	G5
C_s	Coefficient for *lateral-torsional buckling*	2.3.1.2.2
C_w	Torsional warping constant of cross-section	G8.1, 2.3.1
$C_{w,net}$	Net warping constant assuming cross-section *thickness* is zero at hole location(s)	2.3.1
C_{wf}	Torsional warping constant of *flange*	2.3.3, 2.3.3.1, 2.3.3.2
C_y	Compression strain factor	F2.1.1
C_1, C_2, C_3	Axial *buckling* coefficients	I6.2.3
C_1, C_2, C_3, C_4	Coefficients	2.3.4
C1 to C6	Coefficients tabulated in Tables I6.4.1-1 to I6.4.1-3	I6.4.1
C_ϕ	Calibration coefficient	K2.1.1
c	Strip of *flat width* adjacent to hole	1.1.1
c	Coefficient defined in Table 4.2.3.1.1-1	4.2.3.1.1
c_f	Amount of curling displacement	L3
c_i	Horizontal distance from edge of element to centerline of stiffener	1.4.1, 1.4.1.2
c_p	Specific heat	4.2.3.1.2
c_1, c_2	Distance from the centroid to any fiber on the cross-section in the principal coordinate system	3.2
D	Outside diameter of cylindrical tube	E3.3, F2.3, F3.3
D	Overall depth of lip	1.1.4, 1.3
d	*Flat width* of unstiffened element (disregard intermediate stiffeners)	B4.1
d	Out-to-out depth of section in the plane of web	Table G5-2
d	Depth of cross-section	C2.2.1, G6, F3.2, F4, G7.2, I6.2.1, I6.2.3, I6.2.4, I6.4.1, I6.4.2, L3, 1.1.4, 2.3.1.2.1, 2.3.1.2.3
d	Centerline dimension of lip	2.3.3
d	Nominal screw diameter	J4, J4.3.1, J4.3.2, J4.4.1, J4.5.1, J4.5.2
d	Flat depth of lip defined in Figure 1.3-1	1.3
d	Visible diameter of the outer surface of the arc spot weld	J2.2.1, J2.2.2.1, J2.2.2.2, J2.2.4
d	Visible width of arc seam weld	J2.3, J2.3.1, J2.3.2.1, J2.3.2.2
d	Nominal bolt diameter	J3, J3.1, J3.2, J3.3.1, J3.3.2, J3.4, J3.5, J6.2, J6.3

SYMBOLS

Symbol	Definition	Section
d	Fastener diameter measured at near side of embedment or d_s for *PAF* installed such that entire point is located behind far side of the embedment material	J5, J5.2.1, J5.3.1
d_a	Average diameter of arc spot weld at mid-*thickness* of t	J2.2.2.1, J2.2.2.2, J2.2.3, J2.2.4
d_a	Average width of seam weld	J2.3.2.1, J2.3.2.2
d_{ae}	Average embedded diameter, computed as average of installed fastener diameters measured at near side and far side of embedment material or d_s for *PAF* installed such that entire point is located behind far side of embedment material	J5, J5.3.3
d_b	Nominal diameter (body or shank diameter)	M4
d_e	Effective diameter of fused area	J2.2, J2.2.2.1, J2.2.3
d_e	Effective weld diameter	J2.5
d_e	Effective width of arc seam weld at fused surfaces	J2.3.2.1
d_h	Diameter of hole	J3, J6.1, J6.2, J6.3, 1.1.1
d_h	Depth of hole	G3, G6, 1.1.1, 1.1.3
d_h	Screw head diameter or hex washer head integral washer diameter	J4, J4.4, J4.4.2
D	*Specified* dead *load*	4.1.4
$d_{h\text{-}eq}$	Equivalent hole depth for circular and slotted holes	G3
d_o	Out-to-out width of unstiffened element (disregard intermediate stiffeners)	B4.1
$d_{p_{i,j}}$	Distance along roof slope between the ith *purlin* line and the jth anchorage device	I6.4.1
d_s	Reduced *effective width* of stiffener	1.1.4, 1.3
d_s	Nominal shank diameter	J5, J5.1, J5.2.3, J5.3.2, J5.3.3, J5.3.4, J5.3.5
d'_s	*Effective width* of stiffener calculated according to 1.2.1 or 1.2.2	1.3
d_w	Steel washer diameter	J4, J4.4, J4.4.2
d_w	Larger value of screw head or washer diameter	J4.5.1
d'_w	Effective pull-over resistance diameter	J4, J4.4.2
d'_w	Actual diameter of washer or fastener head in contact with retained substrate	J5, J5.2.3
d_1, d_2	Weld offset from flush condition	J2.7
E	Modulus of elasticity of steel, 29,500 ksi (203,000 MPa, or 2,070,000 kg/cm^2)	A3.1.3, E2.1, E3.3, F2.1.1, F2.3, F3.3, G2.3, G4.1, G7.1,

SYMBOLS

Symbol	Definition	Section
		I1.3, I6.2.3, I6.4.1, J2.2.2.1, J5.3.3, L3, 1.1, 1.1.4, 1.3, 1.4.1, 2.3.1, 2.3.1.1.1, 2.3.1.2.1, 2.3.1.2.3, 2.3.1.2.4, 2.3.2.1, 2.3.2.2, 2.3.3.1, 2.3.3.2, 2.3.4, 4.2.3.1
E_T	Modulus of elasticity of steel at temperature (T)	4.2.3.1.5
e	Natural logarithmic base (=2.718)	J5.2.1, K2.1.1
e	Distance between end of each connected part and center of bolt hole	J3.5
e	*Flat width* between first line of connector and edge stiffener	1.1.4
e_{net}	Clear distance between end of material and edge of fastener hole or weld	J6.1
e_{sx}, e_{sy}	Eccentricities of *load* components measured from the shear center and in the x- and y- directions, respectively	C2.2.1
F	Fabrication factor	K2.1.1
F_a	Acceleration-based site coefficient, as defined in NBCC	A3.2.1.1
F_{bs}	Base *stress* parameter (66,000 psi (455 MPa))	J5, J5.2.1
F_c	Critical column *buckling stress*	1.1.4
F_{cr}	Elastic *shear buckling stress*	G2.3, 2.1
F_{cr}	F_{cre} — global (flexural, torsional, or flexural-torsional), $F_{cr\ell}$ — local, or F_{crd} — distortional elastic *buckling stress* in compression	2.1
F_{cr}	F_{cre} — global (lateral-torsional), $F_{cr\ell}$ — local, or F_{crd} — distortional elastic *buckling stress* referenced to the extreme compression fiber	2.1
F_{crd}	Elastic *distortional buckling stress*	2.1, 2.3.3.1, 2.3.3.2
F_{cre}	Critical elastic (*flexural*) *buckling stress*	C1.3.2, E2, E2.1, E3.1, E3.3, I1.2.2, I1.2.2.1, I1.2.2.2, I6.1.1.1, 2.3.1.1
F_{cre}	Critical elastic *lateral-torsional buckling stresses*	F2.1, F3.1, I6.1.2.1, 2.3.1.2
$F_{cr\ell}$	Minimum critical *buckling stress* for cross-section	E2.1, 1.1, 1.1.4
$F_{cr\ell}$	Plate elastic *buckling stress*	1.4.1
$F_{cr\ell}$	Smallest *local buckling stress* of all elements in cross-section	2.1, 2.3.2.1
$F_{cr\ell}$	*Local buckling stress* at extreme compression fiber	2.3.2.2
F_m	Mean value of fabrication factor	I6.3.1, K2.1.1
F_n	Nominal compressive *stress*	E2, E3.1, G7.2, I1.2.2.1, 1.1

SYMBOLS

Symbol	Definition	Section
F_n	Nominal global flexural *stress*	F2.1, F3.1, H2, H3, I6.1.1.2, I6.1.2.2, I6.2.1, I6.2.2
F_n	*Nominal strength* of bolts	J3.4
F_{nt}	*Nominal tensile strength* of bolts	J3.4
F'_{nt}	*Nominal tensile strength* for bolts subject to combination of shear and tension	J3.4
F_{nv}	*Nominal shear strength* of bolts	J3.4
F_{SR}	Design *stress* range	M3
F_{sy}	*Specified minimum yield stress*	A3.1.2, A3.1.3, J2.4.1
F_{TH}	Threshold *fatigue stress* range	M1, M3, M4
F_u	*Tensile strength*	A3.1.3, A3.1.2, D3, J2.2.2.1, J2.2.2.2, J2.2.3, J2.2.4, J2.3.2.1, J2.3.2.2, J2.4.1, J2.7, J3.3.1, J3.5, J4.5.2, J6.1, J6.2, J6.3
F_u	*Tensile strength* of bolt	J3.4
F_{ui}	*Tensile strength* of sheet corresponding to t_i	J4.3.2
F_{uh}	*Tensile strength* of hardened *PAF* steel	J5, J5.2.1, J5.3.1
F_{uv}	*Tensile strength* of *virgin steel* specified by Section A3 or established in accordance with Section K2.3.3	A3.3.2
F_{u1}, F_{u2}	*Tensile strengths* of connected parts corresponding to *thicknesses* t_1 and t_2	J2.6
F_{u1}	*Tensile strength* of member in contact with screw head or washer	J4, J4.3.1, J4.4.2, J4.5.1
F_{u1}	*Tensile strength* of member in contact with *PAF* head or washer	J5, J5.2.3, J5.3.2
F_{u2}	*Tensile strength* of member not in contact with screw head or washer	J4, J4.3.1, J4.4.1, J4.5.2
F_{wy}	Lower value of F_y for beam *web* or F_{ys} for bearing stiffeners	G7.1
F_{xx}	Tensile strength of electrode classification	J2.1, J2.2.2.1, J2.2.2.2, J2.2.3, J2.2.4, J2.3.2.1, J2.3.2.2, J2.4.1, J2.6, J2.7
F_y	*Yield stress*	A3.1.3, A3.3.1, A3.3.2, B4.1, C1.1.1.3, D2, E2, E3.2 E3.3, E4, F2.1, F2.1.1, F2.2, F2.3, F3.1, F3.2, F3.3, G7.1, G2.1, G4.1, G5, G8.1, H1.1, H1.2, H2, H3, H4, I1.3, I6.2.1, I6.2.2, I6.2.4, J2.1, J2.2.3, J2.4.1, J2.2.4, J4.5.2, J6.3, M1, 1.1, 1.1.4, 2.3.1.1.2, 3.2

SYMBOLS

Symbol	Definition	Section
F_{ya}	Average *yield stress* of section	A3.3.2
F_{yc}	Tensile *yield stress* of corners	A3.3.2
F_{yf}	Weighted average tensile *yield stress* of flat portions	A3.3.2, K2.3.2
F_{ys}	*Yield stress* of stiffener steel	G7.1
F_{yst}	Design *yield stress* of stiffener	G4.1
$F_{y,T}$	*Yield stress* at temperature (T)	4.2.3.1.5
F_{yv}	Tensile *yield stress* of *virgin steel* specified by Section A3 or established in accordance with Section K2.3.3	A3.3.2
F_{y2}	*Yield stress* of member not in contact with *PAF* head or washer	J5, J5.3.3
$\overline{F}$	Story shear, in the direction of translation being considered, produced by the lateral forces using *LRFD*, *LSD*, or 1.6 times *ASD load combinations*	C1.2.1.1
f	*Stress* in compression element computed on basis of *effective design width*	1.1, 1.1.1, 1.1.3, 1.1.4, 1.3
f	Uniform compressive *stress* acting on flat element	1.4.1, 1.4.1.1, 1.4.1.2, 1.4.2
f′	*Stress* used in Section 1.3(a) for determining *effective width* of edge stiffener	1.1.4
f_{av}	Average computed *stress* in full unreduced *flange* width	L3
f_c	Compressive *stress* in cover plate or sheet based on *ASD*, *LRFD* or *LSD load combinations*	I1.3
f_d	Computed compressive *stress* in element being considered. Calculations are based on effective section at load for which deflections are determined.	1.1, 1.1.1, 1.1.4, 1.3
f_d	Uniform compressive *stress* acting on flat element. Calculations are based on effective section at load for which deflections are determined.	1.4.1, 1.4.1.1, 1.4.1.2, 1.4.2
f_{d1}, f_{d2}	Computed *stresses* f_1 and f_2 as shown in Figure 1.1.2-1. Calculations are based on effective section at load for which serviceability is determined.	1.1.2
f_{d1}, f_{d2}	Computed *stresses* f_1 and f_2 in unstiffened element, as defined in Figures 1.2.2-1 to 1.2.2-3. Calculations are based on effective section at load for which serviceability is determined.	1.2.2
f_T	*Stress* at temperature (T)	4.2.3.1.5

SYMBOLS

Symbol	Definition	Section
f_v	*Required shear stress* on a bolt	J3.4
f_1, f_2	*Web stresses* defined by Figure 1.1.2-1	1.1.2, 1.1.3
f_1, f_2	*Stresses* at the opposite ends of the *web*	2.3.3.2
f_1, f_2	f_1 is the *stress* at the extreme compression fiber of the *flange* and edge stiffener, f_2 is the *stress* at the *flange*/*web* juncture	2.3.3.2
f_1, f_2	*Stresses* on unstiffened element defined by Figures 1.2.2-1 to 1.2.2-3	1.2.2
G	Shear modulus of steel, 11,300 ksi (78,000 MPa or 795,000 kg/cm^2)	2.3.1, 2.3.3.1, 4.2.3.1.1
g	Vertical distance between two rows of *connections* nearest to top and bottom *flanges*	I1.1
g	Transverse center-to-center spacing between fastener gage lines	J6.2, J6.3
H	Height of story	C1.2.1.1
HRC_p	Rockwell C hardness of *PAF* steel	J5, J5.2.1
h	Depth of flat portion of *web* measured along plane of *web* (disregard intermediate stiffeners)	B4.1
h	Flat depth of *web*	F2.1.1, G2.1, G2.3, G3, G4.1, G5, G6, H3, 1.1.3, 2.3.4
h	Centerline dimension of depth	2.3.3
h	Width of elements adjoining stiffened element	1.4.1
h	Height of lip	J2.7
h_e	Variable determined per Eq. 2.3.3.2-11	2.3.3.2
h_o	Out-to-out depth of *web*	1.1.2, 2.3.3.1, 2.3.3.2
h_o	Overall depth of unstiffened C-section member as defined in Figure 1.2.2-3	1.2.2
h_{st}	Nominal seam height	J2.4.1
h_{wc}	Coped flat *web* depth	J6.1
I	Moment of inertia of cylindrical tube	F3.3
I	Moment of inertia of built-up member about the axis of *flexural buckling*	I1.2.2.1
I	Moment of inertia about axis of *buckling*	2.3.1.1.1
I_a	Adequate moment of inertia of stiffener, so that each component element will behave as a stiffened element	B4.1, 1.3
I_{avg}	Weighted average moment inertia about axis of *buckling*	2.3.1.1.1
I_E	Earthquake importance factor of the structure, as defined in NBCC	A3.2.1.1

SYMBOLS

Symbol	Definition	Section
I_{eff}	Effective moment of inertia	L1, L2
I_g	Moment of inertia about axis of *buckling* for the gross built-up cross-section	I1.2.2.1
I_g	Gross moment of inertia	L2
I_r	Reduced moment of inertia	I1.2.2.1
I_s	Unreduced moment of inertia of stiffener about its own centroidal axis parallel to element to be stiffened	B4.1, 1.3
I_s	Actual moment of inertia of a pair of attached transverse *web* stiffeners, or of a single transverse *web* stiffener, with reference to an axis in the plane of the *web*	G4.1
I_{sp}	Moment of inertia of stiffener about centerline of flat portion of element	1.4.1, 1.4.1.1, 1.4.1.2
I_{st1}	Moment of inertia of the transverse stiffeners required for development of the full shear post *buckling* resistance of the stiffened *web* panels	G4.1
I_x, I_y	Moment of inertia of full unreduced section about x- and y-axis, respectively	C2.2.1, I6.4.1, 2.3.1, 2.3.1.1.4, 2.3.1.2.1, 2.3.1.2.3, 2.3.1.2.4
$I_{x,avg}$, $I_{y,avg}$	Weighted average of moment of inertia about x- and y-axis, respectively	2.3.1, 2.3.1.1.4
I_{xf}, I_{yf}	Moment of inertia of the *flange* about x- and y-axis, respectively	2.3.3, 2.3.3.1, 2.3.3.2
I_{xy}	Product of inertia of full unreduced section about x- and y-axes	C2.2.1, I6.4.1, 2.3.1, 2.3.1.1.4
$I_{xy,avg}$	Weighted average of product of inertia about x- and y-axes	2.3.1, 2.3.1.1.4
I_{xyf}	Product of inertia of *flange* about major and minor centroidal axes	2.3.3, 2.3.3.1, 2.3.3.2
I_1, I_2	Major and minor axis moment of inertia, respectively	3.2
i	Index of level of *notional load*	C1.1.1.2
i	Index of concentrically loaded compression member	C2.3.2
i	Index of sheet	J4.3.2
i	Index of stiffener	1.4.1, 1.4.1.2
i	Index of each *purlin* line	I6.4.1
i	Index of tests	K2.1.1
J	Saint-Venant torsion constant	2.3.1, 2.3.1.2.4
J_{avg}, J_g, J_{net}	Saint-Venant Torsion constant for weighted average, gross and net cross-section, respectively	2.3.1

SYMBOLS

Symbol	Definition	Section
J_f	Saint-Venant torsion constant of compression *flange*, plus edge stiffener about an x-y axis located at the centroid of the *flange*	2.3.3, 2.3.3.1
j	Number of brace anchor ends (j = 1 single side or j = 2 double side).	C2.3.2
j	Index for each anchorage device	I6.4.1
j	Asymmetry property	2.3.1, 2.3.1.2.2
j_{avg}, j_g, j_{net}	Asymmetry property calculated according to Eq. 2.3.1-6 for average, gross and net cross-section, respectively	2.3.1
K	*Effective length factor*	A1.3, 2.3.1.1.1
K′	Brace force factor	C2.2.1
K_a	Lateral stiffness of anchorage device	I6.4.1
$K_{eff_{i,j}}$	Effective lateral stiffness of jth anchorage device with respect to ith *purlin*	I6.4.1
K_{req}	Required stiffness	I6.4.1
K_{sys}	Lateral stiffness of roof system, neglecting anchorage devices	I6.4.1
K_tL_t	*Effective length* for twisting	2.3.1, 2.3.1.1.4
K_{total_i}	Effective lateral stiffness of all elements resisting force P_i	I6.4.1
K_x	*Effective length factor* for *buckling* about x-axis	C1.1.2, C1.2.1.1, C1.3.2
K_xL_x	*Effective length* for *buckling* about x-axis	2.3.1, 2.1.1.1.4
K_y	*Effective length factor* for *buckling* about y-axis	C1.1.2, C1.2.1.1, C1.3.2
K_yL_y	*Effective length* for *buckling* about y-axis	2.3.1, 2.3.1.1.4, 2.3.1.2.1, 2.3.1.2.3, 2.3.1.2.4
KL	*Effective length*	E2.1
KL	*Effective length* of built-up member for the axis of *flexural buckling*	I1.2.2.1
K_fL_f	*Effective length* for coupled *flexural buckling*	2.3.1.1.4
K_1	*Effective length factor* for flexural *buckling* in the plane of bending, K_y or K_x, as applicable, calculated based on the assumption of no lateral translation at member ends	C1.2.1.1
k	Plate *buckling* coefficient	1.1, 1.1.1, 1.1.2, 1.1.4, 1.2.1, 1.2.2, 1.3, 1.4.1, 1.4.2, 2.3.2.1, 2.3.2.2
k	Thermal conductivity	4.2.3.1.3
k_{af}	Reduction factor	I6.2.4
k_d	Plate *buckling* coefficient for *distortional buckling*	1.4.1, 1.4.1.1, 1.4.1.2, 3.4
k_{dnet}	k_d based on net cross-section	3.4

SYMBOLS

Symbol	Definition	Section
k_f	Flexural *stiffness* in the plane of bending as modified in Section C1.2.1.3	C1.2.1.1
k_ℓ	A variable calculated by Eq. 3.3-3	3.3
$k_{\ell net}$	A variable calculated by Eq. 3.3-3 for net section	3.3
k_{loc}	Plate *buckling* coefficient for local sub-element *buckling*	1.4.1, 1.4.1.1, 1.4.1.2
k_s	Ratio of member plastic moment, M_p, and member yield moment, M_y	F3.2
k_{s1}, k_{s2}	k_s as defined in Section F3.2 for M_{1r} and M_{2r}, respectively	3.3, 3.4
k_v	*Shear buckling* coefficient	G2.3, G3, 2.3.4
k_E, k_G, k_y, and k_u	Retention factors	4.2.3.1.1
$\bar{k}_\phi$	Rotational stiffness	2.3.3.1, 2.3.3.2
$\bar{k}_{\phi fe}$	Elastic rotational stiffness provided by *flange* to *flange*/*web* juncture	2.3.3.1, 2.3.3.2, 2.3.4
$\tilde{k}_{\phi fg}$	Geometric rotational stiffness demanded by *flange* from *flange*/*web* juncture	2.3.3.1, 2.3.3.2
$\bar{k}_{\phi we}$	Elastic rotational stiffness provided by *web* to *flange*/*web* juncture	2.3.3.1, 2.3.3.2
$\tilde{k}_{\phi wg}$	Geometric rotational stiffness demanded by the *web* from the *flange*/*web* juncture	2.3.3.1, 2.3.3.2
L	Full span for simple beams, or distance between inflection point for continuous beams, or twice member length for cantilever beams	B4.3
L	Span length	H1.2, I6.4.2, I1.1, I6.4.1
L	Length of weld	J2.1, J2.6, J2.7
L	Length of longitudinal weld or length of *connection*	J6.2
L	Length of seam weld not including circular ends	J2.3.2.1
L	*Unbraced length* of member	C1.2.1.1, 2.3.1.1.1
L	*Specified* occupancy live *load*	4.1.4
L_{av}	Active distance dimension	J6.3
L_b	Longitudinal distance between brace points on the individual concentrically loaded compression member to be braced	C2.3.1
L_{br}	Unsupported length between brace points or other restraints which restrict *distortional buckling* of element	1.4.1, 1.4.1.1, 1.4.1.2
L_{crd}	Critical unbraced length of *distortional buckling*	2.3.3.1, 2.3.3.2, 2.3.3.3
L_d	Minimum of L_{crd} and L_m	2.3.3.1, 2.3.3.2
L_{dh}	Minimum of L_{crd}, L_m and s	2.3.3.3

SYMBOLS

Symbol	Definition	Section
L_{gv}	Distance from free edge to centerline of bolt farthest from edge measured along line of shear failure	J6.3
L_h	Length of hole	G3, G6, 1.1.1, 1.1.3, 2.3.1, 2.3.3.3
$L_{h\text{-}eq}$	Equivalent hole length for circular and slotted holes	G3
L_m	Distance between discrete restraints that restrict *distortional buckling*	2.3.3.1, 2.3.3.2, 2.3.3.3
L_o	Overhang length measured from the edge of bearing to the end of member	G5
L_r	Nominal roof live *load*	5.4.1
L_{st}	Length of bearing stiffener	G7.1
L_{st}	Length of rupture in the staggered plane	J6.3
L_t	*Unbraced length* of member for twisting	2.3.1, 2.3.1.1.4
L_v	*Shear buckling* half-wavelength	2.3.4
L_w	Length of *top arc seam sidelap weld*	J2.4.1
L_0	Length at which *local buckling stress* equals *flexural buckling stress*	E2.1
l	Distance from concentrated load to a brace	C2.2.1
M	Bending moment	L1, L2
M_a	*Available flexural strength* [*factored resistance*] when bending alone is considered, determined in accordance with Section F3	H2
$M_{a\ell o}$	*Available flexural strength* [*factored resistance*] for globally braced member, determined in accordance (1) and (2) in Section H2	H2
$M_{a\ell o}$	*Available flexural strength* [*factored resistance*] for globally braced member, determined in accordance with Section F3 with $F_n = F_y$ or $M_{ne} = M_y$	H3
$M_{a\ell o}$	*Available flexural strength* [*factored resistance*] for globally braced member, determined in accordance with Section H2	H2
$M_{a\ell o}$	*Available flexural strength* [*factored resistance*] about centroidal x-axis in absence of axial *load*, determined in accordance with Section F3 with $F_n = F_y$ or $M_{ne} = M_y$	H3, I1.2.2.1
M_{ax}, M_{ay}	*Available flexural strengths* [*resistances*] about centroidal axes, determined in accordance with Chapter F	H1.1, H1.2, H4

SYMBOLS

Symbol	Definition	Section
M_{axt}, M_{ayt}	*Available flexural strengths* [*resistances*] about centroidal axes	H1.1
M_{cr}	M_{cre}—global (lateral-torsional), $M_{cr\ell}$—local, or M_{crd}—*distortional* elastic *buckling* moment about the axis of bending	2.1
M_{crd}	*Distortional buckling* moment	F4, I6.1.2.3, 2.1, 2.3.3.2
M_{cre}	Global *buckling* (*lateral-torsional buckling*) moment	F2.2, 2.1, 2.3.1.2, I6.1.2.1, 2.3.1.2.1, 2.3.1.2.2, 2.3.1.2.3, 2.3.1.2.4, 3.2
$M_{cr\ell}$	Critical elastic *local buckling* moment	F3.2, I6.1.2.2, 2.1, 2.3.2.2
M_d	*Nominal flexural strength* [*resistance*], M_n, defined in Chapter F with *Direct Strength Method*, but with M_y replaced by M in all equations	L2
M_m	Mean value of material factor	I6.3.1, K2.1.1
M_{max}, M_A, M_B, M_C	Absolute value of moments in unbraced segment, used for determining C_b	2.3.1
M_n	*Nominal flexural strength* [*resistance*]	F1, I6.1.2, I6.2.1, I6.2.2
M_{nd}	*Nominal flexural strength* [*resistance*] for *distortional buckling*	F4, I6.1.2, I6.1.2.3
M_{ne}	*Nominal flexural strength* [*resistance*] for *yielding* and global (*lateral-torsional*) *buckling*	F2, F2.1, F2.1.1, F2.2, F2.3, F3.2, F3.3, H2, H3, I6.1.2, I6.1.2.1, I6.2.1, I6.2.2
$\overline{M}_{ne}$	Lesser of M_y and M_{ne}	F3.2
$M_{n\ell}$	*Nominal flexural strength* [*resistance*] for *local buckling*	F3, F3.1, F3.2, F3.3, I6.1.2, I6.1.2.2
$M_{n\ell e}$	*Nominal flexural strength [resistance]* for elastic *local buckling*	F3.3
$M_{n\ell o}$	*Nominal flexural strength* [*resistance*] for *local buckling* only, as determined from Section F3 with F_n =F_y or M_{ne}=M_y	H3, I6.2.1, I6.2.2
M_p	Member plastic moment	F2.2, F3.2, F4
M_{pnet}	Member plastic moment of net cross-section	F3.2, F4
M_r	Resultant moment of M_{1r} and M_{2r}	3.2
M_y	Member *yield moment* (=$S_{fy}F_y$)	F2.1, F3.2, FF4, H2, H3, H4, I6.2.1, I6.2.2
M_{y3}	Member moment at strain limit	F3.2, F4
M_{ynet}	Member *yield moment* of net cross-section	F3.2, F4
M_{y3net}	Member moment at strain limit	F3.2, F4
M_1, M_2	Smaller and larger end moments in an unbraced segment, respectively	C1.2.1.1, 2.3.3.2

SYMBOLS

Symbol	Definition	Section
M_{1r}, M_{2r}	*Required strength* [effect due to *factored loads*] in bending about the major and minor principal axes, respectively	3.1.1, 3.2, 3.4
M_{1y}, M_{2y}	Moment about the major and minor principal axes, respectively, that causes first yield in the section	3.1.1
$\overline{M}$	*Required* second-order flexural *strength* [moment due to *factored loads*] using *LRFD*, *LSD,* or 1.6 times *ASD load combinations*, as applicable	C1.1.1.1, C1.2.1.1
$\overline{M}$	*Required flexural strengths* [moments due to *factored loads*] in accordance with *ASD*, *LRFD,* or *LSD load combinations*	H2
$\overline{M}$	*Required flexural strength* [moment due to *factored loads*] at, or immediately adjacent to, the point of application of the concentrated *load* or reaction $\overline{P}$ determined in accordance with *ASD*, *LRFD,* or *LSD load combinations*	H3
$\overline{M}_{\ell t}$	Moment from *first-order* elastic *analysis* using *LRFD*, *LSD,* or *ASD load combinations*, as applicable, due to lateral translation of the structure only	C1.2.1.1
$\overline{M}_{nt}$	Moment from *first-order* elastic *analysis* using *LRFD*, *LSD,* or *ASD load combinations*, as applicable, with the structure restrained against lateral translation	C1.2.1.1
$\overline{M}_x$, $\overline{M}_y$	*Required flexural strengths* [moments due to *factored loads*] with respect to centroidal axes in accordance with *ASD*, *LRFD,* or *LSD load combinations*	H1.1, H1.2, H4
$\overline{M}_z$	Torsional moment of force about shear center	C2.2.1
m	Degrees of freedom	K2.1.1
m	Term for determining tensile yield point of corners	A3.3.2
m	Distance from shear center to mid-plane of *web* of C-section	C2.2.1, I1.1, I6.4.1
m	Total number of concentrically loaded compression members to be braced	C2.3.2
m_f	Modification factor for type of bearing *connection*	J3.3.1
N	Bearing length	G5, G6, H3
N	Number of *stress* range fluctuations in design life	M3
N_a	Number of anchorage devices along a line of anchorage	I6.4.1
N_i	*Notional load* applied at level i	C1.1.1.2

SYMBOLS

Symbol	Definition	Section
N_p	Number of *purlin* lines on roof slope	I6.4.1
n	Coefficient	1.3
n	Number of stiffeners in element	1.4.1,1.4.1.1, 1.4.1.2
n	Number of equally spaced intermediate brace locations	C2.3.1
n	Number of anchors in test assembly with same tributary area (for anchor failure), or number of panels with identical spans and loading to failed span (for non-anchor failure)	I6.3.1
n	Number of fasteners on critical cross-section	J6.1
n	Number of threads per inch	M4
n	Total number of tests	K2.1.1
n_b	Number of fasteners along failure path being analyzed	J6.1, J6.2
n_f	Number of intermediate stiffeners in stiffened compression element	B4.1
n_f	Number of rows of bolts	J6.3
n_{fe}	Number of intermediate stiffeners in edge stiffener	B4.1
n_{sh}	Number of shear planes in the block	J6.3
n_{st}	Number of staggered planes in the block	J6.3
n_w	Number of intermediate stiffeners in stiffened element under stress gradient (e.g., *web*)	B4.1
P	Professional factor	B4.2, K2.1.1
P_a	*Available axial strength* [*factored resistance*], determined in accordance with Chapter E	H1.2
P_a	*Available strength* [*factored resistance*] for concentrated *load* or reaction in absence of bending moment, determined in accordance with Section G5 and G6, as applicable	H3
P_{at}	*Available tensile strength* [*resistance*] of arc spot weld	J2.2.3, J2.2.4
P_{av}	*Available shear strength* [*resistance*] of arc spot weld	J2.2.2.1, J2.2.2.2, J2.2.4
P_{av}	*Available shear strength* [*resistance*] of arc seam weld	J2.3.2.1, J2.3.2.2
P_{av}	*Available strength* [*resistance*] of fillet weld	J2.6
P_{av}	*Available shear strength* [*resistance*] of a flare groove weld	J2.7
P_{av}	*Available resistance weld shear strength* [*resistance*]	J2.8
P_{cr}	P_{cre} – global (flexural, torsional, or flexural-torsional), $P_{cr\ell}$ – local, or P_{crd} – distortional elastic *buckling* force in compression	2.1

SYMBOLS

Symbol	Definition	Section
P_{crd}	*Distortional buckling* force (load)	E4, I1.2.4, I6.1.1.3, 2.1, 2.3.3.1
P_{cre}	Global *buckling* force	I1.2.2.1, I6.1.1.1, 2.1, 2.3.1.1, 2.3.1.1.4
P_{cre}	*Flexural buckling* force	2.3.1.1.1, 2.3.1.1.3
P_{cre}	*Flexural-torsional buckling* force	2.3.1.1.2
$P_{cr\ell}$	Critical elastic *local* column *buckling load,* determined in accordance with Appendix 2, including the influence of holes if applicable	E3.2, I1.2.3, I6.1.1.2, 2.1, 2.3.2.1
P_{e1}	Elastic critical *buckling* strength of the member in the plane of bending, calculated based on the assumption of no lateral translation at member ends	C1.2.1.1
$P_{e,story}$	Elastic critical *buckling* strength for the story in the direction of translation being considered, determined by sidesway *buckling* analysis or taken as Eq. C1.2.1.1-7	C1.2.1.1
P_{ex}, P_{ey}	Axial force for *flexural buckling* about x- and y-axis, respectively	2.3.1, 2.3.1.1.2, 2.3.1.1.4, 2.3.1.2.1, 2.3.1.2.2, 2.3.1.2.3
P_{fx}, P_{fy}	Axial force for *flexural buckling* about x- and y-axis, respectively, where the unbraced length is K_fL_f	2.3.1.1.4
P_{fxy}	Axial force for *flexural buckling,* where the unbraced length is K_fL_f	2.3.1.1.4
P_i	Lateral force introduced into system at ith *purlin*	I6.4.1
P_{L_j}	Lateral force to be resisted by the jth anchorage device	I6.4.1
P_m	Mean value of tested-to-predicted load ratios	B4.2
P_m	Mean value of professional factor	K2.1.1
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	C1.2.1.1
P_n	*Nominal web crippling strength* [*resistance*]	G5, H3
P_n	*Nominal axial strength* [*resistance*] of member	E1, I6.1.1, I6.2.3, I6.2.4
P_n	*Nominal axial strength* [*resistance*] of bearing stiffener	G7.1, G7.2
P_n	*Nominal strength* [*resistance*] of groove weld	J2.1
P_n	*Nominal fillet weld strength* [*resistance*]	J2.6
P_n	*Nominal flare groove weld strength* [*resistance*]	J2.7
P_n	*Nominal bolt strength* [*resistance*]	J3.4
P_{nb}	*Nominal bearing strength* [*resistance*]	J3.3.1, J3.3.2
P_{nb}	*Nominal bearing* and *tilting strength* [*resistance*] per *PAF*	J5, J5.3.2

SYMBOLS

Symbol	Definition	Section
P_{nc}	*Nominal web crippling strength* [*resistance*] of C- or Z-section with overhang(s)	G5
P_{nd}	*Nominal axial strength* for *distortional buckling*	E4, I1.2.4, I6.1.1, I6.1.1.3
P_{ne}	*Nominal axial strength* [*resistance*] for yielding and global *buckling*	E2, E3.1, E3.2, I1.2.3, I6.1.1, I6.1.1.1
$P_{n\ell}$	*Nominal axial strength* [*resistance*] for *local buckling*	E3, E3.1, E3.2, E3.3, I1.2.3, I6.1.1, I6.1.1.2
$P_{n\ell o}$	*Nominal compressive strength* [*resistance*] of stiffener determined in accordance with Section E3 with $F_n = F_y$ or $P_{ne} = P_y$, where F_y or P_y is based on *yield stress* of stiffener steel	G7.2
P_{nos}	*Nominal pull-out strength* [*resistance*] in shear per *PAF*	J5, J5.3.3
P_{not}	*Nominal pull-out strength* [*resistance*] of sheet per screw	J4, J4.4.1, J4.5.2
P_{not}	*Nominal pull-out strength* [*resistance*] in tension per *PAF*	J5, J5.2.2
P_{nov}	*Nominal pull-over strength* [*resistance*] of sheet per screw	J4, J4.4.2, J4.5.1
P_{nov}	*Nominal pull-over strength* [*resistance*] per *PAF*	J5, J5.2.3
P_{nr}	*Nominal block shear rupture strength* [*resistance*]	J6.3
P_{nt}	*Nominal tensile strength* [*resistance*]	J2.2.3
P_{nt}	*Nominal tensile rupture strength* [*resistance*]	J6.2
P_{ntp}	*Nominal tensile strength* [*resistance*] of *PAF*	J5, J5.2.1
P_{nts}	*Nominal tension strength* [*resistance*] of screw as reported by manufacturer or determined by independent laboratory testing	J4, J4.4.3, J4.5.3
P_{nv}	*Nominal shear strength* [*resistance*] of arc spot weld	J2.2.2.1, J2.2.2.2
P_{nv}	*Nominal shear strength* [*resistance*] of arc seam weld	J2.3.2.1, J2.3.2.2
P_{nv}	*Nominal shear strength* [*resistance*] of *top arc seam sidelap weld*	J2.4.1
P_{nv}, P_{nv1}, P_{nv2}	*Nominal shear strength* [*resistance*] of a fillet weld	J2.6
P_{nv}	*Nominal shear strength* [*resistance*] of a flare groove weld	J2.7
P_{nv}	*Nominal resistance weld shear strength* [*resistance*]	J2.8
P_{nv}	*Nominal shear strength* [*resistance*] of sheet per screw	J4, J4.3.1, J4.5.1, J4.5.2
P_{nv}	*Nominal shear strength* [*resistance*] of lap *connection* with a single bolt	J3.5

SYMBOLS

Symbol	Definition	Section
P_{nv}	*Nominal shear rupture strength* [*resistance*]	J6.1
P_{nvi}	*Nominal strength* [*resistance*] of individual sheet i	J4.3.2
P_{nvp}	*Nominal shear strength* [*resistance*] of *PAF*	J5, J5.3.1
P_{nvs}	*Nominal shear strength* [*resistance*] of screw as reported by manufacturer or determined by independent laboratory testing	J4, J4.3.3, J4.5.3
P_{nv1}, P_{nv2}	*Nominal shear strength* [*resistance*] corresponding to connected *thicknesses* t_1 and t_2	J2.6
P_r	*Required strength* [effect due to *factored loads*] under axial *load*, compression positive, tension negative	3.1.1, 3.1.2, 3.2
P_s	Concentrated load or reaction based on critical *load combinations* for *ASD*, *LRFD*, and *LSD*	I1.1
P_t	Axial force for *torsional buckling* about shear center	2.3.1, 2.3.1.1.2, 2.3.1.1.3, 2.3.1.1.4, 2.3.1.2.1, 2.3.1.2.2, 2.3.1.2.3
P_{wc}	*Nominal web crippling strength* [*resistance*] for C-section flexural member	G7.2
P_y	Member axial yield strength	C1.1.1.3, E4, 3.1.1, 3.1.2
P_{ynet}	Member yield strength on net cross-section	E3.2, E4
$\overline{P}$	*Design* concentrated *load* [*factored load*] within a distance of 0.3a on each side of a brace, plus 1.4(1-*l*/a) times each required concentrated load located farther than 0.3a but not farther than 1.0a from the brace. The *design* concentrated *load* [*factored load*] is the applied *load,* determined in accordance with the most critical *ASD*, *LRFD*, or *LSD load combinations*, depending on the design method used	C2.2.1
$\overline{P}$	*Required* axial *strength* [compressive force due to *factored loads*] using *LRFD*, *LSD,* or *ASD load combinations*, as applicable	C1.1.1.3, C1.2.1.1, H1.2, H3
$\overline{P}_{\ell t}$	Axial force from *first-order* elastic *analysis* using *LRFD*, *LSD,* or *ASD load combinations*, as applicable, due to lateral translation of the structure only	C1.2.1.1
$\overline{P}_{L1}$, $\overline{P}_{L2}$	Lateral bracing forces	C2.2.1
$\overline{P}_{nt}$	Axial force from *first-order* elastic *analysis* using *LRFD*, *LSD,* or *ASD load combinations*, as applicable, with the structure restrained against lateral translation	C1.2.1.1
$\overline{P}_{ra}$	*Required compressive axial strength* [compressive axial force due to *factored loads*] of individual	C2.3.1

SYMBOLS

Symbol	Definition	Section
	concentrically loaded compression member to be braced, which is calculated in accordance with *ASD*, *LRFD*, or *LSD load combinations* depending on the design method used	
$\overline{P}_{rb}$	*Required brace strength* [brace force due to *factored loads*] to brace a single compression member with an axial *load* $\overline{P}_{ra}$	C2.3.1
$\overline{P}_{rb}$	*Required brace strength* [brace force due to *factored loads*] to brace multiple parallel compression members	C2.3.2
$\overline{P}_{rb,i}$	*Required brace strength* [brace force due to *factored loads*] of (i)th concentrically loaded compression member, which is calculated in accordance with Eq. C2.3.1-1	C2.3.2
$\overline{P}_{story}$	Total vertical *load* supported by the story using *LRFD*, *LSD,* or *ASD load combinations*, as applicable, including loads in columns that are not part of the lateral force-resisting system	C1.2.1.1
$\overline{P}_{v}$	*Required shear strength* [shear force due to *factored loads*], between individual shapes	I1.2.2.1
$\overline{P}_{x}, \overline{P}_{y}$	Components of *design load* [*factored load*] $\overline{P}$ parallel to the x- and y-axis, respectively	C2.2.1
p	Pitch (mm per thread for SI units and cm per thread for MKS units)	M4
Q	First moment of area of connected shape(s) about axis of *buckling* for the gross built-up cross-section	I1.2.2.1
Q_i	*Load effect*	K2.1.1
q	*Design load* [factored load] on beam for determining longitudinal spacing of *connections*	I1.1
R	*Required allowable strength* for *ASD*	B3.1, B3.2.1
R	Modification factor for *distortional* plate *buckling* coefficient	1.4.1
R	Reduction factor	E2.1, E3.3, I6.2.1, I6.2.2, I6.2.4
R	Inside bend radius	A3.3.2, B4.1, G5, H3
R	Radius of outside bend surface	J2.7
R	Nominal *load* due to rain, exclusive of the ponding contribution	5.4.1
R_a	*Available strength* [*factored resistance*]	B3.2
R_a	*Allowable design strength*	B3.2.1, K2.1.2
R_a	*Design strength*	B3.2.2

SYMBOLS

Symbol	Definition	Section
R_a	*Factored resistance*	B3.2.3
R_b	Reduction factor	A3.1.3
R_c	Reduction factor	G6
R_f	Effect of *factored loads*	B3.1, B3.2, B3.2.3
R_I	I_s/I_a	1.3
R_n	*Nominal strength* [*resistance*]	A1.3, B3.2.1, B3.2.2, B3.2.3, K2.2
R_n	*Nominal rupture strength* [*resistance*]	J6
R_n	Average value of all test results	K2.1.1, K2.1.2
$R_{n,i}$	Calculated *nominal strength* [*resistance*] of test i per *rational engineering analysis* model	K2.1.1
R_t	Tested *strength* [*resistance*]	K2.1.1
$R_{t,i}$	Tested *strength* [*resistance*] of test i	K2.1.1
R_u	*Required strength* for *LRFD*	B3.2.2
R_1, R_2	Radius of outside bend surface	J2.7
$\overline{R}$	*Required strength* [effect due to *factored loads*]	B3.1, B3.2
r	Correction factor	I6.2.1
r	Radius of gyration of full unreduced cross-section about axis of *buckling*	E2.1
r	Radius of gyration of full unreduced cross-section area of an individual shape in a built-up member	I1.2.2.1
r	Weld effectiveness reduction factor	J2.2.3
r_i	Minimum radius of gyration of *full unreduced cross-sectional area* of an individual shape in a built-up member	I1.2.2.1
r_o	Polar radius of gyration of about shear center	2.3.1, 2.3.1.1.4, 2.3.1.2.1, 2.3.1.2.2, 2.3.1.2.3
$r_{o,avg}$	Polar radius of gyration about shear center for weighted average, gross and net section	2.3.1
S	$1.28\sqrt{E/f}$	1.3
S	*Specified* variable *load* due to snow	4.1.4
S	Nominal snow *load*	5.4.1
$S_a(T)$	5 percent damped spectral response acceleration, expressed as a ratio to gravitational acceleration, for a period T, as defined in NBCC	A3.2.1.1
S_e	Effective section modulus calculated relative to extreme compression or tension fiber at F_y	F2.1.1
S_{ec}	Effective section modulus calculated at extreme fiber compressive *stress* of F_n	F3.1, 1.2.2

SYMBOLS

Symbol	Definition	Section
S_{et}	Effective section modulus calculated at extreme fiber tension *stress* of F_y	F3.1, 1.2.2
S_f	Elastic section modulus of full unreduced section relative to extreme fiber of first yield	F2.1, F2.3
S_{fc}	Elastic section modulus of full unreduced section relative to extreme compression fiber	F2.1, F3.1, 2.1, 2.3.1.2, 2.3.2.2, 2.3.3.2
S_{fcnet}	Net elastic section modulus relative to extreme compression fiber	F3.1, 2.3.2.2
S_{fnet}	Net section modulus referenced to the extreme fiber in first yield	F3.2
S_{ft}	Section modulus of full unreduced section relative to extreme tension fiber about appropriate axis	H1.1
s	Center-to-center hole spacing	1.1.1, 2.3.1, 2.3.3.3
s	Longitudinal *connection* spacing	I1.1, I1.3, 1.1.4
s	Width of tensile rupture section divided by number of bolt holes in cross-section	J6.2
s′	Longitudinal center-to-center spacing of any consecutive holes	J6.2, J6.3
s_c	Standard deviation of $R_{t,i}$ divided by $R_{n,i}$ for all of the test results	K2.1.1
s_{end}	Clear distance from the hole at ends of member	1.1.1
s_{max}	Maximum permissible longitudinal spacing of welds or other connectors joining two C-sections to form an I-section	I1.1
s_t	Standard deviation of all the test results	K2.1.1
T	Temperature	4.2.3.1.1, 4.2.3.1.2, 4.2.3.1.3, 4.2.3.1.5
T_a	*Available tensile axial strength* [*factored resistance*] determined in accordance with Chapter D	H1.1
T_n	*Nominal tensile strength* [*resistance*]	D1, D2, D3
T_r	*Required strength* [force due to *factored loads*] for *connection* in tension	I1.1
T_s	*Available strength* [*factored resistance*] of *connection* in tension (Chapter J)	I1.1
T_S	Effects due to expansion, contraction, or deflection caused by temperature changes due to the *design basis-fire*. T_S can be taken equal to zero for statically determinate structures or for structures that have sufficient ductility to allow the redistribution of temperature forces before collapse	4.1.4

SYMBOLS

Symbol	Definition	Section
$\overline{T}$	*Required tensile axial strength* [tensile force due to *factored loads*] in accordance with *ASD*, *LRFD,* or *LSD load combinations*	H1.1
$\overline{T}$	*Required tension strength* [tensile force due to *factored loads*] per *connection* fastener determined in accordance with *ASD*, *LRFD,* or *LSD load combinations*	J2.2.4, J4.5.1, J4.5.2, J4.5.3
t	Base steel *thickness* of any element or section	A1.3, A3.1.3, A3.3.2, B4.1, B7.1, E3.3, F2.1.1, F2.3, F3.3, G7.1, G2.1, G2.3, G3, G5, G6, H3, I1.3, I6.2.3, I6.2.4, I6.4.1, J2.2.2.2, J2.2.4, J2.3.2.2, J2.4.1, J2.6, J2.7, J2.8, J3.3.1, J3.3.2, J6.1, J6.2, L3, 1.1, 1.1.1, 1.1.3, 1.1.4, 1.2.2, 1.3, 1.4.1, 1.4.1.1, 1.4.1.2, 1.4.2, 2.3.2.1, 2.3.2.2, 2.3.3, 2.3.3.1, 2.3.3.2, 2.3.3.3, 2.3.4
t	Total combined base steel *thickness* (exclusive of coatings) of sheets involved in shear transfer above plane of maximum shear transfer	J2.2.2.1, J2.3.2.1
t	*Thickness* defined in Table J2.2.3-1	J2.2.3
t	*Thickness* of part in which end distance is measured	J3.5
t_c	Lesser of depth of penetration and t_2	J4, J4.4.1, J4.5.2
t_e	Effective throat dimension of groove weld	J2.1
t_i	*Thickness* of sheet i	J4.3.2
t_i	Thickness of uncompressed glass fiber blanket insulation	I6.2.1
t_r	Modified thickness	2.3.3.3
t_s	*Thickness* of stiffener steel	G7.1
t_{st}	Stiffener *thickness*	G4.1
t_w	Effective throat of weld	J2.6, J2.7
t_w	Steel washer thickness	J4.4.2, J5, J5.2
t_{wf}	Effective throat of groove weld that is filled flush to surface, determined in accordance with Table J2.7-1	J2.7
t_1	*Thickness* of member in contact with screw head or washer	J4, J4.3.1, J4.4, J4.4.2, J4.5.1
t_1	*Thickness* of member in contact with *PAF* head or washer	J5, J5.2.3, J5.3.2
t_2	*Thickness* of member not in contact with screw head	J5, J5.2.2, J5.3.2, J5.3.3

SYMBOLS

Symbol	Definition	Section
t_2	*Thickness* of member not in contact with *PAF* head or washer	J4, J4.3.1, J4.5.1, J4.5.2
t_1, t_2	*Thicknesses* of connected parts	J2.6
U_{bs}	Nonuniform block shear factor	J6.3
$U_{s\ell}$	Shear lag factor	J6.2, J6.3
V_a	*Available shear strength* [*factored resistance*] when shear alone is considered, determined in accordance with Chapter G	H2
V_{cr}	*Shear buckling* force	G2.1, G2.3, G3, 2.1, 2.3.4
V_{crh}	*Shear buckling* force with *web* hole	G3
V_F	Coefficient of variation of fabrication factor	I6.3.1, K2.1.1
V_M	Coefficient of variation of material factor	I6.3.1, K2.1.1
V_n	*Nominal shear strength* [*resistance*]	G2, G2.1, G2.2, G4.1
V_P	Coefficient of variation of tested-to-predicted *load* ratios	B4.2, I6.3.1
V_P	Coefficient of variation of test results, but not less than 0.065	K2.1.1
V_Q	Coefficient of variation of *load effect*	I6.3.1, K2.1.1
V_y	Yield shear force of cross-section	G2.1, G2.2, G3
V_{yh}	Yield shear force with *web* hole	G3
$\overline{V}$	*Required shear strength* [shear force due to *factored loads*] per *connection* fastener, determined in accordance with *ASD*, *LRFD,* or *LSD load combinations*	J2.2.4, J4.5.1, J4.5.2, J4.5.3
$\overline{V}$	*Required shear strength* [shear force due to *factored loads*] in accordance with *ASD*, *LRFD,* or *LSD load combinations*	H2
W_{pi}	Total required vertical *load* supported by ith *purlin* in a single bay	I6.4.1
$\overline{W}$	*Design load* [*factored load*] (applied *load* determined in accordance with the most critical *ASD*, *LRFD,* or *LSD load combinations*, depending on the design method used) within a distance of 0.5a on each side of the brace	C2.2.1
$\overline{W}_x, \overline{W}_y$	Components of *required strength* [*factored load*] $\overline{W}$	C2.2.1
w	*Flat width* of compression *flange*	A3.1.3, B4.3
w	*Flat width* of stiffened compression element (disregard intermediate stiffeners)	B4.1

SYMBOLS

Symbol	Definition	Section
w	*Flat width* of element	F2.1.1, 1.1, 1.1.1, 1.2.1, 1.2.2, 1.3, 2.3.2.1, 2.3.2.2
w	Flat width of stiffened and unstiffened element of bearing stiffener	G7.1
w	Transverse spacing of connectors	1.1.4
w	*Flat width* of narrowest unstiffened compression element tributary to *connections*	I1.3
w′	Equivalent *flat width* for determining *effective width* of edge stiffener	1.1.4
w_f	Width of *flange* projection beyond *web* for I-beams and similar sections, or half distance between *webs* for box- or U-type sections	B4.3, L3
w_f	Face width of weld	J2.7
w_i	Required distributed gravity *load* supported by the i^{th} *purlin* per unit length (determined from the critical *ASD*, *LRFD,* or *LSD load combination* depending on the design method used)	I6.4.1
w_o	Out-to-out width	1.1.1
w_1	Transverse spacing between first and second line of fasteners in compression element	1.1.4
w_1, w_2	Leg of weld	J2.6, J2.7
x	Non-dimensional fastener location	I6.2.3
x	Nearest distance between *web* hole and edge of bearing	G6
x, y	Centroidal axes of the cross-section	C2.2.1, H3, 2.3.1, 2.3.1.1.2, 2.3.1.1.4, 2.3.1.2.1, 2.3.1.2.2, 2.3.1.2.3, 2.3.3
x_{hf}	x distance from centroid of *flange* to *flange*/*web* junction	2.3.3, 2.3.3.1, 2.3.3.2
x_o	Shear center x-coordinate relative to centroid of cross-section	2.3.1, 2.3.1.1.4
$x_{o,avg}$, $x_{o,g}$, $x_{o,net}$	Shear center x-coordinate relative to centroid of cross-section for weighted average, gross and net section, respectively	2.3.1
x_{of}	x distance from centroid of *flange* to shear center of *flange*	2.3.3, 2.3.3.1, 2.3.3.2
$\overline{x}$	Distance from shear plane to centroid of cross-section	J6.2
Y_i	Gravity *load* applied at level i from the *LRFD, LSD load combinations*, or *ASD load combinations*, as applicable	C1.1.1.2, C1.1.1.3

SYMBOLS

Symbol	Definition	Section
y_c	Distance from elastic neutral axis to extreme compression fiber	F3.2, F4
y_{hf}	y distance from centroid of *flange* to *flange*/*web* junction	2.3.3
y_o	Shear center y-coordinate relative to centroid of cross-section	2.3.1, 2.3.1.1.4
$y_{o,avg}$, $y_{o,g}$, $y_{o,net}$	Shear center y-coordinate relative to centroid of cross-section for weighted average, gross and net section	2.3.1
y_{of}	y distance from centroid of *flange* to shear center of *flange*	2.3.3, 2.3.3.1, 2.3.3.2
Z_f	Plastic section modulus	F2.2
Z_{fnet}	Plastic section modulus of the net cross-section	F3.2
α	Coefficient for *purlin* directions	I6.4.1
α	Coefficient for conversion of units	I6.2.3, J3.3.2, J4.4.1, M3
α	Coefficient for strength increase due to overhang	G5
α	Coefficient	I1.3, C1.1.1.2, C1.1.1.3, C1.2.1.1
α	1.0 for storage areas, equipment areas, and service rooms, and 0.5 for other occupancies	4.1.4
α	Coefficient of thermal expansion	4.2.3.1.4
α	Angle between the line of the force and the centerline of the staggered hole	J6.3
α_b	Coefficient	J5.3.2
α_s	1 for cross-sections with one or more corners at the extreme compression fiber, and 0 for other cross-sections	F3.2, F4
α_{s1}, α_{s2}	α_s as defined in Section F4 for M_{1r} and M_{2r}, respectively	3.3, 3.4
α_w	Coefficient differentiating *PAF* types	J5.2.3
β	Coefficient	E2.2
β	Variable	1.4.1.1, 1.4.1.2
β	Coefficient for *flexural-torsional buckling* about x-axis	2.3.1, 2.3.1.1.2, 2.3.1.1.4
β	A value accounting for moment gradient	2.3.3.2
β	Parameter	4.2.3.1.5
β_a	Generalized *Available strength* [*factored resistance*], i.e., capacity, under the combined forces as defined in Section 3.1.2	3.1, 3.1.2
β_{anchor}	Lateral *stiffness* at the brace anchor point(s) and must exceed (m/j) ($\beta_{rb,max}$ / γ_c)	C2.3.2

SYMBOLS

Symbol	Definition	Section
β_c	*Stiffness* of connector used to attach compression member to continuous bracing	C2.3.2
β_{crd}	Magnitude of the generalized demand under the combined forces that cause elastic *distortional buckling*	3.4
β_{cre}	Magnitude of the generalized demand under the combined forces that cause elastic global *buckling*	3.2
$\beta_{cr\ell}$	Magnitude of the generalized demand under the combined forces that cause elastic *local buckling*	3.3
β_n	Generalized *nominal strength* [*resistance*], determined as the minimum of β_{ne}, $\beta_{n\ell}$, or β_{nd} calculated in accordance with Sections 3.2 to 3.4, as applicable	3.1.2
β_{nd}	Generalized *nominal strength* [*resistance*] for *distortional buckling*	3.4
β_{ne}	Generalized *nominal strength* [*resistance*] for global *buckling* and/or *yielding*	3.2, 3.3
β_{neM}	Generalized *nominal strength* [*resistance*] based on flexural strength provisions	3.2
β_{neP}	Generalized *nominal strength* [*resistance*] based on axial strength provisions	3.2
$\beta_{n\ell}$	Magnitude of generalized *local buckling* demand	3.3
β_P	Magnitude of generalized demand under the combined forces that causes the entire section to be plastic	3.2, 3.3
β_o	Target reliability index	I6.3.1, K2.1.1
β_r	Generalized *Required strength* [effect due to *factored loads*], i.e., demand, under the combined forces as defined in Section 3.1.1	3.1, 3.1.1, 3.1.2, 3.2
β_{rb}	Minimum required brace stiffness to brace a single compression member	C2.3.1
β_{rb}	Minimum required stiffness of each brace between the concentrically loaded compression members	C2.3.2
$\beta_{rb,max}$	Largest required brace stiffness of all concentrically loaded compression members calculated in accordance with Section C2.3.1	C2.3.2
β_s	$2y_c/d \geq 0.4$	F3.2, F4
β_{s1}, β_{s2}	β_s as defined in Section F3.2 for M_{1r} and M_{2r}, respectively	3.3, 3.4
β_y	Magnitude of the generalized demand under the combined forces that causes first yield in the section	3.2, 3.3, 3.4

SYMBOLS

Symbol	Definition	Section
β_{ynet}	Value of β_y based on net section	3.3, 3.4
β_{y3}	Magnitude of the generalized demand under the combined forces at strain limit	3.3, 3.4
β_{y3net}	Value of β_{y3} based on net section	3.3, 3.4
$\bar{\beta}_{ne}$	Minimum of β_{ne} and β_{cre}	3.3
γ, γ_i	Coefficients	1.4.1.1, 1.4.1.2
γ_d	$(1 – \cos \varphi_{PM})^{10}$ for P_r in compression and 1 for P_r in tension or no axial load	3.4
γ	Coefficient for *flexural-torsional buckling* about y-axis	2.3.1, 2.3.1.1.4
γ_a	Factor to account for anchor *stiffness*	C2.3.2
γ_c	Factor to account for connector *stiffness*	C2.3.2
γ_i	*Load factor*	K2.1.1
γ_ℓ	Variable	3.3
δ, δ_i	Coefficients	1.4.1.1, 1.4.1.2
ε	Coefficient	2.3.4
ε_y	Yield strain = F_y/E	F2.1.1
ε_T	Strain corresponding to a given *stress* f_T at temperature (T)	4.2.3.1.5
η	Coefficient	J2.7
η_T	Parameter	4.2.3.1.5
θ	Angle between plane of *web* and plane of bearing surface	G5
θ	Angle between vertical and plane of *purlin web*	I6.4.1
θ	Angle between an element and its edge stiffener	1.3, 2.3.3
θ_{12}	Angle variable	3.3, 3.4
λ	Element slenderness factor	F2.1.1, 1.1, 1.1.1, 1.2.2, 1.4.1
λ_c	Slenderness factor for column buckling	E2
λ_c	Slenderness factor	1.1
λ_f	Slenderness factor of *flange* in determining torsion bimoment strength	G8.1
λ_ℓ	Slenderness factor for *local buckling* of column or beam	E3.2, F3.2,
λ_ℓ	Slenderness factor for *local buckling* of column or beam	3.3
λ_d	Slenderness factor for *distortional buckling* of column or beam	E4, F4, 3.4

SYMBOLS

Symbol	Definition	Section
ℓ_{dp}	*PAF* point length	J5, J5.2.2, J5.3.2
λ_t	Slenderness factor	1.1.4
λ_v	Slenderness factor	G2.1, G2.2
λ_w	Slenderness factor of *web* in determining torsion bimoment strength	G8.1
λ_1, λ_2, λ_3, λ_4	Parameters used in determining compression strain factor	F2.1.1
μ	Poisson's ratio of steel = 0.30	G2.3, 1.1, 1.4.1, 2.3.2.1, 2.3.2.2, 2.3.3.1, 2.3.3.2, 2.3.4
ξ_f	*Stress* gradient in *flange* and edge stiffener	2.3.3.2
ξ_{web}	*Stress* gradient in *web*	2.3.3.2
ρ	Reduction factor	1.1, 1.1.4, 1.2.2, 1.4.1
ρ_m	Reduction factor	1.1.4
ρ_{st}	Ratio of design *yield stress* of web and design *yield stress* of stiffener	G4.1
ρ_t	Reduction factor	1.1.4
τ_b	Parameter for reduced stiffness using *second-order analysis*	C1.1.1.3
ϕ	*Resistance factor*	A1.2.6, 1.2.7, A1.3, B3.2.2, B3.2.3, B4, B4.1, B4.2, C2.3.1, G2, H3, I6.2.3, I6.2.4, I6.3.1, I6.4.1, I6.4.2, J2.1, J2.2.2.1, J2.2.2.2, J2.2.3, J2.3.2.1, J2.3.2.2, J2.4.1, J2.6, J2.7, J2.8, J3.3.1, J3.3.2, J3.4, J3.5, J4, J4.3.1, J4.3.2, J4.3.3, J4.4.1, J4.4.2, J4.4.3, J4.5.1, J4.5.2, J4.5.3, J5, J5.2.1, J5.2.2, J5.2.3, J5.3.1, J5.3.2, J5.3.3, J6, K2.1.1, K2.1.2, 3.1.2
ϕ_a	ϕ_c or axial compression, $P_r > 0$ and ϕ_t for axial tension, $P_r < 0$	3.1.2
ϕ_b	*Resistance factor* for bending strength	F1, F2, F2.3, F3, F4, G8.1, H1.1, I6.1.2, I6.2.1, I6.2.2, 3.1.2
ϕ_c	*Resistance factor* for concentrically loaded compression strength	A3.2.1, E1, E2, E3, E4, G7.1, G7.2, I6.1.1, 3.1.2
ϕ_t	*Resistance factor* for tension strength	D1, D2, D3, J3.4 (Appendix B), 3.1.2

SYMBOLS

Symbol	Definition	Section
ϕ_v	*Resistance factor* for shear strength	G2, J3.4 (Appendix B)
ϕ_w	*Resistance factor* for *web crippling* strength	G5
φ	Coefficient	2.3.4
φ_{PM}	Angle (in radians) quantifying relative magnitude of axial and flexural demand as defined in *Eq*. 3.1.2-4	3.1.2, 3.2, 3.4
ω_i	Coefficient	1.4.1.2
ψ	$\lvert f_2/f_1 \rvert$	F2.1.1, 1.1.2, 1.2.2
ψ_f	*Stress* ratio	2.3.3.2
Δ_F	Inter-story drift from *first-order* elastic *analysis* in the direction of translation being considered, due to story shear, $\overline{F}$, computed using the *stiffness* as required by Section C1.2.1.3	C1.2.1.1
Δ_{tf}	Lateral displacement of *purlin* top *flange* at the line of restraint	I6.4.1
Ω	*Safety factor*	A1.2.6, A1.2.7, A1.3, B3.2.1, B4, B4.1, B4.2, C2.3.1, G2, I6.2.3, I6.2.4, I6.3.1, I6.4.1, I6.4.2, J2.1, J2.2.2.1, J2.2.2.2, J2.2.3, J2.3.2.1, J2.3.2.2, J2.4.1, J2.6,J2.7, J2.8, J3.3.1, J3.3.2, J3.4, J3.5, J4, J4.3.1, J4.3.2, J4.3.3, J4.4.1, J4.4.2, J4.4.3, J4.5.1, J4.5.2, J4.5.3, J5, J5.2.1, J5.2.2, J5.2.3, J5.3.1, J5.3.2, J5.3.3, J6, K2.1.2, 3.1.2
Ω_a	Ω_c for axial compression, $P_r > 0$ and Ω_t for axial tension, $P_r < 0$	3.1.2
Ω_b	*Safety factor* for bending strength	F1, F2, F2.3, F3, F4, G8.1, H1.1, I6.1.2, I6.2.1, I6.2.2, 3.1.2
Ω_c	*Safety factor* for concentrically loaded compression strength	A3.2.1, E1, E2, E3, E4, G7.1, G7.2, I6.1.1, 3.1.2
Ω_t	*Safety factor* for tension strength	D1, D2, D3, 3.1.2
Ω_v	*Safety factor* for shear strength	G2
Ω_w	*Safety factor* for *web crippling* strength	G5
$\omega_{n,max}$	Maximum magnitude of normalized unit warping property of cross-section, taken as positive	G8.1

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TABLE OF CONTENTS

NORTH AMERICAN SPECIFICATION FOR THE DESIGN OF COLD-FORMED STEEL STRUCTURAL MEMBERS

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NORTH AMERICAN SPECIFICATION FOR THE DESIGN OF COLD-FORMED STEEL STRUCTURAL MEMBERS

A. GENERAL PROVISIONS

This chapter addresses the scope and applicability of the *Specification*, lists the definitions of the terminology used, summarizes referenced specifications, codes, and standards, and provides requirements for materials.

This chapter is organized as follows:

A1 Scope, Applicability, and Definitions
A2 Referenced Specifications, Codes, and Standards
A3 Material

A1 Scope, Applicability, and Definitions

A1.1 Scope

This *Specification* applies to the design of *structural members* cold-formed to shape from carbon or low-alloy steel sheet, strip, plate, or bar not more than 1 in. (25.4 mm) in *thickness* and used for *load*-carrying purposes in

(a) Buildings, and
(b) Structures other than buildings provided allowances are made for dynamic effects.

A1.2 Applicability

A1.2.1 This *Specification* includes Symbols, Chapters A through M, Appendices A and B, and Appendices 1 through 5 that shall apply as follows:

- Chapters A through M, Appendices 1 through 5—the United States, Mexico, and Canada,
- Appendix A—the United States and Mexico, and
- Appendix B—Canada.

A1.2.2 The symbol ☞$^{\underline{x}}$ is used to point out that additional provisions that are specific to a certain country are provided in the corresponding appendices indicated by the letter(s) "x."

A1.2.3 This *Specification* includes design provisions for *Allowable Strength Design* (*ASD*), *Load and Resistance Factor Design* (*LRFD*), and *Limit States Design* (*LSD*). These design methods shall apply as follows:

- *ASD* and *LRFD*—the United States and Mexico, and
- *LSD*—Canada.

A1.2.4 In this *Specification*, bracketed terms are equivalent terms that apply particularly to *LSD*.

A1.2.5 The *nominal strength* [*resistance*] and stiffness of cold-formed steel components such as elements, members, assemblies, *connections*, and details shall be determined in accordance with the provisions in Chapters B through M, Appendices A and B, and Appendices 1 through 5 of the *Specification*.

A1.2.6 Where the composition or configuration of the components is such that calculation of *available strength* [*factored resistance*] or stiffness cannot be made in accordance with these provisions (excluding those in Chapter K), structural performance shall be established from one of the following:

(a) *Available strength* [*factored resistance*] or stiffness by tests only. Specifically, the *available strength* [*factored resistance*] is determined from tested *nominal strength* [*resistance*] by applying the *safety factors* or the *resistance factors* evaluated in accordance with Section K2.1.1(a);

(b) *Available strength* [*factored resistance*] by *rational engineering analysis* with confirmatory tests. Specifically, the *available strength* [*factored resistance*] is determined from the calculated *nominal strength* [*resistance*] by applying the *safety factors* or *resistance factors* evaluated in accordance with Section K2.1.1(b);

(c) *Available strength* [*factored resistance*] or stiffness by *rational engineering analysis* based on appropriate theory and engineering judgment. Specifically, the *available strength* [*factored resistance*] is determined from the calculated *nominal strength* [*resistance*] by applying the following *safety factors* or *resistance factors*:

For members

Ω = 2.00 (*ASD*)

ϕ = 0.80 (*LRFD*)

= 0.75 (*LSD*)

For *connections*

Ω = 3.00 (*ASD*)

ϕ = 0.55 (*LRFD*)

= 0.50 (*LSD*)

A1.2.7 When *rational engineering analysis* is used in accordance with Section A1.2.6(b) or A1.2.6(c) to determine the *nominal strength* [*resistance*] for a *limit state* already provided in this *Specification*, the *safety factor* shall not be less than the applicable *safety factor* (Ω), nor shall the *resistance factor* exceed the applicable *resistance factor* (ϕ) for the prescribed *limit state*. The determined *safety* and *resistance factors* shall be used in the interaction equations of Chapter H, which involves the applicable *limit states*.

A1.2.8 This *Specification* shall govern over standards referenced in this *Specification*, in matters pertaining to elements falling within the scope of this *Specification* as defined in Section A1.1. Where conflicts between this *Specification* and the *applicable building code* occur, the requirements of the *applicable building code* shall govern. In areas without an *applicable building code*, this *Specification* defines the minimum acceptable standards for elements falling within the scope of this *Specification*, as defined in Section A1.1.

A1.2.9 This *Specification* does not preclude the use of other *approved* materials, assemblies, structures or designs of equivalent performance.

A1.3 Definitions

In this *Specification*, "shall" is used to express a mandatory requirement, i.e., a provision that the user is obliged to satisfy in order to comply with the *Specification*; and "is permitted" is used to express an option or that which is permissible within the limits of the *Specification*. In standards developed by the CSA Group, "is permitted" is expressed by "may."

The following terms are italicized when they appear in the *Specification*. Definitions listed under the *ASD* and *LRFD* Terms sections shall apply to the USA and Mexico, while definitions listed under the *LSD* Terms section shall apply in Canada.

Terms designated with * are usually qualified by the type of *load* effect; for example, *nominal tensile strength*, *available compressive strength*.

Terms designated with ✦ are common AISC-AISI terms that are coordinated between the two standards developers.

General Terms

Applicable Building Code✦. Building code under which the structure is designed.

Approved. Acceptable to the *authority having jurisdiction.*

Authority Having Jurisdiction. An organization, political subdivision, office, or individual charged with the responsibility of administering and enforcing the provisions of the *applicable building code*.

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.

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 *loads* and provides stability for the structural system.

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

*Buckling Strength**. *Nominal strength* [*resistance*] for *instability limit states*.

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.

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

Confirmatory Test. Test made, when desired, on members, *connections*, and assemblies designed in accordance with this *Specification* or its specific references, or *rational engineering analysis,* in order to compare actual to calculated performance.

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

Cross-Sectional Area:

Effective Area. *Effective area*, A_e, calculated using the *effective widths* of component elements in accordance with Appendix 1. If the *effective widths* of all component elements, determined in accordance with Appendix 1, are equal to the actual *flat widths*, it equals the *gross* or *net area*, as applicable.

Full, Unreduced Area. *Full, unreduced area*, A. calculated without considering *local buckling* in the component elements, which equals either the *gross area* or *net area*, as applicable.

Gross Area. *Gross area*, A_g, without deductions for holes, openings, and cutouts.

Net Area. *Net area*, A_{net}, equal to *gross area* less the area of holes, openings, and cutouts.

Curtain Wall Stud. A member in a steel-framed exterior wall system that transfers transverse (out-of-plane) *loads* and is limited to a superimposed axial *load*, exclusive of sheathing materials, of not more than 100 lb/ft (1460 N/m or 1.49 kg/cm), or a superimposed axial *load* of not more than 200 lbs (890 N or 90.7 kg) per stud.

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.

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 members by reducing stiffness and applying *notional loads* in a *second-order analysis*.

Direct Strength Method. A design method that provides predictions of member strengths without the use of *effective widths*.

Distortional Buckling. A mode of *buckling* involving change in cross-sectional shape, excluding *local buckling*.

Doubly-Symmetric Section. A section symmetric about two orthogonal axes through its centroid.

Effective Design Width (Effective Width). *Flat width* of an element reduced for design purposes, also known simply as the *effective width*.

Effective Length. Length of an otherwise identical column of the same strength when analyzed with pinned end conditions.

Effective Length Factor, K. Ratio between the *effective length* and the *unbraced length* of the member.

Effective Length Method. A method of design that addresses *stability* through calculation of *available strength* [*factored resistance*] using the *effective length factor*.

Effective Width Method. A method that considers the *local buckling* of cold-formed steel members by reducing the gross cross-section under a non-linear *stress* distribution to an effective cross-section under a simplified linear *stress* distribution.

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

Factored Load✦. Product of a *load factor* and the *nominal load* [*specified load*].

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

Fire Barrier. Element of construction formed of fire-resisting materials and tested in accordance with an approved standard *fire resistance* test, to demonstrate compliance with the *applicable building code*.

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

First-Order Analysis. Structural analysis in which equilibrium conditions are formulated on the undeformed structure; *second-order effects* are neglected.

Flange of a Section in Bending (Flange). *Flat width* of *flange* including any intermediate stiffeners plus adjoining corners.

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

Flat Width. Width of an element exclusive of corners measured along its plane.

Flat-Width-to-Thickness Ratio (Flat Width Ratio). *Flat width* of an element measured along its plane, divided by its *thickness*.

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

Flexural-Torsional Buckling✦. *Buckling* mode in which a compression member bends and twists simultaneously without change in cross-sectional shape.

Girt✦. Horizontal *structural member* that supports wall panels and is primarily subjected to bending under horizontal *loads*, such as wind *load*.

In-Plane Instability✦. *Limit state* involving *buckling* in the plane of the frame or the member.

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

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

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

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

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

Load Effect✦. Forces, *stresses*, and deformations produced in a *structural component* by the applied *loads*.

Load Factor. A factor defined by the *applicable building code* to take into account the variability in *loads* and the analysis of their effects.

Local Bending✦. *Limit state* of large deformation of a *flange* under a concentrated transverse force.

Local Buckling. *Limit state* of *buckling* of a compression element where the line junctions between elements remain straight and angles between elements do not change.

Local Yielding✦. Yielding that occurs in a local area of an element.

Master Coil. One continuous, weld-free coil as produced by a hot mill, cold mill, metallic coating line, or paint line and identifiable by a unique coil number. In some cases, this coil is cut into smaller coils or slit into narrower coils; however, all of these smaller and/or narrower finished coils are said to have come from the same *master coil* if they are traceable to the original *master coil* number.

Moment Frame✦. Framing system that provides resistance to lateral *loads* and provides *stability* to the structural system primarily by shear and flexure of the framing members and their *connections*.

Multiple-Stiffened Element. Element stiffened between *webs*, or between a *web* and a stiffened edge, by means of intermediate stiffeners parallel to the direction of *stress*.

Non-symmetric Section. Section not symmetric about either an axis or a point.

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*.

Performance Test. Test made on structural members, *connections*, and assemblies whose performance cannot be determined in accordance with Chapters A through J and L through M of this *Specification* or its specific references.

Permanent Load✦. *Load* in which variations over time are rare or of small magnitude. All other *loads* are *variable loads*.

Point-Symmetric Section. Section symmetrical about a point (centroid) such as a Z-section having equal *flanges*.

Power-Actuated Fastener (PAF). Steel fastener, intended to be driven through steel members into embedment material using either powder or gas cartridges, or compressed air or other gas as the energy-driving source.

Power-Actuated Fastener Point. Portion of pointed end of *PAF* shank with varying diameter.

Published Specification. Requirements for a steel listed by a manufacturer, processor, producer, purchaser, or other body, which (a) are generally available in the public domain or are available to the public upon request, (b) are established before the steel is ordered, and (c) as a minimum, specify minimum mechanical properties, chemical composition limits, and, if coated sheet, coating properties.

Purlin✦. Horizontal structural member that supports roof deck and is primarily subjected to bending under vertical *loads* such as snow, wind, or dead *loads*.

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

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

Rational Engineering Analysis✦. Analysis based on theory that is appropriate for the situation, any relevant test data, if available, and sound engineering judgment.

Resistance Factor, ϕ✦. Factor that accounts for unavoidable deviations of the *nominal strength* [*resistance*] from the actual strength and for the manner and consequences of failure.

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

Second-Order Analysis. Structural analysis in which equilibrium conditions are formulated on the deformed structure; *second-order effects* (both *P-δ* and *P-Δ effects*, unless specified otherwise) are included.

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

Service Load✦. *Load* under which *serviceability limit states* are evaluated.

Serviceability Limit State✦. Limiting condition affecting the ability of a structure to preserve its appearance, maintainability, durability, 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 Wall✦. Wall that provides resistance to lateral *loads* in the plane of the wall and provides stability for the structural system.

Singly-Symmetric Section. Section symmetric about only one axis through its centroid.

Specified Minimum Yield Stress✦. Lower limit of *yield stress* specified for a material as defined by ASTM.

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.

Stiffened or Partially Stiffened Compression Elements. Flat compression element (i.e., a plane compression *flange* of a flexural member or a plane *web* or *flange* of a compression member) of which both edges parallel to the direction of *stress* are stiffened either by a *web*, *flange*, stiffening lip, intermediate stiffener, or the like.

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).

Stress. *Stress* as used in this *Specification* means force per unit area.

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 Members. See the definition of Cold-Formed Steel Structural Member.

Sub-Element of a Multiple Stiffened Element. Portion of a multiple stiffened element between adjacent intermediate stiffeners, between *web* and intermediate stiffener, or between edge and intermediate stiffener.

Tensile Strength (of Material)✦. Maximum tensile *stress* that a material is capable of sustaining as defined by ASTM.

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

Thickness. The *thickness*, t, of any element or section is the base steel *thickness*, exclusive of coatings.

Top Arc Seam Sidelap Weld. Arc seam weld applied to the *top sidelap connection*.

Top Sidelap Connection. A *connection* formed by a vertical sheet leg (edge stiffener of deck) inside an overlapping sheet hem, or by vertical sheet legs back-to-back.

Torsional Buckling✦. *Buckling* mode in which a compression member twists about its shear center axis.

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

Unstiffened Compression Elements. Flat compression element stiffened at only one edge parallel to the direction of *stress*.

Variable Load✦. *Load* not classified as *permanent load.*

Virgin Steel. Steel as received from the steel producer or warehouse before being cold worked as a result of fabricating operations.

Virgin Steel Properties. Mechanical properties of *virgin steel* such as *yield stress*, *tensile strength*, and elongation.

Wall Diaphragm. A wall, load-bearing or non-load-bearing, designed to resist forces acting in the plane of the wall (commonly referred to as a "vertical *diaphragm" or "shear wall*").

Web. In a member subjected to flexure, the portion of the section that is joined to two *flanges*, or that is joined to only one *flange* provided it crosses the neutral axis.

Web Crippling✦. *Limit state* of local failure of *web* plate in the immediate vicinity of a concentrated *load* or reaction.

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

Yield Point✦. First *stress* in a material at which an increase in strain occurs without an increase in *stress* as defined by ASTM.

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

Yield Stress✦. Generic term to denote either *yield point* or *yield strength*, as appropriate for the material.

Yielding✦. *Limit state* of inelastic deformation that occurs 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.*

ASD and LRFD Terms (United States and Mexico):

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).

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

*Available Strength**✦. *Design strength* or *allowable strength* as appropriate.

Design Earthquake. The ground motion represented by the design response spectrum as specified in the applicable building code.

*Design Load**✦. Applied *load* determined in accordance with either *LRFD load combinations* or *ASD load combinations*, whichever is applicable.

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

LRFD (Load and Resistance Factor Design)✦. Method of proportioning structural components such that the *design strength* equals or exceeds the *required strength* of the component under the action of the *LRFD load combinations.*

LRFD Load Combination✦. *Load* combination in the *applicable building code* intended for strength design (*Load and Resistance Factor Design*).

*Nominal Load**✦. The magnitudes of the *load* specified by the *applicable building code*.

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

*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*.

Resistance. See the definition of *Nominal Strength*.

Risk Category. A categorization of buildings and other structures for determination of flood, wind, snow, ice, and earthquake *loads* based on the risk associated with unacceptable performance.

Safety Factor, Ω^{+}. Factor that accounts for deviations of the actual strength from the *nominal strength,* deviations of the actual *load* from the *nominal load*, uncertainties in the analysis that transforms the *load* into a *load effect,* and for the manner and consequences of failure.

Seismic Design Category (SDC). A classification assigned by the *applicable building code* to a structure based on its risk category and the severity of the *design earthquake* ground motion at the site.

Span Continuity. Ability of a member to develop moment over a support.

Strength Limit State^{+}. Limiting condition, in which the maximum strength of a structure or its components is reached.

LSD Terms (Canada):

Factored Resistance. Product of *nominal resistance* and appropriate *resistance factor*.

Limit States Design (LSD). A method of proportioning *structural components* (members, connectors, connecting elements, and assemblages) such that no applicable *limit state* is exceeded when the structure is subjected to all appropriate *load* combinations.

Nominal Resistance (Resistance). The capacity of a structure or component to resist the *effects of loads*, determined in accordance with this *Specification* using specified material strengths and dimensions.

Specified Loads. The magnitudes of the *loads* specified by the *applicable building code,* not including *load factors*.

A1.4 Units of Symbols and Terms

Any compatible system of measurement units is permitted to be used in the *Specification*, except where explicitly stated otherwise. The unit systems considered in those sections shall include U.S. customary units (force in kilopounds and length in inches), SI units (force in Newtons and length in millimeters), and MKS units (force in kilograms and length in centimeters).

A2 Referenced Specifications, Codes, and Standards

The following documents or portions thereof are referenced in this *Specification* and shall be considered part of the requirements of this *Specification*. Country-specific codes and standards are listed in Section A2.1 for the United States and Mexico, and Section A2.2 for Canada.

1. American Iron and Steel Institute (AISI), 25 Massachusetts Avenue, NW, Suite 800, Washington, DC 20001:

   AISI S310-24, *North American Standard for the Design of Profiled Steel Diaphragm Panels*

2. American Society of Civil Engineers (ASCE), 1801 Alexander Bell Drive, Reston, VA 20191:

   ASCE/SEI/SFPE 29-05, *Standard Calculation Methods for Structural Fire Protection*

3. American Society of Mechanical Engineers (ASME), Two Park Avenue, New York, NY 10016-5990:

ASME B46.1-2019, *Surface Texture, Surface Roughness, Waviness, and Lay*

4. ASTM International (ASTM), 100 Barr Harbor Drive, West Conshohocken, PA 19428-2959:

ASTM A36/A36M-19, *Standard Specification for Carbon Structural Steel*

ASTM A194/A194M-23, *Standard Specification for Carbon Steel, Alloy Steel, and Stainless Steel Nuts for Bolts for High Pressure or High Temperature Service, or Both*

ASTM A242/A242M-24, *Standard Specification for High-Strength Low-Alloy Structural Steel*

ASTM A283/A283M-24, *Standard Specification for Low and Intermediate Tensile Strength Carbon Steel Plates*

ASTM A307-21, *Standard Specification for Carbon Steel Bolts, Studs, and Threaded Rod 60,000 PSI Tensile Strength*

ASTM A354-17e2, *Standard Specification for Quenched and Tempered Alloy Steel Bolts, Studs, and Other Externally Threaded Fasteners*

ASTM A370-24, *Standard Test Methods and Definitions for Mechanical Testing of Steel Products*

ASTM A449-14(2020), *Standard Specification for Hex Cap Screws, Bolts and Studs, Steel, Heat Treated, 120/105/90 ksi Minimum Tensile Strength, General Use*

ASTM A463/A463M-22, *Standard Specification for Steel Sheet, Aluminum-Coated, by the Hot-Dip Process*

ASTM A500/A500M-23, *Standard Specification for Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes*

ASTM A529/A529M-19, *Standard Specification for High-Strength Carbon-Manganese Steel of Structural Quality*

ASTM A563-23, *Standard Specification for Carbon and Alloy Steel Nuts (Inch and Metric)*

ASTM A572/A572M-21e1, *Standard Specification for High-Strength Low-Alloy Columbium-Vanadium Structural Steel*

ASTM A588/A588M-19, *Standard Specification for High-Strength Low-Alloy Structural Steel, Up to 50 ksi [345 MPa] Minimum Yield Point, With Atmospheric Corrosion Resistance*

ASTM A606/A606M-23, *Standard Specification for Steel, Sheet and Strip, High-Strength, Low-Alloy, Hot-Rolled and Cold-Rolled, With Improved Atmospheric Corrosion Resistance*

ASTM A653/A653M-23, *Standard Specification for Steel Sheet, Zinc-Coated (Galvanized) or Zinc-Iron Alloy-Coated (Galvannealed) by the Hot-Dip Process*

ASTM A751-21, *Standard Test Methods and Practices for Chemical Analysis of Steel Products*

ASTM A792/A792M-23, *Standard Specification for Steel Sheet, 55% Aluminum-Zinc Alloy-Coated by the Hot-Dip Process*

ASTM A847/A847M-21, *Standard Specification for Cold-Formed Welded and Seamless High-Strength, Low-Alloy Structural Tubing With Improved Atmospheric Corrosion Resistance*

ASTM A875/A875M-23, *Standard Specification for Steel Sheet, Zinc-5% Aluminum Alloy-Coated by the Hot-Dip Process*

ASTM A924/A924M-22a, *Standard Specification for General Requirements for Steel Sheet, Metallic-Coated by the Hot Dip Process*

ASTM A1003/A1003M-23, *Standard Specification for Steel Sheet, Carbon, Metallic- and Nonmetallic-Coated for Cold-Formed Framing Members*

ASTM A1008/A1008M-23e1, *Standard Specification for Steel, Sheet, Cold-Rolled, Carbon, Structural, High-Strength Low-Alloy, High-Strength Low-Alloy With Improved Formability, Required Hardness, Solution Hardened, and Bake Hardenable*

ASTM A1011/A1011M-23, *Standard Specification for Steel, Sheet and Strip, Hot-Rolled, Carbon, Structural, High-Strength Low-Alloy, High-Strength Low-Alloy With Improved Formability, and Ultra-High Strength*

ASTM A1018/A1018M-23a, *Standard Specification for Steel, Sheet and Strip, Heavy-Thickness Coils, Hot-Rolled, Carbon, Commercial, Drawing, Structural, High-Strength Low-Alloy, High-Strength Low-Alloy With Improved Formability, and Ultra-High Strength*

ASTM A1039/A1039M-20, *Standard Specification for Steel, Sheet, Hot Rolled, Carbon, Commercial, Structural, and High-Strength Low-Alloy, and Ultra-High Strength, Produced by Twin-Roll Casting Process*

ASTM A1046/A1046M-23, *Standard Specification for Steel Sheet, Zinc-Aluminum-Magnesium Alloy-Coated by the Hot-Dip Process*

ASTM A1058-19, *Standard Test Methods for Mechanical Testing of Steel Products—Metric*

ASTM A1063/A1063M-22, *Standard Specification for Steel Sheet, Twin-Roll Cast, Zinc-Coated (Galvanized) by the Hot-Dip Process*

ASTM A1079-17(2023), *Standard Specification for Steel Sheet, Complex Phase (CP), Dual Phase (DP) and Transformation Induced Plasticity (TRIP), Zinc-Coated (Galvanized) or Zinc-Iron Alloy-Coated (Galvannealed) by the Hot-Dip Process*

ASTM A1083/A1083M-23, *Standard Specification for Steel, Sheet, Cold-Rolled, Carbon, Structural, High-Strength Low-Alloy, Produced by Twin-Roll Casting Process*

ASTM A1085/A1085M-22, *Standard Specification for Cold-Formed Welded Carbon Steel Hollow Structural Sections (HSS)*

ASTM A1088-13(2019), *Standard Specification for Steel, Sheet, Cold-Rolled, Complex Phase (CP), Dual Phase (DP) and Transformation Induced Plasticity (TRIP)*

ASTM E119-22, *Standard Test Methods for Fire Tests of Building Construction and Materials*

ASTM E1592-05(2017), *Standard Test Method for Structural Performance of Sheet Metal Roof and Siding Systems by Uniform Static Air Pressure Difference*

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

ASTM F606/F606M-21, *Standard Test Methods for Determining the Mechanical Properties of Externally and Internally Threaded Fasteners, Washers, Direct Tension Indicators, and Rivets*

ASTM F844-19, *Standard Specification for Washers, Steel, Plain (Flat), Unhardened for General Use*

ASTM F959/F959M-17a, *Standard Specification for Compressible-Washer-Type Direct Tension Indicators for Use With Structural Fasteners, Inch and Metric Series*

ASTM F3125/F3125M-23, *Standard Specification for High Strength Structural Bolts and Assemblies, Steel and Alloy Steel, Heat Treated, Inch Dimensions 120 ksi and 150 ksi Minimum Tensile Strength, and Metric Dimensions 830 MPa and 1040 MPa Minimum Tensile Strength*

User Note:

ASTM F3125 is an umbrella standard including Grades A325, A325M, A490, and A490M, which were previously separate standards.

ASTM F3148-17a, *Standard Specification for High Strength Structural Bolt Assemblies, Steel and*

Alloy Steel, Heat Treated, 144ksi Minimum Tensile Strength, Inch Dimensions

5. CSA Group, 178 Rexdale Boulevard, Toronto, Ontario, Canada, M9W 1R3:

   G40.20-13/G40.21-13(R2023), *General Requirements for Rolled or Welded Structural Quality Steel/Structural Quality Steel*

6. Factory Mutual, Corporate Offices, 270 Central Avenue, Johnston, RI 02919-4949:

   FM 4471, *Approval Standard for Class 1 Metal Roofs*, 2010

7. SAE International, 400 Commonwealth Drive, Warrendale, PA 15096

   SAE J429_201405, *Mechanical and Material Requirements for Externally Threaded Fasteners*

   SAE J995_201707, *Mechanical and Material Requirements for Steel Nuts*

   SAE J2486_201804, *Tension Indicating Washer Tightening Method for Fasteners*

   SAE J2655_201509, *Fastener Part Standard - Washers and Lockwashers (Inch Dimensioned)*

7. Steel Deck Institute, P.O. Box 70, Florence, South Carolina 29503

   ANSI SDI AISI S901-2017(R2024), *Test Standard for Determining the Rotational-Lateral Stiffness of Beam-to-Panel Assemblies*

   ANSI SDI AISI S902-2024, *Test Standard for Determining the Effective Area of Cold-Formed Steel Compression Members*

   ANSI SDI AISI S903-2020(R2024), *Test Standard for Determining the Uniform and Local Ductility of Carbon and Low-Alloy Steels*

   ANSI SDI AISI S904-2017(R2024), *Test Standard for Determining the Tensile and Shear Strengths of Steel Screws*

   ANSI SDI AISI S905-2024, *Test Standard for Determining the Strength and Deformation Characteristics of Cold-Formed Steel Connections*

   ANSI SDI AISI S906-2017(R2024), *Test Standard for Determining the Load-Carrying Strength of Panels and Anchor-to-Panel Attachments for Roof or Siding Systems Tested in Accordance With ASTM E1592*

   ANSI SDI AISI S907-2017(R2024), *Test Standard for Determining the Strength and Stiffness of Cold-Formed Steel Diaphragms by the Cantilever Test Method*

   ANSI SDI AISI S908-2017(R2024), *Test Standard for Determining the Flexural Strength Reduction Factor of Purlins Supporting a Standing Seam Roof System*

   ANSI SDI AISI S909-2017(R2024), *Test Standard for Determining the Web Crippling Strength of Cold-Formed Steel Flexural Members*

   ANSI SDI AISI S910-2017(R2024), *Test Standard for Determining the Distortional Buckling Strength of Cold-Formed Steel Hat-Shaped Compression Members*

   ANSI SDI AISI S911-2017(R2024), *Test Standard for Determining the Flexural Strength of Cold-Formed Steel Hat-Shaped Members*

   ANSI SDI AISI S912-2024, *Test Standard for Determining the Strength of a Roof Panel-to-Purlin-to-Anchorage Device Connection*

   ANSI SDI AISI S913-2017(R2024), *Test Standard for Determining the Strength and Deformation Behavior of Hold-Downs Attached to Cold-Formed Steel Structural Framing*

   ANSI SDI AISI S914-2017(R2024), *Test Standard for Determining the Strength and Deformation Behavior of Joist Connectors Attached to Cold-Formed Steel Structural Framing*

ANSI SDI AISI S915-2020(R2024), *Test Standard for Determining the Strength and Deformation Behavior of Through-the-Web Punchout Cold-Formed Steel Wall Stud Bridging Connectors*

ANSI SDI AISI S916-2020(R2024), *Test Standard for Determining the Strength and Stiffness of Cold-Formed Steel Framed Nonstructural Interior Partition Walls Sheathed With Gypsum Board*

ANSI SDI AISI S917-2017(R2024), *Test Standard for Determining the Fastener-Sheathing Local Translational Stiffness of Sheathed Cold-Formed Steel Assemblies*

ANSI SDI AISI S918-2017(R2024), *Test Standard for Determining the Fastener-Sheathing Rotational Stiffness of Sheathed Cold-Formed Steel Assemblies*

ANSI SDI AISI S919-2017(R2024), *Test Standard for Determining the Flexural Strength and Stiffness of Cold-Formed Steel Nonstructural Members*

ANSI SDI AISI S920-2024, *Test Standard for Determining the Screw Penetration Through Gypsum Board Into Cold-Formed Steel Framing Members*

ANSI SDI AISI S921-2019(R2024), *Test Standard for Determining the Strength and Serviceability of Cold-Formed Steel Truss Assemblies and Components*

ANSI SDI AISI S922-2019(R2024), *Test Standard for Determining the Strength and Stiffness of Bearing-Friction Interference Connector Assemblies in Profiled Steel Panels*

ANSI SDI AISI S923-2024, *Test Standard for Determining the Strength and Stiffness of Shear Connections of Composite Members*

ANSI SDI AISI S924-2020(R2024), *Test Standard for Determining the Effective Flexural Stiffness of Composite Members*

8. U. S. Army Corps of Engineers, 441 G Street NW, Washington, DC 20314-1000:

CEGS-07416, *Guide Specification for Military Construction, Structural Standing Seam Metal Roof (SSSMR) System*, 1995

A2.1 Referenced Specifications, Codes, and Standards for the United States and Mexico

1. American Concrete Institute (ACI), 38800 Country Club Dr., Farmington Hills, MI 48331:

ACI 318-19(22), *Building Code Requirements for Structural Concrete*

2. American Institute of Steel Construction (AISC), 130 East Randolph Street, Suite 2000, Chicago, IL 60601-6219:

ANSI/AISC 360-22, *Specification for Structural Steel Buildings*

3. American Society of Civil Engineers (ASCE), 1801 Alexander Bell Drive, Reston, VA 20191:

ASCE/SEI 7-22, *Minimum Design Loads and Associated Criteria for Buildings and Other Structures*

4. American Welding Society (AWS), 8669 NW 36 Street, # 130, Miami, FL 33166-6672:

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

AWS D1.3/D1.3M: 2018, *Structural Welding Code–Sheet Steel*

AWS C1.1M/C1.1: 2019, *Recommended Practices for Resistance Welding*

A2.2 Referenced Specifications, Codes, and Standards for Canada

1. American Iron and Steel Institute (AISI), 25 Massachusetts Avenue, NW, Suite 800, Washington, DC 20001:

AISI S240-20, *North American Standard for Cold-Formed Steel Structural Framing*

AISI S400-20, *North American Standard for Seismic Design of Cold-Formed Steel Structural Systems*

2. CSA Group, 178 Rexdale Boulevard, Toronto, Ontario, Canada, M9W 1R3:

CAN/CSA A23.3-19, *Design of Concrete Structures*

CSA S16:24, *Design and Construction of Steel Structures*

CSA W47.1:19, *Certification of Companies for Fusion Welding of Steel*

CSA W55.3:08 (R2023), *Certification of Companies for Resistance Welding of Steel and Aluminum*

CSA W59:24, *Welded Steel Construction (Metal Arc Welding)*

3. National Research Council of Canada (NRC), 1200 Montreal Road, Bldg. M-58, Ottawa, Ontario, Canada, K1A 0R6:

National Building Code of Canada (NBCC), 2020

4. Standards Council of Canada, 55 Metcalfe Street, Suite 600, Ottawa, ON K1P 6L5 Canada

CAN/ULC S101-14, *Standard Methods of Fire Endurance Tests of Building Construction and Materials*

A3 Material

This *Specification* requires the use of steels intended for structural applications as defined in general by the specifications of ASTM International listed in this section. The term SS designates structural steels and the terms HSLAS and HSLAS-F designate high-strength low-alloy steels. Steels that do not meet the requirements specified in Sections A3.1 are permitted to be used for structural applications provided Section A3.2 is met.

A3.1 Applicable Steels

This section shall apply to steels that are based on specifications providing mandatory mechanical properties and requiring test reports to confirm those properties.

Steels used in *structural members,* decks, and *connections* shall follow uses and restrictions outlined in this section and subsections, as applicable.

Exception: Requirements for steels used in composite slabs shall be in accordance with the *applicable building code*.

> **User Note:**
> Design of composite steel floor deck is governed by the *applicable building code* and standards published by the Steel Deck Institute (www.sdi.org).

Applicable steels have been grouped by their minimum elongation requirements over a two-inch (50-mm) gage length.

A3.1.1 Steels With a Specified Minimum Elongation of Ten Percent or Greater (Elongation ≥ 10%)

Steel grades listed in Table A3.1.1-1, as well as any other steel for structural applications, are permitted to be used without restriction under the provisions of this *Specification* provided:

(a) The ratio of *tensile strength* to *yield stress* is not less than 1.08; and

(b) The minimum elongation is greater than or equal to either 10 percent in a two-inch (50-mm) gage length or 7 percent in an eight-inch (200-mm) gage length standard specimen tested in accordance with ASTM A370 or ASTM A1058.

Table A3.1.1-1
Steel Grades With Elongation ≥ 10%

Designated Standard	Applicable Grades
ASTM A36/A36M	All Grades
ASTM A242/A242M	All Grades
ASTM A283/A283M	All Grades
ASTM A463/A463M	SS Grades 33 (230), 37 (255), 40 (275), 50 (340) Classes 1 and 3 HSLAS Grades Types A & B, 50 (340), 60 (410), 70 (480), 80 (550)
ASTM A500	All Grades
ASTM A529/A529M	All Grades
ASTM A572/A572M	All Grades
ASTM A588/A588M	All Grades
ASTM A606	All Grades
ASTM A653/A653M	SS Grades 33 (230), 37 (255), 40 (275), 50 (340) Classes 1, 3 and 4, 55 (380), 60 (410)[A], 70 (480)[A] HSLAS and HSLAS-F Grades 40 (275), 50 (340), 55 (380) Classes 1 and 2, 60 (410), 70 (480), 80 (550)
ASTM A792/A792M	SS Grades 33 (230), 37 (255), 40 (275), 50 (340) Classes 1 and 4, 60 (410)[A], 70 (480)[A] HSLAS and HSLAS-F Grades 40 (275), 50 (340), 55 (380) Classes 1 and 2, 60 (410), 70 (480), 80 (550)
ASTM A847/A847M	All Grades
ASTM A875/A875M	SS Grades 33 (230), 37 (255), 40 (275), 50 (340) Classes 1 and 3 HSLAS and HSLAS-F Grades 50 (340), 60 (410), 70 (480), 80 (550)
ASTM A1003/A1003M	ST Grades 33H (230), 37H (255), 40H (275), 50H (340), 55H (380), 60H (410), 70H (480), 80H (550)
ASTM A1008/A1008M	SS Grades 25 (170), 30 (205), 33 (230) Types 1 and 2, 40 (275) Types 1 and 2, 45 (310), 50 (340), 60 (410) HSLAS Grades Classes 1 and 2, 45 (310), 50 (340), 55 (380), 60 (410), 65 (450), 70 (480) HSLAS-F 50 (340), 60 (410), 70 (480), 80 (550)
ASTM A1011/A1011M	SS Grades 30 (205), 33 (230), 36 (250) Types 1 and 2, 40 (275), 45 (310) Types 1 and 2, 50 (340), 55 (380)B, 60 (410)B, 70 (480)B HSLAS Grades Classes 1 and 2, 45 (310), 50 (340), 55 (380), 60 (410), 65 (450), 70 (480) HSLAS-F Grades 50 (340), 60 (410), 70 (480), 80 (550) UHSS Grades Types 1 and 2, 90 (620) and 100 (690)
ASTM A1018/A1018M	SS Grades 30 (205), 33 (230), 36 (250) Types 1 and 2, 40 (275), 45 (310) HSLAS Grades Classes 1 and 2, 45 (310), 50 (340), 55 (380), 60 (410),

	65 (450), 70 (480) HSLAS-F Grades 50 (340), 60 (410), 70 (480), 80 (550) UHSS Grades Types 1 and 2 90 (620), 100 (690)
ASTM A1039/A1039M	SS Grades 30 (205), 33 (230), 36 (250) Types 1 and 2, 40 (275), 45 (310), 50 (340), 55 (380)B, 60 (410)B, 70 (480)B, 80 (550)B HSLAS Grades Classes 1 and 2, 45 (310), 50 (340), 55 (380), 60 (410), 65 (450)
ASTM A1046/A1046M	SS Grades 33 (230), 37 (255), 40 (275), 50 (340) Classes 1, 3 and 4 HSLAS Grades 40 (275), 50 (340), 60 (410), 70 (480), 80 (550) HSLAS-F Grades 40 (275), 50 (340), 60 (410), 70 (480), 80 (550)
ASTM A1063/A1063M	SS Grades 33 (230), 37 (255), 40 (275), 45 (310), 50 (340) HSLAS Grades Classes 1 and 2, 45 (310), 50 (340), 55 (380), 60 (410), 65 (450)
ASTM A1079	CP 600T/350Y, 780T/500Y DP 450T/250Y, 490T/290Y, 590T/340Y, 780T/420Y TRIP 690T/410, 780T/440Y
ASTM A1083/A1083M	SS Grades 25 (170), 30 (305), 33 (230) Types 1 and 2, 40 (275) Types 1 and 2, 45 (310), 50 (340), 60 (410) HSLAS Grades Classes 1 and 2, 45 (310), 50 (340), 55 (380), 60 (410), 65 (450), 70 (480), 80 (550)
ASTM A1085	Grade A
ASTM A1088	CP 600T/350Y, 780T/500Y DP 450T/250Y, 490T/290Y, 590T/340Y, 780T/420Y TRIP 690T/410Y, 780T/440Y
CSA G40.20/G40.21	All Grades

Notes:

A. ASTM A653/A653M SS Grades 60 (410), 70 (480) and ASTM A792/A792M SS Grades 60 (410) and 70 (480) with *thicknesses* less than or equal to 0.028 in. (0.71 mm) are excluded from this elongation group.

B. ASTM A1011/A1011M SS Grades 55 (380), 60 (410) and 70 (480); and ASTM A1039/A1039M SS Grades 55 (380), 60 (410), 70 (480) and 80 (550) with *thicknesses* less than 0.064 in. (1.6 mm) are excluded from this elongation group.

A3.1.2 Steels With a Specified Minimum Elongation From Three Percent to Less Than Ten Percent (3% ≤ Elongation < 10%)

Steel grades listed in Table A3.1.2-1, as well as any other steel for structural applications that has a minimum elongation of 3 percent in a two-inch (50-mm) gage length standard specimen tested in accordance with ASTM A370 or ASTM A1058, are permitted to be used provided that the *available strengths* [*factored resistances*] of *structural members* and *connections* are calculated in accordance with Chapters B through M (excluding welded *connections* in Chapter J), Appendices A and B, and Appendices 1 through 5. For the purposes of these calculations, a reduced *yield stress* 0.9 F_{sy} shall be used in place of F_{sy}, and a reduced *tensile strength* of 0.9 F_u shall be used in place of F_u.

Table A3.1.2-1
Steel Grades With 3% ≤ Elongation < 10%

Designated Standard	Applicable Grades
ASTM A653/A653M	SS Grades 60 (410)[A], 70 (480)[A], 80 (550) Class 3
ASTM A792/A792M	SS Grades 60 (410)[A], 70 (480)[A], 80 (550) Class 3
ASTM 1008/A1008M	SS Grades 70 (480)
ASTM 1011/A1011M	SS Grades 55 (380)[B], 60 (410)[B], 70 (480)[B]
ASTM 1039/A1039M	SS Grades 55 (380)[B], 60 (410)[B], 70 (480)[B], 80 (550)[B] HSLAS Grades Classes 1 and 2, 70 (480), 80 (550)
ASTM 1063/A1063M	SS Grades 55 (380), 60 (410), 70 (480), 80 (550) Class 1 HSLAS Grades Classes 1 and 2, 70 (480), 80 (550)
ASTM A1079	CP 980T/700Y DP 980T/550Y
ASTM 1083/A1083M	SS Grades 70 (480)
ASTM A1088	CP 980T/700Y DP 980T/550Y

Notes:

A. ASTM A653/A653M SS Grades 60 (410), 70 (480) and ASTM A792/A792M SS Grades 60 (410) and 70 (480) with *thicknesses* greater than 0.028 in. (0.71 mm) are excluded from this elongation group.

B. ASTM A1011/A1011M SS Grades 55 (380), 60 (410) and 70 (480); and ASTM A1039/A1039M SS Grades 55 (380), 60 (410), 70 (480) and 80 (550) with *thicknesses* greater than or equal to 0.064 in. (1.6 mm) are excluded from this elongation group.

A3.1.3 Steels With a Specified Minimum Elongation of Less Than Three Percent (Elongation < 3%)

Steel grades listed in Table A3.1.3-1, as well as other steel grades that do not meet the requirements of A3.1.1 or A3.1.2, are permitted to be used only for multiple *web* configurations such as roofing, siding, and floor decking provided the following adjustments are made to the design parameters:

(a) A reduced *specified minimum yield stress*, R_bF_{sy}, is used for determining the *nominal flexural strength* [*resistance*] in Chapter F, for which the reduction factor, R_b, is determined in accordance with (1) or (2):

(1) For stiffened and partially stiffened compression *flanges*

For $w/t \le 0.067E/F_{sy}$

$R_b = 1.0$

For $0.067E/F_{sy} < w/t < 0.974E/F_{sy}$

$$R_b = 1 - 0.26[wF_{sy}/(tE) - 0.067]^{0.4} \quad (Eq.\ A3.1.3\text{-}1)$$

For $0.974E/F_{sy} \le w/t \le 500$

$R_b = 0.75$

(2) For unstiffened compression *flanges*

For $w/t \le 0.0173E/F_{sy}$

$R_b = 1.0$

For $0.0173E/F_{sy} < w/t \leq 60$

$$R_b = 1.079 - 0.6\sqrt{wF_{sy}/(tE)} \quad (Eq.\ A3.1.3\text{-}2)$$

where

w = *Flat width* of compression *flange*

t = *Thickness* of section

E = Modulus of elasticity of steel

F_{sy} = *Specified minimum yield stress* determined in accordance with Section A3.3.1 $\leq$ 80 ksi (550 MPa, or 5620 kg/cm^2)

(b) The *yield stress*, F_y, used for determining *nominal strength* [*resistance*] in Appendix 1 and Chapters C through J exclusive of Chapter F is taken as 75 percent of the *specified minimum yield stress* or 60 ksi (414 MPa or 4220 kg/cm^2), whichever is less, and

(c) The *tensile strength*, F_u, used for determining *nominal strength* [*resistance*] in Chapter J is taken as 80 percent of the specified minimum *tensile strength* or 65 ksi (448 MPa or 4570 kg/cm^2), whichever is less.

Alternatively, the suitability of such steels for any multi-*web* configuration shall be demonstrated by load tests in accordance with the provisions of Section K2.1. *Available strengths* [*factored resistances*] based on these tests shall not exceed the *available strengths* [*factored resistances*] calculated in accordance with Chapters C through J, Appendices A and B, and Appendices 1 through 5, using the *specified minimum yield stress*, F_{sy}, and the specified minimum *tensile strength*, F_u.

Table A3.1.3-1
Steel Grades With Elongation < 3%

Designated Standard	Applicable Grades
ASTM A463/A463M	SS Grade 80 (550)
ASTM A653/A653M	SS Grades 80 (550) Classes 1 and 2
ASTM A792/A792M	SS Grades 80 (550) Classes 1 and 2
ASTM A875/A875M	SS Grade 80 (550)
ASTM A1008/A1008M	SS Grade 80 (550)
ASTM A1046/A1046M	SS Grade 80 (550)
ASTM A1063/A1063M	SS Grades 80 (550) Class 2
ASTM A1083/A1083M	SS Grade 80 (550)

A3.2 Other Steels

The listing in Section A3.1 shall not exclude the use of steel up to and including 1 in. (25.4 mm) in *thickness*, ordered or produced to other than the listed specifications, provided the following requirements are met:

(a) The steel shall conform to the chemical and mechanical requirements of one of the listed specifications or other *published specification*. F_y and F_u shall be the specified minimum values as given in the specified reference specification.

(b) The chemical and mechanical properties shall be determined by the producer, the supplier, or the purchaser, in accordance with the specified reference specification including all general requirements standards cited therein.

(c) The coating properties of coated sheet shall be determined by the producer, the supplier, or the purchaser, in accordance with ASTM A924/A924M.

(d) If the steel is to be welded, its suitability for the intended welding process shall be established by the producer, the supplier, or the purchaser, in accordance with AWS D1.1, AWS D1.3 or CSA W59, as applicable.

These steels shall also meet the permitted uses and restrictions of Section A3.1, as appropriate.

If the identification and documentation of the production of the steel have not been established, then in addition to requirements (a) through (d) in *Specification* Section A3.2, the manufacturer of the cold-formed steel product shall establish that the *yield stress* and *tensile strength* of the *master coil* are at least 10 percent greater than specified in the referenced *published specification*.

A3.2.1 Ductility Requirements of Other Steels

Steels not listed in Section A3.1 and used for *structural members* and *connections* in accordance with Section A3.2 shall comply with the following ductility requirements:

(a) Minimum local elongation in a 1/2-inch (12.7 mm) gage length across the fracture is 20 percent,

(b) Minimum uniform elongation outside the fracture is three percent, and

(c) Uniform and local elongation is determined in accordance with ANSI/SDI AISI S903 or another *approved* test standard.

When material ductility is determined on the basis of these criteria, the use of such material shall be restricted to the design of *purlins*, *girts*, and *curtain wall studs* in accordance with Chapter F, and Sections I6.2.1, I6.2.2, and I6.3.1. *Curtain wall studs* shall also be subject to the restrictions specified in Section A3.2.1.1. For *purlins*, *girts*, and *curtain wall studs* subject to combined axial *load* and bending moment (Section H1), $\frac{\Omega_c P}{P_n}$ shall not exceed 0.15 for *ASD*, $\frac{P_u}{\phi_c P_n}$ shall not exceed 0.15 for *LRFD*, and $\frac{P_f}{\phi_c P_n}$ shall not exceed 0.15 for *LSD*.

A3.2.1.1 Restrictions for Curtain Wall Studs

The use of *curtain wall studs* shall be limited to a wall assembly whose dead *load* divided by its surface area is no greater than 15 psf (0.72 kN/m^2 or 7.32 g/cm^2) in accordance with the following:

(a) In the United States and Mexico, where the building is assigned to *Seismic Design Category* D, E, or F; and

(b) In Canada, where the building has a specified short period spectral acceleration ratio $I_E F_a S_a(0.2)$ greater than 0.35, determined in accordance with the NBCC.

A3.3 Yield Stress and Strength Increase From Cold Work of Forming

A3.3.1 Yield Stress

The *yield stress*, F_y, used in design shall not exceed the *specified minimum yield stress* of steels as listed in Section A3.1, as established in accordance with Section K2, or as increased for cold work of forming in Section A3.3.2.

A3.3.2 Strength Increase From Cold Work of Forming

Strength increase from cold work of forming is permitted by substituting F_{ya} for F_y, where F_{ya} is the average *yield stress* of the full section. Such increase shall be limited to Chapters D, E, F (except as noted), Sections H1, I4, and I6.2; and to sections not subject to strength reduction from *local buckling*. The limits and methods for determining F_{ya} shall be in accordance with (a), (b) and (c).

(a) The design *yield stress*, F_{ya}, of the steel shall be determined on the basis of one of the following methods:

(1) Full section tensile tests [see paragraph (a) of Section K2.3.1],

(2) Stub column tests [see paragraph (b) of Section K2.3.1],

(3) Computed in accordance with Eq. A3.3.2-1:

$$F_{ya} = CF_{yc} + (1 - C)\, F_{yf} \leq F_{uv} \qquad (Eq.\ A3.3.2\text{-}1)$$

where

F_{ya} = Average *yield stress* of full unreduced section of compression members or full *flange* sections of flexural members

C = For compression members, ratio of total corner *cross-sectional area* to total *cross-sectional area* of full section; for flexural members, ratio of total corner *cross-sectional area* of controlling *flange* to full *cross-sectional area* of controlling *flange*

F_{yc} = $B_cF_{yv}/(R/t)^m$, tensile *yield stress* of corners (*Eq*. A3.3.2-2)

Eq. A3.3.2-2 applies only when $F_{uv}/F_{yv} \geq 1.2$, $R/t \leq 7$, and the included angle $\leq 120^{o}$

where

B_c = $3.69\,(F_{uv}/F_{yv}) - 0.819\,(F_{uv}/F_{yv})^2 - 1.79$ (*Eq*. A3.3.2-3)

F_{yv} = Tensile *yield stress* of *virgin steel* specified by Section A3 or established in accordance with Section K2.3.3

R = Inside bend radius

t = *Thickness* of section

m = $0.192\,(F_{uv}/F_{yv}) - 0.068$ (*Eq*. A3.3.2-4)

F_{uv} = *Tensile strength* of *virgin steel* specified by Section A3 or established in accordance with Section K2.3.3

F_{yf} = Weighted average tensile *yield stress* of flat portions established in accordance with Section K2.3.2 or *virgin steel yield stress* if tests are not made

(b) For axially loaded tension members, the *yield stress* of the steel shall be determined by either method (1) or method (3) prescribed in paragraph (a) of this section.

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

B. DESIGN REQUIREMENTS

This chapter addresses general requirements for the design of *cold-formed steel structural members,* assemblies, and systems applicable to the whole *Specification*.

The chapter is organized as follows:

B1 General Provisions
B2 Loads and Load Combinations
B3 Design Basis
B4 Dimensional Limits and Considerations
B5 Member Properties
B6 Fabrication and Erection (reserved)
B7 Quality Control and Quality Assurance
B8 Evaluation of Existing Structures (reserved)

B1 General Provisions

The design of *structural members* and *connections* shall be consistent with the intended behavior of *cold-formed steel* structures and the assumptions made in the *structural analysis*.

B2 Loads and Load Combinations

Loads and *load* combinations shall be as stipulated by the *applicable building code*.

Where no building code is stipulated, the *loads*, *load* combinations, and *nominal loads* [*specified loads*] shall be those stipulated as follows:

(a) In the United States and Mexico, ASCE/SEI 7, *Minimum Design Loads for Buildings and Other Structures*; and
(b) In Canada, *National Building Code of Canada*.

B3 Design Basis

No applicable strength or *serviceability limit state* shall be exceeded when the structure is subjected to the applicable *load* combinations.

Design shall be in accordance with the following methods:

(a) *ASD*, *LRFD,* or a combination of *ASD* and *LRFD*—the United States and Mexico; and
(b) *LSD*—Canada.

B3.1 Required Strength [Effect Due to Factored Loads]

The *required strength* [effect due to *factored loads*] of *structural members* and *connections* shall be determined by *structural analysis* for the appropriate *load* combinations as stipulated in Section B2.

The *required strength* [effect due to *factored loads*] shall be noted as follows:

$\overline{R}$ = *Required strength* [effect due to *factored loads*]
= R in accordance with *ASD load combinations*
= R_u in accordance with *LRFD load combinations*
= R_f in accordance with *LSD load combinations*

B3.2 Design for Strength

Structural members and their *connections* shall be designed to have strength such that the *available strength* [*factored resistance*], R_a, equals or exceeds the *required strength* [effect due to *factored loads*], $\overline{R}$.

Design for strength shall be in accordance with:

(a) Section B3.2.1 for the *Allowable Strength Design* (*ASD*),

(b) Section B3.2.2 for the *Load and Resistance Factor Design* (*LRFD*), or

(c) Section B3.2.3 for the *Limit States Design* (*LSD*).

B3.2.1 Allowable Strength Design (ASD) Requirements

The design shall be performed in accordance with Eqs. B3.2.1-1 and B3.2.1-2:

$$R \le R_a \quad (Eq.\ B3.2.1\text{-}1)$$

$$R_a = R_n/\Omega \quad (Eq.\ B3.2.1\text{-}2)$$

where

R = *Required strength*

R_a = *Allowable strength*

R_n = *Nominal strength* specified in Chapters C through K, and M

Ω = *Safety factor* specified in Chapters C through K, and M

All provisions of this *Specification* shall apply, except for those provisions that are designated specifically for *LRFD* or *LSD*.

B3.2.2 Load and Resistance Factor Design (LRFD) Requirements

The design shall be performed in accordance with Eqs. B3.2.2-1 and B3.2.2-2:

$$R_u \le R_a \quad (Eq.\ B3.2.2\text{-}1)$$

$$R_a = \phi R_n \quad (Eq.\ B3.2.2\text{-}2)$$

where

R_u = *Required strength*

R_a = *Design strength*

ϕ = *Resistance factor* specified in Chapters C through K, and M

R_n = *Nominal strength* specified in Chapters C through K, and M

All provisions of this *Specification* shall apply, except for those provisions that are designated specifically for *ASD* or *LSD*.

B3.2.3 Limit States Design (LSD) Requirements

The design shall be performed in accordance with Eqs. B3.2.3-1 and B3.2.3-2:

$$R_a \ge R_f \quad (Eq.\ B3.2.3\text{-}1)$$

$$R_a = \phi R_n \quad (Eq.\ B3.2.3\text{-}2)$$

where

R_a = *Factored resistance*

R_f = Effect of *factored loads*

ϕ = *Resistance factor* specified in Chapters C through K, and M

R_n = *Nominal resistance* specified in Chapters C through K, and M

All provisions of this *Specification* shall apply, except for those provisions that are designated specifically for *ASD* or *LRFD*.

B3.3 Design for Structural Members

The *available strength* [*factored resistance*] of *cold-formed steel structural members* that meet the geometric and material limitations provided in Section B4 shall be determined in accordance with Chapters D, E, F, G, and H, as applicable, with the *safety* and *resistance factors* provided in the corresponding sections. *Cold-formed steel structural members* outside the limitations provided in Section B4 are permitted to be designed in accordance with Section A1.2.

B3.4 Design for Connections

Connection elements shall be designed in accordance with the provisions of Chapter J. The forces and deformations used in design shall be consistent with the intended performance of the *connection* and the assumptions used in *structural analysis*. Self-limiting inelastic deformations of the *connections* are permitted. At the points of support, beams and trusses shall be restrained against rotation about their longitudinal axis unless other means of restraints against rotation are provided.

B3.4.1 Design for Anchorage to Concrete

Cold-formed steel to concrete anchorage shall be designed according to the *applicable building code*. For cast-in-place or post-installed anchors, *connection* strength controlled by cold-formed steel members or connector components shall be designed in accordance with the provisions of Section J3.

B3.5 Design for Stability

Stability of a structural system and its members shall be determined in accordance with Chapter C.

B3.6 Design of Structural Assemblies and Systems

Cold-formed steel assemblies and systems including *diaphragms* and collectors shall be designed for *load effects* that result from *loads* as stipulated in Section B2. Structural assemblies and systems shall be designed in accordance with the provisions of Chapter I, and in accordance with the provisions of Chapters C through H, and J through M, as applicable.

B3.7 Design for Serviceability

A structure shall be designed to perform its required functions during its expected life. *Serviceability limit states* shall be chosen based on the intended function of the structure and shall be evaluated using realistic *loads* and *load* combinations. The serviceability determination shall be in accordance with Chapter L.

B3.8 Design for Ponding

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

B3.9 Design for Fatigue

Fatigue shall be considered in accordance with Chapter M for *cold-formed steel structural members* and their *connections* subject to repeated loading within the elastic range of *stresses* of frequency and magnitude sufficient to initiate cracking and progressive failure. *Fatigue* need not be considered for seismic effects or for the effects of wind loading on typical building lateral force-resisting systems and building enclosure components.

B3.10 Design for Corrosion Effects

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

B4 Dimensional Limits and Considerations

Either the *Effective Width Method* or the *Direct Strength Method* shall be equally acceptable. When the *Effective Width Method* or the *Direct Strength Method* presented in Chapters E through H is used, the limitations detailed in Section B4.1 shall be met in order to use the *safety* and *resistance factors* provided in Chapters E through H. Members that do not meet the limits of B4.1 shall follow Section B4.2 for determination of the *safety factor*, Ω, or *resistance factor*, ϕ.

B4.1 Limitations for Use of the Effective Width Method or the Direct Strength Method

Members designed in accordance with the *Effective Width Method* or the *Direct Strength Method* and employing the *safety factor*, Ω, or *resistance factor*, ϕ, of Chapters E, F, and H shall fall within the dimensional limitations of Table B4.1-1.

The *webs* of flexural members designed for shear in accordance with Chapter G shall comply with the following h/t limits:

(a) For unreinforced *webs*: $h/t \leq 200$

(b) For *webs* with bearing stiffeners conforming to Section G7.1: $h/t \leq 260$

(c) For *webs* with bearing stiffeners conforming to Section G7.1 and transverse *web* stiffeners conforming to Section G4.1: $h/t \leq 300$

where

h = Depth of flat portion of *web* measured along plane of *web*

t = *Thickness* of individual *web*

The *webs* of flexural members designed for *web crippling* in accordance with Section G5 shall comply with the dimensional limitations in that section.

Table B4.1-1
Limits of Applicability for Member Design by
the Effective Width Method and the Direct Strength Method

Criteria	Limiting Variables[a]	Effective Width Method[b]	Direct Strength Method[c]
Stiffened element in compression	w/t[d]	≤ 500	≤ 500
Edge-stiffened element in compression	b/t	≤ 90 for $I_s \geq I_a$ ≤ 60 for $I_s < I_a$	≤ 160
Unstiffened element in compression	d/t[d]	≤ 60	≤ 60
Stiffened element in bending (e.g. a *web*)	h/t	≤ 300	≤ 300
Inside bend radius	R/t	≤ 10 [e]	≤ 20
Simple edge stiffener length/width ratio	d_o/b_o	≤ 0.7	≤ 0.7
Edge stiffener type		Simple lip only	Simple and complex
Maximum number of intermediate stiffeners in w	n_f	4	4
Maximum number of intermediate stiffeners in b	n_{fe}	2	2
Maximum number of intermediate stiffeners in h	n_w	0	4
Nominal *yield stress*	F_y	≤ 80 ksi (552 MPa) [f]	≤ 95 ksi (655 MPa) [f]

Note:

[a] Variable definitions:

w = *Flat width* of stiffened compression element (disregard intermediate stiffeners)
t = *Thickness* of element
b = *Flat width* of element with edge stiffeners (disregard intermediate stiffeners)
b_o = Out-to-out width of element with edge stiffeners (disregard intermediate stiffeners)
d = *Flat width* of unstiffened element (disregard intermediate stiffeners)
d_o = Out-to-out width of unstiffened element (disregard intermediate stiffeners)
h = Depth of flat portion of *web* measured along plane of *web* (disregard intermediate stiffeners)
I_a = Adequate moment of inertia of edge stiffener as used in Section 1.3
I_s = Moment of inertia of edge stiffener as used in Section 1.3
R = Inside bend radius
n_f = Number of intermediate stiffeners in stiffened compression element
n_{fe} = Number of intermediate stiffeners in edge-stiffened element
n_w = Number of intermediate stiffeners in stiffened element under *stress* gradient (e.g., *web*)
F_y = Nominal *yield stress*

[b] Applies to *local buckling* provisions in Sections E3.1, F3.1, and Appendix 1.

[c] Applies to *local* and *distortional buckling* provisions in Sections E3.2, E4, F3.2, and F4.

[d] Stiffened compression elements with $w/t > 250$ and unstiffened compression elements with $d/t > 30$ are likely to have noticeable deformations prior to developing their full strength.

[e] For inside bend R/t ratios larger than 10, *rational engineering analysis* is permitted.

[f] See Section A3 for additional limitations.

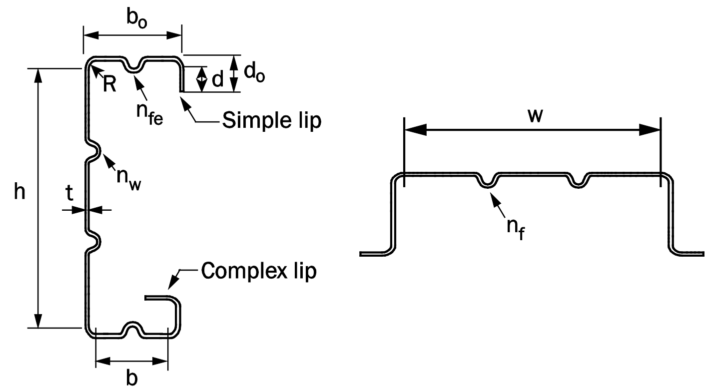


Figure B4.1-1 Illustration of Variables in Table B4.1-1
(Note: The figures are only illustrations of many possible shapes.)

B4.2 Members Falling Outside the Applicability Limits

Members that fall outside of the geometric and material limitations given in Section B4.1 shall be subjected to the provisions of Section A1.2, with the exception that members are permitted to be designed using the *Direct Strength Method* provided the *safety factor*, Ω, and *resistance factor*, ϕ, are determined using (a) or (b), as follows:

(a) Use the *safety factor*, Ω, or *resistance factor*, ϕ, determined by the *rational engineering analysis* clause of Section A1.2.6(c).

(b) Use the existing *safety factor*, Ω, or *resistance factor*, ϕ, in Chapters E through H if in an analysis of test data using Section K2, the predicted *resistance factor*, ϕ, from Section K2 provides an equal or higher ϕ than that used in Chapters E through H.

In the provisions of Section K2, the professional factor, P, shall be the test-to-predicted ratio, where the prediction is that of the *Direct Strength Method*; P_m is the mean of P; and V_P is the coefficient of variation of P. If V_P is less than or equal to 15 percent, C_P is permitted to be set to 1.0. At least three tests shall be conducted.

B4.3 Shear Lag Effects—Short Spans Supporting Concentrated Loads

Where the beam has a span of less than $30w_f$ (w_f as defined below) and carries one concentrated *load*, or several *loads* spaced farther apart than $2w_f$, the *effective design width* of any *flange*, whether in tension or compression, shall be limited by the values in Table B4.3-1.

Table B4.3-1
Short Span, Wide Flanges – Maximum Allowable Ratio of Effective Design Width (b) to Actual Width (w)

L/w_f	Ratio b/w	L/w_f	Ratio b/w
30	1.00	14	0.82
25	0.96	12	0.78
20	0.91	10	0.73
18	0.89	8	0.67
16	0.86	6	0.55

where
L = Full span for simple beams, or the distance between inflection points for continuous beams, or twice the length for cantilever beams
w_f = Width of *flange* projection beyond *web* for I-beam and similar sections, or half the distance between *webs* for box- or U-type sections

For *flanges* of I-beams and similar sections stiffened by lips at the outer edges, w_f shall be taken as the sum of the *flange* projection beyond the *web* plus the depth of the lip.

B5 Member Properties

Properties of cross-sections (*cross-sectional area*, moment of inertia, section modulus, radius of gyration, etc.) shall be determined in accordance with conventional methods of structural design.

Properties used in determining member strengths shall be based on the full cross-section of the members (or net sections where the use of net section is applicable) except where the use of a reduced cross-section, or *effective design width* determined in accordance with Appendix 1, is required.

The section properties used in design for serviceability shall be determined in accordance with Chapter L.

B6 Fabrication and Erection

(Reserved)

B7 Quality Control and Quality Assurance

B7.1 Delivered Minimum Thickness

The uncoated minimum steel *thickness* of the cold-formed steel product as delivered to the job site shall not at any location be less than 95 percent of the *thickness*, t, used in its design; however, lesser *thickness* is permitted at bends, such as corners, due to cold-forming effects.

B8 Evaluation of Existing Structures

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

B9 Design for Fire Conditions

Design for fire conditions shall comply with the fire-protection requirements in the *applicable building code*. As permitted by the *Authority Having Jurisdiction* (AHJ), performance of cold-formed steel systems under *elevated temperatures* due to fire conditions are permitted to be determined in accordance with Appendix 4.

User Note:
Appendix 4 contains provisions on the qualification of cold-formed steel assemblies by (a) testing, (b) analysis, and (c) combined testing and analysis.

C. DESIGN FOR STABILITY

This chapter addresses requirements for the design of structures for *stability*.

The chapter is organized as follows:

C1 Design for System Stability
C2 Member Bracing

C1 Design for System Stability

This chapter addresses requirements for the elastic design of structures for *stability*. System *stability* shall be provided for the structure as a whole and for each of its elements. The effects of all of the following on the *stability* of the structure and its elements shall be considered:

(a) Flexural, shear, and axial member deformations, and all other component and *connection* deformations that contribute to displacements of the structure;
(b) *Second-order effects* (including P-Δ and P-δ *effects*);
(c) Geometric imperfections;
(d) *Stiffness* reductions due to inelasticity, including the effect of residual *stresses* and partial *yielding* of the cross-section;
(e) *Stiffness* reductions due to cross-section deformations or *local* and *distortional buckling*;
(f) Uncertainty in system, member, and *connection stiffness* and strength.

All *load*-dependent effects shall be calculated at a level of loading corresponding to *LRFD load combinations, LSD 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, including the methods identified in Section C1.1, C1.2, or C1.3 within the limitations stated therein.

C1.1 Direct Analysis Method Using Rigorous Second-Order Elastic Analysis

The *direct analysis method* of design, which consists of the calculation of *required strengths* [effects due to *factored loads*] in accordance with Section C1.1.1 and the calculation of *available strengths* [*factored resistance*] in accordance with Section C1.1.2, is permitted for all systems.

C1.1.1 Determination of Required Strengths

For the *direct analysis method* of design, the *required strengths* [effects due to *factored loads*] of components of the structure shall be determined from an analysis conforming to Section C1.1.1.1. The analysis shall include consideration of initial imperfections in accordance with Section C1.1.1.2 and adjustments to *stiffness* in accordance with Section C1.1.1.3.

C1.1.1.1 Analysis

It is permitted to use any elastic analysis method capable of explicit consideration of the P-Δ and P-δ *effects* by capturing the effects of system and member displacements, respectively, on member forces.

Alternatively, it is permitted to use any elastic analysis method capable of explicit consideration of the P-Δ *effects* by capturing the effects of system displacements on member forces. The *required flexural strength* [effect due to *factored loads*], $\overline{M}$, shall then be taken as the moment resulting from such an analysis amplified by B_1, where B_1 is determined in accordance with Section C1.2.1.1.

C1.1.1.2 Consideration of Initial Imperfections

Initial imperfections at the points of member intersection shall be considered as provided by either (a) or (b) below. Additionally, it is permitted, but not required, to consider imperfections in the initial position of points along members.

(a) Direct Geometric Consideration of Initial Imperfections:

In all cases, it is permitted 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.

In the analysis of structures that support gravity *loads* primarily through nominally vertical columns, walls, or frames, where the ratio of maximum *second-order* elastic *analysis* story drift to maximum *first-order* elastic *analysis* story drift (both determined for *LRFD* or *LSD load combinations* or 1.6 times *ASD load combinations*, with *stiffnesses* as specified in Section C1.1.1.3) 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*.

(b) Consideration of Initial Imperfections Through Application of Notional Loads:

For structures that support gravity *loads* primarily through nominally vertical *columns*, walls, or frames, it is permitted 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.

(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 (3), below. The magnitude of the *notional loads* shall be:

$$N_i = (1/240)\alpha Y_i \qquad (Eq.\ C1.1.1.2\text{-}1)$$

where

α = 1.0 (*LRFD* or *LSD*)
= 1.6 (*ASD*)

N_i = *Notional load* applied at level i

Y_i = Gravity *load* applied at level i from *LRFD*, *LSD,* or *ASD load combinations*, as applicable

Where the applicable project or other quality assurance criteria stipulate a more stringent imperfection criteria, (1/240) in the above equation is permitted to be replaced by a lesser value.

(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.

(3) For structures in which the ratio of maximum *second-order* elastic *analysis* story drift to maximum *first-order* elastic *analysis* story drift (both determined for *LRFD load*

combinations or *LSD load* combinations, or 1.6 times *ASD load combinations*, with *stiffnesses* adjusted as specified in Section C1.1.1.3) in all stories is equal to or less than 1.7, it is permitted to apply the *notional load*, N_i, only in gravity-only *load* combinations and not in combinations that include other lateral *loads*.

C1.1.1.3 Modification of Section Stiffness

The analysis of the structure to determine the *required strengths* [effects due to *factored loads*] of components shall use reduced *stiffnesses*, as follows:

(a) A factor of 0.90 shall be applied to all *stiffnesses* considered to contribute to the *stability* of the structure. Additionally, it is permitted, but not required, to also apply the *stiffness* reduction to those members that are not part of the lateral force resisting system.

(b) An additional factor, τ_b, shall be applied to the flexural *stiffnesses* of all members whose flexural *stiffnesses* are considered to contribute to the *stability* of the structure.

For $\alpha\overline{P}/P_y \le 0.5$,

$$\tau_b = 1.0 \qquad (Eq.\ C1.1.1.3\text{-}1)$$

For $\alpha\overline{P}/P_y > 0.5$,

$$\tau_b = 4(\alpha\overline{P}/P_y)[1-(\alpha\overline{P}/P_y)] \qquad (Eq.\ C1.1.1.3\text{-}2)$$

where

α = 1.0 (*LRFD* or *LSD*)
= 1.6 (*ASD*)

$\overline{P}$ = *Required* axial compressive *strength* [compressive force due to *factored loads*] using *LRFD*, *LSD,* or *ASD load combinations*

P_y = Axial yield strength
= F_yA_g (*Eq.* C1.1.1.3-3)

where

F_y = *Yield stress*

A_g = *Gross area* of cross-section

(c) In lieu of using $\tau_b < 1.0$ where $\alpha\overline{P}/P_y > 0.5$, it is permitted to use $\tau_b = 1.0$ for all members if a *notional load* of $(1/1000)\alpha Y_i$ is applied at all levels, in the direction specified in Section C1.1.1.2, in all *load* combinations. These *notional loads* shall be added to those stipulated in Section C1.1.1.2, except that C1.1.1.2(3) shall not apply.

(d) Where components comprised of materials other than cold-formed steel are considered to contribute to the *stability* of the structure, *stiffness* reductions shall be applied to those components as required by the codes and specifications governing their design.

C1.1.2 Determination of Available Strengths [Factored Resistances]

The *available strengths* [*factored resistances*] 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 *flexural buckling effective length factors*, K_y and K_x, of all members shall be taken as unity unless a smaller value can be justified by *rational engineering analysis*.

Bracing intended to define the *unbraced lengths* of members shall have enough *stiffness* and strength to control member movement at the braced points, and shall be designed in accordance with Section C2.

When initial imperfections in the position of points along a member are considered in the analysis in addition to imperfections at the points of intersection as stipulated in Section C1.1.1.2, it is permissible to take the *flexural buckling* strength of the member in the plane of the initial imperfection as the cross-section strength. The *available strengths* [*factored resistances*] due to *torsional*, *flexural-torsional*, *local,* and *distortional buckling* of compression members shall be as specified in Chapter E.

C1.2 Direct Analysis Method Using Amplified First-Order Elastic Analysis

The *direct analysis method* of design, which consists of the calculation of *required strengths* [effects due to *factored loads*] in accordance with Section C1.2.1 and the calculation of *available strengths* [*factored resistance*] in accordance with Section C1.2.2, shall be limited to structures that support *gravity loads* primarily through nominally vertical columns, walls, or frames.

C1.2.1 Determination of Required Strengths [Effects due to Factored Loads]

For the *direct analysis method* of design, the *required strengths* [effects due to *factored loads*] of components of the structure shall be determined from an analysis conforming to Section C1.2.1.1. The analysis shall include consideration of initial imperfections in accordance with Section C1.2.1.2 and adjustments to *stiffness* in accordance with Section C1.2.1.3.

C1.2.1.1 Analysis

The *required flexural strength* [moment due to *factored loads*], $\overline{M}$, and *required axial strength* [axial force due to *factored loads*], $\overline{P}$, of all members shall be determined as follows:

$$\overline{M} = B_1\overline{M}_{nt} + B_2\overline{M}_{\ell t} \quad (Eq.\ C1.2.1.1\text{-}1)$$

$$\overline{P} = \overline{P}_{nt} + B_2\overline{P}_{\ell t} \quad (Eq.\ C1.2.1.1\text{-}2)$$

where

B_1 = Multiplier to account for *P-δ effects*, determined for each member subject to compression and flexure, and each direction of bending of the member in accordance with Eq. C1.2.1.1-3, with B_1 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 using Eq. C1.2.1.1-6

$\overline{M}_{\ell t}$ = Moment from *first-order* elastic *analysis* using *LRFD*, *LSD,* or *ASD load combinations*, as applicable, due to lateral translation of the structure only

$\overline{M}_{nt}$ = Moment from *first-order* elastic *analysis* using *LRFD*, *LSD,* or *ASD load combinations*, as applicable, with the structure restrained against lateral translation

$\overline{M}$ = *Required* second-order flexural *strength* [moment due to *factored loads*] using

LRFD, *LSD* or *ASD load combinations*, as applicable

$\overline{P}_{\ell t}$ = Axial force from *first-order* elastic *analysis* using *LRFD*, *LSD* or *ASD load combinations*, as applicable, due to lateral translation of the structure only

$\overline{P}_{nt}$ = Axial force from *first-order* elastic *analysis* using *LRFD*, *LSD* or *ASD load combinations*, as applicable, with the structure restrained against lateral translation

$\overline{P}$ = *Required* second-order axial *strength* [compressive force due to *factored loads*] using *LRFD*, *LSD* or *ASD load combinations*, as applicable

The *P-δ effect* amplifier B_1 shall be determined in accordance with Eq. C1.2.1.1-3, in which $\overline{P}$ shall be determined by iteration or is permitted to be taken as $\overline{P}_{nt} + \overline{P}_{\ell t}$.

$$B_1 = C_m/(1 - \alpha\overline{P}/P_{e1}) \geq 1.0 \qquad (Eq.\ C1.2.1.1\text{-}3)$$

where

α = 1.00 (*LRFD* or *LSD*)

= 1.60 (*ASD*)

C_m = 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

$$C_m = 0.6 - 0.4(M_1/M_2) \qquad (Eq.\ C1.2.1.1\text{-}4)$$

where

M_1 and M_2 = Smaller and larger moments, respectively, at the ends of that portion of the member unbraced in the plane of bending under consideration. M_1 and M_2 are calculated from a *first-order* elastic *analysis*. 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, 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 member ends

$$= \pi^2 k_f/(K_1 L)^2 \qquad (Eq.\ C1.2.1.1\text{-}5)$$

where

k_f = Flexural *stiffness* in the plane of bending as modified in Section C1.2.1.3

L = Unbraced length of member

K_1 = *Effective length factor* for *flexural buckling* in the plane of bending, K_y or K_x, as applicable, calculated based on the assumption of no lateral translation at member ends

= 1.0 unless analysis justifies a smaller value

The *P-Δ effect* amplifier B_2 for each story and each direction of lateral translation shall be calculated as follows:

$$B_2 = 1/[1 - (\alpha\overline{P}_{story})/P_{e,story})] \geq 1.0 \qquad (Eq.\ C1.2.1.1\text{-}6)$$

where

$\overline{P}_{story}$ = Total vertical *load* supported by the story using *LRFD*, *LSD,* or *ASD load* *combinations*, as applicable, including *loads* in columns that are not part of the lateral force-resisting system

$P_{e,story}$ = Elastic critical *buckling* strength for the story in the direction of translation being considered, determined by sidesway *buckling* analysis or taken as:

$$P_{e,story} = R_M H \overline{F} / \Delta_F \qquad (Eq.\ C1.2.1.1\text{-}7)$$

where

$$R_M = 1.0 - 0.15(P_{mf} / \overline{P}_{story}) \qquad (Eq.\ C1.2.1.1\text{-}8)$$

where

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

H = Height of story

Δ_F = Inter-story drift from *first-order* elastic *analysis* in the direction of translation being considered, due to story shear, $\overline{F}$, computed using the *stiffness* as required by Section C1.2.1.3

$\overline{F}$ = Story shear, in the direction of translation being considered, produced by the lateral forces using *LRFD*, *LSD*, or 1.6 times *ASD load combinations*

Where Δ_F varies over the plan area of the structure in a three-dimensional system with rigid *diaphragms*, it shall be the average drift weighted in proportion to vertical *load* or, alternatively, the maximum drift in the story. In two-dimensional systems with flexible and semi-rigid *diaphragms*, Δ_F shall be evaluated at each independent frame (i.e., line of resistance), or alternatively taken as the maximum drift in the story.

C1.2.1.2 Consideration of Initial Imperfections

Initial imperfections shall be considered as provided by Sections C1.1.1.2(a) or C1.1.1.2(b).

C1.2.1.3 Modification of Section Stiffness

Section stiffness modifications shall be made as required by Section C1.1.1.3.

C1.2.2 Determination of Available Strengths [Factored Resistances]

The *available strengths* [*factored resistances*] of members and *connections* shall be calculated as provided by Section C1.1.2.

C1.3 Effective Length Method

The use of the *effective length method* shall be limited to the following conditions:

(a) The structure supports gravity *loads* primarily through nominally vertical columns, walls, or frames.

(b) The ratio of maximum second-order drift to maximum first-order drift (both determined for *LRFD load combinations, LSD load combinations,* or 1.6 times *ASD load combinations*) in all stories is equal to or less than 1.5, as determined based on nominal unreduced *stiffness*.

C1.3.1 Determination of Required Strengths [Effects of Factored Loads]

For the design, the *required strengths* [effects due to *factored loads*] of components of the structure shall be determined from an analysis conforming to Section C1.3.1.1. The analysis shall include consideration of initial imperfections in accordance with Section C1.3.1.2.

C1.3.1.1 Analysis

The analysis shall be performed in accordance with the requirements of Section C1.2.1.1, except that nominal *stiffnesses* shall be used in the analysis and Section C1.2.1.3 shall not apply.

C1.3.1.2 Consideration of Initial Imperfections

Notional loads shall be applied in the analysis as required by Section C1.1.1.2(b).

C1.3.2 Determination of Available Strengths [Factored Resistances]

The *available strengths* [*factored resistances*] 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 *flexural buckling effective length factors*, K_x and K_y, 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* do not rely on the flexural *stiffness* of columns, K_x and K_y of members subject to compression shall be taken as 1.0, unless *rational engineering 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*, K_x and K_y, or elastic critical *buckling stress*, F_{cre}, 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_x and K_y 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 take K_x or K_y, as applicable, as 1.0 in the design of all columns if the ratio of maximum second-order drift to maximum first-order drift (both determined for *LRFD or LSD load combinations* or 1.6 times *ASD load combinations*) in all stories is equal to or less than 1.1.

Bracing intended to define the *unbraced lengths* of members shall have enough *stiffness* and strength to control member movement at the braced points, and shall be designed in accordance with Section C2.

C2 Member Bracing

☜B

C2.1 Symmetrical Beams and Columns

The provisions of this section shall only apply to Canada. See Section C2.1 of Appendix B.

☜B

C2.2 Bracing of Beams

The *required brace strength* [brace force due to *factored loads*] and *stiffness* are permitted to be determined by a *second-order analysis* in accordance with the requirements of Section C1.

Alternatively, to restrain twisting of C-sections and Z-sections used as beams loaded in the plane of the *web*, the brace force provisions of Section C2.2.1 shall apply only when neither *flange* is connected to deck or sheathing material in such a manner as to effectively restrain lateral deflection of the connected *flange*. When only the top *flange* is so connected, see Section C2.2.2. Also, see Appendix B for additional requirements applicable to Canada. B

Where both *flanges* are connected to deck or sheathing materials in such a manner as to effectively restrain lateral deflection of the connected *flange*, no further bracing is required.

C2.2.1 Neither Flange Connected to Sheathing That Contributes to the Strength and Stability of the C- or Z-Section

Each intermediate brace at the top and bottom *flanges* of C- or Z-section members shall be designed with resistance of $\overline{P}_{L1}$ and $\overline{P}_{L2}$, where $\overline{P}_{L1}$ is the brace force required on the *flange* in the quadrant with both x and y axes positive, and $\overline{P}_{L2}$ is the brace force on the other *flange*. The x-axis shall be designated as the centroidal axis perpendicular to the *web*, and the y-axis shall be designated as the centroidal axis parallel to the *web*. The x and y coordinates shall be oriented such that one of the *flanges* is located in the quadrant with both positive x and y axes. See Figure C2.2.1-1 for illustrations of coordinate systems and positive force directions.

(a) For uniform loads

$$\overline{P}_{L1} = 1.5[\overline{W}_y K' - (\overline{W}_x / 2) + (\overline{M}_z / d)] \quad (Eq.\ C2.2.1\text{-}1)$$

$$\overline{P}_{L2} = 1.5[\overline{W}_y K' - (\overline{W}_x / 2) - (\overline{M}_z / d)] \quad (Eq.\ C2.2.1\text{-}2)$$

When the uniform *load*, $\overline{W}$, acts through the plane of the *web*, i.e., $\overline{W}_y = \overline{W}$ and $\overline{W}_x = 0$:

$$\overline{P}_{L1} = -\overline{P}_{L2} = 1.5(m/d)\overline{W} \quad \text{for C-sections} \quad (Eq.\ C2.2.1\text{-}3)$$

$$\overline{P}_{L1} = \overline{P}_{L2} = 1.5\left(\frac{I_{xy}}{2I_x}\right)\overline{W} \quad \text{for Z-sections} \quad (Eq.\ C2.2.1\text{-}4)$$

where

$\overline{W}_x$, $\overline{W}_y$ = Components of *design load* [*factored load*] $\overline{W}$ parallel to the x- and y-axis, respectively. $\overline{W}_x$ and $\overline{W}_y$ are positive if pointing to the positive x- and y-direction, respectively

where

$\overline{W}$ = *Design load* [*factored load*] (applied *load* determined in accordance with the most critical *ASD*, *LRFD*, or *LSD load combinations*, depending on the design method used) within a distance of 0.5a on each side of the brace

where

a = Longitudinal distance between centerline of braces

K′ = 0 for C-sections

= $I_{xy}/(2I_x)$ for Z-sections (*Eq.* C2.2.1-5)

where

I_{xy} = Product of inertia of full unreduced section about centroidal axes parallel and perpendicular to the *purlin web*

I_x = Moment of inertia of full unreduced section about x-axis

$\overline{M}_z$ = $-\overline{W}_x e_{sy} + \overline{W}_y e_{sx}$, torsional moment of $\overline{W}$ about shear center

where

e_{sx}, e_{sy} = Eccentricities of *load* components measured from the shear center and in the x- and y-directions, respectively

d = Depth of section

m = Distance from shear center to mid-plane of *web* of C-section

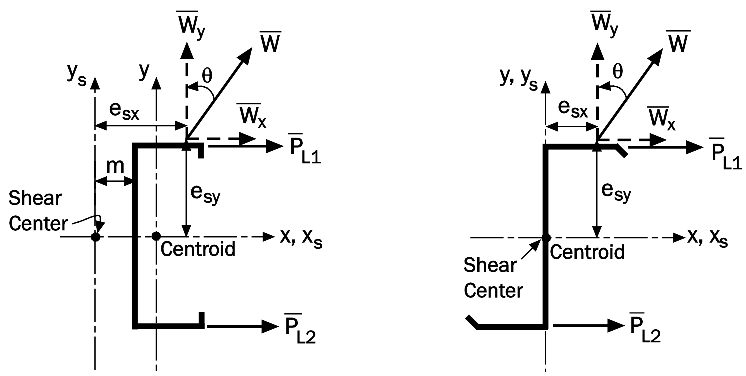


Figure C2.2.1-1 Coordinate Systems and Positive Force Directions

(b) For concentrated loads,

$$\overline{P}_{L1} = \overline{P}_y K' - (\overline{P}_x / 2) + (\overline{M}_z / d) \quad (Eq.\ C2.2.1\text{-}6)$$

$$\overline{P}_{L2} = \overline{P}_y K' - (\overline{P}_x / 2) - (\overline{M}_z / d) \quad (Eq.\ C2.2.1\text{-}7)$$

When a *design load* [*factored load*] acts through the plane of the *web*, i.e., $\overline{P}_y = \overline{P}$ and $\overline{P}_x = 0$:

$$\overline{P}_{L1} = -\overline{P}_{L2} = (m/d)\overline{P} \quad \text{for C-sections} \quad (Eq.\ C2.2.1\text{-}8)$$

$$\overline{P}_{L1} = \overline{P}_{L2} = \left(\frac{I_{xy}}{2I_x}\right)\overline{P} \quad \text{for Z-sections} \quad (Eq.\ C2.2.1\text{-}9)$$

where

$\overline{P}_x, \overline{P}_y$ = Components of *design load* [*factored load*] $\overline{P}$ parallel to the x- and y-axis, respectively. $\overline{P}_x$ and $\overline{P}_y$ are positive if pointing to the positive x- and y-direction, respectively.

$\overline{M}_z$ = $-\overline{P}_x e_{sy} + \overline{P}_y e_{sx}$, torsional moment of $\overline{P}$ about shear center

$\overline{P}$ = *Design* concentrated *load* [*factored load*] within a distance of 0.3a on each side of the brace, plus 1.4(1-*l*/a) times each *design* concentrated *load* [*factored load*] located farther than 0.3a but not farther than 1.0a from the brace. The *design* concentrated *load* [*factored load*] is the applied *load* determined in accordance with the most critical *ASD*, *LRFD*, or *LSD load combinations*, depending on the design method used

where

l = Distance from concentrated *load* to the brace

See Section C2.2.1(a) for definitions of other variables.

The bracing force, $\overline{P}_{L1}$ or $\overline{P}_{L2}$, is positive where restraint is required to prevent the movement of the corresponding *flange* in the negative x-direction.

Where braces are provided, they shall be attached in such a manner as to effectively restrain the section against lateral deflection of both *flanges* at the ends and at any intermediate brace points.

When all *loads* and reactions on a beam are transmitted through members that frame into the section in such a manner as to effectively restrain the section against torsional rotation and lateral displacement, no additional braces shall be required except those required for strength in accordance with Section F3.

C2.2.2 Flange Connected to Sheathing That Contributes to the Strength and Stability of the C- or Z-Section

For purlin roof systems with sheathing attached to the top *flange*, the bracing and the anchorage shall be provided in accordance with Section I6.4.1 for systems with lateral restraints, or Section I6.4.2 for systems with torsional bracing in combination with lateral bracing.

C2.3 Bracing of Axially Loaded Compression Members

Intermediate bracing of compression members shall be designed to restrain member translation and/or twist. The *required brace strength* [brace force or moment due to *factored loads*] and required brace *stiffness* are permitted to be determined by a *second-order analysis* in accordance with the requirements of Section C1.

Alternatively, Section C2.3.1 is permitted to be used for translational bracing of an individual concentrically loaded compression member and Section C2.3.2 is permitted to be used for translational bracing of multiple parallel concentrically loaded compression members.

C2.3.1 Translational Bracing of an Individual Concentrically Loaded Compression Member

To provide an adequate intermediate translation brace (or braces) that will allow an individual concentrically loaded compression member to develop its *required axial strength* [compressive axial force due to *factored loads*] for *flexural buckling*, the *required strength* [brace force due to *factored loads*] acting on the brace (or braces) shall be calculated in accordance with Eq. C2.3.1-1.

$$\overline{P}_{rb} = 0.01\ \overline{P}_{ra} \qquad (Eq.\ C2.3.1\text{-}1)$$

where

$\overline{P}_{rb}$ = *Required brace strength* [brace force due to *factored loads*] to brace a single compression member with an axial *load* $\overline{P}_{ra}$

$\overline{P}_{ra}$ = *Required compressive axial strength* [compressive axial force due to *factored loads*] of an individual concentrically loaded compression member to be braced, which is calculated in accordance with *ASD*, *LRFD*, or *LSD load combinations* depending on the design method used

The required brace *stiffness* of each brace shall equal or exceed β_{rb}, as calculated in Eq. C2.3.1-2:

For *ASD*

$$\beta_{rb} = \frac{2[4-(2/n)]}{L_b}(\Omega\overline{P}_{ra}) \qquad (Eq.\ C2.3.1\text{-}2a)$$

Ω = 2.00

For *LRFD* and *LSD*

$$\beta_{rb} = \frac{2[4-(2/n)]}{L_b}\left(\frac{\overline{P}_{ra}}{\phi}\right) \qquad (Eq.\ C2.3.1\text{-}2b)$$

ϕ = 0.75 for *LRFD*
= 0.70 for *LSD*

where

β_{rb} = Minimum required brace *stiffness* to brace a single compression member

n = Number of equally spaced intermediate brace locations

L_b = Longitudinal distance between brace points on the individual concentrically loaded compression member to be braced

For braces not oriented perpendicular to the braced member, the *required brace strength* [brace force due to *factored loads*] and *stiffness* shall be adjusted for the angle of approach.

C2.3.2 Translational Bracing of Multiple Parallel Concentrically Loaded Compression Members

To provide an adequate intermediate brace (or braces) that will allow multiple parallel concentrically loaded compression members to develop their *required axial strength* [compressive axial force due to *factored loads*], the *required strength* [brace force due to *factored loads*] acting on the brace shall be calculated in accordance with Eq. C2.3.2-1.

$$\overline{P}_{rb} = \frac{0.5}{j}\left(1+\frac{1}{\sqrt{m}}\right)\sum_{i=1}^{m}\overline{P}_{rb,i} \qquad (Eq.\ C2.3.2\text{-}1)$$

where

$\overline{P}_{rb}$ = *Required brace strength* [brace force due to *factored loads*] to brace multiple parallel compression members

$\overline{P}_{rb,i}$ = *Required brace strength* [brace force due to *factored loads*] of the (i)th concentrically loaded compression member, which is calculated in accordance with Eq. C2.3.1-1

m = Total number of concentrically loaded compression members to be braced

j = Number of brace anchor ends (j = 1 single side or j = 2 double side). For a brace to be designed as tension only, two anchor ends shall be provided with j taken as 1.

The *stiffness* of each brace between the concentrically loaded compression members shall equal or exceed the required brace *stiffness*, β_{rb}, as calculated in Eq. C2.3.2-2:

$$\beta_{rb} = \frac{0.4(m/j)^2 + 0.4(m/j) + 0.2}{\gamma_a\gamma_c}\beta_{rb,max} \qquad (Eq.\ C2.3.2\text{-}2)$$

where

β_{rb} = Minimum required *stiffness* of each brace between the concentrically loaded compression members

$\beta_{rb,max}$ = Largest required brace *stiffness* of all concentrically loaded compression members calculated in accordance with Section C2.3.1

γ_a = Factor to account for anchor *stiffness*

= $1-(m/j)(\beta_{rb,max}/\gamma_c)/\beta_{anchor}$ (*Eq*. C2.3.2-3)

β_{anchor} = Lateral *stiffness* at the brace anchor point(s) and must exceed $(m/j)(\beta_{rb,max}/\gamma_c)$

γ_c = Factor to account for connector *stiffness*

= $1-\beta_{rb,max}/\beta_c$ for continuous bracing (*Eq*. C2.3.2-4)

= 1 for intermediate or blocking-type bracing

β_c = *Stiffness* of connector used to attach compression member to continuous bracing and must exceed $\beta_{rb,max}$; when varied the smallest value must be used in Eq. C2.3.2-4

D. MEMBERS IN TENSION

This chapter addresses members subjected to axial tension caused by static forces acting through the centroidal axes.

The chapter is organized as follows:

D1 General Requirements
D2 Yielding of Gross Section
D3 Rupture of Net Section

D1 General Requirements

For axially loaded tension members, the *available tensile strength* [*factored resistance*] ($\phi_t T_n$ or T_n/Ω_t) shall be the lesser of the values obtained in accordance with Sections D2 and D3, where the *nominal strengths* [*resistance*] and the corresponding *safety* and *resistance factors* are provided. The *available strengths* [*factored resistance*] shall be determined in accordance with the applicable design method in Section B3.2.1, B3.2.2, or B3.2.3.

The *nominal tensile strength* [*resistance*] shall also be limited by the *connection* strength of the tension members, which is determined in accordance with the provisions of Chapter J.

D2 Yielding of Gross Section

The *nominal tensile strength* [*resistance*], T_n, due to *yielding* of the gross section shall be determined as follows:

$$T_n = A_g F_y \qquad (Eq.\ D2\text{-}1)$$

Ω_t = 1.67 (*ASD*)

ϕ_t = 0.90 (*LRFD*)

= 0.90 (*LSD*)

where

A_g = *Gross area* of cross-section

F_y = Design *yield stress* as determined in accordance with Section A3.3.1

D3 Rupture of Net Section

The *nominal tensile strength* [*resistance*], T_n, due to rupture of the net section shall be determined as follows:

$$T_n = A_{net} F_u \qquad (Eq.\ D3\text{-}1)$$

Ω_t = 2.00 (*ASD*)

ϕ_t = 0.75 (*LRFD*)

= 0.75 (*LSD*)

where

A_{net} = *Net area* of cross-section

F_u = *Tensile strength* as specified in Section A3.1

E. MEMBERS IN COMPRESSION

This chapter addresses members subjected to concentric axial compression.

This chapter is organized as follows:

Additionally, built-up compression member provisions are provided in:

E1 General Requirements

The *available axial strength* [*factored resistance*] ($\phi_c P_n$ or P_n/Ω_c) shall be the smallest of the values calculated in accordance with Sections E2 to E4 where applicable.

E2 Yielding and Global (Flexural, Flexural-Torsional, and Torsional) Buckling

The *nominal axial strength* [*resistance*], P_{ne}, for *yielding*, and global (*flexural*, *torsional*, or *flexural-torsional*) *buckling* shall be calculated in accordance with this section. The applicable *safety factor* and *resistance factors* given in this section shall be used to determine the *available axial strength* [*factored resistance*] ($\phi_c P_{ne}$ or P_{ne}/Ω_c) in accordance with the applicable design method in Section B3.2.1, B3.2.2, or B3.2.3.

$$P_{ne} = A_g F_n \qquad (Eq.\ E2\text{-}1)$$

Ω_c = 1.80 (*ASD*)

ϕ_c = 0.85 (*LRFD*)

= 0.80 (*LSD*)

where

A_g = *Gross area*

F_n = Compressive *stress* and shall be calculated as follows:

For $\lambda_c \le 1.5$

$$F_n = \left(0.658^{\lambda_c^2}\right)F_y \qquad (Eq.\ E2\text{-}2)$$

For $\lambda_c > 1.5$

$$F_n = \left(\frac{0.877}{\lambda_c^2}\right)F_y \qquad (Eq.\ E2\text{-}3)$$

where

$$\lambda_c = \sqrt{\frac{F_y}{F_{cre}}} \qquad (Eq.\ E2\text{-}4)$$

where

F_{cre} = Elastic global (*flexural*, *torsional*, or *flexural-torsional) buckling stress* determined in accordance with Appendix 2

F_y = *Yield stress*

Concentrically loaded angle sections shall be designed for an additional bending moment as specified in Section H1.2.

E2.1 Reduction for Closed-Box Sections

For a closed-box section made of steel with a specified minimum elongation between three to ten percent, inclusive, the *buckling stress*, F_{cre}, used in Eq. E2-4 shall be multiplied by the reduction factor R when the value of the *effective length* KL is less than 1.1 L_0, where L_0 is given by Eq. E2.1-1, and R is given by Eq. E2.1-2.

$$L_0 = \pi r\sqrt{\frac{E}{F_{cr\ell}}} \quad (Eq.\ E2.1\text{-}1)$$

$$R = \left[0.65 + \frac{0.35(KL)}{1.1L_0}\right]^2 \quad (Eq.\ E2.1\text{-}2)$$

where

L_0 = Length at which *local buckling stress* equals *flexural buckling stress*

r = Radius of gyration of full unreduced cross-section about axis of *buckling*

E = Modulus of elasticity of steel

$F_{cr\ell}$ = Minimum critical *buckling stress* for cross-section calculated by Eq. 1.1-4

KL = *Effective length* determined in accordance with Chapter C

E3 Local Buckling Interacting With Yielding and Global Buckling

The *nominal axial strength* [*resistance*], $P_{n\ell}$, for *local buckling* interacting with *yielding* and global *buckling* shall be calculated in accordance with this section. All members shall be checked for potential reduction in *available strength* [*factored resistance*] due to interaction of the *yielding* or global *buckling* with *local buckling*. This reduction shall be considered through either the *Effective Width Method* of Section E3.1, the *Direct Strength Method* of Section E3.2, or the cylindrical tube provisions of Section E3.3.

The applicable *safety factors* and *resistance factors* given in this section shall be used to determine the *available axial strength* [*factored resistance*] ($\phi_c P_{n\ell}$ or $P_{n\ell}/\Omega_c$) in accordance with the applicable design method in Section B3.2.1, B3.2.2, or B3.2.3.

Ω_c = 1.80 (*ASD*)

ϕ_c = 0.85 (*LRFD*)

= 0.80 (*LSD*)

E3.1 Effective Width Method

For the *Effective Width Method*, the *nominal axial strength* [*resistance*], $P_{n\ell}$, for *local buckling* shall be calculated in accordance with the following:

$$P_{n\ell} = A_e F_n \le P_{ne} \quad (Eq.\ E3.1\text{-}1)$$

where

F_n = Compressive *stress* as defined in Section E2

A_e = *Effective area* calculated at *stress* F_n

P_{ne} = *Nominal strength* [*resistance*] considering *yielding* and global *buckling,* determined in accordance with Section E2

A_e shall be determined from the summation of the *thickness* times the *effective width* of each element comprising the cross-section. The *effective width* of all elements shall be determined in accordance with Appendix 1 at *stress* F_n.

For members with holes, the *effective width* of elements with holes shall be determined in accordance with Section 1.1.1(a) of Appendix 1, subject to the limitations of that section, and F_{cre} shall be multiplied by A_g/A_{net} in Section E2 to determine F_n, where A_{net} is the *net area* of cross-section. If the number of holes in the *effective length* region times the hole length divided by the *effective length* does not exceed 0.015, A_e is permitted to be determined by ignoring the holes.

E3.2 Direct Strength Method

For the *Direct Strength Method*, the *nominal axial strength* [*resistance*], $P_{n\ell}$, for *local buckling* shall be determined as follows for $\lambda_\ell \le 5$:

$$P_{n\ell} = 1.2P_{ne}\left(\frac{1+0.10\lambda_\ell^2}{1+0.55\lambda_\ell^2}\right) \le P_{ne} \qquad (Eq.\ E3.2\text{-}1)$$

where

$\lambda_\ell = \sqrt{P_{ne}/P_{cr\ell}}$ (*Eq*. E3.2-2)

P_{ne} = *Nominal global column strength* [*resistance*] as defined in Section E2

$P_{cr\ell}$ = Critical elastic *local* column *buckling load,* determined in accordance with Appendix 2, including the influence of holes if applicable

> **User Note:**
> It is rare for a section to have *local buckling* slenderness λ_ℓ greater than 5. The *Commentary* provides some rational methods to approximate the strength for high slenderness applications.

For members with holes, $P_{n\ell}$ shall be the lesser of Eq. E3.2-1 and Eq. E3.2-3. Eq. E3.2-1 shall use the lesser of $P_{cr\ell}$ at the gross or net section. Eq. E3.2-3 shall use $P_{cr\ell}$ at the net section.

$$P_{n\ell} = 1.2P_{ynet}\left[\frac{1+0.10\lambda_\ell^2}{1+0.55\lambda_\ell^2\left(\frac{P_{ynet}}{P_{ne}}\right)}\right] \le P_{ynet} \qquad (Eq.\ E3.2\text{-}3)$$

where

$P_{ynet} = A_{net}F_y$ (*Eq*. E3.2-4)

where

A_{net} = *Net area* of cross-section at the location of a hole

F_y = *Yield stress*

E3.3 Cylindrical Tubes

For closed cylindrical tubes having a ratio of outside diameter to wall *thickness*, D/t, not greater than 0.441 E/F_y, the *nominal axial strength* [*resistance*], $P_{n\ell}$, for *local buckling* is permitted to be determined in accordance with Section E3.1 where the *effective area*, A_e, shall be calculated as follows:

$$A_e = A_o + R(A - A_o) \qquad (Eq.\ E3.3\text{-}1)$$

where

$$A_o = \left[\frac{0.037}{(DF_y)/(tE)} + 0.667\right]A \le A \qquad (Eq.\ E3.3\text{-}2)$$

$$R = F_y/(2F_{cre}) \le 1.0 \qquad (Eq.\ E3.3\text{-}3)$$

where

D = Outside diameter of cylindrical tube

F_y = *Yield stress*

t = Wall *thickness*

E = Modulus of elasticity of steel

A = Area of full unreduced cross-section

F_{cre} = Elastic *flexural buckling stress*, determined in accordance with Appendix 2 Section 2.3.1.1

E4 Distortional Buckling

The *nominal axial strength* [*resistance*], P_{nd}, for *distortional buckling* shall be calculated in accordance with this section for $\lambda_d \le 5$. The provisions of this section shall apply to I-, Z-, C-, Hat, and other open cross-section members that employ *flanges* with edge stiffeners or any cross-section with intermediate stiffeners.

Exception: The provisions of this section do not apply to *distortional buckling* modes involving only intermediate stiffeners if $P_{n\ell}$ is determined using Section E3.1 and the intermediate stiffener provisions of Appendix 1 Section 1.4. *Distortional buckling* modes involving edge-stiffened *flanges* are not part of this exception.

The applicable *safety factor* and *resistance factors* given in this section shall be used to determine the *available axial strength* [*factored resistance*] ($\phi_c P_{nd}$ or P_{nd}/Ω_c) in accordance with the applicable design method in Section B3.2.1, B3.2.2, or B3.2.3.

Ω_c = 1.80 (*ASD*)

ϕ_c = 0.85 (*LRFD*)

= 0.80 (*LSD*)

$$P_{nd} = 1.2P_y\left(\frac{1+0.05\lambda_d^2}{1+0.67\lambda_d^2}\right) \le P_y \qquad (Eq.\ E4\text{-}1)$$

where

$$\lambda_d = \sqrt{P_y/P_{crd}} \qquad (Eq.\ E4\text{-}2)$$

P_{crd} = Critical elastic *distortional* column *buckling* force, determined in accordance with Appendix 2

$$P_y = A_gF_y \qquad (Eq.\ E4\text{-}3)$$

where

A_g = *Gross area*

F_y = *Yield stress*

User Note:
It is rare for a section to have *distortional buckling* slenderness λ_d greater than 5. The *Commentary* provides a rational approach to approximate the strength for high slenderness applications.

For members with holes, P_{crd} shall be determined including the influence of holes and P_{nd} shall be determined as follows:

$$P_{nd} = 1.2P_{ynet}\left[\frac{1+0.05\lambda_d^2}{1+0.67\lambda_d^2\left(\dfrac{P_{ynet}}{P_y}\right)}\right] \le P_{ynet} \qquad (Eq.\ E4\text{-}4)$$

where P_{ynet} is the net cross-section yield strength defined in Section E3.2.

F. MEMBERS IN FLEXURE

This chapter addresses members subjected to bending about one principal axis, constrained bending about only one axis (not subject to *lateral-torsional buckling*), or Z-section members about centroidal axis passing through or perpendicular to the *web*. In addition, the member is loaded in a plane that passes through the shear center, is restrained against twisting, or satisfies the requirements of Section H4 for combined bending and torsion.

This chapter is organized as follows:

F1 General Requirements
F2 Yielding and Global (Lateral-Torsional) Buckling
F3 Local Buckling Interacting With Yielding and Global Buckling
F4 Distortional Buckling

Additionally, built-up flexural member provisions are provided in:

I1.1 Flexural Members Composed of Two Back-to-Back C-Sections

F1 General Requirements

The *available flexural strength* [*factored resistance*] ($\phi_b M_n$ or M_n/Ω_b) shall be the smallest of the values calculated in accordance with Sections F2 to F4, where applicable.

F2 Yielding and Global (Lateral-Torsional) Buckling

The *nominal flexural strength* [*resistance*], M_{ne}, for *yielding* and global (*lateral-torsional*) *buckling* shall be calculated using the *Effective Width Method* in accordance with Section F2.1, the *Direct Strength Method* in accordance with Section F2.2, or the cylindrical tube provisions of Section F2.3.

The applicable *safety factor* and *resistance factors* given in this section, unless otherwise specified, shall be used to determine the *available flexural strength* [*factored resistance*] ($\phi_b M_{ne}$ or M_{ne}/Ω_b) in accordance with the applicable design method in Section B3.2.1, B3.2.2, or B3.2.3.

Ω_b = 1.67 (*ASD*)
ϕ_b = 0.90 (*LRFD*)
= 0.90 (*LSD*)

F2.1 Effective Width Method

For the *Effective Width Method*, the *nominal flexural strength* [*resistance*], M_{ne}, for *yielding* and global (*lateral-torsional*) *buckling* considering capacity up to first yield shall be calculated in accordance with Eq. F2.1-1.

$$M_{ne} = S_{fc}F_n \leq M_y \qquad (Eq.\ F2.1\text{-}1)$$

where

M_{ne} = *Nominal flexural strength* [*resistance*] for *yielding* and global *buckling*

S_{fc} = Elastic section modulus of full unreduced section relative to extreme compression fiber

$$M_y = S_fF_y \qquad (Eq.\ F2.1\text{-}2)$$

where
S_f = Elastic section modulus of full unreduced cross-section relative to extreme fiber in first *yielding*
F_y = *Yield stress*

F_n shall be determined as follows:

For $F_{cre} \geq 2.78F_y$

$$F_n = F_y \qquad (Eq.\ F2.1\text{-}3)$$

For $2.78F_y > F_{cre} > 0.56F_y$

$$F_n = \frac{10}{9}F_y\left(1 - \frac{10F_y}{36F_{cre}}\right) \qquad (Eq.\ F2.1\text{-}4)$$

For $F_{cre} \leq 0.56F_y$

$$F_n = F_{cre} \qquad (Eq.\ F2.1\text{-}5)$$

where
F_{cre} = Critical elastic *lateral-torsional buckling stress*, determined in accordance with Appendix 2

F2.1.1 Inelastic Reserve Strength

For the *Effective Width Method*, the inelastic flexural reserve capacity is permitted to be used provided the following conditions are met:

(1) The member is not subject to twisting or to lateral, *torsional*, or *flexural-torsional buckling*.
(2) The effect of cold work of forming is not included in determining the *yield stress* F_y.
(3) The ratio of the depth of the compressed portion of the *web* to its *thickness* does not exceed λ_1 as defined in Eq. F2.1.1-3.
(4) The shear force does not exceed $0.35F_y$ for *ASD*, and $0.6F_y$ for *LRFD* and *LSD* times the *web* area (*thickness* times *flat width*).
(5) The angle between any *web* and the vertical does not exceed 30 degrees.

The *nominal flexural strength* [*resistance*], M_{ne}, shall be the moment at which the compression strain does not exceed $C_y\varepsilon_y$ for any element (no limit is placed on the maximum tensile strain), and M_{ne} shall not exceed $1.25S_eF_y$, where

S_e = Effective section modulus calculated relative to extreme compression or tension fiber at F_y
F_y = *Yield stress*
ε_y = Yield strain

$$= F_y/E \qquad (Eq.\ F2.1.1\text{-}1)$$

where
E = Modulus of elasticity of steel

C_y = Compression strain factor calculated as follows:

(a) For stiffened compression elements without intermediate stiffeners

C_y = 3 when $w/t \leq \lambda_1$

$$C_y = 3-2\left(\frac{w/t-\lambda_1}{\lambda_2-\lambda_1}\right) \text{ when } \lambda_1 < \frac{w}{t} < \lambda_2 \quad (Eq.\ F2.1.1\text{-}2)$$

$C_y = 1$ when $w/t \geq \lambda_2$

where

$$\lambda_1 = \frac{1.11}{\sqrt{F_y/E}} \quad (Eq.\ F2.1.1\text{-}3)$$

$$\lambda_2 = \frac{1.28}{\sqrt{F_y/E}} \quad (Eq.\ F2.1.1\text{-}4)$$

(b) For unstiffened compression elements under *stress* gradient causing compression at one longitudinal edge and tension at the other longitudinal edge:

$C_y = 3$ when $\lambda \leq \lambda_3$

$C_y = 3-2[(\lambda-\lambda_3)/(\lambda_4-\lambda_3)]$ when $\lambda_3 < \lambda < \lambda_4$ (*Eq*. F2.1.1-5)

$C_y = 1$ when $\lambda \geq \lambda_4$

where

λ = Slenderness factor defined in Section 1.2.2

λ_3 = 0.43

$\lambda_4 = 0.673(1+\psi)$ (*Eq*. F2.1.1-6)

where

ψ = A value defined in Section 1.2.2

(c) For all other elements:

$C_y = 1$

M_{ne} shall be calculated considering equilibrium of *stresses*, assuming an ideally elastic-plastic *stress*-strain curve, which is the same in tension as in compression, assuming small deformation, and assuming that plane sections remain plane during bending. Combined bending and *web crippling* shall be checked by the provisions of Section H3.

F2.2 Direct Strength Method

For the *Direct Strength Method*, the *nominal flexural strength* [*resistance*], M_{ne}, for *yielding* and global (*lateral-torsional*) *buckling* shall be calculated as follows:

For $M_{cre} > 0.5M_p$

$$M_{ne} = M_p(1-0.25M_p/M_{cre}) \quad (Eq.\ F2.2\text{-}1)$$

For $M_{cre} \leq 0.5M_p$

$$M_{ne} = M_{cre} \quad (Eq.\ F2.2\text{-}2)$$

where

M_{cre} = Critical elastic *lateral-torsional buckling* moment determined in accordance with Appendix 2

M_p = Member plastic moment

$= Z_fF_y$ (*Eq*. F2.2-3)

where

Z_f = Plastic section modulus

F_y = *Yield stress*

If the effect of cold work of forming is included in determining the *yield stress* F_y, M_{ne} shall not exceed M_y determined in accordance with Section F2.1.

F2.3 Cylindrical Tubes

For closed cylindrical tubes having a ratio of outside diameter to wall *thickness*, D/t, not greater than 0.441 E/F_y, the *nominal flexural strength* [*resistance*], M_{ne}, shall be calculated as follows:

$$M_{ne} = 1.25\, S_f F_y \qquad (Eq.\ F2.3\text{-}1)$$

Ω_b = 1.67 (*ASD*)

ϕ_b = 0.95 (*LRFD*)

= 0.90 (*LSD*)

where

S_f = Elastic section modulus of full unreduced section relative to extreme fiber

F_y = *Yield stress*

F3 Local Buckling Interacting With Yielding and Global Buckling

All members shall be checked for potential reduction in *available strength* [*factored resistance*] due to interaction of the *yielding* or global *buckling* with *local buckling*. This reduction shall be considered through either the *Effective Width Method* of Section F3.1, the *Direct Strength Method* of Section F3.2, or the cylindrical tube provisions of Section F3.3.

The applicable *safety factor* and *resistance factors* given in this section, unless otherwise specified, shall be used to determine the *available flexural strength* [*factored resistance*] ($\phi_b M_{n\ell}$ or $M_{n\ell}/\Omega_b$) in accordance with the applicable design method in Section B3.2.1, B3.2.2, or B3.2.3.

Ω_b = 1.67 (*ASD*)

ϕ_b = 0.90 (*LRFD*)

= 0.90 (*LSD*)

F3.1 Effective Width Method

For the *Effective Width Method*, the *nominal flexural strength* [*resistance*], $M_{n\ell}$, for *local buckling* shall be calculated in accordance with the following:

$$M_{n\ell} = S_{ec} F_n \le S_{et} F_y \qquad (Eq.\ F3.1\text{-}1)$$

where

S_{ec} = Effective section modulus calculated at extreme fiber compressive *stress* of F_n

F_n = Global flexural *stress* as determined in Section F2.1

S_{et} = Effective section modulus calculated at extreme fiber tension *stress* of F_y

F_y = *Yield stress*

S_{ec} and S_{et} shall be determined from the *effective width* of each element comprising the cross-section. The *effective width* of all elements is determined in accordance with Appendix 1 at extreme compressive *stress* F_n.

For members with holes, the elements adjacent to the hole shall be treated as unstiffened elements. S_{ec} and S_{et} shall be determined from the *effective width* in accordance with Appendix 1, and F_{cre} shall be multiplied by S_{fc}/S_{fcnet} in Section F2.1 to determine F_n, where S_{fcnet} is the net section modulus referenced to the extreme compression fiber.

F3.1.1 Local Inelastic Reserve Strength

The Element-Based Method of Section F2.1.1 shall be applied as given in that section. When applicable, *effective design widths* (Appendix 1) shall be used in calculating section properties.

F3.2 Direct Strength Method

For the *Direct Strength Method*, the *nominal flexural strength* [*resistance*], $M_{n\ell}$, for *local buckling* shall be determined as follows for $\lambda_\ell \le 5$:

$$M_{n\ell} = k_s \overline{M}_{ne}\left(\frac{1+0.10\alpha_s\lambda_\ell^2}{1+0.55\beta_s\lambda_\ell^2}\right) \le M_{y3} \qquad (Eq.\ F3.2\text{-}1)$$

where

$$\lambda_\ell = \sqrt{\overline{M}_{ne}/M_{cr\ell}} \qquad (Eq.\ F3.2\text{-}2)$$

User Note:
It is rare for a section to have *local buckling* slenderness λ_ℓ greater than 5. The *Commentary* provides some rational methods to approximate the strength for high slenderness applications.

$\overline{M}_{ne}$ = Lesser of M_{ne} and M_y

M_{ne} = *Nominal flexural strength* [*resistance*] for *lateral-torsional buckling* as defined in Section F2.2

M_y = Member *yield moment* in accordance with Section F2.1

$M_{cr\ell}$ = Critical elastic *local buckling* moment, determined in accordance with Appendix 2, including the influence of holes if applicable

k_s = M_p/M_y (*Eq*. F3.2-3)

M_p = Member plastic moment in accordance with Section F2.2

α_s = 1 for cross-sections with corners at the extreme compression fiber

= 0 for other cross-sections

β_s = $2y_c/d \ge 0.4$ (*Eq*. F3.2-4)

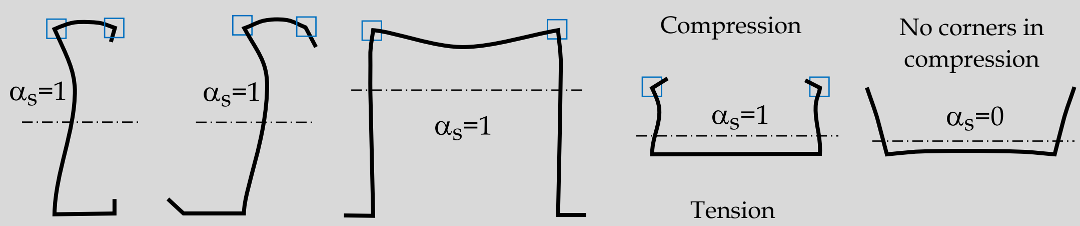
d = Overall depth of cross-section perpendicular to axis of bending

y_c = Distance from elastic neutral axis to extreme compression fiber

M_{y3} = Member moment at strain limit

$$= M_p - (M_p - M_y)/9 \qquad (Eq.\ F3.2\text{-}5)$$

User Note:
Examples of α_s for *local buckling* are shown below. Note that corners do not translate under *local buckling*. For cross-sections with corners between the extreme tension and compression fibers, α_s may conservatively be taken as 0 or determined rationally as discussed in the *Commentary*.



□ = Corners at extreme compression fiber (no translation)

For members with holes, $M_{n\ell}$ shall be the lesser of Eq. F3.2-1 and Eq. F3.2-6. Eq. F3.2-1 shall use the lesser of $M_{cr\ell}$ at the gross or net section. Eq. F3.2-6 shall use $M_{cr\ell}$ at the net section.

$$M_{n\ell} = M_{pnet}\left[\frac{1+0.10\alpha_s\lambda_\ell^2}{1+0.55\beta_s\lambda_\ell^2\left(\dfrac{M_{pnet}}{k_s\overline{M}_{ne}}\right)}\right] \le M_{y3net} \qquad (Eq.\ F3.2\text{-}6)$$

where

M_{pnet} = Member plastic moment of net cross-section

$= Z_{fnet}F_y$ (*Eq*. F3.2-7)

Z_{fnet} = Plastic section modulus of the net cross-section

M_{y3net} = Member moment at strain limit

$= M_{pnet} - (M_{pnet} - M_{ynet})/9$ (*Eq*. F3.2-8)

M_{ynet} = Member yield moment of net cross-section

$= S_{fnet}F_y$ (*Eq*. F3.2-9)

S_{fnet} = Net section modulus referenced to the extreme fiber at first yield

F_y = *Yield stress*

F3.3 Cylindrical Tubes

For closed cylindrical tubes having a ratio of outside diameter to wall *thickness*, D/t, not greater than 0.441 E/F_y, the *nominal flexural strength* [*resistance*], $M_{n\ell}$, for *local buckling* shall be determined as follows, with the *safety factor* and *resistance factors* given in Section F2.3:

For $M_{n\ell e} \ge 3.67\, M_{ne}$

$$M_{n\ell} = M_{ne} \qquad (Eq.\ F3.3\text{-}1)$$

For $3.67\, M_{ne} > M_{n\ell e} \ge 0.826\, M_{ne}$

$$M_{n\ell} = 0.776\, M_{ne} + 0.061\, M_{n\ell e} \qquad (Eq.\ F3.3\text{-}2)$$

For $M_{n\ell e} < 0.826\ M_{ne}$

$$M_{n\ell} = M_{n\ell e} \quad (Eq.\ F3.3\text{-}3)$$

where

M_{ne} = *Nominal flexural strength* [*resistance*] for *yielding* and global *buckling* as defined in Section F2.3

$M_{n\ell e}$ = *Nominal flexural strength* [*resistance*] for elastic *local buckling*

$$= 0.656\ tEI/D^2 \quad (Eq.\ F3.3\text{-}4)$$

t = Wall *thickness*

I = Moment of inertia of cylindrical tube

D = Outside diameter of cylindrical tube

F4 Distortional Buckling

The *nominal flexural strength* [*resistance*], M_{nd}, for *distortional buckling* shall be calculated in accordance with this section for $\lambda_d \le 5$. The provisions of this section shall apply to I-, Z-, C-, and other open cross-section members that employ compression *flanges* with edge stiffeners, U-, Hat, panel, or similar cross-section members with *flanges* in compression, or any cross-section with intermediate stiffeners in compression.

Exception: The provisions of this section do not apply to *distortional buckling* modes involving only intermediate stiffeners if $M_{n\ell}$ is determined using Section F3.1 and the intermediate stiffener provisions of Appendix 1 Section 1.4. *Distortional buckling* modes involving edge-stiffened *flanges* are not part of this exception.

The applicable *safety factor* and *resistance factors* given in this section shall be used to determine the *available flexural strength* [*factored resistance*] ($\phi_b M_{nd}$ or M_{nd}/Ω_b) in accordance with the applicable design method in Section B3.2.1, B3.2.2, or B3.2.3.

Ω_b = 1.67 (*ASD*)

ϕ_b = 0.90 (*LRFD*)

= 0.90 (*LSD*)

$$M_{nd} = M_p \left(\frac{1 + 0.07\alpha_s \lambda_d^2}{1 + 0.60\beta_s \lambda_d^2} \right) \le M_{y3} \quad (Eq.\ F4\text{-}1)$$

where

$$\lambda_d = \sqrt{M_y / M_{crd}} \quad (Eq.\ F4\text{-}2)$$

User Note:
It is rare for a section to have *distortional buckling* slenderness λ_d greater than 5. The *Commentary* provides a rational approach to approximate the strength for high slenderness applications.

M_y = Member yield moment in accordance with Section F2.1

M_{crd} = Critical elastic d*istortional buckling* moment, determined in accordance with Appendix 2

M_p = Member plastic moment in accordance with Section F2.2

α_s = 1 for cross-sections with one or more corners at the extreme compression fiber that do not translate under *distortional buckling*

= 0 for other cross-sections

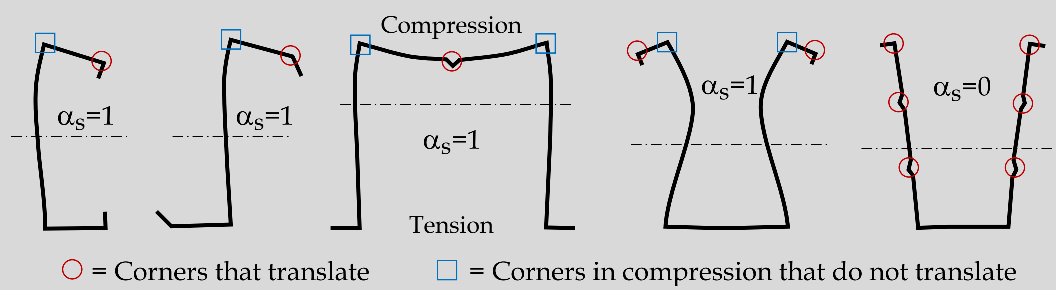
β_s = $2y_c/d \geq 0.4$ (*Eq.* F4-3)

d = Overall depth of cross-section perpendicular to axis of bending

y_c = Distance from elastic neutral axis to extreme compression fiber

M_{y3} = Member moment at strain limit in accordance with Section F3.2

User Note:
Examples of α_s for *distortional buckling* are shown below. For cross-sections with non-translating corners between the extreme tension and compression fibers, α_s may conservatively be taken as 0 or determined rationally as discussed in the *Commentary*.



For members with holes, M_{crd} shall be determined including the influence of holes and M_{nd} shall be calculated as follows:

$$M_{nd} = M_{pnet}\left[\frac{1+0.07\alpha_s\lambda_d^2}{1+0.60\beta_s\lambda_d^2\left(\frac{M_{pnet}}{M_p}\right)}\right] \leq M_{y3net} \quad (Eq.\ F4\text{-}4)$$

where M_{pnet} and M_{y3net} are defined in Section F3.2.

G. MEMBERS IN SHEAR, WEB CRIPPLING, AND TORSION

This chapter addresses member strength for *limit states* not addressed in Chapters D, E, and F, including shear, *web crippling*, and torsion. The design of transverse *web* stiffeners and bearing stiffeners is considered as well.

This chapter is organized as follows:

G1 General Requirements
G2 Shear Strength of Webs Without Holes
G3 Shear Strength of C-Section Webs With Holes
G4 Transverse Web Stiffeners
G5 Web Crippling Strength of Webs Without Holes
G6 Web Crippling Strength of C-Section Webs With Holes
G7 Bearing Stiffeners
G8 Torsion Strength

G1 General Requirements

The *available shear strength* [*factored resistance*] of singly-, doubly-, or point-symmetric cross-section members subject to shear in the plane of *web* shall be determined in accordance with Section G2 for *webs* without holes and Section G3 for *webs* with holes, as applicable. Transverse *web* stiffeners shall be designed in accordance with Section G4, as applicable. Members subjected to concentrated *loads* or reactions on the *web* shall be checked for *web crippling* in accordance with Sections G5 or G6, as applicable. Bearing stiffeners shall be designed in accordance with Section G7, as applicable. Members subjected to torsion shall be checked in accordance with Section G8, as applicable.

G2 Shear Strength of Webs Without Holes

The *nominal shear strength* [*resistance*], V_n, of flexural members without holes in the *web*(*s*) shall be calculated in accordance with this section, as applicable. For flexural members meeting the geometric and material criteria of Section B4, Ω_v and ϕ_v shall be as follows:

Ω_v = 1.67 (*ASD*)
ϕ_v = 0.90 (*LRFD*)
= 0.75 (*LSD*)

For all other flexural members, Ω and ϕ of the *Specification*, Section A1.2.6(c), shall apply. The *available strength* [*factored resistance*] shall be determined in accordance with the applicable design method in Section B3.2.1, B3.2.2, or B3.2.3 of the *Specification*.

G2.1 Flexural Members Without Transverse Web Stiffeners

The *nominal shear strength* [*resistance*], V_n, of flexural members without transverse *web* stiffeners shall be calculated as follows:

$$V_n = \frac{1.2V_y}{1+0.57\lambda_v^2} \leq V_y \qquad (Eq.\ G2.1\text{-}1)$$

where

$$\lambda_v = \sqrt{\frac{V_y}{V_{cr}}} \qquad (Eq.\ G2.1\text{-}2)$$

V_y = Yield shear force of cross-section

$$= 0.6\, htF_y \qquad (Eq.\ G2.1\text{-}3)$$

where

h = Depth of flat portion of *web* measured along plane of *web*

t = *Web thickness*

F_y = Design *yield stress* as determined in accordance with Section A3.3.1

V_{cr} = Elastic *shear buckling* force as defined in Section G2.3 for flat *web* alone, or determined in accordance with Appendix 2 for full cross-section of prequalified (Section B4.1) members

G2.2 Flexural Members With Transverse Web Stiffeners

For a flexural member reinforced with transverse *web* stiffeners meeting the criteria described below, this section is permitted to be used to determine the *nominal shear strength* [*resistance*], V_n, in lieu of Section G2.1.

(a) Transverse *web* stiffener strength and stiffness meet the criteria of Section G4,

(b) Transverse *web* stiffener spacing does not exceed twice the *web* depth,

(c) *Flanges* are restrained from distortion where stiffener spacing is larger than *web* depth, and

(d) Both ends of shear spans are fastened to transverse stiffeners or to supporting members over the full depth of the *web*.

$$V_n = 1.2V_y\left(\frac{1+0.05\lambda_v^2}{1+0.45\lambda_v^2}\right) \le V_y \qquad (Eq.\ G2.2\text{-}1)$$

where V_y and λ_v are defined in Section G2.1.

G2.3 Web Elastic Critical Shear Buckling Force, V_{cr}

The *shear buckling* force, V_{cr}, of a *web* is permitted to be determined in accordance with this section:

$$V_{cr} = A_w F_{cr} \qquad (Eq.\ G2.3\text{-}1)$$

where

A_w = *Web* area

$$= ht \qquad (Eq.\ G2.3\text{-}2)$$

F_{cr} = Elastic shear *buckling stress*

$$= \frac{\pi^2 E k_v}{12(1-\mu^2)(h/t)^2} \qquad (Eq.\ G2.3\text{-}3)$$

where

E = Modulus of elasticity of steel

k_v = *Shear buckling* coefficient calculated in accordance with (a) or (b) as follows:

(a) For unreinforced *webs*, k_v = 5.34

(b) For *webs* with transverse stiffeners satisfying the requirements of Section G4

when $a/h \le 1.0$

$$k_v = 4.00 + \frac{5.34}{(a/h)^2} \quad (Eq.\ G2.3\text{-}4)$$

when $a/h > 1.0$

$$k_v = 5.34 + \frac{4.00}{(a/h)^2} \quad (Eq.\ G2.3\text{-}5)$$

where

a = Shear panel length of unreinforced *web* element

= Clear distance between transverse stiffeners of reinforced *web* elements

Other variables are defined in Section G2.1.

G3 Shear Strength of C-Section Webs With Holes

For C-section *webs* with square, rectangular, circular, or slotted holes, the *available shear strength* [*factored resistance*] shall be calculated in accordance with Section G2, with $V_y = V_{yh}$ and $V_{cr} = V_{crh}$ computed as below within the following limits:

(a) $d_h/h \le 0.8$,

(b) $1.0 \le a/h \le 2.0$ for stiffened *webs*,

(c) $L_h/a \le 0.9$,

(d) $1.0 \le L_h/d_h \le 3.0$,

(e) Holes centered at mid-depth of *web*,

(f) Clear distance between holes ≥ 18 in. (457 mm),

(g) Holes mid-way between stiffeners for stiffened *webs*, and

(h) Non-circular holes corner radii ≥ 2t.

where

d_h= Overall depth of *web* hole

L_h= Overall length of *web* hole taken along the member

h = Depth of flat portion of *web* measured along plane of *web*

t = *Web thickness*

a = Distance between stiffeners for stiffened *webs* or twice the distance from the end of the section to the center of the hole for transversely unstiffened *webs*

For $0 < d_h/h \le 0.10$

$$V_{yh} = V_y \quad (Eq.\ G3\text{-}1)$$

For $0.10 < d_h/h \le 0.80$

$$V_{yh} = V_y\left[1 + a_0\left(\frac{d_h}{h} - 0.1\right) + a_1\left(\frac{d_h}{h} - 0.1\right)^2 + a_2\left(\frac{d_h}{h} - 0.1\right)^3\right] \quad (Eq.\ G3\text{-}2)$$

where

$$a_0 = -0.173 - 0.925\left(\frac{L_h}{d_h}\right) + 0.0524\left(\frac{L_h}{d_h}\right)^2 \quad (Eq.\ G3\text{-}3)$$

$$a_1 = -3.41 + 1.99\left(\frac{L_h}{d_h}\right) - 0.0995\left(\frac{L_h}{d_h}\right)^2 \qquad (Eq.\ G3\text{-}4)$$

$$a_2 = 2.68 - 1.08\left(\frac{L_h}{d_h}\right) + 0.0466\left(\frac{L_h}{d_h}\right)^2 \qquad (Eq.\ G3\text{-}5)$$

V_y = Yield shear force of the cross-section as determined in accordance with Eq. G2.1-3

$$V_{crh} = \alpha_{vh} V_{cr} \qquad (Eq.\ G3\text{-}6)$$

where

$$\alpha_{vh} = \left[1 - 0.4\left(\frac{L_h - d_h}{h}\right)\right]^2 \qquad (Eq.\ G3\text{-}7)$$

The *shear buckling* force, V_{cr}, shall be calculated in accordance with Section G2.3 where the value of k_v for a *web* with a square or rectangular hole shall be determined as follows:

$$k_v = 4.86 + 6.15\left(\frac{h}{a}\right) - 3.63\left(\frac{d_h}{h}\right) - 19.6\left(\frac{d_h}{a}\right) + 13.9\left(\frac{d_h^2}{ah}\right) + 0.57\left(\frac{b_f}{h}\right) \qquad (Eq.\ G3\text{-}8)$$

where

b_f = Overall *flange* width

For circular and slotted holes, d_h and L_h shall be replaced by $d_{h\text{-}eq}$ and $L_{h\text{-}eq}$, respectively, as equivalent values for circular or slotted holes in Eqs. G3-2 to G3-8, where

$$d_{h\text{-}eq} = [0.003(L_h/d_h) + 0.822]d_h \qquad (Eq.\ G3\text{-}9)$$

$$L_{h\text{-}eq} = 0.865A_h / d_{h\text{-}eq} \qquad (Eq.\ G3\text{-}10)$$

A_h = Area of circular hole or slotted hole

G4 Transverse Web Stiffeners

G4.1 Compact Transverse Web Stiffeners

Where transverse *web* stiffeners are required for shear, the spacing shall be based on the *nominal shear strength* [*resistance*], V_n, permitted by Section G2.2, and the ratio a/h shall not exceed 2.0, where a is the distance between transverse stiffeners and h is defined in this section.

This section shall apply to plate and angle type stiffeners which satisfy:

$$b_{st}/t_{st} \le 0.56\sqrt{\frac{E}{F_{yst}}} \qquad (Eq.\ G4.1\text{-}1)$$

where

b_{st} = Stiffener flat width perpendicular to the *web*

t_{st} = Stiffener *thickness*

E = Modulus of elasticity of steel

F_{yst} = Design *yield stress* of stiffener

The actual moment of inertia, I_s, of a pair of attached transverse *web* stiffeners about an axis in the *web* centerline for stiffener pairs, or of a single transverse *web* stiffener about the face in contact with the *web* plate shall have a minimum value calculated in accordance with Eq. G4.1-2 as follows:

I_{st1} = Moment of inertia of the transverse stiffeners required for development of the full shear post *buckling* resistance of the stiffened *web* panels

$$= \frac{h^4 \rho_{st}^{1.3}}{40}\left(\frac{F_y}{E}\right)^{1.5} \qquad (Eq.\ G4.1\text{-}2)$$

where

h = Depth of flat portion of *web* measured along plane of *web*

$$\rho_{st} = \frac{F_y}{F_{yst}} \geq 1.0 \qquad (Eq.\ G4.1\text{-}3)$$

F_y = Design *yield stress* of *web*

The *connections* between the stiffener and the *web* shall ensure full connectivity over the entire *web* depth.

G4.2 Other Transverse Web Stiffeners

The *available strength* [*factored resistance*] of members with transverse *web* stiffeners that do not meet the requirements of Section G4.1, such as stamped or rolled-in stiffeners, shall be determined by tests in accordance with Section K2 or *rational engineering analysis* in accordance with Section A1.2.6(c).

G5 Web Crippling Strength of Webs Without Holes

The *nominal web crippling strength* [*resistance*], P_n, shall be determined in accordance with Eq. G5-1 or Eq. G5-2, as applicable. The *safety factors* and *resistance factors* in Tables G5-1 to G5-5 shall be used to determine the *allowable strength* or *design strength* [*factored resistance*] in accordance with the applicable design method in Section B3.2.1, B3.2.2, or B3.2.3.

$$P_n = Ct^2 F_y \sin\theta \left(1 - C_R\sqrt{\frac{R}{t}}\right)\left(1 + C_N\sqrt{\frac{N}{t}}\right)\left(1 - C_h\sqrt{\frac{h}{t}}\right) \qquad (Eq.\ G5\text{-}1)$$

where:

P_n = *Nominal web crippling strength* [*resistance*]
C = Coefficient from Table G5-1, G5-2, G5-3, G5-4, or G5-5
t = *Web thickness*
F_y = Design *yield stress* as determined in accordance with Section A3.3.1
θ = Angle between plane of web and plane of bearing surface, $45° \leq \theta \leq 90°$
C_R = Inside bend radius coefficient from Table G5-1, G5-2, G5-3, G5-4, or G5-5
R = Inside bend radius
C_N = Bearing length coefficient from Table G5-1, G5-2, G5-3, G5-4, or G5-5
N = Bearing length (3/4 in. (19 mm) minimum)
C_h = *Web* slenderness coefficient from Table G5-1, G5-2, G5-3, G5-4, or G5-5
h = Flat dimension of *web* measured in plane of *web*

Alternatively, for an end one-*flange* loading condition on a C- or Z-section, the *nominal web crippling strength* [*resistance*], P_{nc}, with an overhang on one side, is permitted to be calculated as follows, except that P_{nc} shall not be larger than the interior one-*flange* loading condition:

$$P_{nc} = \alpha P_n \qquad (Eq.\ G5\text{-}2)$$

where

P_{nc} = *Nominal web crippling strength* [*resistance*] of C- and Z-sections with overhang(s)

$$\alpha = \frac{1.34(L_o / h)^{0.26}}{0.009(h / t) + 0.3} \geq 1.0 \qquad (Eq.\ G5\text{-}3)$$

where

L_o = Overhang length measured from edge of bearing to the end of the member

P_n = *Nominal web crippling strength* [*resistance*] with end one-*flange* loading as calculated by Eq. G5-1 and Tables G5-2 and G5-3

Eq. G5-2 shall be limited to $0.5 \leq L_o/h \leq 1.5$ and $h/t \leq 154$. For L_o/h or h/t outside these limits, α=1.

Webs of members in bending for which h/t is greater than 200 shall be provided with means of transmitting concentrated *loads* or reactions directly into the *web*(*s*).

P_n and P_{nc} shall represent the *nominal strengths* [*resistances*] for *load* or reaction for one solid *web* connecting top and bottom *flanges*. For hat, multi-*web* sections and C- or Z-sections, P_n or P_{nc} shall be the *nominal strength* [*resistance*] for a single *web*, and the total *nominal strength* [*resistance*] shall be computed by multiplying P_n or P_{nc} by the number of *webs* at the considered cross-section.

One-*flange* loading or reaction shall be defined as the condition where the clear distance between the bearing edges of adjacent opposite concentrated *loads* or reactions is equal to or greater than 1.5h.

Two-*flange* loading or reaction shall be defined as the condition where the clear distance between the bearing edges of adjacent opposite concentrated *loads* or reactions is less than 1.5h.

End loading or reaction shall be defined as the condition where the distance from the edge of the bearing to the end of the member is equal to or less than 1.5h.

Interior loading or reaction shall be defined as the condition where the distance from the edge of the bearing to the end of the member is greater than 1.5h, except as otherwise noted herein.

Table G5-1 shall apply to I-beams made from two channels connected back-to-back where $h/t \leq 200$, $N/t \leq 210$, $N/h \leq 1.0$, and $\theta = 90°$. See Section G5 of *Commentary* for further explanation.

TABLE G5-1
Safety Factors, Resistance Factors, and Coefficients for Built-Up Sections per Web

Support and Flange Conditions		Load Cases		C	C_R	C_N	C_h	USA and Mexico ASD Ω_w	USA and Mexico LRFD ϕ_w	Canada LSD ϕ_w	Limits
Fastened to Support	Stiffened or Partially Stiffened Flanges	One-Flange Loading or Reaction	End	10	0.14	0.28	0.001	2.00	0.75	0.60	$R/t \leq 5$
			Interior	20.5	0.17	0.11	0.001	1.75	0.85	0.75	$R/t \leq 5$
Unfastened	Stiffened or Partially Stiffened Flanges	One-Flange Loading or Reaction	End	10	0.14	0.28	0.001	2.00	0.75	0.60	$R/t \leq 5$
			Interior	20.5	0.17	0.11	0.001	1.75	0.85	0.75	$R/t \leq 3$
		Two-Flange Loading or Reaction	End	15.5	0.09	0.08	0.04	2.00	0.75	0.65	$R/t \leq 3$
			Interior	36	0.14	0.08	0.04	2.00	0.75	0.65	
	Unstiffened Flanges	One-Flange Loading or Reaction	End	10	0.14	0.28	0.001	2.00	0.75	0.60	$R/t \leq 5$
			Interior	20.5	0.17	0.11	0.001	1.75	0.85	0.75	$R/t \leq 3$

Table G5-2 shall apply to single *web* channel and C-section members where $h/t \le 200$, $N/t \le 210$, $N/h \le 2.0$, and $\theta = 90°$. In Table G5-2, for interior two-*flange* loading or reaction of members having *flanges* fastened to the support, the distance from the edge of the bearing to the end of the member shall be extended at least 2.5h. For unfastened cases, the distance from the edge of the bearing to the end of the member shall be extended at least 1.5h.

TABLE G5-2
Safety Factors, Resistance Factors, and Coefficients for Single Web Channel and C-Sections

Support and Flange Conditions		Load Cases		C	C_R	C_N	C_h	USA and Mexico ASD Ω_w	USA and Mexico LRFD ϕ_w	Canada LSD ϕ_w	Limits
Fastened to Support	Stiffened or Partially Stiffened Flanges	One-Flange Loading or Reaction	End	4	0.14	0.35	0.02	1.75	0.85	0.75	$R/t \le 9$
			Interior	13	0.23	0.14	0.01	1.65	0.90	0.80	$R/t \le 5$
		Two-Flange Loading or Reaction	End	7.5	0.08	0.12	0.048	1.75	0.85	0.75	$R/t \le 12$
			Interior	20	0.10	0.08	0.031	1.75	0.85	0.75	$R/t \le 12$ $d^1 \ge 4.5$ in. (110 mm)
Unfastened	Stiffened or Partially Stiffened Flanges	One-Flange Loading or Reaction	End	4	0.14	0.35	0.02	1.85	0.80	0.70	$R/t \le 5$
			Interior	13	0.23	0.14	0.01	1.65	0.90	0.80	
		Two-Flange Loading or Reaction	End	13	0.32	0.05	0.04	1.65	0.90	0.80	$R/t \le 3$
			Interior	24	0.52	0.15	0.001	1.90	0.80	0.65	
	Unstiffened Flanges	One-Flange Loading or Reaction	End	4	0.40	0.60	0.03	1.80	0.85	0.70	$R/t \le 2$
			Interior	13	0.32	0.10	0.01	1.80	0.85	0.70	$R/t \le 1$
		Two-Flange Loading or Reaction	End	2	0.11	0.37	0.01	2.00	0.75	0.65	$R/t \le 1$
			Interior	13	0.47	0.25	0.04	1.90	0.80	0.65	

Note: [1] d = Out-to-out depth of section in the plane of the *web*

Table G5-3 shall apply to single *web* Z-section members where $h/t \le 200$, $N/t \le 210$, $N/h \le 2.0$, and $\theta = 90°$. In Table G5-3, for interior two-*flange* loading or reaction of members having *flanges* fastened to the support, the distance from the edge of the bearing to the end of the member shall be extended at least 2.5h; for unfastened cases, the distance from the edge of the bearing to the end of the member shall be extended at least 1.5h.

TABLE G5-3
Safety Factors, Resistance Factors, and Coefficients for Single Web Z-Sections

Support and Flange Conditions		Load Cases		C	C_R	C_N	C_h	USA and Mexico		Canada LSD ϕ_w	Limits
								ASD Ω_w	LRFD ϕ_w		
Fastened to Support	Stiffened or Partially Stiffened Flanges	One-Flange Loading or Reaction	End	4	0.14	0.35	0.02	1.75	0.85	0.75	$R/t \le 9$
			Interior	13	0.23	0.14	0.01	1.65	0.90	0.80	$R/t \le 5.5$
		Two-Flange Loading or Reaction	End	9	0.05	0.16	0.052	1.75	0.85	0.75	$R/t \le 12$
			Interior	24	0.07	0.07	0.04	1.85	0.80	0.70	$R/t \le 12$
Unfastened	Stiffened or Partially Stiffened Flanges	One-Flange Loading or Reaction	End	5	0.09	0.02	0.001	1.80	0.85	0.75	$R/t \le 5$
			Interior	13	0.23	0.14	0.01	1.65	0.90	0.80	
		Two-Flange Loading or Reaction	End	13	0.32	0.05	0.04	1.65	0.90	0.80	$R/t \le 3$
			Interior	24	0.52	0.15	0.001	1.90	0.80	0.65	
	Unstiffened Flanges	One-Flange Loading or Reaction	End	4	0.40	0.60	0.03	1.80	0.85	0.70	$R/t \le 2$
			Interior	13	0.32	0.10	0.01	1.80	0.85	0.70	$R/t \le 1$
		Two-Flange Loading or Reaction	End	2	0.11	0.37	0.01	2.00	0.75	0.65	$R/t \le 1$
			Interior	13	0.47	0.25	0.04	1.90	0.80	0.65	

Table G5-4 shall apply to single hat section members where h/t ≤ 200, N/t ≤ 200, N/h ≤ 2, and θ = 90°.

TABLE G5-4
Safety Factors, Resistance Factors, and Coefficients for Single Hat Sections per Web

Support Conditions	Load Cases		C	C_R	C_N	C_h	USA and Mexico ASD Ω_w	USA and Mexico LRFD ϕ_w	Canada LSD ϕ_w	Limits
Fastened to Support	One-Flange Loading or Reaction	End	4	0.25	0.68	0.04	2.00	0.75	0.65	R/t ≤ 5
		Interior	17	0.13	0.13	0.04	1.80	0.85	0.70	R/t ≤ 10
	Two-Flange Loading or Reaction	End	9	0.10	0.07	0.03	1.75	0.85	0.75	R/t ≤ 10
		Interior	10	0.14	0.22	0.02	1.80	0.85	0.75	
Unfastened	One-Flange Loading or Reaction	End	4	0.25	0.68	0.04	2.00	0.75	0.65	R/t ≤ 5
		Interior	17	0.13	0.13	0.04	1.80	0.85	0.70	R/t ≤ 10

Table G5-5 shall apply to multi-*web* section members where h/t ≤ 200, N/t ≤ 210, N/h ≤ 3, and 45° ≤ θ ≤ 90°.

TABLE G5-5
Safety Factors, Resistance Factors, and Coefficients for Multi-Web Deck Sections per Web

Support Conditions	Load Cases		C	C_R	C_N	C_h	USA and Mexico ASD Ω_w	USA and Mexico LRFD ϕ_w	Canada LSD ϕ_w	Limits
Fastened to Support	One-Flange Loading or Reaction	End	4	0.04	0.25	0.025	1.70	0.90	0.80	R/t ≤ 20
		Interior	8	0.10	0.17	0.004	1.75	0.85	0.75	
	Two-Flange Loading or Reaction	End	9	0.12	0.14	0.040	1.80	0.85	0.70	R/t ≤ 10
		Interior	10	0.11	0.21	0.020	1.75	0.85	0.75	
Unfastened	One-Flange Loading or Reaction	End	3	0.04	0.29	0.028	2.45	0.60	0.50	R/t ≤20
		Interior	8	0.10	0.17	0.004	1.75	0.85	0.75	
	Two-Flange Loading or Reaction	End	6	0.16	0.15	0.050	1.65	0.90	0.80	R/t ≤ 5
		Interior	17	0.10	0.10	0.046	1.65	0.90	0.80	

Note: Multi-*web* deck sections are considered unfastened for any support fastener spacing greater than 18 in. (460 mm).

G6 Web Crippling Strength of C-Section Webs With Holes

Where a *web* hole is within the bearing length, a bearing stiffener shall be used.

For beam *webs* with holes, the *available web crippling strength* [*factored resistance*] shall be calculated in accordance with Section G5, multiplied by the reduction factor, R_c, given in this section.

The provisions of this section shall apply within the following limits:

(a) $d_h/h \le 0.7$,

(b) $h/t \le 200$,

(c) Hole centered at mid-depth of *web*,

(d) Clear distance between holes ≥ 18 in. (457 mm),

(e) Distance between end of member and edge of hole ≥ d,

(f) Noncircular holes, corner radii ≥ 2t,

(g) Noncircular holes, $d_h \le 2.5$ in. (63.5 mm) and $L_h \le 4.5$ in. (114 mm),

(h) Circular holes, diameters ≤ 6 in. (152 mm), and

(i) $d_h > 9/16$ in. (14.3 mm).

where

d_h = Depth of *web* hole
h = Depth of flat portion of *web* measured along plane of *web*
t = *Web thickness*
d = Depth of cross-section
L_h = Length of *web* hole

For end one-*flange* reaction (Equation G5-1 with Table G5-2) where a *web* hole is not within the bearing length, the reduction factor, R_c, shall be calculated as follows:

$$R_c = 1.01 - 0.325 d_h/h + 0.083 x/h \le 1.0 \qquad (Eq.\ G6\text{-}1)$$

$N \ge 1$ in. (25.4 mm)

For interior one-*flange* reaction (Equation G5-1 with Table G5-2) where any portion of a *web* hole is not within the bearing length, the reduction factor, R_c, shall be calculated as follows:

$$R_c = 0.90 - 0.047 d_h/h + 0.053 x/h \le 1.0 \qquad (Eq.\ G6\text{-}2)$$

$N \ge 3$ in. (76.2 mm)

where

x = Nearest distance between *web* hole and edge of bearing
N = Bearing length

G7 Bearing Stiffeners

G7.1 Compact Bearing Stiffeners

Bearing stiffeners attached to beam *webs* at points of concentrated *loads* or reactions shall be designed as compression members. Concentrated *loads* or reactions shall be applied directly into the stiffeners, or each stiffener shall be fitted accurately to the flat portion of the *flange* to provide direct *load* bearing into the end of the stiffener. Means for shear transfer between the stiffener and the *web* shall be provided in accordance with Chapter J. For concentrated *loads* or reactions, the *nominal strength* [*resistance*], P_n, shall be the smaller value calculated by (a) and

(b) of this section. The *safety factor* and *resistance factors* provided in this section shall be used to determine the *available strength* [*factored resistance*] in accordance with the applicable design method in Section B3.2.1, B3.2.2, or B3.2.3.

Ω_c = 2.00 (*ASD*)
ϕ_c = 0.85 (*LRFD*)
= 0.80 (*LSD*)

(a) $P_n = F_{wy}A_c$ (*Eq.* G7.1-1)

(b) P_n = *Nominal axial strength* [*resistance*] evaluated in accordance with Section E3.1, with A_e replaced by A_b

where

F_{wy} = Lower value of F_y for beam *web*, or F_{ys} for stiffener section

A_c = $18t^2 + A_s$, for bearing stiffener at interior support or under concentrated *load* (*Eq.* G7.1-2)

= $10t^2 + A_s$, for bearing stiffener at end support (*Eq.* G7.1-3)

where

t = Base steel *thickness* of beam *web*

A_s = *Cross-sectional area* of bearing stiffener

A_b = $b_1t + A_s$, for bearing stiffener at interior support or under concentrated *load* (*Eq.* G7.1-4)

= $b_2t + A_s$, for bearing stiffener at end support (*Eq.* G7.1-5)

where

b_1 = $25t\,[0.0024(L_{st}/t) + 0.72] \le 25t$ (*Eq.* G7.1-6)

b_2 = $12t\,[0.0044(L_{st}/t) + 0.83] \le 12t$ (*Eq.* G7.1-7)

where

L_{st} = Length of bearing stiffener

The w/t_s ratio for the stiffened and unstiffened elements of the bearing stiffener shall not exceed $1.28\sqrt{E/F_{ys}}$ and $0.42\sqrt{E/F_{ys}}$, respectively, where F_{ys} is the *yield stress* of the stiffener steel, and t_s is the *thickness* of the stiffener steel.

G7.2 Stud and Track Type Bearing Stiffeners in C-Section Flexural Members

For two-*flange* loading of C-section flexural members with bearing stiffeners that do not meet the requirements of Section G7.1, the *nominal strength* [*resistance*], P_n, shall be calculated in accordance with Eq. G7.2-1. The *safety factor* and *resistance factors* in this section shall be used to determine the *available strength* [*factored resistance*] in accordance with the applicable design method in Section B3.2.1, B3.2.2, or B3.2.3.

$P_n = 0.7(P_{wc} + P_{n\ell o}) \ge P_{wc}$ (*Eq.* G7.2-1)

Ω_c = 1.70 (*ASD*)
ϕ_c = 0.90 (*LRFD*)
= 0.80 (*LSD*)

where

P_{wc} = *Nominal web crippling strength* [*resistance*] for C-section flexural member, calculated in accordance with Eq. G5-1 for single *web* members, at end or interior locations

$P_{n\ell o}$ = *Nominal compressive strength* [*resistance*] of stiffener determined in accordance with Section E3 with $F_n = F_y$ or $P_{ne} = P_y$, where F_y or P_y is based on *yield stress* of stiffener steel

Eq. G7.2-1 shall apply within the following limits:

(a) Full bearing of the stiffener is required. If the bearing width is narrower than the stiffener such that one of the stiffener *flanges* is unsupported, P_n is reduced by 50 percent.

(b) Stiffeners are C-section stud or track members with a minimum *web* depth of 3-1/2 in. (88.9 mm) and a minimum base steel *thickness* of 0.0329 in. (0.836 mm).

(c) The stiffener is attached to the flexural member *web* with at least three fasteners (screws or bolts).

(d) The distance from the flexural member *flanges* to the first fastener(s) is not less than d/8, where d is the overall depth of the flexural member.

(e) The length of the stiffener is not less than the depth of the flexural member minus 3/8 in. (9.53 mm).

(f) The bearing width is not less than 1-1/2 in. (38.1 mm).

G7.3 Other Stiffeners

The *available strength* [*factored resistance*] of members with stiffeners that do not meet the requirements of Sections G7.1 and G7.2, such as stamped or rolled-in stiffeners, shall be determined by tests in accordance with Section K2 or *rational engineering analysis* in accordance with Section A1.2.6.

G8 Torsion Strength

G8.1 Torsion Bimoment Strength

The provisions of this section shall apply to C-, Z-, I-, Hat-, and any other cross-section members subjected to longitudinal warping torsion *stresses*.

User Note:
Torsional loads produce longitudinal warping *stresses* for most open sections, but these *stresses* are not significant for most closed sections. The *Commentary* to Section G8 provides further guidance.

The *nominal bimoment strength* [*resistance*], B_n, shall be calculated in accordance with this section. The applicable *safety factor* and *resistance factors* given in this section shall be used to determine the *available bimoment strength* [*factored resistance*] ($\phi_b B_n$ or B_n/Ω_b) in accordance with the applicable design method in Section B3.2.1, B3.2.2, or B3.2.3.

$$B_n = B_y \qquad (Eq.\ G8.1\text{-}1)$$

Ω_b = 1.67 (*ASD*)

ϕ_b = 0.90 (*LRFD*)

= 0.90 (*LSD*)

where

B_y = First yield bimoment

$$= F_y C_w/\omega_{n,max} \qquad (Eq.\ G8.1\text{-}2)$$

F_y = *Yield stress*

C_w = Torsional warping constant of cross-section

$\omega_{n,max}$ = Maximum magnitude of normalized unit warping property of cross-section, taken as positive

Alternatively, B_n is permitted to be determined as the smaller value determined in accordance with (a) and (b) considering inelastic reserve and *buckling* of the cross-section.

(a) For modes involving *buckling* of the *web* with or without intermediate stiffeners:

$$B_n = B_p \frac{1+0.09\lambda_w^2}{1+0.23\lambda_w^2} \qquad (Eq.\ G8.1\text{-}3)$$

where

$$\lambda_w = \sqrt{\frac{B_y}{B_{crw}}} \qquad (Eq.\ G8.1\text{-}4)$$

B_{crw} = Bimoment at critical elastic *buckling* of the *web* determined using rational elastic *buckling* analysis in accordance with Appendix 2, including the influence of holes if applicable

B_p = Plastic bimoment

(b) For modes involving *buckling* of the *flange* with or without edge stiffeners:

$$B_n = \frac{B_p}{1+1.11\lambda_f^2} \qquad (Eq.\ G8.1\text{-}5)$$

where

$$\lambda_f = \sqrt{\frac{B_y}{B_{crf}}} \qquad (Eq.\ G8.1\text{-}6)$$

B_{crf} = Bimoment at critical elastic *buckling* of the *flange* determined using rational elastic *buckling* analysis in accordance with Appendix 2, including the influence of holes if applicable

User Note:
Details on the calculation of plastic bimoment, B_p, and simplified formulas for common shapes are provided in the *Commentary*.

For members with holes, the plastic bimoment, B_p, and the properties C_w and $\omega_{n,max}$ shall be based on the net section.

G8.2 Torsion Shear Strength

(Reserved)

H. MEMBERS UNDER COMBINED FORCES

This chapter addresses members subjected to axial force and flexure about one or both axes, flexure and torsion, flexure and shear, and flexure and *web crippling*.

The chapter is organized as follows:

H1 Combined Axial Load and Bending
H2 Combined Bending and Shear
H3 Combined Bending and Web Crippling
H4 Combined Bending and Torsional Loading

H1 Combined Axial Load and Bending

Members subjected to axial load and/or bending shall be evaluated by the interaction equations provided in this section, or in lieu of this section, the provisions in Appendix 3 are permitted.

H1.1 Combined Tensile Axial Load and Bending

The *required strengths* [effects of *factored loads*] $\overline{T}$, $\overline{M}_x$, and $\overline{M}_y$ shall satisfy the following interaction equations:

$$\frac{\overline{M}_x}{M_{axt}}+\frac{\overline{M}_y}{M_{ayt}}+\frac{\overline{T}}{T_a}\le 1.0 \qquad (Eq.\ H1.1\text{-}1)$$

$$\frac{\overline{M}_x}{M_{ax}}+\frac{\overline{M}_y}{M_{ay}}-\frac{\overline{T}}{T_a}\le 1.0 \qquad (Eq.\ H1.1\text{-}2)$$

where

$\overline{M}_x$, $\overline{M}_y$ = *Required flexural strengths* [moment due to *factored loads*] with respect to centroidal axes in accordance with *ASD*, *LRFD,* or *LSD load combinations*

$\overline{T}$ = *Required tensile axial strength* [tensile axial force due to *factored loads*] in accordance with *ASD*, *LRFD,* or *LSD load combinations*

M_{axt}, M_{ayt} = *Available flexural strengths* [*factored resistances*] with respect to centroidal axes in considering tension yielding

= $S_{ft}F_y/\Omega_b$ (*ASD*) (*Eq*. H1.1-3a)

= $\phi_b S_{ft} F_y$ (*LRFD*, *LSD*) (*Eq*. H1.1-3b)

where

S_{ft} = Section modulus of full unreduced section relative to extreme tension fiber about appropriate axis

F_y = Design *yield stress* determined in accordance with Section A3.3.1

Ω_b = 1.67

ϕ_b = 0.90 (*LRFD* and *LSD*)

M_{ax}, M_{ay} = *Available flexural strengths* [*factored resistances*] about centroidal axes in considering compression *buckling*, as determined in accordance with Chapter F

T_a = *Available tensile axial strength* [*factored resistance*], determined in accordance with Chapter D

H1.2 Combined Compressive Axial Load and Bending

The *required strengths* [effects due to *factored loads*] $\overline{P}$, $\overline{M}_x$, and $\overline{M}_y$ shall satisfy Eq. H1.2-1.

For singly-symmetric unstiffened angle sections not subject to *local buckling* at *stress* F_y, $\overline{M}_y$ is permitted to be taken as the *required flexural strength* [moment due to *factored loads*] only. For other angle sections or singly-symmetric unstiffened angles subject to *local buckling* at *stress* level F_y, $\overline{M}_y$ shall be taken either as the *required flexural strength* [moment due to *factored loads*] or the *required flexural strength* [moment due to *factored loads*] plus $(\overline{P})L/1000$, whichever results in a lower permissible value of $\overline{P}$.

$$\frac{\overline{P}}{P_a}+\frac{\overline{M}_x}{M_{ax}}+\frac{\overline{M}_y}{M_{ay}}\le 1.0 \qquad \text{(Eq. H1.2-1)}$$

where

$\overline{P}$ = *Required compressive axial strength* [compressive axial force due to *factored loads*] determined as required in Section C1, in accordance with *ASD*, *LRFD,* or *LSD load combinations*, taken as positive

P_a = *Available axial strength* [*factored resistance*], determined in accordance with Chapter E

$\overline{M}_x$, $\overline{M}_y$ = *Required flexural strengths* [moment due to *factored loads*], determined as required in Section C1, in accordance with *ASD*, *LRFD,* or *LSD load combinations*, taken as positive

M_{ax}, M_{ay} = *Available flexural strengths* [*factored resistances*] about centroidal axes, determined in accordance with Chapter F

H2 Combined Bending and Shear

For beams subjected to combined bending and shear, the *required flexural strength* [moment due to *factored loads*], $\overline{M}$, and the *required shear strength* [shear force due to *factored loads*], $\overline{V}$, shall not exceed M_a and V_a, respectively.

For beams without shear stiffeners as defined in Section G4, the *required flexural strength* [moment due to *factored loads*], $\overline{M}$, and the *required shear strength* [shear force due to *factored loads*], $\overline{V}$, shall also satisfy the following interaction equation:

$$\sqrt{\left(\frac{\overline{M}}{M_{a\ell o}}\right)^2+\left(\frac{\overline{V}}{V_a}\right)^2}\le 1.0 \qquad (\textit{Eq}.\ \text{H2-1})$$

For beams with shear stiffeners as defined in Section G4, when $\overline{M}/M_{a\ell o} > 0.5$ and $\overline{V}/V_a > 0.7$, $\overline{M}$ and $\overline{V}$ shall also satisfy the following interaction equation:

$$0.6\left(\frac{\overline{M}}{M_{a\ell o}}\right)+\left(\frac{\overline{V}}{V_a}\right)\le 1.3 \qquad (Eq.\ H2\text{-}2)$$

where

$\overline{M}$ = *Required flexural strength* [moment due to *factored loads*] in accordance with *ASD*, *LRFD*, or *LSD load combinations*

$\overline{V}$ = *Required shear strength* [shear force due to *factored loads*] in accordance with *ASD*, *LRFD* or *LSD load combinations*

M_a = *Available flexural strength* [*factored resistance*] when bending alone is considered, determined in accordance with Chapter F

V_a = *Available shear strength* [*factored resistance*] when shear alone is considered, determined in accordance with Sections G2 to G4

$M_{a\ell o}$ = *Available flexural strength* [*factored resistance*] for globally braced member determined as follows:

(a) For members without transverse *web* stiffeners, $M_{a\ell o}$ is determined in accordance with Section F3 with $F_n = F_y$ or $M_{ne} = M_y$, and

(b) For members with transverse *web* stiffeners, $M_{a\ell o}$ is the lesser of

(1) *Available strength* [*factored resistance*] determined in accordance with Section F3 with $F_n = F_y$ or $M_{ne} = M_y$, and

(2) *Available strength* [*factored resistance*] determined in accordance with Section F4.

F_n = Global *flexural buckling stress* as defined in Section F2

F_y = *Yield stress*

M_{ne} = *Nominal flexural strength* [*resistance*] considering *yielding* and global *buckling*, determined in accordance with Section F2

M_y = Member *yield moment* in accordance with Section F2.1

H3 Combined Bending and Web Crippling

Unreinforced flat *webs* of shapes subjected to a combination of bending and concentrated *load* or reaction shall be designed such that the moment, $\overline{M}$, and the concentrated *load* or reaction, $\overline{P}$, satisfy $\overline{M} \le M_{a\ell o}$ and $\overline{P} \le P_a$. In addition, the following requirements in (a), (b), and (c), as applicable, shall be satisfied.

(a) For shapes having single unreinforced *webs*, Eq. H3-1 shall be satisfied as follows:

$$0.91\left(\frac{\overline{P}}{P_n}\right)+\left(\frac{\overline{M}}{M_{n\ell o}}\right)\le \frac{1.33}{\Omega} \qquad (ASD) \qquad (Eq.\ H3\text{-}1a)$$

$$0.91\left(\frac{\overline{P}}{P_n}\right)+\left(\frac{\overline{M}}{M_{n\ell o}}\right)\le 1.33\phi \qquad (LRFD \text{ and } LSD) \qquad (Eq.\ H3\text{-}1b)$$

where

Ω = 1.70 (*ASD*)

ϕ = 0.90 (*LRFD*)

= 0.75 (*LSD*)

Exception: At the interior supports of continuous spans, Eq. H3-1 shall not apply to deck or beams with two or more single *webs*, provided the compression edges of adjacent *webs* are laterally supported in the negative moment region by continuous or intermittently connected *flange* elements, rigid cladding, or lateral bracing, and the spacing between adjacent *webs* does not exceed 10 in. (254 mm).

(b) For shapes having multiple unreinforced *webs* such as I-sections made of two C-sections connected back-to-back, or similar sections that provide a high degree of restraint against rotation of the *web* (such as I-sections made by welding two angles to a C-section), Eq. H3-2 shall be satisfied as follows:

$$0.88\left(\frac{\overline{P}}{P_n}\right)+\left(\frac{\overline{M}}{M_{n\ell o}}\right)\le\frac{1.46}{\Omega} \qquad (ASD) \qquad (Eq.\ H3\text{-}2a)$$

$$0.88\left(\frac{\overline{P}}{P_n}\right)+\left(\frac{\overline{M}}{M_{n\ell o}}\right)\le 1.46\phi \qquad (LRFD\ \text{and}\ LSD) \qquad (Eq.\ H3\text{-}2b)$$

where
Ω = 1.70 (*ASD*)
ϕ = 0.90 (*LRFD*)
= 0.75 (*LSD*)

(c) For two nested Z-shapes, Eq. H3-3 shall be satisfied as follows:

$$0.86\left(\frac{\overline{P}}{P_n}\right)+\left(\frac{\overline{M}}{M_{n\ell o}}\right)\le\frac{1.65}{\Omega} \qquad (ASD) \qquad (Eq.\ H3\text{-}3a)$$

$$0.86\left(\frac{\overline{P}}{P_n}\right)+\left(\frac{\overline{M}}{M_{n\ell o}}\right)\le 1.65\phi \qquad (LRFD\ \text{and}\ LSD) \qquad (Eq.\ H3\text{-}3b)$$

where
Ω = 1.70 (*ASD*)
ϕ = 0.90 (*LRFD*)
= 0.80 (*LSD*)

Eq. H3-3 shall apply to shapes that meet the following limits:
(1) $h/t \le 150$,
(2) $N/t \le 140$,
(3) $F_y \le$ 70 ksi (483 MPa or 4920 kg/cm^2), and
(4) $R/t \le 5.5$

where
h = Depth of flat portion of *web* measured along plane of *web*
t = *Web thickness*
N = Bearing length
F_y = *Yield stress*
R = Inside bend radius

The following conditions shall also be satisfied:

(i) The ends of each section are connected to the other section by a minimum of two 1/2 in. (12.7 mm) diameter A307 bolts through the *web*.

(ii) The combined section is connected to the support by a minimum of two 1/2 in. (12.7 mm) diameter A307 bolts through the *flanges*.

(iii) The *webs* of the two sections are in contact.

(iv) The ratio of the thicker to the thinner part does not exceed 1.3.

The following notations shall apply in this section:

$\overline{P}$ = *Required strength* [force due to *factored loads*] for concentrated *load* or reaction in presence of bending moment, determined in accordance with *ASD*, *LRFD,* or *LSD load combinations*

$\overline{M}$ = *Required flexural strength* [moment due to *factored loads*] at, or immediately adjacent to, the point of application of the concentrated *load* or reaction $\overline{P}$, determined in accordance with *ASD*, *LRFD,* or *LSD load combinations*

P_a = *Available strength* [*factored resistance*] for concentrated *load* or reaction in absence of bending moment, determined in accordance with Sections G5 and G6, as applicable

$M_{a\ell o}$ = *Available flexural strength* [*factored resistance*] about centroidal x-axis in absence of axial *load,* determined in accordance with Section F3 with $F_n = F_y$ or $M_{ne} = M_y$

$M_{n\ell o}$ = *Nominal flexural strength* [*resistance*] about centroidal x-axis in absence of axial *load,* determined in accordance with Section F3 with $F_n = F_y$ or $M_{ne} = M_y$

P_n = *Nominal strength* [*resistance*] for concentrated *load* or reaction in absence of bending moment, determined in accordance with Sections G5 and G6, as applicable

F_n = Global flexural *buckling stress* as defined in Section F2

M_{ne} = *Nominal flexural strength* [*resistance*] considering *yielding* and global *buckling*, determined in accordance with Section F2

M_y = Member *yield moment* in accordance with Section F2.1

H4 Combined Bending and Torsion

For members subjected to longitudinal *stresses* from both bending and warping torsion, the *required strengths* [effects due to *factored loads*], $\overline{M}_x$, $\overline{M}_y$ and $\overline{B}$, shall satisfy interaction equation Eq. H4-1, and each individual ratio in Eq. H4-1 shall not exceed unity.

> **User Note:**
> Torsional loads produce longitudinal warping *stresses* for most open sections, but these *stresses* are not significant for most closed sections. The *Commentary* to Section G8 provides further guidance.

$$\frac{\overline{M}_x}{M_{ax}} + \frac{\overline{M}_y}{M_{ay}} + \frac{\overline{B}}{B_a} \le 1.0 \qquad (Eq.\ H4\text{-}1)$$

where

$\overline{M}_x, \overline{M}_y$ = *Required flexural strengths* [moments due to *factored loads*], determined as required in Section C1, in accordance with *ASD*, *LRFD*, or *LSD load combinations*, taken as positive

M_{ax}, M_{ay} = *Available flexural strengths* [*factored resistances*] about centroidal axes determined in accordance with Chapter F. For combined bending and torsion at the location of a discrete lateral-torsional brace, strength reduction due to *lateral-torsional buckling* need not be considered.

$\overline{B}$ = *Required bimoment strength* [bimoment due to *factored loads*], determined as required in Section C1, in accordance with *ASD*, *LRFD*, or *LSD load combinations*, taken as positive

B_a = *Available bimoment strength* [*factored resistance*] determined in accordance with Section G8.1

F_y = *Yield stress*

M_y = Member *yield moment* in accordance with Section F2.1

For C-sections where the bimoment causes tension at the free edge of the flexural compression *flange*, and for Z-sections where the bimoment causes tension at the free edges of the *flanges*, the upper limit of Eq. H4-1 is permitted to be increased to 1.25. The provisions of this section shall not apply if the provisions of Section I6.2.1 or I6.2.2 are used.

I. ASSEMBLIES AND SYSTEMS

This chapter addresses design provisions related to cold-formed steel assemblies and systems. The chapter is organized as follows:

I1 Built-Up Sections
I2 Floor, Roof, or Wall Steel Diaphragm Construction
I3 Mixed Systems
I4 Cold-Formed Steel Light-Frame Construction
I5 Special Bolted Moment Frame Systems
I6 Metal Roof and Wall Systems
I7 Rack Systems

I1 Built-Up Sections

I1.1 Flexural Members Composed of Two Back-to-Back C-Sections

The maximum longitudinal spacing of *connections* (one or more welds or other connectors), s_{max}, joining two C-sections to form an I-section shall be:

$$s_{max} = L / 6 \text{ or } \frac{2gT_s}{mq}, \text{ whichever is smaller} \qquad (Eq.\ I1.1\text{-}1)$$

where

L = Span of beam

g = Vertical distance between two rows of *connections* nearest to top and bottom *flanges*

T_s = *Available strength* [*factored resistance*] of *connection* in tension (Chapter J)

m = Distance from shear center of one C-section to mid-plane of *web*

q = *Design load* [*factored load*] on beam for determining longitudinal spacing of *connections* (See below for methods of determination.)

The *load*, q, shall be obtained by dividing the concentrated *loads* or reactions by the length of bearing. For beams designed for a uniformly distributed *load*, q shall be taken as equal to three times the uniformly distributed *load*, based on the critical *load combinations* for *ASD*, *LRFD*, and *LSD*. If the length of bearing of a concentrated *load* or reaction is smaller than the longitudinal *connection* spacing, s, the *required strength* [force due to *factored loads*] of the *connections* closest to the *load* or reaction shall be calculated as follows:

$$T_r = P_s m/2g \qquad (Eq.\ I1.1\text{-}2)$$

where

P_s = Concentrated *load* [*factored load*] or reaction based on critical *load combinations* for *ASD*, *LRFD*, and *LSD*

T_r = *Required strength* [force due to *factored loads*] of *connection* in tension

The allowable maximum spacing of *connections*, s_{max}, shall depend upon the intensity of the *load* directly at the *connection*. Therefore, if uniform spacing of *connections* is used over the whole length of the beam, it shall be determined at the point of maximum local *load* intensity. In cases where this procedure would result in uneconomically close spacing, either one of the following methods is permitted to be adopted:

(a) The *connection* spacing varies along the beam according to the variation of the *load* intensity, or

(b) Reinforcing cover plates are welded to the *flanges* at points where concentrated *loads* occur. The *available shear strength* [*factored resistance*] of the *connections* joining these plates to the *flanges* is then used for T_s, and g is taken as the depth of the beam.

I1.2 Compression Members Composed of Multiple Cold-Formed Steel Members

I1.2.1 General Requirements

The *available compressive strength* [*factored resistance*] of a built-up member composed of multiple cold-formed steel members shall be determined as follows:

(a) If no interaction between individual members is considered for the strength of the built-up member, the *available strength* [*factored resistance*] of the built-up member is the summation of the *available strength* [*factored resistance*] of the individual members determined in accordance with Chapter E.

(b) Where composite action is considered, the *available strength* [*factored resistance*] of the built-up member is the smallest of the values calculated in accordance with Sections I1.2.2 to I1.2.4, as applicable.

I1.2.2 Yielding and Global Buckling

The *available axial strength* [*factored resistance*] for *yielding* and global (*flexural*, *torsional*, or *flexural-torsional*) *buckling* shall be determined in accordance with Section E2, with the *elastic buckling stress*, F_{cre}, determined in accordance with Section I1.2.2.1 or Section I1.2.2.2.

I1.2.2.1 Elastic Buckling – Prescriptive Requirements

The *elastic buckling stress*, F_{cre}, shall be determined in accordance with Appendix 2 Section 2.3.1.1. If the global *buckling* mode involves flexure that produces shear forces in the connectors between individual shapes, the moment of inertia, I, for the built-up member about the axis of *flexural buckling* shall be replaced by the reduced moment of inertia, I_r, in determining the global *buckling* force, P_{cre}, in Appendix 2, where I_r is given by:

$$I_r = I\frac{(KL/r)^2}{(KL/r)^2 + (a/r_i)^2} \qquad (Eq.\ I1.2.2.1\text{-}1)$$

where

I = Moment of inertia of built-up member about the axis of *flexural buckling*

KL = *Effective length* of built-up member for the axis of *flexural buckling*

r = Radius of gyration of *full unreduced cross-sectional area* of built-up member about the axis of *flexural buckling*

a = Spacing of intermediate fasteners or welds carrying shear between sections

r_i = Minimum radius of gyration of *full unreduced cross-sectional area* of an individual shape in a built-up member

The *connection* strength and spacing shall satisfy the following:

(a) The spacing of intermediate fasteners or welds, a, is limited such that a/r_i does not exceed one-half the governing slenderness ratio, KL/r, of the built-up member.

(b) The intermediate fastener(s) or weld(s) at any longitudinal member tie location are capable of transmitting the *required strength* [force due to *factored loads*] in any direction of 2.5 percent of the *available axial strength* [*factored resistance*] of the built-up member.

(c) The fasteners or welds at the ends of the member *effective length* are capable of transmitting the *required shear strength* [shear force due to *factored loads*], $\overline{P}_v$, between individual shapes determined as follows:

$$\overline{P}_v = \frac{M_{a\ell o}Q}{I_g} \qquad (Eq.\ I1.2.2.1\text{-}2)$$

$M_{a\ell o}$ = *Available flexural strength* [*factored resistance*] about axis of *buckling* for built-up member determined in accordance with Section F3 with $F_n = F_y$ or $M_{ne} = M_y$

Q = First moment of area of connected shape(s) about axis of *buckling* for the gross built-up cross-section

I_g = Moment of inertia about axis of *buckling* for the gross built-up cross-section

Exception: Where a built-up member comprised of two C-Sections oriented back-to-back forming an I-shaped cross-section is fully supported by a bearing surface with adequate strength and *stiffness* to preclude relative end slip of the two sections, these end *connection* provisions are not required.

I1.2.2.2 Elastic Buckling – Rational Analysis

The *elastic buckling stress*, F_{cre}, is permitted to be determined by *rational engineering analysis* in accordance with Appendix 2 Section 2.2 considering elastic member interaction. The *required strength* [shear force due to *factored loads*] of fasteners between members shall be determined from the analysis or in accordance with Section I1.2.2.1.

I1.2.3 Local Buckling Interacting With Yielding and Global Buckling

The *available axial strength* [*factored resistance*] for *local buckling* interacting with *yielding* and global *buckling* shall be calculated in accordance with Section E3, with P_{ne} determined in accordance with Section I1.2.2, and A_e or $P_{cr\ell}$ determined for the entire built-up section. Interaction between elements shall not be considered unless the elements are continuously connected.

I1.2.4 Distortional Buckling

The *available axial strength* [*factored resistance*] for *distortional buckling* shall be calculated in accordance with Section E4. Interaction between individual shapes shall not be considered unless fastener location and spacing are sufficient to increase the critical elastic *distortional buckling* load, P_{crd}, as determined by rational elastic analysis in accordance with Appendix 2 Section 2.2.

I1.3 Spacing of Connections in Cover-Plated Sections

To develop the strength required of the compression element, the spacing, s, in the line of *stress*, of welds, rivets, or bolts connecting a cover plate, sheet, or a non-integral stiffener in compression to another element shall not exceed (a), (b), and (c) as follows:

(a) That which is required to transmit the shear between the connected parts on the basis of the *available strength* [*factored resistance*] per *connection* specified elsewhere herein,

(b) $1.5t\sqrt{E/\alpha f_c}$ (*Eq*. I1.3-1)

where

t = *Thickness* of the cover plate or sheet

E = Modulus of elasticity of steel

f_c = Compressive *stress* in the cover plate or sheet based on *ASD*, *LRFD,* or *LSD load combinations*

α = Coefficient
= 1.67 for *ASD load combinations*, and
= 1.0 for *LRFD* or *LSD load combinations*

(c) Three times the *flat width*, w, of the narrowest unstiffened compression element tributary to the *connections*, but need not be less than $1.11t\sqrt{E/F_y}$ if $w/t < 0.50\sqrt{E/F_y}$, or $1.33t\sqrt{E/F_y}$ if $w/t \geq 0.50\sqrt{E/F_y}$, unless closer spacing is required by (a) or (b) above.

In the case of intermittent fillet welds parallel to the direction of *stress*, the spacing shall be taken as the clear distance between welds, plus 1/2 in. (12.7 mm). In all other cases, the spacing shall be taken as the center-to-center distance between *connections*.

Exception: The requirements of this section do not apply to cover sheets that act only as sheathing material and are not considered *load*-carrying elements.

When any of the limits in (a), (b), or (c) in this section are exceeded, the *effective width* shall be determined in accordance with Section 1.1.4.

I2 Floor, Roof, or Wall Steel Diaphragm Construction

The design of floor, roof or wall steel *diaphragms* constructed with *profiled steel panels* shall be in accordance with the requirements of AISI S310. ☜B

User Note:
AISI S310 determines the strength and *stiffness* of *profiled steel panels* and their *connections* in a *diaphragm* system, but does not address the other components in the system. The design of other *diaphragm* components is governed by the *applicable building code* and various *approved* design standards published by the American Iron and Steel Institute, the American Institute for Steel Construction, the Steel Deck Institute, and American Society of Agricultural and Biological Engineers. Further information on the design of *diaphragms* with *profiled steel panels* is available from the Steel Deck Institute (www.sdi.org).

I3 Mixed Systems

The design of members in mixed systems using cold-formed steel components in conjunction with other materials shall conform to this *Specification* and the applicable specification of the other material.

I3.1 Composite Design

The design of composite members is permitted by *rational engineering analysis* subject to approval by the *authority having jurisdiction*.

In determining the strength and *stiffness* of the shear *connections* and flexural *stiffness* of composite members, the following test standards are permitted to be used:

(1) ANSI/SDI AISI S923, *Test Standard for Determining the Strength and Stiffness of Shear Connection in Composite Members*

(2) ANSI/SDI AISI S924, *Test Standard for Determining the Effective Flexural Stiffness of Composite Members*

I4 Cold-Formed Steel Light-Frame Construction

The design, manufacture, installation, and quality of *structural members* and *connections* utilized in *cold-formed steel* light-frame construction applications shall be in accordance with the *applicable building code*. ☜B

User Note:
The design of *cold-formed steel* light-frame construction is governed by the *applicable building code* and various *approved* design standards published by the American Iron and Steel Institute (www.steel.org), specifically AISI S220, AISI S240, and AISI S400. Additional information on the design of *cold-formed steel* light-frame construction is available from the Cold-Formed Steel Engineers Institute (www.cfsei.org).

I5 Special Bolted Moment Frame Systems

The seismic design of special bolted moment frame systems shall be in accordance with the *applicable building code*.

User Note:
The seismic design of special bolted moment frame systems is governed by the *applicable building code* and an *approved* design standard published by the American Iron and Steel Institute (www.steel.org).

I6 Metal Roof and Wall Systems

The provisions of Sections I6.1 through I6.4 shall apply to metal roof and wall systems that include cold-formed steel members (*girts* and *purlins*), through-fastened wall or roof panels, or standing seam roof panels, as applicable. Members shall be designed in accordance with Section I6.1 or I6.2, as applicable; standing seam roof panel systems shall be designed in accordance with Section I6.3; roof system bracing and anchorage shall be designed in accordance with Section I6.4; and in-plane *diaphragm* shear strength and *stiffness* shall be designed in accordance with Section I2.

I6.1 Member Strength: General Cross-Sections and System Connectivity

I6.1.1 Compression Member Design

The *nominal axial strength* [*resistance*], P_n, shall be the minimum of P_{ne}, $P_{n\ell}$, and P_{nd} as given in Sections I6.1.1.1 to I6.1.1.3. For members meeting the geometric and material limits of Section B4, the *safety* and *resistance factors* shall be as follows:

Ω_c = 1.80 (*ASD*)
ϕ_c = 0.85 (*LRFD*)
= 0.80 (*LSD*)

For all other members, the *safety* and *resistance factors* in Section A1.2.6(c) shall apply. The *available strength* [*factored resistance*] shall be determined in accordance with the applicable method in Section B3.2.1, B3.2.2 or B3.2.3.

I6.1.1.1 Flexural, Torsional, or Flexural-Torsional Buckling

The *nominal compressive strength* [*resistance*], P_{ne}, for *flexural*, *torsional*, or *flexural-torsional buckling* shall be calculated in accordance with Section E2, except F_{cre} or P_{cre} shall be determined including lateral, rotational, and composite *stiffness* provided by the deck or sheathing, bridging and bracing, and *span continuity*.

I6.1.1.2 Local Buckling

The *nominal compressive strength* [*resistance*], $P_{n\ell}$, for *local buckling* shall be calculated in accordance with Section E3, except F_n or $P_{cr\ell}$ shall be determined including lateral, rotational, and composite *stiffness* provided by the deck or sheathing.

I6.1.1.3 Distortional Buckling

The *nominal compressive strength* [*resistance*], P_{nd}, for *distortional buckling* shall be calculated in accordance with Section E4, except P_{crd} shall be determined including lateral, rotational, and composite *stiffness* provided by the deck or sheathing.

I6.1.2 Flexural Member Design

The *nominal flexural strength* [*resistance*], M_n, shall be the minimum of M_{ne}, $M_{n\ell}$, and M_{nd} as given in Sections I6.1.2.1 to I6.1.2.3. For members meeting the geometric and material limits of Section B4, the *safety* and *resistance factors* shall be as follows:

Ω_b = 1.67 (*ASD*)
ϕ_b = 0.90 (*LRFD*)
= 0.85 (*LSD*)

For all other members, the *safety* and *resistance factors* in Section A1.2.6(c) shall apply. The *available strength* [*factored resistance*] shall be determined in accordance with the applicable method in Section B3.2.1, B3.2.2 or B3.2.3.

I6.1.2.1 Lateral-Torsional Buckling

The *nominal flexural strength* [*resistance*], M_{ne}, for *lateral-torsional buckling* shall be calculated in accordance with Section F2, except F_{cre} or M_{cre} shall be determined including lateral, rotational, and composite *stiffness* provided by the deck or sheathing, bridging and bracing, and *span continuity*.

I6.1.2.2 Local Buckling

The *nominal flexural strength* [*resistance*], $M_{n\ell}$, for *local buckling* shall be calculated in accordance with Section F3, except F_n or $M_{cr\ell}$ shall be determined including lateral, rotational, and composite *stiffness* provided by the deck or sheathing.

I6.1.2.3 Distortional Buckling

The *nominal flexural strength* [*resistance*], M_{nd}, for *distortional buckling* of *girts* and *purlins* shall be calculated in accordance with Section F4, except M_{crd} shall be determined including lateral, rotational, and composite *stiffness* provided by the deck or sheathing.

I6.1.3 Member Design for Combined Bending and Torsion

Members subjected to combined bending and torsion shall satisfy the requirements of Section H4, where $\overline{B}$ shall be determined considering the rotational *stiffness* provided by the deck or sheathing.

I6.2 Member Strength: Specific Cross-Sections and System Connectivity

I6.2.1 Flexural Members Having One Flange Through-Fastened to Deck or Sheathing

This section shall not apply to a continuous beam for the region between inflection points adjacent to a support or to a cantilever beam.

The *nominal flexural strength* [*resistance*], M_n, of a C- or Z-section loaded in a plane parallel to the *web*, with the tension *flange* attached to deck or sheathing and with the compression *flange* laterally unbraced, shall be calculated in accordance with Eq. I6.2.1-1. Consideration of *distortional buckling* in accordance with Section F4 shall be excluded. The *safety factor* and *resistance factors* given in this section shall be used to determine the *allowable flexural strength* or *design flexural strength* [*factored resistance*] in accordance with the applicable design method in Section B3.2.1, B3.2.2, or B3.2.3.

$$M_n = R\ M_{n\ell o} \qquad (Eq.\ I6.2.1\text{-}1)$$

Ω_b = 1.67 (*ASD*)

ϕ_b = 0.90 (*LRFD*)

= 0.90 (*LSD*)

where

R = A value obtained from Table I6.2.1-1 for C- or Z-sections

$M_{n\ell o}$ = *Nominal flexural strength* with consideration of *local buckling* only, as determined from Section F3 with $F_n = F_y$ or $M_{ne} = M_y$

TABLE I6.2.1-1
C- or Z-Section R Values

Simple Span		
Member Depth Range, in. (mm)	Profile	R
d ≤ 6.5 (165)	C or Z	0.70
6.5 (165) < d ≤ 8.5 (216)	C or Z	0.65
8.5 (216) < d ≤ 12 (305)	Z	0.50
8.5 (216) < d ≤ 12 (305)	C	0.40
Continuous Span		
Profile	R	
C	0.60	
Z	0.70	

The reduction factor, R, shall be limited to roof and wall systems meeting the following conditions:

(a) Member depth ≤ 12 in. (305 mm),

(b) Member *flanges* with edge stiffeners,

(c) 60 ≤ depth/*thickness* ≤ 170,

(d) 2.8 ≤ depth/*flange* width ≤ 5.5,

(e) *Flange* width ≥ 2.125 in. (54.0 mm),

(f) 16 ≤ *flat width*/*thickness* of *flange* ≤ 43,

(g) For continuous span systems, the lap length at each interior support in each direction (distance from center of support to end of lap) is not less than 1.5d,

(h) Member span length is not greater than 33 feet (10 m),

(i) Both *flanges* are prevented from moving laterally at the supports,

(j) Roof or wall panels are steel sheets with 50 ksi (340 MPa or 3520 kg/cm^2) minimum *yield stress*, and a minimum of 0.018 in. (0.46 mm) base metal *thickness*, having a minimum rib depth of 1-1/8 in. (29 mm), spaced at a maximum of 12 in. (305 mm) on centers and attached in a manner to effectively inhibit relative movement between the panel and member *flange*,

(k) Insulation is glass fiber blanket 0 to 6 in. (152 mm) thick, compressed between the member and panel in a manner consistent with the fastener being used,

(l) Fastener type is, at minimum, No. 12 self-drilling or self-tapping sheet metal screws or 3/16 in. (4.76 mm) rivets, having washers with 1/2 in. (12.7 mm) diameter,

(m) Fasteners are not standoff type screws,

(n) Fasteners are spaced not greater than 12 in. (305 mm) on centers and placed near the center of the member *flange*, and adjacent to the panel high rib, and

(o) The ratio of *tensile strength* to design *yield stress* shall not be less than 1.08.

If variables fall outside any of the above-stated limits, the user shall perform full-scale tests in accordance with Section K2.1 of this *Specification* or apply a *rational engineering analysis* procedure. For continuous *purlin* and *girt* systems in which adjacent bay span lengths vary by more than 20 percent, the R values for the adjacent bays shall be taken from the simple-span values in Table I6.2.1-1. The user is permitted to perform tests in accordance with Section K2.1 as an alternative to the procedure described in this section.

For simple-span members, R shall be reduced for the effects of compressed insulation between the sheeting and the member. The reduction shall be calculated by multiplying R from Table I6.2.1-1 by the following correction factor, r:

$r = 1.00 - 0.01\, t_i$ when t_i is in inches (*Eq*. I6.2.1-2)

$r = 1.00 - 0.0004\, t_i$ when t_i is in millimeters (*Eq*. I6.2.1-3)

where

t_i = Thickness of uncompressed glass fiber blanket insulation

I6.2.2 Flexural Members Having One Flange Fastened to a Standing Seam Roof System

See Section I6.2.2 of Appendix A or B for the provisions of this section. ☜A,B

I6.2.3 Compression Members Having One Flange Through-Fastened to Deck or Sheathing

These provisions shall apply to C- or Z-sections concentrically loaded along their longitudinal axis, with only one *flange* attached to deck or sheathing with through fasteners.

The *nominal axial strength* [*resistance*] of simple span or continuous C- or Z-sections shall be calculated in accordance with (a) and (b). Consideration of *distortional buckling* in accordance with Section E4 shall be excluded.

(a) The weak axis *nominal strength* [*resistance*], P_n, shall be calculated in accordance with Eq. I6.2.3-1. The *safety factor* and *resistance factors* given in this section shall be used to determine the *allowable axial strength* or *design axial strength* [*factored resistance*] in accordance with the applicable design method in Section B3.2.1, B3.2.2, or B3.2.3.

$$P_n = C_1C_2C_3AE/29500 \qquad (Eq.\ I6.2.3\text{-}1)$$

Ω = 1.80 (*ASD*)
ϕ = 0.85 (*LRFD*)
= 0.80 (*LSD*)

where

$$C_1 = (0.79x + 0.54) \qquad (Eq.\ I6.2.3\text{-}2)$$

$$C_2 = (1.17\alpha t + 0.93) \qquad (Eq.\ I6.2.3\text{-}3)$$

$$C_3 = \alpha(2.5b - 1.63d) + 22.8 \qquad (Eq.\ I6.2.3\text{-}4)$$

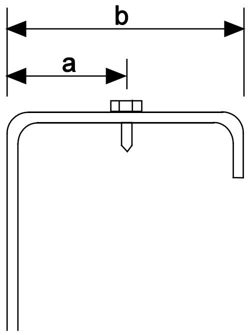
where

x = For Z-sections, fastener distance from outside *web* edge divided by *flange* width, as shown in Figure I6.2.3-1
= For C-sections, *flange* width minus fastener distance from outside *web* edge divided by *flange* width, as shown in Figure I6.2.3-1

α = Coefficient for conversion of units
= 1 when t, b, and d are in inches
= 0.0394 when t, b, and d are in mm
= 0.394 when t, b, and d are in cm

t = C- or Z-section *thickness*
b = C- or Z-section *flange* width
d = C- or Z-section depth

A = *Full unreduced cross-sectional area* of C- or Z-section
E = Modulus of elasticity of steel
= 29,500 ksi for U.S. customary units
= 203,000 MPa for SI units
= 2,070,000 kg/cm^2 for MKS units



For Z-section, $x = \frac{a}{b}$ (*Eq*. I6.2.3-5)

For C-section, $x = \frac{b-a}{b}$ (*Eq*. I6.2.3-6)

Figure I6.2.3-1 Definition of x

Eq. I6.2.3-1 shall be limited to roof and wall systems meeting the following conditions:

(1) $t \le 0.125$ in. (3.22 mm),
(2) 6 in. (152mm) $\le d \le$ 12 in. (305 mm),
(3) *Flanges* are edge-stiffened compression elements,
(4) $70 \le d/t \le 170$,
(5) $2.8 \le d/b \le 5$,
(6) $16 \le$ *flange flat width* / $t \le 50$,
(7) Both *flanges* are prevented from moving laterally at the supports,
(8) Steel roof or steel wall panels with fasteners spaced 12 in. (305 mm) on center or less and having a minimum rotational lateral *stiffness* of 0.0015 k/in./in. (10,300 N/m/m or 0.105 kg/cm/cm) (fastener at mid-flange width for *stiffness* determination) determined in accordance with ANSI/SDI AISI S901,
(9) C- and Z-sections having a minimum *yield stress* of 33 ksi (228 MPa or 2320 kg/cm^2), and
(10) Span length not exceeding 33 feet (10.1 m).

(b) The strong axis *available strength* [*factored resistance*] shall be determined in accordance with Sections E2 and E3.

I6.2.4 Z-Section Compression Members Having One Flange Fastened to a Standing Seam Roof

The provisions of this section shall apply only to the United States and Mexico. See Section I6.2.4 of Appendix A.

☜A

I6.3 Standing Seam Roof Panel Systems

I6.3.1 Strength of Standing Seam Roof Panel Systems

Under gravity loading, the *nominal strength* [*resistance*] of standing seam roof panels shall be determined in accordance with Chapter F of this *Specification* or shall be tested in accordance with ANSI/SDI AISI S906. Under uplift loading, the *nominal strength* [*resistance*] of standing seam roof panel systems shall be determined in accordance with ANSI/SDI AISI S906. Tests shall be performed in accordance with ANSI/SDI AISI S906 with the following exceptions:

(a) The Uplift Pressure Test Procedure for Class 1 Panel Roofs in FM 4471 is permitted.
(b) Existing tests conducted in accordance with CEGS 07416 Uplift Test Procedure prior to

the adoption of these provisions are permitted.

The open-open end configuration, although not prescribed by the ASTM E1592 Test Procedure, is permitted provided the tested end conditions represent the installed condition, and the test follows the requirements given in ANSI/SDI AISI S906. All test results shall be evaluated in accordance with this section.

For *load* combinations that include wind uplift, additional provisions are provided in Section I6.3.1a of Appendix A. ☜A

When the number of physical test assemblies is three (3) or more, *safety factor* and *resistance factors* shall be determined in accordance with the procedures of Section K2.1.1(c) with the following definitions for the variables:

β_o = Target reliability index
= 2.0 for USA and Mexico and 2.5 for Canada for panel flexural limits
= 2.5 for USA and Mexico and 3.0 for Canada for anchor limits

F_m = Mean value of the fabrication factor
= 1.0

M_m = Mean value of the material factor
= 1.1

V_M = Coefficient of variation of the material factor
= 0.08 for anchor failure mode
= 0.10 for other failure modes

V_F = Coefficient of variation of the fabrication factor
= 0.05

V_Q = Coefficient of variation of the *load effect*
= 0.21

V_P = Actual calculated coefficient of variation of the test results, without limit

n = Number of anchors in the test assembly with the same tributary area (for anchor failure) or number of panels with identical spans and loading to the failed span (for non-anchor failures)

The *safety factor*, Ω, shall not be less than 1.67, and the *resistance factor*, ϕ, shall not be greater than 0.9 (*LRFD* and *LSD*).

When the number of physical test assemblies is less than three (3), a *safety factor*, Ω, of 2.0 and a *resistance factor*, ϕ, of 0.8 (*LRFD*) and 0.70 (*LSD*) shall be used.

I6.4 Roof System Bracing and Anchorage

I6.4.1 Anchorage of Bracing for Purlin Roof Systems Under Gravity Load With Top Flange Connected to Metal Sheathing

Anchorage, in the form of a device capable of transferring force from the roof *diaphragm* to a support, shall be provided for roof systems with C-sections or Z-sections, designed in accordance with Chapter F, Section I6.1 or I6.2, having through-fastened or standing seam sheathing attached to the top *flanges*. Each anchorage device shall be designed to resist the force, P_L, determined by Eq. I6.4.1-1 and shall satisfy the minimum *stiffness* requirement of Eq. I6.4.1-7. In addition, *purlins* shall be restrained laterally by the sheathing so that the maximum top *flange* lateral displacements between lines of lateral anchorage resulting from

ASD load combinations (*specified loads* for *LSD*) do not exceed the span length divided by 360.

Anchorage devices shall be located in each *purlin* bay and shall connect to the *purlin* at or near the *purlin* top *flange*. If anchorage devices are not directly connected to all *purlin* lines of each *purlin* bay, provision shall be made to transmit the forces from other *purlin* lines to the anchorage devices. It shall be demonstrated that the required force, P_L, can be transferred to the anchorage device through the roof sheathing and its fastening system. The lateral *stiffness* of the anchorage device shall be determined by analysis or testing. This analysis or testing shall account for the flexibility of the *purlin web* above the attachment of the anchorage device *connection*.

$$P_{L_j} = \sum_{i=1}^{N_p} \left(P_i \frac{K_{eff_{i,j}}}{K_{total_i}} \right) \quad (Eq.\ I6.4.1\text{-}1)$$

where

P_{L_j} = Lateral force to be resisted by the j^{th} anchorage device (positive when restraint is required to prevent *purlins* from translating in the upward roof slope direction)

N_p = Number of *purlin* lines on roof slope

i = Index for each *purlin* line ($i=1, 2, \ldots, N_p$)

j = Index for each anchorage device ($j=1,2, \ldots, N_a$)

N_a = Number of anchorage devices along a line of anchorage

P_i = Lateral force introduced into the system at the i^{th} *purlin*

$$= (C1)W_{p_i} \left\{ \left[\left(\frac{C2}{1000} \right) \frac{I_{xy}L}{I_x d} + (C3) \frac{(m + 0.25b)t}{d^2} \right] \alpha \cos\theta - (C4)\sin\theta \right\} \quad (Eq.\ I6.4.1\text{-}2)$$

where

C1, C2, C3, and C4 = Coefficients tabulated in Tables I6.4.1-1 to I6.4.1-3

W_{p_i} = Total required vertical *load* supported by the i^{th} *purlin* in a single bay

$$= w_i L \quad (Eq.\ I6.4.1\text{-}3)$$

where

w_i = Required distributed gravity *load* supported by the i^{th} *purlin* per unit length (determined from the critical *ASD*, *LRFD*, or *LSD load combination* depending on the design method used)

I_{xy} = Product of inertia of full unreduced section about centroidal axes parallel and perpendicular to the *purlin web* (I_{xy} = 0 for C-sections)

L = *Purlin* span length

m = Distance from shear center to mid-plane of *web* (m = 0 for Z-sections)

b = Top *flange* width of *purlin*

t = *Purlin thickness*

I_x = Moment of inertia of full unreduced section about centroidal axis perpendicular to the *purlin web*

d = Depth of *purlin*

α = +1 for top *flange* facing in the up-slope direction
−1 for top *flange* facing in the down-slope direction

θ = Angle between vertical and plane of *purlin web*

$K_{eff_{i,j}}$ = Effective lateral *stiffness* of the j^{th} anchorage device with respect to the i^{th} *purlin*

$$= \left[\frac{1}{K_a} + \frac{d_{p_{i,j}}}{(C6)LA_pE}\right]^{-1} \qquad (Eq.\ I6.4.1\text{-}4)$$

where

$d_{p_{i,j}}$ = Distance along roof slope between the i^{th} *purlin* line and the j^{th} anchorage device

K_a = Lateral *stiffness* of the anchorage device

C6 = Coefficient tabulated in Tables I6.4.1-1 to I6.4.1-3

A_p = *Gross cross-sectional area* of roof panel per unit width

E = Modulus of elasticity of steel

K_{total_i} = Effective lateral *stiffness* of all elements resisting force P_i

$$= \sum_{j=1}^{N_a}\left(K_{eff_{i,j}}\right) + K_{sys} \qquad (Eq.\ I6.4.1\text{-}5)$$

where

K_{sys} = Lateral *stiffness* of the roof system, neglecting anchorage devices

$$= \left(\frac{C5}{1000}\right)(N_p)\frac{ELt^2}{d^2} \qquad (Eq.\ I6.4.1\text{-}6)$$

where

C5 = Coefficient tabulated in Tables I6.4.1-1 to I6.4.1-3

For multi-span systems, force P_i, calculated in accordance with Eq. I6.4.1-2 and coefficients C1 to C4 from Tables I6.4.1-1 to I6.4.1-3 for the "Exterior Frame Line," "End Bay," or "End Bay Exterior Anchor" cases, shall not be taken as less than 80 percent of the force determined using the coefficients C2 to C4 for the corresponding "All Other Locations" case.

For systems with multiple spans and anchorage devices at supports (support restraints), where the two adjacent bays have different section properties or span lengths, the following procedures are to be used:

(a) The values for P_i in Eq. I6.4.1-1 and Eq. I6.4.1-8 shall be taken as the average of the values found from Eq. I6.4.1-2 evaluated separately for each of the two bays, and

(b) The values of K_{sys} used in Eq. I6.4.1-5 and $K_{eff_{i,j}}$ in Eq. I6.4.1-1 and Eq. I6.4.1-5 shall be calculated using Eq. I6.4.1-4 and Eq. I6.4.1-6, with L, t, and d taken as the average of the values of the two bays.

For systems with multiple spans and anchorage devices at either 1/3 points or mid-points, where the adjacent bays have different section properties or span lengths than the bay under consideration, the following procedures are to be used to account for the influence of the adjacent bays:

(a) The values for P_i in Eq. I6.4.1-1 and Eq. I6.4.1-8 shall be taken as the average of the values found from Eq. I6.4.1-2 evaluated separately for each of the three bays,

(b) The value of K_{sys} in Eq. I6.4.1-5 shall be calculated using Eq. I6.4.1-6, with L, t, and d taken as the average of the values from the three bays,

(c) The values of $K_{eff_{i,j}}$ shall be calculated using Eq. I6.4.1-4, with L taken as the span length of the bay under consideration, and

(d) At an end bay, when computing the average values for P_i or averaging the properties for computing K_{sys}, the averages shall be found by adding the value from the first interior bay and two times the value from the end bay and then dividing the sum by three.

The total effective *stiffness* at each *purlin* shall satisfy the following equation:

$$K_{total\ i} \geq K_{req} \qquad (Eq.\ I6.4.1\text{-}7)$$

where

$$K_{req} = \Omega \frac{20\left|\sum_{i=1}^{N_p} P_i\right|}{d} \ (ASD) \qquad (Eq.\ I6.4.1\text{-}8a)$$

$$K_{req} = \frac{1}{\phi}\frac{20\left|\sum_{i=1}^{N_p} P_i\right|}{d} \ (LRFD, LSD) \qquad (Eq.\ I6.4.1\text{-}8b)$$

Ω = 2.00 (*ASD*)
ϕ = 0.75 (*LRFD*)
= 0.70 (*LSD*)

In lieu of Eqs. I6.4.1-1 through I6.4.1-6, lateral restraint forces are permitted to be determined from alternative analysis. Alternative analysis shall include the first- or second-order effect and account for the effects of roof slope, torsion resulting from applied *loads* eccentric to shear center, torsion resulting from the lateral resistance provided by the sheathing, and *load* applied oblique to the principal axes. Alternative analysis shall also include the effects of the lateral and rotational restraint provided by sheathing attached to the top *flange*. *Stiffness* of the anchorage device shall be considered and shall account for flexibility of the *purlin web* above the attachment of the anchorage device *connection*.

When lateral restraint forces are determined from *rational engineering analysis*, the maximum top *flange* lateral displacement of the *purlin* between lines of lateral bracing resulting from *ASD load combinations* (*specified loads* for *LSD*) shall not exceed the span length divided by 360. The lateral displacement of the *purlin* top *flange* at the line of restraint, Δ_{tf}, shall satisfy Eq. I6.4.1-9a for *ASD load combinations* and Eq. I6.4.1-9b for *LRFD* or *LSD load combinations*:

$$\Delta_{tf} \leq \frac{1}{\Omega}\frac{d}{20} \ (ASD) \qquad (Eq.\ I6.4.1\text{-}9a)$$

$$\Delta_{tf} \leq \phi\frac{d}{20} \ (LRFD, LSD) \qquad (Eq.\ I6.4.1\text{-}9b)$$

Table I6.4.1-1
Coefficients for Support Restraints

			C1	C2	C3	C4	C5	C6
Simple Span	Through Fastened (TF)		0.5	8.2	33	0.99	0.43	0.17
	Standing Seam (SS)		0.5	8.3	28	0.61	0.29	0.051
Multiple Spans	TF	Exterior Frame Line	0.5	14	6.9	0.94	0.073	0.085
		First Interior Frame Line	1.0	4.2	18	0.99	2.5	0.43
		All Other Locations	1.0	6.8	23	0.99	1.8	0.36
	SS	Exterior Frame Line	0.5	13	11	0.35	2.4	0.25
		First Interior Frame Line	1.0	1.7	69	0.77	1.6	0.13
		All Other Locations	1.0	4.3	55	0.71	1.4	0.17

Table I6.4.1-2
Coefficients for Mid-Point Restraints

			C1	C2	C3	C4	C5	C6
Simple Span	Through Fastened (TF)		1.0	7.6	44	0.96	0.75	0.42
	Standing Seam (SS)		1.0	7.5	15	0.62	0.35	0.18
Multiple Spans	TF	End Bay	1.0	8.3	47	0.95	3.1	0.33
		First Interior Bay	1.0	3.6	53	0.92	3.9	0.36
		All Other Locations	1.0	5.4	46	0.93	3.1	0.31
	SS	End Bay	1.0	7.9	19	0.54	2.0	0.080
		First Interior Bay	1.0	2.5	41	0.47	2.6	0.13
		All Other Locations	1.0	4.1	31	0.46	2.7	0.15

Table I6.4.1-3
Coefficients for One-Third Point Restraints

			C1	C2	C3	C4	C5	C6
Simple Span	Through Fastened (TF)		0.5	7.8	42	0.98	0.39	0.40
	Standing Seam (SS)		0.5	7.3	21	0.73	0.19	0.18
Multiple Spans	TF	End Bay Exterior Anchor	0.5	15	17	0.98	0.72	0.043
		End Bay Int. Anchor and 1st Int. Bay Ext. Anchor	0.5	2.4	50	0.96	0.82	0.20
		All Other Locations	0.5	6.1	41	0.96	0.69	0.12
	SS	End Bay Exterior Anchor	0.5	13	13	0.72	0.59	0.035
		End Bay Int. Anchor and 1st Int. Bay Ext. Anchor	0.5	0.84	56	0.64	0.20	0.14
		All Other Locations	0.5	3.8	45	0.65	0.10	0.014

I6.4.2 Alternate Lateral and Stability Bracing for Purlin Roof Systems

Torsional bracing that prevents twist about the longitudinal axis of a member in combination with lateral restraints that resist lateral displacement of the top *flange* at the frame line is permitted in lieu of the requirements of Section I6.4.1. A torsional brace shall prevent torsional rotation of the cross-section at a discrete location along the span of the member. *Connection* of braces shall be made at or near both *flanges* of ordinary open sections, including C- and Z-sections. The effectiveness of torsional braces in preventing torsional rotation of the cross-section and the *required strength* [brace force due to *factored loads*] of lateral restraints at the frame line shall be determined by *rational engineering analysis* or testing. The lateral displacement of the top *flange* of the C- or Z-section at the frame line shall be limited to $d/(20\Omega)$ for *ASD load combinations* or $\phi d/20$ for *LRFD* and *LSD load combinations*, where d is the depth of the C- or Z-section member, Ω is the *safety factor* for *ASD*, and ϕ is the *resistance factor* for *LRFD* and *LSD*. Lateral displacement between frame lines resulting from *ASD load combinations* (*specified loads* for *LSD*) shall be limited to L/180, where L is the span length of the member. For pairs of adjacent *purlins* that provide bracing against twist to each other, external anchorage of torsional brace forces shall not be required.

where
Ω = 2.0 (*ASD*)
ϕ = 0.75 (*LRFD*)
= 0.70 (*LSD*)

I7 Storage Rack Systems

Steel storage rack systems shall be designed and constructed in accordance with the *applicable building code*.

User Note:
The design of steel storage rack systems is governed by the *applicable building code* and various *approved* design standards published by the Rack Manufacturers Institute (www.mhi.org/rmi).

J. CONNECTIONS AND JOINTS

This chapter addresses cold-formed steel-to-steel welded, bolted, screw, and *power-actuated fastener connections*, as well as *connections* of *cold-formed steel structural members* to other materials.

This chapter is organized as follows:

J1 General Provisions
J2 Welded Connections
J3 Bolted Connections
J4 Screw Connections
J5 Power-Actuated Fastener (PAF) Connections
J6 Rupture
J7 Connections to Other Materials

J1 General Provisions

Connections shall be designed to transmit the *required strength* [force due to *factored loads*] acting on the connected members based on the *available strength* [*factored resistance*] of the *connection* (welded, bolt, screw, power actuated fastener or other bearing type *connection*) determined in accordance with J2, J3, J4, or J5 and as limited by J6, with consideration of eccentricity where applicable.

J2 Welded Connections

The design of welded *connections* used for *cold-formed steel structural members* in which the *thickness* of the thinnest connected part is 0.19 in. (4.83 mm) or less shall be based on the following subsections. Additionally, the following specifications or standards shall apply:

For the United States and Mexico:

(a) AWS D1.3, and
(b) AWS C1.1 or AWS C1.3 for resistance welds.

For Canada:

(a) CSA W59, and
(b) CSA W55.3 for resistance welds.

Where the steel is to be welded, a welding procedure suitable for the grade of steel and intended use or service shall be utilized.

For the design of welded *connections* in which the *thickness* of the thinnest connected part is greater than 0.19 in. (4.83 mm), the following specifications or standards shall apply:

(a) ANSI/AISC 360 for the United States and Mexico, and
(b) CSA S16 for Canada.

For *diaphragm* applications, Section I2 shall apply.

See Appendix A or B for additional requirements. ☜A,B

J2.1 Groove Welds in Butt Joints

The *nominal strength* [*resistance*], P_n, of a groove weld in a butt *joint*, welded from one or both sides, shall be determined in accordance with (a) or (b), as applicable. The corresponding *safety factor* and *resistance factors* shall be used to determine the *available strength* [*factored resistance*] in accordance with the applicable design method in Section B3.2.1, B3.2.2, or B3.2.3.

(a) For tension or compression normal to the effective area, the *nominal strength* [*resistance*], P_n, shall be calculated in accordance with Eq. J2.1-1:

$$P_n = Lt_eF_y \qquad (Eq.\ J2.1\text{-}1)$$

Ω = 1.70 (*ASD*)

ϕ = 0.90 (*LRFD*)

= 0.80 (*LSD*)

(b) For shear on the effective area, the *nominal strength* [*resistance*], P_n, shall be the smaller value calculated in accordance with Eqs. J2.1-2 and J2.1-3:

$$P_n = Lt_e\, 0.6F_{xx} \qquad (Eq.\ J2.1\text{-}2)$$

Ω = 1.90 (*ASD*)

ϕ = 0.80 (*LRFD*)

= 0.70 (*LSD*)

$$P_n = Lt_eF_y / \sqrt{3} \qquad (Eq.\ J2.1\text{-}3)$$

Ω = 1.70 (*ASD*)

ϕ = 0.90 (*LRFD*)

= 0.80 (*LSD*)

where

P_n = *Nominal strength* [*resistance*] of groove weld

L = Length of weld

t_e = Effective throat dimension of groove weld

F_y = *Yield stress* of lowest strength base steel

F_{xx} = *Tensile strength* of electrode classification

J2.2 Arc Spot Welds

Arc spot welds, where permitted by this *Specification*, shall be for welding sheet steel to thicker supporting members or sheet to sheet in the flat position. Arc spot welds (puddle welds) shall not be made on steel where the thinnest sheet exceeds 0.19 in. (4.83 mm) in *thickness*, nor through a combination of steel sheets having a total *thickness* over 0.19 in. (4.83 mm).

Weld washers, as shown in Figures J2.2-1 and J2.2-2, shall be used where the *thickness* of the sheet is less than 0.028 in. (0.711 mm). Weld washers shall have a *thickness* between 0.05 in. (1.27 mm) and 0.08 in. (2.03 mm), with a minimum pre-punched hole of 3/8 in. (9.53 mm) in diameter. Sheet-to-sheet welds shall not require weld washers.

Arc spot welds shall be specified by a minimum effective diameter of fused area, d_e. The minimum allowable effective diameter shall be 3/8 in. (9.53 mm).

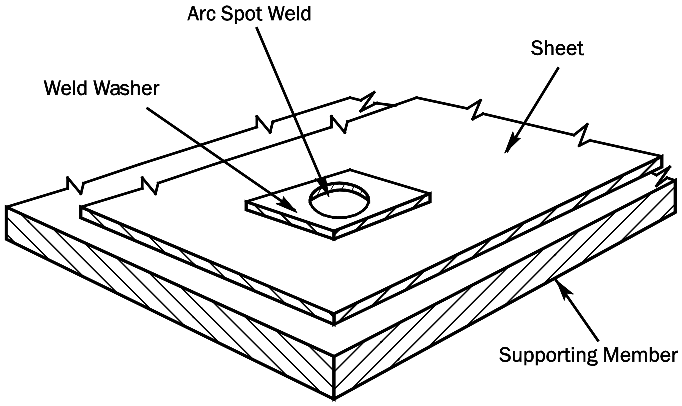


Figure J2.2-1 Typical Weld Washer

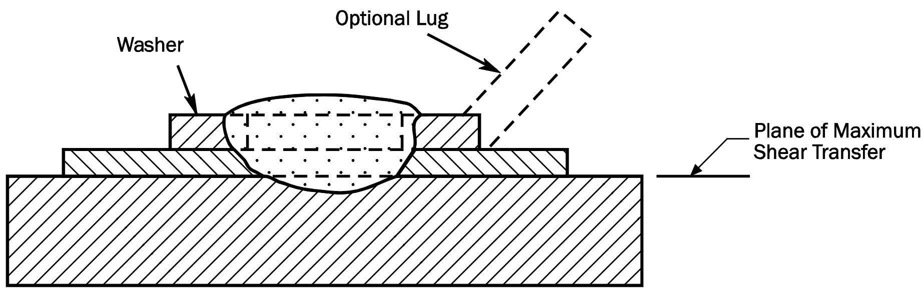


Figure J2.2-2 Arc Spot Weld Using Washer

J2.2.1 Minimum Edge and End Distance

The distance from the centerline of an arc spot weld to the end or edge of the connected member shall not be less than 1.5d and shall satisfy the requirements of Section J6. In no case shall the clear distance between welds and the end or edge of the member be less than 1.0d, where d is the visible diameter of the outer surface of the arc spot weld. See Figures J2.2.1-1 and J2.2.1-2 for details.

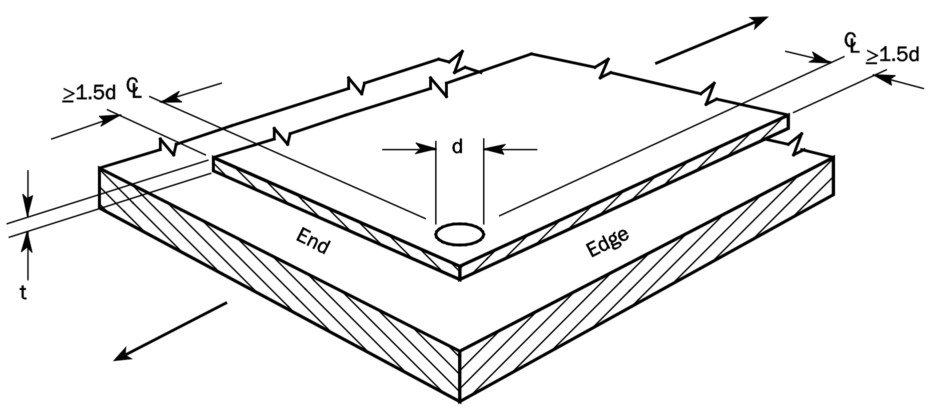


Figure J2.2.1-1 End and Edge Distance for Arc Spot Welds – Single Sheet

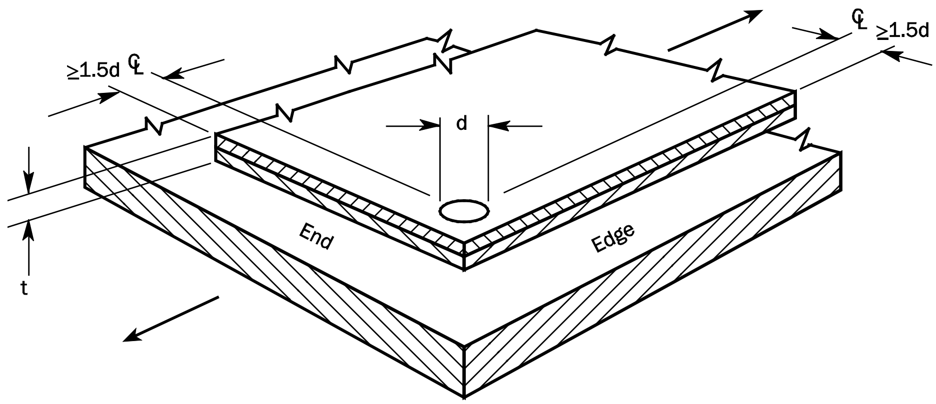


Figure J2.2.1-2 End and Edge Distance for Arc Spot Welds – Multiple Sheets

J2.2.2 Shear

J2.2.2.1 Shear Strength for Sheet(s) Welded to a Thicker Supporting Member

The *available shear strength* [*factored resistance*], P_{av}, of each arc spot weld between the sheet or sheets and a thicker supporting member shall be the smaller value which is computed with the *nominal shear strength* [*resistance*], P_{nv}, determined by using the smaller of (a) and (b), and the corresponding *safety factor* and *resistance factors* applied in accordance with the applicable design method in Section B3.2.1, B3.2.2, or B3.2.3.

(a) $$P_{nv} = \frac{\pi d_e^2}{4} 0.75 F_{xx} \qquad (Eq.\ J2.2.2.1\text{-}1)$$

Ω = 2.60 (*ASD*)
ϕ = 0.60 (*LRFD*)
= 0.50 (*LSD*)

(b) For $(d_a/t) \le 0.815\sqrt{E/F_u}$

$$P_{nv} = 2.20\, t d_a F_u \qquad (Eq.\ J2.2.2.1\text{-}2)$$

Ω = 1.95 (*ASD*)
ϕ = 0.80 (*LRFD*)
= 0.65 (*LSD*)

For $0.815\sqrt{E/F_u} < (d_a/t) < 1.397\sqrt{E/F_u}$

$$P_{nv} = 0.280\left[1 + 5.59\frac{\sqrt{E/F_u}}{d_a/t}\right] t d_a F_u \qquad (Eq.\ J2.2.2.1\text{-}3)$$

Ω = 1.75 (*ASD*)
ϕ = 0.85 (*LRFD*)
= 0.70 (*LSD*)

For $(d_a/t) \ge 1.397\sqrt{E/F_u}$

$$P_{nv} = 1.40\, t d_a F_u \qquad (Eq.\ J2.2.2.1\text{-}4)$$

Ω = 3.30 (*ASD*)
ϕ = 0.45 (*LRFD*)

= 0.35 (*LSD*)

where

P_{nv} = *Nominal shear strength* [*resistance*] of arc spot weld

d_e = Effective diameter of fused area at plane of maximum shear transfer

= Greater of 0.7d – 1.5t and 0.45d (*Eq*. J2.2.2.1-5)

where

d = Visible diameter of outer surface of arc spot weld

t = Total combined base steel *thickness* (exclusive of coatings) of sheets involved in shear transfer above plane of maximum shear transfer

F_{xx} = *Tensile strength* of electrode classification

d_a = Average diameter of arc spot weld at mid-*thickness* of t where d_a = (d – t) for single sheet or multiple sheets not more than four lapped sheets over a supporting member. See Figures J2.2.2.1-1 and J2.2.2.1-2 for diameter definitions.

E = Modulus of elasticity of steel

F_u = *Tensile strength* as determined in accordance with Section A3.1 or A3.2

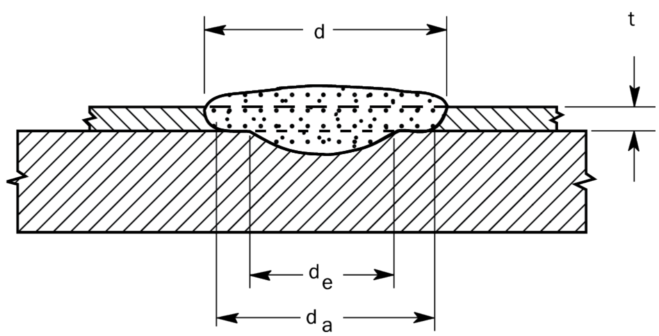


Figure J2.2.2.1-1 Arc Spot Weld – Single Thickness of Sheet

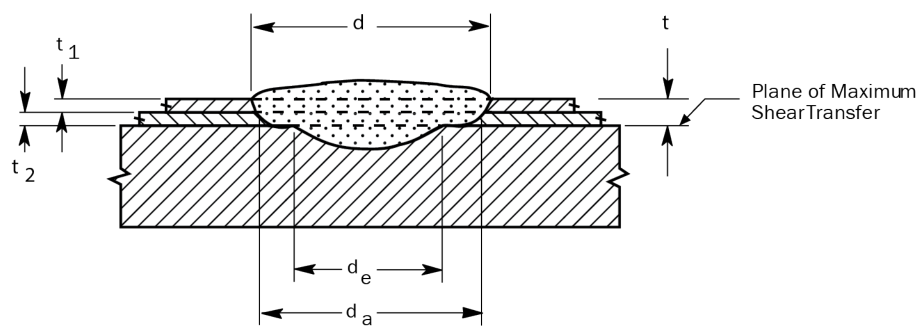


Figure J2.2.2.1-2 Arc Spot Weld – Multiple Thicknesses of Sheet

J2.2.2.2 Shear Strength for Sheet-to-Sheet Connections

The *nominal shear strength* [*resistance*], P_{nv}, for each weld between two sheets of equal *thickness* shall be determined in accordance with Eq. J2.2.2.2-1. The *safety factor* and *resistance factors* in this section shall be used to determine the *available strength* [*factored* *resistance*], P_{av}, in accordance with the applicable design method in Section B3.2.1, B3.2.2, or B3.2.3.

$$P_{nv} = 1.65td_aF_u \quad (Eq.\ J2.2.2.2\text{-}1)$$

Ω = 1.95 (*ASD*)

ϕ = 0.80 (*LRFD*)

= 0.65 (*LSD*)

where

P_{nv} = *Nominal shear strength* [*resistance*] of sheet-to-sheet *connection*

t = Base steel *thickness* (exclusive of coatings) of single welded sheet

d_a = Average diameter of arc spot weld at mid-thickness of t. See Figure J2.2.2.2-1 for diameter definitions

$$= (d - t) \quad (Eq.\ J2.2.2.2\text{-}2)$$

where

d = Visible diameter of the outer surface of arc spot weld

F_u = *Tensile strength* of sheet as determined in accordance with Section A3.1 or A3.2

In addition, the following limits shall apply:

(a) $F_u \leq 59$ ksi (407 MPa or 4150 kg/cm^2),

(b) $F_{xx} > F_u$, and

(c) 0.028 in. (0.71 mm) $\leq t \leq$ 0.0635 in. (1.61 mm).

See Section J2.2.2.1 for definition of F_{xx}.

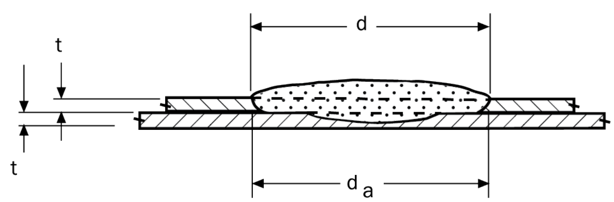


Figure J2.2.2.2-1 Arc Spot Weld – Sheet to Sheet

J2.2.3 Tension

The uplift *available tensile strength* [*factored resistance*], P_{at}, of each concentrically loaded arc spot weld connecting sheet(s) and supporting member shall be the smaller value which is computed with the *nominal tensile strength* [*resistance*], P_{nt}, determined using either Eq. J2.2.3-1 or Eq. J2.2.3-2, as follows, and the *safety factors* and *resistance factors* applied in accordance with the applicable design method in Section B3.2.1, B3.2.2, or B3.2.3.

$$P_{nt} = r\,\frac{\pi d_e^2}{4} F_{xx} \quad (Eq.\ J2.2.3\text{-}1)$$

$$P_{nt} = r\,0.8(F_u/F_y)^2\,t\,d_aF_u \quad (Eq.\ J2.2.3\text{-}2)$$

where

r = Weld effectiveness reduction factor determined in accordance with Table J2.2.3-1

t = *Thickness* as defined in Table J2.2.3-1

See Section J2.2.2.1 for definitions of other variables.

The *safety* and *resistance factors* are provided in Table J2.2.3-1.

Table J2.2.3-1
Safety and Resistance Factors, Thickness Definition, and Weld Effectiveness Reduction

Equation Number	Sheet Configuration	Panel and Deck			Other			Thickness t	Reduction Factor r
		Ω (ASD)	ϕ (LRFD)	ϕ (LSD)	Ω (ASD)	ϕ (LRFD)	ϕ (LSD)		
J2.2.3-1	All	3.05	0.50	0.35	3.90	0.40	0.30	Total *thickness* of sheet(s)	0.5; 1.0 with weld washer
J2.2.3-2	Single or multiple sheets	2.00	0.75	0.60	2.35	0.65	0.50	Total *thickness* of sheet(s)	1.0
	Sidelap *connections* in a deck system	2.90	0.55	0.40	3.50	0.45	0.35	*Thickness* of topmost sheet	1.0
	Eccentrically loaded *connections*	2.30	0.65	0.50	2.75	0.55	0.45	Total *thickness* of sheet(s)	0.5; 1.0 with weld washer

The following limits shall apply:

(a) $t\, d_a\, F_u \leq 3$ kips (13.3 kN or 1360 kg),

(b) $F_{xx} \geq 60$ ksi (410 MPa or 4220 kg/cm^2), and

(c) $F_u \leq 82$ ksi (565 MPa or 5770 kg/cm^2) (of connecting sheets)

Where it is shown by measurement that a given weld procedure consistently gives a larger effective diameter, d_e, or average diameter, d_a, as applicable, this larger diameter is permitted to be used provided the particular welding procedure used for making those welds is followed.

J2.2.4 Combined Shear and Tension on an Arc Spot Weld

For arc spot weld *connections* subjected to a combination of shear and tension, the following interaction check shall be applied:

If $\left(\dfrac{\overline{T}}{P_{at}}\right)^{1.5} \leq 0.15$, no interaction check is required.

If $\left(\dfrac{\overline{T}}{P_{at}}\right)^{1.5} > 0.15$,

$$\left(\frac{\overline{V}}{P_{av}}\right)^{1.5} + \left(\frac{\overline{T}}{P_{at}}\right)^{1.5} \leq 1 \qquad (Eq.\ J2.2.4\text{-}1)$$

where

$\overline{T}$ = *Required tensile strength* [tensile force due to *factored loads*] per *connection* fastener determined in accordance with *ASD*, *LRFD,* or *LSD load combinations*

$\overline{V}$ = *Required shear strength* [shear force due to *factored loads*] per *connection* fastener, determined in accordance with *ASD*, *LRFD,* or *LSD load combinations*

P_{at} = *Available tension strength* [*factored resistance*] as given by Section J2.2.3

P_{av} = *Available shear strength* [*factored resistance*] as given by Section J2.2.2

In addition, the following limitations shall be satisfied:

(a) $F_u \le 105$ ksi (724 MPa or 7380 kg/cm^2),

(b) $F_{xx} \ge 60$ ksi (414 MPa or 4220 kg/cm^2),

(c) $td_aF_u \le 3$ kips (13.3 kN or 1360 kg),

(d) $F_u/F_y \ge 1.02$, and

(e) 0.47 in. (11.9 mm) $\le d \le$ 1.02 in. (25.9 mm).

See Section J2.2.2.1 for definition of variables.

J2.3 Arc Seam Welds

Arc seam welds covered by this *Specification* shall apply only to the following *joints*:

(a) Sheet to thicker supporting member in the flat position (See Figure J2.3-1), and

(b) Sheet to sheet in the horizontal or flat position.

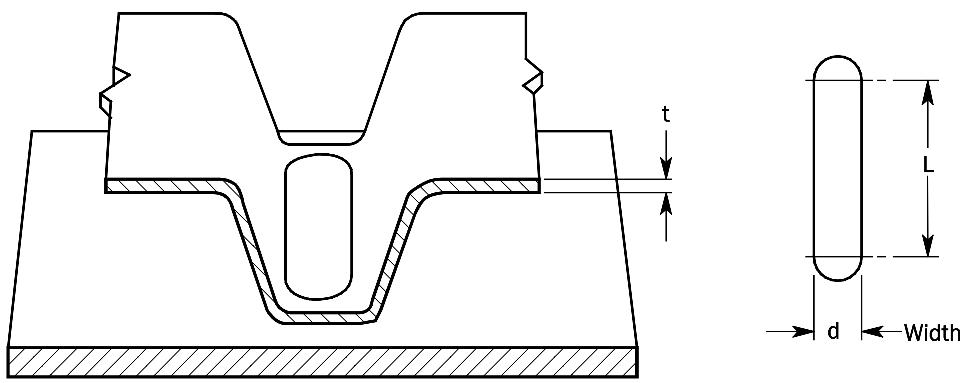


Figure J2.3-1 Arc Seam Welds – Sheet to Supporting Member in Flat Position

J2.3.1 Minimum Edge and End Distance

The distance from the centerline of an arc seam weld to the end or edge of the connected member shall not be less than 1.5d and shall satisfy the requirements of Section J6. In no case shall the clear distance between welds and the end or edge of the member be less than 1.0d, where d is the visible width of the arc seam weld. See Figure J2.3.1-1 for details.

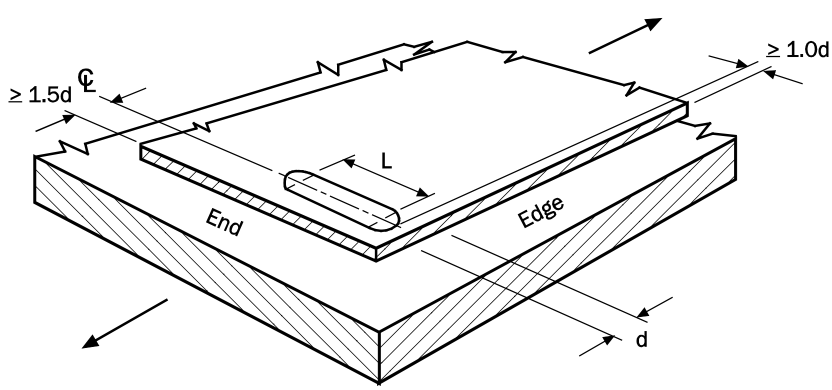


Figure J2.3.1-1 End and Edge Distances for Arc Seam Welds

J2.3.2 Shear

J2.3.2.1 Shear Strength for Sheet(s) Welded to a Thicker Supporting Member

The *available shear strength* [*factored resistance*], P_{av}, of arc seam welds shall be the smaller value which is determined with the *nominal shear strength* [*resistance*], P_{nv}, determined by using either Eq. J2.3.2.1-1 or Eq. J2.3.2.1-2, and the *safety factor* and *resistance factors* applied in accordance with the applicable design method in Section B3.2.1, B3.2.2, or B3.2.3.

$$P_{nv} = \left(\frac{\pi d_e^2}{4} + L d_e\right) 0.75 F_{xx} \qquad (Eq.\ J2.3.2.1\text{-}1)$$

$$P_{nv} = 2.5 t F_u (0.25L + 0.96 d_a) \qquad (Eq.\ J2.3.2.1\text{-}2)$$

Ω = 2.45 (*ASD*)
ϕ = 0.60 (*LRFD*)
= 0.50 (*LSD*)

where
P_{nv} = *Nominal shear strength* [*resistance*] of arc seam weld
d_e = Effective width of arc seam weld at fused surfaces
= 0.7d – 1.5t (*Eq*. J2.3.2.1-3)
where
d = Visible width of arc seam weld
L = Length of seam weld not including circular ends
(For computation purposes, L shall not exceed 3d)
d_a = Average width of arc seam weld
= (d – t) for single or double sheets (*Eq*. J2.3.2.1-4)
F_u, F_{xx}, and t = Values as defined in Section J2.2.2.1

J2.3.2.2 Shear Strength for Sheet-to-Sheet Connections

The *nominal shear strength* [*resistance*], P_{nv}, for each weld between two sheets of equal *thickness* shall be determined in accordance with Eq. J2.3.2.2-1. The *safety factor* and *resistance factors* in this section shall be used to determine the *available strength* [*factored resistance*], P_{av}, in accordance with the applicable design method in Section B3.2.1, B3.2.2 or B3.2.3.

$$P_{nv} = 1.65 t d_a F_u \qquad (Eq.\ J2.3.2.2\text{-}1)$$

Ω = 1.95 (*ASD*)
ϕ = 0.80 (*LRFD*)
= 0.65 (*LSD*)

where
P_{nv} = *Nominal shear strength* [*resistance*] of sheet-to-sheet *connection*
d_a = Average width of arc seam weld at mid-*thickness*. See Figure J2.3.2.2-1 for width definitions
= (d – t) (Eq. J2.3.2.2-2)
where
d = Visible width of the outer surface of arc seam weld

t = Base steel *thickness* (exclusive of coatings) of single welded sheet
F_u = *Tensile strength* of sheet as determined in accordance with Section A3.1 or A3.2

In addition, the following limits shall apply:

(a) $F_u \leq 59$ ksi (407 MPa or 4150 kg/cm^2),

(b) $F_{xx} > F_u$, and

(c) 0.028 in. (0.711 mm) $\leq t \leq$ 0.0635 in. (1.61 mm).

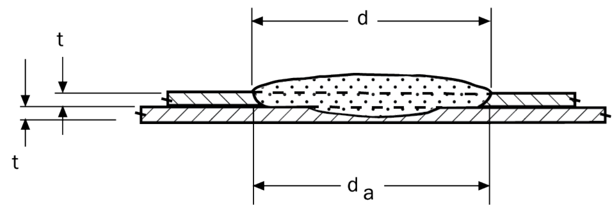


Figure J2.3.2.2-1 Arc Seam Weld – Sheet to Sheet

J2.4 Top Arc Seam Sidelap Welds

J2.4.1 Shear Strength of Top Arc Seam Sidelap Welds

The *nominal shear strength* [*resistance*], P_{nv}, for longitudinal loading of *top arc seam sidelap* welds shall be determined in accordance with Eq. J2.4.1-1. The following limits shall apply:

(a) $h_{st} \leq 1.25$ in. (31.8 mm),

(b) $F_{xx} \geq 60$ ksi (414 MPa),

(c) 0.028 in. (0.711 mm) $\leq t \leq$ 0.064 in. (1.63 mm), and

(d) 1.0 in. (25.4 mm) $\leq L_w \leq$ 2.5 in. (63.5 mm).

where

h_{st} = Nominal seam height. See Figure J2.4.1-1
F_{xx} = *Tensile strength* of electrode classification
L_w = Length of *top arc seam sidelap weld*
t = Base steel *thickness* (exclusive of coatings) of thinner connected sheet

$$P_{nv} = [4.0(F_u/F_{sy})-1.52](t/L_w)^{0.33}L_w t F_u \qquad (Eq.\ J2.4.1\text{-}1)$$

Ω = 2.60 (*ASD*)
ϕ = 0.60 (*LRFD*)
= 0.55 (*LSD*)

where

P_{nv} = *Nominal shear strength* [*resistance*] of *top arc seam sidelap weld*
F_u = Specified minimum *tensile strength* of connected sheets as determined in accordance with Section A3.1.1, A3.1.2, or A3.1.3
F_{sy} = *Specified minimum yield stress* of connected sheets as determined in accordance with Section A3.1.1, A3.1.2, or A3.1.3

It is permitted to exclude the *connection* design reduction specified in Sections A3.1.2, A3.1.3(b), and A3.1.3(c) for *top arc seam welds* provided the arc seam welds meet minimum spacing requirements along steel deck *diaphragm* side laps.

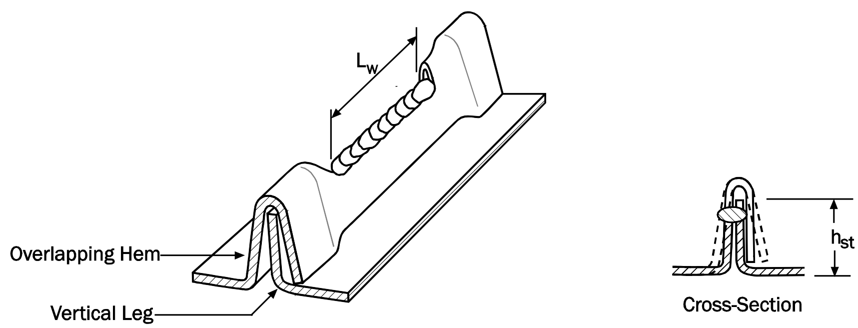
The minimum end distance and the weld spacing shall satisfy the shear rupture requirements in Section J6.

The *top arc seam sidelap weld connection* shall be made as follows:

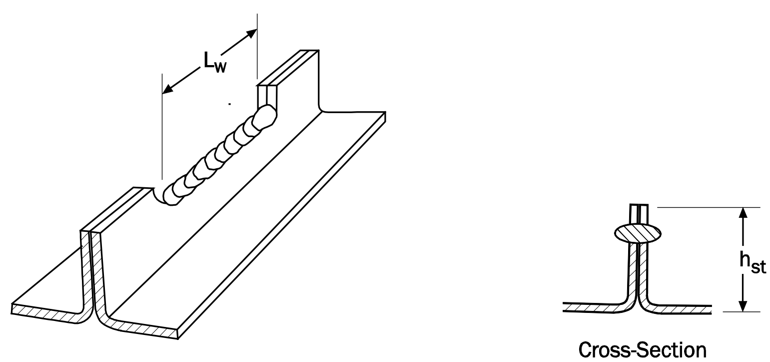
(a) Vertical legs in either vertical leg and overlapping hem *joints* or vertical leg *joints* fit snugly, and

(b) In hem *joints*, the overlapping hem is crimped onto the vertical leg and the crimp length shall be longer than the specified weld length, L_w.

Holes or openings in the hem at either one or both ends of the weld are permitted.



(a) Vertical Leg and Overlapping Hem Joint



(b) Back-to-Back Vertical leg Joint

Figure J2.4.1-1 Top Arc Seam Sidelap Weld

J2.5 Arc Plug Welds

Arc plug welds, where permitted by this *Specification*, shall be designed using the provisions of Section J2.2. The minimum diameter of the hole through which the plug weld is created shall not be less than 3/8 in. (9.53 mm), nor that required to develop an effective weld diameter, d_e, not less than 3/8 in. (9.53 mm).

J2.6 Fillet Welds

Fillet welds covered by this *Specification* shall apply to the welding of *joints* in any position, either:

(1) Sheet to sheet, or

(2) Sheet to thicker steel member.

The *nominal shear strength* [*resistance*], P_{nv}, of a fillet weld shall be the lesser of P_{nv1} and P_{nv2} as determined in accordance with this section. The corresponding *safety factors* and *resistance factors* given in this section shall be used to determine the *available strength* [*factored resistance*], P_{av}, in accordance with the applicable design method in Section B3.2.1, B3.2.2, or B3.2.3.

(a) For longitudinal loading:

For $L/t < 25$

$$P_{nv1} = \left(1 - \frac{0.01L}{t_1}\right) L t_1 F_{u1} \qquad (Eq.\ J2.6\text{-}1)$$

$$P_{nv2} = \left(1 - \frac{0.01L}{t_2}\right) L t_2 F_{u2} \qquad (Eq.\ J2.6\text{-}2)$$

Ω = 2.55 (*ASD*)

ϕ = 0.60 (*LRFD*)

= 0.50 (*LSD*)

For $L/t \geq 25$

$$P_{nv1} = 0.75\ t_1 L F_{u1} \qquad (Eq.\ J2.6\text{-}3)$$

$$P_{nv2} = 0.75\ t_2 L F_{u2} \qquad (Eq.\ J2.6\text{-}4)$$

Ω = 3.05 (*ASD*)

ϕ = 0.50 (*LRFD*)

= 0.40 (*LSD*)

(b) For transverse loading:

$$P_{nv1} = t_1 L F_{u1} \qquad (Eq.\ J2.6\text{-}5)$$

$$P_{nv2} = t_2 L F_{u2} \qquad (Eq.\ J2.6\text{-}6)$$

Ω = 2.35 (*ASD*)

ϕ = 0.65 (*LRFD*)

= 0.60 (*LSD*)

where

t_1, t_2 = *Thickness* of connected parts, as shown in Figures J2.6-1 and J2.6-2

t = Lesser value of t_1 and t_2

F_{u1}, F_{u2} = *Tensile strength* of connected parts corresponding to *thicknesses* t_1 and t_2

P_{nv1}, P_{nv2} = *Nominal shear strength* [*resistance*] corresponding to connected *thicknesses* t_1 and t_2

In addition, for $t > 0.10$ in. (2.54 mm), the *nominal strength* [*resistance*] determined in accordance with (1) and (2) shall not exceed the following value of P_n:

$$P_n = 0.75\ t_w L F_{xx} \qquad (Eq.\ J2.6\text{-}7)$$

Ω = 2.55 (*ASD*)

ϕ = 0.60 (*LRFD*)

= 0.50 (*LSD*)

where

P_n = *Nominal fillet weld strength* [*resistance*]

L = Length of fillet weld

F_{xx} = Tensile strength of electrode classification

t_w = Effective throat

= 0.707 w_1 or 0.707 w_2, whichever is smaller. A larger effective throat is permitted if measurement shows that the welding procedure to be used consistently yields a larger value of t_w.

where

w_1, w_2 = leg of weld (see Figures J2.6-1 and J2.6-2) and $w_1 \le t_1$ in lap *joints*

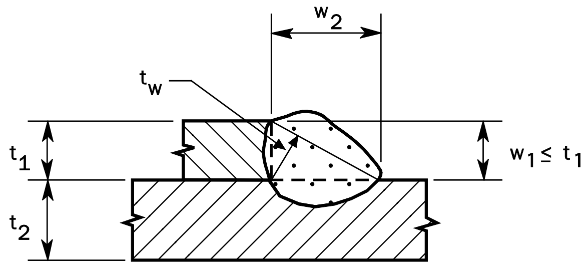


Figure J2.6-1 Fillet Welds – Lap Joint

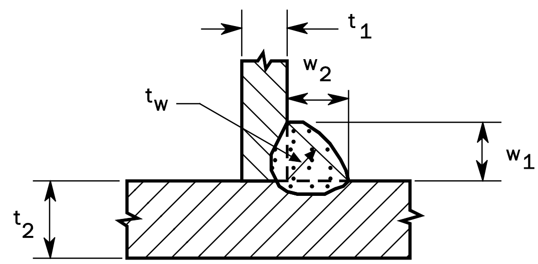


Figure J2.6-2 Fillet Welds – T-Joint

J2.7 Flare Groove Welds

Flare groove welds covered by this *Specification* shall apply to welding of *joints* in any position, either sheet to sheet for flare V-groove welds, sheet to sheet for flare bevel groove welds, or sheet to thicker steel member for flare bevel groove welds.

The *nominal shear strength* [*resistance*], P_{nv}, of a flare groove weld shall be determined in accordance with this section. The corresponding *safety factors* and *resistance factors* given in this section shall be used to determine the *available strength* [*factored resistance*], P_{av}, in accordance with the applicable design method in Section B3.2.1, B3.2.2, or B3.2.3.

Larger effective throat thicknesses, t_w, than those determined by Eq. J2.7-5 or Eq. J2.7-7, as appropriate, are permitted, provided the fabricator can establish by qualification the consistent production of such larger effective throat thicknesses. Qualification shall consist of sectioning the weld normal to its axis, at mid-length and terminal ends. Such sectioning shall be made on a number of combinations of material sizes representative of the range to be used in the fabrication.

(a) For flare bevel groove welds, transverse loading (see Figure J2.7-1):

P_{nv} = $0.833tLF_u$ (*Eq.* J2.7-1)

Ω = 2.55 (*ASD*)

ϕ = 0.60 (*LRFD*)

= 0.50 (*LSD*)

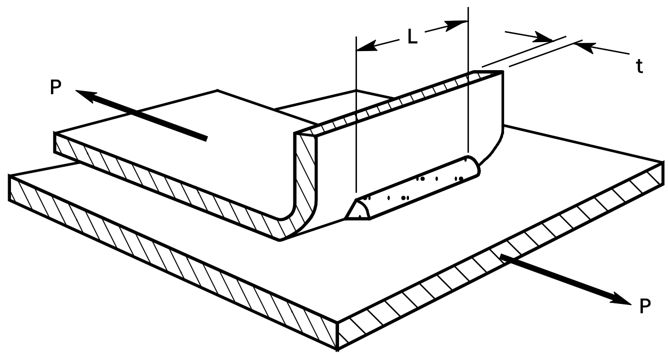


Figure J2.7-1 Flare Bevel Groove Weld

(b) For flare groove welds, longitudinal loading (see Figures J2.7-2 and J2.7-3):

(1) For $t \le t_w < 2t$ or if the lip height, h, is less than weld length, L:

$$P_{nv} = 0.75tLF_u \qquad (Eq.\ J2.7\text{-}2)$$

Ω = 2.80 (*ASD*)
ϕ = 0.55 (*LRFD*)
= 0.45 (*LSD*)

(2) For $t_w \ge 2t$ with the lip height, h, equal to or greater than weld length, L:

$$P_{nv} = 1.50tLF_u \qquad (Eq.\ J2.7\text{-}3)$$

Ω = 2.80 (*ASD*)
ϕ = 0.55 (*LRFD*)
= 0.45 (*LSD*)

(c) For t > 0.10 in. (2.54 mm), the *nominal strength* [*resistance*] determined in accordance with (a) or (b) shall not exceed the value of P_n calculated in accordance with Eq. J2.7-4:

$$P_n = 0.75t_wLF_{xx} \qquad (Eq.\ J2.7\text{-}4)$$

Ω = 2.55 (*ASD*)
ϕ = 0.60 (*LRFD*)
= 0.50 (*LSD*)

where
P_n = *Nominal flare groove weld strength* [*resistance*]
t = *Thickness* of welded member as illustrated in Figures J2.7-1 to J2.7-3
L = Length of weld
F_u and F_{xx} = Values as defined in Section J2.2.2.1
h = Height of lip
t_w = Effective throat of flare groove weld determined using Eq. J2.7-5 or J2.7-7

(1) For a flare-bevel groove weld

$$t_w = \left(w_2 + t_{wf} - R + \sqrt{2Rw_1 - w_1^2}\right)\left(\frac{w_1}{w_f}\right) - R\eta\left(\frac{w_2}{w_f}\right) \qquad (Eq.\ J2.7\text{-}5)$$

where
w_1, w_2 = Leg of weld (see Figure J2.7-4), $w_1 \le R$
t_{wf} = Effective throat of groove weld that is filled flush to the surface, (i.e., w_1 = R, w_2 = 0), determined in accordance with Table J2.7-1

R = Radius of outside bend surface
η = [1 – cos(equivalent angle)] determined in accordance with Table J2.7-1
w_f = Face width of weld

$$= \sqrt{w_1^2 + w_2^2} \qquad (Eq.\ J2.7\text{-}6)$$

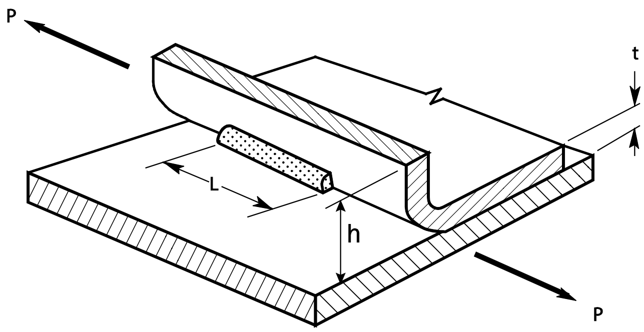


Figure J2.7-2 Shear in Flare Bevel Groove Weld

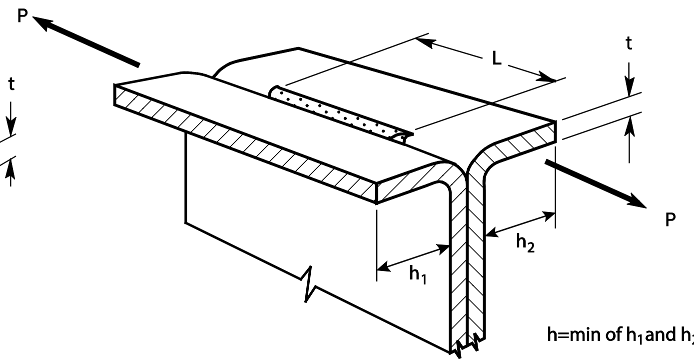


Figure J2.7-3 Shear in Flare V-Groove Weld

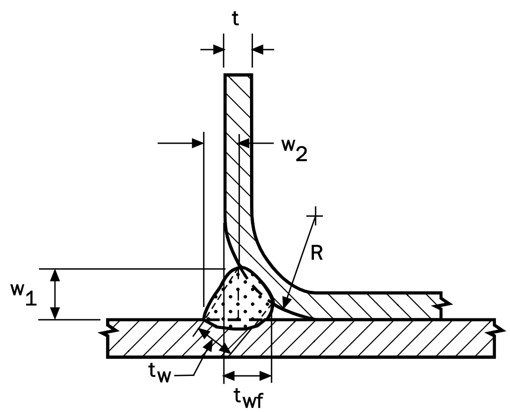


Figure J2.7-4 Flare-Bevel Groove Weld

Table J2.7-1
Flare Bevel Groove Welds

Welding Process	Throat Depth (t_{wf})	η
SMAW, FCAW-S[1]	5/16 R	0.274
GMAW, FCAW-G[2]	5/8 R	0.073
SAW	5/16 R	0.274

Notes:
[1] In Canada, FCAW-S is known as FCAW (self-shielded).
[2] In Canada, FCAW-G is known as FCAW (gas-shielded).

(2) For a flare V-groove weld

t_w = smaller of (t_{wf} – d_1) and (t_{wf} – d_2) (*Eq.* J2.7-7)

where

d_1 and d_2 = Weld offset from flush condition (see Figure J2.7-5)

t_{wf} = Effective throat of groove weld that is filled flush to the surface (i.e. $d_1 = d_2 = 0$), determined in accordance with Table J2.7-2

R_1 and R_2 = Radius of outside bend surface as illustrated in Figure J2.7-5

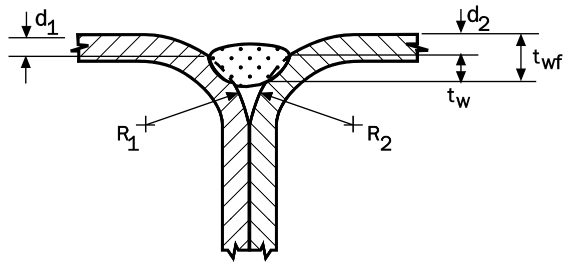


Figure J2.7-5 Flare V-Groove Weld

Table J2.7-2
Flare V-Groove Welds

Welding Process	Throat Depth (t_{wf})
SMAW, FCAW-S[1]	5/8 R
GMAW, FCAW-G[2]	3/4 R
SAW	1/2 R

where
R is the lesser of R_1 and R_2.
Notes:
[1] In Canada, FCAW-S is known as FCAW (self-shielded).
[2] In Canada, FCAW-G is known as FCAW (gas-shielded).

J2.8 Resistance Welds

The *nominal shear strength* [*resistance*], P_{nv}, of resistance (spot) welds shall be determined in accordance with this section. The *safety factor* and *resistance factors* given in this section shall be used to determine the *available strength* [*factored resistance*], P_{av}, in accordance with the applicable design method in Section B3.2.1, B3.2.2, or B3.2.3.

Ω = 2.35 (*ASD*)
ϕ = 0.65 (*LRFD*)
= 0.55 (*LSD*)

When t is in inches and P_{nv} is in kips:

For 0.01 in. ≤ t < 0.14 in.

$$P_{nv} = 144t^{1.47} \qquad (Eq.\ J2.8\text{-}1)$$

For 0.14 in. ≤ t ≤ 0.18 in.

$$P_{nv} = 43.4t + 1.93 \qquad (Eq.\ J2.8\text{-}2)$$

When t is in millimeters and P_{nv} is in kN:

For 0.25 mm ≤ t < 3.56 mm

$P_{nv} = 5.51t^{1.47}$ (*Eq.* J2.8-3)

For 3.56 mm ≤ t ≤ 4.57 mm

$P_{nv} = 7.6t + 8.57$ (*Eq.* J2.8-4)

When t is in centimeters and P_{nv} is in kg:

For 0.025 cm ≤ t < 0.356 cm

$P_{nv} = 16600t^{1.47}$ (*Eq.* J2.8-5)

For 0.356 cm ≤ t ≤ 0.457 cm

$P_{nv} = 7750t + 875$ (*Eq.* J2.8-6)

where

P_{nv} = *Nominal resistance weld shear strength* [*resistance*]

t = *Thickness* of thinnest outside sheet

J3 Bolted Connections

The following design criteria shall apply to steel-to-steel bolted *connections* used for *cold-formed steel structural members* in which the *thickness* of the thinnest connected part is 3/16 in. (4.76 mm) or less. For bolted *connections* in which the *thickness* of the thinnest connected part is greater than 3/16 in. (4.76 mm), the following specifications and standards shall apply:

(a) ANSI/AISC 360 for the United States and Mexico, and
(b) CSA S16 for Canada.

Bolts, nuts, and washers conforming to one of the following ASTM or SAE specifications are approved for use under this *Specification*:

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

ASTM A307 (Type A), *Standard Specification for Carbon Steel Bolts and Studs, 60,000 PSI Tensile Strength*

ASTM A354 (Grade BD), *Standard Specification for Quenched and Tempered Alloy Steel Bolts, Studs, and Other Externally Threaded Fasteners* (for diameter of bolt smaller than 1/2 in.)

ASTM A449, *Standard Specification for Hex Cap Screws, Bolts and Studs, Steel Heat Treated, 120/105/90 ksi Minimum Tensile Strength, General Use*

ASTM A563, *Standard Specification for Carbon and Alloy Steel Nuts*

ASTM A563M, *Standard Specification for Carbon and Alloy Steel Nuts [Metric]*

ASTM F436, *Standard Specification for Hardened Steel Washers*

ASTM F436M, *Standard Specification for Hardened Steel Washers [Metric]*

ASTM F844, *Standard Specification for Washers, Steel, Plain (Flat), Unhardened for General Use*

ASTM F959, *Standard Specification for Compressible Washer-Type Direct Tension Indicators for Use With Structural Fasteners*

ASTM F959M, *Standard Specification for Compressible Washer-Type Direct Tension Indicators for Use With Structural Fasteners [Metric]*

ASTM F3125, *Standard Specification for High Strength Structural Bolts, Steel and Alloy Steel, Heat Treated, 120 ksi (830 MPa) and 150 ksi (1040 MPa) Minimum Tensile Strength, Inch and Metric Dimensions* (for Grades A325, A325M, A490, and A490M only)

ASTM F3148, *Standard Specification for High Strength Structural Bolt Assemblies, Steel and Alloy Steel, Heat Treated, 144 ksi Minimum Tensile Strength, Inch Dimensions*

SAE J429, *Mechanical and Material Requirements for Externally Threaded Fasteners*

SAE J995, *Mechanical and Material Requirements for Steel Nuts*

SAE J2486, *Tension Indicating Washer Tightening Method for Fasteners*

SAE J2655, *Fastener Part Standard - Washers and Lockwashers (Inch Dimensioned)*

When bolts, nuts, and washers other than the above are used, drawings shall clearly indicate the types and sizes of fasteners to be employed and the *nominal strength* [*resistance*] assumed in design.

Nuts and washers shall be appropriate for the *connection* and the corresponding bolt.

Bolts shall be installed and tightened to achieve satisfactory performance of the *connections*.

The holes for bolts shall not exceed the sizes specified in Table J3-1 (J3-1M), except that larger holes are permitted to be used in column base details or structural systems connected to concrete walls.

(a) For Hole Deformation Considered

When the bolt hole deformation is considered in design in accordance with Eq. J3.3.2-1, the following restrictions shall be applied:

(1) Standard holes are used in bolted *connections*, except that oversized and slotted holes are permitted to be used as approved by the designer,
(2) The length of slotted holes is normal to the direction of the shear *load*, and
(3) Washers or backup plates are installed over oversized or slotted holes in an outer ply unless suitable performance is demonstrated by tests in accordance with Section K2.

(b) For Hole Deformation Not Considered

When the bolt hole deformation is not considered in design, oversized holes and short-slotted holes are permitted. The holes for bolts shall not exceed the sizes specified in Table J3-1 (J3-1M).

Slotted or oversized holes shall be taken as standard holes when the holes occur within the lap of lapped or nested Z-members, subject to the following restrictions:

(1) 1/2 in. (12 mm)-diameter bolts only with or without washers or backup plates,
(2) Maximum slot size is 9/16 in. × 7/8 in. (15 mm × 23 mm), slotted vertically,
(3) Maximum oversize hole is 5/8 in. (16 mm) diameter,
(4) Minimum member *thickness* is 0.060 in. (1.52 mm) nominal,
(5) Maximum member *yield stress* is 60 ksi (414 MPa, and 4220 kg/cm^2), and
(6) Minimum lap length measured from center of frame to end of lap is 1.5 times the member depth.

TABLE J3-1
Maximum Size of Bolt Holes, in Inches

Nominal Bolt Diameter, d in.	Standard Hole Diameter, d_h in.	Oversized Hole Diameter, d_h in.	Short-Slotted Hole Dimensions in.	Long-Slotted Hole Dimensions in.	Alternative Short-Slotted Hole[a] Dimensions in.
$d < 1/2$	$d + 1/32$	$d + 1/16$	$(d + 1/32)$ by $(d + 1/4)$	$(d + 1/32)$ by $(2\,1/2\ d)$	
$1/2 \le d < 1$	$d + 1/16$	$d + 1/8$	$(d + 1/16)$ by $(d + 1/4)$	$(d + 1/16)$ by $(2\,1/2\ d)$	$9/16$ by $7/8$
$d = 1$	$1\,1/8$	$1\,1/4$	$(1\,1/8)$ by $(1\,5/16)$	$(1\,1/8)$ by $(2\,1/2)$	
$d \ge 1$	$d + 1/8$	$d + 5/16$	$(d + 1/8)$ by $(d + 3/8)$	$(d + 1/8)$ by $(2\,1/2\ d)$	

Note: [a] The alternative short-slotted hole is only applicable for d=1/2 in.

TABLE J3-1M
Maximum Size of Bolt Holes, in Millimeters

Nominal Bolt Diameter, d mm	Standard Hole Diameter, d_h mm	Oversized Hole Diameter, d_h mm	Short-Slotted Hole Dimensions mm	Long-Slotted Hole Dimensions mm	Alternative Short-Slotted Hole[a] Dimensions mm
$d < 12$	$d + 1$	$d + 2$	$(d + 1)$ by $(d + 6)$	$(d + 1)$ by $(2\,1/2\ d)$	
$12 \le d \le 20$	$d + 2$	$d + 4$	$(d + 2)$ by $(d + 6)$	$(d + 2)$ by $(2\,1/2\ d)$	15 by 23
$20 < d < 24$	$d + 2$	$d + 6$	$(d + 2)$ by $(d + 8)$	$(d + 2)$ by $(2\,1/2\ d)$	
$d = 24$	27	30	27 by 32	27 x 60	
$d > 24$	$d + 3$	$d + 8$	$(d + 3)$ by $(d + 10)$	$(d + 3)$ by $(2\,1/2 d)$	

Note: [a] The alternative short-slotted hole is only applicable for d=12 mm.

J3.1 Minimum Spacing

The distance between the centers of fasteners shall not be less than 3d, where d is the nominal bolt diameter. In addition, the minimum distance between centers of bolt holes shall provide clearance for bolt heads, nuts, washers and the wrench. For oversized and slotted holes, the clear distance between the edges of two adjacent holes shall not be less than 2d.

J3.2 Minimum Edge and End Distances

The distance from the center of a fastener to the edge or end of any part shall not be less than 1.5d and shall satisfy the requirements of Section J6, where d is the nominal bolt diameter. For oversized and slotted holes, the distance between the edge of the hole and the edge or end of the member shall not be less than d.

J3.3 Bearing

The *available bearing strength* [*factored resistance*] of bolted *connections* shall be determined in accordance with Sections J3.3.1 and J3.3.2. For conditions not shown, the *available bearing strength* [*factored resistance*] of bolted *connections* shall be determined by tests.

J3.3.1 Bearing Strength Without Consideration of Bolt Hole Deformation

When deformation around the bolt holes is not a design consideration, the *nominal bearing strength* [*resistance*], P_{nb}, of the connected sheet for each loaded bolt shall be determined in accordance with Eq. J3.3.1-1. The *safety factor* and *resistance factors* given in this section shall be used to determine the *available strength* [*factored resistance*] in accordance with the applicable design method in Section B3.2.1, B3.2.2, or B3.2.3.

$$P_{nb} = C\, m_f\, d\, t\, F_u \qquad (Eq.\ J3.3.1\text{-}1)$$

Ω = 2.50 (*ASD*)

ϕ = 0.60 (*LRFD*)

= 0.50 (*LSD*)

where

C = Bearing factor, determined in accordance with Table J3.3.1-1

m_f = Modification factor for type of bearing *connection*, which is determined according to Table J3.3.1-2

d = Nominal bolt diameter

t = Uncoated sheet *thickness*

F_u = *Tensile strength* of sheet as defined in Section A3.1 or A3.2

Table J3.3.1-1
Bearing Factor, C[1]

	Connections With Standard Holes		Connections With Oversized or Short-Slotted Holes	
Thickness of Connected Part, t, in. (mm)	Ratio of Fastener Diameter to Member Thickness, d/t	C	Ratio of Fastener Diameter to Member Thickness, d/t	C
$0.024 \le t < 0.1875$ ($0.61 \le t < 4.76$)	$d/t < 10$	3.0	$d/t < 7$	3.0
	$10 \le d/t \le 22$	$4 - 0.1(d/t)$	$7 \le d/t \le 18$	$1+14/(d/t)$
	$d/t > 22$	1.8	$d/t > 18$	1.8

Note: [1] Oversized or short-slotted holes within the lap of lapped or nested Z-members as defined in Section J3 are permitted to be considered as standard holes.

Table J3.3.1-2
Modification Factor, m_f, for Type of Bearing Connection[1]

Type of Bearing Connection	m_f
Single Shear and Outside Sheets of Double Shear Connection Using Standard Holes With Washers Under Both Bolt Head and Nut	1.00
Single Shear and Outside Sheets of Double Shear Connection Using Standard Holes Without Washers Under Both Bolt Head and Nut, or With Only One Washer	0.75
Single Shear and Outside Sheets of Double Shear Connection Using Oversized or Short-Slotted Holes Parallel to the Applied Load Without Washers Under Both Bolt Head and Nut, or With Only One Washer	0.70
Single Shear and Outside Sheets of Double Shear Connection Using Short-Slotted Holes Perpendicular to the Applied Load Without Washers Under Both Bolt Head and Nut, or With Only One Washer	0.55
Inside Sheet of Double Shear Connection Using Standard Holes With or Without Washers	1.33
Inside Sheet of Double Shear Connection Using Oversized or Short-Slotted Holes Parallel to the Applied Load With or Without Washers	1.10
Inside Sheet of Double Shear Connection Using Short-Slotted Holes Perpendicular to the Applied Load With or Without Washers	0.90

Note: [1] Oversized or short-slotted holes within the lap of lapped or nested Z-members as defined in Section J3 are permitted to be considered as standard holes.

J3.3.2 Bearing Strength With Consideration of Bolt Hole Deformation

When deformation around a bolt hole is a design consideration, the *nominal bearing strength* [*resistance*], P_{nb}, shall be calculated in accordance with Eq. J3.3.2-1. The *safety factor* and *resistance factors* given in this section shall be used to determine the *available strength* [*factored resistance*] in accordance with the applicable design method in Section B3.2.1, B3.2.2, or B3.2.3. In addition, the *available strength* [*factored resistance*] shall not exceed the *available strength* [*factored resistance*] obtained in accordance with Section J3.3.1.

$$P_{nb} = (4.64\alpha t + 1.53)dtF_u \quad (Eq. J3.3.2-1)$$

Ω = 2.22 (*ASD*)
ϕ = 0.65 (*LRFD*)
= 0.55 (*LSD*)

where
α = Coefficient for conversion of units
= 1 for U.S. customary units (with t in inches)
= 0.0394 for SI units (with t in mm)
= 0.394 for MKS units (with t in cm)

See Section J3.3.1 for definitions of other variables.

J3.4 Shear and Tension in Bolts

See Section J3.4 of Appendix A or B for provisions provided in this section. ☜A,B

J3.5 Shear Strength of Lap Connections with a Single Bolt

For a single bolted lap *connection*, the *nominal strength* [*resistance*], P_{nv}, shall be taken as the smaller of P_{nv} calculated in accordance with Eq. J3.5-1 and Eq. J3.5-2.

$$P_{nv} = t\,e\,F_u \qquad (Eq.\ J3.5\text{-}1)$$

$$P_{nv} = 4.2\,(t^3 d)^{1/2} F_u \qquad (Eq.\ J3.5\text{-}2)$$

Ω = 2.35 (*ASD*)
ϕ = 0.70 (*LRFD*)
= 0.55 (*LSD*)

where
P_{nv} = *Nominal shear strength* [*resistance*] of lap *connection* with a single bolt
t = *Thickness* of part in which end distance is measured
e = Distance between end of each connected part and center of bolt hole
F_u = *Tensile strength* of connected part in which end distance is measured as specified in Section A3.1 or A3.2
d = Nominal bolt diameter

For the inside sheet of double shear *connections* using standard holes with or without washers, P_{nv} shall be calculated in accordance with Eq. J3.5-1, where

Ω = 2.10 (*ASD*)
ϕ = 0.75 (*LRFD*)
= 0.60 (*LSD*)

User note:
The shear strength of a lap *connection* with a single bolt, P_{nv}, needs to be calculated for each part independently, and the minimum capacity of two connected parts should be considered as the *connection* capacity.

J4 Screw Connections

The provisions of this section shall apply to steel-to-steel screw *connections* within specified limitations used for *cold-formed steel structural members*. All provisions in Section J4 shall apply to screws with 0.08 in. (2.03 mm) $\leq d \leq$ 0.25 in. (6.35 mm). The screws shall be thread-forming or thread-cutting, with or without a self-drilling point. Screws shall be installed and tightened in accordance with the manufacturer's recommendations.

For *diaphragm* applications, Section I2 shall be used.

The *safety factor* or *resistance factor* used to determine the *available strength* [*factored resistance*] in accordance with the applicable design method in Section B3.2.1, B3.2.2, or B3.2.3 shall be as indicated for the specific *limit state*.

Alternatively, design values for a particular application are permitted to be based on tests, with the *safety factor*, Ω, and the *resistance factor*, ϕ, determined in accordance with Section K2.

The following notation shall apply to Section J4:

d = Nominal screw diameter
d_h = Screw head diameter or hex washer head integral washer diameter
d_w = Steel washer diameter
d'_w = Effective pull-over resistance diameter
P_{nv} = *Nominal shear strength* [*resistance*] of sheet per screw
P_{nvs} = *Nominal shear strength* [*resistance*] of screw as reported by manufacturer or determined by independent laboratory testing
P_{not} = *Nominal pull-out strength* [*resistance*] of sheet per screw
P_{nov} = *Nominal pull-over strength* [*resistance*] of sheet per screw
P_{nts} = *Nominal tension strength* [*resistance*] of screw as reported by manufacturer or determined by independent laboratory testing
t_1 = *Thickness* of member in contact with screw head or washer
t_2 = *Thickness* of member not in contact with screw head or washer
t_c = Lesser of depth of penetration and *thickness* t_2
F_{u1} = *Tensile strength* of member in contact with screw head or washer
F_{u2} = *Tensile strength* of member not in contact with screw head or washer

J4.1 Minimum Spacing

The distance between the centers of fasteners shall not be less than 3d.

J4.2 Minimum Edge and End Distances

The distance from the center of a fastener to the edge or end of any part shall not be less than 1.5d and shall satisfy the requirements of Section J6.

J4.3 Shear

J4.3.1 Single Shear Connection Strength Limited by Tilting and Bearing

For a single shear *connection*, the *nominal shear strength* [*resistance*] of sheet per screw, P_{nv}, shall be determined in accordance with this section.

For $t_2/t_1 \le 1.0$, P_{nv} shall be taken as the smallest of

$$P_{nv} = 4.2\,(t_2^3 d)^{1/2} F_{u2} \quad (Eq.\ J4.3.1\text{-}1)$$

$$P_{nv} = 2.7\,t_1\,d\,F_{u1} \quad (Eq.\ J4.3.1\text{-}2)$$

$$P_{nv} = 2.7\,t_2\,d\,F_{u2} \quad (Eq.\ J4.3.1\text{-}3)$$

For $t_2/t_1 \ge 2.5$, P_{nv} shall be taken as the smaller of

$$P_{nv} = 2.7\,t_1\,d\,F_{u1} \quad (Eq.\ J4.3.1\text{-}4)$$

$$P_{nv} = 2.7\,t_2\,d\,F_{u2} \quad (Eq.\ J4.3.1\text{-}5)$$

For $1.0 < t_2/t_1 < 2.5$, P_{nv} shall be calculated by linear interpolation between the above two cases.

The following *safety* and *resistance factors* shall be used to determine the *available strength* [*factored resistance*]:

Ω = 2.80 (*ASD*)
ϕ = 0.55 (*LRFD*)
= 0.45 (*LSD*)

J4.3.2 Double Shear Connection Strength Limited by Bearing

For a double shear *connection* in which tilting has been constrained through the double-lapped *connection*, the sheet *nominal shear strength* [*resistance*] shall be determined in accordance with Eq. J4.3.2-1:

$$P_{nvi} = 2.7\, t_i\, d\, F_{ui} \qquad (Eq.\ J4.3.2\text{-}1)$$

where

P_{nvi} = *Nominal strength* [*resistance*] of individual sheet i

t_i = *Thickness* of sheet i

F_{ui} = *Tensile strength* of sheet corresponding to t_i

The force in any sheet within the *connection* shall be determined by structural analysis. The total *nominal shear strength* [*resistance*] of the *connection* shall not exceed the applied load which causes the force in any of the individual sheets to exceed the *nominal shear strength* [*resistance*] of that sheet as determined by Eq. J4.3.2-1.

The following *safety* and *resistance factors* shall be used to determine the *available strength* [*factored resistance*]:

Ω = 2.80 (*ASD*)
ϕ = 0.55 (*LRFD*)
= 0.45 (*LSD*)

J4.3.3 Shear in Screws

The *nominal shear strength* [*resistance*] of the screw shall be taken as P_{nvs}. The following *safety* and *resistance factors* shall be used to determine the *available strength* [*factored resistance*]:

Ω = 3.00 (*ASD*)
ϕ = 0.50 (*LRFD*)
= 0.40 (*LSD*)

Alternatively, the *safety factor* or the *resistance factor* is permitted to be determined in accordance with Section K2.1 and shall be taken as $1.25\Omega \leq 3.0$ (*ASD*), $\phi/1.25 \geq 0.5$ (*LRFD*), or $\phi/1.25 \geq 0.4$ (*LSD*).

J4.4 Tension

For screws that carry tension, the head of the screw or washer, if a washer is provided, shall have a diameter d_h or d_w not less than 5/16 in. (7.94 mm). The nominal washer thickness shall be at least 0.050 in. (1.27 mm) for t_1 greater than 0.027 in. (0.686 mm) and at least 0.024 in. (0.610 mm) for t_1 equal to or less than 0.027 in. (0.686 mm). The washer shall be at least 0.063 in. (1.60 mm) thick when 5/8 in. (15.9 mm) < $d_w \leq$ 3/4 in. (19.1 mm).

J4.4.1 Pull-Out Strength

The *nominal pull-out strength* [*resistance*] of sheet per screw, P_{not}, shall be calculated as follows:

$$P_{not} = 0.85\, t_c\, d\, F_{u2}[1.63(\alpha t_c)^{0.18}] \qquad (Eq.\ J4.4.1\text{-}1)$$

where

α = 1 for t_c in inches

= 0.0394 for t_c in millimeters

The following *safety* and *resistance factors* shall be used to determine the *available strength* [*factored resistance*]:

Ω = 2.80 (*ASD*)
ϕ = 0.55 (*LRFD*)
= 0.45 (*LSD*)

J4.4.2 Pull-Over Strength

The *nominal pull-over strength* [*resistance*] of sheet per screw, P_{nov}, shall be calculated as follows:

$$P_{nov} = 1.5 t_1 d'_w F_{u1} \quad (Eq.\ J4.4.2\text{-}1)$$

Exception: For steel included in Section A3.1.3 (Elongation < 3%) with *thickness* of less than 0.023 in. (0.58 mm), the *nominal strength* [*resistance*] shall be calculated as follows:

$$P_{nov} = 0.90 t_1 d'_w F_{u1} \quad (Eq.\ J4.4.2\text{-}2)$$

where

d'_w = Effective pull-over diameter determined in accordance with (a), (b), or (c) as follows:

(a) For a round head, hex head (Figure J4.4.2-1(1)), pancake screw washer head (Figure J4.4.2-1(2)), or hex washer head (Figure J4.4.2-1(3)) screw with an independent and solid steel washer beneath the screw head:

$$d'_w = d_h + 2t_w + t_1 \leq d_w \quad (Eq.\ J4.4.2\text{-}3)$$

where

t_w = Steel washer thickness

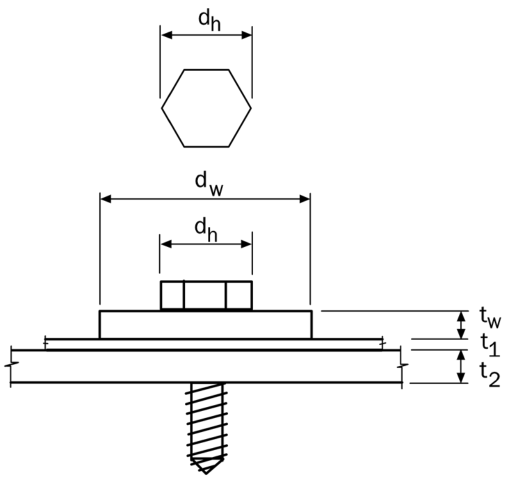
(b) For a round head, a hex head, or a hex washer head screw without an independent washer beneath the screw head:

$d'_w = d_h$ but not larger than 3/4 in. (19.1 mm)

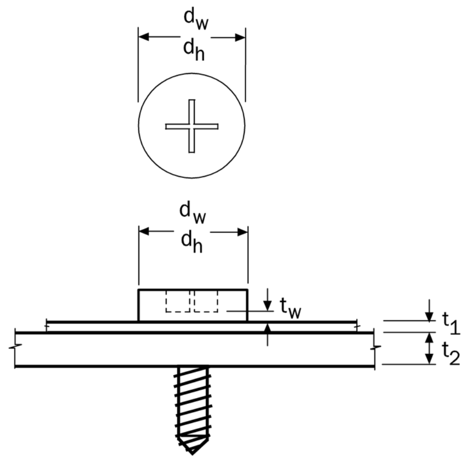
(c) For a domed (non-solid and either independent or integral) washer beneath the screw head (Figure J4.4.2-1(4)), it is permitted to use d'_w as calculated in Eq. J4.4.2-3, where t_w is the thickness of the domed washer. In the equation, d'_w shall not exceed 3/4 in. (19.1 mm).

The following *safety* and *resistance factors* shall be used to determine the *available strength* [*factored resistance*]:

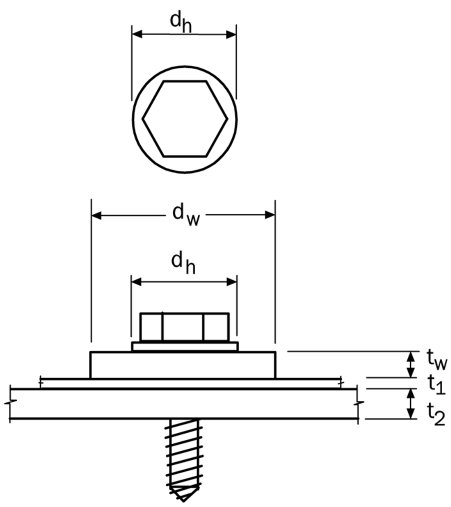
Ω = 2.90 (*ASD*)
ϕ = 0.55 (*LRFD*)
= 0.40 (*LSD*)



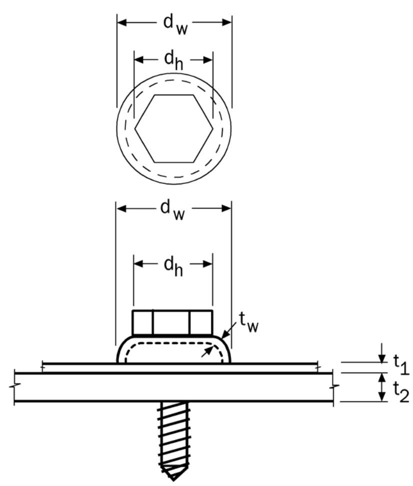
(1) Flat Steel Washer Beneath Hex Head Screw Head



(2) Pancake Screw Washer Head



(3) Flat Steel Washer Beneath Hex Washer Head Screw Head (HWH has Integral Solid Washer)



(4) Domed Washer (Non-Solid) Beneath Screw Head

Figure J4.4.2-1 Screw Pull-Over With Washer

J4.4.3 Tension in Screws

The *nominal tension strength* [*resistance*] of the screw shall be taken as P_{nts}.

The following *safety* and *resistance factors* shall be used to determine the *available strength* [*factored resistance*]:

Ω = 3.00 (*ASD*)
ϕ = 0.50 (*LRFD*)
= 0.40 (*LSD*)

Alternatively, the *safety factor* or the *resistance factor* is permitted to be determined in accordance with Section K2.1 and shall be taken as $1.25\Omega \le 3.0$ (*ASD*), $\phi/1.25 \ge 0.5$ (*LRFD*), or $\phi/1.25 \ge 0.4$ (*LSD*).

J4.5 Combined Shear and Tension

J4.5.1 Combined Shear and Pull-Over

For a screw *connection* subjected to combined shear and pull-over, the *required shear strength* [shear due to *factored loads*], $\overline{V}$, and *required tension strength* [tension due to *factored loads*], $\overline{T}$, shall not exceed the corresponding *available strength* [*factored resistance*] determined by Sections J4.3 and J4.4, respectively.

In addition, the following requirements shall be met:

$$\frac{\overline{V}}{P_{nv}} + 0.71\frac{\overline{T}}{P_{nov}} \le \frac{1.10}{\Omega} \qquad (ASD) \qquad (Eq.\ J4.5.1\text{-}1a)$$

$$\frac{\overline{V}}{P_{nv}} + 0.71\frac{\overline{T}}{P_{nov}} \le 1.10\phi \qquad (LRFD,\ LSD) \qquad (Eq.\ J4.5.1\text{-}1b)$$

where

$\overline{V}$ = *Required shear strength* [shear force due to *factored loads*] per *connection* screw, determined in accordance with *ASD*, *LRFD*, or *LSD load combinations*

$\overline{T}$ = *Required tension strength* [tensile force due to *factored loads*] per *connection* screw, determined in accordance with *ASD*, *LRFD*, or *LSD load combinations*

P_{nv} = *Nominal shear strength* [*resistance*] of sheet per screw

= $2.7t_1dF_{u1}$ (*Eq*. J4.5.1-2)

P_{nov} = *Nominal pull-over strength* [*resistance*] of sheet per screw

= $1.5t_1d_wF_{u1}$ (*Eq*. J4.5.1-3)

where

d_w = Larger of screw head diameter or washer diameter

Ω = 2.35 (*ASD*)

ϕ = 0.65 (*LRFD*)

= 0.55 (*LSD*)

Eq. J4.5.1-1 shall be valid for *connections* that meet the following limits:

(a) 0.0285 in. (0.724 mm) $\le t_1 \le$ 0.0445 in. (1.13 mm),
(b) No. 12 and No. 14 self-drilling screws with or without washers,
(c) $d_w \le$ 0.75 in. (19.1 mm),
(d) Washer dimension limitations of Section J4.4 apply,
(e) $F_{u1} \le$ 70 ksi (483 MPa or 4920 kg/cm^2), and
(f) $t_2/t_1 \ge 2.5$.

For eccentrically loaded *connections* that produce a nonuniform pull-over force on the screw, the *nominal pull-over strength* [*resistance*] shall be taken as 50 percent of P_{nov}.

J4.5.2 Combined Shear and Pull-Out

For a screw *connection* subjected to combined shear and pull-out, the *required shear strength* [shear due to *factored loads*], $\overline{V}$, and *required tension strength* [tension due to *factored loads*], $\overline{T}$, shall not exceed the corresponding *available strength* [*factored resistance*] determined by Sections J4.3 and J4.4, respectively.

In addition, the following requirement shall be met:

$$\frac{\overline{V}}{P_{nv}} + \frac{\overline{T}}{P_{not}} \le \frac{1.15}{\Omega} \qquad (ASD) \qquad (Eq.\ J4.5.2\text{-}1a)$$

$$\frac{\overline{V}}{P_{nv}} + \frac{\overline{T}}{P_{not}} \le 1.15\phi \qquad (LRFD,\ LSD) \qquad (Eq.\ J4.5.2\text{-}1b)$$

where

P_{nv} = *Nominal shear strength* [*resistance*] of sheet per screw

= $4.2(t_2^3 d)^{1/2} F_{u2}$ (*Eq*. J4.5.2-2)

P_{not} = *Nominal pull-out strength* [*resistance*] of sheet per screw

= $0.85 t_c d F_{u2}$ (*Eq*. J4.5.2-3)

Ω = 2.55 (*ASD*)

ϕ = 0.60 (*LRFD*)

= 0.50 (*LSD*)

Other variables are as defined in Section J4.5.1.

Eq. J4.5.2-1 shall be valid for *connections* that meet the following limits:

(a) 0.0297 in. (0.754 mm) $\le t_2 \le$ 0.0724 in. (1.84 mm),

(b) No. 8, 10, 12, or 14 self-drilling screws with or without washers,

(c) $F_{u2} \le$ 121 ksi (834MPa or 8510 kg/cm^2), and

(d) $1.0 \le F_u/F_y \le 1.62$.

J4.5.3 Combined Shear and Tension in Screws

For screws subjected to a combination of shear and tension forces, the *required shear strength* [shear due to *factored loads*], $\overline{V}$, and *required tension strength* [tension due to *factored loads*], $\overline{T}$, shall not exceed the corresponding *available strength* [*factored resistance*] determined by Sections J4.3.3 and J4.4.3, respectively.

In addition, the following requirement shall be met:

$$\frac{\overline{V}}{P_{nvs}} + \frac{\overline{T}}{P_{nts}} \le \frac{1.3}{\Omega} \qquad (ASD) \qquad (Eq.\ J4.5.3\text{-}1a)$$

$$\frac{\overline{V}}{P_{nvs}} + \frac{\overline{T}}{P_{nts}} \le 1.3\phi \qquad (LRFD,\ LSD) \qquad (Eq.\ J4.5.3\text{-}1b)$$

where

$\overline{V}$ = *Required shear strength* [shear force due to *factored loads*], determined in accordance with *ASD*, *LRFD,* or *LSD load combinations*

$\overline{T}$ = *Required tension strength* [tensile force due to *factored loads*], determined in accordance with *ASD*, *LRFD,* or *LSD load combinations*

P_{nvs} = *Nominal shear strength* [*resistance*] of screw as reported by manufacturer or determined by independent laboratory testing

P_{nts} = *Nominal tension strength* [*resistance*] of screw as reported by manufacturer or determined by independent laboratory testing

Ω = 3.00 (*ASD*)

ϕ = 0.50 (*LRFD*)

= 0.40 (*LSD*)

J5 Power-Actuated Fastener (PAF) Connections

The provisions of this section shall apply to steel-to-steel *PAF connections* within specified limitations. The steel *thickness* of the substrate not in contact with the *PAF* head shall be limited to a maximum of 0.75 in. (19.1 mm). The steel *thickness* of the substrate in contact with the *PAF* head shall be limited to a maximum of 0.06 in. (1.52 mm). The washer diameter shall not exceed 0.6 in. (15.2 mm) in computations, although the actual diameter may be larger. The *PAF* shall be produced from heat-treated carbon or stainless steel, hardened to HRC_p ranging from 49 to 61. The *PAF* diameter shall be limited to a range of 0.106 in. (2.69 mm) to 0.206 in. (5.23 mm).

For *diaphragm* applications, the provisions of Section I2 shall be used.

Alternatively, the *available strengths* [*factored resistances*] for any particular application are permitted to be determined through independent laboratory testing, with the *resistance factors*, ϕ, and *safety factors*, Ω, determined in accordance with Section K2. The values of P_{ntp} and P_{nvp} are permitted to be reported by the manufacturer.

The following notation shall apply to Section J5:

a = Major diameter of tapered *PAF* head
d = Fastener diameter measured at near side of embedment
= d_s for *PAF* installed such that entire point is located behind far side of embedment material
d_{ae} = Average embedded diameter, computed as average of installed fastener diameters measured at near side and far side of embedment material
= d_s for *PAF* installed such that entire point is located behind far side of embedment material
d_s = Nominal shank diameter
d'_w = Actual diameter of washer or fastener head in contact with retained substrate
≤ 0.60 in. (15.2 mm) in computation
E = Modulus of elasticity of steel
F_{bs} = Base *stress* parameter
= 66,000 psi (455 MPa or 4640 kg/cm^2)
F_{u1} = *Tensile strength* of member in contact with *PAF* head or washer
F_{uh} = *Tensile strength* of hardened *PAF* steel
F_{y2} = *Yield stress* of member not in contact with *PAF* head or washer
HRC_p = Rockwell C hardness of *PAF* steel
ℓ_{dp} = *PAF* point length. See Figure J5-1
P_{nb} = *Nominal bearing* and *tilting strength* [*resistance*] per *PAF*
P_{nos} = *Nominal pull-out strength* [*resistance*] in shear per *PAF*
P_{not} = *Nominal pull-out strength* [*resistance*] in tension per *PAF*
P_{nov} = *Nominal pull-over strength* [*resistance*] per *PAF*
P_{ntp} = *Nominal tensile strength* [*resistance*] of *PAF*
P_{nvp} = *Nominal shear strength* [*resistance*] of *PAF*
t_1 = *Thickness* of member in contact with *PAF* head or washer
t_2 = *Thickness* of member not in contact with *PAF* head or washer

t_w = Steel washer thickness

Various fastener dimensions used throughout Section J5 are shown in Figure J5-1.

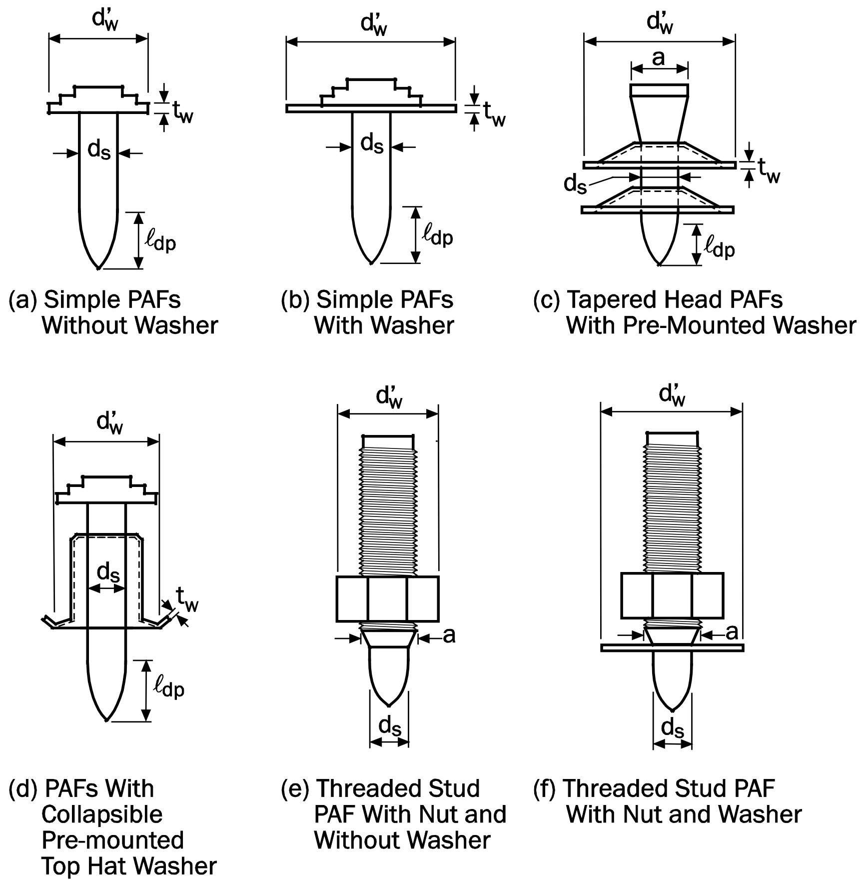


Figure J5-1 Geometric Variables in Power-Actuated Fasteners (PAFs)

J5.1 Minimum Spacing, Edge and End Distances

The minimum center-to-center spacing of the *power-actuated fasteners* (*PAFs*) and the minimum distance from the center of the fastener to any edge of the steel connected part, regardless of the direction of the force, shall be as provided by Table J5.1-1 and shall satisfy the requirements of Section J6.

Table J5.1-1
Minimum Required Edge and Spacing Distances in Steel

PAF Shank Diameter, d_s, in. (mm)	Minimum PAF Spacing in. (mm)	Minimum Edge Distance in. (mm)
0.106 (2.69) ≤ d_s< 0.200 (5.08)	1.00 (25.4)	0.50 (12.7)
0.200 (5.08) ≤ d_s< 0.206 (5.23)	1.60 (40.6)	1.00 (25.4)

J5.2 Power-Actuated Fasteners (PAFs) in Tension

The *available tensile strength* [*factored resistance*] per *PAF* shall be the minimum of the *available strengths* [*factored resistance*] determined by the applicable Sections J5.2.1 through J5.2.3. The washer thickness, t_w, limitations of Section J4 shall apply, except that for tapered head fasteners, the minimum thickness, t_w, shall not be less than 0.039 in. (0.991 mm). The thickness of collapsible pre-mounted top-hat washers shall not exceed 0.020 in. (0.508 mm).

J5.2.1 Tension Strength of Power-Actuated Fasteners (PAFs)

The *nominal tension strength* [*resistance*] of *PAFs*, P_{ntp}, is permitted to be calculated in accordance with Eq. J5.2.1-1, and the following *safety factor* or *resistance factors* shall be applied to determine the *available strength* [*factored resistance*] in accordance with Section B3.2.1, B3.2.2, or B3.2.3:

$$P_{ntp} = \left(d/2\right)^2 \pi F_{uh} \qquad (Eq.\ J5.2.1\text{-}1)$$

Ω = 2.65 (*ASD*)
ϕ = 0.60 (*LRFD*)
= 0.50 (*LSD*)

F_{uh} in Eq. J5.2.1-1 shall be calculated with Eq. J5.2.1-2. Alternatively, for fasteners with HRC_p of 52 or more, F_{uh} is permitted to be taken as 260,000 psi (1790 MPa).

$$F_{uh} = F_{bs} e^{(HRC_p/40)} \qquad (Eq.\ J5.2.1\text{-}2)$$

where
e = 2.718

J5.2.2 Pull-Out Strength

The *nominal pull-out strength* [*resistance*], P_{not}, shall be determined through independent laboratory testing with the *safety factor* or the *resistance factor* determined in accordance with Section K2. Alternatively, for *connections* with the entire *PAF point* length, ℓ_{dp}, below t_2, the following *safety factor* or *resistance factors* are permitted to be applied to the average of the test results. The result is used as the *available strength* [*factored resistance*] in accordance with Section B3.2.1, B3.2.2, or B3.2.3:

Ω = 4.00 (*ASD*)
ϕ = 0.40 (*LRFD*)
= 0.30 (*LSD*)

J5.2.3 Pull-Over Strength

The *nominal pull-over strength* [*resistance*], P_{nov}, is permitted to be computed in accordance with Eq. J5.2.3-1, and the following *safety factor* or *resistance factors* shall be applied to determine the *available strength* [*factored resistance*] in accordance with Section B3.2.1, B3.2.2, or B3.2.3:

$$P_{nov} = \alpha_w t_1 d'_w F_{u1} \qquad (Eq.\ J5.2.3\text{-}1)$$

Ω = 3.00 (*ASD*)
ϕ = 0.50 (*LRFD*)
= 0.40 (*LSD*)

where

α_w = 1.5 for screw-, bolt-, nail-like flat heads or simple *PAF*, with or without head washers (see Figures J5-1(a) and J5-1(b))

= 1.5 for threaded stud *PAFs* and for *PAFs* with tapered standoff heads that achieve pull-over by friction and locking of the pre-mounted washer (see Figure J5-1(c)), with a/d_s ratio of no less than 1.6 and $(a - d_s)$ of no less than 0.12 in. (3.1 mm)

= 1.25 for threaded stud *PAFs* and for *PAFs* with tapered standoff heads that achieve pull-over by friction and locking of pre-mounted washer (see Figure J5-1(c)), with a/d_s ratio of no less than 1.4 and $(a - d_s)$ of no less than 0.08 in. (2.0 mm)

= 2.0 for *PAFs* with collapsible spring washer (see Figure J5-1(d))

J5.3 Power-Actuated Fasteners (PAFs) in Shear

The *available shear strength* [*factored resistance*] shall be the minimum of the *available strengths* [*factored resistances*] determined by the applicable Sections J5.3.1 through J5.3.5.

J5.3.1 Shear Strength of Power-Actuated Fasteners (PAFs)

The *nominal shear strength* [*resistance*] of *PAFs*, P_{nvp}, is permitted to be computed in accordance with Eq. J5.3.1-1, and the *safety factor* and *resistance factors* shall be applied to determine the *available strength* [*factored resistance*] in accordance with Section B3.2.1, B3.2.2, or B3.2.3:

$$P_{nvp} = 0.6(d/2)^2 \pi F_{uh} \qquad (Eq.\ J5.3.1\text{-}1)$$

Ω = 2.65 (*ASD*)

ϕ = 0.60 (*LRFD*)

= 0.55 (*LSD*)

where

F_{uh} is determined in accordance with Section J5.2.1.

J5.3.2 Bearing and Tilting Strength

For *PAFs* embedded such that the entire length of *PAF point* length, ℓ_{dp}, is below t_2, the *nominal bearing and tilting strength* [*resistance*], P_{nb}, is permitted to be computed in accordance with Eq. J5.3.2-1, and the following *safety factor* or *resistance factors* shall be applied to determine the *available strength* [*factored resistance*] in steel in accordance with Section B3.2.1, B3.2.2, or B3.2.3:

$$P_{nb} = \alpha_b d_s t_1 F_{u1} \qquad (Eq.\ J5.3.2\text{-}1)$$

Ω = 2.05 (*ASD*)

ϕ = 0.80 (*LRFD*)

= 0.65 (*LSD*)

where

α_b = 3.7 for *connections* with *PAF* types as shown in Figures J5-1(c) and J5-1(d)

= 3.2 for other types of *PAFs*

Eq. J5.3.2-1 shall apply for *connections* within the following limits:

(a) $t_2/t_1 \geq 2$,

(b) $t_2 \geq 1/8$ in. (3.18 mm), and

(c) 0.146 in. (3.71 mm) $\leq d_s \leq$ 0.177 in. (4.50 mm).

J5.3.3 Pull-Out Strength in Shear

For *PAFs* driven in steel through a depth of at least $0.6t_2$, the *nominal pull-out strength* [*resistance*], P_{nos}, in shear is permitted to be computed in accordance with Eq. J5.3.3-1, and the following *safety factor* and the *resistance factors* shall be applied to determine the *available strength* [*factored resistance*] in accordance with Section B3.2.1, B3.2.2, or B3.2.3:

$$P_{nos} = \frac{d_{ae}^{1.8} t_2^{\,0.2} \left(F_{y2} E^2\right)^{1/3}}{30} \qquad (Eq.\ J5.3.3\text{-}1)$$

Ω = 2.55 (*ASD*)

ϕ = 0.60 (*LRFD*)

= 0.50 (*LSD*)

Eq. J5.3.3-1 shall apply for *connections* within the following limits:

(a) 0.113 in. (2.87 mm) $\leq t_2 \leq$ 3/4 in. (19.1 mm), and

(b) 0.106 in. (2.69 mm) $\leq d_s \leq$ 0.206 in. (5.23 mm).

J5.3.4 Net Section Rupture Strength

The *available strength* [*factored resistance*] due to net cross-section rupture and block shear shall be determined in accordance with Section J6. In computations of net section rupture and block shear limit states, the hole size shall be taken as 1.10 times the nominal *PAF* shank diameter, d_s.

J5.3.5 Shear Strength Limited by Edge Distance

The *available shear strength* [*factored resistance*] limited by edge distance shall be computed in accordance with Section J6.1 and the applicable *safety factor* or the *resistance factors* provided in Table J6-1 shall be applied to determine the *available strength* [*factored resistance*] in accordance with Section B3.2.1, B3.2.2, or B3.2.3. The consideration of edge distance shall be based upon nominal shank diameter, d_s.

J5.4 Combined Shear and Tension

Effects of combined shear and tension on the *PAF connection*, including the interaction due to combined shear and pull-out, combined shear and pull-over, and combined shear and tension on the *PAF*, shall be considered in design.

J6 Rupture

The provisions of this section shall apply to steel-to-steel welded, bolted, screw, and *power-actuated fastener* (*PAF*) *connections* within specified limitations. The design criteria of this section shall apply where the *thickness* of the thinnest connected part is 3/16 in. (4.76 mm) or less. For *connections* where the *thickness* of the thinnest connected part is greater than 3/16 in. (4.76 mm), the following specifications and standards shall apply:

(a) ANSI/AISC 360 for the United States and Mexico, and

(b) CSA S16 for Canada

For *connection* types utilizing welds, bolts, or *PAFs*, the *nominal rupture strength* [*resistance*], R_n, shall be the smallest of the values obtained in accordance with Sections J6.1, J6.2, and J6.3, as applicable. For *connection* types utilizing screws and *PAFs*, the *nominal rupture strength* [*resistance*], R_n, shall be the lesser of the values obtained in accordance with Sections J6.1 and J6.2, as applicable.

The corresponding *safety factor* and *resistance factors* given in Table J6-1 shall be applied to determine the *allowable strength* or *design strength* [*factored resistance*] in accordance with the applicable design method in Section B3.2.1, B3.2.2, or B3.2.3.

Table J6-1
Safety Factors and Resistance Factors for Rupture

Connection Type	Ω (ASD)	ϕ (LRFD)	ϕ (LSD)
Welds	2.50	0.60	0.75
Bolts, Screws, and Power Actuated Fasteners	2.00	0.75	0.75

J6.1 Shear Rupture

The *nominal shear rupture strength* [*resistance*], P_{nv}, shall be calculated in accordance with Eq. J6.1-1 except for lap *connections* with a single bolt, which shall be designed in accordance with Section J3.5.

$$P_{nv} = 0.6\, F_u\, A_{nv} \qquad (Eq.\ J6.1\text{-}1)$$

where

F_u = *Tensile strength* of connected part as specified in Section A3.1 or A3.2

A_{nv} = *Net area* subject to shear (parallel to force):

For a *connection* where each individual fastener pulls through the material towards the limiting edge individually:

$$A_{nv} = 2n\, t\, e_{net} \qquad (Eq.\ J6.1\text{-}2)$$

where

n = Number of fasteners on critical cross-section

t = Base steel *thickness* of section

e_{net} = Clear distance between the end of material and edge of fastener hole or weld

For a beam-end connection where one or more of the *flanges* are coped:

$$A_{nv} = (h_{wc} - n_b d_h)t \qquad (Eq.\ J6.1\text{-}3)$$

where

h_{wc} = Coped flat *web* depth

n_b = Number of fasteners along failure path being analyzed

d_h = Diameter of hole

t = *Thickness* of coped *web*

J6.2 Tension Rupture

The *nominal tensile rupture strength* [*resistance*], P_{nt}, shall be calculated in accordance with Eq. J6.2-1.

$$P_{nt} = F_u A_e \quad (Eq.\ J6.2\text{-}1)$$

where

A_e = Effective *net area* subject to tension

$$= U_{s\ell} A_{nt} \quad (Eq.\ J6.2\text{-}2)$$

where

$U_{s\ell}$ = Shear lag factor determined in Table J6.2-1

A_{nt} = *Net area* subject to tension (perpendicular to force), except as noted in Table J6.2-1

$$= A_g - n_b d_h t + t\Sigma[s'^2/(4g + 2d_h)] \quad (Eq.\ J6.2\text{-}3)$$

where

A_g = *Gross area* of member

s' = Longitudinal center-to-center spacing of any two consecutive holes

g = Transverse center-to-center spacing between fastener gage lines

n_b = Number of fasteners along failure path being analyzed

d_h = Diameter of a standard hole

t = Base steel *thickness* of section

F_u = *Tensile strength* of connected part as specified in Section A3.1 or A3.2

Table J6.2-1
Shear Lag Factors for Connections to Tension Members

Description of Element	Shear Lag Factor, $U_{s\ell}$
(1) For flat sheet *connections* not having staggered hole patterns	$U_{s\ell} = 0.9 + 0.1\ d/s$ (*Eq*. J6.2-4)
(2) For flat sheet *connections* having staggered hole patterns	$U_{s\ell} = 1.0$
(3) For other than flat sheet *connections*	
(a) When load is transmitted only by transverse welds	$U_{s\ell} = 1.0$ and A_{nt} = Area of the directly connected elements
(b) When load is transmitted directly to all the cross-sectional elements	$U_{s\ell} = 1.0$
(c) For *connections* of angle members not meeting (a) or (b) above	For a welded angle: $U_{s\ell} = 1.0 - 1.20\ \bar{x}/L \leq 0.9$ (*Eq*. J6.2-5) but $U_{s\ell}$ shall not be less than 0.4. For a bolted angle: $U_{s\ell} = \dfrac{1}{1.1 + \dfrac{0.5b_1}{b_2 + b_1} + \dfrac{2\bar{x}}{L}}$ (*Eq*. J6.2-6)
(d) For *connections* of channel members not meeting (a) or (b) above	For a welded channel: $U_{s\ell} = 1.0 - 0.36\ \bar{x}/L \leq 0.9$ (*Eq*. J6.2-7) but $U_{s\ell}$ shall not be less than 0.5. For a bolted channel: $U_{s\ell} = \dfrac{1}{1.1 + \dfrac{b_f}{b_w + 2b_f} + \dfrac{\bar{x}}{L}}$ (*Eq*. J6.2-8)

The variables in Table J6.2-1 shall be defined as follows:

$\bar{x}$ = Distance from shear plane to centroid of cross-section
L = Length of longitudinal weld or length of *connection*
s = Width of tensile rupture section divided by number of bolt holes in cross-section
d = Nominal bolt diameter
b_1 = Out-to-out width of angle leg not connected
b_2 = Out-to-out width of angle leg connected
b_f = Out-to-out width of *flange* not connected
b_w = Out-to-out width of *web* connected

J6.3 Block Shear Rupture

The *nominal block shear rupture strength* [*resistance*], P_{nr}, shall be determined from the following:

$$P_{nr} = 0.6F_y A_{av} + U_{s\ell}\, U_{bs}\, F_u\, A_{pt} + A_{st}\sqrt{(0.6F_y \cos\alpha)^2 + (F_u \sin\alpha)^2} \quad (Eq.\ J6.3\text{-}1)$$

where

A_{av} = Active area subject to shear (parallel to force)

= $(A_{gv} + A_{nv})/2$ (*Eq.* J6.3-2a)

= $n_{sh}L_{av}t$ (*Eq.* J6.3-2b)

where

A_{gv} = *Gross area* subject to shear (parallel to force)

A_{nv} = *Net area* subject to shear (parallel to force)

n_{sh} = Number of shear planes in the block

L_{av} = $L_{gv} - (2n_f - 1)\, d_h/4$ (*Eq.* J6.3-3)

where

L_{gv} = Distance from free edge to centerline of bolt farthest from edge measured along line of shear failure

n_f = Number of rows of bolts

d_h = Diameter of standard hole

t = Base steel *thickness* of the section

$U_{s\ell}$ = $0.9 + 0.1d/g$ for *net area* subject to tension (perpendicular to force) (*Eq.* J6.3-4)

where

d = Nominal bolt diameter

g = Transverse center-to-center spacing between fastener gage lines (perpendicular to force)

U_{bs} = Nonuniform block shear factor

= 0.5 for coped beam shear conditions with more than one vertical row of connectors

= 1.0 for all other cases

F_u = *Tensile strength* of connected part as specified in Section A3.1 or A3.2

A_{pt} = *Net area* subject to tension perpendicular to force

A_{st} = Active area in the staggered plane

= $n_{st}\, L_{st}\, t$ (*Eq.* J6.3-5)

where

L_{st} = Length of rupture in the staggered plane

$$= \sqrt{s'^2 + g^2} - d_h + \frac{d_h}{2} 10^{-4\sin\alpha} \qquad (Eq.\ J6.3\text{-}6)$$

α = Angle between the line of the force and the centerline of the staggered hole

= arctan (g/s′)

n_{st} = Number of staggered planes in the block

s′ is as defined in Section J6.2.

F_y = *Yield stress* of connected part as specified in Section A3.1 or A3.2

J7 Connections to Other Materials

In bolted, screw, and *power-actuated fastener connections,* the *available strength* [*factored resistance*] of the *connection* to other materials shall be determined in accordance with Section J7.1.

J7.1 Strength of Connection to Other Materials

J7.1.1 Bearing

Provisions shall be made to transfer bearing forces from steel components covered by this *Specification* to adjacent *structural components* made of other materials.

J7.1.2 Tension

The pull-over shear or tension forces in the steel sheet around the head of the fastener shall be considered, as well as the pull-out force resulting from axial *loads* and bending moments transmitted onto the fastener from various adjacent *structural components* in the assembly.

The *nominal tensile strength* [*resistance*] of the fastener and the *nominal embedment strength* [*resistance*] of the adjacent structural component shall be determined by applicable product code approvals, product specifications, product literature, or combination thereof.

J7.1.3 Shear

Provisions shall be made to transfer shearing forces from steel components covered by this *Specification* to adjacent structural components made of other materials. The *required shear* and/or *bearing strength* [shear or bearing force due to *factored loads*] on the steel components shall not exceed that allowed by this *Specification*. The *available shear strength* [*factored resistance*] on the fasteners and other material shall not be exceeded. Embedment requirements shall be met. Provisions shall also be made for shearing forces in combination with other forces.

K. STRENGTH FOR SPECIAL CASES

This chapter addresses determination of member and *connection* strengths through testing.

The chapter is organized follows:

K1 Test Standards

K2 Tests for Special Cases

K1 Test Standards

The following test standards are permitted to be used to determine the strength, flexibility, or *stiffness* of cold-formed steel members and *connections* via testing:

ANSI/SDI AISI S901, *Test Standard for Determining the Rotational-Lateral Stiffness of Beam-to-Panel Assemblies*

ANSI/SDI AISI S902, *Test Standard for Determining the Effective Area of Cold-Formed Steel Compression Members*

ANSI/SDI AISI S903, *Test Standard for Determining the Uniform and Local Ductility of Carbon and Low-Alloy Steels*

ANSI/SDI AISI S904, *Test Standard for Determining the Tensile and Shear Strengths of Steel Screws*

ANSI/SDI AISI S905, *Test Standard for Determining the Strength and Deformation Characteristics of Cold-Formed Steel Connections*

ANSI/SDI AISI S906, *Test Standard for Determining the Load-Carrying Strength of Panels and Anchor-to-Panel Attachments for Roof or Siding Systems Tested in Accordance With ASTM E1592*

ANSI/SDI AISI S907, *Test Standard for Determining the Strength and Stiffness of Cold-Formed Steel Diaphragms by the Cantilever Test Method*

ANSI/SDI AISI S908, *Test Standard for Determining the Flexural Strength Reduction Factor of Purlins Supporting a Standing Seam Roof System*

ANSI/SDI AISI S909, *Test Standard for Determining the Web Crippling Strength of Cold-Formed Steel Flexural Members*

ANSI/SDI AISI S910, *Test Standard for Determining the Distortional Buckling Strength of Cold-Formed Steel Hat-Shaped Compression Members*

ANSI/SDI AISI S911, *Test Standard for Determining the Flexural Strength of Cold-Formed Steel Hat-Shaped Members*

ANSI/SDI AISI S912, *Test Standard for Determining the Strength of a Roof Panel-to-Purlin-to-Anchorage Device Connection*

ANSI/SDI AISI S913, *Test Standard for Determining the Strength and Deformation Behavior of Hold-Downs Attached to Cold-Formed Steel Structural Framing*

ANSI/SDI AISI S914, *Test Standard for Determining the Strength and Deformation Behavior of Joist Connectors Attached to Cold-Formed Steel Structural Framing*

ANSI/SDI AISI S915, *Test Standard for Through-the-Web Punchout Cold-Formed Steel Wall Stud Bridging Connectors*

ANSI/SDI AISI S916, *Test Standard for Determining the Strength and Stiffness of Cold-Formed Steel Framed Nonstructural Interior Partition Walls Sheathed With Gypsum Board*

ANSI/SDI AISI S917, *Test Standard for Determining the Fastener-Sheathing Local Translational Stiffness of Sheathed Cold-Formed Steel Assemblies*

ANSI/SDI AISI S918, *Test Standard for Determining the Fastener-Sheathing Rotational Stiffness of Sheathed Cold-Formed Steel Assemblies*

ANSI/SDI AISI S919, *Test Standard for Determining the Flexural Strength and Stiffness of Cold-Formed Steel Nonstructural Members*

ANSI/SDI AISI S920, *Test Standard for Determining the Screw Penetration Through Gypsum Board Into Cold-Formed Steel Framing Members*

ANSI/SDI AISI S921, *Test Standard for Determining the Strength and Serviceability of Cold-Formed Steel Truss Assemblies and Components*

ANSI/SDI AISI S922, *Test Standard for Determining the Strength and Stiffness of Bearing-Friction Interference Connector Assemblies in Profiled Steel Panels*

ANSI/SDI AISI S923, *Test Standard for Determining the Strength and Stiffness of Shear Connections of Composite Members*

ANSI/SDI AISI S924, *Test Standard for Determining the Effective Flexural Stiffness of Composite Members*

K2 Tests for Special Cases

Tests shall be made by an independent testing laboratory or by a testing laboratory of a manufacturer.

K2.1 Tests for Determining Structural Performance

K2.1.1 Load and Resistance Factor Design and Limit States Design

Any structural performance that is required to be established by tests in accordance with Section A1.2.6(a) or by *rational engineering analysis* with *confirmatory tests* in accordance with Section A1.2.6(b) shall be evaluated with the following performance procedure:

(a) Evaluation of the test results for use with Section A1.2.6(a) shall be made on the basis of the average value of test data resulting from tests of not fewer than three identical specimens, provided the deviation of any individual test result from the average value obtained from all tests does not exceed ±15 percent. If such deviation from the average value exceeds 15 percent, more tests of the same kind shall be made until the deviation of any individual test result from the average value obtained from all tests does not exceed ±15 percent or until at least three additional tests have been made. No test result shall be eliminated unless a rationale for its exclusion is given. The average value of all tests made shall then be regarded as the *nominal strength* [*resistance*], R_n, for the series of the tests. R_n and the coefficient of variation V_P of the test results shall be determined by statistical analysis.

(b) Evaluation of a *rational engineering analysis* model by *confirmatory* tests for use with Section A1.2.6(b): The correlation coefficient, C_c, between the tested *strength* [*resistance*] (R_t) and the *nominal strength* [*resistance*] (R_n) predicted from the *rational engineering analysis* model shall be greater than or equal to 0.80. Only one limit state is permitted for evaluation of the *rational engineering analysis* model being verified, and the test result shall reflect the *limit state* under consideration.

The *rational engineering analysis* model is only verified within parameters varied in the testing. Extrapolation outside of the tested parameters is not permitted. For each parameter being evaluated:

(1) All other parameters shall be held constant,

(2) The nominally selected values of the parameter to be tested shall not bias the study to a specific region of the parameter, and

(3) A minimum of three tests shall be performed. No test results shall be eliminated unless a rationale for their exclusion is given.

Dimensions and material properties shall be measured for all test specimens. The as-measured dimensions and properties shall be used in determination of the calculated *nominal strength* [*resistance*] ($R_{n,i}$) as employed in determining the *resistance factor* or *safety factor* in accordance with (c). The specified dimensions and properties shall be used in the determination of the calculated *nominal strength* [*resistance*] for design. The bias and variance between the as-measured dimensions and properties and the nominally specified dimensions and properties shall be reflected in the selected material (M_m, V_M) and fabrication (F_m, V_F) factors per Table K2.1.1-1. Otherwise, the selected values of M_m and F_m shall not be greater than in Table K2.1.1-1, and the values of V_M and V_F shall not be less than the values given in Table K2.1.1-1.

(c) The strength of the tested elements, assemblies, *connections*, or members shall satisfy Eq. K2.1.1-1a or Eq. K2.1.1-1b as applicable.

$$\Sigma\gamma_i Q_i \leq \phi R_n \text{ for } LRFD \quad (Eq.\ K2.1.1\text{-}1a)$$

$$\phi R_n \geq \Sigma\gamma_i Q_i \text{ for } LSD \quad (Eq.\ K2.1.1\text{-}1b)$$

where

$\Sigma\gamma_i Q_i$ = *Required strength* [effect of *factored loads*] based on the most critical *load* combination, determined in accordance with Section B2. γ_i and Q_i are *load factors* and *load effects*, respectively.

ϕ = *Resistance factor*

$$= C_\phi (M_m F_m P_m) e^{-\beta_o \sqrt{V_M^2 + V_F^2 + C_P V_P^2 + V_Q^2}} \quad (Eq.\ K2.1.1\text{-}2)$$

where

C_ϕ = Calibration coefficient
= 1.52 for *LRFD*
= 1.42 for *LSD*
= 1.6 for *LRFD* for beams having tension *flange* through-fastened to deck or sheathing and with compression *flange* laterally unbraced
= 1.42 for *LSD* for beams having tension *flange* through-fastened to deck or sheathing and with compression *flange* laterally unbraced

M_m = Mean value of material factor, M, determined by statistical analysis or, where applicable, as limited by Table K2.1.1-1 for type of component involved

F_m = Mean value of fabrication factor, F, determined by statistical analysis or where applicable, as limited by Table K2.1.1-1 for type of component involved

P_m = Mean value of professional factor, P, for tested component
= 1.0, if the *available strength* [*factored resistance*] is determined in accordance with Section K2.1.1(a); or

$$= \frac{\sum_{i=1}^{n} \frac{R_{t,i}}{R_{n,i}}}{n} \quad (Eq.\ K2.1.1\text{-}3)$$

, when the *available strength* [*factored resistance*] is determined in accordance with Section K2.1.1(b)

where
i = Index of tests
= 1 to n
n = Total number of tests
$R_{t,i}$ = Tested *strength* [*resistance*] of test i
$R_{n,i}$ = Calculated *nominal strength* [*resistance*] of test i per *rational engineering analysis* model

e = Natural logarithmic base
= 2.718
β_o = Target reliability index
= 2.5 for *structural members* and 3.5 for *connections* for *LRFD*
= 3.0 for *structural members* and 4.0 for *connections* for *LSD*
= 1.5 for *LRFD* for beams having tension *flange* through-fastened to deck or sheathing and with compression *flange* laterally unbraced
= 3.0 for *LSD* for beams having tension *flange* through-fastened to deck or sheathing and with compression *flange* laterally unbraced
V_M = Coefficient of variation of material factor listed in Table K2.1.1-1 for type of component involved
V_F = Coefficient of variation of fabrication factor listed in Table K2.1.1-1 for type of component involved
C_P = Correction factor
= $(1+1/n)m/(m-2)$ for $n \geq 4$ (*Eq*. K2.1.1-4)
= 5.7 for n = 3

where
n = Number of tests
m = Degrees of freedom
= n - 1

V_P = Coefficient of variation of test results, but not less than 0.065
= $\dfrac{s_t}{R_n}$, if the *available strength* [*factored resistance*] is determined in accordance with Section K2.1.1(a) or (*Eq*. K2.1.1-5)
= $\dfrac{s_c}{P_m}$, if the *available strength* [*factored resistance*] is determined in accordance with Section K2.1.1 (b) (*Eq*. K2.1.1-6)

where
s_t = Standard deviation of all of the test results
s_c = Standard deviation of $R_{t,i}$ divided by $R_{n,i}$ for all of the test results

V_Q = Coefficient of variation of *load effect*
= 0.21 for *LRFD* and *LSD*
= 0.43 for *LRFD* for beams having tension *flange* through-fastened to deck or sheathing and with compression *flange* laterally unbraced
= 0.21 for *LSD* for beams having tension *flange* through-fastened to deck or sheathing and with compression *flange* laterally unbraced ☜B
C_c = Correlation coefficient

$$= \frac{n\sum R_{t,i}R_{n,i} - \left(\sum R_{t,i}\right)\left(\sum R_{n,i}\right)}{\sqrt{n\left(\sum R_{t,i}^2\right) - \left(\sum R_{t,i}\right)^2}\sqrt{n\left(\sum R_{n,i}^2\right) - \left(\sum R_{n,i}\right)^2}} \quad (Eq.\ K2.1.1\text{-}7)$$

R_n = Average value of all test results

The listing in Table K2.1.1-1 shall not exclude the use of other documented statistical data if they are established from sufficient results on material properties and fabrication.

For steels not listed in Section A3.1, the values of M_m and V_M shall be determined by the statistical analysis for the materials used.

When distortions interfere with the proper functioning of the specimen in actual use, the load effects based on the critical load combination at the occurrence of the acceptable distortion shall also satisfy Eq. K2.1.1-1a or Eq. K2.1.1-1b, as applicable, except that the *resistance factor*, ϕ, shall be taken as unity and the load factor for dead load shall be taken as 1.0.

(d) For strength determined in accordance with Section K2.1.1(a) or K2.1.1(b), the mechanical properties of the steel sheet shall be determined based on representative samples of the material taken from the test specimen or the flat sheet used to form the test specimen. Alternatively, for connectors or devices that are too small to obtain standard size or sub-size tensile specimens per ASTM A370, and are produced from steel sheet coils that have not undergone a secondary process to alter the mechanical or chemical properties, mechanical properties are permitted to be determined based on mill certificates, and the mean value of the material factor, M_m, shall be equal to 0.85. If the *yield stress* of the steel is larger than the specified value, the test results shall be adjusted down to the *specified minimum yield stress* of the steel that the manufacturer intends to use. The test results shall not be adjusted upward if the *yield stress* of the test specimen is less than the *specified minimum yield stress*. Similar adjustments shall be made on the basis of *tensile strength* instead of *yield stress* where *tensile strength* is the critical factor.

Consideration shall also be given to any variation or differences between the design *thickness* and the *thickness* of the specimens used in the tests.

TABLE K2.1.1-1
Statistical Data for the Determination of Resistance Factor

Type of Component	M_m	V_M	F_m	V_F
Members				
Tension	1.10	0.10	1.00	0.05
Compression	1.10	0.10	1.00	0.05
Flexure	1.10	0.10	1.00	0.05
Shear and *Web Crippling*	1.10	0.10	1.00	0.05
Under Combined Forces	1.05	0.10	1.00	0.05
Other Member Limit States[1]	1.00	0.10	1.00	0.05
Connections and Joints				
Welded *Connections*	1.10	0.10	1.00	0.10
Bolted *Connections*	1.10	0.08	1.00	0.05
Screw *Connections*	1.10	0.10	1.00	0.10
Shear Strength Limited by Tilting and *Bearing*	1.10	0.08	1.00	0.05
Power-Actuated Fasteners	1.10	0.10	1.00	0.10
Other Connectors or Fasteners[2]	1.10	0.10	1.00	0.15
Connections to Structural Concrete	1.10	0.10	0.90	0.10
Connections to Wood	1.10	0.15	1.00	0.15

Notes:

[1] For member limit states captured in testing but not covered in AISI S100.

[2] For steel-to-steel connectors and fasteners not already listed in the table.

K2.1.2 Allowable Strength Design

Where the composition or configuration of elements, assemblies, *connections*, or details of *cold-formed steel structural members* are such that calculation of their strength cannot be made in accordance with the provisions of this *Specification*, their structural performance shall be established from tests and evaluated in accordance with Section K2.1.1, except as modified in this section for *allowable strength design*.

The *allowable strength* shall be calculated as follows:

$$R_a = R_n/\Omega \qquad (Eq.\ K2.1.2\text{-}1)$$

where

R_n = Average value of all test results

Ω = *Safety factor*

$$= \frac{1.6}{\phi} \qquad (Eq.\ K2.1.2\text{-}2)$$

where

ϕ = A value evaluated in accordance with Section K2.1.1

The *required strength* shall be determined from *ASD load combinations* as described in Section B2.

K2.2 Tests for Confirming Structural Performance

For *structural members*, *connections*, and assemblies for which the *nominal strength* [*resistance*] is computed in accordance with this *Specification* or its specific references, *confirmatory tests* are permitted to be made to demonstrate the strength is not less than the *nominal strength* [*resistance*], R_n, specified in this *Specification* or its specific references for the type of behavior involved.

K2.3 Tests for Determining Mechanical Properties

K2.3.1 Full Section

Tests for determination of mechanical properties of full sections to be used in Section A3.3.2 shall be conducted in accordance with this section:

(a) Tensile testing procedures shall conform to the requirements of ASTM A370.

(b) Compressive *yield stress* determinations shall be made by means of compression tests of short specimens of the section. See ANSI/SDI AISI S902.

The compressive *yield stress* shall be taken as the smaller value of either the maximum compressive strength of the sections divided by the *cross-sectional area* or the *stress* defined by one of the following methods:

(1) For sharp-yielding steel, the *yield stress* is determined by the autographic diagram method or by the total strain under load method.

(2) For gradual-yielding steel, the *yield stress* is determined by the strain under load method or by the 0.2 percent offset method.

When the total strain under load method is used, there shall be evidence that the *yield stress* so determined is within five (5) percent with the *yield stress* that would be determined by the 0.2 percent offset method.

(c) Where the principal effect of the loading to which the member will be subjected in service will be to produce bending *stresses*, the *yield stress* shall be determined for the *flanges* only. In determining such *yield stress*, each specimen shall consist of one complete *flange* plus a portion of the *web* of such *flat width* ratio that the value of ρ for the specimen is unity.

(d) For acceptance and control purposes, one full section test shall be made from each *master coil*.

(e) At the option of the manufacturer, either tension or compression tests are permitted to be used for routine acceptance and control purposes, provided the manufacturer demonstrates that such tests reliably indicate the *yield stress* of the section when subjected to the kind of *stress* under which the member is to be used.

K2.3.2 Flat Elements of Formed Sections

Tests for determining mechanical properties of flat elements of formed sections and representative mechanical properties of *virgin steel* to be used in Section A3.3.2 shall be made in accordance with this section.

The *yield stress* of flats, F_{yf}, shall be established by means of a weighted average of the *yield stresses* of standard tensile coupons taken longitudinally from the flat portions of a representative cold-formed member. The weighted average shall be the sum of the products of the average *yield stress* for each flat portion times its *cross-sectional area*, divided by the total area of flats in the cross-section. Although the exact number of such coupons will depend on the shape of the member, i.e., on the number of flats in the cross-section, at least one tensile coupon shall be taken from the middle of each flat. If the actual virgin *yield stress* exceeds the *specified minimum yield stress*, the *yield stress* of the flats, F_{yf}, shall be adjusted by multiplying the test values by the ratio of the *specified minimum yield stress* to the actual virgin *yield stress*.

K2.3.3 Virgin Steel

The following provisions shall apply to steel produced to other than the ASTM Specifications listed in Section A3.1 when used in sections for which the increased *yield stress* of the steel after cold forming is computed from the *virgin steel properties* in accordance with Section A3.3.2. For acceptance and control purposes, at least four tensile specimens shall be taken from each *master coil* for the establishment of the representative values of the virgin tensile *yield stress* and *tensile strength*. Specimens shall be taken longitudinally from the quarter points of the width near the outer end of the coil.

L. DESIGN FOR SERVICEABILITY

This chapter addresses the serviceability determination using the *Effective Width Method* and *Direct Strength Method*, and flange curling.

The chapter is organized as follows:

L1 Serviceability Determination for the Effective Width Method
L2 Serviceability Determination for the Direct Strength Method
L3 Flange Curling

Reduced stiffness values used in the *direct analysis method*, described in Chapter C, are not intended for use with the provisions of this chapter.

L1 Serviceability Determination for the Effective Width Method

The bending deflection at any moment, M, due to *service loads* is permitted to be determined by using the effective moment of inertia, I_{eff}, determined in accordance with Appendix 1.

L2 Serviceability Determination for the Direct Strength Method

The bending deflection at any moment, M, due to *service loads* is permitted to be determined by reducing the gross moment of inertia, I_g, to an effective moment of inertia for deflection, as given in Eq. L2-1:

$$I_{eff} = I_g(M_d/M) \le I_g \qquad (Eq.\ L2\text{-}1)$$

where

M_d = *Nominal flexural strength* [*resistance*], M_n, defined in Chapter F with *Direct Strength Method*, but with M_y replaced by M in all equations

M = Moment due to *service loads* on member to be considered ($M \le M_y$)

L3 Flange Curling

Where the *flange* of a flexural member is unusually wide and it is desired to limit the maximum amount of curling or movement of the *flange* toward the neutral axis, Eq. L3-1 is permitted to be applied to compression and tension *flanges*, either stiffened or unstiffened, as follows:

$$w_f = \sqrt{0.061tdE/f_{av}}\sqrt[4]{(100c_f/d)} \qquad (Eq.\ L3\text{-}1)$$

where

w_f = Width of *flange* projecting beyond *web*, or half of distance between *webs* for box- or U-type beams

t = *Flange thickness*

d = Depth of beam

E = Modulus of elasticity of steel

f_{av} = Average *stress* in full unreduced *flange* width. (Where members are designed by the *effective design width* procedure, the average *stress* equals the maximum *stress* multiplied by the ratio of the *effective design width* to the actual width.)

c_f = Amount of curling displacement

M. DESIGN FOR FATIGUE

This chapter addresses *cold-formed steel structural members* and *connections* subject to cyclic loading within the elastic range of *stresses* of frequency and magnitude sufficient to initiate cracking and progressive failure, which defines the limit state of *fatigue*.

This chapter is organized as follows:

M1 General

M2 Calculation of Maximum Stresses and Stress Ranges

M3 Design Stress Range

M4 Bolts and Threaded Parts

M5 Special Fabrication Requirements

M1 General

When cyclic loading is a design consideration, the provisions of this chapter shall apply to *stresses* calculated on the basis of *ASD load combinations* [*specified loads*]. The maximum permitted tensile *stress* shall be 0.6 F_y.

Stress range shall be defined as the magnitude of the change in *stress* due to the application or removal of the live *load* [*specified* live *load*]. In the case of a *stress* reversal, the *stress* range shall be computed as the sum of the absolute values of maximum repeated tensile and compressive *stresses* or the sum of the absolute values of maximum shearing *stresses* of opposite direction at the point of probable crack initiation.

Fatigue need not be considered for seismic effects or for the effects of wind loading on typical building lateral force-resisting systems and building enclosure components. *Fatigue* need not be considered when the live *load* [*specified* live *load*] *stress* range is less than the threshold *stress* range, F_{TH}, given in Table M1-1.

Evaluation of *fatigue* strength [*resistance*] shall not be required if the number of cycles of application of live *load* [*specified* live *load*] is less than 20,000.

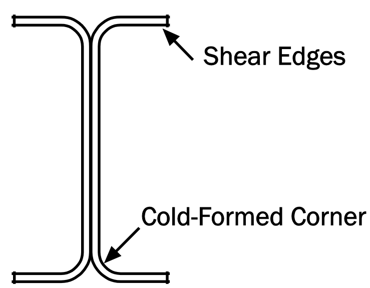
The *fatigue* strength [*resistance*] determined by the provisions of this chapter shall be applicable to structures with corrosion protection or subject only to non-aggressive atmospheres.

The *fatigue* strength [*resistance*] determined by the provisions of this chapter shall be applicable only to structures subject to temperatures not exceeding 300°F (149°C).

The contract documents shall either provide complete details including weld sizes, or specify the planned cycle life and the maximum range of moments, shears, and reactions for the *connections*.

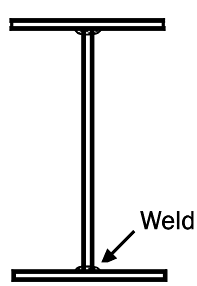
Table M1-1
Fatigue Design Parameters for Cold-Formed Steel Structures

Description	Stress Category	Constant C_f	Threshold F_{TH}, ksi (MPa) [kg/cm^2]	Reference Figure
As-received base metal and components with as-rolled surfaces, including sheared edges and cold-formed corners	I	$3.2x10^{10}$	25 (172) [1760]	M1-1
As-received base metal and weld metal in members connected by continuous longitudinal welds	II	$1.0x10^{10}$	15 (103) [1050]	M1-2
Welded attachments to a plate or a beam, transverse fillet welds, and continuous longitudinal fillet welds less than or equal to 2 in. (50.8 mm), bolt and screw *connections*, and spot welds	III	$3.2x10^9$	16 (110) [1120]	M1-3, M1-4
Longitudinal fillet-welded attachments exceeding 2 in. (50.8 mm) but less than or equal to 4 in. (101.6 mm) parallel to the direction of the applied stress, and intermittent welds parallel to the direction of the applied force	IV	$1.0x10^9$	9 (62) [633]	M1-3(b)



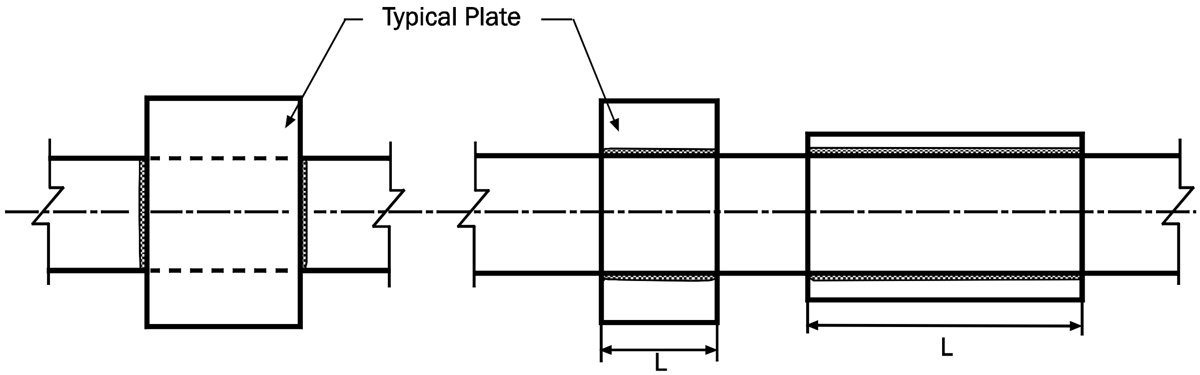
Cold-Formed Steel Channels, Stress Category I

Figure M1-1 Typical Detail for Stress Category I



Welded I Beam, Stress Category II

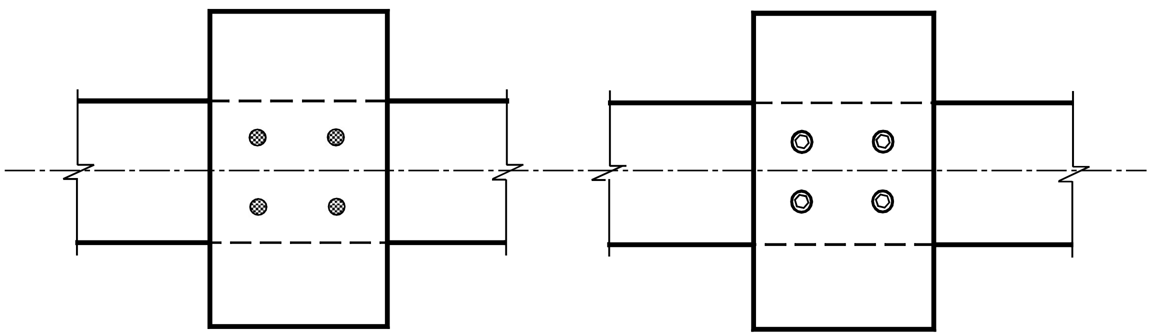
Figure M1-2 Typical Detail for Stress Category II



(a) Transverse Welds
Stress Category III

(b) Longitudinal Welds
For Stress Category III, $L \leq 2$ in. (50.8 mm)
For Stress Category IV, 2 in. (50.8 mm) $< L \leq 4$ in. (101.6 mm)

Figure M1-3 Typical Attachments for Stress Categories III and IV



(a) Arc Spot or Plug Weld

(b) Screws

Figure M1-4 Typical Attachments for Stress Category III

M2 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 applicable.

In the case of axial *stress* combined with bending, the maximum *stresses* of each kind shall be those determined for concurrent arrangements of 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 stressed 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.

M3 Design Stress Range

The range of *stress* shall not exceed the design *stress* range computed using Eq. M3-1 for all *stress* categories as follows:

$$F_{SR} = (\alpha C_f/N)^{0.333} \geq F_{TH} \quad (Eq.\ M3\text{-}1)$$

where

F_{SR} = Design *stress* range

α = Coefficient for conversion of units
= 1 for U.S. customary units
= 327 for SI units
= 352,000 for MKS units

C_f = Constant from Table M1-1

N = Number of *stress* range fluctuations in design life
= Number of *stress* range fluctuations per day × 365 × years of design life

F_{TH} = Threshold *fatigue stress* range, maximum *stress* range for indefinite design life from Table M1-1

M4 Bolts and Threaded Parts

For mechanically fastened *connections* loaded in shear, the maximum range of *stress* in the connected material shall not exceed the design *stress* range computed using Equation M3-1. The factor C_f shall be taken as 22×10^8. The threshold *stress*, F_{TH}, shall be taken as 7 ksi (48 MPa or 492 kg/cm^2).

For not-fully-tightened high-strength bolts, common bolts, and threaded anchor rods with cut, ground, or rolled threads, the maximum range of tensile *stress* on the net tensile area from applied axial *load* and moment plus *load* due to prying action shall not exceed the design *stress* range computed using Eq. M3-1. The factor C_f shall be taken as 3.9×10^8. The threshold *stress*, F_{TH}, shall be taken as 7 ksi (48 MPa or 492 kg/cm^2). The net tensile area shall be calculated by Eq. M4-1a or M4-1b as applicable.

$$A_t = (\pi/4)\,[d_b - (0.9743/n)]^2 \quad \text{for U.S. Customary units} \quad (Eq.\ M4\text{-}1a)$$

$$A_t = (\pi/4)\,[d_b - (0.9382p)]^2 \quad \text{for SI or MKS units} \quad (Eq.\ M4\text{-}1b)$$

where:

A_t = Net tensile area

d_b = Nominal diameter (body or shank diameter)

n = Number of threads per inch

p = Pitch (mm per thread for SI units and cm per thread for MKS units)

M5 Special Fabrication Requirements

Backing bars in welded *connections* that are parallel to the *stress* field are permitted to remain in place, and if used, shall be continuous.

Backing bars that are perpendicular to the *stress* field, if used, shall be removed and the *joint* back gouged and welded.

Flame-cut edges subject to cyclic *stress* ranges shall have a surface roughness not to exceed

1,000 μin. (25 μm) in accordance with ASME B46.1.

Re-entrant corners at cuts, copes, and weld access holes shall form a radius of not less than 3/8 in. (9.53 mm) by pre-drilling or sub-punching 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 contour to provide a radiused transition, free of notches, with a surface roughness not to exceed 1,000 μin. (25 μm) in accordance with ASME B46.1 or other equivalent approved standards.

For transverse butt *joints* in regions of high tensile *stress*, weld tabs shall be used to provide for cascading the weld termination outside the finished *joint*. End dams shall not be used. Weld tabs shall be removed and the end of the weld finished flush with the edge of the member. **Exception**: Weld tabs shall not be required for sheet material if the welding procedures used result in smooth, flush edges.

APPENDIX 1, EFFECTIVE WIDTH OF ELEMENTS

This appendix addresses the *Effective Width Method* for elements on cold-formed steel cross-sections subject to compression *stress*. The effective section properties are used to determine the member strengths and deflections.

This appendix is organized as follows:

1.1 Effective Width of Uniformly Compressed Stiffened Elements
1.2 Effective Width of Unstiffened Elements
1.3 Effective Width of Uniformly Compressed Elements With a Simple Lip Edge Stiffener
1.4 Effective Width of Stiffened Elements With Single or Multiple Intermediate Stiffeners or Edge-Stiffened Elements With Intermediate Stiffener(s)

1.1 Effective Width of Uniformly Compressed Stiffened Elements

(a) Strength Determination

The *effective width*, b, shall be calculated as follows:

$$b = \rho w \qquad (Eq.\ 1.1\text{-}1)$$

where

w = *Flat width* as shown in Figure 1.1-1

ρ = Local reduction factor

= 1 when $\lambda \le 0.673$

$$= (1 - 0.22/\lambda)/\lambda \quad \text{when } \lambda > 0.673 \qquad (Eq.\ 1.1\text{-}2)$$

where

λ = Slenderness factor

$$= \sqrt{\frac{f}{F_{cr\ell}}} \qquad (Eq.\ 1.1\text{-}3)$$

where

f = Compressive *stress* in element considered, which is computed as follows:

For flexural members:

(1) For *local buckling* interacting with yielding and global *buckling*, f is the *stress* in the compression element calculated based on the extreme compression fiber at F_n, or the extreme tension fiber at F_y, in accordance with Section F3.1.

(2) For inelastic reserve strength in accordance with Section F2.1.1, f is the *stress* in the compression element calculated based on the extreme compression or tension fiber at F_y.

For compression members, f is equal to F_n as determined in accordance with Chapter E.

$$F_{cr\ell} = k\frac{\pi^2 E}{12(1-\mu^2)}\left(\frac{t}{w}\right)^2 \qquad (Eq.\ 1.1\text{-}4)$$

where

k = Plate *buckling* coefficient

= 4 for stiffened elements supported by a *web* on each longitudinal edge. Values for different types of elements are given in the applicable sections.

E = Modulus of elasticity of steel

t = *Thickness* of uniformly compressed stiffened element
μ = Poisson's ratio of steel

(b) Serviceability Determination

The *effective width*, b_d, used in determining serviceability shall be calculated as follows:

$$b_d = \rho w \qquad (Eq.\ 1.1\text{-}5)$$

where
w = *Flat width*
ρ = Local reduction factor determined by either of the following two procedures:
(1) Procedure I:

A conservative estimate of the *effective width* is obtained from Section 1.1(a) by substituting f_d for f, where f_d is the computed compressive *stress* in the element being considered.

(2) Procedure II:

For stiffened elements supported by a *web* on each longitudinal edge, an improved estimate of the *effective width* is obtained by calculating ρ as follows:

$\rho = 1$ when $\lambda \le 0.673$

$$\rho = (1.358 - 0.461/\lambda)/\lambda \text{ when } 0.673 < \lambda < \lambda_c \qquad (Eq.\ 1.1\text{-}6)$$

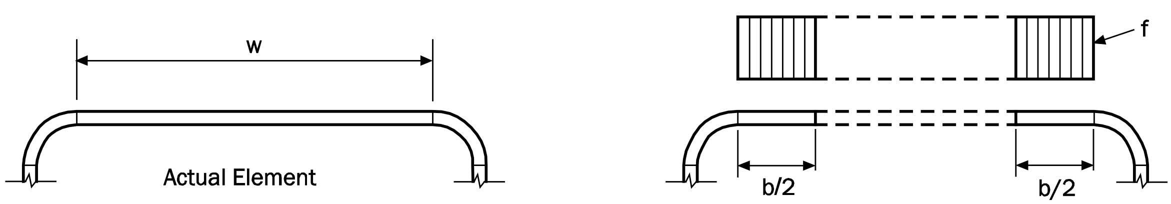
$$\rho = \left(0.41 + 0.59\sqrt{F_y/f_d} - 0.22/\lambda\right)/\lambda \text{ when } \lambda \ge \lambda_c \qquad (Eq.\ 1.1\text{-}7)$$

$\rho \le 1$ for all cases.

where
λ = Slenderness factor as defined by Eq. 1.1-3, except that f_d is substituted for f

$$\lambda_c = 0.256 + 0.328\,(w/t)\sqrt{F_y/E} \qquad (Eq.\ 1.1\text{-}8)$$

F_y = *Yield stress*



Effective Element, b, and Stress, f, on Effective Elements

Figure 1.1-1 Stiffened Elements

1.1.1 Uniformly Compressed Stiffened Elements With Circular or Noncircular Holes

(a) Strength Determination

For circular holes:

The *effective width*, b, shall be calculated by either Eq. 1.1.1-1 or Eq. 1.1.1-2 as follows:

For $0.50 \geq \dfrac{d_h}{w} \geq 0$, and $\dfrac{w}{t} \leq 70$, and

the distance between centers of holes $\geq 0.50w$ and $\geq 3d_h$

$$b = w - d_h \quad \text{when } \lambda \leq 0.673 \qquad (Eq.\ 1.1.1\text{-}1)$$

$$b = \frac{w\left[1 - \dfrac{(0.22)}{\lambda} - \dfrac{(0.8d_h)}{w} + \dfrac{(0.085d_h)}{w\lambda}\right]}{\lambda} \quad \text{when } \lambda > 0.673 \qquad (Eq.\ 1.1.1\text{-}2)$$

In all cases, $b \leq w - d_h$

where

w = *Flat width*

t = *Thickness* of element

d_h = Diameter of holes

λ = Slenderness factor as defined in Section 1.1 with $k = 4.0$

For noncircular holes:

A uniformly compressed stiffened element with noncircular holes shall be assumed to consist of two unstiffened strips of *flat width*, c, adjacent to the holes (see Figure 1.1.1-1). The *effective width*, b, of each unstiffened strip adjacent to the hole shall be determined in accordance with Section 1.1(a), except that the plate *buckling* coefficient, k, shall be taken as 0.43 and w as c. These provisions shall be applicable within the following limits:

(1) Center-to-center hole spacing, $s \geq 24$ in. (610 mm),

(2) Clear distance from the hole at ends, $s_{end} \geq 10$ in. (254 mm),

(3) Depth of hole, $d_h \leq 2.5$ in. (63.5 mm),

(4) Length of hole, $L_h \leq 4.5$ in. (114 mm), and

(5) Ratio of the depth of hole, d_h, to the out-to-out width, w_o, $d_h/w_o \leq 0.5$.

Alternatively, the *effective width*, b, is permitted to be determined by stub-column tests in accordance with the test procedure, ANSI/SDI AISI S902.

(b) Serviceability Determination

The *effective width*, b_d, used in determining serviceability shall be equal to b calculated in accordance with Procedure I of Section 1.1(b), except that f_d is substituted for f, where f_d is the computed compressive stress in the element being considered.

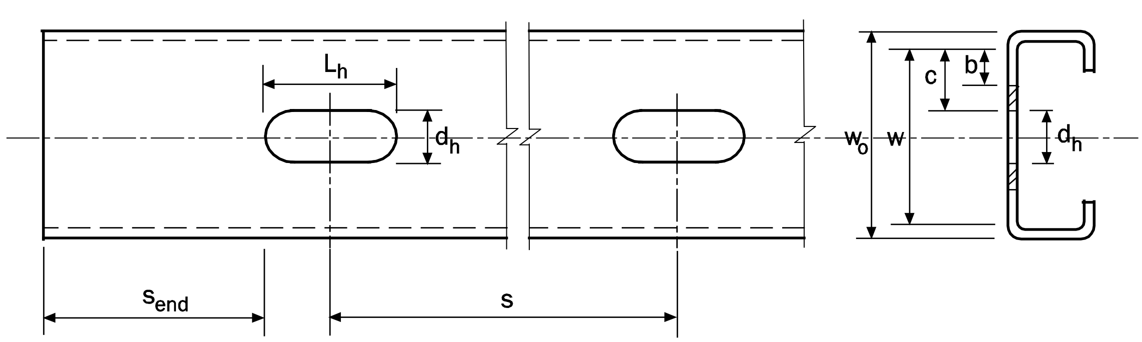


Figure 1.1.1-1 Uniformly Compressed Stiffened Elements With Noncircular Holes

1.1.2 Webs and Other Stiffened Elements Under Stress Gradient

The following notation shall apply in this section:

b_1 = *Effective width*, dimension defined in Figure 1.1.2-1

b_2 = *Effective width*, dimension defined in Figure 1.1.2-1

b_e = *Effective width*, b, determined in accordance with Section 1.1, with f_1 substituted for f and with k determined as given in this section

b_o = Out-to-out width of the compression *flange* as defined in Figure 1.1.2-2

f_1, f_2 = *Stresses* shown in Figure 1.1.2-1 calculated on the basis of effective section. Where f_1 and f_2 are both compression, $f_1 \geq f_2$

h_o = Out-to-out depth of *web* as defined in Figure 1.1.2-2

k = Plate *buckling* coefficient

ψ = $|f_2/f_1|$ (absolute value) (*Eq.* 1.1.2-1)

(a) Strength Determination

(1) For *webs* under *stress* gradient (f_1 in compression and f_2 in tension as shown in Figure 1.1.2-1(a)), the *effective widths* and plate *buckling* coefficient shall be calculated as follows:

$k = 4 + 2(1 + \psi)^3 + 2(1 + \psi)$ (*Eq.* 1.1.2-2)

For $h_o/b_o \leq 4$

$b_1 = b_e/(3 + \psi)$ (*Eq.* 1.1.2-3)

$b_2 = b_e/2$ when $\psi > 0.236$ (*Eq.* 1.1.2-4)

$b_2 = b_e - b_1$ when $\psi \leq 0.236$ (*Eq.* 1.1.2-5)

In addition, $b_1 + b_2$ shall not exceed the compression portion of the *web* calculated on the basis of effective section.

For $h_o/b_o > 4$

$b_1 = b_e/(3 + \psi)$ (*Eq.* 1.1.2-6)

$b_2 = b_e/(1 + \psi) - b_1$ (*Eq.* 1.1.2-7)

(2) For other stiffened elements under *stress* gradient (f_1 and f_2 in compression as shown in Figure 1.1.2-1(b)):

$k = 4 + 2(1 - \psi)^3 + 2(1 - \psi)$ (*Eq.* 1.1.2-8)

$b_1 = b_e/(3 - \psi)$ (*Eq.* 1.1.2-9)

$b_2 = b_e - b_1$ (*Eq.* 1.1.2-10)

(b) Serviceability Determination

The *effective widths* used in determining serviceability shall be calculated in accordance with Section 1.1.2(a) except that f_{d1} and f_{d2} are substituted for f_1 and f_2, where f_{d1} and f_{d2} are the computed *stresses* f_1 and f_2 based on the effective section at the *load* for which serviceability is determined.

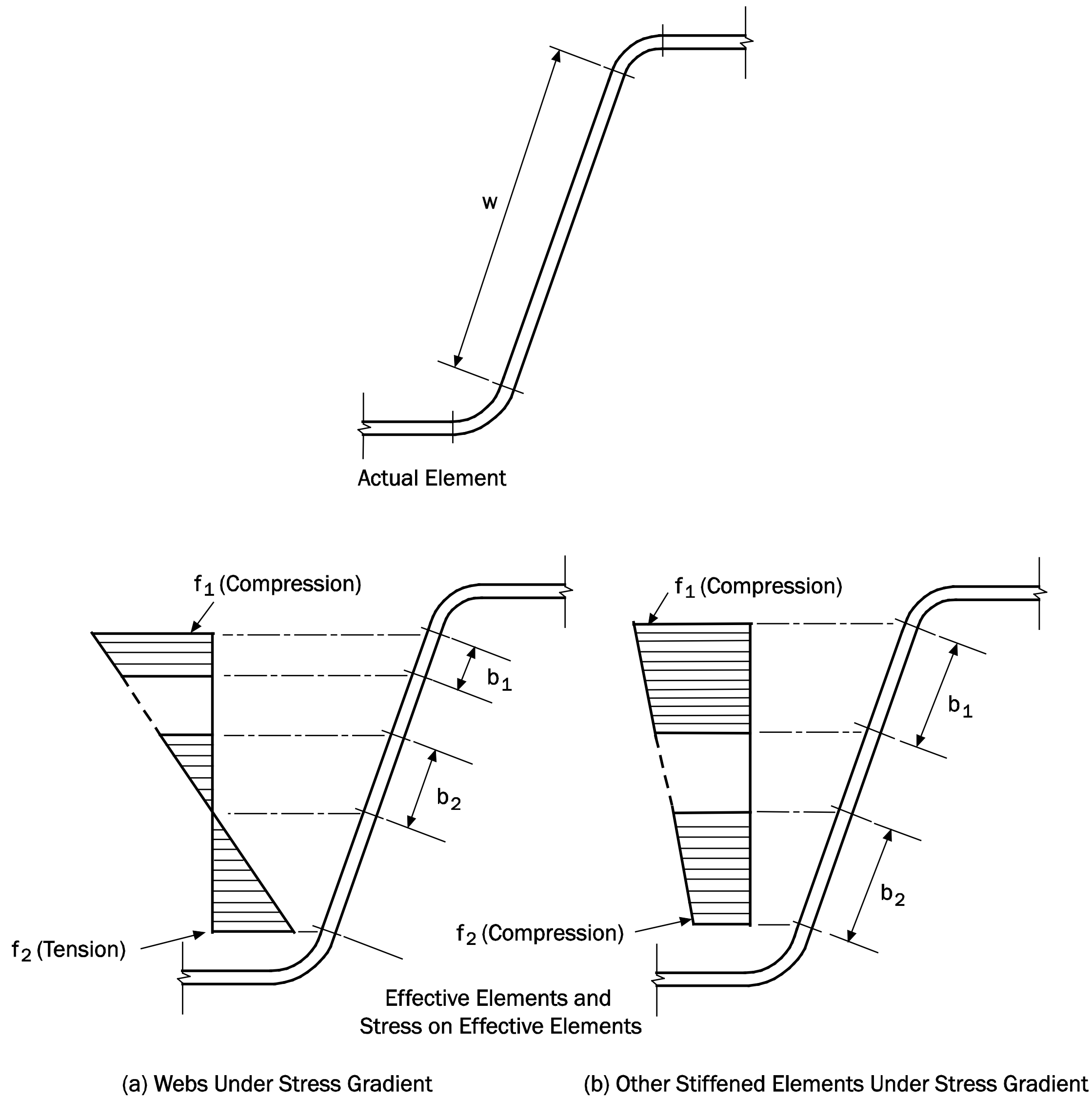


Figure 1.1.2-1 Webs and Other Stiffened Elements Under Stress Gradient

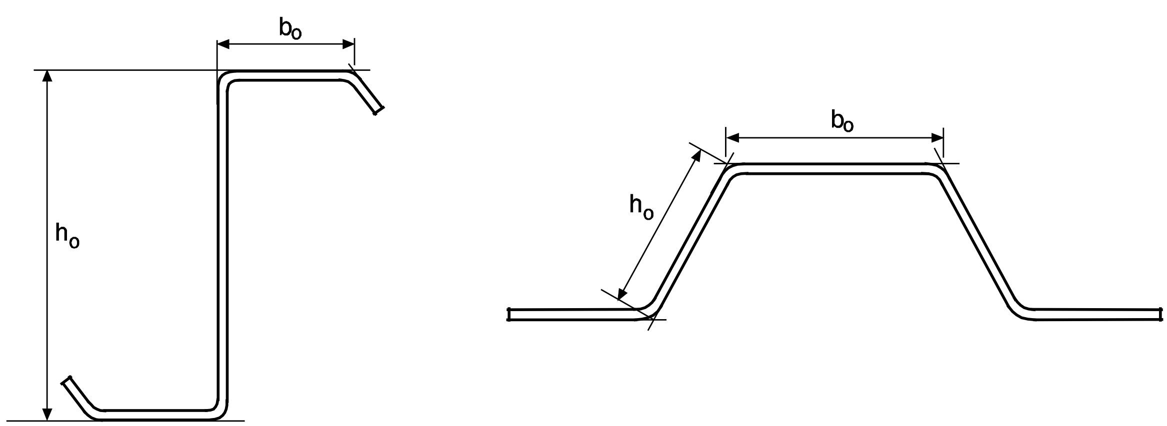


Figure 1.1.2-2 Out-to-Out Dimensions of Webs and Stiffened Elements Under Stress Gradient

1.1.3 C-Section Webs With Holes Under Stress Gradient

The provisions of Section 1.1.3 shall apply within the following limits:

(1) $d_h/h \leq 0.7$,

(2) $h/t \leq 200$,

(3) Holes centered at mid-depth of *web*,

(4) Clear distance between holes ≥ 18 in. (457 mm),

(5) Noncircular holes, corner radii ≥ 2t,

(6) Noncircular holes, $d_h \leq$ 2.5 in. (63.5 mm) and $L_h \leq$ 4.5 in. (114 mm),

(7) Circular holes, diameter ≤ 6 in. (152 mm), and

(8) $d_h >$ 9/16 in. (14.3 mm).

where

d_h = Depth of *web* hole

h = Depth of flat portion of *web* measured along plane of *web*

t = *Thickness* of *web*

L_h = Length of *web* hole

(a) *Strength Determination*

When $d_h/h < 0.38$, the *effective widths*, b_1 and b_2, as illustrated in Figure 1.1.2-1, shall be determined in accordance with Section 1.1.2(a) by assuming no hole exists in the *web*.

When $d_h/h \geq 0.38$, the *effective width* shall be determined in accordance with Section 1.2.2(a), assuming the compression portion of the *web* consists of an unstiffened element adjacent to the hole, using f_1 and f_2 as shown in Figure 1.2.2-1(a). Alternatively, the *effective width* of the unstiffened element is permitted to be calculated in accordance with Section 1.2.1(a) with $f = f_1$ as shown in Figure 1.1.2-1.

(b) *Serviceability Determination*

The *effective widths* shall be determined in accordance with Section 1.1.2(b) by assuming no hole exists in the *web*.

1.1.4 Uniformly Compressed Elements Restrained by Intermittent Connections

The provisions of this section shall apply to compressed elements of flexural members only. The provisions shall be limited to multiple flute built-up members having edge-stiffened cover plates. When the spacing of fasteners, s, of a uniformly compressed element restrained by intermittent *connections* is not greater than the limits specified in Section I1.3, the *effective width* shall be calculated in accordance with Section 1.1. When the spacing of fasteners is greater than the limits specified in Section I1.3, the *effective width* shall be determined in accordance with (a) and (b) below.

(a) *Strength Determination*

The *effective width* of the uniformly compressed element restrained by intermittent *connections* shall be determined as follows:

(1) When $f < F_c$, the *effective width* of the compression element between *connection* lines shall be calculated in accordance with Section 1.1(a).

(2) When $f \geq F_c$, the *effective width* of the compression element between *connection* lines shall be calculated in accordance with Section 1.1(a), except that the reduction factor, ρ, shall be the

lesser of the value determined in accordance with Section 1.1 and the value determined by Eq. 1.1.4-1:

$$\rho = \rho_t \rho_m \qquad (Eq.\ 1.1.4\text{-}1)$$

where

$\rho_t = 1.0$ for $\lambda_t \le 0.673$

$$\rho_t = (1.0 - 0.22/\lambda_t)/\lambda_t \quad \text{for } \lambda_t > 0.673 \qquad (Eq.\ 1.1.4\text{-}2)$$

where

$$\lambda_t = \sqrt{\frac{F_c}{F_{cr\ell}}} \qquad (Eq.\ 1.1.4\text{-}3)$$

where

F_c = Critical column *buckling stress* of compression element

$$= 3.29\,E/(s/t)^2 \qquad (Eq.\ 1.1.4\text{-}4)$$

where

s = Center-to-center spacing of connectors in line of compression *stress*

E = Modulus of elasticity of steel

t = *Thickness* of cover plate in compression

$F_{cr\ell}$ = Critical *buckling stress* defined in Eq. 1.1-4 where w is the transverse spacing of connectors

$$\rho_m = 8\left(\frac{F_y}{f}\right)\sqrt{\frac{tF_c}{df}} \le 1.0 \qquad (Eq.\ 1.1.4\text{-}5)$$

where

F_y = Design *yield stress* of the compression element restrained by intermittent *connections*

d = Overall depth of the built-up member

f = *Stress* in compression element restrained by intermittent *connections* when the controlling extreme fiber *stress* is F_y

The provisions of this section shall apply to shapes that meet the following limits:

(1) 1.5 in. (38.1 mm) ≤ d ≤ 7.5 in. (191 mm),

(2) 0.035 in. (0.889 mm) ≤ t ≤ 0.060 in. (1.52 mm),

(3) 2.0 in. (50.8 mm) ≤ s ≤ 8.0 in. (203 mm),

(4) 33 ksi (228 MPa or 2320 kg/cm^2) ≤ F_y ≤ 60 ksi (414 MPa or 4220 kg/cm^2), and

(5) 100 ≤ w/t ≤ 350.

The *effective width* of the edge stiffener and the flat portion, e, shall be determined in accordance with Section 1.3(a) with modifications as follows:

For $f < F_c$

$$w = e \qquad (Eq.\ 1.1.4\text{-}6)$$

For $f \ge F_c$

For the flat portion, e, the *effective width*, b, in Eqs. 1.3-4 and 1.3-5 shall be calculated in accordance with Section 1.1(a) with

(i) w taken as e,

(ii) if $D/e \le 0.8$

k is determined in accordance with Table 1.3-1

if $D/e > 0.8$

k=1.25, and

(iii) ρ calculated using Eq. 1.1.4-1 in lieu of Eq. 1.1-2.

where

w = *Flat width* of element measured between longitudinal *connection* lines and exclusive of radii at stiffeners

e = *Flat width* between the first line of connector and the edge stiffener. See Figure 1.1.4-1

D = Overall length of stiffener as defined in Section 1.3

For the edge stiffener, d_s and I_a shall be determined using w′ and f′ in lieu of w and f, respectively.

$w' = 2e + \text{minimum of } (0.75s \text{ and } w_1)$ (*Eq*. 1.1.4-7)

$f' = \text{Maximum of } (\rho_m f \text{ and } F_c)$ (*Eq*. 1.1.4-8)

where

f′ = *Stress* used in Section 1.3(a) for determining *effective width* of edge stiffener

F_c = *Buckling stress* of cover plate determined in accordance with Eq. 1.1.4-4

w′= Equivalent *flat width* for determining the *effective width* of edge stiffener

w_1= Transverse spacing between the first and the second line of connectors in the compression element. See Figure 1.1.4-1.

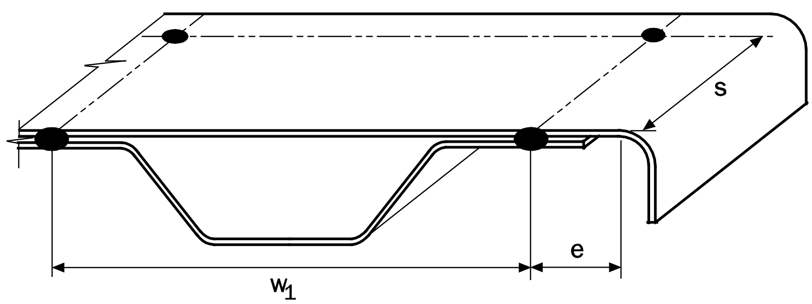


Figure 1.1.4-1 Dimension Illustration of Cellular Deck

The provisions of this section shall not apply to single flute members having compression plates with edge stiffeners.

(b) Serviceability Determination

The *effective width* of the uniformly compressed element restrained by intermittent *connections* used for computing deflection shall be determined in accordance with Section 1.1.4(a) except that:

(1) f_d shall be substituted for f, where f_d is the computed compression *stress* in the element being considered at *service load*, and

(2) The maximum extreme fiber *stress* in the built-up member shall be substituted for F_y.

1.2 Effective Width of Unstiffened Elements

1.2.1 Uniformly Compressed Unstiffened Elements

(a) Strength Determination

The *effective width*, b, shall be determined in accordance with Section 1.1(a), except that the plate *buckling* coefficient, k, shall be taken as 0.43 and w as defined in Figure 1.2.1-1.

(b) Serviceability Determination

The *effective width*, b_d, used in determining serviceability shall be calculated in accordance with Procedure I of Section 1.1(b), except that f_d is substituted for f and k = 0.43.

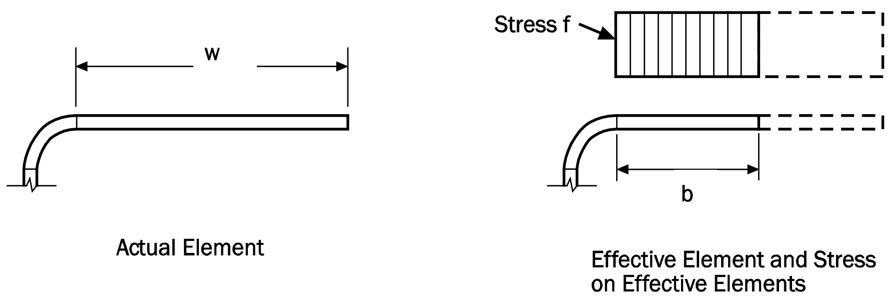


Figure 1.2.1-1 Unstiffened Element With Uniform Compression

1.2.2 Unstiffened Elements and Edge Stiffeners With Stress Gradient

The following notation shall apply in this section:

b = *Effective width* measured from the supported edge, determined in accordance with Section 1.1(a), with f equal to the maximum compressive *stress* on the effective element and with k and ρ being determined in accordance with this section

b_o = Overall width of unstiffened element of unstiffened C-section member as defined in Fig. 1.2.2-3

f_1, f_2 = *Stresses*, shown in Figures 1.2.2-1, 1.2.2-2, and 1.2.2-3. Where f_1 and f_2 are both compression, $f_1 \geq f_2$.

h_o = Overall depth of unstiffened C-section member. See Figure 1.2.2-3

k = Plate *buckling* coefficient defined in this section or, otherwise, as defined in Section 1.1(a)

t = *Thickness* of element

w = *Flat width* of unstiffened element, where $w/t \leq 60$

ψ = $|f_2/ f_1|$ (absolute value) (*Eq.* 1.2.2-1)

λ = Slenderness factor defined in Section 1.1(a) with f equal to the maximum compressive *stress* on the effective element

ρ = Reduction factor defined in this section or, otherwise, as defined in Section 1.1(a)

(a) Strength Determination

The *effective width*, b, of an unstiffened element under *stress* gradient shall be determined in accordance with Section 1.1(a) with *stress*, f, equal to the maximum compressive *stress* on the effective element and the plate *buckling* coefficient, k, determined in accordance with this section, unless otherwise noted. For the cases where f_1 is in compression and f_2 is in tension, ρ in Section 1.1(a) shall be determined in accordance with this section.

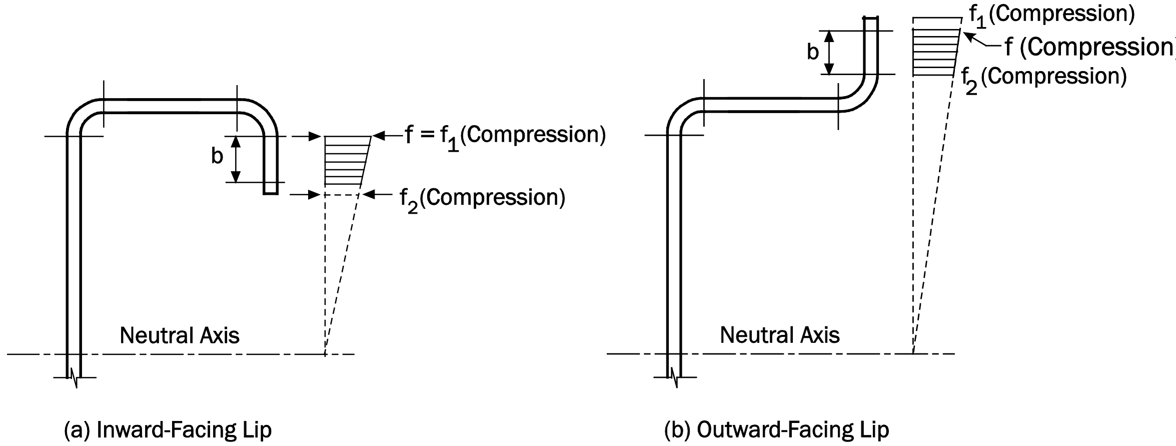


Figure 1.2.2-1 Unstiffened Elements Under Stress Gradient, Both Longitudinal Edges in Compression

(1) When both f_1 and f_2 are in compression (Figure 1.2.2-1), the plate *buckling* coefficient shall be calculated in accordance with either Eq. 1.2.2-2 or Eq. 1.2.2-3 as follows:

If the *stress* decreases toward the unsupported edge (Figure 1.2.2-1(a)):

$$k = \frac{0.578}{\psi + 0.34} \quad \text{(Eq. 1.2.2-2)}$$

If the *stress* increases toward the unsupported edge (Figure 1.2.2-1(b)):

$$k = 0.57 - 0.21\psi + 0.07\psi^2 \quad \text{(Eq. 1.2.2-3)}$$

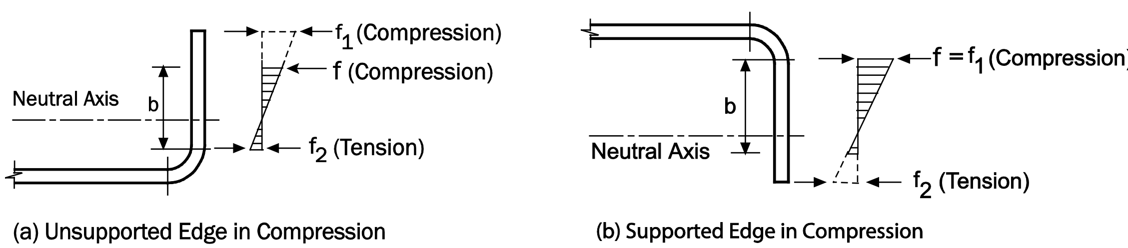


Figure 1.2.2-2 Unstiffened Elements Under Stress Gradient, One Longitudinal Edge in Compression and the Other Longitudinal Edge in Tension

(2) When f_1 is in compression and f_2 in tension (Fig. 1.2.2-2), the reduction factor and plate *buckling* coefficient shall be calculated as follows:

(i) If the unsupported edge is in compression (Figure 1.2.2-2(a)):

$$\rho = 1 \quad \text{when } \lambda \le 0.673(1+\psi)$$

$$\rho = (1+\psi)\frac{\left(1 - \frac{0.22(1+\psi)}{\lambda}\right)}{\lambda} \quad \text{when } \lambda > 0.673(1+\psi) \quad \text{(Eq. 1.2.2-4)}$$

$$k = 0.57 + 0.21\psi + 0.07\psi^2 \quad \text{(Eq. 1.2.2-5)}$$

(ii) If the supported edge is in compression (Fig. 1.2.2-2(b)):

For $\psi < 1$

$\rho = 1$ when $\lambda \le 0.673$

$$\rho = (1-\psi)\frac{\left(1-\frac{0.22}{\lambda}\right)}{\lambda}+\psi \quad \text{when } \lambda > 0.673 \qquad (Eq.\ 1.2.2\text{-}6)$$

$$k = 1.70+5\psi+17.1\psi^2 \qquad (Eq.\ 1.2.2\text{-}7)$$

For $\psi \ge 1$,

$\rho = 1$

The *effective width*, b, of the unstiffened elements of an unstiffened C-section member is permitted to be determined using the following alternative methods, as applicable:

Alternative 1 for unstiffened C-sections: When the unsupported edge is in compression and the supported edge is in tension (Figure 1.2.2-3 (a)):

$$b = w \quad \text{when } \lambda \le 0.856 \qquad (Eq.\ 1.2.2\text{-}8)$$

$$b = \rho w \quad \text{when } \lambda > 0.856 \qquad (Eq.\ 1.2.2\text{-}9)$$

where

$$\rho = 0.925/\sqrt{\lambda} \qquad (Eq.\ 1.2.2\text{-}10)$$

$$k = 0.145(b_o/h_o) + 1.256 \qquad (Eq.\ 1.2.2\text{-}11)$$

$0.1 \le b_o/h_o \le 1.0$

Alternative 2 for unstiffened C-sections: When the supported edge is in compression and the unsupported edge is in tension (Figure 1.2.2-3(b)), the *effective width* is determined in accordance with Section 1.1.2.

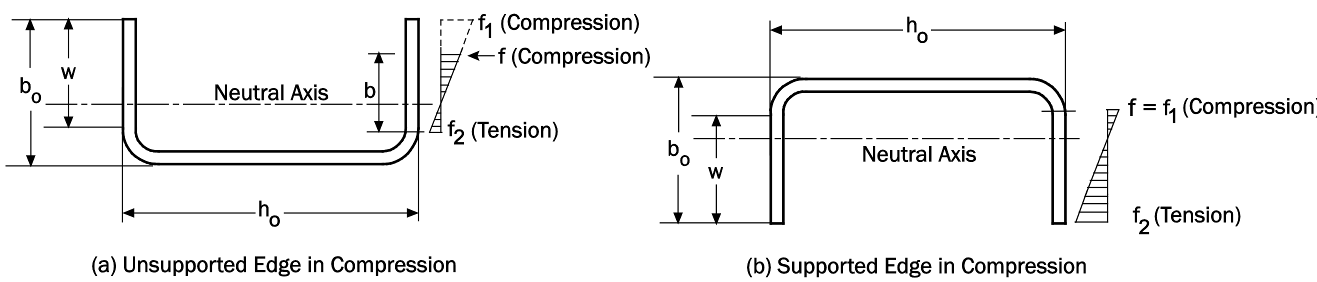


Figure 1.2.2-3 Unstiffened Elements of C-Section Under Stress Gradient for Alternative Methods

Where *stress*, f_1, occurs at the unsupported edge as in Figures 1.2.2-1(b), 1.2.2-2(a), and 1.2.2-3(a), the design *stress*, f, shall be taken at the extreme fiber of the effective section, and f_1 is the calculated *stress*, based on the effective section, at the edge of the gross section. If the only elements not fully effective are unstiffened elements with *stress* gradient, as in Figure 1.2.2-3(a), the *stresses* f_1 and f_2 are permitted to be based on the gross section, f taken equal to f_1, and iteration is not required.

In calculating the effective section modulus, S_{ec} or S_{et}, in Section F3.1, the extreme compression fiber in Figures 1.2.2-1(b), 1.2.2-2(a), and 1.2.2-3(a) shall be taken as the edge of the effective section closer to the unsupported edge, and the extreme tension fiber in Figures 1.2.2-2(b) and 1.2.2-3(b) shall be taken as the edge of the effective section closer to the unsupported edge.

(b) Serviceability Determination

The *effective width,* b_d, used in determining serviceability shall be calculated in accordance with Section 1.2.2(a), except that f_{d1} and f_{d2} are substituted for f_1 and f_2, respectively, where f_{d1} and f_{d2} are the computed *stresses* f_1 and f_2 as shown in Figures 1.2.2-1, 1.2.2-2, and 1.2.2-3, respectively, at the *load* for which serviceability is determined.

1.3 Effective Width of Uniformly Compressed Elements With a Simple Lip Edge Stiffener

The *effective widths* of uniformly compressed elements with a simple lip edge stiffener shall be calculated in accordance with (a) for strength determination and (b) for serviceability determination.

(a) Strength Determination

For $w/t \le 0.328S$:

$I_a = 0$ (no edge stiffener needed)

$b = w$ (*Eq.* 1.3-1)

$b_1 = b_2 = w/2$ (see Figure 1.3-1) (*Eq.* 1.3-2)

$d_s = d'_s$ (*Eq.* 1.3-3)

For $w/t > 0.328S$

$b_1 = (b/2)\,(R_I)$ (see Figure 1.3-1) (*Eq.* 1.3-4)

$b_2 = b - b_1$ (see Figure 1.3-1) (*Eq.* 1.3-5)

$d_s = d'_s\,(R_I)$ (*Eq.* 1.3-6)

where

$S = 1.28\sqrt{E/f}$ (*Eq.* 1.3-7)

where

E = Modulus of elasticity of steel

f = *Stress* in compression flange

w = Flat dimension of *flange* (see Figure 1.3-1)

t = *Thickness* of section

I_a = Adequate moment of inertia of stiffener, so that each component element will behave as a stiffened element

$= 399t^4\left(\frac{w/t}{S} - 0.328\right)^3 \le t^4\left(115\frac{w/t}{S} + 5\right)$ (*Eq.* 1.3-8)

b = *Effective design width*

b_1, b_2 = Portions of *effective design width* (see Figure 1.3-1)

d_s = Reduced *effective width* of stiffener (see Figure 1.3-1), which is used in computing overall effective section properties

d'_s = *Effective width* of stiffener calculated in accordance with Section 1.2.1 or 1.2.2 (see Figure 1.3-1)

$(R_I) = I_s/I_a \le 1$ (*Eq.* 1.3-9)

where

I_s = Unreduced moment of inertia of stiffener about its own centroidal axis parallel to element to be stiffened. For edge stiffeners, the round corner between stiffener and element to be stiffened is not considered a part of the stiffener.

$= (d^3 t \sin^2\theta)/12$ (*Eq.* 1.3-10)

See Figure 1.3-1 for definitions of other dimensional variables.

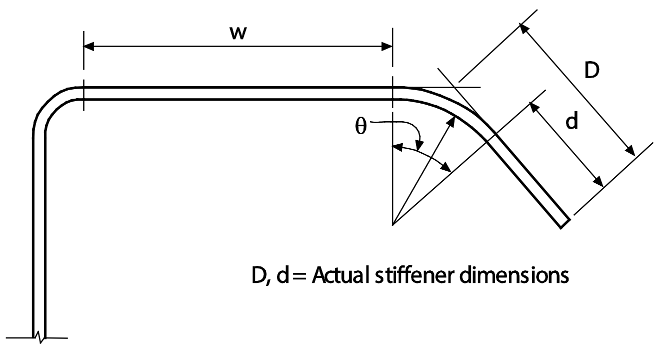
The *effective width*, b, in Eqs. 1.3-4 and 1.3-5 shall be calculated in accordance with Section 1.1.1 with the plate *buckling* coefficient, k, as given in Table 1.3-1 below:

Table 1.3-1
Determination of Plate Buckling Coefficient, k

Simple Lip Edge Stiffener (140° ≥ θ ≥ 40°)	
$D/w \le 0.25$	$0.25 < D/w \le 0.8$
$3.57(R_I)^n + 0.43 \le 4$	$(4.82 - \frac{5D}{w})(R_I)^n + 0.43 \le 4$

where

$$n = \left(0.582 - \frac{w/t}{4S}\right) \ge \frac{1}{3} \qquad (Eq.\ 1.3\text{-}11)$$



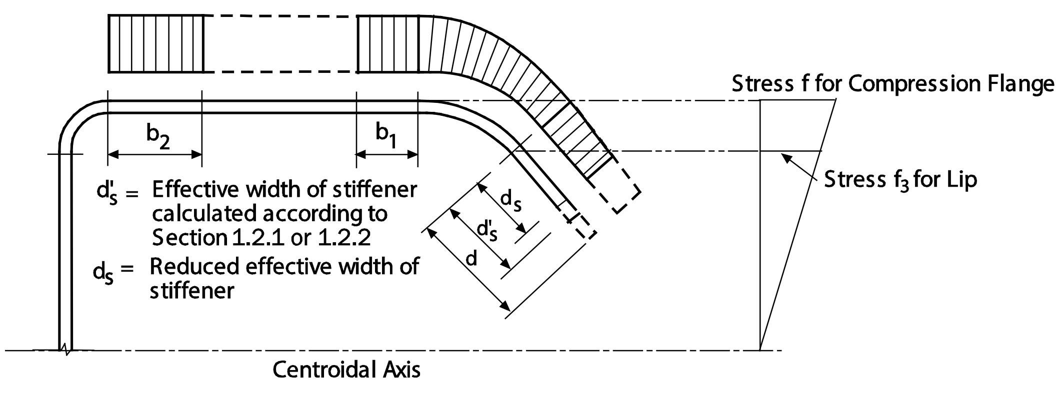


Figure 1.3-1 Element With Simple Lip Edge Stiffener

(b) Serviceability Determination

The *effective width*, b_d, used in determining serviceability shall be calculated as in Section 1.3(a), except that f_d is substituted for f, where f_d is computed compressive *stress* in the effective section at the *load* for which serviceability is determined.

1.4 Effective Width of Stiffened Elements With Single or Multiple Intermediate Stiffeners or Edge-Stiffened Elements With Intermediate Stiffener(s)

1.4.1 Effective Width of Uniformly Compressed Stiffened Elements With Single or Multiple Intermediate Stiffeners

The following notations shall apply in this section:

A_g = *Gross area* of element including stiffeners
A_s = *Gross area* of stiffener
b_e = *Effective width* of element, located at centroid of element including stiffeners; see Figure 1.4.1-2
b_o = Total *flat width* of stiffened element; see Figure 1.4.1-1
b_p = Largest sub-element *flat width*; see Figure 1.4.1-1
c_i = Horizontal distance from edge of element to centerline(s) of stiffener(s); see Figure 1.4.1-1
E = Modulus of elasticity of steel
$F_{cr\ell}$ = Plate elastic *buckling stress*
f = Uniform compressive *stress* acting on flat element
h = Width of elements adjoining stiffened element (e.g., depth of *web* in hat section with multiple intermediate stiffeners in compression *flange* is equal to h; if adjoining elements have different widths, use smallest one)
I_{sp} = Moment of inertia of stiffener about centerline of flat portion of element. The radii that connect the stiffener to the flat can be included.
k = Plate *buckling* coefficient of element
k_d = Plate *buckling* coefficient for *distortional buckling*
k_{loc} = Plate *buckling* coefficient for local sub-element *buckling*
L_{br} = Unsupported length between brace points or other restraints that restrict *distortional buckling* of element
R = Modification factor for *distortional* plate *buckling* coefficient
n = Number of stiffeners in element
t = Element *thickness*
i = Index for stiffener "i"
λ = Slenderness factor
μ = Poisson's ratio of steel
ρ = Reduction factor

The *effective width* shall be calculated in accordance with Eq. 1.4.1-1 as follows:

$$b_e = \rho\left(\frac{A_g}{t}\right) \quad \text{(Eq. 1.4.1-1)}$$

where

ρ = 1 when $\lambda \le 0.673$

$$\rho = (1 - 0.22/\lambda)/\lambda \quad \text{when } \lambda > 0.673 \quad \text{(Eq. 1.4.1-2)}$$

where

$$\lambda = \sqrt{\frac{f}{F_{cr\ell}}} \qquad (Eq.\ 1.4.1\text{-}3)$$

where

$$F_{cr\ell} = k\frac{\pi^2 E}{12(1-\mu^2)}\left(\frac{t}{b_o}\right)^2 \qquad (Eq.\ 1.4.1\text{-}4)$$

The plate *buckling* coefficient, k, shall be determined from the minimum of Rk_d and k_{loc}, as determined in accordance with Section 1.4.1.1 or 1.4.1.2, as applicable.

k = the minimum of Rk_d and k_{loc} (*Eq.* 1.4.1-5)

R = 2 when $b_o/h < 1$

$$R = \frac{11 - b_o/h}{5} \geq \frac{1}{2} \quad \text{when } b_o/h \geq 1 \qquad (Eq.\ 1.4.1\text{-}6)$$

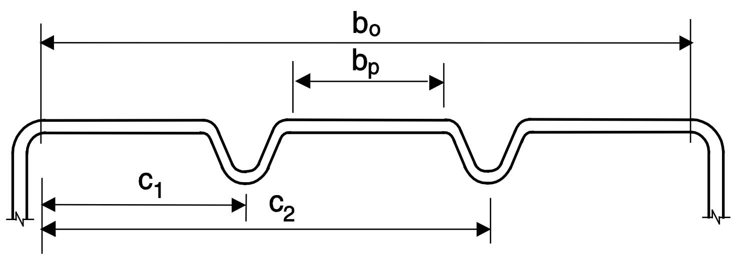


Figure 1.4.1-1 Plate Widths and Stiffener Locations

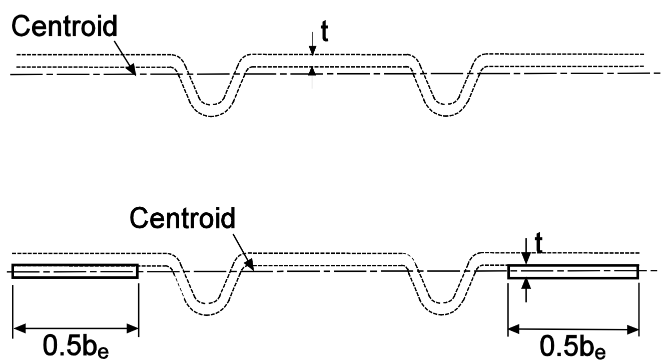


Figure 1.4.1-2 Effective Width Locations

1.4.1.1 Specific Case: Single or n Identical Stiffeners, Equally Spaced

For uniformly compressed elements with single or multiple identical and equally spaced stiffeners, the plate *buckling* coefficients and *effective widths* shall be calculated as follows:

(a) Strength Determination

$$k_{loc} = 4(b_o/b_p)^2 \qquad (Eq.\ 1.4.1.1\text{-}1)$$

$$k_d = \frac{(1+\beta^2)^2 + \gamma(1+n)}{\beta^2(1+\delta(n+1))} \quad (Eq.\ 1.4.1.1\text{-}2)$$

where

$$\beta = (1+\gamma(n+1))^{1/4} \quad (Eq.\ 1.4.1.1\text{-}3)$$

where

$$\gamma = \frac{10.92 I_{sp}}{b_o t^3} \quad (Eq.\ 1.4.1.1\text{-}4)$$

$$\delta = \frac{A_s}{b_o t} \quad (Eq.\ 1.4.1.1\text{-}5)$$

If $L_{br} < \beta b_o$, L_{br}/b_o is permitted to be substituted for β to account for increased capacity due to bracing.

(b) Serviceability Determination

The *effective width*, b_d, used in determining serviceability shall be calculated as in Section 1.4.1.1(a), except that f_d is substituted for f, where f_d is the computed compressive *stress* in the element being considered based on the effective section at the *load* for which serviceability is determined.

1.4.1.2 General Case: Arbitrary Stiffener Size, Location, and Number

For uniformly compressed stiffened elements with stiffeners of arbitrary size, location, and number, the plate *buckling* coefficients and *effective widths* shall be calculated as follows:

(a) Strength Determination

$$k_{loc} = 4(b_o/b_p)^2 \quad (Eq.\ 1.4.1.2\text{-}1)$$

$$k_d = \frac{(1+\beta^2)^2 + 2\sum_{i=1}^{n}\gamma_i\omega_i}{\beta^2\left(1+2\sum_{i=1}^{n}\delta_i\omega_i\right)} \quad (Eq.\ 1.4.1.2\text{-}2)$$

where

$$\beta = \left(2\sum_{i=1}^{n}\gamma_i\omega_i + 1\right)^{1/4} \quad (Eq.\ 1.4.1.2\text{-}3)$$

where

$$\gamma_i = \frac{10.92 (I_{sp})_i}{b_o t^3} \quad (Eq.\ 1.4.1.2\text{-}4)$$

$$\omega_i = \sin^2\left(\pi\frac{c_i}{b_o}\right) \quad (Eq.\ 1.4.1.2\text{-}5)$$

$$\delta_i = \frac{(A_s)_i}{b_o t} \quad (Eq.\ 1.4.1.2\text{-}6)$$

If $L_{br} < \beta b_o$, L_{br}/b_o is permitted to be substituted for β to account for increased capacity due to bracing.

(b) *Serviceability Determination*

The *effective width*, b_d, used in determining serviceability shall be calculated as in Section 1.4.1.2(a), except that f_d is substituted for f, where f_d is the computed compressive *stress* in the element being considered based on the effective section at the *load* for which serviceability is determined.

1.4.2 Edge-Stiffened Elements With Intermediate Stiffener(s)

(a) *Strength Determination*

For edge-stiffened elements with intermediate stiffener(s), the *effective width*, b_e, shall be determined as follows:

If $b_o/t \leq 0.328S$, the element is fully effective and no *local buckling* reduction is required.

If $b_o/t > 0.328S$, the plate *buckling* coefficient, k, is determined in accordance with Section 1.3, but with b_o replacing w in all expressions:

If k calculated from Section 1.3 is less than 4.0 (k < 4), the intermediate stiffener(s) is ignored and the provisions of Section 1.3 are followed for calculation of the *effective width*.

If k calculated from Section 1.3 is equal to 4.0 (k = 4), the *effective width* of the edge-stiffened element is calculated from the provisions of Section 1.4.1, with the following exception:

R calculated in accordance with Section 1.4.1 is less than or equal to 1.

where

b_o = Total *flat width* of edge-stiffened element

See Sections 1.3 and 1.4.1 for definitions of other variables.

(b) *Serviceability Determination*

The *effective width*, b_d, used in determining serviceability shall be calculated as in Section 1.4.2(a), except that f_d is substituted for f, where f_d is the computed compressive *stress* in the element being considered based on the effective section at the *load* for which serviceability is determined.

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APPENDIX 2, ELASTIC BUCKLING ANALYSIS OF MEMBERS

This appendix addresses the elastic *buckling stress* and *stress* resultant (force or moment) that are used for the determination of member strength in the *Specification*.

Elastic *buckling* occurs at a load in which the equilibrium of the member (approximated with linear elastic material) is neutral between two alternative states: buckled and straight. Thin-walled cold-formed steel members may have at least three relevant elastic *buckling* modes: *local*, *distortional*, and global. The global *buckling* mode includes *flexural*, *torsional*, or *flexural-torsional buckling* for columns, and *lateral-torsional buckling* for beams. This appendix provides a means to determine all three relevant *buckling* modes for use in the design process.

This appendix is organized as follows:

2.1 General Provisions

2.2 Numerical Solutions

2.3 Analytical Solutions

2.1 General Provisions

The elastic *buckling stresses* or elastic *buckling stress* resultants (forces or moments) that are used in the *Specification* Chapters D through H are permitted to be calculated numerically in accordance with Section 2.2, analytically in accordance with Section 2.3, or in any combination.

In compression, global, *local*, and *distortional buckling* conversion between force and *stress* shall use the *gross area*, except where a reduced (e.g., net or effective) area is explicitly required by the *Specification*. Therefore:

$$P_{cr} = A_g F_{cr} \qquad (Eq.\ 2.1\text{-}1)$$

where

P_{cr} = P_{cre} — global (flexural, torsional, or flexural-torsional), $P_{cr\ell}$ — local, or P_{crd} — distortional elastic *buckling* force in compression

F_{cr} = F_{cre} — global (flexural, torsional, or flexural-torsional), $F_{cr\ell}$ — local, or F_{crd} — distortional elastic *buckling stress* in compression

A_g = *Gross* cross-sectional *area*

In flexure, global, *local*, and *distortional buckling* conversion between moment and *stress* at the extreme compression fiber shall use the gross section modulus, except where a reduced (e.g., net or effective) section modulus is explicitly required by the *Specification*. Therefore:

$$M_{cr} = S_{fc} F_{cr} \qquad (Eq.\ 2.1\text{-}2)$$

where

M_{cr} = M_{cre} — global (lateral-torsional), $M_{cr\ell}$ — local, or M_{crd} — distortional elastic *buckling* moment about the axis of bending

F_{cr} = F_{cre} — global (lateral-torsional), $F_{cr\ell}$ — local, or F_{crd} — distortional elastic *buckling stress* referenced to the extreme compression fiber

S_{fc} = Gross elastic section modulus referenced to the extreme compression fiber

In shear, shear *buckling* conversion between force and *stress* shall use the *web gross area*, except where a reduced area is explicitly required by the *Specification*. Therefore:

$$V_{cr} = F_{cr} A_w \qquad (Eq.\ 2.1\text{-}3)$$

where

V_{cr} = Shear elastic *buckling* force

F_{cr} = Shear elastic *buckling stress*

A_w = *Web gross area*

User Note:
The *Specification* uses both *stress* and *stress* resultants (force, moment, etc.) in elastic *buckling* analysis. In particular, *Effective Width Method* calculations (e.g., Section E3.1) and traditional column and beam *buckling* formulas use *stress* (F_{cr}), while the *Direct Strength Method* (e.g., Section E3.2) uses *stress* resultants (P_{cr}). Numerical solutions are also performed as *stress* or *stress* resultants; either is adequate, but conversion of results between *stress* and *stress* resultant may be needed in order to use *Specification* equations.

2.2 Numerical Solutions

Any numerical elastic *buckling* solution that includes the relevant mechanics for the *buckling* mode under consideration is permitted to be utilized.

User Note:
A number of numerical methods, and related software programs, are known to be accurate for *local*, *distortional*, and global *buckling*, including the finite strip method utilizing plate bending strips for discretizing the cross-section, the finite element method utilizing plate or shell finite elements for discretizing the cross-section, and generalized beam theory with appropriate cross-section modes added for *local* and *distortional buckling*. See the *Commentary* for greater elaboration on the application of these numerical methods, including methods for members with holes, members with bracing, etc.

For *local buckling*, the impact of plate bending and cross-sectional distortion on the elastic *buckling* mode shall be considered.

For *distortional buckling*, the impact of plate bending and cross-sectional distortion, including distortion resulting from longitudinal strains, shall be considered.

For *shear buckling* (a specialized case of *local* or *distortional buckling* or both), the interaction of shear and longitudinal *stresses* on plate bending and cross-sectional distortion shall be considered.

For global *buckling*, the interaction of bending and torsion (i.e., *flexural-torsional buckling* or *lateral-torsional buckling*), particularly for cross-sections that are not doubly symmetric, shall be considered.

User Note:
Most conventional beam finite elements used in structural analysis software do not include the interaction of bending and torsion and should be used with care for global elastic *buckling* determination.

2.3 Analytical Solutions

The analytical solutions provided in this section are permitted to be used for the given boundary conditions and cross-section geometry. For other boundary conditions or cross-section geometry, numerical analysis as detailed in Section 2.2 shall be used.

User Note:
The global *buckling* provisions are based on members with discrete supports without additional restraint from attached components such as deck or sheathing, and will generally give conservative

results where such restraint is provided. The column *buckling* solutions are based on axial *load* applied at the centroid of the cross-section. The *lateral-torsional buckling* solutions are based on transverse loads applied at the shear center of the cross-section, whereas other load locations could increase or decrease the *buckling* moment.

2.3.1 Global Buckling

The global *buckling* forces and moments for *cold-formed steel structural members* are permitted to be determined analytically in accordance with this section. The provisions of this section and its subsections make use of the following variables:

P_{ex} = Axial force for *flexural buckling* about x-axis

$$= \frac{\pi^2 EI_x}{(K_x L_x)^2} \qquad (Eq.\ 2.3.1\text{-}1)$$

P_{ey} = Axial force for *flexural buckling* about y-axis

$$= \frac{\pi^2 EI_y}{(K_y L_y)^2} \qquad (Eq.\ 2.3.1\text{-}2)$$

P_t = Axial force for *torsional buckling* about shear center

$$= \frac{1}{r_o^2}\left[GJ + \frac{\pi^2 EC_w}{(K_t L_t)^2}\right] \qquad (Eq.\ 2.3.1\text{-}3)$$

β = Coefficient for *flexural-torsional buckling* about x-axis

$$= 1 - \left(\frac{x_o}{r_o}\right)^2 \left(\frac{K_t L_t}{K_x L_x}\right)^2 \qquad (Eq.\ 2.3.1\text{-}4)$$

γ = Coefficient for *flexural-torsional buckling* about y-axis

$$= 1 - \left(\frac{y_o}{r_o}\right)^2 \left(\frac{K_t L_t}{K_y L_y}\right)^2 \qquad (Eq.\ 2.3.1\text{-}5)$$

j = Asymmetry property

$$= \frac{1}{2I_y}\left(\int_A x^3 dA + \int_A xy^2 dA\right) - x_o \qquad (Eq.\ 2.3.1\text{-}6)$$

where

E = Modulus of elasticity of steel

= 29,500 ksi (203,000 MPa, or 2,070,000 kg/cm^2)

G = Shear modulus of steel

= 11,300 ksi (78,000 MPa or 795,000 kg/cm^2)

A = Full unreduced *cross-sectional area*

C_w = Torsion warping constant of cross-section

I_x = Moment of inertia of full unreduced cross-section about x-axis

I_y = Moment of inertia of full unreduced cross-section about y-axis

I_{xy} = Product of inertia of full unreduced cross-section about x- and y- axes

J = Saint-Venant torsion constant

K_x = *Effective length factor* for *buckling* about x-axis in accordance with Chapter C
K_y = *Effective length factor* for *buckling* about y-axis in accordance with Chapter C
K_t = *Effective length factor* for twisting determined in accordance with Chapter C
L_x = *Unbraced length* of member for *buckling* about x-axis
L_y = *Unbraced length* of member for *buckling* about y-axis
L_t = *Unbraced length* of member for twisting
x_o = Shear center x-coordinate relative to centroid of cross-section
y_o = Shear center y-coordinate relative to centroid of cross-section
r_o = Polar radius of gyration about shear center

$$= \sqrt{I_x / A + I_y / A + x_o^2 + y_o^2} \qquad (Eq.\ 2.3.1\text{-}7)$$

User Note:
The *unbraced lengths* L_x, L_y, and L_t are the distances between discrete support or brace locations that restrain translation and/or twisting. Locations of zero moment (inflection points) are not considered restraint locations. The bending coefficient C_b accounts for moment gradient including cases with moment reversal.

$$C_b = \frac{12.5M_{max}}{2.5M_{max} + 3M_A + 4M_B + 3M_C} \qquad (Eq.\ 2.3.1\text{-}8)$$

where
M_{max} = Absolute value of maximum moment in unbraced segment
M_A = Absolute value of moment at quarter point of unbraced segment
M_B = Absolute value of moment at centerline of unbraced segment
M_C = Absolute value of moment at three-quarter point of unbraced segment

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

For members without holes, the section properties defined above shall be based on the gross cross-section. For members with holes, section properties A, I_x, I_y, I_{xy}, J, j, x_o, y_o, r_o, and C_w, shall be replaced by A_{avg}, $I_{x,avg}$, $I_{y,avg}$, $I_{xy,avg}$, J_{avg}, j_{avg}, $x_{o,avg}$, $y_{o,avg}$, $r_{o,avg}$, and $C_{w,net}$, respectively in Eqs. 2.3.1-1 to 2.3.1-7, defined as follows:

$$A_{avg} = [A_g(s - L_h) + A_{net}L_h] / s \qquad (Eq.\ 2.3.1\text{-}9)$$

$$I_{x,avg} = [I_{x,g}(s - L_h) + I_{x,net}L_h] / s \qquad (Eq.\ 2.3.1\text{-}10)$$

$$I_{y,avg} = [I_{y,g}(s - L_h) + I_{y,net}L_h] / s \qquad (Eq.\ 2.3.1\text{-}11)$$

$$I_{xy,avg} = [I_{xy,g}(s - L_h) + I_{xy,net}L_h] / s \qquad (Eq.\ 2.3.1\text{-}12)$$

$$J_{avg} = [J_g(s - L_h) + J_{net}L_h] / s \qquad (Eq.\ 2.3.1\text{-}13)$$

$$j_{avg} = [j_g(s - L_h) + j_{net}L_h] / s \qquad (Eq.\ 2.3.1\text{-}14)$$

$$x_{o,avg} = [x_{o,g}(s - L_h) + x_{o,net}L_h] / s \qquad (Eq.\ 2.3.1\text{-}15)$$

$$y_{o,avg} = [y_{o,g}(s - L_h) + y_{o,net}L_h] / s \qquad (Eq.\ 2.3.1\text{-}16)$$

$r_{o,avg}$ = Polar radius of gyration calculated using Eq. 2.3.1-7 with modified section properties A_{avg}, $I_{x,avg}$, $I_{y,avg}$, $x_{o,avg}$, and $y_{o,avg}$

$C_{w,net}$ = Net warping constant assuming the cross-section *thickness* is zero at hole location(s)

where

A_g, A_{net} = *Gross* and *net cross-sectional area*, respectively

$I_{x,g}$, $I_{x,net}$ = Moment of inertia of gross and net cross-section about x-axis, respectively

$I_{y,g}$, $I_{y,net}$ = Moment of inertia of gross and net cross-section about y-axis, respectively

$I_{xy,g}$, $I_{xy,net}$ = Product of inertia of gross and net cross-section, respectively

J_g, J_{net} = Saint-Venant torsion constant of gross and net cross-section, respectively

j_g, j_{net} = Asymmetry property calculated according to Eq. 2.3.1-6 for gross and net cross-section, respectively

$x_{o,g}$, $x_{o,net}$ = Shear center x-coordinate relative to centroid for gross and net cross-section, respectively

$y_{o,g}$, $y_{o,net}$ = Shear center y-coordinate relative to centroid for gross and net cross-section, respectively

L_h = Length of each hole. For unequal hole lengths, it is permitted to conservatively use the largest hole length.

s = Center-to-center hole spacing. For non-uniformly spaced holes, it is permitted to conservatively use the closest center-to-center hole spacing. For a single hole, it is permitted to conservatively use half the unbraced length about axis of *buckling*.

Exception: For members with holes designed using the *Effective Width Method*, where hole sizes meet the limitations of Appendix 1.1.1 for compression members, or Appendix 1.1.3 for flexural members, global *buckling* forces and moments are permitted to be calculated using gross section properties.

2.3.1.1 Global Buckling for Compression Members (F_{cre}, P_{cre})

The global *buckling stress*, F_{cre}, shall be calculated as follows:

$$F_{cre} = P_{cre}/A_g \qquad (Eq.\ 2.3.1.1\text{-}1)$$

where

P_{cre} = Smallest global *buckling* force of member as determined in accordance with Sections 2.3.1.1.1 to 2.3.1.1.4, as applicable

A_g = *Gross* cross-sectional *area*

2.3.1.1.1 Sections Not Subject to Torsional or Flexural-Torsional Buckling

For *doubly-symmetric sections*, closed cross-sections, or any other cross-sections that can be shown not to be subjected to *torsional* or *flexural-torsional buckling*, the elastic *flexural buckling* force, P_{cre}, shall be calculated as follows:

$$P_{cre} = \frac{\pi^2 EI}{(KL)^2} \qquad (Eq.\ 2.3.1.1.1\text{-}1)$$

where

K = *Effective length factor* determined in accordance with Chapter C

L = *Unbraced length* about the axis of *buckling*

I = Moment of inertia about axis of *buckling*

P_{cre} shall be the smallest *flexural buckling* force for the member. For sections where the bracing directions do not align with the principal axes, P_{cre} shall also be checked for *buckling* about the minor principal axis using the largest unbraced length between adjacent brace locations.

For members with holes, I shall be replaced by I_{avg} as defined in Section 2.3.1.

2.3.1.1.2 Singly-Symmetric Sections Subject to Flexural-Torsional Buckling

For *singly-symmetric sections* subject to *flexural-torsional buckling*, the *buckling* force, P_{cre}, shall be taken as the smaller of P_{cre} calculated in accordance with Section 2.3.1.1.1 and P_{cre} calculated as follows, where the x-axis is the axis of symmetry:

$$P_{cre} = \frac{1}{2\beta}\left[(P_{ex} + P_t) - \sqrt{(P_{ex} + P_t)^2 - 4\beta P_{ex} P_t}\right] \quad (Eq.\ 2.3.1.1.2\text{-}1)$$

For members with holes, P_{ex}, P_t and β shall include the influence of holes in accordance with Section 2.3.1.

For *singly-symmetric* unstiffened angle *sections* not subject to *local buckling* at *stress* F_y, P_{cre} shall be computed using Section 2.3.1.1.1.

2.3.1.1.3 Doubly- or Point-Symmetric Sections Subject to Torsional Buckling

For *doubly-symmetric sections* and *point-symmetric sections* subject to *torsional buckling*, P_{cre} shall be taken as the smaller of P_t as defined in Section 2.3.1 and P_{cre} as calculated in Section 2.3.1.1.1, including the influence of holes if applicable.

2.3.1.1.4 Non-Symmetric Sections

For any cross-section, including *non-symmetric sections*, it is permitted to determine the global *buckling* force, P_{cre}, as the smallest value given by Eq. 2.3.1.1.4-1, Eq. 2.3.1.1.4-2, and the smallest positive root of Eq. 2.3.1.1.4-3 where x and y are the two perpendicular centroidal axes.

$$P_{cre} = \frac{1}{2\beta}\left[(P_{ex} + P_t) - \sqrt{(P_{ex} + P_t)^2 - 4\beta P_{ex} P_t}\right] \quad (Eq.\ 2.3.1.1.4\text{-}1)$$

$$P_{cre} = \frac{1}{2\gamma}\left[(P_{ey} + P_t) - \sqrt{(P_{ey} + P_t)^2 - 4\gamma P_{ey} P_t}\right] \quad (Eq.\ 2.3.1.1.4\text{-}2)$$

$$\begin{aligned}&(P_{cre} - P_{fx})(P_{cre} - P_{fy})(P_{cre} - P_t) - P_{cre}^2 (P_{cre} - P_{fy})\left(\frac{x_o}{r_o}\right)^2 \left(\frac{K_t L_t}{K_f L_f}\right)^2 \\ &- P_{cre}^2 (P_{cre} - P_{fx})\left(\frac{y_o}{r_o}\right)^2 \left(\frac{K_t L_t}{K_f L_f}\right)^2 + 2P_{cre}^2 P_{fxy}\left(\frac{x_o y_o}{r_o^2}\right)\left(\frac{K_t L_t}{K_f L_f}\right)^2 - (P_{cre} - P_t)P_{fxy}^2 = 0\end{aligned}$$

(*Eq.* 2.3.1.1.4-3)

where

$$P_{fx} = \frac{\pi^2 EI_x}{(K_f L_f)^2} \quad (Eq.\ 2.3.1.1.4\text{-}4)$$

$$P_{fy} = \frac{\pi^2 EI_y}{(K_f L_f)^2} \quad (Eq.\ 2.3.1.1.4\text{-}5)$$

$$P_{fxy} = \frac{\pi^2 EI_{xy}}{(K_f L_f)^2} \quad (Eq.\ 2.3.1.1.4\text{-}6)$$

where

K_fL_f = *Effective length* for coupled *flexural buckling*

= Smaller of K_xL_x and K_yL_y

For members with holes, P_{ex}, P_{ey}, P_t, β, and γ shall include the influence of holes in accordance with Section 2.3.1. I_x, I_y, and I_{xy} shall be replaced with $I_{x,avg}$, $I_{y,avg}$ and $I_{xy,avg}$, respectively, as defined in Section 2.3.1.

User Note:
If $K_xL_x < K_yL_y$, Eq. 2.3.1.1.4-1 does not control. If $K_yL_y < K_xL_x$, Eq. 2.3.1.1.4-2 does not control.
If $K_xL_x = K_yL_y$, the smallest root of Eq. 2.3.1.1.4-3 controls over Eqs. 2.3.1.1.4-1 and 2.3.1.1.4-2.

2.3.1.2 Global Buckling for Flexural Members (F_{cre}, M_{cre})

The global *buckling stress*, F_{cre}, shall be calculated as follows:

$$F_{cre} = M_{cre}/S_{fc} \quad (Eq.\ 2.3.1.2\text{-}1)$$

where

M_{cre} = Global *buckling* moment of member as determined in accordance with Sections 2.3.1.2.1 to 2.3.1.2.4, as applicable

S_{fc} = Gross elastic section modulus referenced to the extreme compression fiber

2.3.1.2.1 Sections Bending About Symmetric Axis

The global (*lateral-torsional*) *buckling* moment, M_{cre}, for *singly-* or *doubly-symmetric sections* bending about the symmetric x-axis, shall be calculated as follows:

$$M_{cre} = C_b r_o \sqrt{P_{ey} P_t} \quad (Eq.\ 2.3.1.2.1\text{-}1)$$

Alternatively, for *doubly-symmetric* I-sections bending about the x-axis, M_{cre} is permitted to be calculated using the following equation:

$$M_{cre} = \frac{C_b \pi^2 EdI_y}{2(K_y L_y)^2} \quad (Eq.\ 2.3.1.2.1\text{-}2)$$

where

d = Depth of cross-section

For members with holes, P_{ey}, P_t, r_o, and I_y shall include the influence of holes in accordance with Section 2.3.1.

2.3.1.2.2 Sections Bending About Non-Symmetric Principal Axis

The global (*lateral-torsional*) *buckling* moment, M_{cre}, for *singly-symmetric sections* bending about the centroidal y-axis perpendicular to the symmetric x-axis, or any cross-section bending about a non-symmetric principal y-axis, shall be calculated as follows:

$$M_{cre} = C_b P_{ex}\left(C_s|j| + \sqrt{j^2 + r_o^2 P_t / P_{ex}}\right) \quad (Eq.\ 2.3.1.2.2\text{-}1)$$

where

C_s = +1 for moment causing compression on shear center side of centroid

= -1 for moment causing tension on shear center side of centroid

For members with holes, P_{ex}, P_t, r_o, and j shall include the influence of holes in accordance with Section 2.3.1.

2.3.1.2.3 Point-Symmetric Sections

The global (*lateral-torsional*) *buckling* moment, M_{cre}, for *point-symmetric Z-sections* bending about an x-axis that is perpendicular to the *web* and through the centroid shall be calculated as follows:

$$M_{cre} = \frac{C_b r_o}{2}\sqrt{P_{ey} P_t} \quad (Eq.\ 2.3.1.2.3\text{-}1)$$

Alternatively, M_{cre} is permitted to be calculated using Eq. 2.3.1.2.3-2:

$$M_{cre} = \frac{C_b \pi^2 E d I_y}{4(K_y L_y)^2} \quad (Eq. 2.3.1.2.3\text{-}2)$$

where

d = Depth of cross-section

For members with holes, P_{ey}, P_t, r_o, and I_y shall include the influence of holes in accordance with Section 2.3.1.

2.3.1.2.4 Closed-Box Section

The global (*lateral-torsional*) *buckling* moment, M_{cre}, for closed-box sections shall be calculated as follows:

$$M_{cre} = \frac{C_b \pi}{K_y L_y}\sqrt{E I_y G J} \quad (Eq.\ 2.3.1.2.4\text{-}1)$$

For members with holes, I_y and J shall include the influence of holes in accordance with Section 2.3.1.

2.3.1.2.5 Biaxial Bending

The global (*lateral-torsional*) *buckling* moment of a member subject to biaxial bending shall be determined as follows:

$$M_{cre} = C_b P_e\left(-j + \sqrt{j^2 + r_o^2 P_t / P_e}\right) \quad (Eq.\ 2.3.1.2.5\text{-}1)$$

C_b = Bending coefficient defined in Appendix 2 Section 2.3.1

$$P_e = \frac{\pi^2 E I_1 I_2}{I_b (KL)^2} \quad (Eq.\ 2.3.1.2.5\text{-}2)$$

I_1, I_2 = Moments of inertia about the major and minor principal axes, respectively

I_b = Moment of inertia about the axis of bending

$$= I_1\left(\frac{M_{1r}}{M_r}\right)^2 + I_2\left(\frac{M_{2r}}{M_r}\right)^2 \quad (Eq.\ 2.3.1.2.5\text{-}3)$$

M_{1r} = *Required strength* [moment due to *factored loads*] in bending about the major principal axis

M_{2r} = *Required strength* [moment due to *factored loads*] in bending about the minor principal axis

$$M_r = \sqrt{M_{1r}^2 + M_{2r}^2} \quad (Eq.\ 2.3.1.2.5\text{-}4)$$

KL = *Effective length* for *lateral-torsional buckling* in accordance with Chapter C

$$j = (y_a - y_o)\frac{M_{1r}}{M_r} - (x_a - x_o)\frac{M_{2r}}{M_r} \quad (Eq.\ 2.3.1.2.5\text{-}5)$$

$$x_a = \frac{1}{2I_y}\int_A x(x^2 + y^2)dA \quad (Eq.\ 2.3.1.2.5\text{-}6)$$

$$y_a = \frac{1}{2I_x}\int_A y(x^2 + y^2)dA \quad (Eq.\ 2.3.1.2.5\text{-}7)$$

The x- and y-axes used in calculating j, x_o, y_o, x_a, and y_a correspond to the principal centroidal 1- and 2-axes, respectively, as shown in Figure 2.3.1.2.5-1. The moment directions shown in Figure 2.3.1.2.5-1 are positive. The variables E, P_t, r_o, x_o, and y_o are defined in Appendix 2 Section 2.3.1.

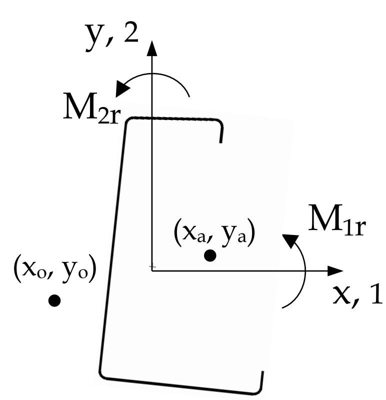


Figure 2.3.1.2.5-1 Biaxial Bending

2.3.2 Local Buckling

The *local buckling* forces and moments for *cold-formed steel structural members* are permitted to be determined analytically in accordance with this section.

2.3.2.1 Local Buckling for Compression Members ($F_{cr\ell}$, $P_{cr\ell}$)

The *local buckling* force, $P_{cr\ell}$, of a member shall be based on the lowest *buckling stress* among elements in the cross-section as follows:

$$P_{cr\ell} = A_g F_{cr\ell} \qquad (Eq.\ 2.3.2.1\text{-}1)$$

where

A_g = *Gross cross-sectional area*

$F_{cr\ell}$ = Smallest *local buckling stress* of all elements in cross-section

$$= k\frac{\pi^2 E}{12(1-\mu^2)}\left(\frac{t}{w}\right)^2 \qquad (Eq.\ 2.3.2.1\text{-}2)$$

where

k = Plate *buckling* coefficient provided in Appendix 1 for different types of elements and supporting conditions

E = Modulus of elasticity of steel

t = Element *thickness*

μ = Poisson's ratio of steel

w = Element flat width

User Note:
Determining the *local buckling* force by using the smallest of the element (*flange*, *web*, lip, etc.) *local buckling stresses* can be very conservative if one element is much more slender than the rest of the elements in the cross-section. Numerical solutions or more advanced analytical solutions are recommended in this case.

The *local buckling stress* for elements with holes shall be calculated as both unstiffened elements at the hole location and as a separate element where the hole is not located. For the unstiffened elements at the hole location, the *buckling stress* shall be modified to account for the net section by multiplying by the ratio A_{net}/A_g.

2.3.2.2 Local Buckling for Flexural Members ($F_{cr\ell}$, $M_{cr\ell}$)

The *local buckling* moment, $M_{cr\ell}$, of a member shall be based on the smallest *buckling stress* among elements in the cross-section, referenced to the extreme compression fiber, as follows:

$$M_{cr\ell} = S_{fc} F_{cr\ell} \qquad (Eq.\ 2.3.2.2.\text{-}1)$$

where

S_{fc} = Gross elastic cross-sectional modulus referenced to the extreme compression fiber

$F_{cr\ell}$ = *Local buckling stress* at extreme compression fiber

$$= k\frac{\pi^2 E}{12(1-\mu^2)}\left(\frac{t}{w}\right)^2 \qquad (Eq.\ 2.3.2.2\text{-}2)$$

where

k = Plate *buckling* coefficient, provided in Appendix 1 for different types of elements and supporting conditions

E = Modulus of elasticity of steel

t = Element *thickness*
μ = Poisson's ratio of steel
w = Element *flat width*

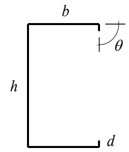
User Note:
The first step in the application of this method is the determination of the *local buckling stress* of all the elements (*flange*, *web*, lip, etc.). The *local buckling* moment or *stress* is controlled by the element *local buckling stress* that results in the smallest *stress* level when linearly extrapolated to the extreme compression fiber.

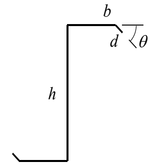
The *local buckling stress* for elements with holes shall be calculated as both unstiffened elements at the hole location and as a separate element where the hole is not located. For the unstiffened elements at the hole location, the *buckling stress* shall be modified to account for the net section by multiplying the *buckling stress* times the ratio S_{fcnet}/S_{fc}, where S_{fcnet} is the net section modulus referenced to the extreme compression fiber.

2.3.3 Distortional Buckling

The *distortional buckling* forces and moments for *cold-formed steel structural members* are permitted to be determined analytically in accordance with this section. Table 2.3.3-1 provides geometric properties for a *flange* with a simple lip stiffener to be used in this section. Other types of stiffeners are permitted.

Table 2.3.3-1
Geometric Flange Plus Lip Properties for C- and Z-Sections[1, 2,3]





$A_f = (b+d)t$	$A_f = (b+d)t$
$J_f = \frac{1}{3}bt^3 + \frac{1}{3}dt^3$	$J_f = \frac{1}{3}bt^3 + \frac{1}{3}dt^3$
$I_{xf} = \dfrac{t\left(t^2b^2 + 4bd^3 + t^2bd + d^4\right)}{12(b+d)}$	$I_{xf} = \dfrac{t\left(t^2b^2 + 4bd^3 - 4bd^3\cos^2(\theta) + t^2bd + d^4 - d^4\cos^2(\theta)\right)}{12(b+d)}$
$I_{yf} = \dfrac{t\left(b^4 + 4db^3\right)}{12(b+d)}$	$I_{yf} = \dfrac{t\left(b^4 + 4db^3 + 6d^2b^2\cos(\theta) + 4d^3b\cos^2(\theta) + d^4\cos^2(\theta)\right)}{12(b+d)}$
$I_{xyf} = \dfrac{tb^2d^2}{4(b+d)}$	$I_{xyf} = \dfrac{tbd^2\sin(\theta)\left(b + d\cos(\theta)\right)}{4(b+d)}$
$C_{wf} = 0$	$C_{wf} = 0$
$x_{of} = \dfrac{b^2}{2(b+d)}$	$x_{of} = \dfrac{b^2 - d^2\cos(\theta)}{2(b+d)}$
$x_{hf} = \dfrac{-\left(b^2 + 2db\right)}{2(b+d)}$	$x_{hf} = \dfrac{-\left(b^2 + 2db + d^2\cos(\theta)\right)}{2(b+d)}$
$y_{hf} = y_{of} = \dfrac{-d^2}{2(b+d)}$	$y_{hf} = y_{of} = \dfrac{-d^2\sin(\theta)}{2(b+d)}$

Notes:

1. b, d, and h are mid-line dimensions of cross-section.
2. x-y axis system is located at the centroid of the *flange* with x positive to the right from the centroid, perpendicular to the *web*, and y positive down from the centroid, parallel to the *web*. Table 2.3.3-1 does not include the effect of corner radius. More refined values are permitted.
3. Variables are defined as follows:

A_f = *Cross-sectional area* of *flange*
t = *Thickness* of cross-section
J_f = St. Venant torsion constant of *flange*
I_{xf} = x-axis moment of inertia of *flange*
I_{yf} = y-axis moment of inertia of *flange*
I_{xyf} = Product of the moment of inertia of *flange*
C_{wf} = Warping torsion constant of *flange*
x_{of} = x distance from centroid of *flange* to shear center of *flange*
y_{of} = y distance from centroid of *flange* to shear center of *flange*
x_{hf} = x distance from centroid of *flange* to *flange*/*web* junction
y_{hf} = y distance from centroid of *flange* to *flange*/*web* junction

2.3.3.1 Distortional Buckling for Compression Members (F_{crd}, P_{crd})

The provisions of this section shall apply to any open cross-section with stiffened *flanges* of equal dimension where the stiffener is either a simple lip or a complex edge stiffener. The elastic *distortional buckling* load, P_{crd}, shall be calculated as follows:

$$P_{crd} = A_g F_{crd} \quad (Eq.\ 2.3.3.1\text{-}1)$$

where

A_g = *Gross cross-sectional area*

$$F_{crd} = \frac{\bar{k}_{\phi fe} + \bar{k}_{\phi we} + \bar{k}_{\phi}}{\tilde{k}_{\phi fg} + \tilde{k}_{\phi wg}} \quad (Eq.\ 2.3.3.1\text{-}2)$$

where

$\bar{k}_{\phi fe}$ = Elastic rotational stiffness provided by the *flange* to the *flange*/*web* juncture

$$= \left(\frac{\pi}{L_d}\right)^4 \left[EC_{wf} + EI_{xf}\left(x_{of} - x_{hf}\right)^2\left(1 - \frac{I_{xyf}^2}{I_{xf}I_{yf}}\right)\right] + \left(\frac{\pi}{L_d}\right)^2 GJ_f \quad (Eq.\ 2.3.3.1\text{-}3)$$

$\bar{k}_{\phi we}$ = Elastic rotational stiffness provided by the *web* to *flange*/*web* juncture

$$= \frac{Et^3}{12(1-\mu^2)}\left(\frac{2}{h_o}\right) \quad (Eq.\ 2.3.3.1\text{-}4)$$

where

h_o = Out-to-out *web* depth (See Figure 1.1.2-2)

t = Base steel *thickness*

μ = Poisson's ratio of steel

$\bar{k}_{\phi}$ = Continuous rotational stiffness (i.e., per unit length) provided by a component (brace, panel, sheathing) that restrains rotation about the *flange*/*web* juncture of a member

= 0 for unrestrained *flange*

$\bar{k}_{\phi}$ is permitted to be conservatively taken as zero. If rotational stiffness provided to the two *flanges* is dissimilar, the smaller rotational stiffness shall be used. For sheathing-based restraint, AISI S240 Appendix 1 is permitted to be used for analytical determination or ANSI/SDI AISI S918 for test-based determination of continuous rotational stiffness.

$\tilde{k}_{\phi fg}$ = Geometric rotational stiffness demanded by *flange* from *flange*/*web* juncture

$$= \left(\frac{\pi}{L_d}\right)^2 \left\{I_{xf} + I_{yf} + A_f\left[x_{hf}^2 + y_{of}^2 - 2y_{of}\left(x_{of} - x_{hf}\right)\left(\frac{I_{xyf}}{I_{yf}}\right) + \left(x_{of} - x_{hf}\right)^2\left(\frac{I_{xyf}}{I_{yf}}\right)^2\right]\right\}$$

(*Eq.* 2.3.3.1-5)

$\tilde{k}_{\phi wg}$ = Geometric rotational stiffness demanded by *web* from *flange*/*web* juncture

$$= \left(\frac{\pi}{L_d}\right)^2 \frac{th_o^3}{60} \quad (Eq.\ 2.3.3.1\text{-}6)$$

where

L_d = Minimum of L_{crd} and L_m

where

$$L_{crd} = \pi h_o \left\{ \frac{6(1-\mu^2)}{t^3 h_o^3} \left[C_{wf} + I_{xf}\left(x_{of} - x_{hf}\right)^2 \left(1 - \frac{I_{xyf}^2}{I_{xf} I_{yf}}\right)\right]\right\}^{1/4} \quad (Eq.\ 2.3.3.1\text{-}7)$$

L_m = Distance between discrete restraints that restrict *distortional buckling* (for *flanges* with continuous rotational stiffness provided by restraining components, $L_m = L_{crd}$)

Variables A_f, J_f, I_{xf}, I_{yf}, I_{xyf}, C_{wf}, x_{of}, y_{of}, and x_{hf} are defined in Table 2.3.3-1, and other variables are defined in Section 2.3.1. For members subject to *distortional buckling* which do not meet the geometric criteria of this section, a numerical solution shall be used in accordance with Section 2.2.

2.3.3.2 Distortional Buckling for Flexural Members (F_{crd}, M_{crd})

The provisions of this section are permitted to apply to any open cross-section with a single *web* and edge-stiffened *flanges* extending to one side of the *web,* where the stiffener is either a simple lip or a complex edge stiffener. The elastic *distortional buckling* moment, M_{crd}, shall be calculated as follows:

$$M_{crd} = S_{fc} F_{crd} \quad (Eq.\ 2.3.3.2\text{-}1)$$

where

$$F_{crd} = \beta \frac{\bar{k}_{\phi fe} + \bar{k}_{\phi we} + \bar{k}_{\phi}}{\tilde{k}_{\phi fg} + \tilde{k}_{\phi wg}} \quad (Eq.\ 2.3.3.2\text{-}2)$$

where

β = A value accounting for moment gradient, which is permitted to be conservatively taken as 1.0

$$= 1.0 \le 1 + 0.4(L_d/L_m)^{0.7}(1 + M_1/M_2)^{0.7} \le 1.3 \quad (Eq.\ 2.3.3.2\text{-}3)$$

where

L_d = Minimum of L_{crd} and L_m

where

L_{crd} = Critical unbraced length for *distortional buckling*, given by Eq. 2.3.3.2-4 or Eq. 2.3.3.2-8

L_m = Distance between discrete restraints that restrict *distortional buckling* (for *flanges* with continuous rotational stiffness provided by restraining components, $L_m = L_{crd}$)

M_1 and M_2 = Smaller and larger end moments, respectively, in the unbraced segment (L_m) of the beam; M_1/M_2 is positive when the moments cause reverse curvature and negative when bent in single curvature

$\bar{k}_{\phi}$ = Continuous rotational stiffness (i.e., per unit length) provided by a component (brace, panel, sheathing) that restrains rotation about the *flange*/*web* juncture of a member

= 0 for unrestrained compression *flange*

$\bar{k}_{\phi}$ is permitted to be conservatively taken as zero. For sheathing-based restraint, AISI S240 Appendix 1 is permitted to be used for analytical determination or ANSI/SDI AISI S918 for test-based determination of continuous rotational stiffness.

Variables S_{fc}, L_{crd}, $\bar{k}_{\phi fe}$, $\bar{k}_{\phi we}$, $\tilde{k}_{\phi fg}$, $\tilde{k}_{\phi wg}$ are defined as follows:

(a) For bending about the axis perpendicular to the *web*:

S_{fc} = Gross elastic cross-sectional modulus referenced to the compression fiber at the *flange/web* juncture, the point at which h_o is measured

$$L_{crd} = \pi h_o \left\{ \frac{4(1-\mu^2)}{t^3 h_o^3}\left[C_{wf} + I_{xf}\left(x_{of} - x_{hf}\right)^2 \left(1 - \frac{I_{xyf}^2}{I_{xf} I_{yf}}\right)\right] + \frac{1}{720}\right\}^{1/4} \qquad (Eq.\ 2.3.3.2\text{-}4)$$

$\bar{k}_{\phi fe}$ = Elastic rotational stiffness provided by the *flange* to the *flange/web* juncture, given in Eq. 2.3.3.1-3

$\bar{k}_{\phi we}$ = Elastic rotational stiffness provided by the *web* to the *flange/web* juncture

$$= \frac{Et^3}{12(1-\mu^2)}\left(\frac{3}{h_o}\right)\left[1 + \frac{2}{15}\left(\frac{\pi h_o}{L_d}\right)^2 + \frac{1}{720}\left(\frac{\pi h_o}{L_d}\right)^4\right] \qquad (Eq.\ 2.3.3.2\text{-}5)$$

$\tilde{k}_{\phi fg}$ = Geometric rotational stiffness demanded by the *flange* from the *flange/web* juncture

$$= \left(\frac{\pi}{L_d}\right)^2 \left\{ I_{xf} + I_{yf} + A_f\left[x_{hf}^2 + y_{of}^2 - 2y_{of}\left(x_{of} - x_{hf}\right)\left(\frac{I_{xyf}}{I_{yf}}\right)\right]\right\} \qquad (Eq.\ 2.3.3.2\text{-}6)$$

$\tilde{k}_{\phi wg}$ = Geometric rotational stiffness demanded by the *web* from the *flange/web* juncture

$$= \left(\frac{\pi}{L_d}\right)^2 \left(\frac{t h_o^3}{240}\right)\left[\frac{1110 + 810(1-\xi_{web}) + 8\left(\frac{\pi h_o}{L_d}\right)^2 + \left(\frac{\pi h_o}{L_d}\right)^4}{420 + 28\left(\frac{\pi h_o}{L_d}\right)^2 + \left(\frac{\pi h_o}{L_d}\right)^4}\right] \qquad (Eq.\ 2.3.3.2\text{-}7)$$

where

ξ_{web} = $(f_1 - f_2)/f_1$, *stress* gradient in the *web*, where f_1 and f_2 are the *stresses* at the opposite ends of the *web*, $f_1 > f_2$, compression is positive, tension is negative, and the *stresses* are calculated on the basis of the gross section (e.g., pure symmetrical bending, $f_2 = -f_1$, $\xi_{web} = 2$)

(b) For bending about the axis parallel to the *web*, with the *web* in tension:

S_{fc} = Gross elastic cross-sectional modulus referenced to the extreme compression fiber of the *flange* and stiffener

$$L_{crd} = \pi h_e \left\{ \frac{6(1-\mu^2)}{t^3 h_e^3} \left[C_{wf} + I_{xf}(x_{of} - x_{hf})^2 \left(1 - \frac{I_{xyf}^2}{I_{xf} I_{yf}} \right) \right] + \frac{1}{120} \right\}^{1/4} \qquad (\textit{Eq. } 2.3.3.2\text{-}8)$$

$\bar{k}_{\phi fe}$ = Elastic rotational stiffness provided by the *flange* to the *flange*/*web* juncture, given in Eq. 2.3.3.1-3

$\bar{k}_{\phi we}$ = Elastic rotational stiffness provided by the *web* to the *flange*/*web* juncture

$$= \frac{Et^3}{12(1-\mu^2)} \left(\frac{2}{h_e} \right) \left[1 + \frac{1}{6} \left(\frac{\pi h_e}{L_d} \right)^2 + \frac{1}{120} \left(\frac{\pi h_e}{L_d} \right)^4 \right] \qquad (\textit{Eq. } 2.3.3.2\text{-}9)$$

$\tilde{k}_{\phi fg}$ = Geometric rotational stiffness demanded by the *flange* from the *flange*/*web* juncture

$$= \left(\frac{\pi}{L_d} \right)^2 \left\{ I_{xf} + I_{yf} + A_f \left[x_{hf}^2 + y_{of}^2 - 2y_{of}(x_{of} - x_{hf}) \left(\frac{I_{xyf}}{I_{yf}} \right) \right] \right\} \psi_f + \left(\frac{\pi}{L_d} \right)^2 I_{yf} \xi_f \qquad (\textit{Eq. } 2.3.3.2\text{-}10)$$

$\tilde{k}_{\phi wg}$ = 0

$$h_e = 3.5 \sqrt{\frac{I_{yf}}{A_f} + x_{hf}^2} \qquad (\textit{Eq. } 2.3.3.2\text{-}11)$$

ξ_f = $(f_1 - f_2)/f_1$, *stress* gradient in the *flange* and edge stiffener, where f_1 is the *stress* at the extreme compression fiber of the *flange* and edge stiffener, f_2 is the *stress* at the *flange*/*web* juncture, compression is positive, tension is negative, and the *stresses* are calculated on the basis of the gross cross-section (see Figure 2.3.3.2-1)

ψ_f = f_{cg}/f_1, *stress* ratio in the *flange* and edge stiffener, where f_1 is the *stress* at the extreme compression fiber of the *flange* and edge stiffener, f_{cg} is the *stress* at the centroid of the *flange* and edge stiffener, compression is positive, tension is negative, and the *stresses* are calculated on the basis of the gross cross-section (see Figure 2.3.3.2-1)

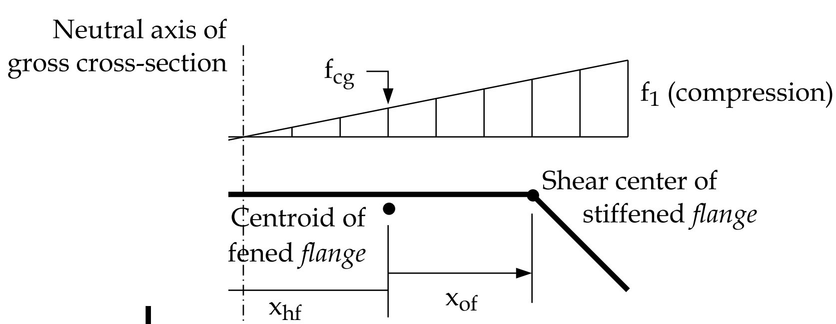


Figure 2.3.3.2-1 Flange Stresses for Bending About Axis Parallel to Web

All other variables are defined in Section 2.3.3.1.

For members subject to *distortional buckling* which do not meet the geometric criteria of this section, a numerical solution shall be used in accordance with Section 2.2.

2.3.3.3 Distortional Buckling for Members With Holes

For members meeting the geometric criteria of Sections 2.3.3.1 and 2.3.3.2, and having a single hole in the *web* or a single line of *web* holes longitudinally, the *distortional buckling* force and moment shall be determined in accordance with Sections 2.3.3.1 and 2.3.3.2, respectively, provided that *thickness*, t, in Eqs. 2.3.3.1-4, 2.3.3.1-6, 2.3.3.2-5, 2.3.3.2-7, and 2.3.3.2-9 be replaced by modified thickness, t_r, as follows:

For $L_h \le L_{dh}$

$$t_r = t\left(1-\frac{L_h}{L_{dh}}\right)^{1/3} \qquad (Eq.\ 2.3.3.3\text{-}1)$$

For $L_h > L_{dh}$

$t_r = 0$

where

t = *Thickness* of *web*

L_h = Hole length

L_{dh} = Minimum of L_{crd}, L_m and s

L_{crd} = D*istortional buckling* half-wavelength of member with gross cross-section, determined numerically or using Eq. 2.3.3.1-7, Eq. 2.3.3.2-4 or Eq. 2.3.3.2-8

L_m = Distance between discrete restraints that restrict *distortional buckling* (for *flanges* with continuous rotational stiffness provided by restraining components, $L_m = L_{crd}$)

s = Longitudinal center-to-center hole spacing

For members meeting the geometric criteria of Sections 2.3.3.1 and 2.3.3.2, and having other *web* hole patterns, the *distortional buckling* force and moment shall be determined in accordance with Sections 2.3.3.1 and 2.3.3.2, respectively, provided that *thickness*, t, in Eqs. 2.3.3.1-4, 2.3.3.1-6, 2.3.3.1-7, 2.3.3.2-4, 2.3.3.2-5, 2.3.3.2-7, 2.3.3.2-8, and 2.3.3.2-9 are replaced by modified thickness, t_r, as follows:

$$t_r = t\left(\frac{A_{web,net}}{A_{web,gross}}\right)^{1/3} \ge 0 \qquad (Eq.\ 2.3.3.3\text{-}2)$$

where

t = *Thickness* of *web*

$A_{web,net}$ = *Web* surface area along member length subtracting the hole areas

$A_{web,gross}$ = *Web* surface area along member length

For *distortional buckling* of other members with holes, a numerical solution shall be used in accordance with Section 2.2.

2.3.4 Shear Buckling (V_{cr})

The elastic *shear buckling* force, V_{cr}, is permitted to be determined as follows:

$$V_{cr} = \frac{\pi^2 E k_v h t}{12(1-\mu^2)(h/t)^2} \quad (Eq.\ 2.3.4\text{-}1)$$

where
V_{cr} = Elastic *shear buckling* force of the *web*
E = Modulus of elasticity of steel
μ = Poisson's ratio of steel
h = Depth of the flat portion of *web* measured along the plane of the *web*
t = *Thickness* of *web*
k_v = *Shear buckling* coefficient calculated in accordance with Section G2.3, or for any open cross-section with a single *web* and single edge-stiffened compression *flange* extending to one side of the *web* where the stiffener is either a simple lip or a complex edge stiffener, the *shear buckling* coefficient may be calculated as follows:

$$k_v = \frac{0.9}{\sin 2\varphi}\left[\frac{1}{(L_v/h)^2\cos^2\varphi} + C_1(L_v/h)^2\cos^2\varphi + C_2(1+2\sin^2\varphi)\right] \quad (Eq.\ 2.3.4\text{-}2)$$

where

$$\varphi = \arccos\left(\sqrt{C_3 + \sqrt{{C_3}^2 + C_4}}\right) \quad (Eq.\ 2.3.4\text{-}3)$$

$$C_1 = \frac{5.143\varepsilon^2 + 64.58\varepsilon + 108.6}{\varepsilon^2 + 20.57\varepsilon + 108.6} \quad (Eq.\ 2.3.4\text{-}4)$$

$$C_2 = \frac{2.472\varepsilon^2 + 41.14\varepsilon + 217.2}{\varepsilon^2 + 20.57\varepsilon + 108.6} \quad (Eq.\ 2.3.4\text{-}5)$$

$$C_3 = \frac{1.5C_2 - \dfrac{2}{(L_v/h)^2}}{4C_2 + C_1(L_v/h)^2} \quad (Eq.\ 2.3.4\text{-}6)$$

$$C_4 = \frac{\dfrac{3}{(L_v/h)^2}}{4C_2 + C_1(L_v/h)^2} \quad (Eq.\ 2.3.4\text{-}7)$$

L_v= *Shear buckling* half-wavelength taken as the minimum of 0.85h and the distance between transverse *web* stiffeners meeting the requirements of Section G4

$$\varepsilon = \frac{\bar{k}_{\phi fe} h}{Et^3/[12(1-\mu^2)]} \quad (Eq.\ 2.3.4\text{-}8)$$

where
$\bar{k}_{\phi fe}$ = Elastic rotational stiffness provided by the *flange* to the *flange*/*web* juncture, as given in Eq. 2.3.3.1-3 with L_d taken as L_v

APPENDIX 3, MEMBERS UNDER COMBINED FORCES – ALTERNATIVE PROCEDURE

This appendix provides an alternative procedure for evaluating strength *limit states* for *structural members* under combined forces consisting of axial load and/or bending.

This appendix is organized as follows:

3.1 General Requirements
3.2 Yielding and Global Buckling
3.3 Local Buckling Interacting With Yielding and Global Buckling
3.4 Distortional Buckling

3.1 General Requirements

Structural members shall be designed such that the generalized *available strength* [*factored resistance*] under a specific combination of forces is greater than or equal to the generalized *required strength* [effect due to *factored loads*] as follows:

$$\beta_r \le \beta_a \qquad (Eq.\ 3.1\text{-}1)$$

where

β_r = Generalized *Required strength* [effect due to *factored loads*], i.e., demand, under the combined forces as defined in Section 3.1.1

β_a = Generalized *Available strength* [*factored resistance*], i.e., capacity, under the combined forces as defined in Section 3.1.2

User Note:
The term "generalized" in the β definitions refers to the vector summation of the loads, normalized to first yield. The commentary provides further detail.

3.1.1 Required Strength Under Combined Forces

The generalized *required strength* [effect due to *factored loads*] for axial load and bending shall be defined as:

$$\beta_r = \sqrt{\left(\frac{P_r}{P_y}\right)^2 + \left(\frac{M_{1r}}{M_{1y}}\right)^2 + \left(\frac{M_{2r}}{M_{2y}}\right)^2} \qquad (Eq.\ 3.1.1\text{-}1)$$

where

M_{1r} = *Required strength* [effect due to *factored loads*] in bending about the major principal axis

M_{2r} = *Required strength* [effect due to *factored loads*] in bending about the minor principal axis

P_r = *Required strength* [effect due to *factored loads*] under axial load, compression positive, tension negative

M_{1y} = Moment about the major principal axis that causes first yield in the section

M_{2y} = Moment about the minor principal axis that causes first yield in the section

P_y = Axial load that causes yield in the section

3.1.2 Available Strength Under Combined Forces

Design for strength shall be in accordance with *LRFD*, *ASD*, or *LSD* as appropriate:

$\beta_a = \phi\beta_n$ (*LRFD*, *LSD*) (*Eq.* 3.1.2-1a)

$= \beta_n / \Omega$ (*ASD*) (*Eq.* 3.1.2-1b)

where

β_a = Generalized *available strength* [*factored resistance*]

β_n = Generalized *nominal strength* [*resistance*], determined as the minimum of β_{ne}, $\beta_{n\ell}$, or β_{nd} calculated in accordance with Sections 3.2 to 3.4, as applicable

ϕ = R*esistance factor*

$= \phi_a + (\phi_b - \phi_a)\sin(\varphi_{PM})$ (*Eq.* 3.1.2-2)

where

$\phi_a = \phi_c$ for axial compression, $P_r > 0$

= 0.85 (*LRFD*)

= 0.80 (*LSD*)

= ϕ_t for axial tension, $P_r < 0$

= 0.90 (*LRFD, LSD*)

ϕ_b = 0.90 (*LRFD*, *LSD*)

Ω = S*afety factor* defined as follows:

$= \Omega_a + (\Omega_b - \Omega_a)\sin(\varphi_{PM})$ (*Eq.* 3.1.2-3)

where

$\Omega_a = \Omega_c$ for axial compression, $P_r > 0$

= 1.80 (*ASD*)

= Ω_t for axial tension, $P_r < 0$

= 1.67 (*ASD*)

Ω_b = 1.67 (*ASD*)

$\varphi_{PM} = \cos^{-1}\left((P_r / P_y)/\beta_r\right)$ (*Eq.* 3.1.2-4)

Variables P_r, P_y, and β_r are defined in Section 3.1.1.

User Note:
For axial demand P_r, compression is positive and tension is negative. This implies $0 \le \varphi_{PM} < \pi/2$ includes axial compression in the generalized demand, and $\pi/2 \le \varphi_{PM} \le \pi$ includes axial tension in the generalized demand.

User Note:
For the special case of biaxial bending without axial load (i.e., $P_r = 0$) many equations simplify. For Section 3.1, $\phi = \phi_b$ and $\Omega = \Omega_b$. See Sections 3.2 and 3.4 for other simplifications.

3.2 Yielding and Global Buckling, β_{ne}

The generalized *nominal strength* [*resistance*], β_{ne}, for global *buckling* and/or *yielding* shall be calculated as follow:

For *load* including axial compression ($P_r > 0$)

$$\beta_{ne} = \beta_{neP} + (\beta_{neM} - \beta_{neP}) \sin \varphi_{PM} \qquad (Eq.\ 3.2\text{-}1)$$

For *load* including axial tension or no axial *load* ($P_r \leq 0$)

$$\beta_{ne} = \beta_{neM} \qquad (Eq.\ 3.2\text{-}2)$$

where

β_{neP} = Generalized *nominal strength* [*resistance*] based on axial strength provisions calculated as follows:

For $\beta_{cre} > 0.444\beta_p$; $\beta_{neP} = 0.658^{(\beta_p/\beta_{cre})}\beta_p$ (*Eq.* 3.2-3)

For $\beta_{cre} \leq 0.444\beta_p$; $\beta_{neP} = 0.877\ \beta_{cre}$ (*Eq.* 3.2-4)

β_{neM} = Generalized *nominal strength* [*resistance*] based on flexural strength provisions calculated as follows:

For $\beta_{cre} > 0.5\beta_p$; $\beta_{neM} = \beta_p\ (1\text{-}0.25\beta_p/\beta_{cre})$ (*Eq.* 3.2-5)

For $\beta_{cre} \leq 0.5\beta_p$; $\beta_{neM} = \beta_{cre}$ (*Eq.* 3.2-6)

φ_{PM} = Angle (in radians) quantifying relative magnitude of axial and flexural demand as defined in *Eq*. 3.1.2-4

β_{cre} = Magnitude of the generalized demand under the combined forces that cause elastic global *buckling*, including the influence of holes if applicable

User Note:
Elastic global *buckling,* commonly known as *lateral-torsional buckling* in beams and *flexural* (or *flexural-torsional*) *buckling* in columns, is considered here under the *stresses* developed from the combined forces, not under each separate case. This may readily be accomplished with numerical methods such as the Finite Strip Method and Finite Element Method as discussed further in Appendix 2 and *Commentary* Section 3.2.

For the special case of biaxial bending without axial load (i.e., $P_r = 0$), the global *buckling* moment about the resultant axis of bending, M_{cre}, may be calculated per Appendix 2 Section 2.3.1.2.5, noting that $\beta_{cre} = \dfrac{M_{cre}}{M_r}\beta_r$ where $M_r = \sqrt{M_{1r}^2 + M_{2r}^2}$.

β_p = Magnitude of generalized demand under the combined forces that causes the entire section to be plastic

User Note:
β_p is the generalized equivalent to M_p in flexure. It is a multiple of the generalized demand at which the cross-section is fully plastic, where all *stresses* are either positive or negative F_y. The *stress* resultants: P, M_1, M_2 must be proportional to the required demands and increased until full plasticity is reached: i.e., $\beta_p / \beta_r = P/P_r = M_1/M_{1r} = M_2/M_{2r}$. This is relatively complex to solve without numerical tools. See *Commentary* Section 3.2 for additional discussion.

It is permitted to calculate β_{ne} using β_p determined in accordance with Eq. 3.2-7 provided that β_{ne} is limited to a maximum of β_y.

$\beta_p = (1 + 0.2 \sin \varphi_{PM})\beta_y$ (*Eq.* 3.2-7)

where

β_y = Magnitude of the generalized demand under the combined forces that causes first yield in the section

User Note:
β_y is a scalar multiple of the generalized demand β_r. The multiple is the ratio of the *yield stress* to the absolute value of the maximum *stress* developed in the section due to the combined forces, i.e.:

$$\beta_y = \left(F_y \Big/ \max\left| \frac{P_r}{A} + \frac{M_{1r}c_2}{I_1} + \frac{M_{2r}c_1}{I_2} \right| \right) \beta_r \qquad \text{(Eq. 3.2-8)}$$

where A, I_1, and I_2 are the cross-section area and major and minor axis moment of inertia respectively, and c_1 and c_2 are the distance from the centroid to any fiber on the cross-section in the principal coordinate system. This calculation can be done by hand, but must be checked for all fibers in the cross-section, thus numerical tools are recommended. See *Commentary* Section 3.2 for additional discussion.

If the effect of cold work of forming is included in determining the *yield stress* F_y, β_{ne} shall not exceed β_y.

3.3 Local Buckling Interacting With Yielding and Global Buckling, $\beta_{n\ell}$

The *nominal strength* [*resistance*] for *local buckling*, *yielding*, and interaction with global *buckling* where applicable, shall be calculated in accordance with this section for $\lambda_\ell \le 5$.

$$\beta_{n\ell} = k_\ell \bar{\beta}_{ne} \frac{1 + a_\ell \lambda_\ell^2}{1 + b_\ell \lambda_\ell^2} \le \beta_{y3} \qquad (Eq.\ 3.3\text{-}1)$$

where

$$\lambda_\ell = \sqrt{\frac{\bar{\beta}_{ne}}{\beta_{cr\ell}}} \qquad (Eq.\ 3.3\text{-}2)$$

$\bar{\beta}_{ne} = \min(\beta_{ne}, \beta_y)$

β_{ne} = A variable defined in Section 3.2

β_y = A variable defined in Section 3.2

$\beta_{cr\ell}$ = Magnitude of generalized demand under the combined forces that cause elastic *local buckling*

$k_\ell = 1.2(1 - \gamma_\ell) + [k_{s1} + (k_{s2} - k_{s1}) \sin^2 \theta_{12}]\, \gamma_\ell$ (*Eq.* 3.3-3)

$a_\ell = 0.10(1 - \gamma_\ell) + 0.10[\alpha_{s1} + (\alpha_{s2} - \alpha_{s1}) \sin^2 \theta_{12}]\, \gamma_\ell$ (*Eq.* 3.3-4)

$b_\ell = 0.55(1 - \gamma_\ell) + 0.55[\beta_{s1} + (\beta_{s2} - \beta_{s1}) \sin^2 \theta_{12}]\, \gamma_\ell$ (*Eq.* 3.3-5)

$\gamma_\ell = (1 - \cos \varphi_{PM})^4$ for P_r in compression (*Eq.* 3.3-6)

$= 1$ for P_r in tension or no axial load

$$\theta_{12} = \tan^{-1}\left(\frac{M_{2r} / M_{2y}}{M_{1r} / M_{1y}} \right) \qquad (Eq.\ 3.3\text{-}7)$$

$$\beta_{y3} = \beta_y + \frac{8}{9}\left(\beta_p - \beta_y\right) \qquad (Eq.\ 3.3\text{-}8)$$

k_{s1}, k_{s2} = k_s as defined in Section F3.2 for M_{1r} and M_{2r}, respectively
α_{s1}, α_{s2} = α_s as defined in Section F3.2 for M_{1r} and M_{2r}, respectively
β_{s1}, β_{s2} = β_s as defined in Section F3.2 for M_{1r} and M_{2r}, respectively

User Note:
For a C-section bending only about the major principal axis, M_{2r}=0, θ_{12}=0, and the coefficients are simplified to: $k_\ell=1.2(1-\gamma_\ell)+k_{s1}\gamma_\ell$, $a_\ell=0.10$, $b_\ell=0.55$.

The variables M_{1r}, M_{1y}, M_{2r}, M_{2y}, and φ_{PM} are defined in Section 3.1. The variables β_{ne}, β_y, and β_p are defined in Section 3.2. β_p is permitted to be conservatively taken as β_y.

For members with holes, $\beta_{n\ell}$ shall be the lesser of Eq. 3.3-1 and Eq. 3.3-9. Eq. 3.3-1 shall use the lesser of $\beta_{cr\ell}$ at the gross or net section. Eq. 3.3-9 shall use $\beta_{cr\ell}$ at the net section.

$$\beta_{n\ell} = k_{\ell net}\beta_{ynet}\frac{1+a_\ell\lambda_\ell^2}{1+b_\ell\lambda_\ell^2\left(\dfrac{k_{\ell net}\beta_{ynet}}{k_\ell\beta_{ne}}\right)} \le \beta_{y3net} \qquad (Eq.\ 3.3\text{-}9)$$

where $k_{\ell net}$, β_{ynet}, and β_{y3net} are the values of k_ℓ, β_y, and β_{y3} based on the net cross-section.

User Note:
β_{ynet} is β_r times the ratio F_y/f_{max} where f_{max} is the maximum *stress* in the net section resulting from the demands P_r, M_{1r}, and M_{2r}. β_{pnet} is the multiple of β_r at which the net section is fully plastic. In both cases, β_r remains normalized to gross section first yield strengths P_y, M_{1y}, and M_{2y}.

3.4 Distortional Buckling, β_{nd}

The *nominal strength* [*resistance*] for *distortional buckling* and *yielding* where applicable, shall be calculated in accordance with this section for $\lambda_d \le 5$.

$$\beta_{nd} = k_d\beta_y\frac{1+a_d\lambda_d^2}{1+b_d\lambda_d^2} \le \beta_{y3} \qquad (Eq.\ 3.4\text{-}1)$$

where

$$\lambda_d = \sqrt{\frac{\beta_y}{\beta_{crd}}} \qquad (Eq.\ 3.4\text{-}2)$$

β_y = As defined in Section 3.2
β_{crd} = Magnitude of the generalized demand under the combined forces that cause elastic *distortional buckling*

$$k_d = 1.2(1-\gamma_d) + [k_{s1} + (k_{s2}-k_{s1})\sin^2\theta_{12}]\,\gamma_d \qquad (Eq.\ 3.4\text{-}3)$$

$$a_d = 0.05(1-\gamma_d) + 0.07[\alpha_{s1} + (\alpha_{s2}-\alpha_{s1})\sin^2\theta_{12}]\,\gamma_d \qquad (Eq.\ 3.4\text{-}4)$$

$$b_d = 0.67(1-\gamma_d) + 0.60[\beta_{s1} + (\beta_{s2}-\beta_{s1})\sin^2\theta_{12}]\,\gamma_d \qquad (Eq.\ 3.4\text{-}5)$$

$$\gamma_d = (1-\cos\varphi_{PM})^{10} \text{ for } P_r \text{ in compression} \qquad (Eq.\ 3.4\text{-}6)$$

= 1 for P_r in tension or no axial load

k_{s1}, k_{s2} = k_s as defined in Section F3.2 for M_{1r} and M_{2r}, respectively

α_{s1}, α_{s2} = α_s as defined in Section F4 for M_{1r} and M_{2r}, respectively

β_{s1}, β_{s2} = β_s as defined in Section F4 for M_{1r} and M_{2r}, respectively

The variables M_{1r}, M_{2r}, and φ_{PM} are defined in Section 3.1. The variable β_y is defined in Section 3.2. The variables θ_{12} and β_{y3} are defined in Section 3.3.

For members with holes, β_{crd} shall be determined including the influence of holes and β_{nd} shall be calculated as follows:

$$\beta_{nd} = k_{dnet}\beta_{ynet}\frac{1+a_d\lambda_d^2}{1+b_d\lambda_d^2\left(\dfrac{k_{dnet}\beta_{ynet}}{k_d\beta_y}\right)} \le \beta_{y3net} \qquad (Eq.\ 3.4\text{-}7)$$

where k_{dnet}, β_{ynet}, and β_{y3net} are the values of k_d, β_y, and β_{y3} based on the net cross-section.

APPENDIX 4, ALTERNATIVE METHOD FOR PERFORMANCE OF COLD-FORMED STEEL SYSTEMS UNDER ELEVATED TEMPERATURES DUE TO FIRE CONDITIONS

This Appendix provides criteria for the design and evaluation of *cold-formed steel structural components, members*, assemblies, and systems under *elevated temperature* due to 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 *cold-formed steel structural components*, *members*, assemblies and systems at *elevated temperatures*. Cold-formed steel is generally used in combination with fire protection materials, thus this Appendix provides criteria for the properties and behavior of unprotected and fire-protected cold-formed steel members and materials. The Appendix applies to components and members with plate *thickness* up to 0.14 in. (3.5 mm). The Appendix is used when the temperature level in materials due to fire conditions exceeds 150°F (65°C).

The Section is organized as follows:

4.1 General Provisions
4.2 Design by Analysis
4.3 Design by Qualification Testing
4.4 Design by Combination of Analysis and Testing

4.1 General Provisions

The methods contained in this section provide regulatory evidence of compliance in accordance with the design applications outlined in this section.

4.1.1 Performance Objective

Cold-formed steel structural components, *members*, assemblies, and 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.

Where fire protection materials are required, the performance of the fire-protected cold-formed steel member and system shall be assessed.

Deformation criteria shall be applied where the means of providing structural *fire resistance*, or the design criteria for *fire barriers*, requires evaluation of the deformation of the load-carrying structure.

Within the compartment of fire origin, forces and deformations from the *design-basis fire* shall not cause a breach of horizontal or vertical *compartmentation*.

4.1.2 General Structural Integrity

The cold-formed steel structural frame shall be capable of providing the strength and deformation capacity to withstand, as a system, the structural actions developed during the fire within the prescribed limits of deformation. The structural system shall be designed to sustain local damage with the structural system as a whole remaining stable. Load paths shall be provided to transfer forces from the exposed region to the final point of resistance.

4.1.3 Design Requirements

The analysis methods in Section 4.2 are permitted to be used to determine the anticipated performance of *cold-formed steel structural components* and systems when subjected to *design-basis fire* scenarios.

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 Section 4.2 shall be performed using the *Load and Resistance Factor Design* method in accordance with the provisions of Section B3.2.2 (*LRFD*), or in Canada the limit states design method in accordance with the provisions of Section B3.2.3 (*LSD*).

The qualification testing methods in Section 4.3 are permitted to be used to determine the *fire resistance* of *cold-formed steel structural components* subject to the standardized fire testing protocols required by the *applicable building code*.

Combination of analysis as detailed in Section 4.2 and testing as detailed in Section 4.3 are permitted as detailed in Section 4.4.

4.1.4 Load Combinations and Required Strength

For *LRFD*, in the absence of *applicable building code* provisions for design under fire exposures, the *required strength* of the structure and its elements shall be determined from the gravity load combination specified in ASCE/SEI 7 Section 2.5, *Load Combinations for Extraordinary Events*.

Unless otherwise stipulated by the *applicable building code*, the *nominal* dead *load*, *nominal* occupancy live *load*, and *nominal* snow *load* used in the combination shall be the *nominal loads* specified in ASCE/SEI 7. For roof structures, the *nominal* roof live *load* shall be used instead of the *nominal* occupancy live *load*.

For *LSD*, in the absence of *applicable building code* provisions for design under fire exposures, the effect due to *factored loads* of the structure and its elements shall be determined from the gravity load combination as follows:

$$D + T_S + (\alpha L \text{ or } 0.25\,S) \qquad (Eq.\ 4.1.4\text{-}1)$$

where

D = *Specified* dead *load*

T_S = Effects due to expansion, contraction, or deflection caused by temperature changes due to the *design basis-fire*. T_S can be taken equal to zero for statically determinate structures or for structures that have sufficient ductility to allow the redistribution of temperature forces before collapse

α = 1.0 for storage areas, equipment areas, and service rooms
= 0.5 for other occupancies

L = *Specified* occupancy live *load*

S = *Specified* variable *load* due to snow

Unless otherwise stipulated by the *applicable building code*, D, L and S shall be the *specified loads* in the NBCC.

For both *LRFD* and *LSD, second-order effects* that consider both *P-Δ* and *P-δ effects* shall be considered in the load combination, consistent with Chapter C.

User Note:
A notional load as defined in C.1.1.1.2 is one way to consider initial imperfections when an analysis is performed to consider the *second-order effects*.

4.1.5 Connections

The performance of fire-protection for a *connection* shall be at least equivalent to the performance of the fire-protection for the connected members.

4.2 Design by Analysis

It is permitted to design *cold-formed steel structural components*, *members*, and systems for fire conditions in accordance with the requirements of this section.

4.2.1 Design-Basis Fire

A *design-basis fire* shall be identified to describe the heating conditions for the structure. These heating conditions shall relate to the fuel commodities and compartment characteristics present in the assumed fire area. The fuel load density based on the occupancy of the space shall be considered when determining the total fuel load. Heating conditions shall be specified either in terms of a heat flux or temperature of the upper gas layer created by the fire. The variation of the heating conditions with time shall be determined for the duration of the fire including the decay phase.

The fire duration in a particular area shall be determined from the total combustible mass, or fuel *load* in the space. In the case of either a localized fire or a post-*flashover* compartment fire, the fire duration shall be determined as the total combustible mass divided by the mass loss rate.

The analysis methods in Section 4.2 are permitted to be used in accordance with the provisions for alternative materials, designs and methods as permitted by the *applicable building code*. When the analysis methods in Section 4.2 are used to demonstrate equivalency to hourly ratings based on qualification testing in Section 4.3, the *design-basis fire* is permitted to be determined in accordance with ASTM E119 in the United States and Mexico and CAN/ULC-S101 in Canada.

(a) Localized Fires

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.

(b) 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 *design-basis fire* shall include fuel *load*, ventilation characteristics of the space (natural and mechanical), compartment dimensions, and thermal characteristics of the compartment boundary.

4.2.2 Temperatures in Cold-formed Steel and Fire-Protected Cold-Formed Steel Structural Systems under Fire Conditions

Temperatures within structural components, members and frames due to the heating conditions posed by the *design-basis fire* shall be determined by a heat transfer analysis.

The heat transfer analysis shall also consider the thermal response of the fire protection material.

4.2.3 Mechanical and Thermal Properties at Elevated Temperatures

The deterioration in strength and stiffness of structural components, members and systems shall be taken into account in the structural analysis.

The material properties used in the equations shall be as defined in Section A3.1.1 or Section A3.1.2 of this *Specification*. For steels with unique composition or manufacturing, it shall be permitted to determine material properties at *elevated temperatures* from testing.

4.2.3.1 Steel

4.2.3.1.1 Variation of Strength and Modulus of Elasticity With Temperature

The mechanical properties for modulus of elasticity, E, shear modulus, G, *yield stress*, F_y, and *tensile strength*, F_u used in the *Specification* shall be modified due to the influence of temperature (T), by multiplying by retention factors k_E, k_G, k_y, and k_u, respectively. For steels conforming to Section A3.1.1 or Section A3.1.2, and for temperatures between 68 °F (20 °C) and 1472 °F (800 °C), these retention factors shall be taken as follows:

$$k_E,\ k_G,\ k_y,\ k_u = (1-c)\frac{1-x^b}{1+ax^b}+c \qquad (Eq.\ 4.2.3.1.1\text{-}1)$$

where

$$x = \frac{T - T_1}{T_2 - T_1} \qquad (Eq.\ 4.2.3.1.1\text{-}2)$$

T_1 = 68 °F (20 °C)

T_2 = 1832 °F (1000 °C)

a, b, c = Coefficients as defined in Table 4.2.3.1.1-1

Table 4.2.3.1.1-1 Coefficients for Retention Factors of Steel Mechanical Properties

Retention factor	a	b	c
k_E and k_G	8	3	0.04
k_y	20	4	0.03
k_u	185	7	0.04

4.2.3.1.2 Variation of Specific Heat With Temperature

For steels conforming to Section A3.1.1 or Section A3.1.2, the influence of temperature (T) on the specific heat (c_p) shall be taken as follows:

For temperature (T) in °F and specific heat (c_p) in BTU/lb°F:

68 °F ≤ T < 1112 °F:

$$c_p = 0.0983 + 1.11 \times 10^{-4}T - 1.34 \times 10^{-7}T^2 + 9.11 \times 10^{-11}T^3 \quad (Eq.\ 4.2.3.1.2\text{-}1)$$

1112 °F ≤ T < 1355 °F:

$$c_p = 0.1594 + 5.602/(1360.4 - T) \quad (Eq.\ 4.2.3.1.2\text{-}2)$$

1355 °F ≤ T < 1652 °F:

$$c_p = 0.1304 + 7.678/(T - 1347.8) \quad (Eq.\ 4.2.3.1.2\text{-}3)$$

1652 °F ≤ T < 2192 °F:

$$c_p = 0.1556$$

For temperature (T) in °C and specific heat (c_p) in J/kgK:

20 °C ≤ T < 600 °C:

$$c_p = 425 + 7.73 \times 10^{-1}T - 1.69 \times 10^{-3}T^2 + 2.22 \times 10^{-6}T^3 \quad (Eq.\ 4.2.3.1.2\text{-}4)$$

600 °C ≤ T < 735 °C:

$$c_p = 666 + 13002/(738 - T) \quad (Eq.\ 4.2.3.1.2\text{-}5)$$

735 °C ≤ T < 900 °C:

$$c_p = 545 + 17820/(T - 731) \quad (Eq.\ 4.2.3.1.2\text{-}6)$$

900 °C ≤ T < 1200 °C:

$$c_p = 650$$

4.2.3.1.3 Variation of Thermal Conductivity With Temperature

For steels conforming to Section A3.1.1 or Section A3.1.2, the influence of temperature (T) on thermal conductivity (k) shall be taken as follows:

For temperature (T) in °F and thermal conductivity (k) in BTU/h-ft°F:

68 °F ≤ T < 1472 °F: $k = 31.5 - 0.0107T$ (*Eq*. 4.2.3.1.3-1)

1472 °F ≤ T < 2192 °F: $k = 15.8$

For temperature (T) in °C and thermal conductivity (k) in W/mK:

20 °C ≤ T < 800 °C: $k = 54 - 3.33 \times 10^{-2}T$ (*Eq*. 4.2.3.1.3-2)

800 °C ≤ T < 1200 °C: $k = 27.3$

4.2.3.1.4 Thermal Elongation

For calculations at temperatures above 150 °F (66 °C), the coefficient of thermal expansion for steels conforming to Section A3.1.1 or Section A3.1.2 shall be

$\alpha = 7.8 \times 10^{-6}$/°F ($1.4 \times 10^{-5}$/°C)

4.2.3.1.5 Variation of Stress-Strain Relationship With Temperature

The influence of temperature (T) on the *stress*-strain relationship for cold-formed steels shall be taken as follows:

$$\varepsilon_T = \frac{f_T}{E_T} + \beta\left(\frac{F_{y,T}}{E_T}\right)\left(\frac{f_T}{F_{y,T}}\right)^{\eta_T} \quad (Eq.\ 4.2.3.1.5\text{-}1)$$

where

ε_T = Strain corresponding to a given *stress* f_T at temperature (T)

E_T = Modulus of elasticity of steel at temperature (T)

$F_{y,T}$ = *Yield stress* at temperature (T)

η_T and β = Parameters determined as follows:

For steels conforming to Section A3.1.1:

68 °F (20 °C) ≤ T < 572 °F (300 °C):

An elastic-perfectly-plastic model based on E_T and $F_{y,T}$ shall be used.

572 °F (300 °C) ≤ T < 1472 °F (800 °C):

$\beta = 1.5$

$\eta_T = 4.26 \times 10^{-5}T^2 - 5.021 \times 10^{-2}T + 20.775$ for T in °F (*Eq*. 4.2.3.1.5-2a)

$\eta_T = 1.38 \times 10^{-4}T^2 - 8.5468 \times 10^{-2}T + 19.212$ for T in °C (*Eq*. 4.2.3.1.5-2b)

For steels conforming to Section A3.1.2:

68 °F (20 °C) ≤ T < 1472 °F (800 °C):

$\beta = 0.86$

$\eta_T = -5.23 \times 10^{-8}T^3 + 1.593 \times 10^{-4}T^2 - 0.1553T + 67.462$ for T in °F (*Eq*. 4.2.3.1.5-3a)

$\eta_T = -3.05 \times 10^{-7}T^3 + 5.0 \times 10^{-4}T^2 - 0.2615T + 62.653$ for T in °C (*Eq*. 4.2.3.1.5-3b)

4.2.3.2 Fire Protective Boards/Materials

The thermal performance of fire protection materials shall be considered when predicting the heat transfer for the *design-basis fire*. The mechanical performance of fire protection materials at *elevated temperature* shall be considered if the engineer is relying on the fire protection material to add structural resistance to the *cold-formed steel structural members*.

User Note:
Cold-formed steel structural members invariably need fire-protection, with gypsum wallboard being the most common material. If the wallboard is relied upon for bracing the cold-formed steel members (e.g., in a sheathing braced wall) then the degradation of the bracing supplied by the board at *elevated temperature* must be considered along with the heat transfer.

4.2.3.3 Connectors

The thermal and mechanical performance of connectors at *elevated temperatures* shall be considered.

4.2.3.4 Concrete

For concrete, material properties at *elevated temperatures* shall be determined in accordance with AISC 360 in the United States and Mexico and CSA S16 Annex K in Canada.

4.2.4 Structural Integrity and Strength at Elevated Temperatures

Conformance of the structural system to the requirements of structural integrity and strength 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*.

Structural members at *elevated temperature* shall be designed in accordance with the provisions of Section B3.3.

Connections at *elevated temperature* shall be designed in accordance with the provisions of Section B3.4.

Where the means of providing *fire resistance* requires the evaluation of deformation criteria, the deformation of the structural system, or members thereof, under the *design-basis fire* shall not exceed the prescribed limits.

It is permitted to include membrane action of composite floor slabs for *fire resistance* if the design provides for the effects of increased connection tensile forces and redistributed gravity load demands on the adjacent framing supports.

4.2.4.1 Design by Advanced Methods of Analysis

Design by advanced methods of analysis is permitted for the design of cold-formed steel members and components for fire conditions. The *design-basis fire* exposure shall be that determined in accordance with 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 4.2.3.

The following effects of on the stability of the structure and its elements shall be considered:

(1) The deterioration in strength and *stiffness* with increasing temperature,

(2) The effects of thermal expansion and thermal bowing,

(3) Inelastic behavior and load redistribution, and large deformations,

(4) Time-dependent effects such as creep, and

(5) Uncertainties resulting from variability in material properties at *elevated temperature*.

Boundary conditions and *connection* fixity shall represent the proposed structural design. Material properties shall be defined as per Section 4.2.3.

Any *rational engineering analysis* method of design for stability that considers all of the listed effects is permitted; this includes the methods identified in Sections C1.1, C1.2, or C1.3.

The resulting analysis shall address all relevant *limit states*, such as excessive deflections, *connection* ruptures, global *buckling*, and *local* and *distortional buckling*.

4.2.4.2 Design by Simple Methods of Analysis

The methods of analysis in this section are permitted to be used for the evaluation of the performance of individual *cold-formed steel structural members* at *elevated temperatures* during exposure to fire.

The support and restraint conditions (forces, moments and boundary conditions) applicable at normal temperatures are permitted to be assumed to remain unchanged throughout the fire exposure.

It is permitted to model the thermal response of steel and fire-protected steel members using a one-dimensional heat transfer equation with heat input as determined by the *design-basis fire* defined in Section 4.2.1.

The design strength shall be determined as in Section B3.2.2 for *LRFD* and B3.2.3 for *LSD*. The *nominal strength* [*resistance*], R_n, shall be calculated as follows.

(a) *Members subject to uniform temperature distributions*

For members subject to uniform or near-uniform temperature distributions in applications such as beams or columns, ambient temperature design capacity rules shall be used with appropriately reduced mechanical properties in accordance with Section 4.2.3, at the temperature developed by the *design-basis fire*.

(b) *Members subject to non-uniform temperature distributions*

Reduced mechanical properties in accordance with Section 4.2.3, at the temperature developed by the *design-basis fire*, shall be used. The *required strength* [effect due to *factored loads*] shall consider thermal bowing and any other second-order response consistent with the non-uniform temperature distribution. The *available strength* [*factored resistance*] shall consider *local*, *distortional*, and global *buckling*, and *yielding* in the cross-section consistent with the non-uniform temperature distribution.

4.3 Design by Qualification Testing

Structural members and components in cold-formed steel buildings shall be qualified for the rating period in conformance with ASTM E119 in the United States and Mexico, and with CAN/ULC-S101 in Canada. Demonstration of compliance with these requirements using the procedures specified for steel construction in Section 5 of ASCE/SEI/SFPE 29 is permitted.

4.4 Design by Combination of Analysis and Testing

Cold-formed steel structural members and *components* are permitted to be qualified based on combinations of both analysis and testing. Temperature distributions of Section 4.2.2 is permitted to be established from testing consistent with the *design-basis fire* of Section 4.2.1 and then used to perform analysis consistent with Sections 4.2.3 and 4.2.4. Structural integrity of Section 4.2.4 is permitted to be established by testing with *elevated temperatures* predicted by analysis consistent with Sections 4.2.1 to 4.2.3.

User Note:
These provisions provide a means to leverage ASTM E119 for similar assemblies by performing analysis instead of additional testing, or to test unique members or assemblages under temperature to establish their strength. Other combinations of analysis and testing are also possible.

APPENDIX 5, EVALUATION OF EXISTING STRUCTURES

This appendix applies to the evaluation of the strength and *stiffness* under static *loads* of existing structures by structural analysis, by *load* tests, or by a combination of structural analysis and *load* tests when specified. This appendix does not address seismic evaluation of existing buildings, nor does this appendix address *load* testing for the effects of seismic *loads* or dynamic *loads*. Section 5.4 is only applicable to static vertical gravity *loads* applied to existing roofs or floors.

The Appendix is organized as follows:

5.1. General Provisions
5.2. Material Properties
5.3. Evaluation by Structural Analysis
5.4. Evaluation by Load Tests
5.5. Evaluation Report

5.1 General Provisions

These provisions shall be applicable when the evaluation of existing *cold-formed steel structural members* and *connections* is specified for (a) verification of a specific set of design loadings or (b) determination of the *available strength* [*factored resistance*] of the cold-formed steel *load*-resisting member or system. The evaluation shall be performed by structural analysis (Section 5.3), by *load* tests (Section 5.4), or by a combination of structural analysis and *load* tests, when specified. Where *load* tests are used, the structure shall first be analyzed, and a testing plan developed, including a written test procedure. The plan shall consider catastrophic collapse and excessive levels of permanent deformation, and shall include procedures to preclude either occurrence during testing.

5.2 Material Properties

For evaluations in accordance with this appendix, steel grades other than those listed in Section A3.1 are permitted.

5.2.1 Determination of Required Tests

The specific tests and their corresponding locations that are required from Sections 5.2.2 through 5.2.5 shall be determined. The use of applicable project records is permitted to reduce or eliminate the need for testing.

5.2.2 Material Properties

Material properties of members shall be established via tensile coupon testing for use in evaluation by structural analysis (Section 5.3) or *load* tests (Section 5.4). Such properties shall include the *yield stress*, *tensile strength* and percent elongation. Certified material test reports or certified reports of tests made by the manufacturer of the cold-formed steel product or an approved testing laboratory are permitted for this purpose. Otherwise, material tests shall be conducted in accordance with ASTM A370 from samples taken from components of the structure.

5.2.3 Chemical Composition

Where welding is anticipated for repair or modification of existing structures, the chemical composition of the steel shall be determined for use in preparing a welding procedure specification. Results from certified material test reports or certified reports of tests made by the manufacturer of cold-formed steel components or a testing laboratory in accordance with ASTM procedures are permitted for this purpose. Otherwise, analyses shall be conducted in accordance with ASTM A751 from the samples used to determine tensile properties or from samples taken from the same locations.

5.2.4 Weld Metal

Where structural performance is dependent on existing welded *connections*, representative samples of weld metal shall be obtained. Chemical analysis and mechanical tests shall be made to characterize the weld metal. A determination shall be made of the magnitude and consequences of imperfections. If the applicable requirements of AWS D1.1/D1.1M, AWS D1.3/D1.3M, CSA W59, and CSA W55.3 are not met, the need for remedial action shall be determined.

5.2.5 Fasteners

Representative samples of fasteners shall be visually inspected to determine markings and classifications. Where it is not possible to classify fasteners by visual inspection, representative samples shall be taken and tested to determine *tensile strength* in accordance with ASTM F606/F606M or ANSI/SDI AISI S904, as applicable.

5.3 Evaluation by Structural Analysis

5.3.1 Dimensional Data

All dimensions used in the evaluation, such as spans, column or stud heights, member spacings, bracing locations, cross-section dimensions, *thicknesses*, and *connection* details, shall be determined from a field survey. Alternatively, it is permitted to determine such dimensions from applicable construction or manufacturer documents with field verification of critical dimensions.

5.3.2 Strength Evaluation

Forces (*load effects*) in members and *connections* shall be determined by structural analysis complying with Chapter C and 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* [*factored resistance*] of members and *connections* shall be determined from applicable provisions of Chapters B through M, excluding Chapter K, of this *Specification*.

5.3.3 Serviceability Evaluation

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

5.4 Evaluation by Load Tests

The requirements of this section shall only be applicable to static vertical gravity *loads* applied to existing roofs or floors.

5.4.1 Determination of Load Rating by Testing

To determine the *load* rating of an existing floor or roof structure by testing, a test *load* shall be applied incrementally in accordance with the testing plan. At a minimum, the testing plan shall include a visual assessment, structural analysis, loading protocol, and shoring and bracing plan. The structure shall be visually inspected for signs of distress or imminent failure at each load level. Measures shall be taken to prevent collapse if these or any other unusual conditions are encountered.

The tested strength of the structure shall be taken as the maximum applied test load plus the in-situ dead load. The live load rating of a floor structure shall be determined by setting the tested strength equal to the controlling load combination (e.g., 1.2D+1.6L (*LRFD*), 1.25D+1.5L (*LSD*)), where D is the nominal [specified] dead *load* and L is the nominal [specified] live *load* rating for the structure. For roof structures, L_r, R or S shall be substituted for L, where

L_r = Nominal [specified] roof live *load*

R = Nominal [specified] *load* due to rain, exclusive of the ponding contribution

S = Nominal [specified] snow *load*

Other *load* combinations shall be used where justified or required by the *applicable building codes* or the *authority having jurisdiction*. Wind (W) and Seismic (E) are not within the scope of this Appendix.

Periodic unloading is permitted once the *service load* level is attained and after the onset of inelastic structural behavior is identified to document the amount of permanent set and the magnitude of the inelastic deformations. Deformations of the structure, such as member deflections, shall be monitored at critical locations during the test, referenced to the initial position before loading. It shall be demonstrated, while maintaining maximum test *load* for one hour that the (vertical) deformation of the structure does not increase by more than 10% above that at the beginning of the holding period. It is permitted to repeat the loading sequence, if necessary, to demonstrate compliance.

Deformations of the structure shall also be recorded 24 hours after the test loading is removed to determine the amount of permanent set. Where it is not feasible to load test the entire structure, a segment or zone representative of the most critical conditions shall be selected.

5.4.2 Serviceability Evaluation

When *load* tests are prescribed, the structure shall be loaded incrementally to the *service load* level. The service test *load* shall be held for a period of one hour, and deformations shall be recorded at the beginning and at the end of the one-hour holding period.

5.5 Evaluation Report

After the evaluation of an existing structure has been completed, a report documenting the evaluation shall be prepared. 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 the manufacturer of the cold-formed steel product, construction documents, material test reports, and auxiliary material testing shall also be reported. The report shall indicate whether the structure, including all members and *connections*, can withstand the expected *load* effects.

ANSI/SDI AISI S100-2024

Appendix A

Provisions Applicable to the United States and Mexico

PREFACE TO APPENDIX A

Specification Chapters A through M contain design provisions that are applicable to Canada, Mexico, and the United States, and accommodate those provisions that may be partially applicable to certain countries. Appendix A provides *Specification* provisions that apply only to the United States and Mexico.

Also included in Appendix A are technical items where full agreement between countries was not reached. Such items include certain provisions pertaining to the design of:

(a) Beams and compression members (C- and Z-sections) for standing seam roofs, and

(b) Bolted and welded *connections*.

Efforts are being made to minimize these differences in future editions of the *Specification*.

APPENDIX A, PROVISIONS APPLICABLE TO THE UNITED STATES AND MEXICO

Specification Chapters A through M contain design provisions that are applicable to Canada, Mexico, and the United States, and accommodate those provisions that may be partially applicable to certain countries. This appendix addresses design provisions or supplements to Chapters A through M that specifically apply to the United States and Mexico. This appendix is considered mandatory for applications in the United States and Mexico.

A section number ending with the letter "a" indicates that the provisions herein supplement the corresponding section in Chapters A through M of the *Specification*. A section number not ending with the letter "a" indicates that the section gives the entire design provision.

I6.2.2 Flexural Members Having One Flange Fastened to a Standing Seam Roof System

The *available flexural strength* of a C- or Z-section, loaded in a plane parallel to the *web* with the top *flange* supporting a standing seam roof system, shall be determined using discrete point bracing and the provisions of Section F3, or shall be calculated in accordance with this section, where consideration of *distortional buckling* in accordance with Section F4 is permitted to be excluded.

The *safety factor* and the *resistance factor* provided in this section shall be applied to the *nominal strength*, M_n, calculated by Eq. I6.2.2-1 to determine the *available strengths* in accordance with the applicable design method in Section B3.2.1 or B3.2.2.

$$M_n = RM_{n\ell o} \quad (Eq.\ I6.2.2\text{-}1)$$

Ω_b = 1.67 (*ASD*)

ϕ_b = 0.90 (*LRFD*)

where

R = Reduction factor determined in accordance with ANSI/SDI AISI S908

$M_{n\ell o}$ = *Nominal flexural strength* with consideration of *local buckling* only, as determined from Section F3 with $F_n = F_y$ or $M_{ne} = M_y$

I6.2.4 Z-Section Compression Members Having One Flange Fastened to a Standing Seam Roof

These provisions shall apply to Z-sections concentrically loaded along their longitudinal axis, with only one *flange* attached to standing seam roof panels. Alternatively, design values for a particular system are permitted to be based on discrete point bracing locations, or on tests in accordance with Section K2.

The *nominal axial strength*, P_n, of simple span or continuous Z-sections shall be calculated in accordance with (a) and (b). Consideration of *distortional buckling* in accordance with Section E4 is permitted to be excluded.

Unless otherwise specified, the *safety factor* and the *resistance factor* provided in this section shall be used to determine the *available strengths* in accordance with the applicable design method in Section B3.2.1 or B3.2.2.

(a) For weak axis *available strength*

$$P_n = k_{af}RF_yA \quad (Eq.\ I6.2.4\text{-}1)$$

Ω = 1.80 (*ASD*)

ϕ = 0.85 (*LRFD*)

where

For $d/t \le 90$

$k_{af} = 0.36$

For $90 < d/t \le 130$

$$k_{af} = 0.72 - \frac{d}{250t} \qquad (Eq.\ I6.2.4\text{-}2)$$

For $d/t > 130$

$k_{af} = 0.20$

R = Reduction factor determined from uplift tests performed using ANSI/SDI AISI S908

A = *Full unreduced cross-sectional area* of Z-section

d = Z-section depth

t = Z-section *thickness*

F_y = Design *yield stress* determined in accordance with Section A3.3.1

Eq. I6.2.4-1 shall be limited to roof systems meeting the following conditions:

(1) *Purlin thickness*, 0.054 in. (1.37 mm) $\le t \le$ 0.125 in. (3.22 mm),

(2) 6 in. (152 mm) $\le d \le$ 12 in. (305 mm),

(3) *Flanges* are edge-*stiffened compression elements,*

(4) $70 \le d/t \le 170$,

(5) $2.8 \le d/b < 5$, where b = Z-section *flange* width,

(6) $16 \le \dfrac{\text{flange flat width}}{t} < 50$,

(7) Both *flanges* are prevented from moving laterally at the supports, and

(8) *Yield stress*, $F_y \le 70$ ksi (483 MPa or 4920 kg/cm^2).

(b) The *available strength* about the strong axis shall be determined in accordance with Sections E2 and E3.

I6.3.1a Strength of Standing Seam Roof Panel Systems

In addition to the provisions provided in Section I6.3.1, for *load combinations* that include wind uplift, the *nominal wind load*, to be applied to the standing seam roof panel, clips and fasteners, is permitted to be multiplied by 0.67 provided the tested system and wind load evaluation satisfy the following conditions:

(a) The roof system is tested in accordance with ANSI/SDI AISI S906.

(b) The wind load is calculated using ASCE/SEI 7 for components and cladding.

User Note:

These provisions can be used with the 2005, 2010, 2016, or 2022 edition of ASCE 7.

References:

ASCE/SEI 7-05, *Minimum Design Loads for Buildings and Other Structures*, ASCE, 2005.

ASCE/SEI 7-10, *Minimum Design Loads for Buildings and Other Structures*, ASCE, 2010.

ASCE/SEI 7-16, *Minimum Design Loads and Associated Criteria for Buildings and Other Structures,* ASCE, 2016.

(c) The area of the roof being evaluated is in the roof edge or corner zones, as defined in

ASCE/SEI 7; i.e., the 0.67 factor does not apply to the field of the roof beyond the edge and corner zones. The *nominal wind load* applied to edge and corner zones, after the 0.67 multiplier is applied, shall not be less than the *nominal wind load* applied to the field of the roof.

(d) The base metal *thickness* of the standing seam roof panel is greater than or equal to 0.023 in. (0.59 mm) and less than or equal to 0.030 in. (0.77 mm).

(e) For trapezoidal profile standing seam roof panels, the distance between sidelaps is no greater than 24 in. (610 mm).

(f) For vertical rib profile standing seam roof panels, the distance between sidelaps is no greater than 18 in. (460 mm).

(g) The observed failure mode of the tested system is one of the following:

  (1) The standing seam roof clip mechanically fails by separating from the panel sidelap.

  (2) The standing seam roof clip mechanically fails by the sliding tab separating from the stationary base.

J2a Welded Connections

Welders and welding procedures shall be qualified as specified in AWS D1.3. These provisions shall apply to the welding positions as listed in Table J2a.

TABLE J2a
Welding Positions Covered

Connection	Welding Position					
	Square Groove Butt Weld	Arc Spot Weld	Arc Seam Weld	Fillet Weld, Lap or T	Flare Bevel Groove	Flare V-Groove Weld
Sheet to Sheet	F H V OH	— — — —	F H — —	F H V OH	F H V OH	F H V OH
Sheet to Supporting Member	— — — —	F — — —	F — — —	F H V OH	F H V OH	— — — —

(F = Flat, H = Horizontal, V = Vertical, OH = Overhead)

J3.4 Shear and Tension in Bolts

The *nominal bolt strength*, P_n, resulting from shear, tension, or a combination of shear and tension shall be calculated in accordance with this section. The *safety factor* and the *resistance factor* given in this section shall be used to determine the *available strengths* in accordance with the applicable design method in Section B3.2.1 or B3.2.2.

$$P_n = A_b F_n \quad (Eq.\ J3.4\text{-}1)$$

$\Omega = 2.00$ (*ASD*)

ϕ = 0.75 (*LRFD*)

where

A_b = Gross *cross-sectional area* of bolt

F_n = *Nominal strength,* ksi (MPa), determined in accordance with (a) or (b) as follows:

(a) When bolts are subjected to shear only or tension only, F_n shall be given by F_{nv} or F_{nt} in Tables J3.4-1(a) and J3.4-1(b).

The pull-over strength of the connected sheet at the bolt head, nut, or washer shall be considered where bolt tension is involved. See Section J6.2.

(b) When bolts are subjected to a combination of shear and tension, F_n is given by F'_{nt} in Eq. J3.4-2 or J3.4-3 as follows:

For *ASD*

$$F'_{nt} = 1.3\,F_{nt} - \frac{\Omega F_{nt}}{F_{nv}} f_v \le F_{nt} \qquad (Eq.\ J3.4\text{-}2)$$

For *LRFD*

$$F'_{nt} = 1.3\,F_{nt} - \frac{F_{nt}}{\phi F_{nv}} f_v \le F_{nt} \qquad (Eq.\ J3.4\text{-}3)$$

where

F'_{nt} = *Nominal tensile strength* modified to include the effects of *required shear strength*, ksi (MPa)

F_{nt} = *Nominal tensile strength* from Tables J3.4-1(a) and J3.4-1(b)

F_{nv} = *Nominal shear strength* from Tables J3.4-1(a) and J3.4-1(b)

f_v = *Required shear strength*, ksi (MPa)

In addition, the *required shear strength*, f_v, shall not exceed the *allowable shear strength*, F_{nv} / Ω (*ASD*), or the *design shear strength*, ϕF_{nv} (*LRFD*), of the fastener.

In Tables J3.4-1(a) and J3.4-1(b), the *nominal shear strength* shall apply to bolts in holes as limited by Table J3-1 (J3-1M). Washers or back-up plates shall be installed over long-slotted holes, and the capacity of *connections* using long-slotted holes shall be determined by *load* tests in accordance with Section K2.

TABLE J3.4-1(a)
Nominal Tensile and Shear Strengths for ASTM Bolts

Bolt Type [a, b]	Nominal Tensile Strength F_{nt}, ksi (MPa)		Nominal Shear Strength F_{nv}, ksi (MPa) [c]	
Bolt diameter, d, in. (mm)	1/4 in. ≤ d <1/2 in. (6.4 mm ≤ d < 12 mm)	d ≥ 1/2 in. (12 mm)	1/4 in. ≤ d <1/2 in. (6.4 mm ≤ d < 12 mm)	d ≥ 1/2 in. (12 mm)
ASTM A307 Grade A Bolts	40 (280)	45 (310)	24 (169) [d]	27 (188) [d]
ASTM F3125 Grade A325/A325M Bolts: • Threads Included • Threads Excluded	NA	90 (620)	NA	 54 (372) 68 (457)
ASTM A354 Grade BD Bolts: • Threads Included • Threads Excluded	101 (700)	113 (780)	 61 (411) 84 (579)	 68 (457) 84 (579)
ASTM A449 Bolts: • Threads Included • Threads Excluded	81 (560)	90 (620)	 48 (334) 68 (457)	 54 (372) 68 (457)
ASTM F3125 Grade A490/A490M Bolts: • Threads Included • Threads Excluded	NA	113 (780)	NA	 68 (457) 84 (579)
ASTM F3148 Bolts: • Threads Included • Threads Excluded	NA	108 (745)	NA	 65 (448) 81 (558)
Threaded Parts: • Threads Included • Threads Excluded	0.675 F_u [e]	0.75 F_u	 0.400 F_u 0.563 F_u	 0.450 F_u 0.563 F_u

Notes:

a. "Threads Included" refers to when threads are not excluded from shear planes.

b. "Threads Excluded" refers to when threads are excluded from shear planes.

c. For end-loaded *connections* with a fastener pattern length greater than 38 in. (965 mm), F_{nv} should be reduced to 83.3 percent of the tabulated values. Fastener pattern length is the maximum distance parallel to the line of force between the centerline of the bolts connecting two parts with one faying surface.

d. Threads permitted in shear planes.

e. *Tensile strength* of bolt or threaded part.

TABLE J3.4-1(b)
Nominal Tensile and Shear Strengths for SAE Bolts

Bolt Type[a, b]	Nominal Tensile Strength F_{nt}, ksi (MPa)			Nominal Shear Strength F_{nv}, ksi (MPa)[c]		
Bolt diameter, d, in. (mm)	1/4 ≤ d <1/2 (6.4 ≤ d < 12.7)	1/2 ≤ d ≤ 3/4 (12.7 ≤ d ≤ 19.1)	d>3/4 (d>19.1)	1/4 ≤ d <1/2 (6.4 ≤ d < 12.7)	1/2 ≤ d ≤3/4 (12.7 ≤ d ≤ 19.1)	d>3/4 (d>19.1)
SAE J429 Grade 2 Bolts: • Threads Included • Threads Excluded	50 (345)	56 (386)	45 (310)	30 (207) 42 (290)	33 (228) 42 (290)	27 (186) 34 (234)
Bolt diameter, d, in. (mm)	1/4 ≤ d <1/2 (6.4 ≤ d < 12.7)	1/2 ≤ d ≤ 1 (12.7 ≤ d ≤ 25.4)	d>1 (d>25.4)	1/4 ≤ d <1/2 (6.4 ≤ d < 12.7)	1/2 ≤ d ≤ 1 (12.7 ≤ d ≤ 25.4)	d> 1 (d>25.4)
SAE J429 Grade 5 Bolts: • Threads Included • Threads Excluded	81 (558)	90 (621)	79 (545)	48 (331) 68 (469)	54 (372) 68(469)	47 (324) 59 (407)
Bolt diameter, d, in. (mm)	1/4 ≤ d <1/2 (6.4 ≤ d < 12.7)	d>1/2 (d>12.7)		1/4 ≤ d <1/2 (6.4 ≤ d < 12.7)	d>1/2 (d>12.7)	
SAE J429 Grade 8 Bolts: • Threads Included • Threads Excluded	101 (696)	113 (779)		60 (414) 84 (579)	68 (469) 84 (579)	
Threaded Parts: • Included • Excluded	$0.675F_u$[d]	$0.75\ F_u$		$0.400\ F_u$ $0.563\ F_u$	$0.450\ F_u$ $0.563\ F_u$	

Notes:

a. "Threads Included" refers to when threads are not excluded from shear planes.

b. "Threads Excluded" refers to when threads are excluded from shear planes.

c. For end-loaded *connections* with a fastener pattern length greater than 38 in. (965 mm), F_{nv} should be reduced to 83.3 percent of the tabulated values. Fastener pattern length is the maximum distance parallel to the line of force between the centerline of the bolts connecting two parts with one faying surface.

d. *Tensile strength* of bolt or threaded part.

ANSI/SDI AISI S100-2024

Appendix B

Provisions Applicable to Canada

PREFACE TO APPENDIX B

Specification Chapters A through M contain design provisions that are applicable to Canada, Mexico, and the United States, and accommodate those provisions that may be partially applicable to certain countries. This appendix addresses *Specification* provisions that are applicable only to Canada.

While this document is referred to as a "*Specification,*" in Canada it is considered a "*Standard*."

Also included in Appendix B are technical items where full agreement between the three countries was not reached. The most noteworthy of these items are the following:

(a) Beams (C- and Z-sections) for standing seam roofs,

(b) Bolted and welded connections, and

(c) Lateral and stability bracing.

Efforts will be made to minimize these differences in future editions of the *Specification.*

In Canada, SI units are the units of record for the purpose of this *Specification*.

APPENDIX B, PROVISIONS APPLICABLE TO CANADA

Specification Chapters A through M contain design provisions that are applicable to Canada, Mexico, and the United States, and accommodate those provisions that may be partially applicable to certain countries. This appendix is considered mandatory for applications in Canada.

A section number ending with the letter "a" indicates that the provisions herein supplement the corresponding section in Chapters A through M of the *Specification*. A section number not ending with the letter "a" indicates that the section gives the entire design provision.

C2a Lateral and Stability Bracing

Structural members and assemblies shall be adequately braced to prevent collapse and to maintain their integrity during the anticipated service life of the structure. Care shall be taken to ensure that the bracing of the entire structural system is complete, particularly when there is interdependence between walls, floors, or roofs acting as *diaphragms*.

Erection diagrams shall show the details of the essential bracing requirements, including any details necessary to ensure the effectiveness of the bracing or bracing system.

The spacing of braces shall not be greater than the unbraced length assumed in the design of the member or component being braced.

C2.1 Symmetrical Beams and Columns

Discrete bracing of axially loaded compression members shall meet the requirements specified in Section C2.3 of the *Specification*. In addition, the provisions of Sections C2.1.1 and C2.1.2 of this appendix shall apply to symmetric sections in compression or bending in which the applied *load* does not induce twist.

C2.1.1 Discrete Bracing for Beams

The *factored resistance* of braces shall be at least 2 percent of the factored compressive force in the compressive *flange* of a member in bending at the braced location. When more than one brace acts at a common location and the nature of the braces is such that combined action is possible, the bracing force may be shared proportionately. The slenderness ratio of compressive braces shall not exceed 200.

C2.1.2 Bracing by Deck, Slab, or Sheathing for Beams and Columns

The *factored resistance* of the attachments along the entire length of the braced member shall be at least 5 percent of either the maximum factored compressive force in a compressive member or the maximum factored compressive force in the compressive *flange* of a member in bending.

C2.2a C-Section and Z-Section Beams

The provisions of Sections C2.2.2, C2.2.3, and C2.2.4 of this appendix apply to members in bending in which the applied *load* in the plane of the *web* induces twist. Braces shall be designed to avoid local crippling at the points of attachment to the member.

C2.2.2 Discrete Bracing

Braces shall be connected so as to effectively restrain both *flanges* of the section at the ends and at intervals not greater than one-quarter of the span length in such a manner as to prevent tipping at the ends and lateral deflection of either *flange* in either direction at the intermediate braces. Fewer braces may be used if this approach can be shown to be acceptable by *rational engineering analysis*, testing, or Section I6.2.1 of the *Specification*, taking into account the effects of both lateral and torsional displacements.

If fewer braces are used (when shown to be acceptable by *rational engineering analysis* or testing), those sections used as *purlins* with "floating"-type roof sheathings that allow for expansion and contraction independent of the *purlins* shall have a minimum of one brace per bay for spans ≤ 7 m and two braces per bay for spans > 7 m.

If one-third or more of the total *load* on the member is concentrated over a length of one-twelfth or less of the span of the beam, an additional brace shall be placed at or near the centre of this loaded length.

C2.2.3 One Flange Braced by Deck, Slab, or Sheathing

The *factored resistance* of the attachment of the continuous deck, slab, or sheathing shall be in accordance with Section C2.1.2 of this appendix. Discrete bracing shall be provided to restrain the *flange* that is not braced by the deck, slab, or sheathing. The spacing of discrete bracing shall be in accordance with Section C2.2.2 of this appendix.

C2.2.4 Both Flanges Braced by Deck, Slab, or Sheathing

The *factored resistance* of the attachment shall be as given by Section C2.1.2 of this appendix.

I2 Floor, Roof, or Wall Steel Diaphragm Construction

The following AISI standards shall be applied, as applicable, for *diaphragm* design: AISI S240 and AISI S400.

I4 Cold-Formed Steel Light-Frame Construction

The design, manufacture, installation, and quality of *structural members* and *connections* utilized in *cold-formed steel light-frame construction* applications shall be in accordance with AISI S240 and, as applicable, the seismic requirements of AISI S400.

I6a Metal Roof and Wall Systems

I6.2.2 Flexural Members Having One Flange Fastened to a Standing Seam Roof System

This type of member shall have discrete bracing in accordance with Section C2.2.2 of this appendix.

J2a Welded Connections

Fabricators and erectors performing arc welding shall comply with the requirements of CSA W47.1 (Division 1 or Division 2). The work may be sublet to a Division 3 fabricator or erector;

however, the Division 1 or Division 2 fabricator or erector shall retain responsibility for the sublet work. Fabricators and erectors performing resistance welding shall comply with the requirements of CSA W55.3.

Note: In Canada, accreditation of welding inspection bodies is provided by the Standards Council of Canada.

Where at least one of the connected parts is between 0.70 mm and 4.76 mm in base steel *thickness*, welding shall conform to the requirements contained herein and shall be performed in accordance with the applicable requirements of CSA W59. Except as provided in Section J2.2 of the *Specification*, where at least one of the connected parts is less than 0.70 mm in base steel *thickness*, welds shall be considered to have no structural value unless a value is substantiated by appropriate tests. For arc spot welds connecting sheets to a thicker supporting member, the applicable base steel *thickness* limits shall be 0.70 mm to 5.84 mm.

The resistance in tension or compression of butt welds shall be the same as that prescribed for the lower strength of base metal being joined. The butt weld shall fully penetrate the *joint*.

J3.4 Shear and Tension in Bolts

For ASTM A307 bolts less than 12.7 mm in diameter and SAE J429 bolts, refer to Tables J3.4-1(a), J3.4-1(b), and J3.4-2 of this appendix. For all other bolts, refer to CSA S16. The *resistance factors* given in this section shall be used to determine the *available strengths* in accordance with the applicable design method in Section B3.2.3.

ϕ_t = 0.65 *resistance factor* for tension of a bolt

ϕ_v = 0.55 *resistance factor* for shear of a bolt

The *nominal bolt resistance*, P_n, resulting from shear, tension, or a combination of shear and tension shall be calculated as follows:

$$P_n = A_b F_n \qquad (Eq.\ J3.4\text{-}1)$$

where

A_b = Gross *cross-sectional area* of bolt

F_n = A value determined in accordance with Items (a) and (b) below, as applicable:

(a) When bolts are subjected to shear or tension,
F_n is given by F_{nt} or F_{nv} in Table J3.4-1.

(b) When bolts are subjected to a combination of shear and tension,
F_n is given by F'_{nt} in Table J3.4-2.

The pull-over resistance of the connected sheet at the bolt head, nut, or washer shall be considered where bolt tension is involved. See Section J6.2 of the *Specification*.

TABLE J3.4-1(a)
Nominal Tensile and Shear Stresses for ASTM Bolts

Description of Bolts	Nominal Tensile Stress, F_{nt} (MPa)	Resistance Factor, ϕ_t	Nominal Shear Stress, F_{nv} (MPa)	Resistance Factor, ϕ_v
A307 Bolts, Grade A 6.4 mm ≤ d < 12.7 mm	279	0.65	165	0.55

TABLE J3.4-1(b)
Nominal Tensile and Shear Stresses for SAE Bolts

Bolt Type[a, b]	Nominal Tensile Strength F_{nt} (MPa)			Nominal Shear Strength F_{nv} (MPa)		
Bolt diameter, d, mm	6.4 ≤ d < 12.7	12.7 ≤ d ≤ 19.1	d>19.1	6.4 ≤ d < 12.7	12.7 ≤ d ≤ 19.1	d>19.1
SAE J429 Grade 2 Bolts: • Threads Included • Threads Excluded	345	386	310	207 290	228 290	186 234
Bolt diameter, d, mm	6.4 ≤ d < 12	12.7 ≤ d ≤ 25.4	d>25.4	6.4 ≤ d < 12	12.7 ≤ d ≤ 25.4	d>25.4
SAE J429 Grade 5 Bolts: • Threads Included • Threads Excluded	558	621	545	331 469	372 469	324 407
Bolt diameter, d, mm	6.4 ≤ d < 12.7	d>12.7		6.4 ≤ d < 12.7)	d>12.7	
SAE J429 Grade 8 Bolts: • Threads Included • Threads Excluded	696	779		414 579	469 579	
Threaded Parts: • Threads Included • Threads Excluded	$0.675F_u$[c]	$0.75\ F_u$		$0.400\ F_u$ $0.563\ F_u$	$0.450\ F_u$ $0.563\ F_u$	

Notes:
a. "Threads Included" refers to when threads are not excluded from shear planes.
b. "Threads Excluded" refers to when threads are excluded from shear planes.
c. *Tensile strength* of bolt or threaded part.

TABLE J3.4-2
Nominal Tensile Stress for Bolts
Subjected to the Combination of Shear and Tension

Description of Bolts	Nominal Tensile Stress, F'_{nt} (MPa)	Resistance Factor, ϕ_t
A307 Bolts, Grade A When 6.4 mm ≤ d < 12.7 mm	$324 - 2.4f_v \leq 279$	0.65
SAE J429	Refer to CSA S16	

Note: The actual shear *stress*, f_v, shall also satisfy Table J3.4-1 of this appendix.

K2.1.1a Load and Resistance Factor Design and Limit States Design

To calculate the *resistance factor* of an interior partition wall stud that is in a composite steel-framed wall system with gypsum sheathing attached to both *flanges* and that is limited to a transverse (out-of-plane) *specified load* of not more than 0.5 kPa, a superimposed *specified* axial *load*, exclusive of sheathing materials, of not more than 1.46 kN/m, or a superimposed *specified* axial *load* not more than 0.89 kN, the following shall apply:

(a) C_ϕ = 1.42,
(b) M_m = 1.10,
(c) F_m = 1.00,
(d) V_M = 0.10,
(e) V_F = 0.05, and
(f) β_o = 1.82.

Commentary on
North American Specification
for the Design of Cold-Formed
Steel Structural Members

2024 Edition

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First Printing, December 2024

PREFACE

This document provides a commentary on the 2024 edition of the *North American Specification for the Design of Cold-Formed Steel Structural Members.*

The purpose of the *Commentary* is: (a) to provide a record of the reasoning behind, and justification for, the various provisions of the *North American Specification* by cross-referencing the published supporting research data, and to discuss the changes made in the current *Specification;* (b) to offer a brief but coherent presentation of the characteristics and performance of cold-formed steel structures to structural engineers and other interested individuals; (c) to furnish the background material for a study of cold-formed steel design methods to educators and students; and (d) to provide the needed information to those who will be responsible for future revisions of the *Specification.* The readers who wish to have more complete information, or who may have questions which are not answered by the abbreviated presentation of this *Commentary*, should refer to the original research publications.

Consistent with the *Specification*, the *Commentary* contains a main document, Chapters A through M, Appendices 1 through 5, and the country-specific provisions Appendices A and B. A symbol ☞**A,B** is used in the main document to point out that additional discussions are provided in the corresponding country-specific provisions in Appendices A or B.

This *Commentary* is published along with the publication of the *Specification*.

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TABLE OF CONTENTS
COMMENTARY ON THE NORTH AMERICAN SPECIFICATION FOR THE DESIGN OF COLD-FORMED STEEL STRUCTURAL MEMBERS

COMMENTARY ON THE NORTH AMERICAN SPECIFICATION FOR THE DESIGN OF COLD-FORMED STEEL STRUCTURAL MEMBERS

INTRODUCTION

Cold-formed steel members have been used economically for building construction and other applications (Winter, 1959a, 1959b; Yu and LaBoube, 2010). These types of sections are cold-formed from steel sheet, strip, plate or flat bar in roll-forming machines or by press brake or bending operations. The *thicknesses* of steel sheets or strips generally used for *cold-formed steel structural members* range from 0.0147 in. (0.373 mm) to about 1/4 in. (6.35 mm). Steel plates and bars as thick as 1 in. (25.4 mm) can be cold-formed successfully into structural shapes.

In general, *cold-formed steel structural members* can offer several advantages for building construction (Winter, 1970; Yu and LaBoube, 2010): (1) Light members can be manufactured for relatively light loads and/or short spans, (2) Unusual sectional configurations can be produced economically by cold-forming operations and consequently favorable strength-to-weight ratios can be obtained, (3) Load-carrying panels and decks can provide useful surfaces for floor, roof and wall construction, and in some cases they can also provide enclosed cells for electrical and other conduits, and (4) Panels and decks not only withstand loads normal to their surfaces, but they can also act as shear *diaphragms* to resist forces in their own planes if they are adequately interconnected to each other and to supporting members.

The use of cold-formed steel members in building construction began around the 1850s. However, in North America, such steel members were not widely used in buildings until the publication of the first edition of the American Iron and Steel Institute (AISI) *Specification* in 1946 (AISI, 1946). This first design standard was primarily based on the research work sponsored by AISI at Cornell University since 1939. It was revised subsequently by the AISI Committee in 1956, 1960, 1962, 1968, 1980, and 1986 to reflect the technical developments and the results of continuing research. In 1991, AISI published the first edition of the *Load and Resistance Factor Design Specification for Cold-Formed Steel Structural Members* (AISI, 1991). Both *Allowable Stress Design* (*ASD*) and *Load and Resistance Factor Design* (*LRFD*) *Specifications* were combined into a single document in 1996.

In Canada, the CSA Group (CSA) published its first edition of *Design of Light Gauge Steel Structural Members* in 1963 based on the 1962 edition of the AISI *Specification*. Subsequent editions were published in 1974, 1984, 1989 and 1994. The Canadian *Standard for Cold Formed Steel Structural Members* (CSA, 1994) was based on the *Limit States Design* (*LSD*) method.

In Mexico, *cold-formed steel structural members* have also been designed on the basis of AISI *Specifications*. The 1962 edition of the AISI *Design Manual* (AISI, 1962) was translated into Spanish in 1965 (Camara, 1965).

The first edition of the *North American Specification* (AISI, 2001), applicable to the United States, Canada and Mexico, was published in 2001. This 2001 edition of the *Specification* was developed on the basis of the 1996 AISI *Specification* with the 1999 *Supplement* (AISI, 1996, 1999), the 1994 CSA *Standard* (CSA, 1994), and subsequent developments. In the *North American Specification*, the *ASD* and *LRFD* methods are used in the United States and Mexico, while the *LSD* method is used in Canada. The *North American Specification* was revised and updated in 2004, 2007, 2010, 2012, and 2016 (AISI, 2004; AISI, 2007a; AISI, 2010; AISI, 2012a; and AISI, 2016a) as new technology was adopted. The *Direct Strength Method* was introduced in 2004 (AISI, 2004) as an alternative design method. The *second-order analysis* of structural systems was added in 2007 (AISI, 2007a). In the

2012 edition of the *Specification*, the added design provisions included the design of *power-actuated fasteners* and the *Direct Strength Method* for determining compression and flexural strength of perforated members, shear strength for non-perforated members, and member reserve capacities.

In 2016, the *North American Specification for the Design of Cold-Formed Steel Structural Members* was reorganized—The *Direct Strength Method* was moved into Chapters D through H of the *Specification* and is considered as an equivalent design method to the *Effective Width Method*; the provisions for determining the *effective width* of elements were moved into Appendix 1; and the determination of *buckling loads* was moved into Appendix 2. The provisions were reorganized to be consistent, where possible, with the layout of the AISC Specification. Accordingly, the *Commentary* on the *Specification* has also been revised and reorganized.

In addition to the issuance of the design *Specification*, AISI also published the first edition of the *Design Manual* in 1949 (AISI, 1949). This *Allowable Stress Design* manual was revised in 1956, 1961, 1962, 1968, 1977, 1983, and 1986. In 1991, the *LRFD Design Manual* was published for using the *Load and Resistance Factor Design* criteria. The AISI 1996 *Cold-Formed Steel Design Manual* was prepared for the combined AISI *ASD* and *LRFD Specifications*. The *Cold-Formed Steel Design Manual* was updated in 2002, 2008, 2013 and 2017 (AISI, 2002; AISI, 2008; AISI, 2012; AISI, 2017) as the *Specification* was revised (AISI, 2001; AISI, 2007a; AISI 2012a; and AISI, 2016a).

During the period from 1958 through 1983, AISI published *Commentaries* on several editions of the AISI design *Specifications*, which were prepared by Professor George Winter of Cornell University in 1958, 1961, 1962, and 1970. Since 1983, the format used for the AISI *Commentary* has been such that the same section numbers are used for the *Commentary* as for the *Specification*. The *Commentary* on the 1996 AISI *Specification* was prepared by Professor Wei-Wen Yu of the University of Missouri-Rolla (Yu, 1996). The 2001 edition of the *Commentary* (AISI, 2001) was based on the *Commentary* for the 1996 AISI *Specification*.

The current edition of the *Commentary* is updated for the 2024 edition of the *North American Specification*. It contains Chapters A through M, Appendices 1 through 5, and Appendices A and B, where commentary on provisions that are only applicable to a specific country is included in the corresponding lettered appendix.

As in previous editions of the *Commentary*, this document contains a brief presentation of the characteristics and performance of *cold-formed steel structural members*, *connections* and assemblies. In addition, it provides a record of the reasoning behind, and the justification for, various provisions of the *Specification*. A cross-reference is provided between various design provisions and the published research data.

In this *Commentary,* the individual sections, equations, figures, and tables are identified by the same notation as in the *Specification* and the material is presented in the same sequence. Bracketed terms used in the *Commentary* are equivalent terms that apply particularly to the *LSD* method in Canada.

The *Specification* and *Commentary* are intended for use by design professionals with demonstrated engineering competence in their fields.

A. GENERAL PROVISIONS

A1 Scope, Applicability, and Definitions

A1.1 Scope

The cross-sectional configurations, manufacturing processes and fabrication practices of *cold-formed steel structural members* differ in several respects from those of hot-rolled steel shapes. For cold-formed steel sections, the forming process is performed at, or near, room temperature by the use of bending brakes, press brakes, or roll-forming machines. Some of the significant differences between cold-formed sections and hot-rolled shapes are: (1) absence of the residual *stresses* caused by uneven cooling due to hot-rolling, (2) lack of corner fillets, (3) presence of increased *yield stress* with decreased proportional limit and ductility resulting from cold-forming, (4) presence of cold-reducing *stresses* when cold-rolled steel stock has not been finally annealed, (5) prevalence of elements having large width-to-*thickness* ratios, (6) rounded corners, and (7) different characteristics of *stress*-strain curves that can be either the sharp-yielding type or gradual-yielding type.

The *Specification* is applicable only to cold-formed sections not more than 1 inch (25.4 mm) in *thickness*. Research conducted at the University of Missouri-Rolla (Yu, Liu, and McKinney, 1973b and 1974) has verified the applicability of the *Specification*'s provisions for such cases.

In view of the fact that most of the design provisions have been developed on the basis of experimental work subject to static loading, the *Specification* is intended for the design of *cold-formed steel structural members* to be used for load-carrying purposes in buildings. For structures other than buildings, appropriate allowances should be made for dynamic effects.

A1.2 Applicability

The *Specification* is limited to the design of steel structural members cold-formed from carbon or low-alloy sheet, strip, plate or bar. The design can be made by using either the *Allowable Strength Design* (*ASD*) method or the *Load and Resistance Factor Design* (*LRFD*) method for the United States and Mexico. Only the *Limit States Design* (*LSD*) method is permitted in Canada.

In this *Commentary*, the bracketed terms are equivalent terms that apply particularly to *LSD*. A symbol ☞X is used to point out that additional provisions are provided in the country-specific appendices as indicated by the letter, x.

Because of the diverse forms of *cold-formed steel structural members* and *connections*, it is not possible to cover all design configurations by the design rules presented in the *Specification*. For those special cases where the *available strength* [*factored resistance*] and/or stiffness cannot be determined, it can be established by:

(a) Testing in accordance with the provisions of Section K2.1.1(a),

(b) *Rational engineering analysis* and confirmatory testing evaluated in accordance with the provisions of Section K2.1.1(b), or

(c) *Rational engineering analysis* only in accordance with the provisions of Section A1.2.6(c). Prior to 2001, the only option in such cases was testing. Since 2001, in recognition of the fact that this was not always practical or necessary, the *rational engineering analysis* options were added. It is essential that such analysis be based on theory that is appropriate for the situation and sound engineering judgment. *Specification* Section A1.2.6(b) was added for components that have significant geometric variations such that it becomes impractical to test each variation in accordance with *Specification* Section A1.2.6(a). This is particularly useful when the following applies:

(1) A form of cold-formed steel component is being evaluated that is outside the scope of the *Specification,*

(2) The member or assembly being evaluated has a degree of variation, such as variations in cross-sectional dimensions, that makes it impractical to test each individual variation,

(3) More accurate *safety* and *resistance factors* than those prescribed by Section A1.2.6(c) are desired, and

(4) A test program can be conducted in accordance with Section K2.

In any case, *safety* and *resistance factors* given in *Specification* Section A1.2.6(c) should not be used if applicable *safety factors* or *resistance factors* in *Specification* Chapters A through M, Appendices 1 through 5, and Appendices A and B are more conservative. These provisions must not be used to circumvent the intent of the *Specification*. Where the provisions of Chapters B through J and L through M of the *Specification* and Appendices A and B apply, those provisions must be used and cannot be avoided by testing or *rational engineering analysis*.

In 2022, provisions were added to clarify that when the *safety* and *resistance factors* are determined in accordance with *Specification* Section A1.2.6, these *safety* and *resistance factors* are to be used in interaction equations of Chapter H. Some sections in Chapter H list *safety* and *resistance factors* that assume the *nominal strength* [*resistance*] was determined in accordance with Chapters D through M or Appendices 1 through 5, A or B. For cases where *Specification* Section A1.2.6 is used, the use of the *safety* and *resistance factors* listed in Chapter H rather than those determined in accordance with Section A1.2.6 could be unconservative.

In order to provide better alignment between the testing provisions of Section K2 and the provisions for *rational engineering analysis*, the *safety factor* for the *rational engineering analysis* of *connections* was adjusted upward from $\Omega = 2.5$ to $\Omega = 3.0$ in 2016. Compatible adjustments were also made to the accompanying *resistance factors*, ϕ, for *rational engineering analysis*.

A1.3 Definitions

Many of the definitions in *Specification* Section A1.3 for *ASD*, *LRFD* and *LSD* are self-explanatory. Only those which are not self-explanatory are briefly discussed below.

General Terms

Effective Design Width

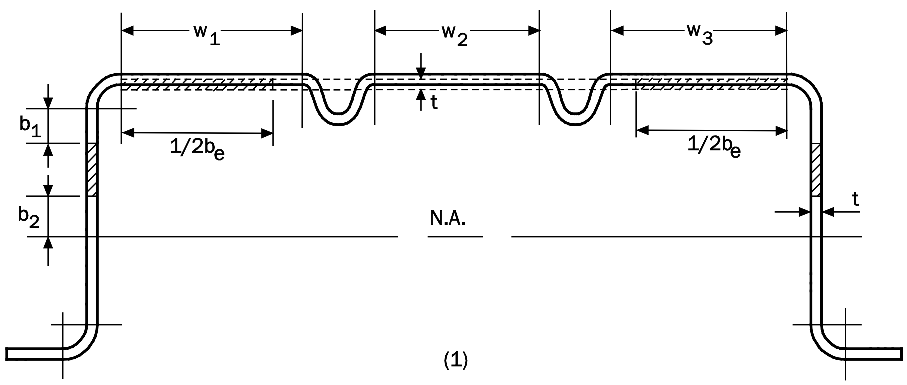
The *effective design width* is a concept which facilitates taking account of *local buckling* and post-*buckling* strength for compression elements. The effect of shear lag on short, wide *flanges* is also handled by using an *effective design width*. These matters are treated in *Specification* Appendix 1, and the corresponding *effective widths* are discussed in the *Commentary* on that appendix.

Multiple-Stiffened Elements

Multiple-stiffened elements of two cross-sections are shown in Figure C-A1.3-1. Each of the two outer sub-elements of cross-section (1) is stiffened by a *web* and an intermediate stiffener while the middle sub-element is stiffened by two intermediate stiffeners. The two sub-elements of cross-section (2) are stiffened by a *web* and the attached intermediate middle stiffener.

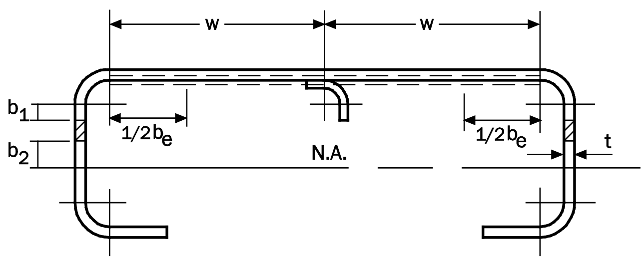
Stiffened or Partially Stiffened Compression Elements

Stiffened compression elements of various cross-sections are shown in Figure C-A1.3-2 in which Cross-sections (1) through (5) are for flexural members, and Cross-sections (6) through (9) are for compression members. Cross-sections (1) and (2) each have a *web* and a lip to stiffen the compression element (i.e., the compression *flange*), the ineffective portion of which is shown shaded. For the explanation of these ineffective portions, see the discussion of *Effective Design Width* and Appendix 1. Cross-sections (3), (4), and (5) show compression elements stiffened by two *webs*. Cross-sections (6) and (8) show edge-stiffened *flange* elements that have a vertical element (*web*) and an edge stiffener (lip) to stiffen the elements while the *web* itself is stiffened by the *flanges*. Cross-section (7) has four compression elements stiffening each other, and cross-section (9) has each stiffened element stiffened by a lip and the other stiffened element.



(1)

Multiple-Stiffened Hat-Section



(2)

Multiple-Stiffened Inverted "U"-Type Section

Flexural Members, Such as Beams

Figure C-A1.3-1 Multiple-Stiffened Compression Elements

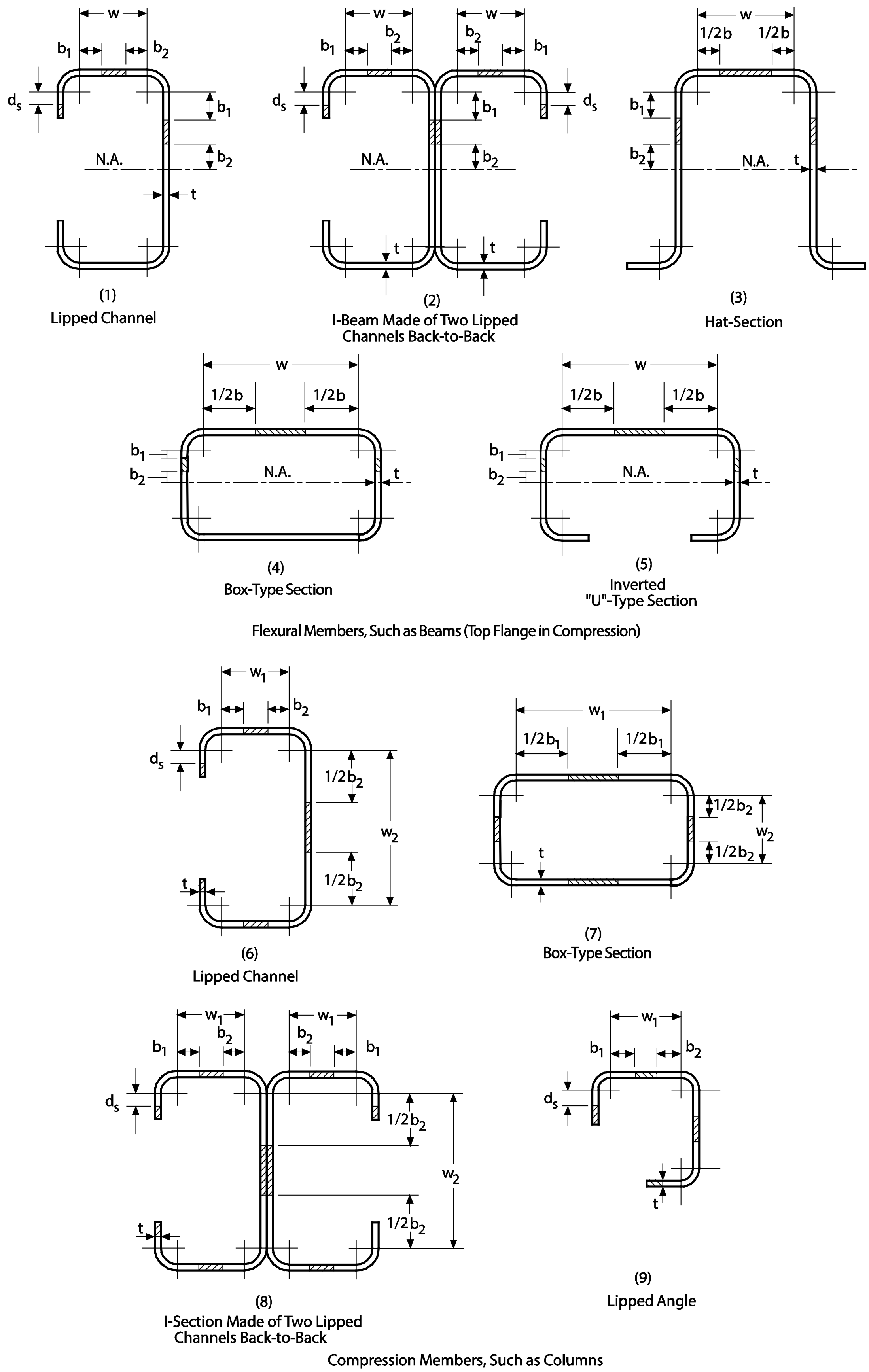


Figure C-A1.3-2 Stiffened Compression Elements

Thickness

In calculating section properties, the reduction in *thickness* that occurs at corner bends is ignored, and the base metal *thickness* of the flat steel stock, exclusive of coatings, is used in all calculations for load-carrying purposes.

Flexural-Torsional Buckling

The 1968 edition of the *Specification* pioneered methods for computing column loads of cold-formed steel cross-sections prone to *buckling* by simultaneous twisting and bending. This complex behavior may result in lower column loads than would result from primary *buckling* by flexure alone.

Unstiffened Compression Elements

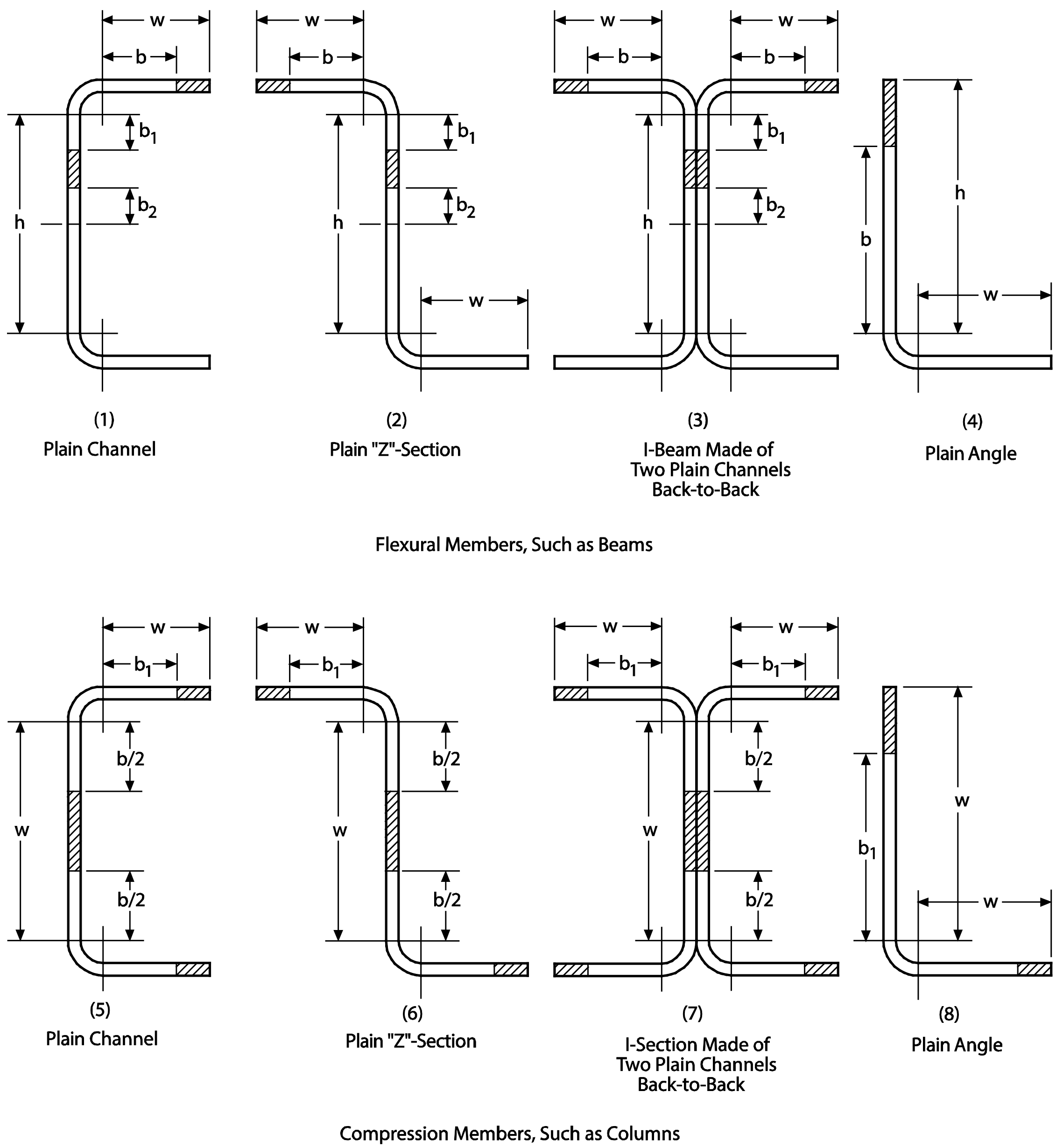


Figure C-A1.3-3 Unstiffened Compression Elements

Unstiffened elements of various cross-sections are shown in Figure C-A1.3-3, in which Cross-sections (1) through (4) are for flexural members and cross-sections (5) through (8) are for compression members. Cross-sections (1), (2), and (3) have only a *web* to stiffen the compression *flange* element. The legs of Cross-section (4) provide mutual stiffening action to each other along their common edges. Cross-sections (5), (6), and (7), acting as columns, have vertical stiffened elements (*webs*) which provide support for one edge of the unstiffened *flange* elements. The legs of Cross-section (8) provide mutual stiffening action to each other.

ASD and LRFD Terms (United States and Mexico)

ASD (Allowable Strength Design, formerly referred to as Allowable Stress Design)

Allowable Strength Design (*ASD*) is a method of designing *structural components* such that the *allowable strength* (force or moment) permitted by various sections of the *Specification* is not exceeded when the structure is subjected to all appropriate *loads* and *load* combinations in accordance with *Specification* Section B2. See also *Specification* Section B3.2.1 for *ASD* requirements.

LRFD (Load and Resistance Factor Design)

Load and Resistance Factor Design (*LRFD*) is a method of designing *structural components* such that the applicable limit state is not exceeded when the structure is subjected to all appropriate *loads* and *load* combinations in accordance with *Specification* Section B2. See also *Specification* Section B3.2.2 for *LRFD* requirements.

LSD Terms (Canada)

LSD (Limit States Design)

Limit States Design (*LSD*) is a method of designing *structural components* such that the applicable limit state is not exceeded when the structure is subjected to all appropriate *loads* and *load* combinations in accordance with *Specification* Section B2. See also *Specification* Section B3.2.3 for *LSD* requirements.

In the *Specification*, the terminologies for *Limit States Design* (*LSD*) are given in brackets parallel to those for *Load and Resistance Factor Design* (*LRFD*). The inclusion of *LSD* terminology is intended to help engineers who are familiar with *LSD* better understand the *Specification*.

It should be noted that the design concept used for the *LRFD* and the *LSD* methods is the same, except that the *load factors*, load combinations, assumed dead-to-live *load* ratios, and target reliability indexes are slightly different. In most cases, the same *nominal strength* [*resistance*] equations are used for *ASD*, *LRFD*, and *LSD* approaches.

A1.4 Units of Symbols and Terms

The nondimensional character of the majority of the *Specification* provisions is intended to facilitate design in any compatible systems of units (U.S. customary, SI or metric, and MKS systems).

The conversion of U.S. customary into SI metric units and MKS systems are given in parentheses throughout the entire text of the *Specification* and *Commentary*. Table C-A1.4-1 is a conversion table for these three different units.

Table C-A1.4-1
Conversion Table

	To Convert	To	Multiply by
Length	in.	mm	25.4
	mm	in.	0.03937
	ft	m	0.30480
	m	ft	3.28084
Area	in^2	mm^2	645.160
	mm^2	in^2	0.00155
	ft^2	m^2	0.09290
	m^2	ft^2	10.7639
Force	kip	kN	4.448
	kip	kg	453.5
	lb	N	4.448
	lb	kg	0.4535
	kN	kip	0.2248
	kN	kg	101.96
	kg	kip	0.0022
	kg	N	9.808
Stress	ksi	MPa	6.895
	ksi	kg/cm^2	70.30
	MPa	ksi	0.145
	MPa	kg/cm^2	10.196
	kg/cm^2	ksi	0.0142
	kg/cm^2	MPa	0.0981

A2 Referenced Specifications, Codes, and Standards

Other specifications and standards to which the *Specification* makes references have been listed and updated in *Specification* Section A2 to provide the effective dates of these standards at the time of approval of this *Specification*. References for country-specific provisions are provided in *Specification* Section A2.1 for the U.S. and Mexico and A2.2 for Canada.

Additional references which the designer may use for related information are listed in the *Commentary* section, References.

A3 Material

A3.1 Applicable Steels

ASTM International and CSA Group are the basic sources of steel designations for use with the *Specification. Specification* Section A3.1 contains the complete list of steel standards that are accepted by the *Specification.* Dates of issue are included in *Specification* Section A2.

In 2012, the list of applicable steels was enhanced by categorizing them into three groups based on the specified minimum elongation in a 2-inch (50-mm) gage length: ten (10) percent or greater elongation, three (3) percent to ten (10) percent elongation, and less than three (3) percent elongation. This eliminated the need to identify specific steel grades in subsequent sections.

In the 1996 *Specification*, the ASTM A446 Standard was replaced by the ASTM A653/A653M Standard. At the same time, the ASTM A283/A283M Standard, High-Strength, Low-Alloy Steel (HSLAS) Grades 70 (480) and 80 (550) of ASTM A653/A653M and ASTM A715 were added.

In 2001, the ASTM A1008/A1008M and ASTM A1011/A1011M Standards replaced the ASTM A570/A570M, ASTM A607, ASTM A611, and ASTM A715 Standards. ASTM A1003/A1003M was added to the list of the applicable steels.

In 2007, the ASTM A1039 Standard was added to the list of the applicable steels. For all grades of steel, ASTM A1039 complies with the minimum required F_u/F_y ratio of 1.08. *Thicknesses* equal to or greater than 0.064 in. (1.6 mm) and less than or equal to 0.078 in. (2.0 mm) also meet the minimum elongation requirements of *Specification* Section A3.1.1 and no reduction in the *specified minimum yield stress* is required. However, steel *thicknesses* less than 0.064 in. (1.6 mm) with *yield stresses* greater than 55 ksi (380 MPa) do not meet the requirements of *Specification* Section A3.1.1 and are subject to the limitations of *Specification* Section A3.1.2.

In 2012, the ASTM A1063/A1063M Standard was added to the list of the applicable steels. The ASTM A1063/A1063M Standard is intended to be a match to ASTM A653/A653M, but the materials are produced using a “twin-roll casting process,” which is also used to produce materials conforming to the ASTM A1039/A1039M Standard.

In 2022, the ASTM A463/A463M, ASTM A1018/A1018M, ASTM A1046/A1046M, ASTM A1079, ASTM A1083/A1083M, and ASTM A1088 Standards were added to the list of *Specification* Section A3.1. These standards reflect additional high-strength steel grades that are available. These added high-strength steels may not all be applicable for welded construction. Where the steel is to be welded, a welding procedure suitable for the grade of the steel and intended use or service is to be utilized.

The important material properties for the design of cold-formed steel members are *yield stress*, *tensile strength*, and ductility. Ductility is the ability of steel to undergo sizable plastic or permanent strains before fracturing and is important both for structural safety and for cold-forming. It is usually measured by the elongation in a 2-inch (50-mm) gage length. The ratio of the *tensile strength* to the *yield stress* is also an important material property; this is an indication of strain hardening and the ability of the material to redistribute *stress*.

A3.1.1 Steels With a Specified Minimum Elongation of Ten Percent or Greater (Elongation ≥ 10%)

Material specifications for low-carbon sheet and strip steels with *specified minimum yield stress* from 24 to 50 ksi (165 to 345 MPa or 1690 to 3520 kg/cm^2) provide specified minimum elongations in a 2-inch (50-mm) gage length of 11 to 30 percent, thus easily meeting the 10-percent minimum requirement for this category. Steels with *yield stresses* higher than 50 ksi (345 MPa or 3520 kg/cm^2) are often produced as low-alloy steels in order to meet these ductility requirements. Elongations are determined in accordance with ASTM test method A370 (A1058).

For the listed standards, the *yield stresses* of steels range from 24 to 80 ksi (165 to 550 MPa or 1690 to 5620 kg/cm^2) and the *tensile strengths* vary from 42 to 100 ksi (290 to 690 MPa or 2950 to 7030 kg/cm^2). The tensile-to-yield ratios are not less than 1.13, and the elongations are not less than 10 percent. The conditions for use of steels that have a defined ductility of at least three percent (3%) are outlined in *Specification* Section A3.1.2. The conditions for use of steels that have a defined ductility of less than three percent (3%) are outlined in *Specification* Section A3.1.3.

For ASTM A1003/A1003M steel, even though the minimum *tensile strength* is not specified in the ASTM Standard for each of Types H and L steels, the footnote of Table 2 of the Standard states that for Type H steels, the ratio of *tensile strength* to *yield stress* shall not be less than 1.08. Thus, a conservative value of $F_u = 1.08\ F_y$ can be used for the design of cold-formed steel members using Type H steels. Based on the same standard, a conservative value of $F_u = F_y$ can be used for the design of *purlins* and *girts* using Type L steels. In 2004, the *Specification* listing of ASTM A1003/A1003M steel was revised to list only the grades designated Type H, because this is the only grade that satisfies the criterion for unrestricted usage. Grades designated Type L can still be used but are subject to the restrictions of *Specification* Section A3.2.1.

Certain grades of ASTM A653, A792, A1011, and A1039 have elongations that vary based upon the *thickness* of the material. Exceptions are provided for those steels that do not belong to the designated group.

A3.1.2 Steels With a Specified Minimum Elongation From Three Percent to Less Than Ten Percent (3% ≤ Elongation < 10%)

Steels listed in this section have specified minimum elongations less than the 10 percent limitation for unlimited utilization within the *Specification*. However, they do have some defined ductility.

For the determination of the tension strength of members and *connections* in Grade 80 (550) Class 3 steels produced to ASTM A653/A653M and A792/A792M, tension tests on sheet steels and shear tests on *connections* using steel produced to Australian Standard AS1397 G550 (Standards Australia, 2001), which is similar in minimum ductility (2%) to ASTM A792 Grade 80 (550) Class 3 (minimum ductility 3%), were performed at the University of Sydney by Rogers and Hancock. These included sheet steels in tension with and without perforations (Rogers and Hancock, 1997), bolted *connections* in shear (Rogers and Hancock, 1998; Rogers and Hancock, 1999b), screw *connections* in shear (Rogers and Hancock, 1999a), and sheet steel fracture toughness tests (Rogers and Hancock, 2001).

For the determination of the compression strength of members of Grade 80 (550) Class 3 steels produced to ASTM A653/A653M and A792/792M, compression tests of steel produced to Australian Standard AS1397 G550 (which is similar to ASTM A792 Grade 80 (550) Class 3) were performed at the University of Sydney by Yang and Hancock (2004a, 2004b), and Yang, Hancock and Rasmussen (2004). For short-box sections where $F_n = F_y$, the study (Yang and Hancock, 2004a) shows that the limit of the *yield stress* used in design can be 90 percent of the *specified minimum yield stress,* F_{sy}, for low-ductility steels. For edge-stiffened elements with intermediate stiffener(s), stub compression testing on channel sections (Yang and Hancock, 2004b) confirms the provisions given in *Specification* Section 1.4.2. For long column tests of channel sections (Yang and Hancock, 2004b), *distortional* *buckling* as well as the interaction of *local* and *distortional buckling* controls the design. The use of 0.9 F_{sy} in the *distortional buckling* equations produces reliable results.

A3.1.3 Steels With a Specified Minimum Elongation of Less Than Three Percent (Elongation < 3%)

Steels of this group have a *specified minimum yield stress* of 80 ksi (550 MPa or 5620 kg/cm^2), a specified minimum *tensile strength* of 82 ksi (565 MPa or 5770 kg/cm^2), and no stipulated minimum elongation in a 2-inch (50-mm) gage length. These steels do not have adequate ductility as defined by *Specification* Section A3.1.1. These low-ductility steels permit only limited amounts of cold forming, require fairly large corner radii, and have other limits on their applicability for structural framing members. Their use has been limited in *Specification* Section A3.1.3 to particular multiple-*web* configurations such as roofing, siding, and floor decking.

In the past, the *yield stress* used in design was limited to 75 percent of the *specified minimum yield stress*, or 60 ksi (414 MPa or 4220 kg/cm^2), and the *tensile strength* used in design was limited to 75 percent of the specified minimum *tensile strength*, or 62 ksi (427 MPa or 4360 kg/cm^2), whichever was lower. Koka, Yu, and LaBoube (1997) studied this criterion for *connections* using SS Grade 80 (550) of A653/A653M steel. In 2022, the upper limit on F_u was adjusted to 80 percent of the specified minimum *tensile strength*, or 65 ksi (448 MPa or 4570 kg/cm^2), whichever is lower, which aligns with commonly used Grade 50 steels. The modest increase from 62 ksi (427 MPa or 4360 kg/cm^2) to 65 ksi (448 MPa or 4570 kg/cm^2) did not negatively affect the recalibrated *safety* and *resistance factors* for welds and screws determined in the research (Blackburn and Sputo, 2016; and Stevens and Sputo, 2019). This introduced a higher *safety factor*, but still made low-ductility steels, such as *SS* Grade 80 and Grade E, useful for the named applications.

Based on the UMR research findings (Wu, Yu, and LaBoube, 1996), Equation A3.1.3-1 was added in *Specification* Section A3.1.3 to determine the reduced *yield stress*, R_bF_{sy}, for the calculation of the *nominal flexural strength* [*resistance*] of multiple-*web* sections such as roofing, siding and floor decking (AISI, 1999). For the unstiffened compression *flange*, Equation A3.1.3-2 was added on the basis of a 1988 UMR study (Pan and Yu, 1988). This revision allows the use of a higher *nominal bending strength* [*resistance*] than previous editions of the *Specification.* When the multiple-*web* section is composed of both stiffened and unstiffened compression *flange* elements, the smallest R_b should be used to determine the reduced *yield stress* for use on the entire section. Different values of the reduced *yield stress* could be used for positive and negative moments.

The equations provided in *Specification* Section A3.1.3 can also be used for calculating the *nominal flexural strength* [*resistance*] when the *available strengths* [*factored resistances*] are determined on the basis of tests as permitted by *rational engineering analysis*.

It should be noted that the exception clause in *Specification* Section A3.1 should be followed for steel deck used for composite slabs when the deck is used as the tensile reinforcement.

For the calculation of *web crippling* strength of deck panels, although the UMR study (Wu, Yu, and LaBoube, 1997) shows that the *specified minimum yield stress* can be used to calculate the *web crippling* strength of deck panels, the *Specification* provides a more conservative approach. The lesser of 0.75 F_{sy} and 60 ksi (414 MPa or 4220 kg/cm^2) is used to determine both the *web crippling* strength (*Specification* Section G5) and the shear strength (*Specification* Section G2) for the low-ductility steels. This is consistent with the previous edition of the *Specification*.

Load tests are permitted, but not for the purpose of using higher loads than can be calculated under *Specification* Chapters D through M.

A3.2 Other Steels

Although the use of the steel standards listed in *Specification* Section A3.1 is encouraged, other steels may also be used in cold-formed steel structures, provided they satisfy the requirements stipulated in *Specification* Section A3.2.

ASTM and CSA Group material standards include references to general requirements standards that cover information such as dimensional tolerances and testing protocols that are similar across a set of material standards to minimize duplication and inconsistencies. For sheet steel used for cold-formed products, the typical general requirements standards are as follows:

(a) For coated sheets, ASTM A924/A924M-14 or CSA G40.20-13, as applicable;
(b) For hot-rolled or cold-rolled sheet and strip, ASTM A568/A568M-15 or CSA G40.20-13, as applicable;
(c) For plate and bar, ASTM A6/A6M-14 or CSA G40.20-13, as applicable;
(d) For hollow structural sections (carbon steel), ASTM A500/A500M-21, ASTM A1085/A1085M-15 or CSA G40.20-13/G40.21-13, as applicable;
(e) For hollow structural sections (HSLAS steel), ASTM A847/A847M-14 or CSA G40.20-13, as applicable.

In 2004, these requirements were clarified and revised. The *Specification* has long required that such "other steels" conform to the chemical and mechanical requirements of one of the listed specifications or "other *published specification*." Specific requirements for a *published specification* have been detailed in the definitions under *Specification* Section A1.3, General Terms. It is important to note that, by this definition, published requirements must be established before the steel is ordered, not by a post-order screening process. The requirements must include minimum tensile properties, chemical composition limits, and for coated sheets, coating properties. Testing procedures must be in accordance with the referenced ASTM or CSA Group specifications. A proprietary specification of a manufacturer, purchaser, or producer could qualify as a *published specification* if it meets the definition requirements.

As an example of these *Specification* provisions, it would not be permissible to establish a minimum *yield stress* or minimum *tensile strength* greater than that ordered to a standard ASTM grade by reviewing mill test reports or conducting additional tests. However, it would be permissible to publish a manufacturer's or producer's specification before the steel is ordered requiring that such enhanced properties be furnished as a minimum. Testing to verify that the minimum properties are achieved could be done by the manufacturer or the producer. The intent of these provisions is to ensure that the material factor, M_m (see *Specification* Section K2), will be maintained at about 1.10, corresponding to an assumed typical 10 percent overrun in tensile properties for ASTM grades.

Where the material is used for fabrication by welding, care must be exercised in selection of chemical composition or mechanical properties to ensure compatibility with the welding process and its potential effect on altering the tensile properties.

Special additional requirements have been added to qualify unidentified material. In such a case, the manufacturer must run tensile tests sufficient to establish that the *yield stress* and *tensile strength* of each *master coil* are at least 10 percent greater than the applicable *published specification*. As used here, *master coil* refers to the coil being processed by the manufacturer. Of course, the testing must always be adequate to ensure that specified minimum properties are met, as well as the ductility requirements of *Specification* Section A3.1.1, A3.1.2, or A3.1.3 as desired.

A3.2.1 Ductility Requirements of Other Steels

In 1968, because new steels of higher strengths were being developed, sometimes with lower elongations, the question of how much elongation is really needed in a structure was the focus of a study initiated at Cornell University. Steels were studied that had *yield strengths* ranging from 45 to 100 ksi (310 to 690 MPa or 3160 to 7030 kg/cm^2), elongations in 2 inches (50-mm) ranging from 50 to 1.3 percent, and *tensile strength*-to-*yield strength* ratios ranging from 1.51 to 1.00 (Dhalla, Errera and Winter, 1971; Dhalla and Winter, 1974a; Dhalla and Winter, 1974b). The investigators developed elongation requirements for ductile steels. These measurements are more accurate but cumbersome to make; therefore, the investigators recommended the following determination for adequately ductile steels: (1) The *tensile strength*-to-*yield strength* ratio shall not be less than 1.08, and (2) The total elongation in a 2-inch (50-mm) gage length shall not be less than 10 percent, or not less than 7 percent in an 8-inch (200-mm) gage length. Also, the *Specification* limits the use of Chapters D through J to adequately ductile steels. In lieu of the *tensile strength*-to-*yield strength* limit of 1.08, the *Specification* permits the use of elongation requirements using the measurement technique as given by Dhalla and Winter (1974a) (Yu and LaBoube, 2010). Further information on the test procedure should be obtained from AISI S903, *Test Standard for Determining Uniform and Local Ductility of Carbon and Low-Alloy Steels*. Because of limited experimental verification of the structural performance of members using materials having a *tensile strength*-to-*yield strength* ratio less than 1.08 (Macadam et al., 1988), the *Specification* limits the use of this material to *purlins*, *girts*, and *curtain wall studs* meeting the elastic design requirements of Sections F2, F3, I6.2.1, I6.2.2, I6.3.1, and additional country-specific requirements given in the appendices. Thus, the use of such steels in other applications is prohibited. However, in *purlins*, *girts*, and *curtain wall studs* (with special country-specific requirements given in the appendices), concurrent axial loads of relatively small magnitude are acceptable providing the requirements of *Specification* Section H1.2 are met and $\Omega_c P/P_n$ does not exceed 0.15 for *allowable strength design*, $P_u/\phi_c P_n$ does not exceed 0.15 for the *Load and Resistance Factor Design*, and $P_f/\phi_c P_n$ does not exceed 0.15 for the *Limit States Design*.

In 2007, *curtain wall studs* were added to the applications for materials having a *tensile strength*-to-*yield strength* ratio less than 1.08. *Curtain wall studs* are repetitive framing members that are typically spaced more closely than *purlins* and *girts*. *Curtain wall studs* are analogous to vertical girts; as such, they are not subjected to snow or other significant sustained gravity *loads*.

With the addition of the provisions of *Specification* Section A3.1.2 in 2012, the use of the alternative approach for the limited range of structural usage is largely superseded by the provisions of *Specification* Section A3.1.2.

A3.2.1.1 Restrictions for Curtain Wall Studs

Pending future research regarding the cyclic performance of *connections*, an exception is noted on use of lower ductility steels as defined in Section A3.2.1 for *curtain wall studs* supporting heavyweight exterior walls in high seismic areas.

A3.3 Yield Stress and Strength Increase From Cold Work of Forming

A3.3.1 Yield Stress

The strength of *cold-formed steel structural members* depends on the *yield stress*, except in those cases where elastic *local buckling* or overall *buckling* is critical. Because the *stress*-strain curve of steel sheet or strip can be either the sharp-yielding type (Figure C-A3.3.1-1(a)) or gradual-yielding type (Figure C-A3.3.1-1(b)), the method for determining the *yield point* for sharp-yielding steel and the *yield strength* for gradual-yielding steel are based on ASTM Standard A370 (ASTM, 2015). As shown in Figure C-A3.3.1-2(a), the *yield point* for sharp-yielding steel is defined by the *stress* level of the plateau. For gradual-yielding steel, the *stress*-strain curve is rounded out at the "knee" and the *yield strength* is determined by either the offset method (Figure C-A3.3.1-2(b)) or the extension under the *load* method (Figure C-A3.3.1-2(c)). The term *yield stress* used in the *Specification* applies to either *yield point* or *yield strength*. Section 1.2 of the AISI *Design Manual* (AISI, 2017) lists the minimum mechanical properties specified by the ASTM specifications for various steels.

The strength of members that are governed by *buckling* depends not only on the *yield stress* but also on the modulus of elasticity of steel, E, and the tangent modulus of steel, E_t. The modulus of elasticity is defined by the slope of the initial straight portion of the *stress*-strain curve (Figure C-A3.3.1-1). The measured values of E on the basis of the standard methods usually range from 29,000 to 30,000 ksi (200 to 207 GPa or 2.0×10^6 to 2.1×10^6 kg/cm^2). A value of 29,500 ksi (203 GPa or 2.07×10^6 kg/cm^2) is used in the *Specification* for design purposes. The tangent modulus is defined by the slope of the *stress*-strain curve at any *stress* level, as shown in Figure C-A3.3.1-1(b).

For sharp-yielding steels, E_t = E up to the *yield point*, but with gradual-yielding steels, E_t = E only up to the proportional limit, f_{pr}. Once the *stress* exceeds the proportional limit, the tangent modulus, E_t, becomes progressively smaller than the initial modulus of elasticity.

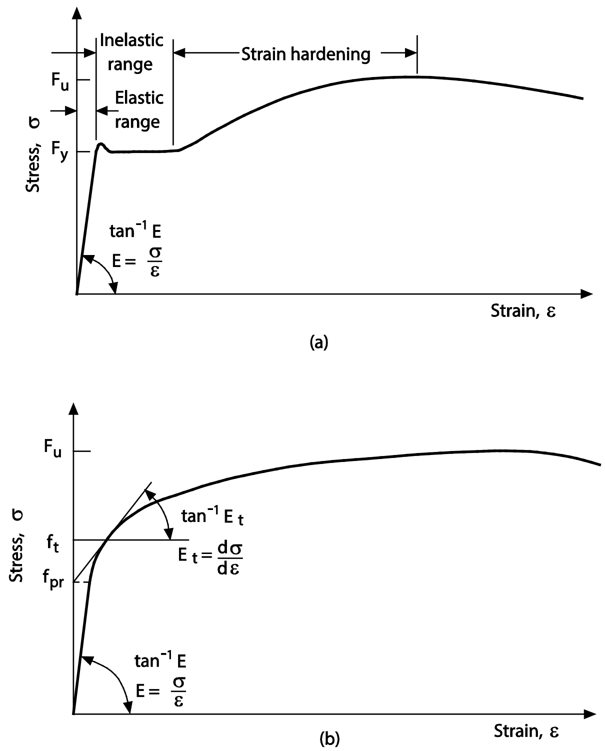


Figure C-A3.3.1-1 Stress-Strain Curves of Carbon Steel Sheet or Strip

(a) Sharp Yielding, (b) Gradual Yielding

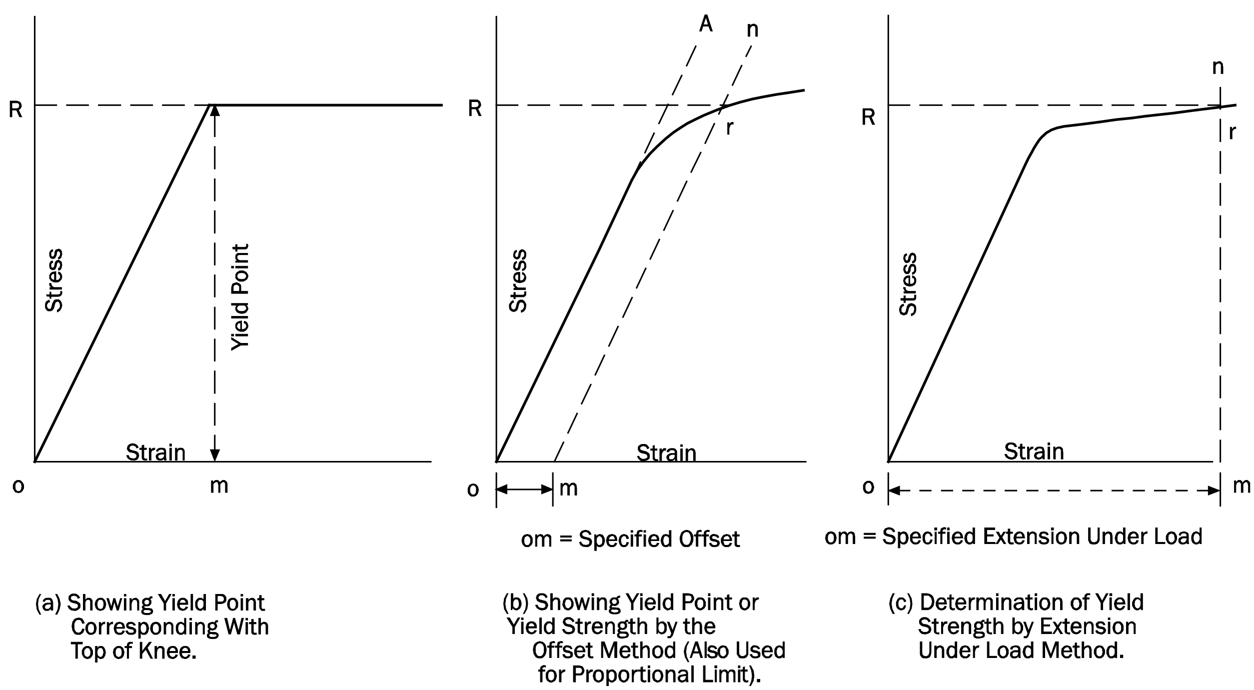


Figure C-A3.3.1-2 Stress-Strain Diagrams Showing Methods of Yield Point and Yield Strength Determination

Various *buckling* provisions of the *Specification* have been written for gradual-yielding steels whose proportional limit is not lower than about 70 percent of the *specified minimum yield stress*.

Determination of proportional limits for informational purposes can be done simply by using the offset method shown in Figure C-A3.3.1-2(b) with the distance "om" equal to 0.0001 length/length (0.01 percent offset) and calling the *stress* R where "mn" intersects the *stress*-strain curve at "r", the proportional limit.

A3.3.2 Strength Increase From Cold Work of Forming

The mechanical properties of the flat steel sheet, strip, plate or bar, such as *yield stress*, *tensile strength*, and elongation may be substantially different from the properties exhibited by the cold-formed steel sections. Figure C-A3.3.2-1 illustrates the increase of *yield stress* and *tensile strength* from those of the virgin material at the section locations in a cold-formed steel channel section and a joist chord (Karren and Winter, 1967). This difference can be attributed to cold working of the material during the cold-forming process.

The influence of cold work on mechanical properties was investigated by Chajes, Britvec, Winter, Karren, and Uribe at Cornell University in the 1960s (Chajes, Britvec, and Winter, 1963; Karren, 1967; Karren and Winter, 1967; Winter and Uribe, 1968). It was found that the changes of mechanical properties due to cold-stretching are caused mainly by strain-hardening and strain-aging, as illustrated in Figure C-A3.3.2-2 (Chajes, Britvec, and Winter, 1963). In this figure, Curve A represents the *stress*-strain curve of the virgin material. Curve B is due to unloading in the strain-hardening range, Curve C represents immediate reloading, and Curve D is the *stress*-strain curve of reloading after strain-aging. It is interesting to note that the *yield stresses* of both Curves C and D are higher than the *yield point* of the virgin material and that the ductility decreases after strain hardening and strain aging.

Cornell research also revealed that the effects of cold work on the mechanical properties of corners usually depend on: (1) the type of steel, (2) the type of *stress* (compression or tension), (3) the direction of *stress* with respect to the direction of cold work (transverse or longitudinal), (4) the F_u/F_y ratio, (5) the inside radius-to-*thickness* ratio (R/t), and (6) the amount of cold work. Among the above items, the F_u/F_y and R/t ratios are the most important factors to affect the change in mechanical properties of formed sections. Virgin material with a large F_u/F_y ratio possesses a large potential for strain hardening. Consequently, as the F_u/F_y ratio increases, the effect of cold work on the increase in the *yield stress* of steel increases. Small inside radius-to-*thickness* ratios, R/t, correspond to a large degree of cold work in a corner and therefore, for a given material, the smaller the R/t ratio, the larger the increase in *yield stress*.

Investigating the influence of cold work, Karren derived the following equations for the ratio of corner *yield stress*-to-virgin *yield stress* (Karren, 1967):

$$\frac{F_{yc}}{F_{yv}} = \frac{B_c}{(R/t)^m} \qquad \text{(C-A3.3.2-1)}$$

where

$$B_c = 3.69\frac{F_{uv}}{F_{yv}} - 0.819\left(\frac{F_{uv}}{F_{yv}}\right)^2 - 1.79$$

$$m = 0.192\frac{F_{uv}}{F_{yv}} - 0.068$$

F_{yc} = Corner *yield stress*
F_{yv} = Virgin *yield stress*
F_{uv} = Virgin ultimate *tensile strength*
R = Inside bend radius
t = Sheet *thickness*

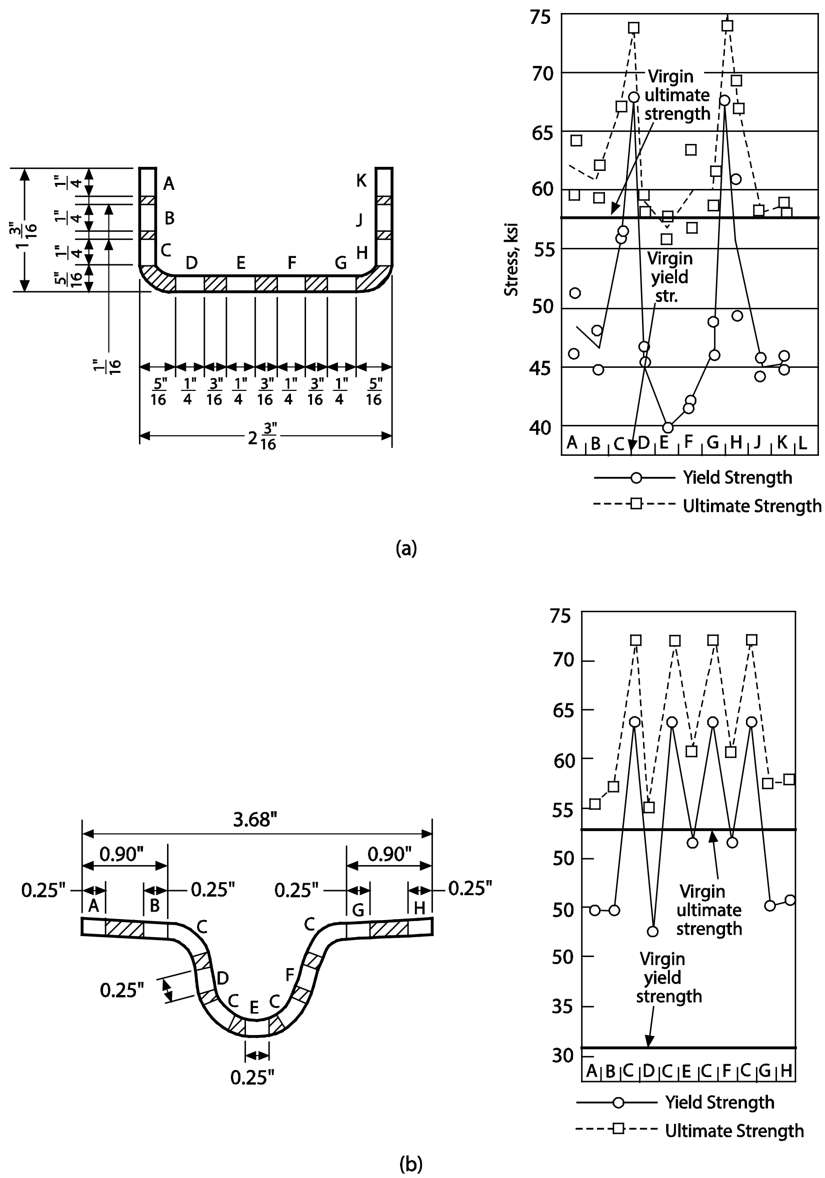


Figure C-A3.3.2-1 Effect of Cold Work on Mechanical Properties in Cold-Formed Steel Sections. (a) Channel Section, (b) Joist Chord

With regard to the full-section properties, the tensile *yield stress* of the full section may be approximated by using a weighted average as follows:

$$F_{ya} = CF_{yc} + (1 - C)F_{yf} \tag{C-A3.3.2-2}$$

where

F_{ya} = Full-section tensile *yield stress*

F_{yc} = Average tensile *yield stress* of corners = $B_c F_{yv}/(R/t)^m$

F_{yf} = Average tensile *yield stress* of flats

C = Ratio of corner area to total cross-sectional area. For flexural members having unequal *flanges*, the one giving a smaller C value is considered to be the controlling *flange*

Good agreements between the computed and the tested *stress*-strain characteristics for a channel section and a joist chord section were demonstrated by Karren and Winter (Karren and Winter, 1967).

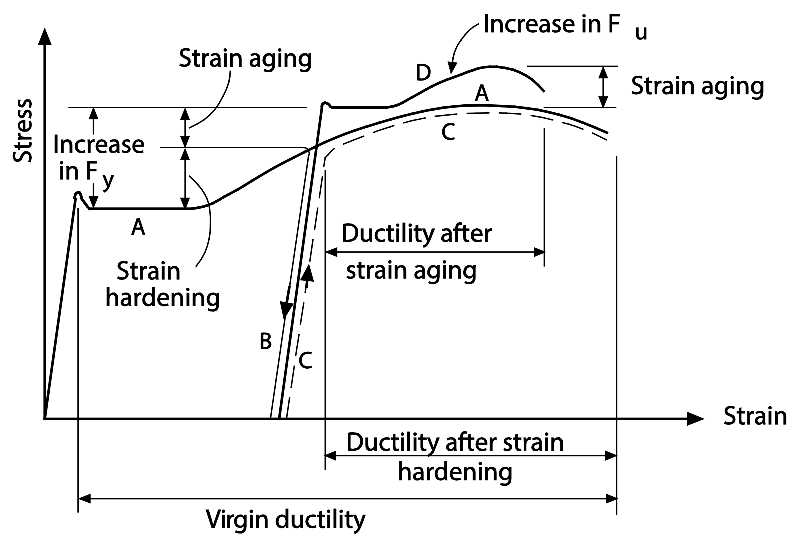


Figure C-A3.3.2-2 Effect of Strain Hardening and Strain Aging on Stress-Strain Characteristics

The limitation $F_{ya} \le F_{uv}$ places an upper bound on the average *yield stress*. The intent of the upper bound is to limit *stresses* in flat elements that may not see significant increases in *yield stress* and *tensile strength* as compared to the *virgin steel properties*.

Subsequent studies have been made by numerous investigators. These investigations dealt with the cold-formed sections having large R/t ratios and thick materials. They also considered residual *stress* distribution, simplification of design methods, and other related subjects. For details, see Yu and LaBoube (2010).

In 1962, the *Specification* permitted the utilization of cold work of forming on the basis of full section tests. Since 1968, the *Specification* has allowed the use of the increased average *yield stress* of the section, F_{ya}, to be determined by: (1) full section tensile tests, (2) stub column tests, or (3) computed in accordance with Equation C-A3.3.2-2. However, such a strength increase is limited only to relatively compact sections designed according to *Specification* Chapter D (tension members), Chapter F (bending strength excluding the use of inelastic reserve capacity), Chapter E (concentrically loaded compression members), Section H1 (combined axial *load* and bending), Section I4 (cold-formed steel light-frame construction), and Sections I6.1 and I6.2 (*purlins*, *girts* and other members). A design example in the *Cold-Formed Steel Design Manual* (AISI, 2017) demonstrates the use of strength increase from cold work of forming for a channel section to be used as a beam.

Prior to 2016, the requirements for applying the provisions of strength increase from cold work of forming were written for using the *Effective Width Method*. The requirements were revised in 2016 to make the provisions also applicable to the *Direct Strength Method*. The strength increase from cold work of forming is applicable to sections that are not subject to strength reduction from *local buckling*. This requires the cross-section to be fully effective when using the *Effective Width Method*, or $\lambda_\ell \leq 0.776$ in *Specification* Section E3.2 or F3.2 when using the *Direct Strength Method*.

In the development of the AISI *LRFD Specification,* the following statistical data on material and cross-sectional properties were developed by Rang, Galambos and Yu (1979a and 1979b) for use in the derivation of *resistance factors* ϕ:

$(F_y)_m = 1.10F_y \qquad M_m = 1.10 \qquad V_{fy} = V_M = 0.10$

$(F_{ya})_m = 1.10F_{ya} \qquad M_m = 1.10 \qquad V_{Fya} = V_M = 0.11$

$(F_u)_m = 1.10F_u \qquad M_m = 1.10 \qquad V_{Fu} = V_M = 0.08$

$F_m = 1.00 \qquad V_F = 0.05$

In the above expressions, m refers to mean value; V represents coefficient of variation; M and F are, respectively, the ratios of the actual-to-the-nominal material property and cross-sectional fabrication property; and F_y, F_{ya}, and F_u are, respectively, the *specified minimum yield stress*, the average *yield stress* including the effect of cold forming, and the specified minimum *tensile strength*.

These statistical data are based on the analysis of many samples (Rang et al., 1978), and are representative properties of materials and cross-sections used in the industrial application of cold-formed steel structures.

B. DESIGN REQUIREMENTS

B1 General Provisions

This *Specification* provides design provisions for cold-formed steel members and structural assemblies. *Specification* Section B1 provides the essential design requirements: the design of members and their *connections* should be consistent with the intended use of the structure and the assumptions made in the analysis of the structure.

B2 Loads and Load Combinations

Loads and *load combinations* should be determined in accordance with *applicable building code*. In the absence of an *applicable building code*, ASCE/SEI 7, *Minimum Design Loads for Buildings and Other Structures*, should be followed in the United States and Mexico, and the *National Building Code of Canada* (*NBCC*) should be followed in Canada.

During construction, when steel decks are used to support wet concrete, they should be designed to carry the concrete dead *load*, the steel dead *load*, and the construction live *load*. When the *ASD* or *LRFD* method is used, the construction *loads* and *load* combinations should be based on the sequential loading of concrete as specified in ANSI/SDI SD-2022 (SDI, 2022).

When the *LSD* method is used, CSSBI 12M (2024), *Standard for Composite Steel Deck*, should be followed.

B3 Design Basis

As stated in *Specification* Section B3, design should be based on the principle that no applicable strength or serviceability limit state is exceeded when the structure is subjected to *load effects* corresponding to the applicable *load* combinations.

A *limit state* is the condition at which the structural usefulness of a *load*-carrying element or member is impaired to such an extent that it becomes unsafe for the occupants of the structure, or the element no longer performs its intended function. Typical *limit states* for cold-formed steel members are excessive deflection, *yielding*, *buckling* and attainment of maximum strength after *local buckling* (i.e., post-*buckling* strength). These *limit states* have been established through experience in practice or in the laboratory, and they have been thoroughly investigated through analytical and experimental research. The background for the establishment of the *limit states* is extensively documented (Winter, 1970; Peköz, 1986b; and Yu and LaBoube, 2010), and a continuing research effort provides further improvement in understanding them.

Three design methods are provided in the *Specification* for strength: *Allowable Strength Design* (*ASD*), *Load and Resistance Factor Design* (*LRFD*), and *Limit States Design* (*LSD*). Both *Allowable Strength Design* (*ASD*) and *Load and Resistance Factor Design* (*LRFD*) are applicable only in the United States and Mexico, while the *Limit States Design* (*LSD*) is applicable in Canada. *ASD* and *LRFD* are distinct methods. They are not identical and not interchangeable. Indiscriminate use of combinations of the *ASD* and *LRFD* methods could result in unpredictable performance or unsafe design. There are, however, circumstances in which the two methods could be used in the design, modification or renovation of a structural system without conflict, such as providing modifications to a structural floor system of an older building after assessing the as-built conditions.

In the *ASD*, *LRFD* and *LSD* methods, two types of *limit states* are considered. They are: (1) the *limit state* of the strength required to resist the extreme *loads* during the intended life of the structure, and (2) the *limit state* of the ability of the structure to perform its intended function during its life. These two *limit states* are usually referred to as the *limit state* of strength and *limit state* of serviceability. The *ASD*, *LRFD* and *LSD* methods focus on the *limit state* of strength in *Specification* Sections B3.2.1, B3.2.2, and B3.2.3, respectively; and the *limit state* of serviceability in *Specification* Section B3.7.

B3.1 Required Strength [Effect Due to Factored Loads]

Generally, design is performed by elastic analysis. The *required strength* [effect due to *factored loads*] is determined by the appropriate methods of *structural analysis*. In some circumstances, as in the proportioning of stability bracing members that carry no calculated forces, the *required strength* [effect due to *factored loads*] is explicitly stated in the *Specification*.

B3.2 Design for Strength

The *Allowable Strength Design* method has been featured in AISI *Specifications* beginning with the 1946 edition. It is included in the *Specification* along with the *LRFD* and the *LSD* methods for use in the United States, Mexico, and Canada since the 2001 edition.

B3.2.1 Allowable Strength Design (ASD) Requirements

In the *Allowable Strength Design* approach, the *required strengths* (bending moments, axial forces, and shear forces) in structural members are computed by accepted methods of *structural analysis* for the specified nominal or working *loads* for all applicable load combinations determined according to *Specification* Section B2. These *required strengths* are not to exceed the *allowable strengths* permitted by the *Specification*. According to *Specification* Section B3.2.1, the *allowable strength* is determined by dividing the *nominal strength* by a *safety factor* as follows:

$$R \leq R_n/\Omega \tag{C-B3.2.1-1}$$

where

R = *Required strength*

R_n = *Nominal strength*

Ω = *Safety factor*

The fundamental nature of the *safety factor* is to compensate for uncertainties inherent in the design, fabrication, or erection of building components, as well as uncertainties in the estimation of applied *loads*. Appropriate *safety factors* are explicitly specified in various sections of the *Specification*. Through experience, it has been established that the present *safety factors* provide satisfactory design. It should be noted that the *ASD* method employs only one *safety factor* for a given condition regardless of the type of *load*. Serviceability is addressed in *Specification* Section B3.7.

B3.2.2 Load and Resistance Factor Design (LRFD) Requirements

For the *limit state* of strength, the general format of the *LRFD* method is expressed by the following equation:

$$\Sigma\gamma_i Q_i \leq \phi R_n \tag{C-B3.2.2-1}$$

or

$$R_u \leq \phi R_n$$

where

R_u = $\Sigma\gamma_i Q_i$ = *Required strength*

R_n = *Nominal resistance*

ϕ = *Resistance factor*

γ_i = *Load factors*

Qi = *Load effects*

ϕR_n = *Design strength*

The *nominal resistance* is the strength of the element or member for a given *limit state*, computed for nominal section properties and for minimum specified material properties according to the appropriate analytical model which defines the strength. The *resistance factor*, ϕ, accounts for the uncertainties and variabilities inherent in the R_n, and it is usually less than unity. The *load effects*, Q_i, are the forces on the cross-section (i.e., bending moment, axial force, or shear force) determined from the specified *nominal loads* by *structural analysis* and γ_i are the corresponding *load factors*, which account for the uncertainties and variabilities of the *loads*.

The advantages of *LRFD* are: (1) the uncertainties and the variabilities of different types of *loads* and resistances are different (e.g., dead *load* is less variable than wind *load*), and so these differences can be accounted for by use of multiple factors, and (2) by using probability theory, designs can ideally achieve a more consistent reliability. Thus, *LRFD* provides the basis for a more rational and refined design method than is possible with the *ASD* method.

(a) Probabilistic Concepts

Safety factors or *load factors* are provided against the uncertainties and variabilities which are inherent in the design process. Structural design consists of comparing nominal *load effects* Q to *nominal resistances* R, but both Q and R are random parameters (see Figure C-B3.2.2-1). A *limit state* is violated if R<Q. While the possibility of this event ever occurring is never zero, a successful design should, nevertheless, have only an acceptably small probability of exceeding the limit state. If the exact probability distributions of Q and R were known, then the probability of (R - Q) < 0 could be exactly determined for any design. In general, the distributions of Q and R are not known, and only the means, Q_m and R_m, and the standard deviations, σ_Q and σ_R, are available. Nevertheless, it is possible to determine relative reliabilities of several designs by the scheme illustrated in Figure C-B3.2.2-2. The distribution curve shown is for *ln*(R/Q), and a *limit state* is exceeded when *ln*(R/Q) ≤ 0. The area under *ln*(R/Q) ≤ 0 is the probability of violating the *limit state*. The size of this area is dependent on the distance between the origin and the mean of *ln*(R/Q). For given statistical data R_m, Q_m, σ_R and σ_Q, the area under *ln*(R/Q) ≤ 0 can be varied by changing the value of β (Figure C-B3.2.2-2), since $\beta\sigma_{ln(R/Q)} = ln(R/Q)_m$, from which approximately

$$\beta = \frac{ln(R_m / Q_m)}{\sqrt{V_R^2 + V_Q^2}} \qquad \text{(C-B3.2.2-2)}$$

where $V_R = \sigma_R/R_m$ and $V_Q = \sigma_Q/Q_m$, the coefficients of variation of R and Q, respectively. The index, β, is called the "reliability index," and it is a relative measure of the safety of the design. When two designs are compared, the one with the larger β is more reliable.

The concept of the reliability index can be used for determining the relative reliability inherent in current design, and it can be used in testing out the reliability of new design formats, as illustrated by the following example of a simply supported beam, braced against *distortional buckling* and *lateral-torsional buckling*, subjected to dead and live loading and designed considering *local buckling* using the *Effective Width Method*.

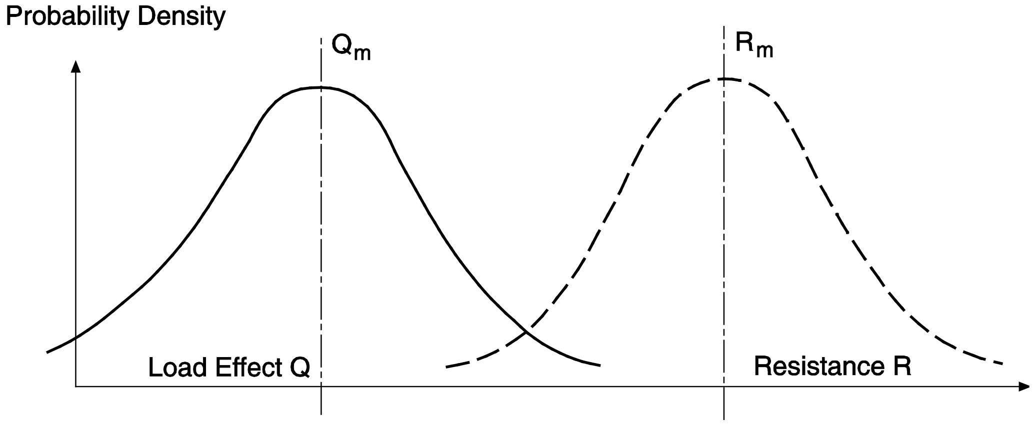


Figure C-B3.2.2-1 Definition of the Randomness Q and R

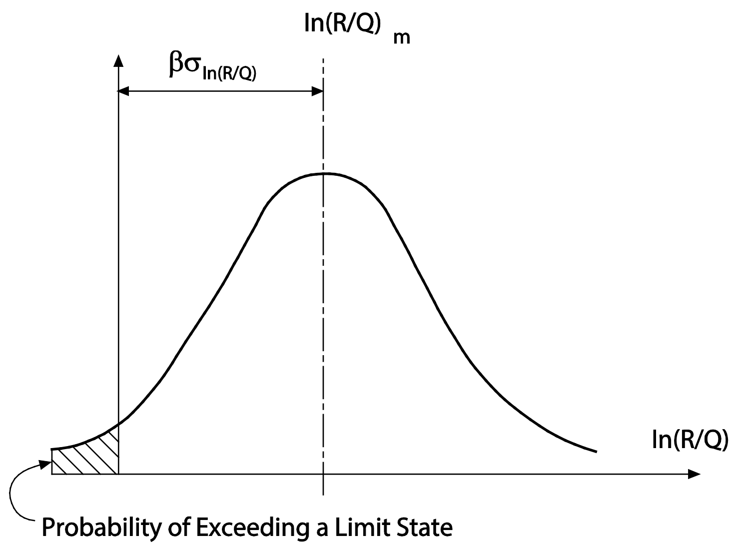


Figure C-B3.2.2-2 Definition of the Reliability Index β

The *ASD* design requirement of the *Specification* for such a beam is

$$S_eF_y/\Omega = (L_s^2 s/8)(D+L) \tag{C-B3.2.2-3}$$

where

S_e = Elastic section modulus based on the effective section

Ω = 1.67 = *Safety factor* for bending

F_y = Specified *yield stress*

L_s = Span length, and

s = Beam spacing

D and L are, respectively, the code-specified dead and live *load* intensities.

The mean *resistance* is defined as (Ravindra and Galambos, 1978):

$$R_m = R_n(P_m M_m F_m) \quad \text{(C-B3.2.2-4)}$$

In the above equation, R_n is the *nominal resistance*, which in this case is

$$R_n = S_e F_y \quad \text{(C-B3.2.2-5)}$$

that is, the nominal moment predicted on the basis of the post-*buckling* strength of the compression *flange* and the *web* using the *Effective Width Method*. The mean values P_m, M_m, and F_m, and the corresponding coefficients of variation V_P, V_M, and V_F, are the statistical parameters, which define the variability of the *resistance*:

P_m = Mean ratio of the experimentally determined moment to the predicted moment for the actual material and cross-sectional properties of the test specimens

M_m = Mean ratio of the actual *yield stress* to the minimum specified value

F_m = Mean ratio of the actual section modulus to the specified (nominal) value

The coefficient of variation of R equals

$$V_R = \sqrt{V_P^2 + V_M^2 + V_F^2} \quad \text{(C-B3.2.2-6)}$$

The values of these data were obtained from examining available tests prior to 1990 on beams having different compression *flanges* with partially and fully effective *flanges* and *webs*, and from analyzing data on *yield stress* values from tests and cross-sectional dimensions from many measurements. This information was developed from research (Hsiao, Yu, and Galambos, 1988a and 1990; Hsiao, 1989) and is given below:

$P_m = 1.11$, $V_P = 0.09$; $M_m = 1.10$, $V_M = 0.10$; $F_m = 1.0$, $V_F = 0.05$ and thus

$R_m = 1.22R_n$ and $V_R = 0.14$.

The mean *load effect* is equal to

$$Q_m = (L_s^2 s/8)(D_m + L_m) \quad \text{(C-B3.2.2-7)}$$

and

$$V_Q = \frac{\sqrt{(D_m V_D)^2 + (L_m V_L)^2}}{D_m + L_m} \quad \text{(C-B3.2.2-8)}$$

where D_m and L_m are the mean dead and live *load* intensities, respectively, and V_D and V_L are the corresponding coefficients of variation.

Load statistics have been analyzed in a study of the National Bureau of Standards (NBS) (Ellingwood et al., 1980), where it was shown that $D_m = 1.05D$, $V_D = 0.1$; $L_m = L$, $V_L = 0.25$.

The mean live *load* intensity equals the code live *load* intensity if the tributary area is small enough so that no live *load* reduction is included. Substitution of the *load* statistics into Equations C-B3.2.2-7 and C-B3.2.2-8 gives:

$$Q_m = \frac{L_s^2 s}{8}\left(\frac{1.05D}{L} + 1\right)L \quad \text{(C-B3.2.2-9)}$$

$$V_Q = \frac{\sqrt{(1.05D/L)^2 V_D^2 + V_L^2}}{(1.05D/L + 1)} \quad \text{(C-B3.2.2-10)}$$

Q_m and V_Q thus depend on the dead-to-live *load* ratio. Cold-formed steel beams typically have small D/L ratios, which may vary for different applications. Different D/L ratio may be assumed by different countries for developing design criteria. The impact of D/L ratio on the reliability is also provided in Meimand and Schafer (2014). For the purposes of checking the reliability of these *LRFD* criteria, it has been assumed that D/L = 1/5, and so

$Q_m = 1.21L(L_s^2\, s/8)$ and $V_Q = 0.21$.

From Equations C-B3.2.2-3 and C-B3.2.2-5, the *nominal resistance*, R_n, can be obtained for $D/L = 1/5$ and $\Omega = 1.67$ as follows:

$$R_n = 2L(L_s^2 s/8)$$

In order to determine the reliability index, β, from Equation C-B3.2.2-2, the R_m/Q_m ratio is required by considering $R_m = 1.22R_n$:

$$\frac{R_m}{Q_m} = \frac{1.22x2.0xL(L_s^2 s/8)}{1.21L(L_s^2 s/8)} = 2.02$$

Therefore, from Equation C-B3.2.2-2,

$$\beta = \frac{ln(2.02)}{\sqrt{0.14^2 + 0.21^2}} = 2.79$$

Of itself, β= 2.79 for beams having different compression *flanges* with partially and fully effective *flanges* and *webs* designed by the *Specification* means nothing. However, when this is compared to β for other types of cold-formed steel members, and to β for designs of various types from hot-rolled steel shapes or even for other materials, then it is possible to say that this particular cold-formed steel beam has about an average reliability (Galambos et al., 1982).

(b) Basis for LRFD of Cold-Formed Steel Structures

A great deal of work has been performed for determining the values of the reliability index, β, inherent in traditional design as exemplified by the current structural design specifications such as the ANSI/AISC 360 for hot-rolled steel, the AISI *Specification* for cold-formed steel, the ACI 318 Code for reinforced concrete members, etc. The studies for hot-rolled steel are summarized by Ravindra and Galambos (1978), where many other papers are also referenced which contain additional data. The determination of β for cold-formed steel elements or members is presented in several research reports of the University of Missouri-Rolla (Hsiao, Yu, and Galambos, 1988a; Rang, Galambos, and Yu, 1979a, 1979b, 1979c, and 1979d; Supornsilaphachai, Galambos, and Yu, 1979), where both the basic research data as well as the β's inherent in the *Specification* are presented in great detail. The β's computed in the above-referenced publications were developed with slightly different *load* statistics than those of this *Commentary,* but the essential conclusions remain the same.

The entire set of data for hot-rolled steel and cold-formed steel designs, as well as data for reinforced concrete, aluminum, laminated timber, and masonry walls, was reanalyzed by Ellingwood, Galambos, MacGregor, and Cornell (Ellingwood et al., 1980; Galambos et al., 1982; Ellingwood et al., 1982) using (a) updated *load* statistics and (b) a more advanced level of probability analysis which was able to incorporate probability distributions and to describe the true distributions more realistically. The details of this extensive reanalysis are presented by the investigators. Only the final conclusions from the analysis are summarized below.

The values of the reliability index, β, vary considerably for the different kinds of loading, the different types of construction, and the different types of members within a given material design specification. In order to achieve more consistent reliability, it was suggested by Ellingwood, et al. (1982) that the following values of β would provide this improved consistency while at the same time give, on the average, essentially the same design by the

LRFD method as is obtained by prior designs for all materials of construction. Ellingwood's recommended target reliability indices, β_o, were for members with gravity loading: $\beta_o = 3.0$, for *connections* with gravity loading: $\beta_o = 4.5$, and for wind loading: $\beta_o = 2.5$. These target reliability indices are the ones inherent in the *load factors* first recommended in the ASCE 7-98 Load Standard (ASCE, 1998).

For simply supported, braced cold-formed steel beams with stiffened *flanges*, which were designed according to the *Allowable Strength Design* method in the current *Specification* or to any previous version of the *Specification*, it was shown that for the representative dead-to-live *load* ratio of 1/5, the reliability index $\beta = 2.79$. Considering the fact that for other such *load* ratios, or for other types of members, the reliability index inherent in current cold-formed steel construction could be more or less than this value of 2.79, a somewhat lower target reliability index of $\beta_o = 2.5$ is recommended as a lower limit in the United States for members with gravity *loads*.

The *resistance factors*, ϕ, were selected such that $\beta_o = 2.5$ is essentially the lower bound of the actual β's for members supporting gravity *loads*. In order to ensure that failure of a structure is not initiated in the *connections*, a higher target reliability of $\beta_o = 3.5$ is recommended for *joints* and fasteners in the United States. These two targets of 2.5 and 3.5 for members and *connections*, respectively, are somewhat lower than those recommended by the ASCE 7-98 (i.e., 3.0 and 4.5, respectively), but they are essentially the same targets as the basis for the AISC *LRFD Specification* (AISC, 1999).

For wind loading, the same ASCE target reliability index of $\beta_o = 2.5$ is used for *connections* in the U.S. *LRFD* method. For flexural members such as individual *purlins*, *girts*, panels, and roof decks subjected to the combination of dead and wind *loads*, the target reliability index, β_o, used in the United States is reduced to 1.5. With this reduced target reliability index, the design based on the U.S. *LRFD* method is comparable to the U.S. *Allowable Strength Design* method.

(c) Resistance Factors

The following portions of this *Commentary* present the background for the *resistance factors*, ϕ, which are recommended for various members and *connections* in Chapters D through J. These ϕ factors are determined in conformance with the ASCE/SEI 7 *load factors* to provide approximately a target reliability index β_o of 2.5 for members and 3.5 for *connections*, respectively, for a typical *load* combination 1.2D+1.6L. For practical reasons, it is desirable to have relatively few different *resistance factors*, and so the actual values of β will differ from the derived targets. This means that:

$$\phi R_n = c(1.2D+1.6L) = (1.2D/L+1.6)cL \qquad \text{(C-B3.2.2-11)}$$

where c is the deterministic influence coefficient translating *load* intensities to *load effects*.

By assuming D/L = 1/5, Equations C-B3.2.2-11 and C-B3.2.2-9 can be rewritten as follows:

$$R_n = 1.84(cL/\phi) \qquad \text{(C-B3.2.2-12)}$$

$$Q_m = (1.05D/L+1)cL = 1.21cL \qquad \text{(C-B3.2.2-13)}$$

Therefore,

$$R_m/Q_m = (1.521/\phi)(R_m/R_n) \qquad \text{(C-B3.2.2-14)}$$

The ϕ factor can be computed from Equation C-B3.2.2-15 on the basis of Equations C-

B3.2.2-2, C-B3.2.2-4 and C-B3.2.2-14 (Hsiao, Yu and Galambos, 1988b, AISI 1996):

$$\phi = 1.521\ (P_m M_m F_m)\exp(-\beta_o\sqrt{V_R^2 + V_Q^2}) \qquad \text{(C-B3.2.2-15)}$$

in which β_o is the target reliability index. Other symbols were defined previously. For other *load* combinations and *load* ratios, corrected values for the 1.521 pre-factor (known as C_ϕ) and V_Q are provided in Meimand and Schafer (2014).

By knowing the ϕ factor, the corresponding *safety factor*, Ω, for *Allowable Strength Design* can be computed for the *load* combination 1.2D+1.6L as follows:

$$\Omega = (1.2D/L + 1.6)/[\phi(D/L + 1)] \qquad \text{(C-B3.2.2-16)}$$

where D/L is the dead-to-live *load* ratio for the given condition.

B3.2.3 Limit States Design (LSD) Requirements

In *Limit States Design*, the resistance of a *structural component* is checked against the various *limit states*. For the ultimate *limit states* resistance, the structural member must retain its *load*-carrying capacity up to the *factored load* levels. For *serviceability limit states*, the performance of the structure must be satisfactory at *specified load* levels. *Specified loads* are those prescribed by the *National Building Code of Canada* (*NBCC*). Examples of serviceability requirements include deflections and the possibility of vibrations.

For the *limit state* of strength, the general format of the *LSD* method is expressed by the following equation:

$$\phi R_n \geq \Sigma\gamma_i Q_i \qquad \text{(C-B3.2.3-1)}$$

or

$$\phi R_n \geq R_f$$

where

R_f = $\Sigma\gamma_i Q_i$ = Effect of *factored loads*

R_n = *Nominal resistance*

ϕ = *Resistance factor*

γ_i = *Load factors*

Q_i = *Load effects*

ϕR_n = *Factored resistance*

The *nominal resistance* is the strength of the element or member for a given *limit state*, computed for nominal section properties and for minimum specified material properties according to the appropriate analytical model which defines the *resistance*. The *factored resistance* is given by the product ϕR_n, where ϕ is the *resistance factor,* which is applied to the *nominal* member *resistance*, R_n. The *resistance factor* is intended to take into account the fact that the resistance of the member may be less than anticipated, due to variability of the material properties, dimensions, and workmanship, as well as the type of failure and uncertainty in the prediction of the resistance. The *resistance factor* does not, however, cover gross human errors. Human errors cause most structural failures and typically these human errors are "gross" errors. Gross errors are completely unpredictable and are not covered by the overall *safety factor* inherent in buildings.

The *NBCC* defines a set of *load factors*, *load* combination factors, and specified minimum *loads* to be used in the design, hence fixing the position of the nominal *load* distribution and the *factored load* distribution. The design standard is then obligated to specify the appropriate resistance function.

The *load effects*, Q_i, are the forces on the cross-section (i.e., bending moment, axial force, or shear force) determined from the specified *nominal loads* by *structural analysis*, and γ_i are the corresponding *load factors*, which account for the uncertainties and variabilities of the *loads*.

In *Limit States Design*, structural reliability is specified in terms of a safety index, β, determined through a statistical analysis of the *loads* and resistances. The safety index is directly related to the structural reliability of the design; hence, increasing β increases the reliability, and decreasing β decreases the reliability. The safety index, β, is also directly related to the *load* and *resistance factors* used in the design.

Those responsible for writing a design standard are given the *load* distribution and *load factors*, and must calibrate the *resistance factors*, ϕ, such that the safety index, β, reaches a certain target value. The technical committee responsible for CSA Group Standard S136 elected to use a target safety index of 3.0 for members and 4.0 for *connections*.

In order to determine the loading for calibration, it was assumed that 80 percent of cold-formed steel is used in panel form (e.g., roof or floor deck, wall panels, etc.) and the remaining 20 percent for structural sections (*purlins*, *girts*, studs, etc.). An effective *load factor* was arrived at by assuming live-to-dead *load* ratios and their relative frequencies of occurrence.

Probabilistic studies show that consistent probabilities of failure are determined for all live-to-dead *load* ratios when a live *load factor* of 1.50 and a dead *load factor* of 1.25 are used.

Since the design basis for the *LSD* and the *LRFD* is the same, further discussions on how to obtain a *resistance factor* using probability analysis can be obtained from Section B3.2.2(c) of the *Commentary*. However, attention should be paid to the fact that target values for members and *connections* as well as the dead-to-live *load* ratio may vary from country to country. These variations lead to differences in *resistance factors*. The dead-to-live *load* ratio used in Canada is assumed to be 1:3 (or 1/3), and the target of the reliability index for *cold-formed steel structural members* is 3.0 for members and 4.0 for *connections*. These target values are consistent with those used in other CSA Group design standards.

B3.3 Design of Structural Members

For the design of cold-formed steel axial or flexural members, consideration should be given to several design features: (a) axial or bending strength and combined axial and bending, (b) shear strength of *webs* and combined bending and shear, (c) *web crippling* strength and combined bending and *web crippling*, (d) bracing requirements, and (e) serviceability. For some cases, special consideration should also be given to shear lag and *flange* curling due to the use of thin material. The design provisions for items (a), (b) and (c) are provided in *Specification* Chapters D, E, F, G and H, and Sections I6.1, I6.2, and I6.3; while Item (d), the requirements for lateral and stability bracing, is given in *Specification* Sections C2 and I6.4; and Item (e) is covered in Chapter L. The treatments for *flange* curling and shear lag are discussed in Sections L3 and B4.3 of the *Commentary*, respectively.

Rational engineering analysis is permitted to be used if the section geometry or material properties are outside the limitations given in *Specification* Section B4.

Example problems are given in Parts II and III of the AISI *Cold-Formed Steel Design Manual* (AISI, 2017) for the design of flexural and axial members.

B3.4 Design of Connections

Specification Section B3.4 provides the charging language for Chapter J on the design of *connections*. Chapter J covers the proportioning of the individual elements of a *connection* (welds, bolts, screws, and *power-actuated fasteners*, etc.) once the *load effects* on the *connection* are known. Section B3.4 establishes that the modeling assumptions associated with the structural analysis must be consistent with the conditions used in Chapter J to proportion the connecting elements.

B3.5 Design for Stability

Design for stability needs to consider the stability of the structural system and also the stability of its individual members. Design provisions are provided in *Specification* Chapter C.

B3.6 Design of Structural Assemblies and Systems

Specification Section B3.6 provides charging language on the design of cold-formed steel assemblies and systems included in *Specification* Chapter I. Chapter I provides design provisions for cold-formed steel built-up members and metal roof and wall systems; and references design standards for *diaphragm*, light-frame construction, and rack systems.

B3.7 Design for Serviceability

Serviceability limit states are conditions under which a structure can no longer perform its intended functions. Safety and strength considerations are generally not affected by *serviceability limit states*. However, serviceability criteria are essential to ensure functional performance and economy of design.

Common conditions which may require serviceability limits are:

(a) Excessive deflections or rotations which may affect the appearance or functional use of the structure. Deflections which may cause damage to non-structural elements should be considered.
(b) Excessive vibrations which may cause occupant discomfort or equipment malfunctions.
(c) Deterioration over time, which may include corrosion or appearance considerations.

When checking serviceability, the designer should consider appropriate *service loads*, the response of the structure, and the reaction of building occupants.

Service loads that may require consideration include static *loads*, snow or rain *loads*, temperature fluctuations, and dynamic *loads* from human activities, wind-induced effects, or the operation of equipment. The *service loads* are actual *loads* that act on the structure at an arbitrary point in time. Appropriate *service loads* for checking *serviceability limit states* may only be a fraction of the *nominal loads*.

The response of the structure to *service loads* can normally be analyzed assuming linear elastic behavior. However, members that accumulate residual deformations under *service loads* may require consideration of this long-term behavior.

Serviceability limits depend on the function of the structure and on the perceptions of the observer. In contrast to the strength *limit states*, it is not possible to specify general serviceability limits that are applicable to all structures. The *Specification* does not contain explicit requirements; however, guidance is generally provided by the *applicable building code*. In the absence of specific criteria, guidelines may be found in Fisher and West (1990), Ellingwood

(1989), Murray (1991), AISC (2010a) and ATC (1999).

B3.8 Design for Ponding

Ponding refers to the retention of water due solely to the deflection of flat roof framing. The amount of accumulated water is dependent on the *stiffness* of the framing. Unbounded incremental deflections due to the incremental increase in retained water can result in the collapse of the roof. The problem becomes catastrophic when more water causes more deflection, resulting in more room for more water until the roof collapses.

The *Specification* requires that design for ponding be considered if water is impounded on the roof. Camber and deflections due to *load*s acting concurrently with rain or snow meltwater *loads* can be considered in establishing the initial conditions.

Determination of ponding stability is typically done by structural analysis where the rain *loads* are increased commensurate with incremental deflections of the framing system under the accumulated rainwater assuming the primary roof drains are blocked.

ANSI/AISC 360 Appendix 2 (AISC, 2010a) can be used for considering ponding stability, except the effective section properties as defined in the *Specification* should be used. The effective section properties should be calculated based on the *load* cases and combinations consistent with the requirements of ANSI/AISC 360 Appendix 2 (AISC, 2010a) for checking the ponding stability and the specific circumstances of the roof configuration considered.

For Canada, Commentary H of the *User's Guide - NBC 2010, Structural Commentary (Part 4 of Division B)* (NBC, 2010) can be used to determine the stiffness when ponding *instability* will occur. When calculating the stiffness, the effective section properties as defined in the *Specification* must be used.

B3.9 Design for Fatigue

Section B3.9 provides the charging language for Chapter M on the design of fatigue for *cold-formed steel structural members* and *connections*. Fatigue may occur when the structure is subjected to cyclic or repetitive *load,* which results in repetitive tensile *stresses* in the *connections* and the members. Fatigue, however, does not need to be considered for seismic *load* or wind *load* due to either infrequent *load* cycle or infrequent high *load* magnitude that would cause fatigue.

B3.10 Design for Corrosion Effects

Steel members may deteriorate in some service environments. This deterioration may appear either as external corrosion, which would be visible upon inspection, or as undetected changes that would reduce member strength. The designer should recognize these problems by either factoring a specific amount of tolerance for damage into the design, or providing adequate protection (for example, coatings or cathodic protection) or planned maintenance programs or both so that such problems do not occur.

B4 Dimensional Limits and Considerations

The *Specification* permits two methods for the basic design of members in Chapters E through H, including the *Effective Width Method* and the *Direct Strength Method*. The *Specification* indicates no preference between the two methods as either provides consistent levels of reliability even though they may not result in numerically equal answers.

B4.1 Limitations for Use of the Effective Width Method or Direct Strength Method

In 2016, the applicability limitations of the *Effective Width Method* and the *Direct Strength Method* were merged into this section and simplified. To some extent, these limitations are arbitrary; however, the provided limitations give practical limits on the applicability of the design methods and reflect serviceability limitations, limitations borne from practice, and in some cases limitations of available testing or other verification methods. In 2022, these limitations were clarified to separate shear and *web crippling* limits from *local* and *distortional buckling* limits.

The *Effective Width Method* limitations originate with the work of Winter (1970). The limits for stiffened elements in bending were updated in 1980 based on the studies conducted at the University of Missouri-Rolla in the 1970s (LaBoube and Yu, 1978a, 1978b, and 1982b; Hetrakul and Yu, 1978 and 1980; Nguyen and Yu, 1978a and 1978b) and aligned with the AISC Specification (AISC, 1989) at that time.

The *effective width* provisions of Appendix 1 provide no reductions for corners. For inside bend radius-to-*thickness* ratios (R/t) in excess of 10, this is shown to be unconservative based on the studies of Sarawit (2003), and Zeinoddini and Schafer (2010). For members with large radius-to-*thickness*, the *Direct Strength Method* may be employed, which is applicable for radius-to-thickness ratios R/t less than 20. Alternatively, the *Specification* specifically allows for *rational engineering analysis*. Using an equivalent centerline model to determine the *effective width* of the flats or appropriately reducing the plate *buckling* coefficient are examples of such rational engineering analyses.

In Zeinoddini and Schafer (2010), the following method is shown to provide a rational reduction for $10 < R/t \leq 20$. A reduced plate *buckling* coefficient, k_R, is determined by applying reduction factors based on the R/t value at each edge of the element. For unstiffened elements, only one reduction factor is applied. The plate *buckling* coefficient, k_R, which replaces k in Appendix 1, is determined as follows:

$$k_R = k\, R_{R1}\, R_{R2} \quad \text{(C-B4.1-1)}$$

where

k = Plate *buckling* coefficient determined in accordance with *Specification* Appendix 1, as applicable

$$R_{R1} = 1.08 - (R_1/t)/50 \leq 1.0 \quad \text{(C-B4.1-2)}$$

$$R_{R2} = 1.08 - (R_2/t)/50 \leq 1.0 \quad \text{(C-B4.1-3)}$$

where

R_1, R_2 = Inside bend radius. See Figure C-B4.1-1

t = *Thickness* of element. See Figure C-B4.1-1

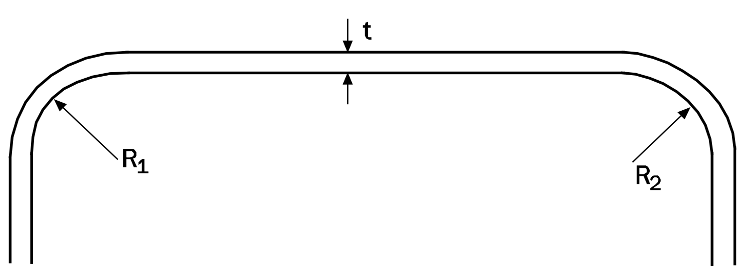


Figure C-B4.1-1 Corner Radius

Engineers are reminded that when *rational engineering analysis* methods are employed, such as presented here for r/t>10, the *safety* and *resistance factors* of Section A1.2.6(c) apply.

Prior to 2016, the *Specification* provided detailed dimensional limits for all cross-sections using the *Direct Strength Method (DSM)*. This approach was simplified and made parallel to the *Effective Width Method* limitations in 2016. The limits employed in Table B4.1-1 are based on the limits of available testing and judgment. The reliability of the *Direct Strength Method* within these limitations is detailed in Schafer (2008) and were based on testing of concentrically loaded, pin-ended cold-formed steel columns (Kwon and Hancock, 1992; Lau and Hancock, 1987; Loughlan, 1979; Miller and Peköz, 1994; Mulligan, 1983; Polyzois, et al., 1993; Thomasson, 1978); laterally braced beams (Cohen, 1987; Ellifritt, et al., 1997; LaBoube and Yu, 1978; Moreyara, 1993; Phung and Yu, 1978; Rogers, 1995; Schardt and Schrade, 1982; Schuster, 1992; Shan, et al., 1994; Willis and Wallace, 1990) and laterally braced hats and decks (Acharya and Schuster, 1998; Bernard, 1993; Desmond, 1977; Höglund, 1980; König, 1978; Papazian, et al., 1994). Application to complex lip stiffeners was verified in Schafer, et al. (2006) and application to inside bend radius-to-*thickness* ratio limits up to 20 was verified in Zeinoddini and Schafer (2010). Application of the *DSM* to sections with multiple stiffeners in the *web* for bending is given in Pham and Hancock, 2013; with multiple stiffeners in the *web* for shear in Pham and Hancock, 2012a; and with a single large intermediate stiffener in the *web* for shear in Pham and Hancock, 2015.

B4.2 Members Falling Outside the Application Limits

In general, members that are outside the applicability limits of Section B4.1 default to the general criteria in Section A1.2.6; however, the *Direct Strength Method* provides a general approach to design that is often applicable outside of the provided limits. Recognizing this, the *Specification* provides specific guidance when the *Direct Strength Method* is applied outside of Table B4.1-1. For example, companies with proprietary sections may wish to perform their own testing and follow Section K2 of the *Specification* to justify the use of the Ω and ϕ factors for a particular cross-section in *Specification* Chapters D through I. When such testing is performed, the provisions of *Specification* Sections B4.2 provide some relief from the sample size correction factor, C_P, of *Specification* Section K2. Based on the existing data, the largest observed V_P for the categories within *Specification* Table B4.1-1 is 15 percent (AISI, 2006; Schafer, 2008). Therefore, as long as the tested section, over at least three tests, exhibits a $V_P < 15$ percent, then the section is assumed to be similar to the much larger database of tested sections used to calibrate the *Direct Strength Method* and the correction for small sample sizes is not required, and, therefore, C_p is set to 1.0. If the ϕ generated from *Specification* Section K2 is higher than that of Chapters E and F, this is evidence that the section behaves as a section that satisfies *Specification* Table B4.1-1.

It is not anticipated that member testing is necessarily required for all relevant *limit states*: *local*, *distortional* and global *buckling*. An engineer may only require testing to reflect a single common condition for the member, with a minimum of three tests in that condition. However, beams and columns should be treated as separate entities. A manufacturer who cannot establish a common condition for a product may choose to perform testing in each of the *limit states* to ensure reliable performance in any condition. Engineering judgment is required. Note that for the purposes of this section, the test results in *Specification* Section K2 are replaced by test-to-predicted ratios. The prediction is that of the *Direct Strength Method* using the actual material and cross-sectional properties from the tests. The P_m parameter, taken as equal to one in *Specification* Section K2, is taken instead as the mean of the test-to-predicted ratios, and V_P is the accompanying coefficient of variation.

Users of the *Direct Strength Method* should be aware that beams within the limits of Table B4.1-1 with large *flat width-to-thickness ratios* in the compression *flange* will be conservatively predicted by the *Direct Strength Method* when compared to the *Effective Width Method* (Schafer and Peköz, 1998). However, the same beam with small longitudinal stiffeners in the compression *flange* will be well-predicted using the *Direct Strength Method*.

Alternatively, member geometries that are outside the limits of *Specification* Table B4.1-1 may still use provisions given in Chapters E and F, but with the increased Ω and reduced ϕ factors consistent with any *rational engineering analysis* method as prescribed in Section A1.2.6 of the *Specification*.

B4.3 Shear Lag Effects — Short Spans Supporting Concentrated Loads

For the beams of usual shapes, the normal *stresses* are induced in the *flanges* through shear *stresses* transferred from the *web* to the *flange*. These shear *stresses* produce shear strains in the *flange* which, for ordinary dimensions, have negligible effects. However, if *flanges* are unusually wide (relative to their length), these shear strains have the effect that the normal bending *stresses* in the *flanges* decrease with increasing distance from the *web*. This phenomenon is known as shear lag. It results in a nonuniform *stress* distribution across the width of the *flange*, similar to that in stiffened compression elements (see Section 1.1 of the *Commentary*), though for entirely different reasons. The simplest way of accounting for this *stress* variation in design is to replace the nonuniformly stressed flange of actual width, w_f, by one of reduced, *effective width* subject to uniform *stress* (Winter, 1970).

Theoretical analyses by various investigators have arrived at results which differ numerically (Roark, 1965). The provisions of *Specification* Section B4.3 are based on the analysis and supporting experimental evidence obtained by detailed *stress* measurements on 11 beams (Winter, 1940). In fact, the values of *effective widths* in *Specification* Table B4.3-1 are taken directly from Curve A of Figure 4 of Winter (1940).

It will be noted that according to *Specification* Section B4.3, the use of a reduced width for stable, wide *flanges* is required only for concentrated *load* as shown in Figure C-B4.3-1. For uniform *load*, it is seen from Curve B of the figure that the width reduction due to shear lag for any unrealistically large span-width ratios is so small as to be practically negligible.

The phenomenon of shear lag is of considerable consequence in naval architecture and aircraft design. However, in cold-formed steel construction, it is infrequent that beams are so wide as to require significant reductions according to *Specification* Section B4.3. For design purpose, see the example in the AISI *Design Manual* (AISI, 2017).

For beams designed by the *Direct Strength Method*, the shear lag check of Section B4.3 may be reasonably applied by assuming that the member strength (M_n/M_y) reduces proportional to the reduced *flange* effectiveness (b/w). For short spans under concentrated *loads*, *web crippling* (not shear lag) is typically the controlling *limit state* for members with an unstiffened *web*.

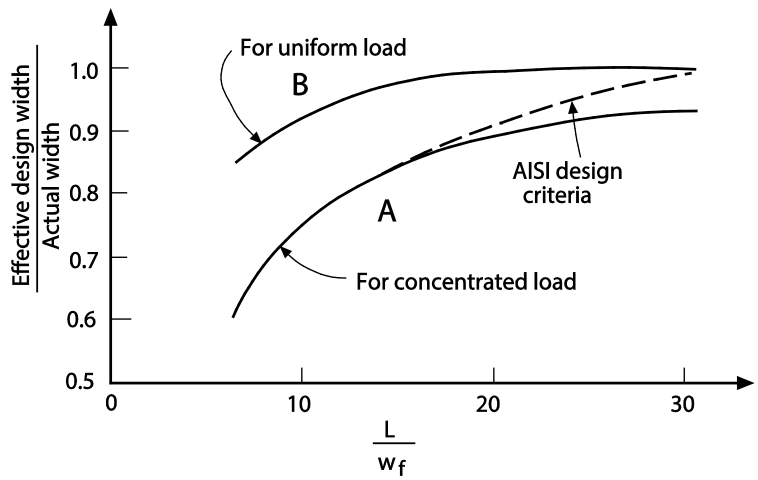


Figure C-B4.3-1 Analytical Curves for Determining Effective Width of Flange of Short Span Beams

B5 Member Properties

The geometric properties of a member (i.e., area, moment of inertia, section modulus, radius of gyration, etc.) are evaluated using conventional methods of structural design. These properties are based upon full cross-section dimensions, *effective widths,* or net section, as applicable.

Effective Width Method

For the design of tension members, both gross and net sections are employed when computing the *nominal tensile strength* [*resistance*] of the axially loaded tension members.

For flexural members and axially loaded compression members, both full and effective dimensions are used to compute cross-sectional properties. The full dimensions are used when calculating the critical *load* or moment, while the effective dimensions, evaluated at the *stress* corresponding to the critical *load* or moment, are used to calculate the *nominal strength* [*resistance*]. For serviceability consideration, the effective dimension should be determined for the compressive *stress* in the element corresponding to the *service load*. Peköz (1986a and 1986b) discussed this concept in more detail.

Section 3 of Part I of the AISI *Design Manual* (AISI, 2017) deals with the calculation of cross-sectional properties for C-sections, Z-sections, angles, hat sections, and decks.

Direct Strength Method

The *Direct Strength Method* uses the gross or net cross-section properties in member design. It considers *local buckling* through the whole cross-section and takes the interaction of the elements into consideration.

B6 Fabrication and Erection

(Reserved)

B7 Quality Control and Quality Assurance

In this edition of the *Specification*, only the delivered minimum *thickness* is addressed under this section. Other quality control and quality assurance issues may be considered in future editions.

B7.1 Delivered Minimum Thickness

Sheet and strip steels, both coated and uncoated, may be ordered to nominal or minimum *thickness*. If the steel is ordered to minimum *thickness*, all *thickness* tolerances are over (+) and nothing under (-). If the steel is ordered to nominal *thickness*, the ASTM *thickness* tolerances are divided equally between over and under. The mill tolerance for the variation between nominal and minimum thicknesses as published in the ASTM A568 Standard can range from 5 percent to 15 percent of the nominal thickness for common cold-formed sheet thicknesses. The addition of the 95 percent provision to the AISI *Specification* limited the difference between minimum delivered thickness and nominal (design) thickness to no more than 5 percent. A portion of the *safety factor* or *resistance factor* may be considered to cover the minor negative *thickness* tolerance created by this 95 percent provision.

Generally, *thickness* measurements should be made in the center of *flanges*. For decking and siding, measurements should be made as close as practical to the center of the first full flat of the section. *Thickness* measurements should not be made closer to edges than the minimum distances specified in the ASTM A568 Standard.

The responsibility of meeting this requirement for a cold-formed product is clearly that of the manufacturer of the product, not the steel producer.

B8 Evaluation of Existing Structures

In 2024, *Specification* Appendix 5 was added for the evaluation of existing structures. The corresponding commentary was provided in Appendix 5 of this document.

B9 Design for Fire Conditions

This section provides the charging language for Appendix 4 on structural design for *fire resistance*. Traditional fire design utilizes qualification testing. At the discretion of the EOR and AHJ, Appendix 4 permits (a) analysis, or (b) combination of testing and analysis for providing *fire resistance*. Design by analysis is addressed in Appendix 4, Section 4.2; and design by combination of analysis and testing is addressed in Appendix 4, Section 4.4.

C. DESIGN FOR STABILITY

C1 Design for System Stability

In previous editions of the *Specification*, concluding with its 2012 edition, the primary technique for considering system stability was the *effective length method*, mainly structured as first introduced in the 1961 AISC Specification (AISC, 1961). Characteristic for this approach was that the member strength calculation models employing an *effective length factor*, K, were relied upon for considering the effects of residual *stresses* and geometric imperfections. Consideration of various sources of deformation, such as those at *connections* and those resulting from member shear, were not previously prescribed and the manner in which they were captured was largely dependent on the standard practice of various constituent industries and the judgment of individual professionals. The structure of the *Specification* implied the usage of *first-order* elastic *analysis* of a geometrically undisturbed structure, where the *second-order effects* were crudely captured through the approximate amplifiers embedded in the interaction equations. In 2006, based on the study by Sarawit and Peköz (2006) and the similar methodology in ANSI/AISC 360-05 (AISC, 2005), Appendix 2 of the *Specification* (AISI, 2007a and 2012a) incorporating a *notional load* approach was added. Supplied as an alternative to the *effective length method*, the *notional load* approach required that the member and system *second-order effects* be considered directly through an elastic analysis capable of establishing equilibrium on a deformed structure. In this analysis, initial imperfections were captured through the application of notional forces while *stiffnesses* used in such an analysis were reduced to model the effect of section softening due to inelastic deformations, including residual *stresses*, and to account for the strength reduction factor applied to column strength. For further background on AISI S100-12 (AISI, 2012a), the user is referred to the *Commentary* to AISI S100-12 (AISI, 2012b).

Similar to ANSI/AISC 360-10 (AISC, 2010a), recognizing the interrelated roles of analysis and member proportioning in assessing and assuring the overall system stability, the *Specification* introduced the concept of "method of design." Therein, the term "design" refers to the comprehensive process of determining the *required* and *available* member *strength* [effects due to *factored loads* and member *factored resistance*], thus incorporating analysis, definition of imperfections, identifying sources of deformation, and determining the member strength. As described above, it is possible to capture many such effects either through determination of required forces (analysis) or through determination of *available strength* [*factored resistance*] (member proportioning). It is, therefore, crucial that the processes of determining the *required* and *available* member strengths [effects of *factored loads* and *factored resistance*] within any particular method of design are compatible.

In 2016, the *Specification* was reorganized whereby the interaction equations were decoupled from the analysis requirements and specific effects affecting system *stability*. In addition, the *Specification* relaxed the requirement that the bending moment ($\overline{M}$) should be defined with respect to the centroidal axis of the effective section. For ideally pin-ended beam-columns, when determining applied bending from a compressive force, eccentricity from the line of compressive action may be increased if the effective centroid (accounting for *local buckling*) is considered. However, for continuous members or members with end restraint or members with support restrained in a manner that reduces the neutral axis eccentricity between gross and effective sections, this phenomenon is minor as the line of action of the force moves with the *buckling* deformations due to the continuity of the structure, and calculation of the *required bending moment* [moment due to *factored loads*] about the gross centroidal axes is appropriate.

The *Specification* permits the usage of any method of design capable of assessing the *stability* of both the system and each of its individual members, provided it considers items (a) through (f) from *Specification* Section C1. The *Specification* offers three such design methods, subject to the limitations stipulated within each of the methods. However, it is not the intention of the *Specification* to prefer any of the methods of design enclosed therein, including approaches incorporating inelastic analyses, or to prevent the usage of any methods of design not stated therein, provided such a method considers the above items.

The *load*-displacement response resulting from a *second-order* elastic *analysis* is nonlinear. For this reason, and to assure that consistent reliability can be achieved through deployment of *LRFD*, *LSD*, or *ASD*, all *load*-dependent effects must be determined using either *LRFD* or *LSD load combinations* or 1.6 times the *ASD load combinations*. Subsequently, if *ASD* is used in the design, such effect should be divided by 1.6 to arrive at required member forces. Consequently, application of *ASD* in this regard may be conservative in systems for which the live-to-dead *load* ratio is relatively low.

Unbraced length, as used in *Specification* Section C1, is considered to occur between distinct bracing points possessing adequate strength and *stiffness* to restrain their translation and/or rotation, as applicable. Methods of satisfying the bracing requirement are provided in *Specification* Section C2. The requirements of *Specification* Section C2 are not applicable to bracing that is included in the analysis of the overall structure as part of the overall force-resisting system.

Stiffness modification requirements of *Specification* Section C1.1 and C1.2 are intended only for the strength and *stability* checks under factored *load* combinations, as prescribed in those sections. An analysis utilizing such *stiffnesses* may not be suitable for many displacement-related design considerations. Unreduced (nominal) stiffness is considered appropriate for considering serviceability, such as deflection, drift and vibrations, or for calculating many other *stiffness*-based properties or design checks, including period, seismic drift, and seismic *stability* factor.

The design of structures for stability often requires an analysis. Specifications worldwide are providing greater guidance on performing such analyses. Due to differences in terminology around the world, it can be challenging for the engineer to quickly assess analysis requirements in design and analysis capabilities in software. To aid the engineer, a series of analysis definitions that are intended to be reasonably comprehensive with respect to current terminology have been developed and are provided in this section. The terms are being coordinated between AISI and AISC and are intended to be used in future provisions. The definitions are ordered by level of complexity. In addition, explanatory (parenthetical) information follows each definition.

Structural Analysis. Determination of *load effects* on members and *connections* based on principles of structural mechanics, typically using a model. (*Structural Analysis* is the means by which ASCE/SEI 7 or other applicable *loads* are turned into demands on the members, *connections*, or structure as a whole, i.e., *load effects*.)

Frame Structural Analysis. A common form of *structural analysis* that employs one-dimensional framework elements in its structural model. (Framework elements are known as beam or line elements. These elements do not include cross-section distortion or cross-section imperfections. Many line elements do not include the effects of cross-section asymmetry or warping torsion, both potentially important issues for many cold-formed steel members in a *frame structural analysis*.)

Linear/First-order Analysis (LA). *Structural analysis* in which equilibrium conditions are formulated on the undeformed structure. (*Linear*, or *first-order* (elastic) *analysis* is the only form of analysis for which superposition applies, making it popular for handling many different load cases, but it is the most limited in terms of its predictive capability.)

Linear Buckling Analysis (LBA). *Structural analysis* to determine the *load* at which the equilibrium of the structure and/or member, approximated with linear elastic material, is neutral between two alternative states: buckled and straight. (Also known as elastic critical load or eigen-*buckling* analysis, linear buckling analysis may refer to *buckling* of the frame or the member cross-section; typically referred to as frame *buckling* analysis and cross-section *buckling* analysis respectively.)

Geometric Nonlinear (Second-order) Analysis (GNA). *Structural analysis* in which equilibrium conditions are formulated on the deformed structure, and the material remains elastic. (*Geometric nonlinear/second-order analysis* includes, but is not limited to, such effects as the additional moments developed due to deformations in the structure/frame (P-Δ), member (P-δ), and cross-section (P-δ_c). Equilibrium in the deformed structure is not limited to *P-delta effects*; other common effects such as the redistribution of moments between the major and minor axis as a member twists also commonly occur. Approximate geometric nonlinear analysis methods using simple amplification expressions such as B_1 and B_2 of Section C1.2 have long been used in codes and standards such as this *Specification*, and are appropriate when used with care.)

Geometric Nonlinear (Second-order) Analysis with Imperfections (GNIA). *Structural analysis* in which equilibrium conditions are formulated on the deformed structure, the material remains elastic, and geometric imperfections are included. (This form of analysis is the basis for the *Direct Analysis Method* of Chapter C of this *Specification*. When imperfections for the structure/framework (Δ) and along the member (δ) are both considered, this form of analysis is also known as Direct Modeling of Member Imperfections, and AISC 360 Appendix 1 provides guidance on application. Cross-section imperfections (δ_c) may be necessary to adequately capture the behavior of cold-formed steel members; however, this may require sophisticated analysis tools such as shell finite elements. The magnitude and distribution of imperfections can have a pronounced influence on the results in many structures. The phrase "geometric imperfections are included" does not require that all geometric imperfections be explicitly modeled, rather that their effect be included. In some cases this is efficiently performed through the use of notional loads.)

Material Nonlinear/Plastic Analysis (MNA). *Structural analysis* in which equilibrium conditions are formulated on the undeformed structure, but considering non-linear and/or inelastic materials. (Also known as first-order inelastic analysis, plastic hinge analysis, or plastic design, material nonlinear analysis is a method long used for understanding the limits of redistribution of moments and finding collapse capacities in frameworks.)

Nonlinear Buckling Analysis (NBA). *Structural analysis* to determine the incremental *load*, beyond a known stable equilibrium, in which the equilibrium of the structure and/or member is neutral between two alternative states. (Also known as an iterative *buckling* analysis, inelastic critical load, or inelastic eigen-*buckling* analysis. This form of nonlinear *buckling* analysis can be particularly useful for analyzing bracing of members in the inelastic range.)

Geometric and Material Nonlinear (Second-order) Inelastic Analysis with Imperfections (GMNIA). *Structural analysis* in which equilibrium conditions are formulated on the deformed structure, the potential for non-linearity and/or inelasticity of the material is included, and imperfections, both geometric and material, are included. (Also known as (nonlinear) collapse analysis or advanced analysis, this method is generally regarded as the highest level of analysis. Engineers still must recognize the limitations of their analysis choice; for example, *frame structural analysis* requires special considerations if employed with cold-formed steel members. Cross-section imperfections (δ_c) and residual *stresses* and strains may be necessary to adequately capture the behavior of members; however, this may require sophisticated analysis tools such as shell finite elements. GMNIA with shell finite elements, for collapse analysis prediction of CFS assemblies, is commonly used in research. Analysis results can be sensitive to the boundary conditions, element formulation, solver, as well as material and imperfection modeling choices, which should be understood by the engineer prior to applying GMNIA results in design.)

C1.1 Direct Analysis Method Using Rigorous Second-Order Elastic Analysis

The provisions of this section are based on Sarawit (2003), Sarawit and Peköz (2006), and ANSI/AISC 360-10 (AISC, 2010a). This method of design effectively incorporates the *notional load* approach, previously included in Appendix 2 of AISI S100-12. The study by Sarawit and Peköz on industrial steel storage racks at Cornell University (Sarawit, 2003) was sponsored by the Rack Manufacturers Institute and the American Iron and Steel Institute. The subject of *notional loads* is discussed fully in the Commentary to Chapter C of ANSI/AISC 360-10 (2010a). The application of the *direct analysis method* to cold-formed steel structures has to consider the items (a) through (f) listed in *Specification* Section C1, including frequently encountered *flexural-torsional buckling*, semi-rigid *joints* and local *instabilities*. In Sarawit (2003), and Sarawit and Peköz (2006), it was shown that the *direct analysis method* gives more accurate results than the *effective length method*.

Required strengths [effects due to *factored loads*] are determined by analysis according to *Specification* Section C1.1.1 and the members have to satisfy the provisions of Section H1 of the *Specification*. The work by Sarawit and Peköz is based on a linear moment-axial interaction equation, as depicted in *Specification* Section H1.2. It is the position of the committee that such a model adequately captures the interaction of a wide variety of cold-formed steel shapes subject to different forms of *buckling*, axes of bending and *buckling* modes for the design methods proposed herein, including the *direct analysis method*. Further background on interaction equations is provided in the commentary to *Specification* Section H1.2.

Since the frame *stability* is considered by the *direct analysis method*, *nominal axial strength* [*resistance*] in *Specification* Chapters D and E should be determined considering the *flexural buckling effective length* equal to the *unbraced length* (i.e., $K_x = K_y = 1.0$). It is important to recognize that the application of the *direct analysis method* does not alter the torsional *effective length factor*, K_t, which could be larger or smaller than 1.0, depending on the member boundary conditions. As an example, one can consider the case of a C-section cantilevered column with torsional and flexural fixity at the base. If designed using the *direct analysis method*, the calculations of *available strength* [*factored resistance*] would be based on $K_x = K_y = 1.0$ when computing *flexural buckling stresses* as prescribed by Chapter E. However, a $K_t = 2.0$ would be used in computing the *flexural-torsional buckling stress*.

Any type of *second-order* elastic *analysis* capable of establishing static equilibrium on the displaced structure is permitted. Two examples of such analyses are the stability functions approach and the geometric stiffness approach. The latter is typically implemented in commercially available software. It is required to carry out a *second-order analysis* that considers both the effect of *loads* acting on the deflected shape of a member between *joints* or nodes (*P-δ effects*) and the effect of *loads* acting on the displaced location of *joints* or nodes in a structure (*P-Δ effects*). On a member level, *P-δ effects* need to be modeled explicitly. One possible method of accomplishing this is to employ an elastic analysis capable of capturing only *P-Δ effects* whereby *P-δ effects* are accounted for by modeling individual columns as a serious of short column segments separated by intermediate nodes. These intermediate nodes do not need to account for the initial out-of-straightness for the member. This is because for members, the design equations based on *flexural buckling* column curves include the presence of the initial imperfections along the member length.

As an alternative to an elastic method of analysis capable of capturing both *P-Δ* and *P-δ effects*, users are permitted to employ a mixed approach, whereby *P-Δ effects* are captured explicitly in the analysis with the results of such an analysis subsequently amplified by the coefficient B_1, as defined in *Specification* Section C1.2. This method of analysis is typical of commercially available analysis software commonly used in practice. Relatively small conservativism occurs in moment frame systems due to the application of B_1 to both sway and non-sway components of the calculated moment.

Second-order frame *analysis* within the *direct analysis method* of design is permitted either on the out-of-plumb geometry without *notional loads* or on the plumb geometry by applying *notional loads* or minimum lateral *loads* as defined in *Specification* Section C1.1.1.2. Initial displacements similar in configuration to both displacements due to loading and anticipated *buckling* modes should be considered in the modeling of imperfections. The imperfections required to be 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 prescribed by the standard practice governing the member fabrication. The magnitude of the initial displacements should be based on permissible construction tolerances, such as those specified in the AISI S202, *Code of Standard Practice for Cold-Formed Steel Structural Framing* (AISI, 2011), AISC 303, *Code of Standard Practice* (AISC, 2010c), other governing requirements, as applicable, or on actual imperfections, if known. 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. The *notional loads* can also lead to additional overturning effects, which are not fictitious. An out-of-plumbness of 1/240, based on the Rack Manufacturer's Institute Specification, RMI MH16.1:2008 (RMI, 2008), is selected as appropriate or conservative for use in a wide variety of cold-formed steel structures. An out-of-plumbness of 1/500, representing the maximum tolerance on column plumbness, is specified in the AISC 303. The usage of values smaller than 1/240 is permitted provided such values are substantiated by the applicable quality assurance standard or project-specific requirements. Various codes, such as EN 1993-1 (ECS, 2005), provide criteria and methods of computing the initial imperfection as a function of the number of stories and the number of participating columns in the resistance plane. 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. The *notional load* concept is applicable to all types of structures, but the specific requirements (1) through (3) given in *Specification* C1.1.1.2(b) are applicable only for the particular class of structure identified therein.

If *second-order* elastic *analysis* is used, whereby the effects of inelasticity and uncertainty are not explicitly included in the analysis, all *stiffnesses* maintaining the *stability* of the system are to be reduced as specified in *Specification* Section C1.1.1.3. In the application of the *direct analysis method* in ANSI/AISC 360-10 (AISC, 2010a), this results in a stiffness reduction factor of 0.8 for slender columns, capable of resisting factored axial loads of up to $0.5P_y$. The factor of 0.8 is equivalent to the margin of safety implied by a strength reduction factor 0.9, prescribed in the ANSI/AISC 360-10 Chapter E, multiplied by the elastic *flexural buckling* column curve adjustment coefficient of 0.877. In the development of the *direct analysis method*, as implemented in ANSI/AISC 360-10, a distributed plasticity analysis was used. It can be shown, using distributed plasticity analysis, that using a factored elastic modulus, E, and *yield stress*, F_y, in the analysis will yield the same P-M interaction curve as if nominal values of E and F_y were used in the analysis and subsequently the abscissa and the ordinate of the P-M interaction curve are factored (White et al., 2006). However, a ten percent (10%) reduction in member *stiffness* EI, namely multiplying EI by 0.9, is substantiated by Sarawit and Peköz for the cold-formed steel members whose required axial force does not exceed $0.5P_y$. Specifically, Sarawit and Peköz (2006) showed that for typical industrial storage rack frames with a wide variety of section properties, configurations, and behavior modes, a reduction of 10 percent in member *stiffnesses* results in an increased conservatism of 10 percent in the calculated *load*-carrying capacity. A 20 percent (20%) reduction in member *stiffnesses* would lead to an increased conservatism of 20 percent (20%) in the calculated *load*-carrying capacity. However, a parametric study of individual columns in Sarawit and Peköz (2006) shows that some unconservative results can be obtained in a few instances if the *stiffness* of members is not reduced in the analysis. Reducing the stiffness by 10 percent gives satisfactory results for these cases.

The study by Sarawit and Peköz did not incorporate the members with sections subjected to the required axial force in excess of $0.5P_y$. It is furthermore not expected that such sections would be commonly found in cold-formed steel framing applications. However, to address the full scope of the *Specification*, the committee's position is that such occurrences could be adequately addressed by applying the ANSI/AISC 360-10 (2010a) *stiffness* modifier, τ_b, which in addition to 0.9, has the role of capturing additional *stiffness* softness characteristic for stockier columns with the axial force approaching P_y. This conclusion is further supported by the study by Ziemian and Kissell (2010) on aluminum members which noted a limited impact of τ_b even for fairly stocky columns. In addition, the study findings suggest the ability to use a higher value of the reduced stiffness given the usage of a linear interaction diagram compared to a multi-linear bulged-forward interaction diagram in ANSI/AISC 360-10 (2010a).

Initial imperfections, as considered in this design method when performing an analysis, refer to the imperfections at the points of member intersections (i.e., column out-of-plumbness). Column out-of-straightness, referring to the initial imperfection occurring between the points of member intersections, is in turn not considered in the analysis, but its influence is considered in the column strength curves when computing the *available strength* [*factored resistance*] per Chapter E. In certain cases, the user may experience difficulty in determining the *effective* member *length* (i.e., $KL_x=L_x$ and $KL_y=L_y$) for use in computations of *available strength* per Chapter E. An example of such a difficulty would be the exercise of determining the *effective* *length* of a gable long-span portal frame rafter. In such a case, the user may directly model initial imperfections along the length of the member and in exchange determine the *available strength* [*factored resistance*] based on the strength of the member section (i.e, L_x and L_y of zero). The initial displacements should be considered in the direction in which the effective member length KL=L is taken as zero. Similarly, even when member length is less ambiguous, such as in the cases of typical floor columns with clearly defined points of member intersection, it is permitted to explicitly include the column out-of-straightness in the analysis, and in exchange determining the *available* axial *strength* [*factored resistance*] considering *local* and *distortional buckling* only. Consideration of *torsional* and *flexural-torsional buckling* would be unaffected by this option.

It should be noted that the *nominal axial* and *flexural strengths* [*resistances*] computed per Chapters D, E, F, G, H, I, K and M are not intended to be calculated using the reduced value of *stiffness*.

C1.2 Direct Analysis Method Using Amplified First-Order Elastic Analysis

The design method presented in this section is identical to that offered in *Specification* Section C1.1, except that it is permitted to perform the design using an amplified *first-order* elastic *analysis*. With this approach, the non-sway and sway components of the member moments resulting from a *first-order* elastic *analysis* are amplified by factors B_1 and B_2, respectively. Additionally, sway moment amplification will translate into additional axial forces in the system columns and thereby amplification of the column forces resulting from sway effects by the factor B_2 is required as well. The amplified *first-order* elastic *analysis* method, as configured here, was first introduced in the 1986 AISC *LRFD* Specification (AISC, 1986), and has also been used historically in some form in other major design specifications, such as ACI 318-14 (ACI, 2014). Both B_1 and B_2 represent nonlinear algebraic convergence functions relating the member forces from the undeformed structure equilibrium to the forces in a displaced member and structure, respectively. Consequently, diverging values of these functions will indicate *instability*. Similarly, a large value of B_2 is associated with a *stability* critical system. The reader will recognize the mathematical similarity of the amplifier included in the P-M interaction equation in AISI S100-12 (AISI, 2012a) with B_1 and B_2. The advantage of B_1 and B_2 over the amplifier previously used in the *Specification* is their ability to distinguish between the non-sway (member) and sway (story) effects and consequently capture the appropriate level of amplification associated with each effect. Consequently, it avoids potentially grossly conservative or unconservative results resulting from the application of a single amplifier previously contained in the *Specification* interaction equations. As B_1 and B_2 are stiffness-based terms representing an integral part of the analysis process, their application within the framework of *Direct Analysis Method* of design must be associated with the same *stiffness* reductions as mandated by the method for use in the *first-order* elastic *analysis*, with the exception that B_1 must be computed with a reduced *stiffness* even for members not contributing to the system *stability*. The evaluation of B_2 as configured in these provisions is based upon the story drift approach, rather than column *buckling* analogy. As a result, this analysis can be employed with any method of design, without the need to determine the *effective length factor* associated with story sway. $\overline{F}$ and Δ_F in *Specification* Equation C1.2.1.1-7 may be based on any lateral loading that provides a representative value of story lateral *stiffness*, $\overline{F}/\Delta_F$. The derivation of B_1 and B_2 was presented in many references, including Chen and Lui (1991).

The user should maintain awareness of the fact that the amplification of first-order load-effects are actual system effects in the form of additional member, *connection*, bracing, foundation and anchorage forces. These additional forces should be accounted for in design. As a result of logistical convenience within the commercially available design-analysis software packages, when amplified *first-order analysis* is used, B_1 and B_2 are typically applied at the member proportioning stage, thus excluding such forces from the result of the analysis. Consequently, care should be taken to incorporate the effect of the amplifiers in the proportioning of other elements of the system, such as those listed above.

Further background on this method of analysis is presented in the Commentary to the ANSI/AISC 360-10 (2010b).

C1.3 Effective Length Method

The design method presented in this *Specification* section represents the traditional method of design, first introduced in the 1961 AISC Specification (AISC, 1961). Recognizing its traditional association with the amplified *first-order* elastic *analysis* in the 1986, 1993 and 1999 AISC Specifications, it is presented in such a form herein, though it is not the intention of the committee to limit the usages of other methods of analysis compatible with the *effective length method* framework as long as the items (a) through (f) listed in *Specification* Section C1 are considered. Notwithstanding the different formulation of *second-order effect* amplifiers, the *effective length method* historically constituted the primary approach of design for *stability* up until and including the 2012 edition of the *Specification* (AISI, 2012a), and the sole such approach before the 2007 edition of the *Specification* (AISI 2007a) which introduced the *notional load* approach (*direct analysis method*) in its Appendix 2.

Unlike the methods of design set forth in *Specification* Sections C1.1 and C1.2, the *effective length method* relies on the calculations of *available strength* [*factored resistance*] through the application of *effective length* (typically larger than the actual *unbraced* member *length*) and the empirical column curves, incorporating a modified elastic and inelastic *buckling* range, to capture the effects of geometric imperfections and loss of *stiffness* due to residual *stresses*, local *yielding* as the capacity is approached, as well as other effects. As a result of this, the analysis, performed using nominal *stiffnesses*, need only capture the *P-Δ* and *P-δ effects*. Also, given the application of the *effective length factor* in member proportioning, notional forces are not required to safely configure a column solely on the basis of the axial forces and in-plane bending. Unfortunately, the application of the *effective length factor* for *flexural buckling* does not impart the forces resulting from initial imperfections into beams, framing *connections*, *stability* braces, foundations and base anchorage, which is particularly critical in designs controlled by gravity *load* combinations. For this reason, the *Specification* stipulates the application of notional forces, as described in *Specification* Section C1.1 in conjunction with all gravity *load* combinations.

In the design, many systems can be classified as sway and non-sway. For the former, *effective length factor*, K_x or K_y, as applicable, will be larger than 1.0; and for the latter, K_x or K_y, as applicable, can typically be taken as 1.0 or less, depending upon specific boundary conditions.

The calculation of the *effective length factor*, K, for *flexural buckling* depends upon the axis of bending, frame configuration, boundary conditions, and the *stiffness* properties of the column and the members attached thereto. For further information on various methods of computing K, the user is referred to Chen and Lui (1991), AISC Specification (2010a) and ASCE Task

Committee on Effective Length Method (1997).

When B_2 exceeds 1.5, this method of design is not permitted. Specifically, research found that the method considerably underestimates the internal system forces when B_2 exceeds 1.5, where B_2 is evaluated on the basis of unreduced (nominal) *stiffness* (White et al., 2006).

C2 Member Bracing

The provisions of this section cover the design of torsion (also known as primary, first-order, or load-resisting) bracing in Section C2.2 and *stability* (also known as secondary, second-order, or deformation-resisting) bracing in Sections C2.1 and C2.3.

Torsion bracing develops forces even when equilibrium in the undeformed shape is considered. For example, bracing designed to resist twist in a C-section loaded in the plane of the *web* develops forces due to the location of the shear center not coinciding with the *web*, and is considered torsion, or first-order, bracing. Also, bracing designed to resist twist in a Z-section will develop forces when it is desired to have loading and response occur in a geometric axis that does not coincide with a cross-section principal axis, and is also considered torsion, or first-order, bracing. First-order or torsion braces, traditionally, are designed with strength criteria alone. The forces that develop to directly resist the first-order demands in torsion braces scale directly with the applied *loads* and can be significant. The relatively large magnitude of the brace forces and the fact that they may be predicted independently of brace *stiffness* makes their design criteria slightly simpler than *stability* bracing, as explained below. A *First-Order Analysis* that includes cross-section torsion can provide a means to predict bracing forces for torsion braces.

Stability bracing, on the other hand, is used to prevent a member from *buckling*. *Stability* bracing receives forces only if equilibrium in the deformed (buckled) shape requires forces in the braces. For example, bracing designed to resist minor-axis *flexural buckling* in a C-section is considered a *stability*, or second-order, bracing. If *stability* braces are stiff enough, they only develop very small forces. As a result, *stability* braces are typically designed with both *stiffness* and strength criteria. A *second-order analysis* that includes the potential *buckling* deformations the brace is intended to restrict along with appropriate imperfections can provide a means to predict bracing forces for *stability* braces as detailed further in Section C2.3.

Cases where a brace may need to act as both a torsion and a *stability* brace are possible. In such cases, the strength demands for torsion braces generally exceed those for *stability* bra ☜B traditionally have been employed without additional consideration for behavior as a *stability* brace. For other cases beyond the scope of the *Specification*, brace forces predicted from a proper *second-order analysis* that can capture both torsion and *stability* brace demands are recommended.

C2.1 Symmetrical Beams and Columns

There are no simple, generally accepted techniques for determining the *required strength* [effect due to *factored loads*] and *stiffness* for discrete braces in steel construction. Winter (1960) offered a partial solution and others have extended this knowledge (Haussler, 1964; Haussler and Pahers, 1973; Lutz and Fisher, 1985; Salmon and Johnson, 1990; Yura, 1993; SSRC, 1993). The design engineer is encouraged to seek out the stated references to obtain guidance for design of a brace or brace system.

☜B

C2.2 Bracing of Beams

C-sections and Z-sections used as beams to support transverse *loads* applied in the plane of the *web* may twist and deflect laterally unless adequate lateral supports are provided. The force and moments in a brace developing from *lateral-torsional buckling* deformation can be calculated with a *second-order analysis* as specified in *Specification* Section C1 by considering the brace member as part of the structural system. Section C2.2 of the *Specification* includes the requirements for spacing and design of braces when neither *flange* of the beam is braced by deck or sheathing material. The bracing requirements for members having one *flange* connected to deck or sheathing materials are provided in *Specification* Section I6.4.1. A curtainwall bracing design example is available in "AISI RP18-2: Design Example for Analytical Modeling of a Curtainwall and Considering the Effects of Bridging (All-Steel Design Approach)" (AISI, 2018). AISI D110-16 (AISI, 2016) provides an example of how to check intermediate wall stud bridging ultimate strength for torsional demand moments. ☜B

C2.2.1 Neither Flange Connected to Sheathing That Contributes to the Strength and Stability of the Section

(a) Bracing of C-Section Beams

If C-sections are used singly as beams, rather than being paired to form I-sections, they should be braced at intervals so as to prevent them from rotating in the manner indicated in Figure C-C2.2.1-1. Figure C-C2.2.1-2, for simplicity, shows two C-sections braced at intervals against each other. The situation is evidently much the same as in the composite I-section of Figure C-I1.1-2, except that the role of the connectors is now played by the braces. The difference is that the two C-sections are not in contact, and that the spacing of braces is generally considerably larger than the connector spacing. In consequence, each C-section may actually rotate very slightly between braces, and this will cause some additional *stresses*, which superimpose on the usual, simple bending *stresses*. Bracing should be so arranged that: (1) these additional *stresses* are small enough not to reduce the *load*-carrying capacity of the C-section (as compared to what it would be in the continuously braced condition), and (2) rotations should be kept small enough to be unobjectionable on the order of one to two degrees.

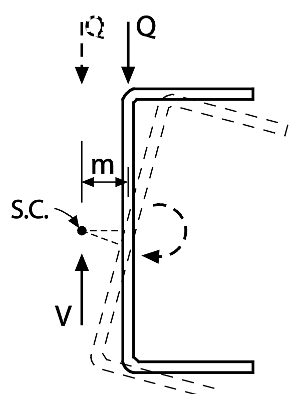


Figure C-C2.2.1-1 Rotation of C-Section Beams

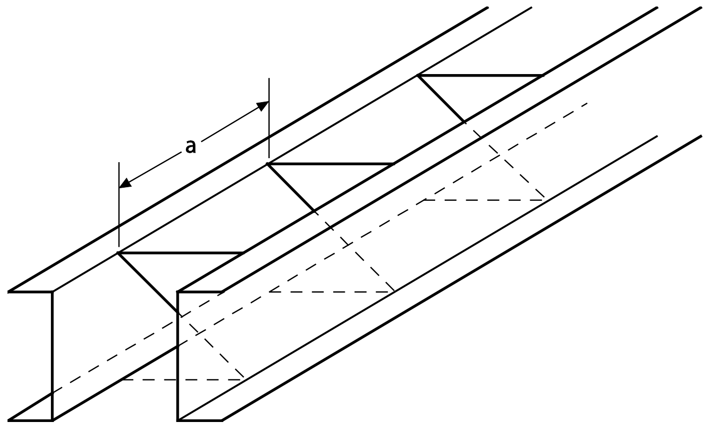


Figure C-C2.2.1-2 Two C-Sections Braced at Intervals Against Each Other

In order to obtain the information for developing bracing provisions, different C-section shapes were tested at Cornell University (Winter, 1970). Each of these was tested with full, continuous bracing; without any bracing; and with intermediate bracing at two different spacings. In addition to this experimental work, an approximate method of analysis was developed and checked against the test results. A condensed account of this work was given by Winter, Lansing and McCalley (1949b). It is indicated in the reference that the above requirements are satisfied for most distributions of beam *load* if between supports not less than three equidistant braces are placed (i.e., at quarter-points of the span, or closer). The exception is the case where a large part of the total *load* of the beam is concentrated over a short portion of the span; in this case, an additional brace should be placed at such a *load*. Correspondingly, previous editions of the AISI *Specification* (AISI, 1986; AISI, 1991) provided that the distance between braces should not be greater than one-quarter of the span and defined the conditions under which an additional brace should be placed at a *load* concentration.

For such braces to be effective, it is necessary that their spacing be appropriately limited and their strength should suffice to provide the force required to prevent the C-section from rotating. It is also necessary to determine the forces that will act in braces, such as those forces shown in Figure C-C2.2.1-3. These forces are found if one considers that the action of a *load* applied in the plane of the *web* (which causes a torque Qm) is equivalent to that same *load* when applied at the shear center (where it causes no torque) plus two forces P = Qm/d which, together, produce the same torque Qm. As is sketched in Figure C-C2.2.1-4 and shown in some detail by Winter, Lansing and McCalley (1949b), each half of the channel can then be regarded as a continuous beam loaded by the horizontal forces and supported at the brace points. The horizontal brace force is then, simply, the appropriate reaction of this continuous beam. The provisions of *Specification* Section C2.2.1 provide expressions for determining bracing forces P_{L1} and P_{L2}, which the braces are required to resist at each *flange*.

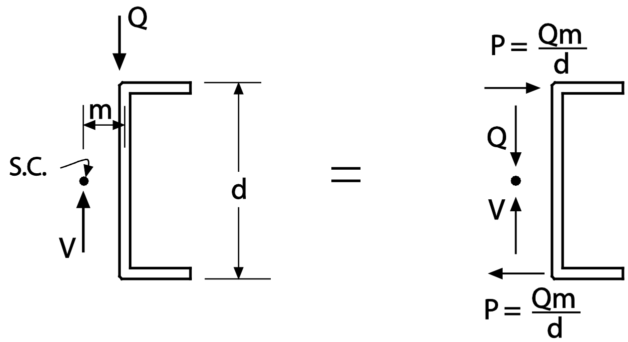


Figure C-C2.2.1-3 Lateral Forces Applied to C-Section

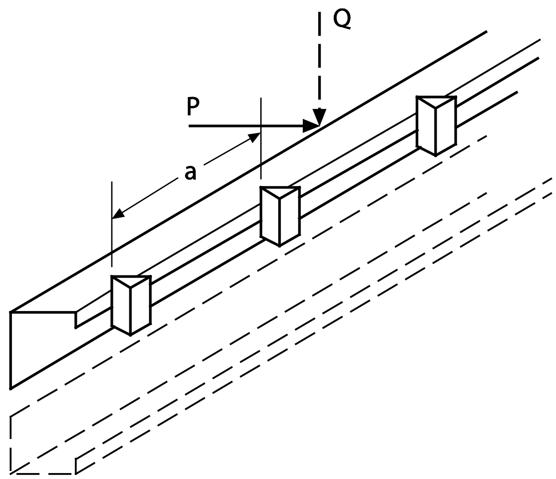


Figure C-C2.2.1-4 Half of C-Section Treated as a Continuous Beam Loaded by Horizontal Forces

(b) Bracing of Z-Section Beams

Most Z-sections are anti-symmetrical about the vertical and horizontal centroidal axes; i.e., they are point-symmetrical. In view of this, the centroid and the shear center coincide and are located at the midpoint of the *web*. A *load* applied in the plane of the *web* has, then, no lever arm about the shear center (m = 0) and does not tend to produce the kind of rotation that a similar *load* would produce on a C-section. However, in Z-sections the principal axes are oblique to the *web* (Figure C-C2.2.1-5). A *load* applied in the plane of the *web*, resolved in the direction of the two axes, produces deflections along each of them. By projecting these deflections onto the horizontal and vertical planes, it is found that a Z-beam loaded vertically in the plane of the *web* deflects not only vertically but also horizontally. If such deflection is permitted to occur, then the *loads*, moving sideways with the beam, are no longer in the same plane with the reactions at the ends. In consequence, the *loads* produce a twisting moment about the line connecting the reactions. In this manner it is seen that a Z-beam, unbraced between ends and loaded in the plane of the *web*, deflects laterally and also twists. Not only are these deformations likely to interfere with the proper functioning of the beam, but the additional *stresses* caused by them produce failure at a *load* considerably lower than when the same beam is used fully braced.

In order to obtain information for developing appropriate bracing provisions, tests have been carried out on three different Z-sections at Cornell University, unbraced as well as with variously spaced intermediate braces. In addition, an approximate method of analysis has been developed and checked against the test results. An account of this work was given by Zetlin and Winter (1955b). Briefly, it is shown that intermittently braced Z-beams can be analyzed in much the same way as intermittently braced C-beams. It is merely necessary, at the point of each actual vertical *load* Q, to apply a fictitious horizontal *load*, $Q(I_{xy}/I_x)$ or $Q[I_{xy}/(2I_x)]$, to each *flange*. One can then compute the vertical and horizontal deflections, and the corresponding *stresses*, in conventional ways by utilizing the convenient axes x and y (rather than 1 and 2, Figure C-C2.2.1-5), except that certain modified section properties have to be used. To control the lateral deflection, brace forces, P, must statically balance the fictitious force.

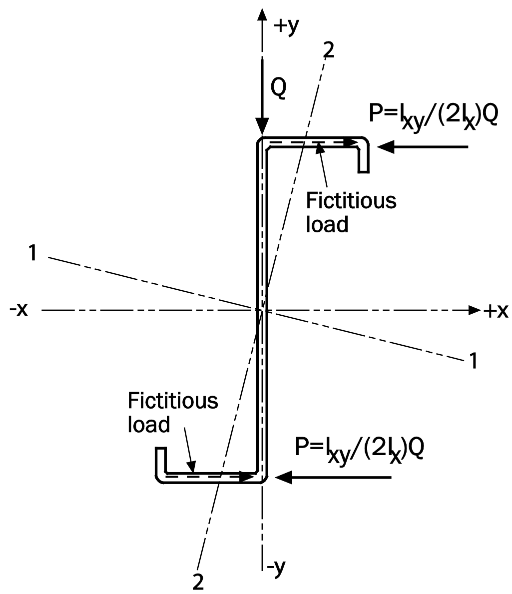


Figure C-C2.2.1-5 Principal Axis of Z-Section

In this manner it has been shown that as to location of braces, the same provisions that apply to C-sections are also adequate for Z-sections. Likewise, the forces in the braces are again obtained as the reactions of continuous beams horizontally loaded by fictitious loads, P. It should, however, be noted that the direction of the bracing forces in Z-beams is different from the direction in C-beams. In the Z-beam, the bracing forces are acting in the same direction, as shown in Figure C-C2.2.1-5, in order to constrain bending of the section about the axis x-x. The directions of the bracing forces in the C-beam *flanges* are in the opposite direction, as shown in Figure C-C2.2.1-3, in order to resist the torsion caused by the applied *load*. In the previous edition of the *Specification*, the magnitude of the Z-beam bracing force was shown as $P = Q(I_{xy}/I_x)$ on each *flange*. In 2001, this force was corrected to $P = Q[I_{xy}/(2I_x)]$.

(c) Slope Effect and Eccentricity

For a C- or Z-section member subjected to an arbitrary *load*, bracing forces, P_{L1} and P_{L2}, on *flanges* need to resist: (1) force component P_x that is perpendicular to the *web*, (2) the torsion caused by eccentricity about the shear center, and (3) for the Z-section member, the lateral movement caused by component P_y, that is parallel to the *web*.

To develop a set of equations applicable to any loading conditions, the x and y axes are oriented such that one of the *flanges* is located in the quadrant with both x and y axes positive. Since the torsion should be calculated about the shear center, coordinates x_s and y_s, that go through the shear center and parallel to x and y axes, are established. *Load* eccentricities e_x and e_y should be measured based on x_s and y_s coordinate system.

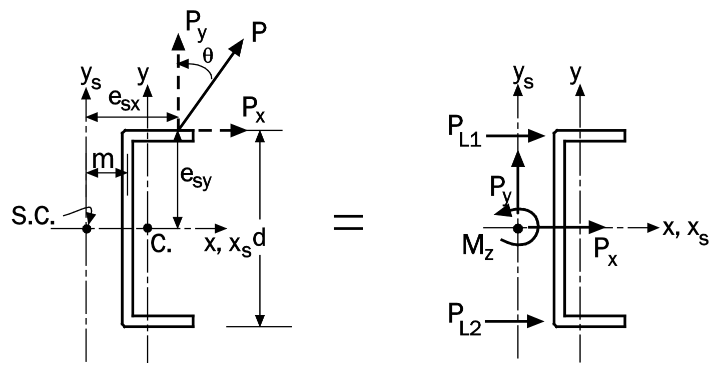


Figure C-C2.2.1-6 C-Section Member Subjected to a Concentrated Load

For the C-section member as shown in Figure C-C2.2.1-6, the bracing forces on both *flanges* are given in Equations C-C2.2.1-1 and C-C2.2.1-2.

$$P_{L1} = -\frac{P_x}{2} + \frac{M_z}{d} \tag{C-C2.2.1-1}$$

$$P_{L2} = -\frac{P_x}{2} - \frac{M_z}{d} \tag{C-C2.2.1-2}$$

$$M_z = -P_x e_{sy} + P_y e_{sx} \tag{C-C2.2.1-3}$$

where d = Overall depth of the *web*; e_{sx}, e_{sy} = Eccentricities of *load* about the shear center in x_s- and y_s-direction, respectively; P_x, P_y = Components of *load* in x- and y-direction, respectively; M_z = Torsional moment about the shear center; and P_{L1} = Bracing force applied to the *flange* located in the quadrant with both positive x and y axes, and P_{L2} = Bracing force applied on the other *flange*. Positive P_{L1} and P_{L2} indicate that a restraint is required to prevent the movement of the corresponding *flange* in the negative x-direction.

For a special case where *load*, Q, is through the *web*, as shown in Figure C-C2.2.1-3, P_y =-Q, P_x=0; e_{sx} = m, e_{sy} = d/2, and from Equation C-C2.2.1-3, M_z = -Qm. Therefore:

$$P_{L1} = -Qm/d \tag{C-C2.2.1-4}$$

$$P_{L2} = Qm/d \tag{C-C2.2.1-5}$$

In which, m = Distance from centerline of *web* to the shear center.

For the Z-section member as shown in Figure C-C2.2.1-7, bracing forces, P_{L1} and P_{L2}, are given in Equations C-C2.2.1-6 and C-C2.2.1-7.

$$P_{L1} = P_y\left(\frac{I_{xy}}{2I_x}\right) - \frac{P_x}{2} + \frac{M_z}{d} \tag{C-C2.2.1-6}$$

$$P_{L2} = P_y\left(\frac{I_{xy}}{2I_x}\right) - \frac{P_x}{2} - \frac{M_z}{d} \tag{C-C2.2.1-7}$$

where I_x, I_{xy} = Unreduced moment of inertia and product of inertia, respectively. Other variables are defined under the discussion for C-section members.

Assuming that a gravity *load*, P, acts at 1/3 of the top *flange* width, b_f, and the Z-section member rests on a sloped roof with an angle of θ, P_x = -Psinθ; P_y = -Pcosθ; e_{sx} = b_f/3; e_{sy} = d/2 and M_z = Psinθ(d/2) - Pcosθ(b_f/3). Substituting the above expressions into Equations C-C2.2.1-6 and C-C2.2.1-7 results in:

$$P_{L1} = -P\cos\theta(\frac{I_{xy}}{2I_x}) + P\sin\theta - \frac{Pb_f\cos\theta}{3d} \quad \text{(C-C2.2.1-8)}$$

$$P_{L2} = -P\cos\theta(\frac{I_{xy}}{2I_x}) + \frac{Pb_f\cos\theta}{3d} \quad \text{(C-C2.2.1-9)}$$

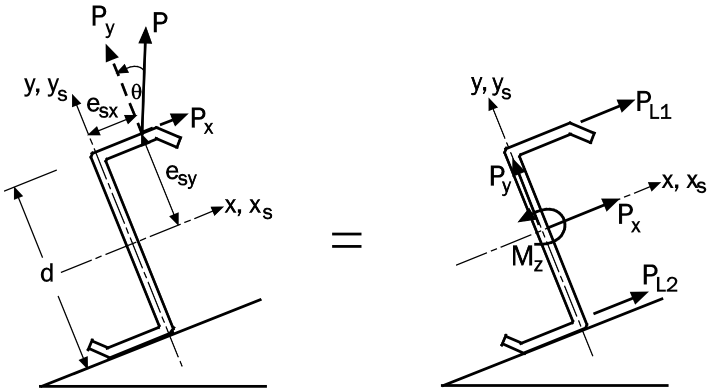


Figure C-C2.2.1-7 A Z-Section Member Subjected to an Arbitrary Load

In considering the distribution of *loads* and the braces along the member length, it is required that the resistance at each brace location along the member length be greater than or equal to the *design load* within a distance of 0.5a on each side of the brace for distributed *loads*. For concentrated *loads*, the resistance at each brace location should be greater than or equal to the concentrated *load* within a distance 0.3a on each side of the brace, plus 1.4(1-*l*/a) times each *load* located farther than 0.3a but not farther than 1.0a from the brace. In the above, "a" is the distance between centerline of braces along the member length and "*l*" is the distance from concentrated *load* to the brace to be considered.

In *Specification* Section C2.2.1, a top-bar is added to the variables designed as the *design load,* which are calculated in accordance with *ASD*, *LRFD*, or *LSD load combinations* depending on the design method used.

(d) Spacing of Braces

During the period from 1956 through 1996, the AISI *Specification* required that braces be attached both to the top and bottom *flanges* of the beam, at the ends and at intervals not greater than one-quarter of the span length, in such a manner as to prevent tipping at the ends and lateral deflection of either *flange* in either direction at intermediate braces. The *lateral-torsional buckling* equations provided in *Specification* Sections F2 and F3 can be used to predict the moment capacity of the member. Beam tests conducted by Ellifritt, Sputo and Haynes (1992) have shown that for typical sections, a mid-span brace may reduce *service load* horizontal deflections and rotations by as much as 80 percent when compared to a completely unbraced beam. However, the restraining effect of braces may change the failure mode from *lateral-torsional buckling* to *distortional buckling* of the *flange* and lip at a brace point.

The natural tendency of the member under vertical *load* is to twist and translate in such a manner as to relieve the compression on the lip. When such movement is restrained by intermediate braces, the compression on the stiffening lip is not relieved, and may increase. In this case, local *distortional buckling* may occur at *loads* lower than that predicted by the *lateral-torsional buckling* equations of *Specification* Sections F2 and F3.

Research (Ellifritt, Sputo and Haynes, 1992) has also shown that the *lateral-torsional buckling* equations of *Specification* Sections F2 and F3 predict *loads*, which are conservative for cases where one mid-span brace is used but may be unconservative where more than one intermediate brace is used. Based on such research findings, Section C2.2.1 of the *Specification* was revised in 1996 to eliminate the requirement of quarter-point bracing. It is suggested that, minimally, a mid-span brace be used for C-section and Z-section beams to control lateral deflection and rotation at *service loads*. The *lateral-torsional buckling* strength of an open cross-section member should be determined by *Specification* Sections F2 and F3 using the distance between centerlines of braces "a" as the unbraced length of the member "L" in all design equations. In any case, the user is permitted to perform tests, in accordance with *Specification* Section K2.1, as an alternative, or use a rigorous analysis, which accounts for biaxial bending and torsion.

Section C2.2.1 of the *Specification* provides the lateral forces for which these discrete braces must be designed.

The *Specification* permits omission of discrete braces when all *loads* and reactions on a beam are transmitted through members that frame into the section in such a manner as to effectively restrain the member against torsional rotation and lateral displacement. Frequently, this occurs in the end walls of metal buildings.

In 2007, the title of this section was changed to clarify that it is and was formerly anticipated that the C- and Z-sections covered by these provisions would be supporting sheathing and be loaded as a result of providing this support function. The revised title reflects that the supported sheathing is not contributing to the strength and stiffness of these members by virtue of the nature of its connection to the C- and Z-sections.

C2.2.2 Flange Connected to Sheathing That Contributes to the Strength and Stability of the C- or Z-Section

This section of the *Specification* reminds users that *stability* should be considered in accordance with provisions of Section I6.4.1 or I6.4.2 for members with sheathing attached. See commentary for those sections for detail. ☜B

C2.3 Bracing of Axially Loaded Compression Members

Intermediate bracing of axially loaded compression members is achieved by limiting translation and/or twist at locations along the member. Such bracing should be designed for both strength and *stiffness*.

Per the requirements of Section C1.1, a rigorous *second-order analysis* may be used to establish the *required brace strength* [brace force due to *factored loads*] and required brace *stiffness* for bracing a compression member. The analysis includes consideration of the initial out-of-straightness of the compression member as well as the bracing member properties, *connections,* and anchoring details.

Alternatively, bracing intended to restrain translation can be designed using the provisions provided in *Specification* Sections C2.3.1 and C2.3.2.

Compression members with *singly-symmetric*, *point-symmetric*, or asymmetric cross-*sections* may have a torsional *stiffness* that is small enough in which *flexural-torsional buckling* deformation develops under *load*. In such cases, adequate brace strength and *stiffness* should be provided to limit translation and twist at the compression member's brace point. Testing or advanced modeling is required to predict these torsional bracing requirements. Sputo and Turner (2006) offer a concise summary of cold-formed steel bracing practices.

C2.3.1 Translational Bracing of an Individual Concentrically Loaded Compression Member

The strength and *stiffness* requirements for the translational bracing of a single compression member were developed from a study by Green, et al. (2004) and adaptation of requirements in the AISC Specification (AISC, 2016a). These bracing provisions ensure that an individual concentrically loaded compression member can develop its *required compressive axial strength* [compressive axial force due to *factored loads*]; however, they do not necessarily allow individual concentrically loaded compression members to develop their fully braced capacity at an effective length equal to the length between braces. The required bracing *stiffness* ensures that the translation at the brace point is limited until the axial *loads* equal the *required strength* [compressive axial force due to *factored loads*], P_{ra}, which is determined in accordance with the applied *load* combinations for the corresponding design method of *ASD*, *LRFD*, or *LSD*. It is important to note that a compression member braced to these provisions has an *available strength* [*factored resistance*] equal to the *required strength* [compressive axial force due to *factored loads*], but not in excess of this *required strength* [compressive axial force due to *factored loads*]. If the *available strength* [*factored resistance*] of the compression member needs to exceed $\overline{P}_{ra}$, then the *required brace strength* [brace force due to *factored loads*] and required brace stiffness designed for $\overline{P}_{ra}$ should be increased. For example, if the *available strength* [*factored resistance*] of the compression member must equal the fully braced compression strength of the member, the *required axial compressive strength* [compressive axial force due to *factored loads*], $\overline{P}_{ra}$, in *Specification* Equations C2.3.1-1, C2.3.1-2a and C2.3.1-2b should be replaced by the fully braced *available strength* [*factored resistance*] of the compression member, P_n/Ω_c for *ASD* or $\phi_c P_n$ for *LRFD* or *LSD*.

The requirements for brace *stiffness* for a single compression member are similar to the AISC provisions (AISC, 2016a), with the exception that the number of equally spaced brace locations along the length of the member is accounted for by including the term (4-2/n) per Yura (1995). As a simplification, AISC assumes the most severe case (many intermediate braces with n = infinity), but this simplification is considered too conservative for cold-formed steel structures. Analytical modeling by Sputo and Beery (2006) has shown that these provisions may be applied to members of varied cross-sections. The *safety factor* (Ω=2.0) and *resistance factor* (ϕ=0.75) for calculating required brace *stiffness* in *Specification* Equations C2.3.1-2a and C2.3.1-2b are the same as those used in the AISC provisions (AISC, 2016a).

The strength and *stiffness* requirements provided assume that the brace is perpendicular to the compression member being braced and located in the plane of *buckling*. For brace members with a non-perpendicular angle, the *required brace strength* [brace force due to *factored loads*] and *stiffness* should be increased as follows:

$$\overline{P}'_{rb} = \frac{\overline{P}_{rb}}{\cos\theta\cos\phi} \qquad \text{(C-C2.3.1-1)}$$

where

$\overline{P}'_{rb}$ = *Required strength* [brace force due to *factored loads*] of the non-perpendicular brace

θ, ϕ = Angles of brace from perpendicular and plane of *buckling*, respectively (see Figure C-C2.3.1-1)

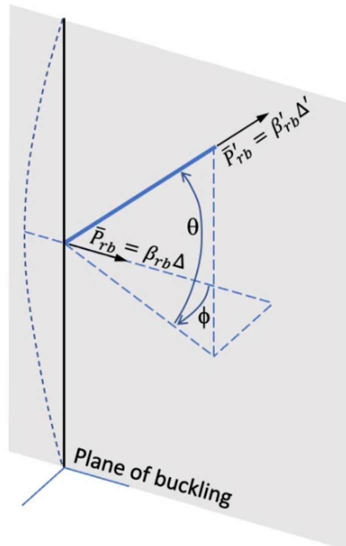


Figure C-C2.3.1-1 Non-Perpendicular Brace

The required *stiffness* of a perpendicular brace, β_{rb}, is

$$\beta_{rb} = \frac{\overline{P}_{rb}}{\Delta} \tag{C-C2.3.1-2}$$

where

Δ = Lateral displacement of brace point

And, the required *stiffness* of the non-perpendicular brace, β'_{rb}, is

$$\beta'_{rb} = \frac{\overline{P}'_{rb}}{\Delta'} \tag{C-C2.3.1-3}$$

where

Δ' = Axial deformation of non-perpendicular brace

The relationship between this axial deformation and the lateral displacement is given by

$$\Delta'_{rb} = \Delta_{rb} \cos\theta \cos\phi \tag{C-C2.3.1-4}$$

Substituting Equations C-C2.3.1-1, C-C2.3.1-2, and C-C2.3.1-4 into Equation C-C2.3.1-3,

$$\beta'_{rb} = \frac{\beta_{rb}}{(\cos\theta\cos\phi)^2} \tag{C-C2.3.1-5}$$

Per Figure C-C2.3.1-2, the required brace *stiffness*, β_{rb}, for an individual compression member should include the stiffness contributions of the bracing members, β_{brace}, the *connections*, β_{conn}, and the lateral *stiffness* at the anchor location, β_{anchor}, and is represented by the inverse of sum of the flexibilities

$$\frac{1}{\beta_{rb}} = \frac{1}{\beta_{brace}} + \frac{1}{\beta_{conn}} + \frac{1}{\beta_{anchor}} \tag{C-C2.3.1-6}$$

The lateral *stiffness* at the anchor location, β_{anchor}, is computed from a simple structural analysis in which this *stiffness* value would be taken as the force required to displace the anchorage system by a unit displacement. As illustrated in Figure C-C2.3.1-2, β_{anchor} should include both the lateral *stiffness* of the anchor and the *stiffness* of the *connection* where the bracing attaches to the anchorage.

Once the *required brace strength* [brace force due to *factored loads*] and required *stiffness* are determined in accordance with *Specification* Equations C2.3.1-1 and C2.3.1-2, the brace member should then be designed in accordance with *Specification* Section B3.2.1, B3.2.2, or B3.2.3, as appropriate, and with the *safety* and *resistance factors* determined in accordance with the applicable *Specification* section.

Additional brace strength and *stiffness* at brace points may be required to brace members that may also be subject to bending, torsion, or torsional-flexural *stresses*. Bracing for these effects are not accounted for in Section C2.3.1 and should be determined through rational analysis or other methods.

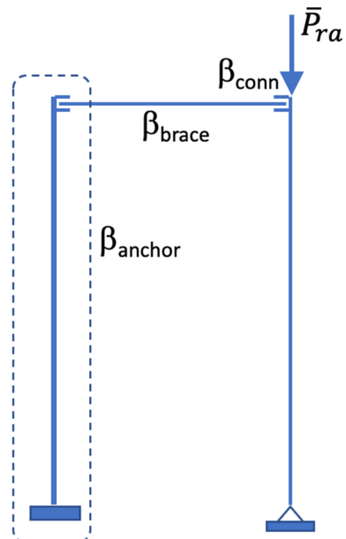


Figure C-C2.3.1-2 Single Member Bracing Configuration

C2.3.2 Translational Bracing of Multiple Parallel Concentrically Loaded Compression Members

The strength and *stiffness* requirements for an individual brace should be increased when it is used to provide stability for more than one compression member. In lieu of using a *second-order analysis* in conjunction with the requirements provided in Section C1, strength and *stiffness* requirements are provided in this section.

Employing the ΣP or lean-on concept (Yura, 2008), the *required brace strength* [brace force due to *factored loads*] is taken as the sum of the individual strengths (per *Specification* Equation C2.3.1-1) that are required to brace each of the m compression members. When the bracing of such compression members terminates at two end anchorages and the brace members are designed for both compression and tension, the total *required brace strength* [brace force due to *factored loads*] may be reduced by a factor of j = 2. To account for the possibility that the imperfections in all compression members are not of the same magnitude and perhaps not *buckling* towards the same direction, this total required brace force can be further reduced by $0.5(1+1/\sqrt{m})$. This expression is similar to the reduction factors of $1/\sqrt{m}$ suggested by Chen and Tong (1994), and $(0.2+0.8/\sqrt{m})$ as recommended by the Canadian Standards Association (CSA Group, 2014).

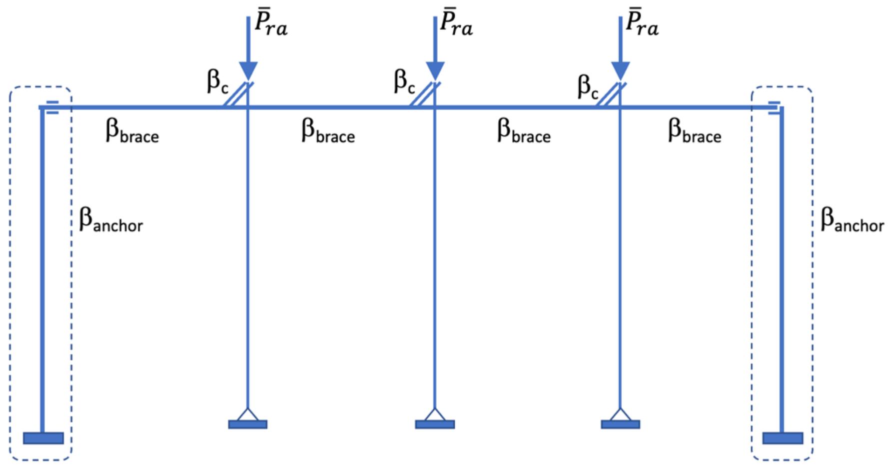
As with the strength requirement, the required *stiffness* of a brace should also be increased when it is used to provide stability to multiple compression members. Based on the work by Ziemian and Ziemian (2017, 2021), with insight provided by Blum, et al. (2014) and Sputo and Beery (2008), the required brace *stiffness*, $\beta_{rb,max}$, which would be computed using *Specification* Equation C2.3.1-2 for the compression member with largest *required axial strength* [force due to *factored load*], should be increased according to *Specification* Equation C2.3.2-2.

In general, the bracing in systems with multiple compression members can be considered as either continuous (Figure C-C2.3.2-1a) or discontinuous (Figure C-C2.3.2-1b). Consider, for example, two bridging possibilities for a system of axially loaded cold-formed steel studs. The bracing in a tension-compression system would be continuous through the *webs* of the studs with the bracing force required for each compression member being transferred to the brace via a connector. With forces accumulating only in the brace, and not within the connectors, the *stiffness* of the bracing member β_{brace} can be taken directly as β_{rb} (from Eq. C2.3.2-2). It is noted that the factor γ_c (Eq. C2.3.2-4) accounts for the flexibility of the connector *stiffness*. In contrast, the bracing in an intermediate or blocking system is discontinuous, with individual brace segments spanning between pairs of compression members. In this case, the bracing forces accumulate in both the brace segments and the *connections* attaching the brace segments to the compression members. As a result, the brace and the *connections* at each of its ends are in series, and the minimum required *stiffness* β_{rb} is represented by the inverse of the sum of the flexibilities:

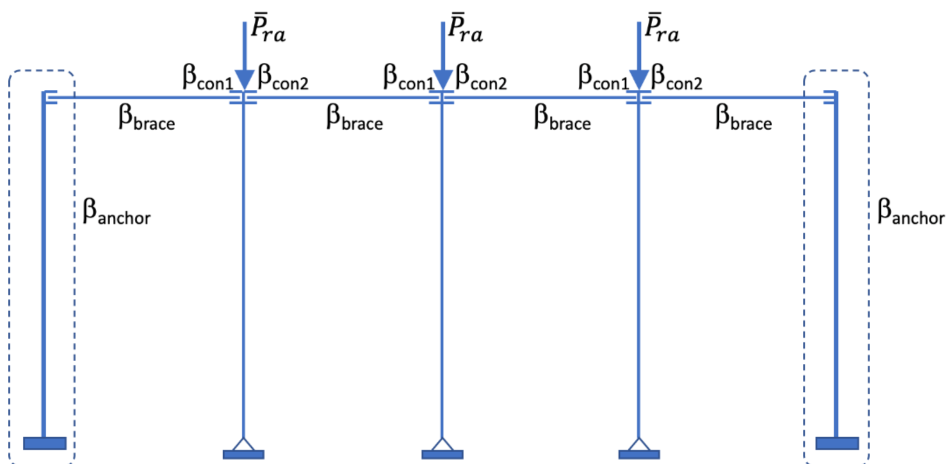
$$\frac{1}{\beta_{rb}} = \frac{1}{\beta_{brace}} + \frac{1}{\beta_{con1}} + \frac{1}{\beta_{con2}} \qquad \text{(C-C2.3.2-1)}$$

in which, β_{rb}, comes from *Specification* Eq. C2.3.2-2, β_{brace} is the *stiffness* of the bracing member, and β_{con1} and β_{con2} are the *stiffnesses* of the *connections* at the ends of the brace segments.

Regardless of whether the bracing system is continuous or discontinuous, the accumulated bracing force needs to be resolved at an anchor or termination point(s). The flexibility of this anchor system is represented in the factor γ_a (Eq. C2.3.2-3). The anchor system can have significant influence on the *stiffness* demands of the brace and its *stiffness* β_{anchor} should include both the anchoring member or subassemblage and the *connection* used to attach the brace to the anchor (Figure C-C2.3.2-1).



(a) Continuous



(b) Discontinuous

Figure C-C2.3.2-1 Bracing Configurations (m = 3, j =2)

D. MEMBERS IN TENSION

In 2010, the provisions for tension members were consolidated and moved from the country-specific appendices to the main *Specification*. The *available tensile strength* [*factored resistance*] of axially loaded cold-formed steel tension members is determined either by yielding of the *gross area* of the cross-section or by rupture of the *net area* of the cross-section. At locations of *connections*, the *nominal tensile strength* [*resistance*] is also limited by the *available strengths* [*factored resistances*] specified in *Specification* Chapter J for tension in connected parts.

D2 Yielding of Gross Section

Yielding in the gross section indirectly provides a limit on the deformation that a tension member can achieve. The definition of yielding in the gross section to determine the tensile strength is well established in hot-rolled steel construction.

The *resistance factor* ϕ_t = 0.90 and *safety factor* Ω_t = 1.67 used for yielding of the gross section are consistent with the factors used in ANSI/AISC 360 Specification (AISC, 2010a) and CSA S16 Specification (CSA, 2009).

D3 Rupture of Net Section

The *resistance factor* of ϕ_t = 0.75 and *safety factor* of Ω_t = 2.00 used for rupture of the net section are consistent with the factors used in the ANSI/AISC 360 Specification (AISC, 2010a) and CSA S16 Specification (CSA, 2009).

E. MEMBERS IN COMPRESSION

E1 General Requirements

Cold-formed steel column members should be designed considering *yielding* and global (*flexural*, *flexural-torsional* and *torsional*) *buckling* in accordance with *Specification* Section E2, *local buckling* with *yielding* and global *buckling* in accordance with *Specification* Section E3, and *distortional buckling* in accordance with *Specification* Section E4, as applicable.

Two approaches can be used in column design: the *Effective Width Method* (*EWM*) and the *Direct Strength Method* (*DSM*). The *EWM* traditionally addressed *local* and global *buckling*. In 2004, the *distortional buckling* strength prediction using *DSM* was adopted as an alternative method.

The calibration of the *EWM* has been reported in the *Commentary* in the 1991 edition of the AISI *Specification*. A brief discussion of the *DSM* is provided herein. In considering column *yielding* and global *buckling*, the *DSM* is essentially the same as the *EWM*. However, the approach of the two methods in predicting the strength due to *local buckling* is different. The *DSM* strength curves for *local* and *distortional buckling* of a fully braced column are presented in Figure C-E1-1. The curves are presented as a function of slenderness, which in this case refers to slenderness in the local or distortional mode, as opposed to traditional long column slenderness. Inelastic and post-*buckling* regimes are observed for both *local* and *distortional buckling* modes. The magnitude of the post-*buckling* reserve for the *distortional buckling* mode is less than the *local buckling* mode, as may be observed by the location of the strength curves in relation to the critical elastic *buckling* curve.

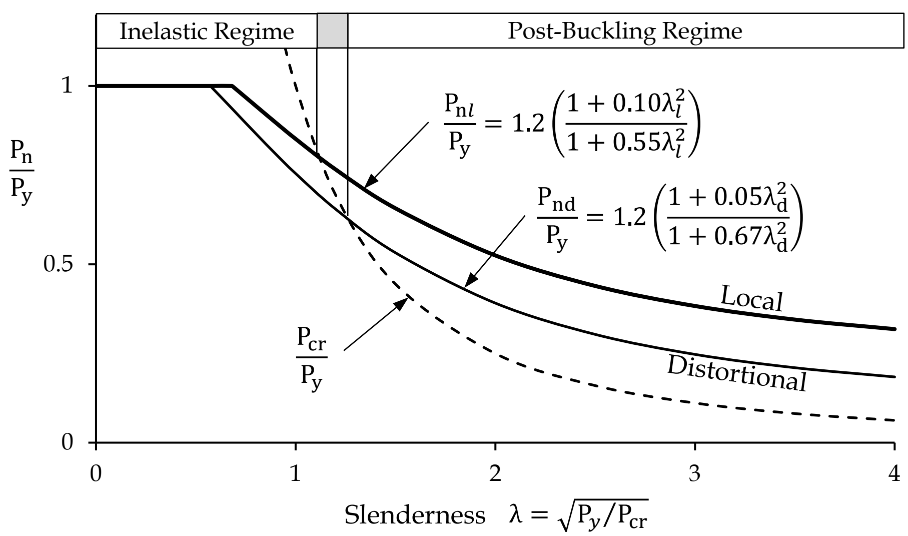


Figure C-E1-1 Local and Distortional Direct Strength Curves for a Braced Column ($P_{ne} = P_y$)

The development and calibration of the *DSM* provisions for columns are reported in Schafer (2000, 2002). Changes to the form of the strength curve equations in 2024 were developed by Glauz (2023), providing equivalent strength predictions and consistent with the equation forms used for the *DSM* flexural strength curves. The reliability of the column provisions was determined using the test data defined by the limits of Section B4.1 and the provisions of Section K2 of the *Specification*. Based on a target reliability, β_o, of 2.5, a *resistance factor*, ϕ, of 0.84 was calculated for all the investigated columns. Based on this information, the *safety* and *resistance factors* of Chapter E were determined for the prequalified members. For the United States and Mexico, ϕ = 0.85 was selected; while for Canada, ϕ = 0.80, since a slightly higher reliability, β_o, of 3.0 is employed. The *safety factor*, Ω, was back-calculated from ϕ at an assumed dead-to-live *load* ratio of 1 to 5. Since the range of prequalified members is relatively large, extensions of the *DSM* to geometries outside the prequalified set is allowed. Given the uncertain nature of this extension, increased *safety factors* and reduced *resistance factors* are applied in that case, per the *rational engineering analysis* provisions of Section A1.2.6(c) of the *Specification*.

The development and calibration of the *DSM* provisions for columns with holes was performed with experimental and simulation databases as reported in Moen and Schafer (2009a) and summarized in Moen and Schafer (2011). Note that both databases contain only lipped Cee cross-sections with discrete *web* holes because this is what was available in the research literature at the time. However, the philosophy of employing elastic *buckling* parameters ($P_{cr\ell}$, P_{crd}, P_{cre}) to predict the ultimate strength of cold-formed steel columns with holes was thoroughly validated in Moen and Schafer (2009a), and is assumed to hold true for other cross-section shapes.

The generality of the *DSM* approach for holes was demonstrated across experiments and nonlinear finite element analysis collapse simulations across a wide range of spacing, shape, and size of holes for both cold-formed steel columns and beams. Based on a target reliability, β_o, of 2.5, the *resistance factor*, ϕ, was calculated as 0.94 (experiments) and 0.89 (simulations) for columns with holes predicted to fail from local-global *buckling* interaction. For columns with holes predicted to experience a *distortional buckling* failure mode, ϕ was calculated as 0.96 (experiments) and 0.91 (simulations). The prediction accuracy for *DSM* for members with holes is greater than that for members without holes (Ganesan and Moen, 2012). The new form of *DSM* equations developed by Glauz (2023) for *local* and *distortional buckling* of columns further improved the reliability of the strength curves for members with holes.

E2 Yielding and Global (Flexural, Flexural-Torsional and Torsional) Buckling

In this section, the *limit states* of *yielding* and overall column *buckling* are discussed. In 2022, the elastic global *buckling* equations were moved from *Specification* Section E2 to Appendix 2 to consolidate all elastic *buckling* provisions in the appendix.

A. Yielding

It is well known that a very short, compact column under an axial *load* may fail by yielding. The yield load is determined by Equation C-E2-1:

$$P_y = A_g F_y \tag{C-E2-1}$$

where A_g is the *gross area* of the column and F_y is the *yield stress* of steel.

B. Global Buckling of Columns

(a) Elastic Buckling Stress

A slender, axially loaded column may fail by overall *flexural buckling* if the cross-section of the column is a *doubly-symmetric* shape, closed shape (square or rectangular tube), cylindrical shape, or *point-symmetric* shape. For *singly-symmetric* shapes, *flexural buckling* is one of the possible failure modes. Wall studs connected with sheathing material can also fail by *flexural buckling*.

The elastic critical *buckling* load for a long column can be determined by the following Euler equation:

$$P_{cre} = \frac{\pi^2 EI}{(KL)^2} \tag{C-E2-2}$$

where P_{cre} is the column *buckling* load in the elastic range, E is the modulus of elasticity, I is the moment of inertia, K is the *effective length factor*, and L is the unbraced length. Accordingly, the elastic column *buckling stress* is

$$F_{cre} = \frac{P_{cre}}{A_g} = \frac{\pi^2 E}{(KL/r)^2} \tag{C-E2-3}$$

in which r is the radius of gyration of the full cross-section, and KL/r is the effective slenderness ratio.

Section 2.3.1.1 of *Commentary* Appendix 2 covers elastic *buckling* of compression members in greater detail, addressing *effective length factors*, *torsional buckling*, *flexural-torsional buckling*, and non-symmetric shapes.

(b) Nominal Axial Strength [Resistance] for Locally Stable Columns

If the individual components of compression members have small w/t ratios, *local buckling* will not occur before the compressive *stress* reaches the column *buckling stress* or the *yield stress* of steel. In the 1996 *Specification*, the design equations for calculating the inelastic and elastic *flexural buckling stresses* were aligned with those used in the AISC *LRFD Specification* (AISC, 1993) as follows:

$$\text{For } \lambda_c \le 1.5, \quad F_n = (0.658^{\lambda_c^2})F_y \tag{C-E2-4}$$

$$\text{For } \lambda_c > 1.5, \quad F_n = \left(\frac{0.877}{\lambda_c^2}\right)F_y \tag{C-E2-5}$$

where F_n is the nominal *flexural buckling stress* which can be either in the elastic range or in the inelastic range depending on the value of $\lambda_c = \sqrt{F_y / F_{cre}}$, and F_{cre} is the elastic *flexural buckling stress* calculated by using Equation C-E2-3. Together these equations provide a continuous strength curve as shown in Figure C-E2-1. The *nominal axial strength* [*resistance*] is the nominal *flexural buckling stress* times the *gross area* as given in Equation E2-1 of the *Specification.*

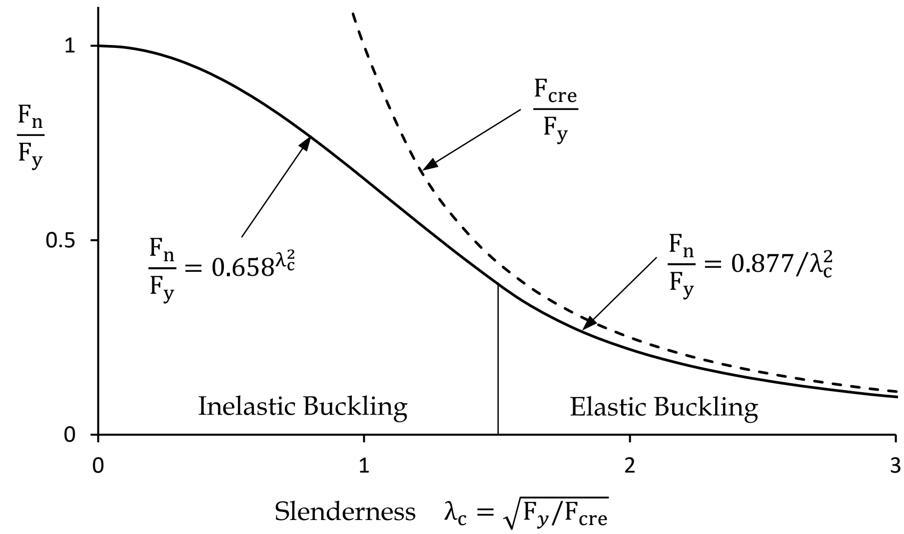


Figure C-E2-1 Global Buckling Stress Curve for Columns

C. *Additional Design Consideration for Angles*

During the development of a unified approach to the design of cold-formed steel members, Peköz realized the possibility of a reduction in column strength due to initial sweep (out-of-straightness) of angle sections. Based on an evaluation of the available test results, an initial out-of-straightness of L/1000 was recommended by Peköz for the design of concentrically loaded compression angle members and beam-columns in the 1986 edition of the AISI *Specification*. Those requirements were retained in Sections E2 and H1.2 of the *Specification*. A study conducted at the University of Sydney (Popovic, Hancock, and Rasmussen, 1999) indicated that for the design of *singly-symmetric* unstiffened angle sections under the axial compression *load*, the required additional moment about the minor principal axis due to initial sweep should only be applied to those angle sections subjected to *local buckling* at *stress* F_y. Consequently, clarification was made in the 2001 edition of the AISI *Specification*, and is retained in Section H1.2 of this edition.

Specification Equations E2-1 to E2-3 have been shown to be conservative in predicting the experimental failure loads obtained from tests of concentrically loaded pin-ended and fixed-ended angle columns. Tested columns exhibit end supports fixed with respect to warping and major-axis flexure, but pinned or fixed with respect to minor-axis flexure. Tests were performed by Popovic, et al. (1999) and Chodraui, et al. (2006) for columns with minor-axis pin-ends, and by Popovic, et al. (1999) and Young (2004, 2005) for columns with fixed-ends. The above underestimation is essentially due to the fact that *Specification* Equations E2-1 to E2-3: (1) account twice for the local/flexural-torsional effects (Rasmussen, 2005), and (2) disregard the beneficial effect of the warping fixity (Shifferaw and Schafer, 2014). Dinis, et al. (2012) and Mesacasa, et al. (2014) investigated the mechanics of these phenomena and showed that the collapse of intermediate plain angle columns is governed by the interaction between major-axis *flexural-torsional buckling* (akin, but not identical, to *local bucking*) and minor-axis *flexural buckling*. Due to effective centroid shift effects (Young and Rasmussen, 1999), this interaction is much stronger in pin-ended columns. Several design methods/approaches have been proposed to estimate more accurately the angle column failure loads, thus accounting for the increased strength due to the warping fixity (e.g., Young, 2004; Rasmussen, 2005; Silvestre, et al., 2013; Shifferaw and Schafer, 2014; and Dinis and Camotim, 2015). Using flexural-torsional strength curves (instead of the *local buckling* strength curves), the research finding of angle end-fixity is valid for columns with pin-ends and fixed-ends, and provides reliable prediction of column failure loads.

D. *Slenderness Ratios*

The slenderness ratio, KL/r, of all compression members should preferably not exceed 200, except that during construction only, KL/r should not exceed 300. In 1999, the above recommendations were moved from the *Specification* to the *Commentary*.

The maximum slenderness ratios on compression and tension members have been stipulated in steel design standards for many years but are not mandatory in the AISI *Specification*.

The KL/r limit of 300 is still recommended for most tension members in order to control serviceability issues such as handling, sag and vibration. The limit is not mandatory, however, because there are a number of applications where it can be shown that such factors are not detrimental to the performance of the structure or assembly of which the member is a part. Flat strap tension bracing is a common example of an acceptable type of tension member where the KL/r limit of 300 is routinely exceeded.

The compression member KL/r limits are recommended not only to control handling, sag and vibration serviceability issues, but also to flag possible strength concerns. The AISI *Specification* provisions adequately predict the capacities of slender columns and beam-columns, but the resulting strengths are quite small and the members relatively inefficient. Slender members are also very sensitive to eccentrically applied axial *load* because the moment magnification will be large.

E2.1 Reduction for Closed-Box Sections

For calculating the *nominal strength* [*resistance*] of concentrically loaded compression members with a closed-box section, *Specification* Equations E2.1-1 and E2.1-2, based on the University of Sydney research findings (Yang, Hancock and Rasmussen, 2004), were added in the *Specification* Section E2.1 when determining the *nominal axial strength* [*resistance*] according to Sections E2 and E3. The reduction factor R specified in *Specification* Equation E2.1-2 is to be applied to the *buckling stress*, F_{cre}, and allows for the interaction of *local* and *flexural* (Euler) *buckling* of thin high-strength low-ductility steel sections. The reduction factor varies with length from 0.4225 at KL = 0 to 1.0 at KL = $1.1L_0$, where L_0 is the length at which the *local buckling stress* equals the *flexural buckling stress*.

E3 Local Buckling Interacting With Yielding and Global Buckling

The discussion in Section E2 refers to members subject to global (*flexural*, *flexural-torsional* and *torsional) buckling*, but made up of elements whose w/t ratios are small enough so that no *local buckling* will occur. For shapes which are sufficiently thin, i.e., with w/t ratios sufficiently large, *local buckling* can combine with global *buckling*. For this case, the effect of *local buckling* on the global *buckling* strength can be handled by using the *Effective Width Method*, which applies the *effective area*, A_e, determined at the *stress* F_n; or the *Direct Strength Method*, which takes into consideration the *local* and global *buckling* interaction in the strength predication equations.

The *Effective Width Method*'s approach to *local buckling* is to conceptualize the member as a collection of "elements" and investigate *local buckling* of each element separately.

The *Direct Strength Method* provides a means to incorporate all relevant global *buckling* modes into the design process. Further, all *buckling* modes are determined for the member as a whole rather than element by element. This ensures that compatibility and equilibrium are maintained at element junctures. Consider, as an example, the lipped C-section shown in pure compression in Figure C-2.2.2-2(a). The member's *local* elastic *buckling* load from the analysis is:

$P_{cr\ell}$ = 0.12 x 48.42 kips = 5.81 kips (25.84 kN)

The column has a *gross area* (A_g) of 0.881 in^2 (568.4 mm^2); therefore,

$f_{cr\ell} = P_{cr\ell}/A_g$ = 6.59 ksi (45.44 MPa)

The *Effective Width Method* determines a plate *buckling* coefficient, k, for each element, then f_{cr}, and finally the *effective width*. The centerline dimensions (ignoring corner radii) are h = 8.94 in. (227.1 mm), b = 2.44 in. (62.00 mm), d = 0.744 in. (18.88 mm), and t = 0.059 in. (1.499 mm), the critical *buckling stress*, f_{cr}, of each element as determined from Appendix 1 of the *Specification*:

lip: k = 0.43, $f_{cr\ell\text{-lip}} = 0.43[\pi^2E/(12(1-\mu^2))](t/d)^2$ = 72.1 ksi (497 MPa)

flange: k = 4, $f_{cr\ell\text{-flange}} = 4.0[\pi^2E/(12(1-\mu^2))](t/b)^2$ = 62.4 ksi (430 MPa)

web: k = 4, $f_{cr\ell\text{-web}} = 4.0[\pi^2E/(12(1-\mu^2))](t/h)^2$ = 4.6 ksi (32.0 MPa)

Each element predicts a different *buckling stress*, even though the member is a connected group. These differences in the *buckling stress* are ignored in the *Effective Width Method*. The high *flange* and lip *buckling stresses* have little relevance given the low *web buckling stress*. The finite strip analysis, which includes the interaction amongst the elements, shows that the *flange* aids the *web* significantly in *local buckling*, increasing the *web buckling stress* from 4.6 ksi (32.0 MPa) to 6.59 ksi (45.4 MPa), but the *buckling stress* in the *flange* and lip are much reduced due to the same interaction.

The *Direct Strength Method* is a robust method, but the *Effective Width Method*, which has been used by design engineers since the 1986 edition of the *Specification*, also provides a comprehensive and reliable design solution.

E3.1 Effective Width Method

For cold-formed steel compression members with large w/t ratios, *local buckling* of individual component plates may occur before the applied *load* reaches the *nominal axial strength* [*resistance*] determined by Equation C-E2-1. The interaction effect of the *local* and overall column *buckling* may result in a reduction of the overall column strength. From 1946 through 1986, the effect of *local buckling* on column strength was considered in the AISI *Specification* by using a form factor, Q, in the determination of allowable *stress* for the design of axially loaded compression members (Winter, 1970; Yu and LaBoube, 2010). Even though the Q-factor method was used successfully for the design of cold-formed steel compression members, research work conducted at Cornell University and other institutions has shown that this method can be improved. On the basis of the test results and analytical studies of DeWolf, Peköz, Winter, and Mulligan (DeWolf, Peköz and Winter, 1974; Mulligan and Peköz, 1984) and Peköz's development of a unified approach for the design of cold-formed steel members (Peköz, 1986b), the Q-factor method was eliminated in the 1986 edition of the AISI *Specification*. In order to reflect the effect of *local buckling* on the reduction of column strength, the *nominal axial strength* [*resistance*] is determined by the critical column *buckling stress* and the *effective area*, A_e, instead of the full sectional area. When A_e cannot be calculated, such as when the compression member has dimensions or geometry beyond the range of applicability of the AISI *Specification*, the *effective area,* A_e, can be determined experimentally by stub column tests using AISI S902, *Stub-Column Test Method for Effective Area of Cold-Formed Steel Columns* (AISI, 2013c). For a more in-depth discussion of the background for these provisions, see Peköz (1986b). Therefore, the *nominal axial strength* [*resistance*] of cold-formed steel compression members can be determined by the following equation:

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

where F_{cr} is either elastic *buckling stress* or inelastic *buckling stress*, whichever is applicable, and A_e is the *effective area* at F_{cr}.

In the *Effective Width Method*, column *nominal strength* [*resistance*] is calculated by multiplying the nominal column *buckling stress*, F_n, by the *effective area*, A_e, calculated at F_n. This accounts for *local buckling* reductions in the actual column strength (i.e., local-global interaction).

Research at the University of Sydney (Popovic, Hancock, and Rasmussen, 1999) has shown that *singly-symmetric* unstiffened cold-formed steel angles, which have a fully effective cross-section under *yield stress*, do not fail in a flexural-torsional mode and can be designed based on *flexural buckling* alone as specified in *Specification* Appendix 2 Section 2.3.1.1.1. There is also no need to include a *load* eccentricity for these sections when using *Specification* Section H1.2 as explained in Item C of *Commentary* Section E2.

For members with holes, the provisions in *Specification* Appendix Section 1.1.1 (a) should be used in determining the *effective area*, A_e. The *buckling stress*, F_{cre}, for members with holes determined in accordance with Appendix 2 is a gross section *stress*. Therefore, this *stress* must be multiplied by A_g/A_{net} to establish the *stress* at the net section.

The *Specification* permits ignoring the hole effect if the number of holes in the *effective length* region times the hole length divided by the *effective length* does not exceed 0.015.

E3.2 Direct Strength Method

In the *Direct Strength Method*, the *local buckling* strength considers local-global interaction if not fully braced, i.e., the global *buckling* strength (P_{ne}) is less than the yield strength (P_y). The calibration of the *Direct Strength Method* has been provided in *Commentary* Section E1.

The *nominal strength* [*resistance*] of compression members without holes is provided in *Specification* Section E3.2 with P_{ne} determined in accordance with Section E2.

The *Direct Strength Method* (*DSM*) approach to columns with holes utilizes the elastic *buckling* properties of a cold-formed steel column ($P_{cr\ell}$, P_{crd}, and P_{cre}), including the influence of holes (e.g., flat punched holes in studs, patterned holes in rack sections, holes with edge stiffeners) to predict ultimate strength. In most cases, holes decrease the elastic *buckling* properties, $P_{cr\ell}$, P_{crd}, and P_{cre}, which increase a column's local (λ_ℓ), distortional (λ_d) and global (λ_c) slenderness and lower the predicted strength. Simplified methods for predicting $P_{cr\ell}$, P_{crd}, and P_{cre} including holes are presented in Appendix 2. Alternatively, full finite element elastic eigen-*buckling* analysis can be performed.

The *DSM* strength prediction expression has been modified to limit the maximum strength of a column with holes to the capacity of the net cross-section, P_{ynet} (Moen and Schafer, 2011) and was further refined by Glauz (2023). Figure C-E3.2-1 compares the *local buckling* strength prediction for columns with holes having different ratios of P_{ynet}/P_{ne}. As local slenderness increases, the prediction transitions from P_{ynet} to the strength curve used for columns without holes. The transition is implemented to reflect the change in failure mode as slenderness increases, from *yielding* at the net section to *local buckling*.

The provisions of this section are limited to *local buckling* slenderness, λ_ℓ, less than 5 due to uncertainty of the strength predictions at high slenderness. The limits of applicability in *Specification* Section B4.1 already restrict slenderness to a practical range, but it is still possible to exceed the imposed slenderness limit. One rational approach to high slenderness applications is to limit the *local buckling* strength using Equation C-E3.2-1 which approximates the original form of *DSM* equation approaching zero at high slenderness.

$$P_{n\ell} \le \frac{P_{ne}}{\lambda_\ell^{0.8}} \tag{C-E3.2-1}$$

Another rational approach for approximating the strength of slender sections is to ignore the large flat widths and exclude most of the material from those elements. This will reduce the yield load P_y and increase the critical elastic *local buckling* load $P_{cr\ell}$, resulting in a lower slenderness which may allow use of the *Specification* equations.

For columns with holes, the *DSM* strength prediction expression was modified to limit the maximum strength to the capacity of the net cross-section (Moen and Schafer, 2009a) and was further refined with a transition curve by Glauz (2023). Figure C-E3.2-1 compares the *local buckling* strength prediction for columns with holes having different ratios of P_{ynet}/P_{ne}. As local slenderness increases, the prediction transitions from P_{ynet} to the strength curve used for columns without holes. The transition is implemented to reflect the change in failure mode as slenderness increases, from yielding at the net section to *local buckling*.

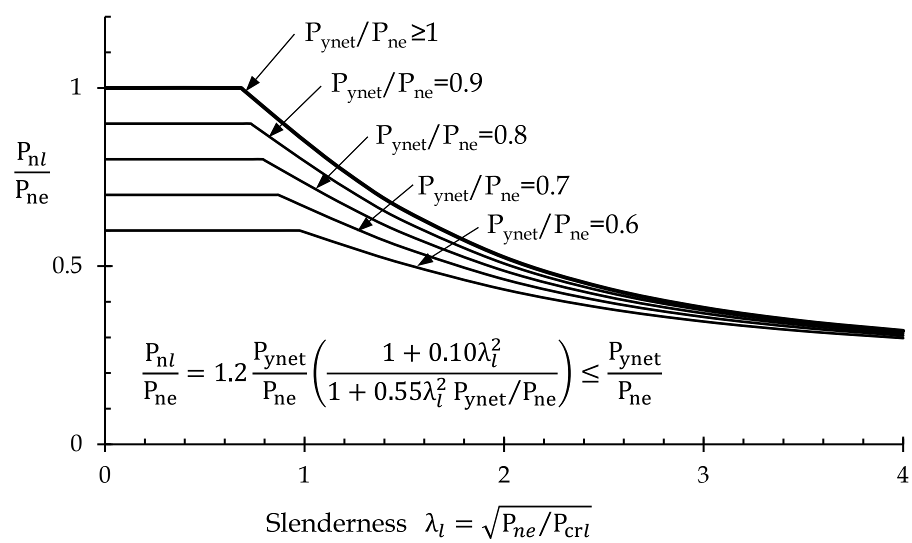


Figure C-E3.2-1 DSM for Local Buckling of Columns with Holes

Holes are common in cold-formed steel members, and their presence reduces *structural member* strength as defined by *Direct Strength Method* equations in *Specification* Section E3.2 for compression members and Section F3.2 for flexural members. Hole influence on strength can be counterintuitive and difficult to predict just with engineering judgment alone. Therefore, the strength reduction should be calculated, even if the holes are small. Rules of thumb on the influence of holes in both compression and flexural members are: (1) rectangular or elongated holes typically reduce *local buckling* strength more than square and circular holes; (2) *web* holes always decrease *distortional buckling* strength; (3) holes always reduce global (Euler) *buckling* strength; (4) the more holes along a member, the more the strength decreases; (5) hole patterns, such as those typically present in storage rack columns, can reduce strength as much as discrete holes; and (6) adding edge stiffeners to holes increases *local buckling* strength more than *distortional buckling* and global *buckling* strength.

In an approximate strength check, the influence of holes on unlapped compression or flexural members can be ignored when the sum of the length of holes along the member is less than or equal to 10 percent of the member length ($\Sigma(L_h/L)\leq 0.10$); the maximum hole depth (width) is greater than or equal to 25 percent of the hole length ($d_h/L_h \geq 0.25$); and the *net* cross-sectional *area* is greater than or equal to 95 percent of the gross cross-sectional area ($A_{net}/A_g \geq 0.95$). Members meeting these limits are expected to have a capacity reduction of 5 percent or less caused by the presence of holes.

E3.3 Cylindrical Tubes

Closed thin-walled cylindrical tubes are economical sections for compression and torsional members because of their large ratio of radius of gyration to area, the same radius of gyration in all directions, and the large torsional rigidity. Like other cold-formed steel compression members, cylindrical tubes must be designed to provide adequate safety not only against overall column *buckling* but also against *local buckling*. It is well known that the classic theory

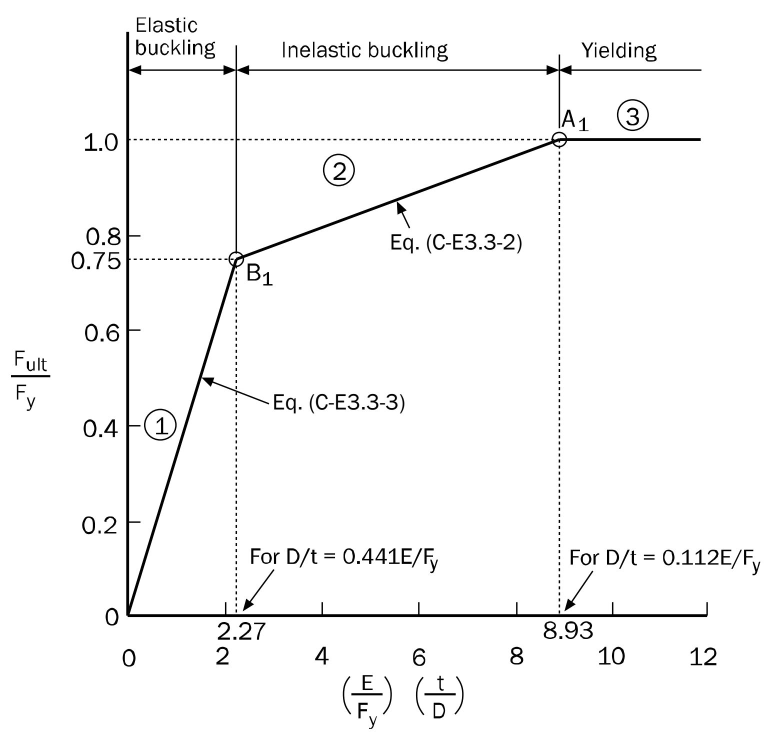


Figure C-E3.3-1 Critical Stress of Cylindrical Tubes for Local Buckling

of *local buckling* of longitudinally compressed cylinders overestimates the actual *buckling* strength, and that inevitable imperfections and residual *stresses* reduce the actual strength of compressed tubes radically below the theoretical value. For this reason, the design provisions for *local buckling* have been based largely on test results.

Considering the post-*buckling* behavior of the axially compressed cylinder and the important effect of the initial imperfection, the design provisions included in the AISI *Specification* were originally based on Plantema's graphic representation and the additional results of cylindrical shell tests made by Wilson and Newmark at the University of Illinois (Winter, 1970).

From the tests of compressed tubes, Plantema found that the ratio F_{ult}/F_y depends on the parameter $(E/F_y)(t/D)$, in which t is the wall *thickness*, D is the mean diameter of the tube, and F_{ult} is the ultimate *stress* or collapse *stress*. As shown in Figure C-E3.3-1, Line 1 corresponds to the collapse *stress* below the proportional limit, Line 2 corresponds to the collapse *stress* between the proportional limit and the *yield stress*, and Line 3 represents the collapse stress occurring at *yield stress*. In the range of Line 3, *local buckling* will not occur before yielding. In Ranges 1 and 2, *local buckling* occurs before the *yield stress* is reached. The cylindrical tubes should be designed to safeguard against *local buckling*.

Based on a conservative approach, the *Specification* specifies that when the D/t ratio is smaller than or equal to $0.112E/F_y$, the member shall be designed for yielding. This provision is based on point A_1, for which $(E/F_y)(t/D) = 8.93$.

When $0.112E/F_y < D/t < 0.441E/F_y$, the design of cylindrical tubes is based on the inelastic *local buckling* criteria. For the purpose of developing a design equation for inelastic *buckling*, point B_1 was selected to represent the proportional limit. For point B_1,

$$\left(\frac{E}{F_y}\right)\left(\frac{t}{D}\right) = 2.27, \qquad \frac{F_{ult}}{F_y} = 0.75 \tag{C-E3.3-1}$$

Using line A_1B_1, the maximum *stress* of cylindrical tubes can be represented by

$$\frac{F_{ult}}{F_y} = 0.037\left(\frac{E}{F_y}\right)\left(\frac{t}{D}\right) + 0.667 \tag{C-E3.3-2}$$

When $D/t \geq 0.441E/F_y$, the following equation represents Line 1 for elastic *local buckling stress*:

$$\frac{F_{ult}}{F_y} = 0.328\left(\frac{E}{F_y}\right)\left(\frac{t}{D}\right) \tag{C-E3.3-3}$$

The correlations between the available test data and Equations C-E3.3-2 and C-E3.3-3 are shown in Figure C-E3.3-2. The definition of symbol "D" was changed from "mean diameter" to "outside diameter" in the 1986 AISI *Specification* in order to be consistent with the general practice.

Specification Section E3.3 is only applicable to members having a ratio of outside diameter-to-wall *thickness*, D/t, not greater than $0.441E/F_y$ because the design of extremely thin tubes will be governed by elastic *local buckling* resulting in an uneconomical design. In addition, cylindrical tubes with unusually large D/t ratios are very sensitive to geometric imperfections.

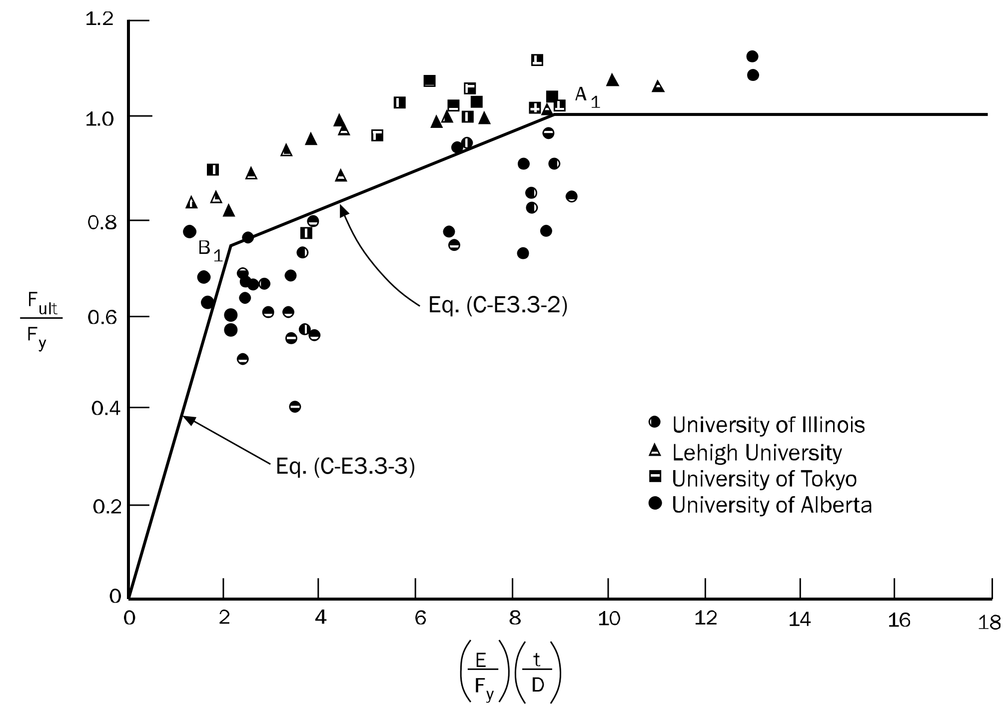


Figure C-E3.3-2 Correlation Between Test Data and AISI Criteria for Local Buckling of Cylindrical Tubes Under Axial Compression

When closed cylindrical tubes are used as concentrically loaded compression members, the *nominal axial strength* [*resistance*] is determined by the same equation as given in *Specification* Section E2, except that: (1) the nominal *buckling stress*, F_e, is determined only for *flexural buckling*, and (2) the *effective area*, A_e, is calculated by Equation C-E3.3-4:

$$A_e = [1-(1-R^2)(1-A_o/A)]A \quad \text{(C-E3.3-4)}$$

where

$$R = \sqrt{F_y / 2F_{cre}} \quad \text{(C-E3.3-5)}$$

$$A_o = \left(\frac{0.037}{DF_y / tE} + 0.667\right)A \le A \quad \text{(C-E3.3-6)}$$

A = area of the unreduced cross-section.

Equation C-E3.3-6 is used for computing the reduced area due to *local buckling*. It is derived from Equation C-E3.3-2 for inelastic *local buckling stress* (Yu and LaBoube, 2010).

In 1999, the coefficient, R, was limited to one (1.0) so that the *effective area*, A_e, will always be less than or equal to the *unreduced cross-sectional area*, A. To simplify the equations, $R = F_y/(2F_{cre})$ is used rather than $R = \sqrt{F_y/(2F_{cre})}$ as in the previous edition of the AISI *Specification*. The equation for the *effective area* is simplified to $A_e = A_o + R(A - A_o)$ as given in Equation E3.3-1 of the *Specification*.

E4 Distortional Buckling

The expression used for *distortional buckling* strength of columns is shown in Figure C-E1-1 and is discussed in Section E1. Based on experimental test data and on the success of the Australian/New Zealand code (see Hancock et al., 2001 for discussion and Hancock et al., 1994 for further details), the *distortional buckling* strength is limited to P_y instead of P_{ne}. This presumes that *distortional buckling* failures are independent of long-column behavior, i.e., little if any distortional-global interaction exists. See Appendix 2 for information on rational analysis methods for calculation of P_{crd}.

A. Members Without Holes

Distortional buckling is an *instability* that may occur in members with edge-stiffened *flanges*, such as lipped C- and Z-sections. As shown in Figure C-E4-1, this *buckling* mode is characterized by *instability* of the entire *flange*, as the *flange* along with the edge stiffener rotates about the junction of the *flange* and the *web*. The length of the *buckling* wave in *distortional buckling* is considerably longer than *local buckling*, and noticeably shorter than *flexural* or *flexural-torsional buckling*. The *Specification* provisions of Appendix 1 Section 1.3 partially account for *distortional buckling*, but research has shown that a separate *limit state* check is required (Schafer, 2002). Thus, in 2007, treating *distortional buckling* as a separate *limit state*, *Specification* Section E4 was added to address *distortional buckling* in columns and *Specification* Section F4 was added to address *distortional buckling* in beams.

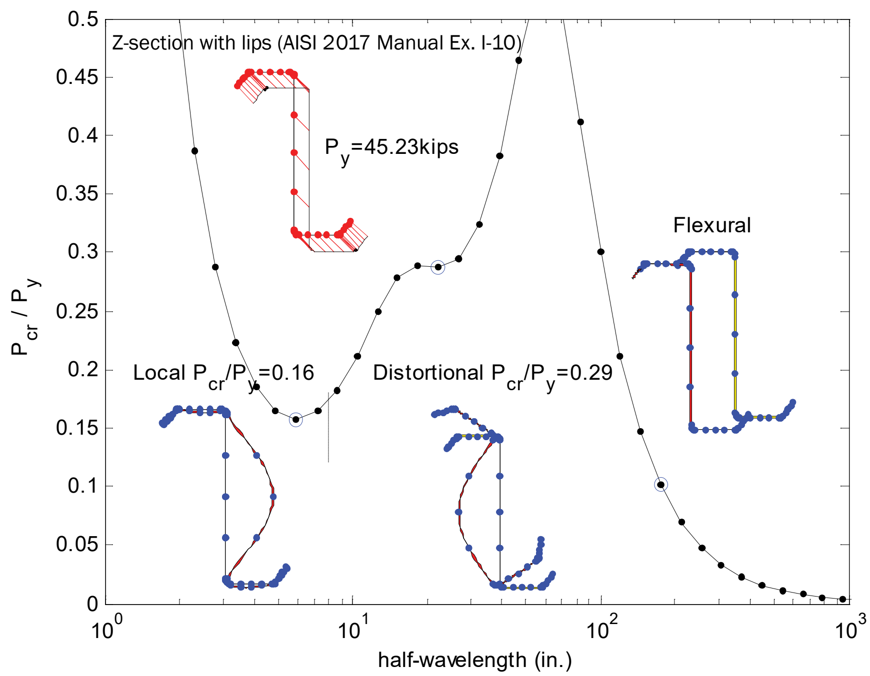


Figure C-E4-1 Rational Elastic Buckling Analysis of a Z-Section Under Compression Showing Local, Distortional, and Flexural Buckling Modes

Distortional buckling can also occur in cross-sections with intermediate stiffeners, as in Figure C-E4-2. This is treated as *local buckling* when using the *Effective Width Method*, but is recognized as *distortional buckling* when using the *Direct Strength Method*. The strength provisions of *Specification* Section E4 apply to this form of *distortional buckling* as well.

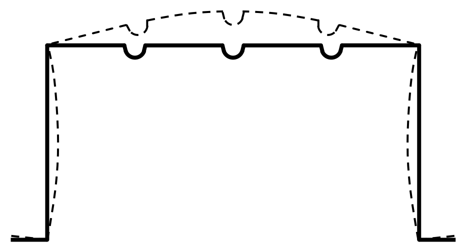


Figure C-E4-2 Distortional Buckling of Element With Intermediate Stiffeners

Determination of the *nominal strength* [*resistance*] in *distortional buckling* (*Specification* Equation E4-1) was validated by testing. Calibration of the *safety* and *resistance factors* for *Specification* Equation E4-1 is provided in *Commentary* Section E1. The Australian/New Zealand Specification (AS/NZS 4600) has used an expression similar to the previous form of *Specification* Equation E4-1, but yielding slightly less conservative strength predictions.

Distortional buckling is unlikely to control the strength of a column if: (a) the *web* is slender and triggers *local buckling* far in advance of *distortional buckling*, as is the case for many common C-sections, (b) edge stiffeners are sufficiently stiff and thus stabilize the *flange* (as is often the case for C-sections, but typically not for Z-sections due to the use of sloping lip stiffeners), (c) unbraced lengths are long and *flexural* or *flexural-torsional buckling* strength limits the capacity, or (d) adequate rotational restraint is provided to the *flanges* from attachments (panels, sheathing, etc.).

The primary difficulty in calculating the strength in *distortional buckling* is to efficiently estimate the elastic *distortional buckling* load, P_{crd}. Recognizing the complexity of this calculation, Appendix 2 provides two alternatives: (a) numerical solutions, or (b) analytical formulas for C- and Z-section members and any open section with a single *web* and *flanges* of the same dimension. See the Appendix 2 commentary for further discussion. The Appendix 2 commentary also provides a simplified analytical formula method that may be useful in preliminary design, and was specifically derived as a conservative simplification to *Specification* Section 2.3.3.1.

The provisions of this section are limited to *distortional buckling* slenderness, λ_d, less than 5 due to uncertainty of the strength predictions at high slenderness. The limits of applicability in *Specification* Section B4.1 already restrict slenderness to a practical range, but it is still possible to exceed the imposed slenderness limit. One rational approach to high slenderness applications is to limit the *distortional buckling* strength using Equation C-E4-1 which approximates the original form of *DSM* equation approaching zero at high slenderness.

$$P_{nd} \leq \frac{P_y}{\lambda_d^{1.2}} \qquad \text{(C-E4-1)}$$

B. Members With Holes

For columns with holes, the *DSM* strength prediction expression was modified to limit the maximum strength to the capacity of the net cross-section with a linear transition to the *distortional buckling* strength curve (Moen and Schafer, 2009a) which was further refined to a single smooth curve by Glauz (2023). Figure C-E4-3 compares the *distortional buckling* strength prediction for columns with holes having different ratios of P_{ynet}/P_y. As slenderness increases, the prediction transitions from P_{ynet} to the strength curve used for columns without holes (P_{ynet}/P_y=1). The transition is implemented to reflect the change in failure mode as slenderness increases, from yielding at the net section to *distortional buckling*.

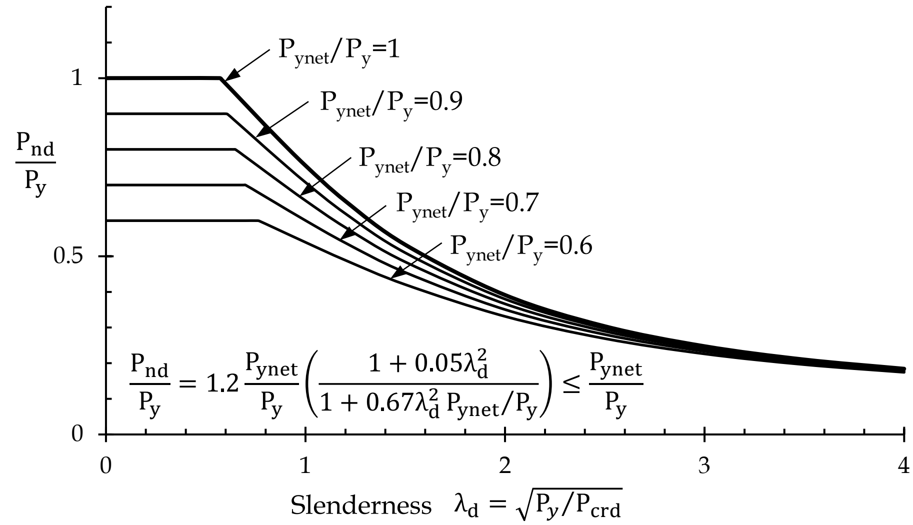


Figure C-E4-3 DSM Distortional Buckling Strength Curves for Columns with Holes

The extension of the *DSM* approach to columns with holes utilizes the elastic *buckling* properties of a cold-formed steel column ($P_{cr\ell}$, P_{crd}, and P_{cre}), including the influence of holes to predict ultimate strength. In most cases, holes decrease the elastic *buckling* properties, $P_{cr\ell}$, P_{crd}, and P_{cre}, which increases a column's local (λ_ℓ), distortional (λ_d) and global (λ_c) slenderness and lowers the predicted strength. Simplified methods for predicting $P_{cr\ell}$, P_{crd}, and P_{cre} including holes are presented in Appendix 2. Alternatively, full finite element elastic eigen-*buckling* analysis can be performed.

When determining P_{nd} for members with holes, P_{crd} is required to be determined including the influence of holes. Appendix 2 Section 2.3.3.1 specifies $P_{crd} = A_gF_{crd}$. For members with holes, only F_{crd} must include the influence of holes. The cross-sectional area should not be modified. Thus, for members with holes, P_{crd} = A_g(gross area ignoring holes) × F_{crd}(elastic *distortional buckling stress* including the influence of holes).

F. MEMBERS IN FLEXURE

This chapter provides the design requirements for flexural members. If the member is subject to biaxial bending, the provisions of Chapter H are used to evaluate the combined effects for bending about two principal axes or for Z-sections bending about the axes parallel and perpendicular to the *web*. For a flexural member constrained to bend about only one axis, the provisions of Chapter F can be applied regardless of the bending axis because unsymmetric bending and *lateral-torsional buckling* are prevented.

In 2022, the design provisions for bearing stiffeners were moved to Chapter G.

F1 General Requirements

In general, a common *nominal strength* [*resistance*] equation is provided in the *Specification* for a given *limit state* with a required *safety factor* (Ω) for *Allowable Strength Design* (*ASD*) and a *resistance factor* (ϕ) for *Load and Resistance Factor Design* (*LRFD*) or *Limit States Design* (*LSD*). Design provisions that are applicable to a specific country are provided in the corresponding lettered appendix.

The thin-walled nature of cold-formed beams complicates behavior and design. Elastic *buckling* analysis reveals at least three *buckling* modes: *local*, *distortional*, and *lateral-torsional buckling* (for members in strong-axis bending) that must be considered in design. Bending strengths of flexural members are determined by considering *yielding* and global (*lateral-torsional*) *buckling* in *Specification* Section F2, *local buckling* interaction with global *buckling* in *Specification* Section F3, and *distortional buckling* in *Specification* Section F4. The member flexural strength is the least of the strengths after considering the above *buckling* modes.

Like column design, two approaches can be used in beam design: *Effective Width Method* (*EWM*) and *Direct Strength Method* (*DSM*). The *EWM* traditionally addressed *local* and global *buckling*. In 2004, the *distortional buckling* strength prediction using *DSM* was adopted.

In considering flexural member *yielding* and global *buckling*, the *DSM* follows a similar practice as the *EWM*. The *Effective Width Method* provides the *lateral-torsional buckling* strength in terms of an extreme fiber compressive *stress*, F_n (*Specification* Equation F2.1-1) based on the *yield stress* and elastic *buckling stress*. In the *DSM*, the *lateral-torsional buckling* strength is determined using the plastic moment and elastic *buckling* moment. The *DSM* emerged through the combination of more refined methods for *local* and *distortional buckling* prediction, improved understanding of the post-*buckling* strength and imperfection sensitivity in *distortional buckling*, and the relatively large amount of available experimental data.

In the *Effective Width Method*, for beams that are not fully braced and locally unstable, beam strength is calculated by multiplying the predicted *stress* for failure in *lateral-torsional buckling*, F_n, by the effective section modulus, S_e, determined at *stress* F_n. This accounts for *local buckling* reductions in the *lateral-torsional buckling* strength (i.e., local-global interaction). In the *DSM*, this calculation is broken into two parts: the *lateral-torsional buckling* strength without any reduction for *local buckling* (M_{ne}), and the strength considering local-global interaction ($M_{n\ell}$).

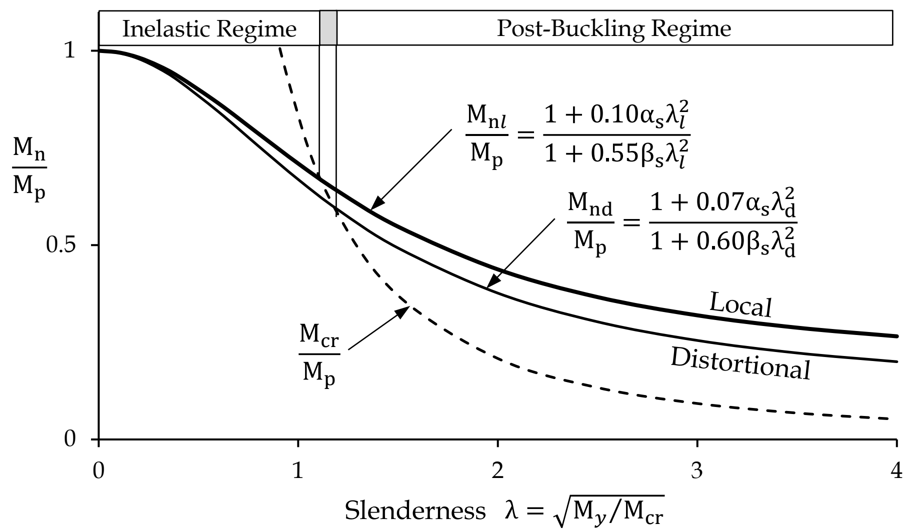


Figure C-F1-1 Local and Distortional Direct Strength Curves for a Beam Braced Against Lateral-Torsional Buckling ($M_{ne} \geq M_y$, k_s=1.2, α_s=1, β_s=1)

The *DSM* strength curves for *local* and *distortional buckling* of a beam fully braced against *lateral-torsional buckling* are presented in Figure C-F1-1 and compared to the critical elastic *buckling* curve. The post-*buckling* reserve for the local mode is predicted to be greater than that of the distortional mode.

The *DSM* equations were first established using the form of the Winter plate *buckling* formula as shown in Equation C-F1-1. This form worked well for symmetrical sections but was found to be less accurate for flexural members unsymmetric about the axis of bending.

$$\frac{M_n}{M_y} = \left(1 - \frac{c}{\lambda^\eta}\right)\frac{1}{\lambda^\eta} = \left[1 - c\left(\frac{M_{cr}}{M_y}\right)^{0.5\eta}\right]\left(\frac{M_{cr}}{M_y}\right)^{0.5\eta} \quad \text{(C-F1-1)}$$

Investigation by Glauz and Schafer (2022) identified general modifications to this equation to improve the accuracy for unsymmetric sections. This work included the development of a new form of strength curves to capture strength prediction throughout the entire range of slenderness including inelastic reserve strength (M_n>M_y). Equation C-F1-2 shows this general form, which more directly accommodates the necessary modifiers for unsymmetric sections. For consistency and simplicity, this form was adopted for all *DSM* strength curves in 2024.

$$\frac{M_n}{M_p} = \frac{1 + a\lambda^2}{1 + b\lambda^2} = \frac{M_{cr} + aM_y}{M_{cr} + bM_y} \quad \text{(C-F1-2)}$$

If members are laterally supported, *local buckling* strength is proportioned to the nominal section strength (*Specification* Section F3.1 or F3.2). Since *distortional buckling* has an intermediate *buckling* half wavelength, *distortional buckling* still needs to be considered even for braced members. See the *Direct Strength Method Design Guide* (AISI, 2006) for detailed discussion and design examples. If they are laterally unbraced, the *limit state* is *lateral-torsional buckling* and possible interaction with *local buckling* (*Specification* Sections F2 and F3).

The extension of the *DSM* approach to beams with holes utilizes the elastic *buckling* properties of a cold-formed steel beam ($M_{cr\ell}$, M_{crd}, and M_{cre}) including the influence of holes to predict ultimate strength. In most cases, holes decrease $M_{cr\ell}$, M_{crd}, and M_{cre}; this increases the beam's local (λ_ℓ), distortional (λ_d) and global (λ_c) slenderness and lowers the predicted strength. Simplified methods for predicting $M_{cr\ell}$, M_{crd}, and M_{cre} including holes are presented in Appendix 2. Alternatively, full finite element elastic eigen-*buckling* analysis can be performed.

The calibration of the *Effective Width Method* was reviewed in the *Commentary* of the 1991 edition of the *Specification*. A brief discussion of the *DSM* calibration is provided herein. The reliability of the *DSM* beam provisions was determined using test data defined by the limits of Section B4.1 and the provisions of Section K2 of the *Specification*.

Based on a target reliability, β_o, of 2.5, a *resistance factor*, ϕ, of 0.90 was calculated for all of the investigated beams. Based on this information, the *safety* and *resistance factors* of *Specification* Chapter F were determined for the prequalified members. The *safety factor*, Ω, is back-calculated from ϕ at an assumed dead-to-live *load* ratio of 1 to 5. Since the range of prequalified members is relatively large, extensions of the *DSM* to geometries outside the prequalified set are allowed. However, given the uncertain nature of this extension, increased *safety factors* and reduced *resistance factors* are applied in that case, per the *rational engineering analysis* provisions of Section A1.2.6(c) of the *Specification*.

The development and calibration of the *DSM* provisions for beams with holes were performed with a simulation database as reported in Moen and Schafer (2009a) and a set of 12 beam experiments summarized in Moen, et al. (2012). Note that the simulations and experiments only considered lipped Cee cross-sections with discrete *web* holes. However, the philosophy of employing elastic *buckling* parameters ($M_{cr\ell}$, M_{crd}, M_{cre}) to predict the ultimate strength of cold-formed steel beams with holes, validated in Moen and Schafer (2009a), is assumed to hold true for other cross-sectional shapes.

Resistance factors for beams with holes were calculated by *limit state* with Section K2 of the main *Specification*. Based on a target reliability, β_o, of 2.5, the *resistance factor*, ϕ, was calculated with the simulation database as 0.95 for laterally braced beams predicted to fail from *local buckling*. For beams predicted to experience a *distortional buckling* failure mode, ϕ was calculated with the simulation database as 0.91 and with the Moen, et al. (2012) experiments as 0.94. The new form of *DSM* equations introduced by Glauz and Schafer (2022) for *local* and *distortional buckling* of beams further improved the reliability of the strength curves for members with holes (Glauz, 2023).

Many of the flexural strength provisions in Chapter F utilize the plastic moment, M_p, where all longitudinal *stresses* in the cross-section are at yield in tension or compression. M_p is determined using the plastic section modulus, Z, which is the sum of the first moments of area of the tension and compression regions relative to the plastic neutral axis. For sections bending about an axis of symmetry, the plastic neutral axis is the axis of symmetry. Otherwise, the plastic neutral axis must be located such that half the cross-sectional area exists on each side.

$$M_p = ZF_y \tag{C-F1-3}$$

For bending about $\bar{x}$-axis and $\bar{y}$-axis:

$$Z_x = \int y_t dA + \int y_c dA = \frac{1}{2}A\bar{y}_t + \frac{1}{2}A\bar{y}_c \tag{C-F1-4}$$

$$Z_y = \int x_t dA + \int x_c dA = \frac{1}{2}A\bar{x}_t + \frac{1}{2}A\bar{x}_c \tag{C-F1-5}$$

where $\bar{y}_t, \bar{y}_c, \bar{x}_t, \bar{x}_c$ are the y and x distances from the plastic neutral axis to the centroid of areas in tension or compression.

Formulas are given below for plastic section moduli of common shapes using sharp corners as shown in Figure C-F1-2, where h, b, and d are centerline dimensions, and t is the *thickness*. These formulas are based on thin-wall cross-sections applicable to h/t>40 and b/t>20. Rounded corners will reduce the plastic section modulus due to the reduced areas at the corners. These formulas apply to unlipped sections by setting d = 0. Equations C-F1-8 and C-F1-13 approximate the plastic neutral axis location as the centerline of the *web* element (h dimension), although the actual location is slightly off centerline but still within the material *thickness*.

For C-Sections where d < h/2:

$$Z_x = ht\left[\frac{h}{4} + b + d\left(1 - \frac{d}{h}\right)\right] \tag{C-F1-6}$$

$$Z_y = hbt\left[\frac{1}{2} + \frac{b}{2h} - \frac{h}{8b} + \frac{d}{h}\left(1 + \frac{h}{2b} - \frac{d}{2b}\right)\right] \text{ for h < 2b+2d} \tag{C-F1-7}$$

$$Z_y = bt(b + 2d) \text{ for h} \geq \text{2b+2d} \tag{C-F1-8}$$

For Z-Sections where d < h/2:

$$Z_x = ht\left[\frac{h}{4} + b + d\left(1 - \frac{d}{h}\sin\theta\right)\right] \tag{C-F1-9}$$

$$Z_y = bt\left[b + d\left(2 + \frac{d}{b}\cos\theta\right)\right] \tag{C-F1-10}$$

For Hat-Sections where d < h/2+b:

$$Z_x = ht\left[\frac{h}{4} + b + d\left(1 + \frac{d}{h}\right)\right] \tag{C-F1-11}$$

$$Z_y = hbt\left[\frac{1}{2} + \frac{b}{2h} - \frac{h}{8b} + \frac{d}{h}\left(1 + \frac{h}{2b} - \frac{d}{2b}\right)\right] \text{ for h < 2b+2d} \tag{C-F1-12}$$

$$Z_y = bt(b + 2d) \text{ for h} \geq \text{2b+2d} \tag{C-F1-13}$$

For Angle Sections where d < b:

$$Z_x = \frac{t}{\sqrt{2}}(b^2 + 2bd - d^2) \tag{C-F1-14}$$

$$Z_y = \frac{t}{2\sqrt{2}}(b + d)^2 \tag{C-F1-15}$$

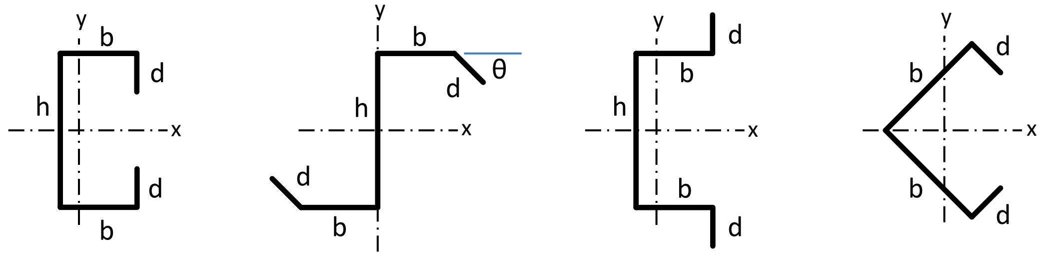


Figure C-F1-2 Centerline Dimensions for C, Z, Hat and Angle Sections

F2 Yielding and Global (Lateral-Torsional) Buckling

The bending capacity of flexural members can be limited by *yielding* or the *lateral-torsional buckling* strength of the member depending on the member's lateral unbraced length. The design provisions for determining the *nominal lateral-torsional buckling strength* [*resistance*] are given in *Specification* Section F2.

F2.1 Effective Width Method

In this section, the *limit states* of *yielding* and global (lateral-torsional) *buckling* are discussed. In 2022, the elastic global *buckling* equations were moved from *Specification* Section F2.1 to Appendix 2 to consolidate all elastic *buckling* provisions in the appendix.

A. *Initiation of Yielding*

For compact beams with short unbraced lengths, the member may fail by *yielding*. The *yield* moment is determined by Equation C-F2.1-1:

$$M_y = S_f F_y \qquad \text{(C-F2.1-1)}$$

where S_f is the unreduced elastic section modulus, and F_y is the *yield stress*.

B. *Lateral-Torsional Buckling*

Section 2.3.1.2 of *Commentary* Appendix 2 covers elastic *lateral-torsional buckling* of flexural members in detail for various types of sections, including the impact of moment gradients.

The equations for elastic *buckling* of cold-formed steel members in bending are applicable when the computed theoretical *buckling stress* is less than or equal to the proportional limit. When the computed *stress* exceeds the proportional limit, the beam behavior will be governed by inelastic *buckling*.

The following equation for determining the inelastic *buckling stress*, F_n, was adopted in the 1996 edition of the *Specification*:

$$F_n = \frac{10}{9} F_y \left(1 - \frac{10F_y}{36F_{cre}} \right) \qquad \text{(C-F2.1-2)}$$

where F_{cre} is the elastic critical *lateral-torsional buckling stress*.

As specified in *Specification* Section F2.1, *lateral-torsional buckling* is considered to be elastic up to a *stress* equal to $0.56F_y$. The inelastic region is defined by a Johnson parabola from $0.56F_y$ to $(10/9)F_y$ at an unsupported length of zero. The (10/9) factor is based on the partial plastification of the section in bending (Galambos, 1963) and is a conservative approximation of the shape factor (M_p/M_y). A flat plateau is created by limiting the maximum *stress* to F_y, which enables the calculation of the maximum unsupported length for which there is no *stress* reduction below F_y due to lateral-torsional *instability*. This maximum unsupported length can be calculated by setting F_n equal to F_y in Equation C-F2.1-2.

This inelastic *lateral-torsional buckling* curve has been confirmed by research in beam-columns (Peköz and Sumer, 1992) and wall studs (Niu and Peköz, 1994).

F2.1.1 Inelastic Reserve Strength

Prior to 1980, the inelastic reserve capacity of beams was not included in the *Specification* because most cold-formed steel shapes have large width-to-*thickness* ratios that are considerably in excess of the limits required by plastic design.

In the 1970s and early 1980s, research work on the inelastic strength of cold-formed steel beams was carried out by Reck, Peköz, Winter, and Yener at Cornell University (Reck, Peköz and Winter, 1975; Yener and Peköz, 1985a, 1985b). These studies showed that the *inelastic reserve* strength of cold-formed steel beams due to partial plastification of the cross-section and the moment redistribution of statically indeterminate beams can be significant for certain shapes. With proper care, this reserve strength can be utilized to achieve more economical design of such members.

In order to utilize the *available inelastic reserve strength* [*factored resistance*] of certain cold-formed steel beams, design provisions based on the partial plastification of the cross-section were added in the 1980 edition of the *Specification*. The same provisions are retained in this edition of the *Specification*. According to Section F2.1.1 of the *Specification*, the *nominal section strength* [*resistance*], M_n, of those beams satisfying certain specific limitations can be determined on the basis of the inelastic reserve capacity with a limit of $1.25M_y$, where M_y is the effective *yield moment*. The ratio of M_n/M_y represents the inelastic reserve strength of a beam cross-section.

The *nominal moment* [*resistance*], M_n, is the maximum bending capacity of the beam by considering the inelastic reserve strength through partial plastification of the cross-section. The inelastic *stress* distribution in the cross-section depends on the maximum strain in the compression *flange*, ε_{cu}. Based on the Cornell research work on hat sections having stiffened compression *flanges* (Reck, Peköz and Winter, 1975), the AISI design provision limits the maximum compression strain to be $C_y\varepsilon_y$, where C_y is a compression strain factor determined using the equations provided in *Specification* Section F2.1.1 (a) as shown in Figure C-F2.1.1-1.

On the basis of the maximum compression strain, ε_{cu}, allowed in the *Specification*, the neutral axis can be located using Equation C-F2.1.1-1 and the *nominal moment* [*resistance*] M_n can be determined using Equation C-F2.1.1-2:

$$\int \sigma dA = 0 \tag{C-F2.1.1-1}$$

$$\int \sigma y dA = M_n \tag{C-F2.1.1-2}$$

where σ is the *stress* in the cross-section, and y is the distance measured from the neutral axis to the *yield stress*.

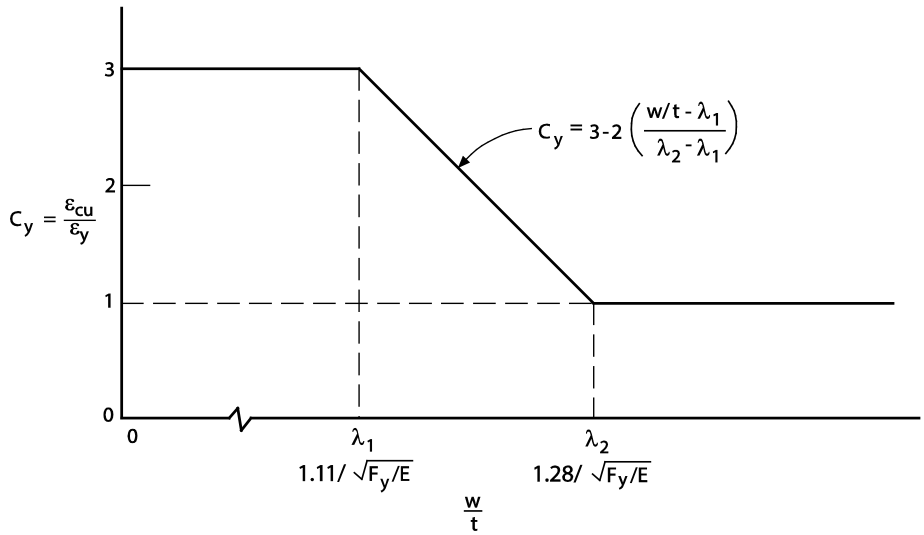


Figure C-F2.1.1-1 Factor C_y for Stiffened Compression Elements Without Intermediate Stiffeners

The calculation of M_n based on inelastic reserve capacity is illustrated in Part I of the AISI *Cold-Formed Steel Design Manual* (AISI, 2017) and the textbook by Yu and LaBoube (2010).

In 2001, the shear force upper limit was clarified. The *stress* upper limit is $0.35F_y$ for *ASD* and $0.6F_y$ for *LRFD* and *LSD* in the *Specification*.

Additional equations were provided in *Specification* Section F2.1.1(b) since 2004 for determining the *nominal moment strength* [*resistance*], M_n, based on inelastic reserve capacity, for sections containing *unstiffened compression elements* under *stress* gradient. Based on research by Bambach and Rasmussen (2002b, 2002c) on I- and plain channel sections in minor axis bending, a compression strain factor, C_y, determines the maximum compressive strain on the unstiffened element of the section. The C_y values are dependent on the *stress* ratio, ψ, and slenderness ratio, λ, of the unstiffened element, determined in accordance with Section 1.2.2(a) of the *Specification*.

F2.2 Direct Strength Method

The original *DSM lateral-torsional buckling* strength followed the *stress*-based form of the *EWM*, with the inelastic *buckling* strength given by Equation C-F2.2-1.

$$M_{ne} = \frac{10}{9}M_y\left(1-\frac{(10/9)M_y}{4M_{cre}}\right) \le M_y \qquad \text{(C-F2.2-1)}$$

The 2012 *Specification* introduced inelastic reserve strength (M_{ne}>M_y) with a linear transition between the yield moment M_y and the plastic moment M_p in the slenderness range from λ_e=0.60 to λ_e=0.23 as shown in Figure C-F2.2-1.

In the 2016 *Specification*, the *DSM* provisions were incorporated into the main *Specification* and the *lateral-torsional buckling* strength utilized the design *stress* F_n from the *EWM*. This *stress* is specifically for the extreme compression fiber to provide the proper element *stresses* for *effective width* calculations. For members unsymmetric about the axis of bending, this approach resulted in discontinuities for strengths above M_y as documented by Glauz (2023).

In 2024, the provisions reverted to the 2012 *Specification* equation, except that the $\frac{10}{9}M_y$ term, which was a conservative estimate of the plastic moment, is replaced with M_p. This enables the inelastic *buckling* strength equation to be used above the first yield, eliminating the need for a separate transition equation. Figure C-F2.2-1 compares the current strength curve to S100-12 and AISC (2005, 2010a, 2016a) provisions.

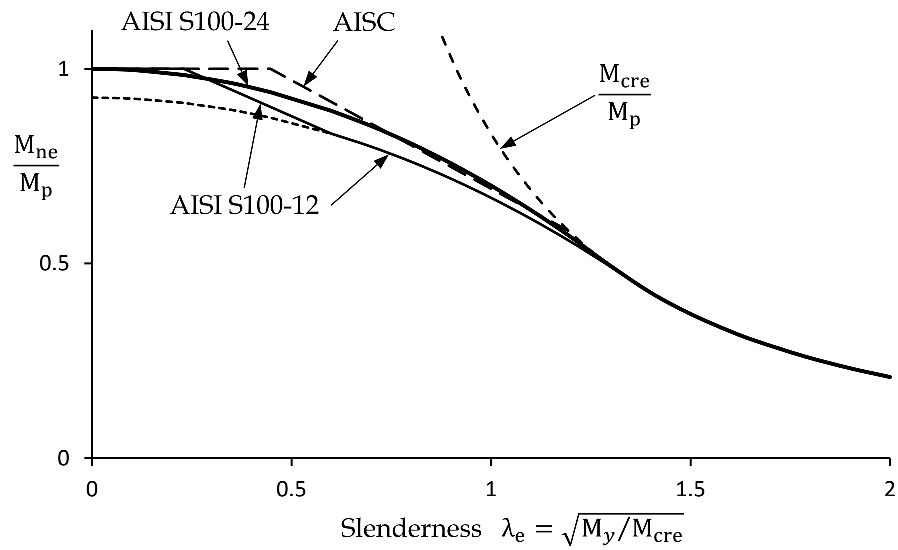


Figure C-F2.2-1 Direct Strength Curves for Lateral-Torsional Buckling

F2.3 Cylindrical Tubes

The discussion on cylindrical tube behavior and *buckling* modes is provided in *Commentary* Section E3.3. It should be noted that the design provisions of *Specification* Sections E3.3, F2.3, and F3.3 are applicable only for members having a ratio of outside diameter-to-wall *thickness*, D/t, not greater than $0.441E/F_y$ because the design of extremely thin tubes will be governed by elastic *local buckling*, resulting in an uneconomical design. In addition, cylindrical tubes with unusually large D/t ratios are very sensitive to geometric imperfections.

For thick cylinders in bending, the initiation of *yielding* does not represent a failure condition as is generally assumed for axial loading. Failure is at the plastic moment capacity, which is at least 1.29 times the moment at first yielding. *Specification* Equation F2.3-1 uses an assumed minimum shape factor of 1.25, which is a slight reduction to limit the maximum bending *stress* to $0.75F_y$, a value typically used for solid sections in bending for the *ASD* method. The reduction also brings the criteria closer to a lower bound for inelastic *local buckling*, which is discussed in *Commentary* Section F3.3.

F3 Local Buckling Interacting With Yielding and Global Buckling

F3.1 Effective Width Method

For locally unstable beams, the interaction of the *local buckling* of the compression elements and overall *lateral-torsional buckling* of members may result in a reduction of the *lateral-torsional buckling* strength of the member. The effect of *local buckling* on the critical moment is considered by Equation F3.1-1 of the *Specification* by using the elastic section modulus, S_e, based on an effective section.

Using the *nominal lateral-torsional buckling strength* [*resistance*] determined in accordance with *Specification* Equation F3.1-1 with a *resistance factor* of ϕ_b = 0.90, the reliability indexes of β vary from 2.4 to 3.8 for the *LRFD* method.

For locally stable beams, the nominal moment, M_n, of the cross-section is the effective *yield moment*, M_y, determined on the basis of the *effective areas* of *flanges* and the beam *web*. The *effective width* of the compression *flange* and the effective depth of the *web* can be computed from the design equations given in Appendix 1 of the *Specification*.

Similar to the design of hot-rolled steel shapes, the *yield moment*, M_y, of a cold-formed steel beam is defined as the moment at which an outer fiber (tension, compression, or both) first attains the *yield stress* of the steel. This is the maximum bending capacity to be used in elastic design. Figure C-F3.1-1 shows several types of *stress* distributions for *yield moment* based on different locations of the neutral axis. For balanced sections (Figure C-F3.1-1(a)), the outer fibers in the compression and tension *flanges* reach the *yield stress* at the same time. However, if the neutral axis is eccentrically located, as shown in Figures C-F3.1-1(b) and (c), the initial *yielding* takes place in the tension *flange* for case (b) and in the compression *flange* for case (c).

Accordingly, the *nominal section strength* [*resistance*] for initiation of *yielding* is calculated by using Equation C-F3.1-1:

$$M_n = S_e F_y \tag{C-F3.1-1}$$

where

F_y = Design *yield stress*

S_e = Elastic section modulus of the effective section calculated with the extreme compression or tension fiber at F_y.

For cold-formed steel design, S_e is usually computed by using one of the following two cases:

1. If the neutral axis is closer to the tension than to the compression *flange*, the maximum *stress* occurs in the compression *flange*, and therefore the plate slenderness ratio λ and the *effective width* of the compression *flange* are determined by the w/t ratio and f = F_y. Of course, this procedure is also applicable to those beams for which the neutral axis is located at the mid-depth of the section.
2. If the neutral axis is closer to the compression than to the tension *flange*, the maximum *stress* of F_y occurs in the tension *flange*. The *stress* in the compression *flange* depends on the location of the neutral axis, which is determined by the *effective area* of the section. The latter cannot be determined unless the compressive *stress* is known. The closed-form solution of this type of design is possible but would be a very tedious and complex procedure. It is therefore customary to determine the sectional properties of the section by successive approximation.

Prior to the 2008 edition of the AISI *Specification*, the *design flexural strength* [*factored resistance*], $\phi_b M_n$, employed different ϕ_b factors depending on the compression *flange*. Based on the 1991 edition of the *Specification* and the work of Hsiao, Yu and Galambos (1988a), unstiffened *flanges* were specified at ϕ_b= 0.90 and edge-stiffened or stiffened *flanges* at ϕ_b = 0.95 (*ASD* used one Ω factor for all cases). Examination of more recently available test data (Schafer and Trestain, 2002; Yu and Schafer, 2003) and consideration of the fact that the higher ϕ_b existed in part due to inelastic reserve strength, which is already addressed in *Specification* Section F2.1.1, a uniform ϕ_b= 0.90 was adopted for all members. This change also removed a conflict with the ϕ_b factors adopted

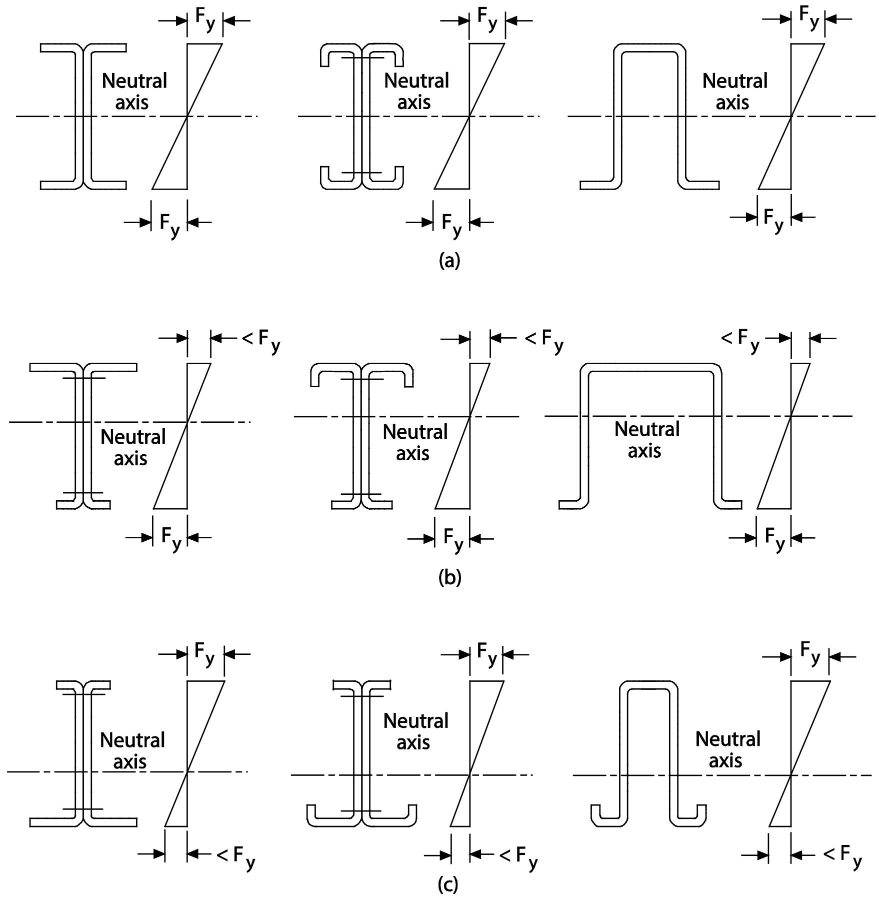


Figure C-F3.1-1 Stress Distribution for Yield Moment: (a) Balanced Sections, (b) Neutral Axis Close to Compression Flange, and (c) Neutral Axis Close to Tension Flange

in 2007 for the *Specification*, when the member is fully effective.

For members with holes, the elements beside the holes are considered as unstiffened elements. The effective widths are then determined in accordance with Appendix 1 of the *Specification*. The *buckling stress*, F_{cre}, for members with holes determined in accordance with Appendix 2 is a gross section compressive *stress*. Therefore, this *stress* must be multiplied by S_{fc}/S_{fcnet} to establish the compressive *stress* at the net section.

F3.1.1 Local Inelastic Reserve Strength

Specification Section F2.1.1 should be used for determining the inelastic reserve strength, as applicable. The discussion of inelastic reserve strength has been provided in *Commentary* Section F2.1.1.

F3.2 Direct Strength Method

In the *Direct Strength Method* (*DSM*), the *local buckling* strength may involve local-global interaction. This interaction is only considered if the *lateral-torsional buckling strength* (M_{ne}) is less than the *yield moment* (M_y); otherwise, the lesser of M_{ne} and $M_{n\ell}$ controls, both of which may include inelastic reserve strength. The calibration of the *DSM* for beams was discussed in

Commentary Section F1.

The expression used for *local buckling* strength of beams is shown in Figure C-F1-1 and is discussed in Section F1. The use of the *DSM* for *local buckling* and the development of the original empirical strength expression are given in Schafer and Peköz (1998). The current strength expression was developed by Glauz and Schafer (2022) and adopted in 2024 to improve the strength predictions for members unsymmetric about the axis of bending and members that lose overall depth in post-*buckling*.

The provisions of this section are limited to *local buckling* slenderness, λ_ℓ, less than 5 due to uncertainty of the strength predictions at high slenderness. The limits of applicability in *Specification* Section B4.1 already restrict slenderness to a practical range, but it is still possible to exceed the imposed slenderness limit. One rational approach to high slenderness applications is to limit the *local buckling* strength using Eq. C-F3.2-1 which approximates the original form of *DSM* equation approaching zero at high slenderness and accommodates the factors k_s and β_s.

$$M_{n\ell} \leq \frac{k_s \overline{M}_{ne}}{1.2\beta_s \lambda_\ell^{0.8}} \tag{C-F3.2-1}$$

Another rational approach for approximating the strength of slender sections is to ignore the large flat widths and exclude most of the material from those elements. This will reduce the *yield moment* M_y and increase the critical elastic *local buckling* moment $M_{cr\ell}$, resulting in a lower slenderness which may allow use of the *Specification* equations.

Members unsymmetric about the axis of bending exhibit greater post-*buckling stress* redistribution, particularly as plastification ensues. The *local buckling* strength curve (*Specification* Equation F3.2-1) is a single equation referenced from the maximum strength of k_sM_{ne} (M_p for a fully braced beam), where k_s is the shape factor (M_p/M_y). This equation utilizes two modifiers for the strength impact of *stress* redistribution: section factor α_s and symmetry factor β_s. The influence of these factors is illustrated in Figure C-F3.2-1, where α_s=1 and β_s=1 for common flexural members.

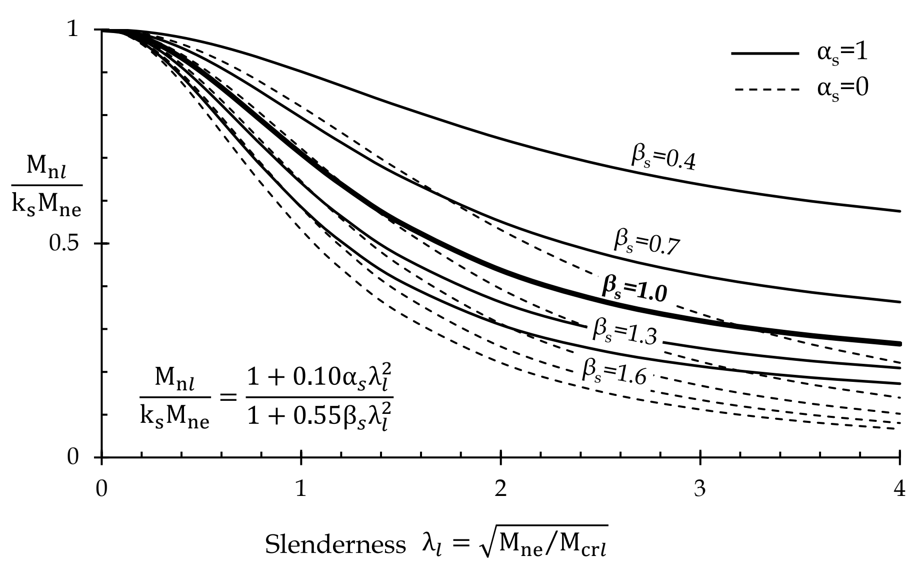


Figure C-F3.2-1 DSM for Local Buckling of Beams ($M_{ne} \leq M_y$)

For sections symmetric about the axis of bending, the symmetry factor β_s is 1. For sections with the first yield in tension, β_s is less than 1 and the post-*buckling stress* redistribution is favorable to the flexural strength. For sections with the first yield in compression, β_s is greater than 1 and the post-*buckling stress* redistribution is unfavorable to the flexural strength. Figure C-F3.2-2 shows examples of β_s for sections unsymmetric about the axis of bending.

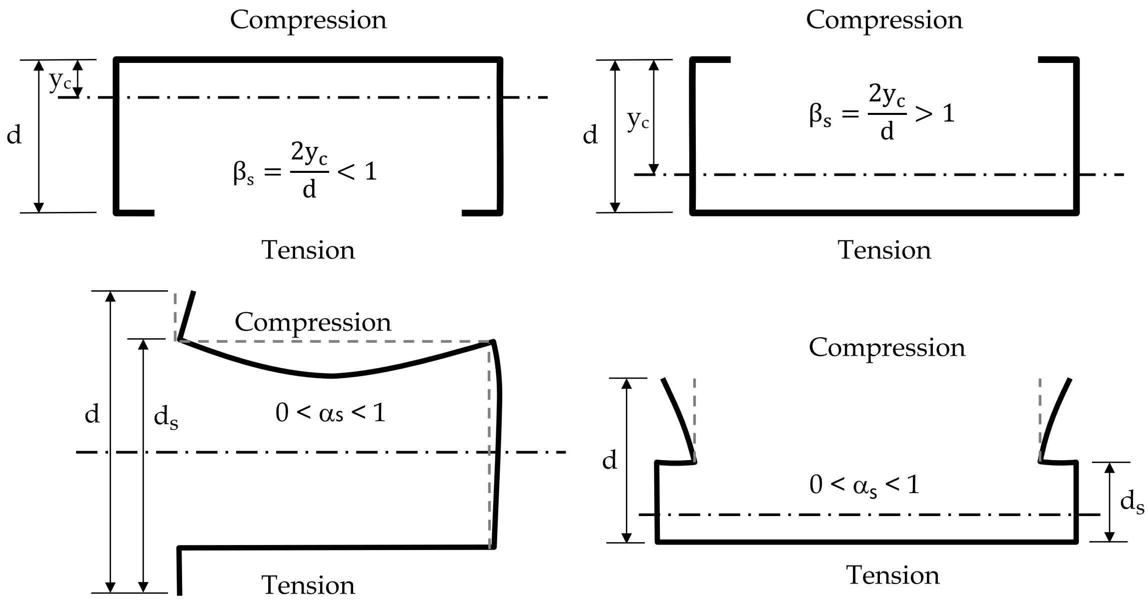


Figure C-F3.2-2 DSM Factors Influencing Local Buckling Strength

For open flexural members with free edges in compression, the *local buckling* flexural strength drops more rapidly as slenderness increases due to additional *stress* redistribution. The section factor α_s is used to account for this strength reduction. Cross-sections having fold lines (corners) between the extreme tension and compression fiber may have a flexural strength represented by α_s between 0 and 1 as shown in Figure C-F3.2-2. The rational approach suggested by Glauz and Schafer (2022) is to linearly interpolate α_s as d_s/d, where d_s is the depth of the cross-section excluding unstiffened elements and d is the overall depth.

Sections that utilize α_s=0 may still have greater *DSM* strengths than predicted by the *EWM*. Figure C-F3.2-3 shows the strength comparison for a track section in minor axis bending. The *DSM* strength for this section is up to 80% higher than the *EWM* strength because the shape factor (M_p/M_y) is about 1.8. Inelastic reserve for the *EWM* does provide some additional strength, up to 25% as limited by *Specification* Section F2.2.1. The *DSM* strength for this case is shown to be more accurate as documented by Glauz and Schafer (2022).

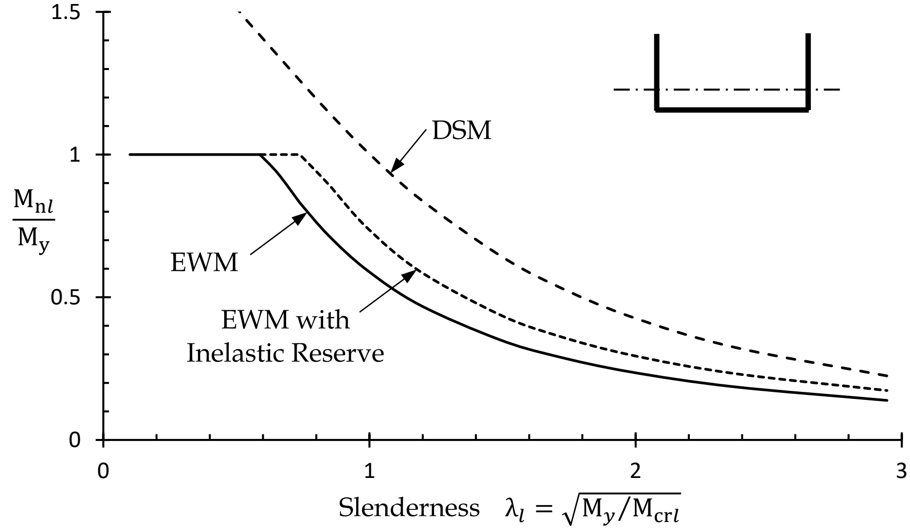


Figure C-F3.2-3 Comparison of EWM and DSM for Track in Minor Axis Bending

For beams with holes, the *DSM* strength prediction expression was modified to limit the maximum strength to the capacity of the net cross-section (Moen and Schafer, 2009a) and was further refined with a transition curve by Glauz (2023). Figure C-F3.2-4 compares the *local buckling* strength prediction for beams with holes having different ratios of $M_{pnet}/(k_s M_{ne})$. As local slenderness increases, the prediction transitions from M_{pnet} to the strength curve used for beams without holes. The transition is implemented to reflect the change in failure mode as slenderness increases, from yielding at the net section to *local buckling*.

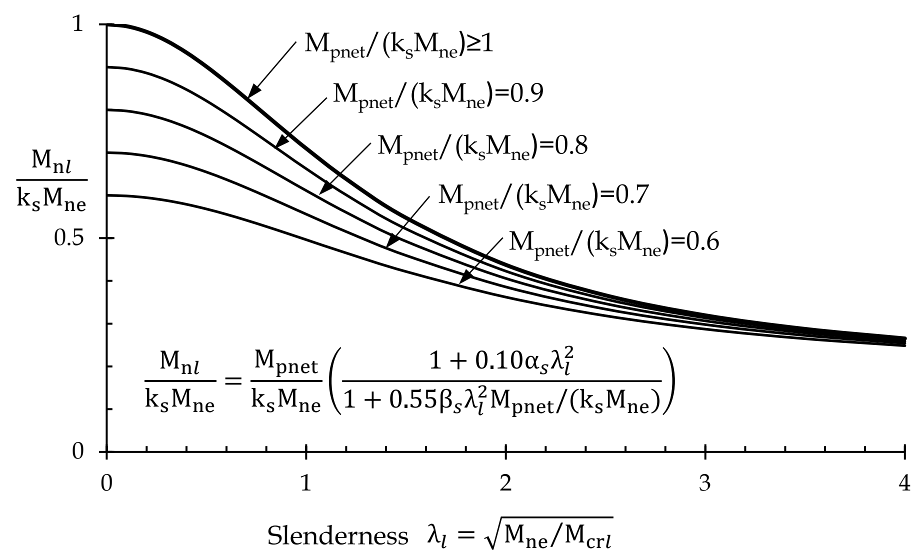


Figure C-F3.2-4 DSM for Local Buckling of Beams With Holes ($M_{ne} \leq M_y$, k_s=1.2, α_s=1, β_s=1)

More discussions are provided in *Commentary* Section E3.2 regarding hole influences on member strength, including the treatment of stiffened holes (Grey and Moen, 2011; Moen and Yu, 2010). Liu et al. (2022) further confirmed the suitability of *DSM* for C-section beams with edge stiffened holes as long as the elastic *buckling* solution includes the effect of the edge stiffened holes.

F3.3 Cylindrical Tubes

Specification Equations F3.3-1, F3.3-2 and F3.3-3 are based upon the work reported by Sherman (1985). All three equations for determining the *nominal flexural strength* [*resistance*] are shown in Figure C-F3.3-1 as originally defined using *stresses*. These equations have been used in the *Specification* since 1986 and are retained in this edition, but now restated using moments. The *safety factor*, Ω_b, and the *resistance factor,* ϕ_b, are the same as used in *Specification* Section F2.3 for inelastic reserve bending strength.

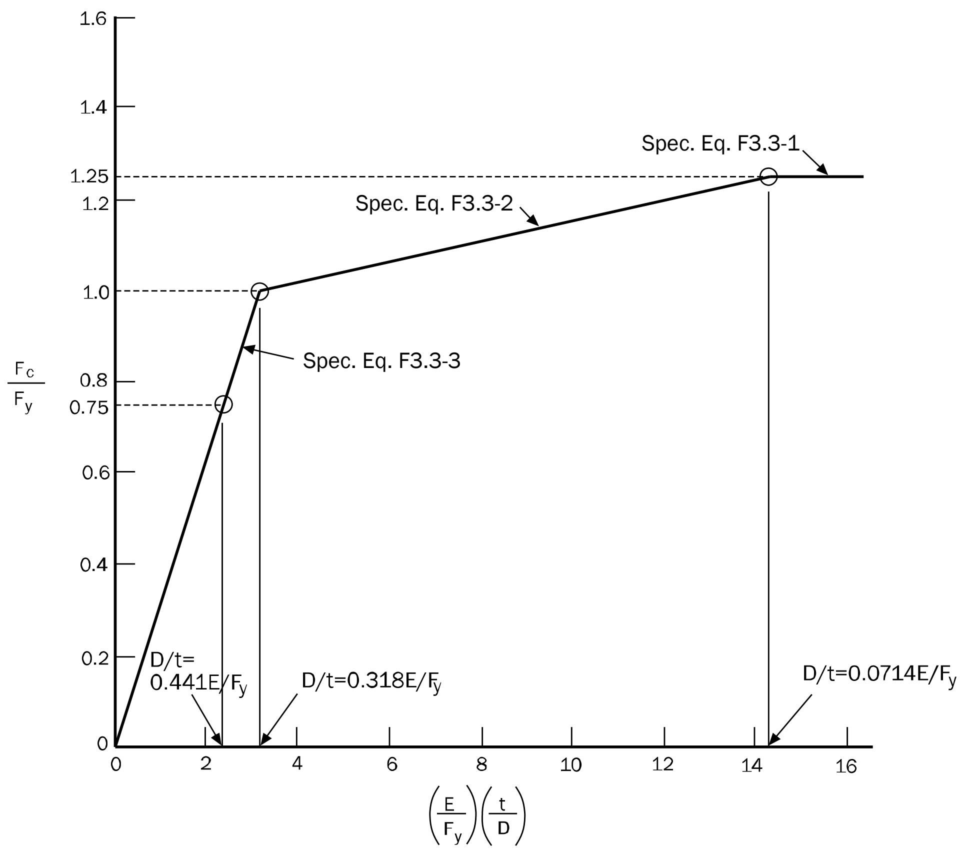


Figure C-F3.3-1 Nominal Flexural Strength of Cylindrical Tubes

F4 Distortional Buckling

Distortional buckling is an *instability* that may occur in members with edge-stiffened *flanges*, such as lipped C- and Z-sections. As shown in Figure C-F4-1, this *buckling* mode is characterized by *instability* of the entire *flange*, as the *flange* along with the edge stiffener rotates about the junction of the compression *flange* and the *web*. The length of the *buckling* wave in *distortional buckling* is considerably longer than *local buckling*, and noticeably shorter than *lateral-torsional buckling*. The *Specification* provisions of Section 1.3 partially account for *distortional buckling*, but research has shown that a separate limit state check is required (Ellifritt, Sputo, and Haynes, 1992; Hancock, Rogers, and Schuster, 1996; Kavanagh and Ellifritt, 1994; Schafer and Peköz, 1999;

Hancock, 1997; Yu and Schafer, 2003 and 2006). Thus, in 2007, provisions were added to address *distortional buckling* as a separate *limit state*.

Distortional buckling can also occur in flexural members with laterally unbraced edge-stiffened *flanges*, as in Figure C-2.3.1.2-4, as well as sections with intermediate stiffeners in compression. The strength provisions of *Specification* Section F4 apply to all these types of *distortional buckling*. Note that *buckling* of intermediate stiffeners is treated as *local buckling* when using the *Effective Width Method*, but is recognized as *distortional buckling* when using the *Direct Strength Method*.

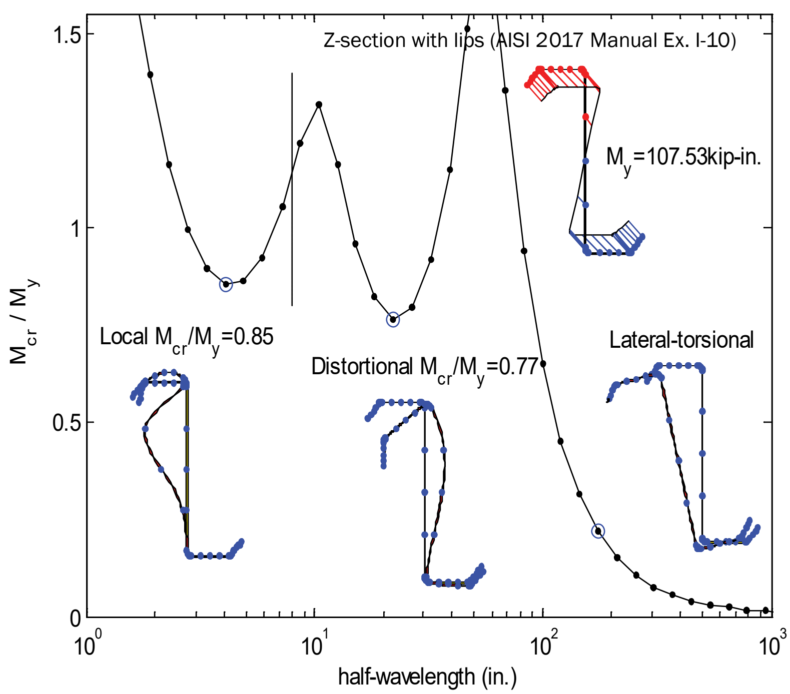


Figure C-F4-1 Rational Elastic Buckling Analysis of a Z-Section Under Restrained Bending Showing Local, Distortional, and Lateral-Torsional Buckling Modes

The expression used for *distortional buckling* strength of beams is shown in Figure C-F1-1 and is discussed in Section F1. The use of the *DSM* for *distortional buckling* and the development of the original empirical strength expression are given in Schafer and Peköz (1999). The current strength expression was developed by Glauz and Schafer (2022) to improve the strength predictions for members unsymmetric about the axis of bending and members that lose overall depth in post-*buckling*.

The provisions of this section are limited to *distortional buckling* slenderness, λ_d, less than 5 due to uncertainty of the strength predictions at high slenderness. The limits of applicability in *Specification* Section B4.1 already restrict slenderness to a practical range, but it is still possible to exceed the imposed slenderness limit. One rational approach to high slenderness applications is to limit the *distortional buckling* strength using Equation C-F4-1 which approximates the original form of *DSM* equation approaching zero at high slenderness and accommodates the factors k_s and β_s.

$$M_{nd} \leq \frac{k_s M_y}{1.2\beta_s \lambda_d} \tag{C-F4-1}$$

Members unsymmetric about the axis of bending exhibit greater post-*buckling stress* redistribution, particularly as plastification ensues. The *distortional buckling* strength curve is a single equation referenced from the maximum inelastic reserve strength of M_p, and utilizes two modifiers for the strength impact of *stress* redistribution: section factor α_s and symmetry factor β_s. The influence of these factors is illustrated in Figure C-F4-2, where α_s=1 and β_s=1 for common sections.

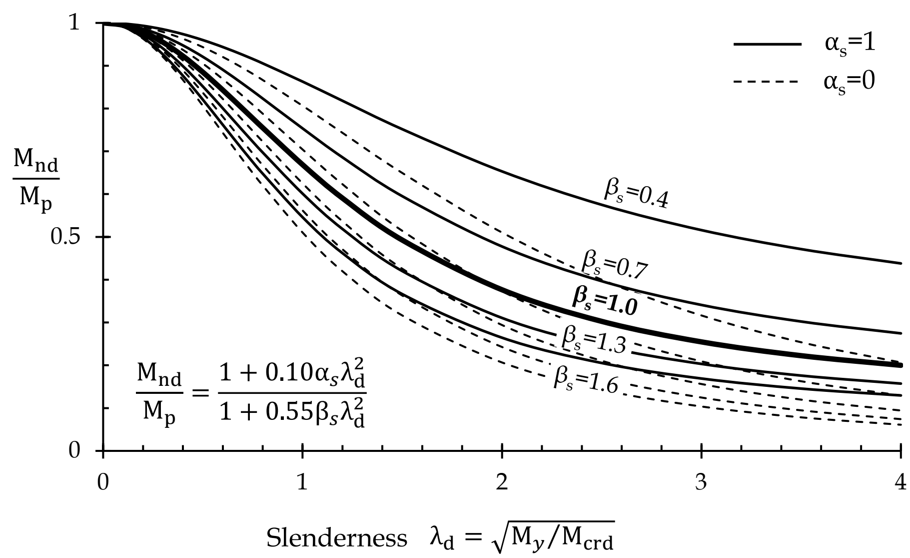


Figure C-F4-2 DSM for Distortional Buckling of Beams

The symmetry factor β_s is the same for *local* and *distortional buckling*. For sections symmetric about the axis of bending, β_s is 1. For sections with the first yield in tension, β_s is less than 1 and the post-*buckling stress* redistribution is favorable to the flexural strength. For sections with the first yield in compression, β_s is greater than 1 and the post-*buckling stress* redistribution is unfavorable to the flexural strength. Figure C-F3.2-2 shows examples of β_s for sections unsymmetric about the axis of bending.

For open flexural members with free edges in compression, the *distortional buckling* flexural strength drops more rapidly as slenderness increases. The section factor α_s is used to account for this strength reduction. For cross-sections with fold lines (corners) at the extreme compression fiber that do not translate under *distortional buckling*, α_s=1. For cross-sections where the only non-translating fold lines are at the extreme tension fiber, α_s=0. Cross-sections having non-translating fold lines between the extreme tension and compression fiber may have a flexural strength represented by α_s between 0 and 1 as shown in Figure C-F4-3. The rational approach suggested by Glauz and Schafer (2022) is to linearly interpolate α_s as d_s/d, where d_s is the depth of the cross-section excluding distorted elements and d is the overall depth.

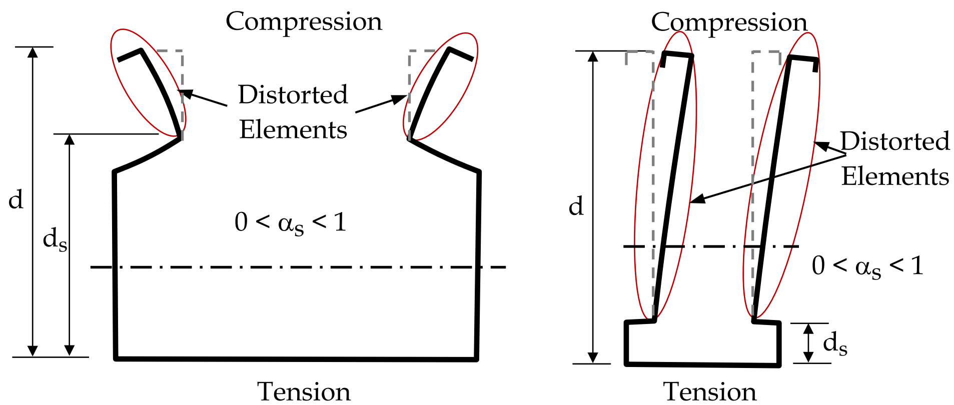


Figure C-F4-3 Section Factor for Distortional Buckling of Beams

Distortional buckling is unlikely to control the strength if: (a) edge stiffeners are sufficiently stiff and thus stabilize the *flange* (as is often the case for C-sections, but typically not for Z-sections due to the use of sloping lips), (b) unbraced lengths are long and *lateral-torsional buckling* strength limits the capacity, or (c) adequate rotational restraint is provided to the compression *flange* from attachments (panels, sheathing, etc.).

The primary difficulty in calculating the strength in *distortional buckling* is to efficiently estimate the elastic *distortional buckling* moment, M_{crd}. Recognizing the complexity of this calculation, Appendix 2 provides two alternatives: (a) numerical solutions, or (b) analytical formulas for C- and Z-section members and any open section with a single *web* and single edge-stiffened compression *flange*. See Appendix 2 commentary for further discussion. The Appendix 2 commentary also provides a simplified analytical formula that may be useful in preliminary design, and was specifically derived as a conservative simplification to *Specification* Section 2.3.3.2.

For beams with holes, the *DSM* strength prediction expression was modified to limit the maximum strength to the capacity of the net cross-section with a linear transition to the *distortional buckling* strength curve (Moen and Schafer, 2009a) which was further refined to a single smooth curve by Glauz (2023). Figure C-F4-4 compares the *distortional buckling* strength prediction for beams with holes having different ratios of M_{pnet}/M_p. As slenderness increases, the prediction transitions from M_{pnet} to the strength curve used for beams without holes (M_{pnet}/M_p=1). The transition is implemented to reflect the change in failure mode as slenderness increases, from *yielding* at the net section to *distortional buckling*.

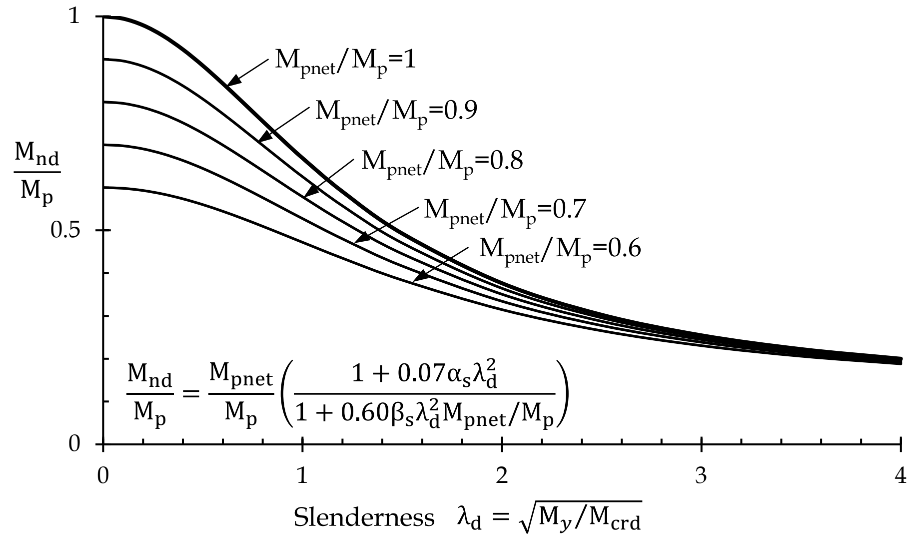


Figure C-F4-4 DSM for Distortional Buckling of Beams With Holes (k_s=1.2, α_s=1, β_s=1)

When determining M_{nd} for members with holes, M_{crd} is required to be determined including the influence of holes. *Specification* Appendix 2 Section 2.3.3.2 specifies M_{crd} = $S_{fc}F_{crd}$. For members with holes, only F_{crd} must include the influence of holes. The section modulus should not be modified. Thus, for members with holes, M_{crd} = S_{fc} (gross elastic section modulus ignoring holes) × F_{crd} (elastic *distortional buckling stress* including the influence of holes).

G. MEMBERS IN SHEAR, WEB CRIPPLING, AND TORSION

G1 General Requirements

Chapter G defines the shear strength of flexural members, *web* crippling strength, and torsion strength. The design of transverse *web* stiffeners and the design of bearing stiffeners are also treated.

G2 Shear Strength [Resistance] of Webs Without Holes

Prior to the 2001 edition, the AISI *ASD Specification* (AISI, 1986) used three different *safety factors* when evaluating the *allowable shear strength* of an unreinforced *web* because it was intended to use the same *nominal strength* [*resistance*] equations for the AISI and AISC Specifications. To simplify the design of shear using only one *safety factor* for *ASD* and one *resistance factor* for *LRFD*, Craig (1999) carried out a calibration using the data by LaBoube and Yu (LaBoube, 1978a). Based on this work, in the 2001 edition of the *Specification*, the constant used to determine the shear strength due to inelastic *buckling* was reduced from 0.64 to 0.60. In addition, the *ASD safety factor* for yielding, elastic and inelastic *buckling* was taken as 1.60, with a corresponding *resistance factor* of 0.95 for *LRFD* and 0.80 for *LSD*.

In 2022, the *resistance factor* was changed to 0.90 for *LRFD* and 0.75 for *LSD* based on historic test data (LaBoube and Yu, 1978) and more recent test data (Keerthan and Mahendran, 2015; Pham, Zelenkin and Hancock, 2017; Pham, Pham and Hancock, 2020). The *safety factor* was changed to 1.67 for *ASD*.

G2.1 Flexural Members Without Transverse Web Stiffeners

In 2022, the shear strength equations in *Specification* Section G2.1 for flexural members without transverse *web* stiffeners were changed to the *Direct Strength Method* (*DSM*) format accounting for the shear post-*buckling* at higher h/t ratios. The main purpose of these revised design equations was to raise the design strength in the elastic *buckling* range because there is always some post-*buckling* strength in shear even for unstiffened elements as demonstrated by testing (Pham, Zelenkin and Hancock, 2017; Pham, Pham and Hancock, 2020). This aligns to some degree with the AISC 360 Specification (AISC, 2016a) where *Specification* Section G2 only has the *yielding* and inelastic *buckling* zones with no elastic *buckling* zone at higher slenderness.

For *webs* having small h/t ratios, shear *yielding* controls the design. For unstiffened *webs* having large h/t ratios, elastic *shear buckling* (Yu and LaBoube, 2010) and some degree of *shear* post-*buckling* (Pham, Zelenkin and Hancock 2017; Pham, Pham and Hancock, 2020) control the design. The original calibration of the revised strength equations is provided in Pham, Pham and Hancock (2020). Subsequent calibration to the current *DSM* form given by *Specification* Equation G2.1-1 is provided by Glauz (2023), and was adopted in 2024. The strength curves for *webs* with and without transverse stiffeners are presented in Figure C-G2.1-1.

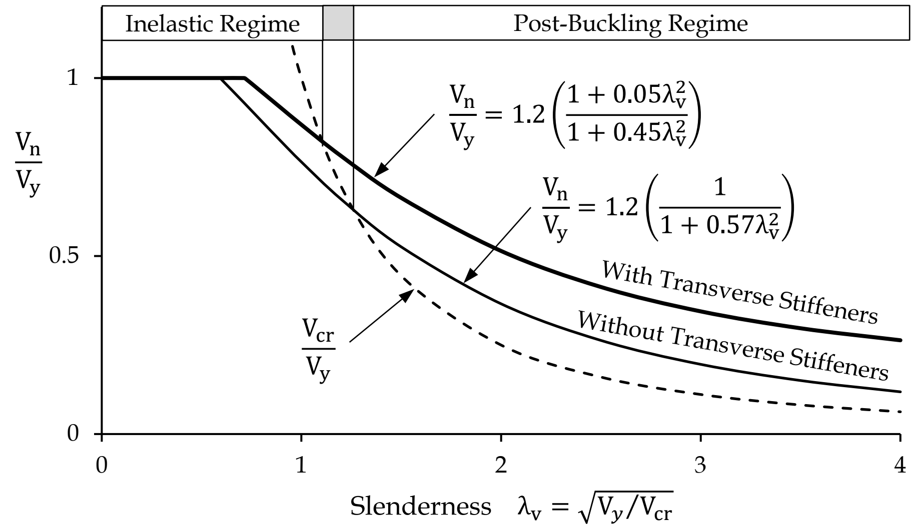


Figure C-G2.1-1 DSM Strength Curves for Shear

G2.2 Flexural Members With Transverse Web Stiffeners

In 2016, the *Direct Strength Method* (*DSM*) format equations for determining the *nominal shear strength* [*resistance*] were adopted for flexural members with transverse *web* stiffeners in Section G2.2. Validation for the *shear* post-*buckling* equations in *DSM* format has been confirmed (Pham and Hancock, 2012a) by tests on high-strength steel C-sections in shear, and combined bending and shear. Sections with transverse *web* stiffeners have considerable tension field action (Pham and Hancock, 2012b). A larger data set has been used to confirm these equations (Pham and Hancock, 2012a; Keerthan and Mahendran, 2015; Pham, Zelenkin and Hancock, 2017; Pham, Pham and Hancock, 2018; Pham, Pham, Rogers and Hancock, 2019). Prior to 2016, the transverse stiffener effect was only considered in the shear *buckling* coefficient k_v. Tension field action was, however, not considered.

For *webs* having small h/t ratios, shear *yielding* controls the design. For *webs* having larger h/t ratios, transverse *web* stiffeners enable significant tension field action (Pham and Hancock, 2012b) resulting in a higher design strength equation. Calibration to the current *DSM* form given by *Specification* Equation G2.2-1 is provided by Glauz (2023), and was adopted in 2024. The strength curves for *webs* with and without transverse stiffeners are presented in Figure C-G2.1-1.

G2.3 Web Elastic Critical Shear Buckling Force, V_{cr}

Specification Section G2.3 provides a simple analytical solution for shear *buckling* force, V_{cr}, of an unreinforced *web*. However, for prequalified *webs* according to *Specification* Table B4.1-1, a numerical analysis approach should be considered in accordance with Appendix 2, which provides for the contribution of the transverse stiffeners in *buckling* analysis.

G3 Shear Strength of C-Section Webs With Holes

For C-section *webs* with holes, Schuster, et al. (1995) and Shan, et al. (1994) investigated the degradation in *web* shear strength due to the presence of a *web* perforation. The test program considered a constant shear distribution across the perforation, and included d_0/h ratios ranging from 0.20 to 0.78, and h/t ratios of 91 to 168. Schuster's equation for reduction factor, q_s, was developed with due consideration for the potential range of both punched and field-cut holes. Three-hole geometries—rectangular with corner fillets, circular, and diamond—were considered in the test program. Eiler (1997) extended the work of Schuster and Shan for the case of constant shear along the longitudinal axis of the perforation. He also studied linearly varying shear, but this case is not included in the *Specification*. The development of Eiler's reduction factor, q_s, utilized the test data of both Schuster, et al. (1995) and Shan, et al. (1994).

The method of applying reduction factor q_s to consider the hole effect was used in the *Specification* up to the 2016 edition. The data from Schuster, et al. (1995), Shan, et al. (1994) and Eiler (1997) have been included to calibrate the *DSM* methodology described below.

A *DSM* design model was developed for square and circular *web* holes in C-sections in shear by Pham S.H., et al. (2017a) and Pham, S.H., et al. (2020). The model is based on a Vierendeel truss collapse mechanism across the hole and uses the yield shear force V_{yh} and *shear buckling* load V_{crh}. The method has been validated for stiffened *webs* according to Section G2.2 against test results by Keerthan and Mahendran (2013) for circular holes; Pham, C.H., et al. (2014, 2016) and Pham, C.H. and Hancock (2020) for square holes. The parameters varied with a wider range than the previous empirical approach (i.e., using the q_s reduction factor) and the method can be applied to square and circular holes. In 2020, the method has been extended to elongated rectangular and slotted holes by Pham, D.K., et al. (2020). The method is much more reliable than the previous empirical approach since it accounts for the influences of both the depth of the hole, d_h and the length of the hole, L_h. There is no reduction in V_y for holes with d_h/h less than or equal to 0.1. The methodology is applicable to square, rectangular, circular, and slotted holes as shown in Figure C-G3-1. *Buckling* coefficients for square holes were derived by Pham, S.H., et al. (2017b) using an artificial neural network for a wide range of parameters (h/a, d_h/h, L_h/a, ah/A_h) where a is the shear span between transverse stiffeners, the hole dimensions d_h and L_h are shown in Fig. C-G3-1, and other variables are defined in the *Specification* Section G3. The revised *buckling* coefficients were derived in Pham, S.H., et al. (2020), and for circular and slotted holes in Pham, V.B., et al. (2020). The yield shear force, V_{yh} varies from an unreduced value of V_y for small *web* holes to a shear yielding value including the hole effect. V_{yh} is computed based upon a theoretical Vierendeel plastic collapse mechanism for sufficiently large square and circular *web* holes as derived in Pham, S.H., et al. (2017a) and Pham, S.H., et al. (2020). The methodology was extended by Pham, D.K., et al. (2020) to account for the influence of the aspect ratios of the *web* holes (L_h/d_h) and the expression is simplified using a cubic polynomial as given in *Specification* Equation G3-2 by Pham, V.B., et al. (2020).

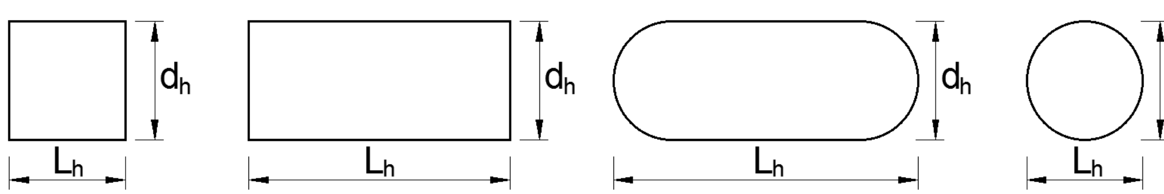


Figure C-G3-1 Illustration of Hole Dimensions

G4 Transverse Web Stiffeners

G4.1 Compact Transverse Web Stiffeners

The requirements for transverse *web* stiffeners included in *Specification* Section G4.1 up until 2024 were primarily adopted from the AISC Specification (1978). In the 2016 and earlier editions of the *Specification*, the equations for determining the minimum required moment of inertia and the minimum required *gross area* of attached transverse *web* stiffeners were based on the studies summarized by Nguyen and Yu (1978a).

In the 2016 AISC Specification (2016a), these requirements were changed to be based on the demands on the stiffener flexural rigidity when the post-*buckling* resistance of the *web* is relied upon. The revised AISC requirements are based on the research of Kim and White (2013). These requirements were confirmed for cold-formed steel channel sections in Pham, S.H., Pham, C.H. and Hancock, G.J. (2020). The requirements in *Specification* Section G4.1 were therefore revised in the 2024 edition to align with the AISC 2016 requirements. Only the case where the full tension field action (TFA) is specified is included in the revised Section G4.1 as validated in Pham, S.H., Pham, C.H. and Hancock, G.J. (2020). The requirement that "The *connections* between the stiffener and the *web* shall ensure full connectivity over the entire *web* depth" are based on the 2016 AISC Specification where welding is normally used. For cold-formed steel channel sections as tested in Pham, S.H., Pham, C.H. and Hancock, G.J. (2020a), 6 evenly spaced screws over the full depth with h/t ratio up to 133 were used to ensure that the shear buckles in the post-*buckling* range at the center of the *web* was fully restrained and were found to be adequate to ensure full connectivity.

G4.2 Other Transverse Web Stiffeners

Tests on rolled-in transverse *web* stiffeners covered in *Specification* Section G4.2 were not conducted in the experimental program reported by Nguyen and Yu (1978 a), and Pham, S.H., Pham, C.H. and Hancock, G.J. (2020a). Lacking reliable information, the *available strength* [*factored resistance*] of stiffeners should be determined by special tests, or *rational engineering analysis*.

G5 Web Crippling Strength of Webs Without Holes

Since cold-formed steel flexural members generally have large *web* slenderness ratios, the *webs* of such members may cripple due to the high local intensity of the *load* or reaction. Figure C-G5-1 shows typical *web crippling* failure modes of unreinforced single hat sections (Figure C-G5-1(a)) and of I-sections (Figure C-G5-1(b)) unfastened to the support.

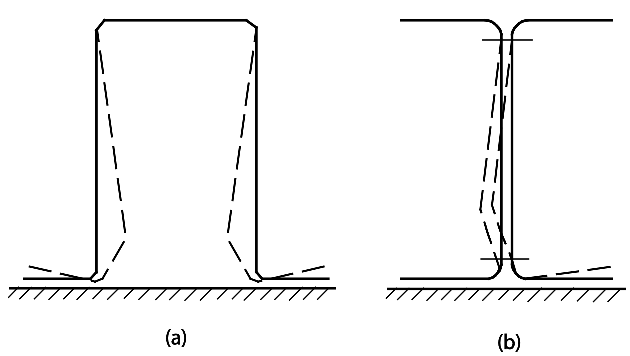


Figure C-G5-1 Web Crippling of Cold-Formed Steel Sections

In the past, the *buckling* problem of plates and the *web crippling* behavior of cold-formed steel members under locally distributed edge loading have been studied by numerous investigators (Yu and LaBoube, 2010). It has been found that the theoretical analysis of *web crippling* for cold-formed steel flexural members is rather complicated because it involves the following factors: (1) nonuniform *stress* distribution under the applied *load* and adjacent portions of the *web*, (2) elastic and inelastic stability of the *web* element, (3) *local yielding* in the immediate region of *load* application, (4) bending produced by eccentric *load* (or reaction) when it is applied on the bearing *flange* at a distance beyond the curved transition of the *web*, (5) initial out-of-plane imperfection of plate elements, (6) various edge restraints provided by beam *flanges* and interaction between *flange* and *web* elements, and (7) inclined *webs* for decks and panels.

For these reasons, the present AISI design provision for *web crippling* is based on the extensive experimental investigations conducted at Cornell University by Winter and Pian (1946) and Zetlin (1955a); at the University of Missouri-Rolla by Hetrakul and Yu (1978 and 1979), Yu (1981), Santaputra (1986), Santaputra, Parks and Yu (1989), Bhakta, LaBoube and Yu (1992), Langan, Yu and LaBoube (1994), Cain, LaBoube and Yu (1995) and Wu, Yu and LaBoube (1997); at the University of Waterloo by Wing (1981), Wing and Schuster (1982), Prabakaran (1993), Gerges (1997), Gerges and Schuster (1998), Prabakaran and Schuster (1998), Beshara (1999), and Beshara and Schuster (2000 and 2000a); and at the University of Sydney by Young and Hancock (1998). In these experimental investigations, the *web crippling* tests were carried out under the following four loading conditions for beams having single unreinforced *webs* and I-beams, single hat sections and multi-*web* deck sections:

1. End one-*flange* (EOF) loading
2. Interior one-*flange* (IOF) loading
3. End two-*flange* (ETF) loading
4. Interior two-*flange* (ITF) loading

All loading conditions are illustrated in Figure C-G5-2. In Figures (a) and (b), the distances between bearing plates were kept to no less than 1.5 times the *web* depth in order to avoid the two-*flange* loading action. Application of the various *load* cases is shown in Figure C-G5-3 and the assumed reaction or *load* distributions are illustrated in Figure C-G5-4.

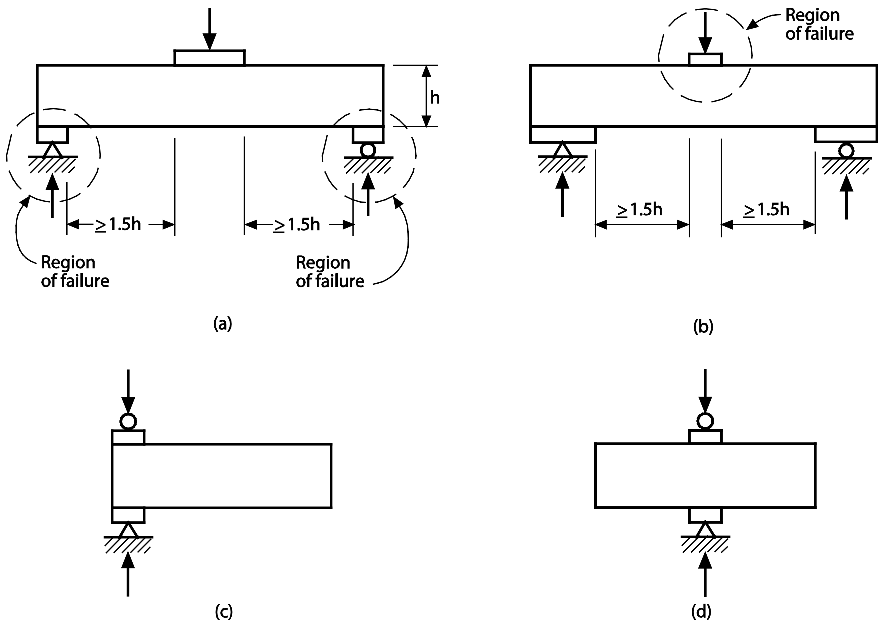


Figure C-G5-2 Loading Conditions for Web Crippling Tests: (a) EOF Loading, (b) IOF Loading, (c) ETF Loading, (d) ITF Loading

In the 1996 edition of the AISI *Specification*, and in previous editions, different *web crippling* equations were used for the various loading conditions stated above. These equations were based on experimental evidence (Winter, 1970; Hetrakul and Yu, 1978) and the assumed distributions of *loads* or reactions acting on the *web* as shown in Figure C-G5-4. The equations were also based on the type of section geometry, i.e., shapes having single *webs* and I-sections (made of two channels connected back-to-back, by welding two angles to a channel, or by connecting three channels). C-and Z-sections, single hat sections and multi-*web* deck sections were considered in the single *web* member category. I-sections made of two channels connected back-to-back by a line of connectors near each *flange* or similar sections that provide a high degree of restraint against rotation of the *web* were treated separately. In addition, different equations were used for sections with stiffened or partially stiffened *flanges* and sections with unstiffened *flanges*.

Prabakaran (1993) and Prabakaran and Schuster (1998) developed one consistent unified *web crippling* equation with variable coefficients (*Specification* Equation G5-1). These coefficients accommodate one- or two-*flange* loading for both end and interior loading conditions of various section geometries. Beshara (1999) extended the work of Prabakaran and Schuster (1998) by developing new *web crippling* coefficients using the available data as summarized by Beshara and Schuster (2000). The *web crippling* coefficients are summarized in Tables G5-1 to G5-5 of the *Specification* and the parametric limitations given are based on the experimental data that was used in the development of the *web crippling* coefficients. From *Specification* Equation G5-1, it can be seen that the *nominal web crippling strength* [*resistance*] of cold-formed steel members depends on an overall *web crippling* coefficient, C; the *web thickness*, t; the *yield stress*, F_y; the *web* inclination angle, θ; the inside bend radius coefficient, C_R; the inside bend radius ratio, R/t; the bearing length coefficient, C_N; the bearing length ratio, N/t; the *web* slenderness coefficient, C_h; and the *web* slenderness ratio, h/t.

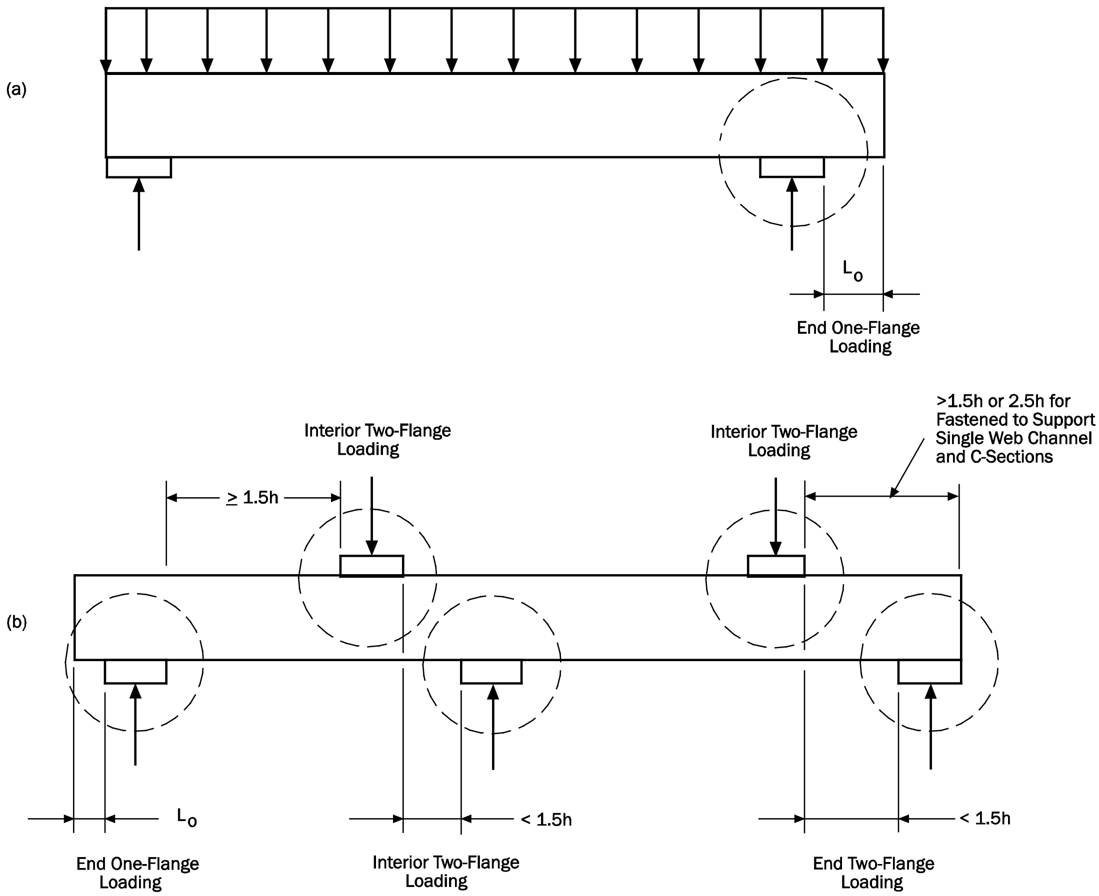


Figure C-G5-3 Application of Loading Case

This new equation is presented in a normalized format and is nondimensional, allowing for any consistent system of measurement to be used. Consideration was given to whether or not the test specimens were fastened to the bearing plate/support during testing. It was discovered that some of the test specimens in the literature were not fastened to the bearing plate/support during testing, which can make a considerable difference in the *web crippling* capacity of certain sections and loading conditions. Therefore, it was decided to separate the data on the basis of members being fastened to the bearing plate/support and those not being fastened to the bearing plate/support. The fastened-to-the-bearing plate/support data in the literature were primarily based on specimens being bolted to the bearing plate/support; hence, a few control tests were carried out by Schuster, the results of which are contained in Beshara (1999), using self-drilling screws to establish the *web crippling* integrity in comparison to the bolted data. Based on these tests, the specimens with self-drilling screws performed equally well in comparison to the specimens with bolts. Fastened-to-the-bearing plate/support in practice can be achieved by either using bolts, self-drilling/self-tapping screws or by welding. What is important is that the *flange* elements are restrained from rotating at the location of *load* application. In fact, in most cases, the *flanges* are frequently completely restrained against rotation by some type of sheathing material that is attached to the *flanges*.

The data was further separated in the *Specification* based on section type, as follows:

1) Built-up sections (Table G5-1),
2) Single *web* channel and C-sections (Table G5-2),
3) Single *web* Z-sections (Table G5-3),
4) Single hat sections (Table G5-4), and
5) Multi-web deck sections (Table G5-5).

Calibrations were carried out by Beshara and Schuster (2000) in accordance with Supornsilaphachai, Galambos and Yu (1979) to establish the *safety factors*, Ω, and the *resistance factors*, ϕ, for each *web crippling* case. Based on these calibrations, different *safety factors* and corresponding *resistance factors* are presented in the *web crippling* coefficient tables for the particular *load* case and section type. In 2005, the *safety* and the *resistance factors* for built-up and single hat sections with interior one-*flange* loading case were revised based on a more consistent calibration. For the fastened built-up sections, the factors were revised from 1.65 to 1.75 (for *ASD*), 0.90 to 0.85 (for *LRFD*) and 0.80 to 0.75 (for *LSD*). For the fastened single hat section, the factors were revised from 1.90 to 1.80 (for *ASD*) and 0.80 to 0.85 (for *LRFD*). For the unfastened single hat sections, the factors were revised from 1.70 to 1.80 (for *ASD*), 0.90 to 0.80 (for *LRFD*) and 0.75 to 0.70 (for *LSD*). Also in 2005, the coefficients for built-up sections were revised to remove inconsistencies between unfastened and fastened conditions and to better reflect the calibration for the *safety factor* and the *resistance factors*. Also, a minimum bearing length of 3/4 in. (19 mm) was introduced based on the data used in the development of the *web crippling* coefficients. For fastened-to-support single *web* C- and Z-section members under interior two-*flange* loading or reaction, the distance from the edge of bearing to the end of the member (Figure C-G5-2(d)) must be extended at least 2.5h. This requirement is necessary because a total of 5h specimen length was used for the test setup shown in Figure C-G5-2(d) (Beshara, 1999). The 2.5h length is conservatively taken from the edge of bearing rather than the centerline of bearing.

The assumed distributions of *loads* or reactions acting on the *web* of a member, as shown in Figure C-G5-4, are independent of the flexural response of the member. Due to the flexural action, the point of bearing will vary relative to the plane of bearing, resulting in a nonuniform bearing *load* distribution on the *web*. The value of P_n will vary because of a transition from the interior one-*flange* loading (Figure C-G5-4(b)) to the end one-*flange* loading (Figure C-G5-4(a)) condition. These discrete conditions represent the experimental basis on which the design provisions were founded (Winter, 1970; Hetrakul and Yu, 1978). Based on additional updated calibrations, the *resistance factor* for Canada *LSD* for the unfastened interior one-*flange* loading (IOF) case in *Specification* Table G5-4 was changed from 0.75 to 0.70 in 2004.

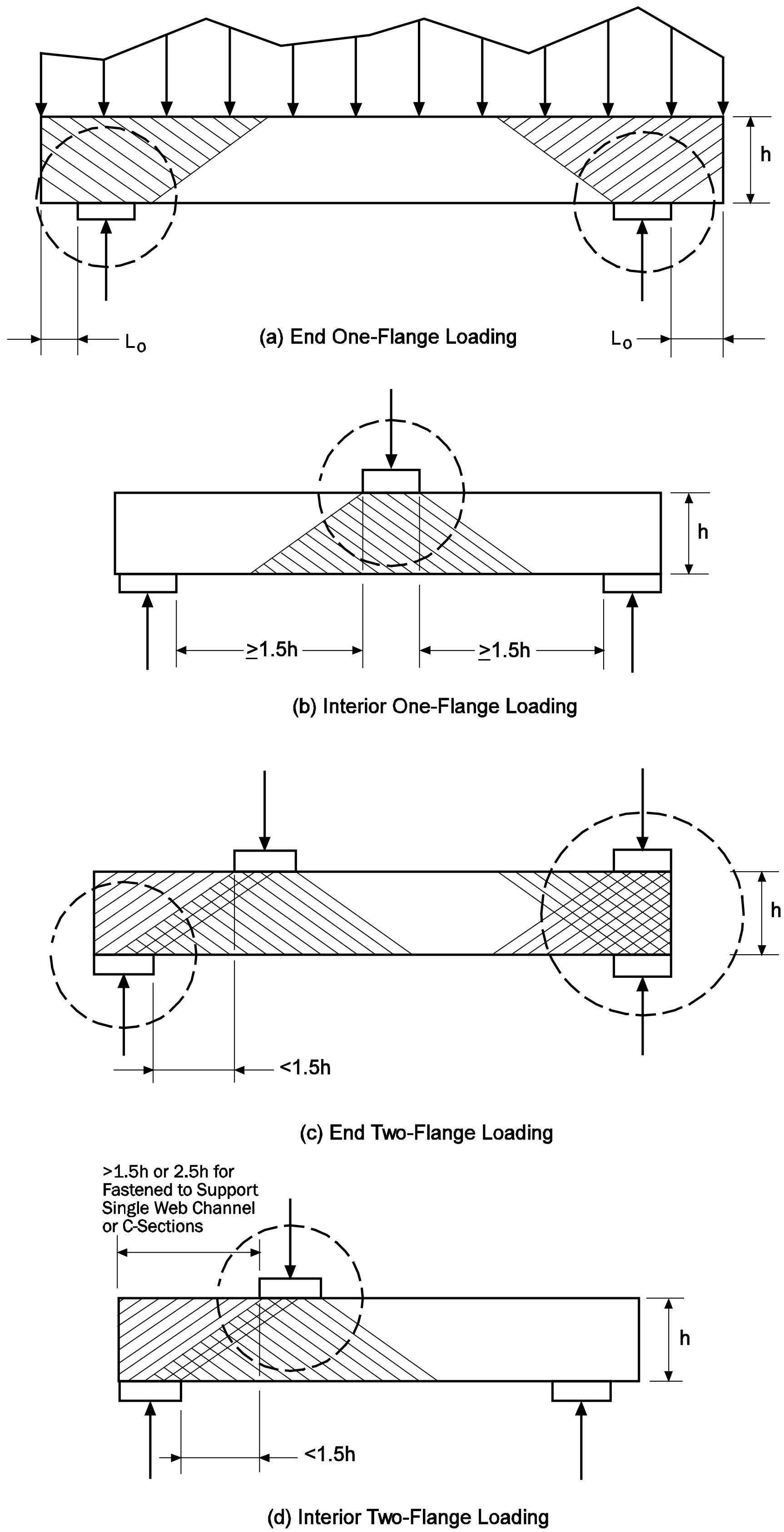


Figure C-G5-4 Assumed Distribution of Reaction or Load

In the case of unfastened built-up members such as I-sections (not fastened to the bearing plate/support), the available data was for specimens that were fastened together with a row of fasteners near each *flange* line of the member (Winter and Pian, 1946) and Hetrakul and Yu (1978) as shown in Figure C-G5-5(a). For the fastened built-up member data of I-sections (fastened to the bearing plate/support), the specimens were fastened together with two rows of fasteners located symmetrically near the centerline length of the member, as shown in Figure C-G5-5(b) (Bhakta, LaBoube and Yu, 1992).

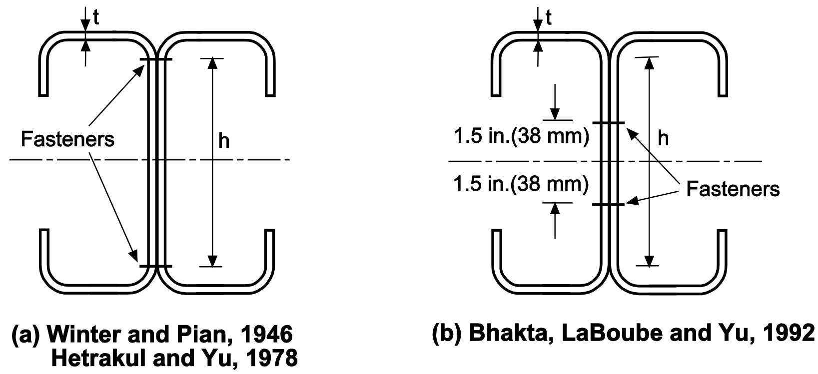


Figure C-G5-5 Typical Bolt Pattern for I-Section Test Specimens

In *Specification* Table G5-1, the heading was changed in 2012 to indicate that the resulting *nominal web crippling strength* [*resistance*] is per *web*.

The research indicates that a Z-section having its end support *flange* bolted to the section's supporting member through two 1/2-in. (12.7-mm) diameter bolts will experience an increase in end one-*flange web crippling* capacity (Bhakta, LaBoube and Yu, 1992; Cain, LaBoube and Yu, 1995). The increase in *load*-carrying capacity was shown to range from 27 to 55 percent for the sections under the limitations prescribed in the *Specification*. A lower-bound value of 30 percent increase was permitted in *Specification* Section G5 of the 1996 *Specification*. This is now incorporated under "Fastened to Support" condition.

In 2005, the R/t limit in *Specification* Table G5-3 regarding interior one-*flange* loading for fastened Z-sections was changed from 5 to 5.5 to achieve consistency with *Specification* Equation H3-3, which stipulates a limit of R/t = 5.5.

For two nested Z-sections, the 1996 *Specification* permitted the use of a slightly different *safety factor* and *resistance factor* for the interior one-*flange* loading condition. This is no longer required since the new *web crippling* approach now takes this into account in *Specification* Table G5-3 of the *Specification* under the category of "Fastened to Support" for the interior one-*flange* loading case.

The coefficients in *Specification* Table G5-4 for one-*flange* loading or reaction with fastened to support condition are based on those with unfastened to support condition. For consistency, the R/t ratios for unfastened to support condition were revised in 2009 to be the same as the values of fastened to support condition. The table heading was changed to indicate that the resulting *nominal web crippling strength* [*resistance*] is per *web*.

The previous *web crippling* coefficients in Table G5-5 for end one-*flange* loading (EOF) of multi-*web* deck sections in the design provisions (AISI 2001) were based on limited data. This data was based on specimens that were not fastened to the support during testing; hence, the previous coefficients for this case were also being used conservatively for the case of fastened to the support. Based on extensive testing, *web crippling* coefficients were developed by James A.

Wallace (2003) for both the unfastened and fastened cases of EOF loading. Calibrations were also carried out to establish the respective *safety factors* and *resistance factors*. The R/t ratio for interior one-*flange* loading with fastened to support condition was revised in 2012 to be consistent with the corresponding interior one-*flange* loading value of the unfastened condition. The heading of Table G5-5 was changed to indicate that the resulting *nominal web crippling strength* [*resistance*] is per *web*. A note was also added to the table to indicate that multi-*web* deck sections are considered unfastened for any support fastener spacing greater than 18 in. (460 mm) (Wallace, 2004).

In 2004, the definitions of "one-*flange* loading" and "two-*flange* loading" were revised according to the test setup, specimen lengths, development of *web crippling* coefficients, and calibration of *safety factors* and *resistance factors*. In Figures C-G5-3 and C-G5-4 of the *Commentary*, the distances from the edge of bearing to the end of the member were revised to be consistent with the *Specification*.

Specification Equation G5-2 for single *web* C- and Z-sections with an overhang or overhangs is based on a study of the behavior and resultant failure *loads* from an end one-*flange* loading investigation performed at the University of Missouri-Rolla (Holesapple and LaBoube, 2002). This equation is applicable within the limits of the investigation. The UMR test results indicated that in some situations with overhangs, the interior one-*flange* loading capacity may not be achieved, and the interior one-*flange* loading condition was therefore removed from Figures C-G5-3 and C-G5-4.

Tests were conducted on fastened to support, stiffened *flange*, single *web* 3-½ in. (88.9 mm) C-sections subjected to interior two-*flange* loading or reactions (ITF) that indicate the *web crippling* equation is unconservative by about 25 percent. Therefore, in 2012, the application of the *web crippling* equation was limited to a *web* depth greater than or equal to 4-½ in. (110 mm) or more to be consistent with the tests conducted by Schuster and Bashera in 1999. This revision was based on the *web crippling* test observations (Yu, 2009 and 2009a).

G6 Web Crippling Strength of C-Section Webs With Holes

Studies by Langan, et al. (1994), Uphoff (1996) and Deshmukh (1996) quantified the reduction in *web crippling* capacity when a hole is present in a *web* element. These studies investigated both the end one-*flange* and interior one-*flange* loading conditions for h/t and d_h/h ratios as large as 200 and 0.81, respectively. The studies revealed that the reduction in *web crippling* strength is influenced primarily by the size of the hole as reflected in the d_h/h ratio and the location of the hole, x/h ratio.

The provisions for circular and non-circular holes also apply to any hole pattern that fits within an equivalent virtual hole. Figure C-1.1.3-1 illustrates the L_h and d_h that may be used for a multiple hole pattern that fits within a non-circular virtual hole. Figure C-1.1.3-2 illustrates the d_h that may be used for a rectangular hole that fits within a circular virtual hole. For each case, the design provisions apply to the geometry of the virtual hole geometry, not the actual hole or holes.

G7 Bearing Stiffeners

G7.1 Compact Bearing Stiffeners

Design requirements for attached bearing stiffeners (previously called transverse stiffeners) were added in the 1980 *Specification* and the same design equations are retained in Section G7 of the current *Specification*. The term "transverse stiffener" was changed to "bearing stiffeners" in 2004. These provisions apply to bearing stiffeners whose elements are proportioned to prevent *local buckling*. The *nominal strength* [*resistance*] equation given in Item (a) of *Specification* Section G7.1 serves to prevent end crushing of the bearing stiffeners, while the *nominal strength* [*resistance*] equation given in Item (b) is to prevent column-type *buckling* of the *web*-stiffeners. The equations for computing the *effective areas* (A_b and A_c) and the *effective widths* (b_1 and b_2) were adopted from Nguyen and Yu (1978a) with minor modifications.

The available experimental data on cold-formed steel bearing stiffeners were evaluated by Hsiao, Yu and Galambos (1988a). A total of 61 tests were examined. The *resistance factor* of 0.85 used for the *LRFD* method was selected on the basis of the statistical data. The corresponding reliability indices vary from 3.32 to 3.41.

In 1999, the upper limit of w/t_s ratio for the unstiffened elements of cold-formed steel bearing stiffeners was revised from $0.37\sqrt{E/F_{ys}}$ to $0.42\sqrt{E/F_{ys}}$ for the reason that the former was calculated based on the *Allowable Strength Design* approach, while the latter is based on the *effective area* approach. The revision provided the same basis for the stiffened and unstiffened elements of cold-formed steel bearing stiffeners.

G7.2 Stud and Track Type Bearing Stiffeners in C-Section Flexural Members

The provisions of this section are based on the research by Fox and Schuster (2001), which investigated the behavior of stud and track type bearing stiffeners in cold-formed steel C-section flexural members. These stiffeners fall outside of the scope of *Specification* Section G7.1 and may contain elements that are not fully effective at *yield stress.* The research program investigated bearing stiffeners subjected to two-*flange* loading at both interior and end locations, and with the stiffener positioned between the member *flanges* and on the back of the member. A total of 263 tests were carried out on different stiffened C-section assemblies. The design expression in *Specification* Section G7.2 is a simplified method applicable within the limits of the test program. A more detailed beam-column design method is described in Fox (2002).

G7.3 Other Stiffeners

Tests on rolled-in stiffeners covered in *Specification* Section G7.3 were not conducted in the experimental program reported by Nguyen and Yu (1978). Lacking reliable information, the *available strength* [*factored resistance*] of stiffeners should be determined by special tests.

G8 Torsion Strength

Members subject to torsion are resisted by St. Venant (pure) torsion, warping torsion, or both. Closed sections are resisted predominantly by pure torsion, whereas open sections are resisted predominantly by warping torsion. Open sections with thicker steel exhibit more pure torsion and less warping torsion than those with thinner material. Warping end restraints also influence the levels of pure and warping torsion. Open sections where all elements pass through the shear center (such as angles, tees, and cruciform shapes) have negligible warping resistance, which therefore rely on pure torsion alone and may exhibit excessive twisting. To quantify these general cases:

if $L_t > 20\sqrt{EC_w / (GJ)}$, warping torsion can be neglected;

if $L_t < 0.5\sqrt{EC_w / (GJ)}$, pure torsion can be neglected;

if L_t is around $3\sqrt{EC_w / (GJ)}$, torsion is resisted fairly evenly by warping and pure torsion;

where L_t is the unbraced length for twisting, EC_w is the warping rigidity and GJ is the pure torsion rigidity.

The *Specification* does not address serviceability for twisting, but this may be an important consideration in design.

Warping torsion produces both shear *stresses* and longitudinal *stresses*. Warping longitudinal *stresses* have a greater impact on member capacity than warping shear *stresses*, thus Section G8.1 addresses this longitudinal *stress* limit state. Pure torsion produces only shear *stresses*. The *limit state* for pure torsion shear is not yet addressed in the *Specification*.

G8.1 Torsion Bimoment Strength

The term bimoment refers to the cross-section *stress* resultant due to longitudinal torsional warping. The *nominal bimoment strength* [*resistance*] given by *Specification* Equation G8.1-1 is the value at which the highest longitudinal warping *stress* reaches yield. Research by Bian, et al. (2016) demonstrated that many cross-sections can have significant inelastic reserve, where the torsional strength could reach twice the magnitude at first yield. The alternative strength provisions given by *Specification* Equations G8.1-3 and G8.1-5 allow partial plastification, limited by the *buckling strength* of the cross-section. This approach was developed by Xia et al. (2022) and requires the additional effort of determining the plastic bimoment and critical elastic *buckling* bimoments.

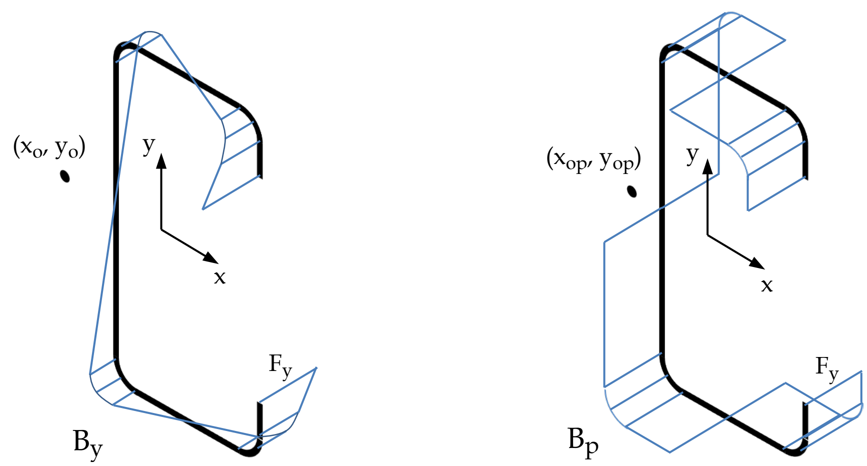


Figure C-G8.1-1 Torsion Bimoment: First Yield and Fully Plastic

The bimoment *stress* distributions at first yield and fully plastic are illustrated in Figure C-G8.1-1. The first yield bimoment, B_y, is calculated by Equation C-G8.1-1, where $\omega_{n,max}$ is the maximum magnitude of ω_n, which is the value of the sectorial coordinate ω_o based on the elastic shear center (x_o, y_o), normalized by subtracting the mean value of ω_o ($\overline{\omega}_o$). The plastic bimoment, B_p, is calculated by Equation C-G8.1-3, where ω_{np} is the value of the sectorial coordinate ω_{op} based on the plastic shear center (x_{op}, y_{op}), normalized by subtracting the median value of ω_{op} ($\tilde{\omega}_{op}$) such that equal areas of positive and negative ω_{np} exist (axial equilibrium). The plastic shear center is located such that the resulting bimoment *yield stresses* produce no net moment about the x- or y-axes (flexural equilibrium). The plastic shear center is typically closer to the centroid than the elastic shear center, and generally requires iteration to converge on its location.

$$B_y = \frac{F_y}{\omega_{n,max}} \int_0^{\ell} \omega_n^2 t ds = \frac{F_y C_w}{\omega_{n,max}} \tag{C-G8.1-1}$$

$$\omega_n = \omega_o - \frac{1}{A}\int_0^{\ell} \omega_o t ds = \omega_o - \bar{\omega}_o \tag{C-G8.1-2}$$

$$B_p = F_y \int_0^{\ell} \left|\omega_{np}\right| t ds = F_y Z_w \tag{C-G8.1-3}$$

$$\omega_{np} = \omega_{op} - \tilde{\omega}_{op} \tag{C-G8.1-4}$$

Sectorial coordinates and other variables are explained in *Commentary* Section 2.3.1.1.4. For members with holes, the torsion properties are calculated at the net cross-section where t = 0 at hole locations.

The B_y and B_p calculations can be complex for the general case. For the common shapes shown in Figure C-G8.1-2, symmetry and sharp corners simplify the calculations to the following formulas as derived in Glauz (2023c), where h, b, and d are centerline dimensions, t is the *thickness*, and m is the elastic shear center offset from centerline of *web*. These formulas also apply to sections without lip elements by using d = 0.

C-Sections:

$$\omega_{n,max} = \frac{h}{2}(b-m) + d(b+m) \tag{C-G8.1-5}$$

$$B_p = hb^2 tF_y \left[\frac{1}{2} + \frac{d}{b}\left(1 + \frac{d}{h}\right)\right] \quad \text{for } b \le b_1 \tag{C-G8.1-6}$$

$$B_p = hb^2 tF_y \left[\frac{1}{4} + \frac{h}{8b} - \frac{b_1^2}{4b^2} + \frac{d}{b}\left(\frac{1}{2} + \frac{3d}{2h}\right)\right] \quad \text{for } b > b_1 \tag{C-G8.1-7}$$

where

$$b_1 = h/4 - d + d^2/h \tag{C-G8.1-8}$$

Z-Sections:

$$\omega_{n,max} = \frac{hb(h+b) + d(h+2b+d)(2b\sin\theta + h\cos\theta)}{2h+4b+4d} \tag{C-G8.1-9}$$

$$B_p = hb^2 tF_y \left[\frac{1}{2} + \frac{d}{b}\left(1 + \frac{d}{h}\sin\theta + \frac{d}{2b}\cos\theta\right)\right] \quad \text{for } b \le b_1 \tag{C-G8.1-10}$$

$$B_p = hb^2 tF_y \left[\frac{1}{4} + \frac{h}{4b} - \frac{b_1^2}{4b^2} + \frac{d}{b}\left(\frac{1}{2} + \frac{d}{h}\sin\theta + \frac{d}{2b}\cos\theta\right)\right] \quad \text{for } b > b_1 \tag{C-G8.1-11}$$

where

$$b_1 = h/2 - d \tag{C-G8.1-12}$$

Hat-Sections:

$$\omega_{n,max} = \max\left[\frac{h}{2}(b-m),\ \ d(b+m) - \frac{h}{2}(b-m)\right] \tag{C-G8.1-13}$$

$$B_p = hb^2tF_y\left[\frac{1}{2}+\frac{d}{b}\left(1-\frac{d}{h}\right)\right] \quad \text{for } b \le b_1 \qquad \text{(C-G8.1-14)}$$

$$B_p = hb^2tF_y\left[\frac{1}{4}+\frac{h}{8b}-\frac{b_1^2}{4b^2}+\frac{d}{b}\left(\frac{1}{2}-\frac{3d}{2h}\right)\right] \quad \text{for } b > b_1 \text{ and } d \le d_o \qquad \text{(C-G8.1-15)}$$

$$B_p \approx hb^2tF_y \times \max\left[\frac{1}{4}+\frac{h}{6b},\quad \frac{1}{4}+\frac{h}{8b}+\frac{d^2}{h^2}\left(\frac{1}{4}+\frac{3h}{8b}\right)\right] \text{ for } d_o < d \le h \text{ and } b \le h \qquad \text{(C-G8.1-16)}$$

where

$$b_1 = h/4 - d - d^2/h \qquad \text{(C-G8.1-17)}$$

$$d_o = \frac{h}{2}\left(\frac{b+b_1}{3b-b_1}\right) \qquad \text{(C-G8.1-18)}$$

Angle Sections:

$$\omega_{n,max} = bd-(b-d)m/\sqrt{2} \qquad \text{(C-G8.1-19)}$$

$$B_p = bd^2tF_y \qquad \text{(C-G8.1-20)}$$

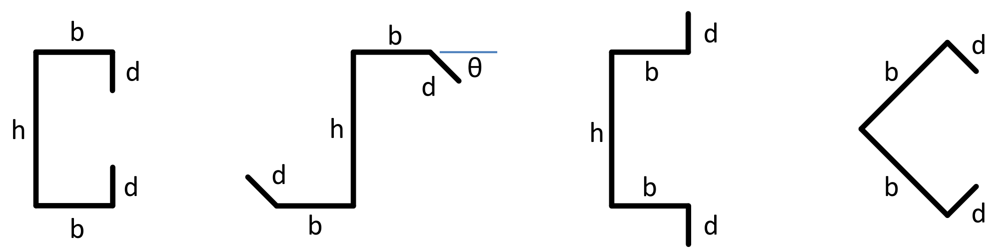
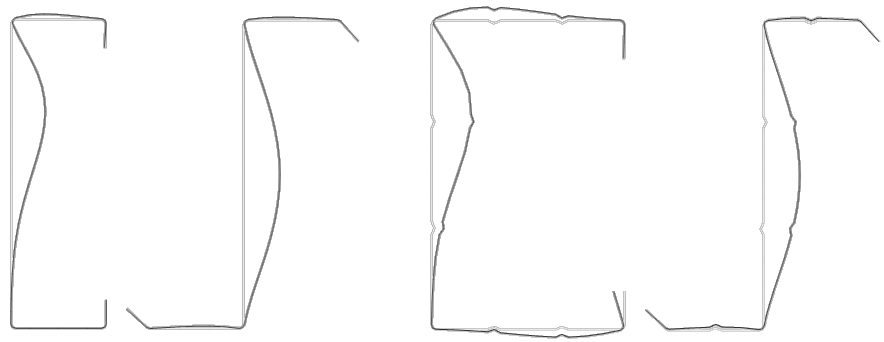
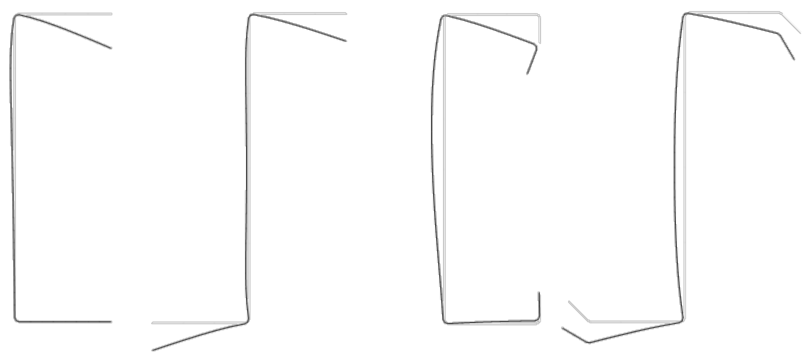


Figure C-G8.1-2 Dimensions for C, Z, Hat and Angle Sections

The critical elastic *buckling* bimoment is the magnitude of bimoment causing *buckling* of the *web* (B_{crw}) or *flange* (B_{crf}) as illustrated in Figure C-G8.1-3. This can be determined using a finite strip or finite element analysis, similar to compression and bending as described in *Commentary* Section 2.2.2. The freely available program, CUFSM, provides this capability.



a) Buckling of Web With or Without Intermediate Stiffeners



b) Buckling of Flange With or Without Edge Stiffeners

Figure C-G8.1-3 Bimoment Elastic Buckling Examples

G8.2 Torsion Shear Strength

(Reserved)

H. MEMBERS UNDER COMBINED FORCES

H1 Combined Axial Load and Bending

In the 1996 edition of the AISI *Specification*, the design provisions for combined axial *load* and bending were expanded to include expressions for the design of members subjected to combined tensile axial *load* and bending. Since the 2001 edition, combined axial and bending for the *Limit States Design* (*LSD*) method has been added. The design approach of the *LSD* method is the same as the *LRFD* method.

H1.1 Combined Tensile Axial Load and Bending

These provisions apply to concurrent bending and tensile axial *load*. If bending can occur without the presence of tensile axial *load*, the member must also conform to the provisions of *Specification* Chapters E, F, Sections I4, I6.1, and I6.2. Care must be taken not to overestimate the tensile *load*, as this could be unconservative.

Specification Equation H1.1-1 provides a design criterion to prevent *yielding* of the tension *flange* of a member under combined tensile axial *load* and bending. Therefore, the *available flexural strengths* [*factored resistances*], M_{axt} and M_{ayt}, are calculated based on the section modulus of full unreduced section relative to the extreme tension fiber. *Specification* Equation H1.1-2 provides a design criterion to prevent failure of the compression *flange*.

H1.2 Combined Compressive Axial Load and Bending

Cold-formed steel members under a combination of compressive axial *load* and bending are usually referred to as beam-columns. The bending may result from eccentric loading, transverse *loads*, or applied moments. Such members are often found in framed structures, trusses, and exterior wall studs. For the design of such members, interaction equations have been developed for locally stable and unstable beam-columns on the basis of thorough comparison with rigorous theory and verified by the available test results (Peköz, 1986a; Peköz and Sumer, 1992).

The structural behavior of beam-columns depends on the shape and dimensions of the cross-section, the location of the applied eccentric *load*, the column length, the end restraint, and the condition of bracing.

In 2007, the *Specification* introduced the *second-order analysis*, which contained the *direct analysis method* approach as an optional method for structural *stability* analysis. In 2016, the *Specification* was reorganized and it provides three methods of design for system *stability*: the *direct analysis method* using rigorous *second-order* elastic *analysis* (Section C1.1), the *direct analysis method* using amplified first-order elastic analysis (Section C1.2) and the *effective length method* (Section C1.3). Since moment magnifications are considered in the system analysis in accordance with *Specification* Section C1, Section H1.2 was revised accordingly by deleting the terms relating to moment amplification ($1/\alpha$) and moment gradient (C_m) as these effects are now handled in Chapter C.

When a beam-column is subjected to an axial *load* $\overline{P}$ and end moments $\overline{M}$ as shown in Figure C-H1.2-1(a), the combined axial and bending *stress* in compression is given in Equation C-H1.2-1 as long as the member remains straight:

$$\bar{f} = \frac{\bar{P}}{A} + \frac{\bar{M}}{S} \tag{C-H1.2-1}$$

$$= \bar{f}_a + \bar{f}_b$$

where

$\bar{f}$ = Combined *stress* in compression

$\bar{P}$ = *Required* axial *load* determined in accordance with *ASD*, *LRFD* or *LSD load combinations*

A = Cross-sectional area

$\bar{M}$ = *Required* bending moment determined in accordance with *ASD*, *LRFD* or *LSD load combinations*

S = Section modulus

$\bar{f}_a$ = Axial compressive *stress*

$\bar{f}_b$ = Bending *stress* in compression

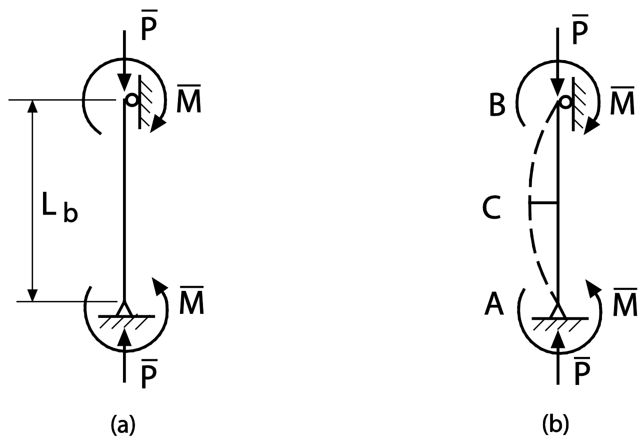


Figure C-H1.2-1 Beam-Column Subjected to Axial Loads and End Moments

In the design of a beam-column by using the *ASD*, *LRFD* or *LSD* method, the combined *stress* should be limited by certain available *stress* F_a; that is,

$$\bar{f}_a + \bar{f}_b \leq F_a$$

or

$$\frac{\bar{f}_a}{F_a} + \frac{\bar{f}_b}{F_a} \leq 1.0 \tag{C-H1.2-2}$$

As specified in Sections F2 and F3, I6.1, I6.2 and Chapter E of the *Specification*, the *safety factor* or *resistance factor* for the design of compression members is different from the *safety factor* or *resistance factor* for beam design. Therefore, Equation C-H1.2-2 may be modified as follows:

$$\frac{\bar{f}_a}{F_{a\_axial}} + \frac{\bar{f}_b}{F_{a\_bending}} \leq 1.0 \tag{C-H1.2-3}$$

where

F_{a_axial} = Available *stress* for the design of compression members

$F_{b\_bending}$ = Available *stress* for the design of beams

If the strength ratio is used instead of the *stress* ratio, Equation C-H1.2-3 can be rewritten as follows:

$$\frac{\overline{P}}{P_a}+\frac{\overline{M}}{M_a}\le 1.0 \qquad \text{(C-H1.2-4)}$$

where

P_a = *Available compressive strength* [*factored resistance*] determined in accordance with Chapter E

M_a = *Available flexural strength* [*factored resistance*] determined in accordance with Chapter F and Sections I6.1 and I6.2, as applicable

Equation C-H1.2-4 is a well-known interaction equation which has been adopted in several specifications for the design of beam-columns. It can be used with reasonable accuracy for short members and members subjected to a relatively small axial *load*. It should be realized that in practical applications, when end moments are applied to the member, it will be bent as shown in Figure C-H1.2-1(b) due to the applied moment, $\overline{M}$, and the secondary moment resulting from the applied axial *load,* $\overline{P}$, and the deflection of the member. This is why the increase of moment in the member due to member deformation (*P-δ effect*), and story sway (*P-Δ effect*), as well as initial imperfections, need to be considered in determining member forces. See Section C1 *Commentary* for further information.

The *Specification* interaction equation contains three strength ratios: axial, bending about the x-axis, and bending about the y-axis. For each strength ratio, the strength can be controlled by either *yielding*, global *buckling*, *local buckling*, or *distortional buckling*. The actual failure mode for a beam-column is either *local buckling* interacting with *yielding* and global *buckling*, or *distortional buckling* interacting with *yielding*. It is therefore conservative to determine the maximum strength ratio from each of the load effects (axial, bending about the x-axis, and bending about the y-axis) and sum these ratios in the interaction equation, even if the *limit states* are different. For a more precise strength that reflects the two actual failure modes for a beam-column, *Specification* Equation H1.2-1 can be applied separately for the two *limit states*: once with all *available strengths* based on *local buckling* interacting with *yielding* and global *buckling* (Eq. C-H1.2-5a), and once with all *available strengths* based on *distortional buckling* interacting with *yielding* (Eq. C-H1.2-5b). The committee determined this added complexity was not warranted in the *Specification*, but recognized these refinements may sometimes be useful and thus provided them here.

$$\frac{\overline{P}}{P_{a\ell}}+\frac{\overline{M}_x}{M_{a\ell x}}+\frac{\overline{M}_y}{M_{a\ell y}}\le 1.0 \qquad \text{(C-H1.2-5a)}$$

$$\frac{\overline{P}}{P_{ad}}+\frac{\overline{M}_x}{M_{adx}}+\frac{\overline{M}_y}{M_{ady}}\le 1.0 \qquad \text{(C-H1.2-5b)}$$

where

$\overline{P}$, $\overline{M}_x$, $\overline{M}_y$ = *Required strengths* [forces and moments due to *factored loads*] based on *ASD*, *LRFD* or *LSD load combinations*

$P_{a\ell}$, $M_{a\ell x}$, $M_{a\ell y}$ = *Available strengths* [*factored resistances*] considering *local buckling* interacting with *yielding* and global *buckling*, not to exceed the *available strengths* [*resistances*] for *yielding* and global *buckling*

P_{ad}, M_{adx}, M_{ady} = *Available strengths* [*factored resistances*] considering *distortional* *buckling* interacting with *yielding*

In 2016, the *Specification* relaxed the requirement that the bending moment ($\overline{M}$) should be defined with respect to the centroidal axis of the effective section. The increased eccentricity due to *local buckling* may exist in an ideally simply-supported member; it becomes minor in continuous members or members with ends restrained so as to reduce such eccentricity. Further, the *Direct Strength Method* utilized in Chapter F for the *available flexural strength* [*factored resistance*], M_a, has shown that accurate bending strength may be determined without consideration of neutral axis shift. In such an approach, the designer does not calculate effective properties or effective axes, and thus it is consistent to remove such a requirement from the beam-column interaction check. Research indicates that use of the gross centroidal axes is adequate for cold-formed steel beam-columns (Torabian, et al. 2013, 2014), and additional work is ongoing.

For the design of angle sections using the *ASD, LRFD* or *LSD* method, the required additional bending moment of PL/1000 about the minor principal axis is discussed in Item C of Chapter E of the *Commentary*.

H2 Combined Bending and Shear

For cantilever beams and continuous beams, high bending *stresses* often combine with high shear *stresses* at the supports. Such beam *webs* must be safeguarded against *buckling* due to the combination of bending and shear *stresses*.

For disjointed flat rectangular plates, the critical combination of bending and shear *stresses* can be approximated by the following interaction equation (Bleich, 1952), which is part of a unit circle:

$$\left(\frac{f_b}{f_{cr}}\right)^2 + \left(\frac{\tau}{\tau_{cr}}\right)^2 = 1.0 \qquad \text{(C-H2-1)}$$

or

$$\sqrt{\left(\frac{f_b}{f_{cr}}\right)^2 + \left(\frac{\tau}{\tau_{cr}}\right)^2} = 1.0 \qquad \text{(C-H2-2)}$$

where f_b is the actual compressive bending *stress*, f_{cr} is the theoretical *buckling stress* in pure bending, τ is the actual shear *stress*, and τ_{cr} is the theoretical *buckling stress* in pure shear. The above equation was found to be conservative for beam *webs* with adequate shear stiffeners, for which a diagonal tension field action may be developed. Based on the studies made by LaBoube and Yu (1978b), Equation C-H2-3 was developed for beam *webs* with shear stiffeners satisfying the requirements of *Specification* Section G4.

$$0.6\frac{f_b}{f_{b_{max}}} + \frac{\tau}{\tau_{max}} = 1.3 \qquad \text{(C-H2-3)}$$

Equation C-H2-3 was added to the AISI *Specification* in 1980. The correlations between Equation C-H2-3 and the test results of beam *webs* having a diagonal tension field action are shown in Figure C-H2-1.

Since 1986, the AISI *Specification* has used strength ratios (i.e., moment ratio for bending and force ratio for shear) instead of *stress* ratios for the interaction equations. *Specification* Equations H2-1 and H2-2 are based on Equations C-H2-2 and C-H2-3, respectively, by using the *available flexural strength* [*factored resistance*], $M_{a\ell o}$, and the *available shear strength* [*factored resistance*], V_a.

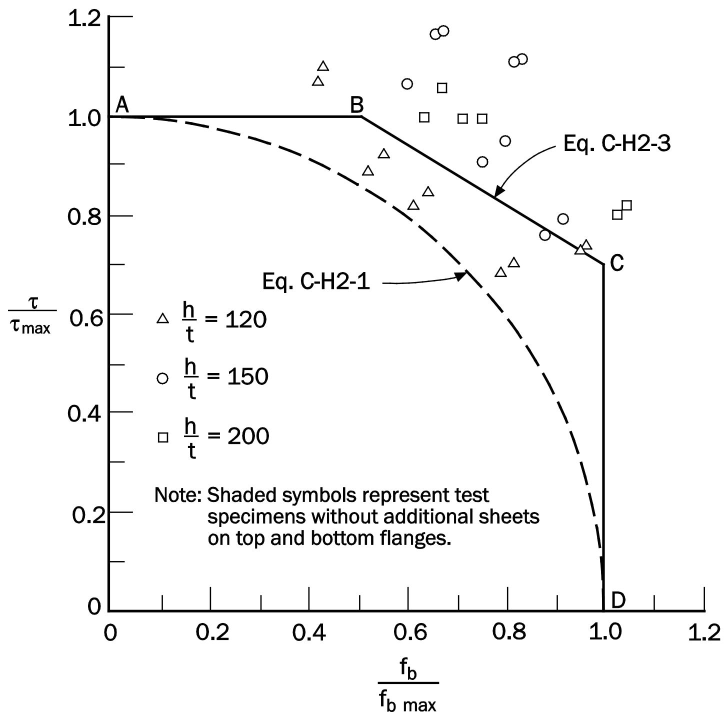


Figure C-H2-1 Interaction Diagram for τ/τ_{max} and f_b/f_{bmax}

The *available flexural strength* [*factored resistance*], $M_{a\ell o}$, for *local buckling* from *Specification* Section F3.1 or F3.2 has been used in the interaction equations since combined bending and shear occur in regions of high moment gradient where *distortional buckling* is unlikely to play a significant role. *Distortional buckling* is checked independently in *Specification* Section F4. Validation of this approach has been confirmed from tests of lapped *purlins* (Pham and Hancock, 2009b) and tests on high-strength steel C-sections in combined bending and shear (Pham and Hancock, 2012a). However, where tension field action given by *Specification* Equation G2.2-1 is used to compute V_a, then *flange* distortion of unrestrained *flanges* requires that *distortional buckling* be considered when computing $M_{a\ell o}$ (Pham and Hancock, 2012a).

H3 Combined Bending and Web Crippling

This *Specification* contains interaction equations for the combination of bending and *web crippling*. *Specification* Equations H3-1 and H3-2 are based on an evaluation of available experimental data using the *web crippling* equation included in the 2001 edition of the *Specification* (LaBoube, Schuster, and Wallace, 2002). The experimental data is based on research studies conducted at the University of Missouri-Rolla (Hetrakul and Yu, 1978 and 1980; Yu, 1981 and 2000), Cornell University (Winter and Pian, 1946), and the University of Sydney (Young and Hancock, 2000). For embossed *webs*, *crippling* strength should be determined by tests according to *Specification* Section K2.

The exception clause included in *Specification* Section H3 for single unreinforced *webs* applies to the interior supports of continuous spans using decks and beams, as shown in Figure C-H3-1. Results of continuous beam tests of steel decks (Yu, 1981) and several independent studies by manufacturers indicate that, for these types of members, the post-*buckling* behavior of *webs* at interior supports differs from the type of failure mode occurring under concentrated *loads* on single-span beams. This post-*buckling* strength enables the member to redistribute the moments in continuous spans. For this reason, *Specification* Equation H3-1 is not applicable to the interaction between bending and the reaction at interior supports of continuous spans. This exception clause applies only to the members shown in Figure C-H3-1 and similar situations explicitly described in *Specification* Section H3.

The exception clause should be interpreted to mean that the effects of combined bending and *web crippling* need not be checked for determining *load*-carrying capacity. Furthermore, the positive bending resistance of the beam should be at least 90 percent of the negative bending resistance in order to ensure the safety implied by the *Specification*.

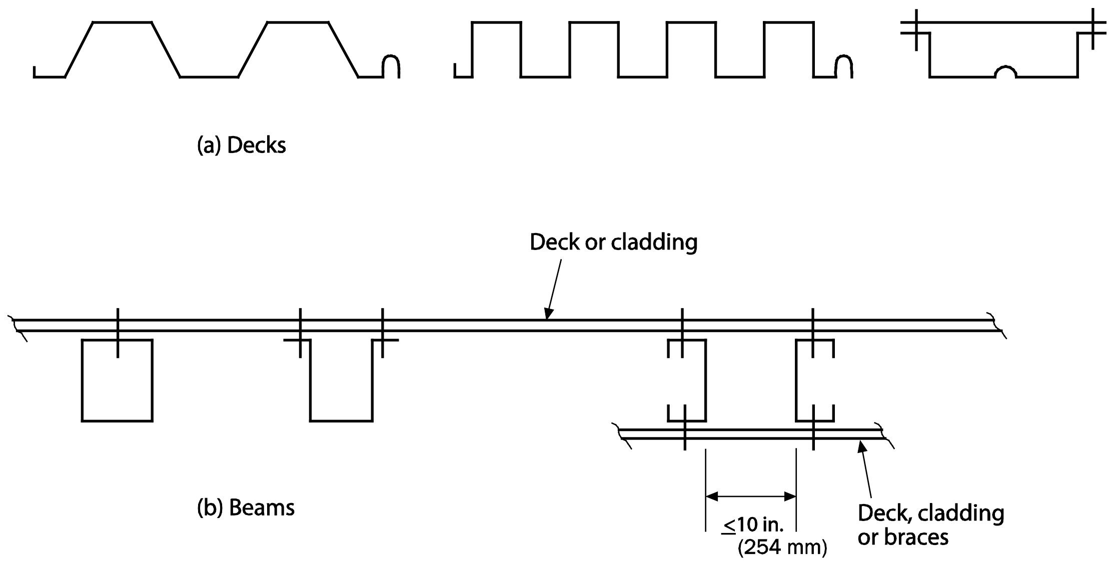


Figure C-H3-1 Sections Used for Exception Clause of Specification Section H3

Using this procedure, the *service loads* may: (1) produce slight deformations in the member over the support, (2) increase the actual compressive bending *stresses* over the support to as high as 0.8 F_y, and (3) result in additional bending deflection of up to 22 percent due to elastic moment redistribution.

If *load*-carrying capacity is not the primary design concern because of the behavior described above, the designer is urged to use *Specification* Equation H3-1.

In 1996, additional design information was added to *Specification* Section H3(c) for two nested Z-shapes. These design provisions are based on the research conducted at the University of Wisconsin-Milwaukee, University of Missouri-Rolla, and a metal building manufacturer (LaBoube, Nunnery and Hodges, 1994). The *web crippling* and bending behavior of unreinforced nested *web* elements is enhanced because of the interaction of the nested *webs*. The design equation is based on the experimental results obtained from 14 nested *web* configurations. These configurations are typically used by the metal building industry.

In 2003, based on the test data of LaBoube, Nunnery, and Hodges (1994), the interaction equation for the combined effects of bending and *web crippling* was reevaluated because a new *web crippling* equation was adopted for Section G5 of the *Specification*.

In the development of the original *LRFD* equations, a total of 551 tests were calibrated for combined bending and *web crippling* strength. Based on ϕ_w = 0.75 for single unreinforced *webs* and ϕ_w = 0.80 for I-sections, the values of the reliability index vary from 2.5 to 3.3 as summarized in the AISI *Commentary* (AISI, 1991).

H4 Combined Bending and Torsion

When the transverse *loads* applied to a bending member do not pass through the shear center of the cross-section of the member, twisting and torsional *stresses* can develop. The torsional *stresses* consist of pure torsional shear *stresses*, shear *stresses* due to warping, and normal *stresses* due to warping. References, such as the AISC Steel Design Guide "Torsional Analysis of Structural Steel Members" (AISC, 1997a), describe the effect of torsion and how these *stresses* may be calculated. *Stresses* due to warping are of particular concern because they are the primary means by which thin cold-formed cross-sections resist torsion (Bian, et al., 2016) and they directly interact with other normal *stresses*.

Open cold-formed steel sections have little resistance to torsion, thus severe twisting and large *stresses* can develop. In many situations, however, the *connection* between a beam and the member delivering the *load* to the beam is such that it constrains twisting and in effect causes the resultant *load* to act as though it is delivered through the shear center. In such cases the torsional effects do not occur. Positive *connections* between the *load*-bearing *flange* and supported elements, in general, prevent torsional effects. An example of this is a *purlin* supporting a through-fastened roof panel that will prevent movement in the plane of the roof panel. It is important that the designer ensure that torsion is adequately constrained when evaluating a specific situation.

In situations where torsional loading cannot be avoided, discrete bracing will reduce the torsional effects. For most situations, the maximum torsional warping *stresses* will occur at discrete brace locations. Torsional bracing at the third points of the span would be adequate for most light construction applications. The bracing should be designed to prevent torsional twisting at the braced points.

Provisions for the interaction between bending and torsion were first introduced in the 2007 edition of the *Specification*. These provisions limited the combined longitudinal *stresses* resulting from bending and torsional warping. In 2022, these provisions were changed to a linear interaction equation involving major and minor axis bending, and torsional bimoment. In 2024, this interaction equation was refined based on the research by Xia et al. (2023) to utilize the *bimoment buckling strength* with inelastic reserve and to consider the flexural failure modes of *lateral-torsional buckling* and *distortional buckling*. For members with discrete bracing, the location of maximum bimoment is typically at a brace where *lateral-torsional buckling* need not be considered. For members without discrete bracing, the controlling location will generally be within the span, for which *lateral-torsional buckling* must be considered. A higher interaction limit is permitted for C- and Z-sections where the bimoment reduces the flexural *stresses* at the free edge of the compression *flange*. This occurs for an unbraced member with gravity load on the compression *flange*, and at an intermediate brace point with gravity load on the other side of the shear center.

The applied bimoment is calculated as $EC_w\theta''$, where E is the modulus of elasticity, C_w is the torsional warping constant, and θ'' is the second derivative of the twist angle with respect to the longitudinal axis of the member resulting from the torsional loads applied to the member. This can be determined using a beam analysis which properly accounts for torsional warping behavior. Other sources exist for manual calculation of torsional warping (e.g., AISC, 1997a).

I. ASSEMBLIES AND SYSTEMS

I1 Built-Up Sections

I-sections made by connecting two C-sections back-to-back are one type of built-up section that is often used as either flexural or compression members. Cases (2) and (8) of Figure C-A1.3-2 and Cases (3) and (7) of Figure C-A1.3-3 show several built-up I-sections. For built-up flexural members, the *Specification* is limited to two back-to-back C-sections. For built-up compression members, other sections can be used.

I1.1 Flexural Members Composed of Two Back-to-Back C-Sections

For the I-sections to be used as flexural members, the longitudinal spacing of connectors is limited by Equation I1.1-1 of the *Specification*. The first requirement is an arbitrarily selected limit to prevent any possible excessive distortion of the top *flange* between connectors. The second requirement is based on the strength and arrangement of connectors and the intensity of the *load* acting on the beam (Yu and LaBoube, 2010).

The second requirement for maximum spacing of connectors required by *Specification* Equation I1.1-1 is based on the fact that the shear center of the C-section is neither coincident with nor located in the plane of the *web*; and that when a *load*, Q, is applied in the plane of the *web*, it produces a twisting moment, Qm, about its shear center, as shown in Figure C-I1.1-1. The tensile force of the top connector, T_s, can then be computed from the equality of the twisting moment, Qm, and the resisting moment, $T_s g$; that is:

$$Qm = T_s g \tag{C-I1.1-1}$$

$$T_s = \frac{Qm}{g} \tag{C-I1.1-2}$$

Considering that q is the intensity of the *load* and that s is the spacing of connectors as shown in Figure C-I1.1-2, the applied *load* is Q = qs/2. The maximum spacing, s_{max}, used in the *Specification* can easily be obtained by substituting the above value of Q into Equation C-I1.1-2 of this *Commentary*. The determination of the *load* intensity, q, is based upon the type of loading applied to the beam. The requirement of three times the uniformly distributed *load* is applied to reflect that the assumed uniform *load* will not really be uniform. The *Specification* prescribes a conservative estimate of the applied loading to account for the likely concentration of *loads* near the welds or other connectors that join the two C-sections.

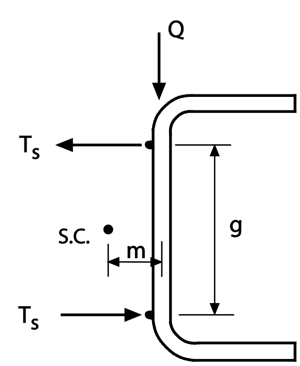


Figure C-I1.1-1 Tensile Force Developed in the Connector for C-Section

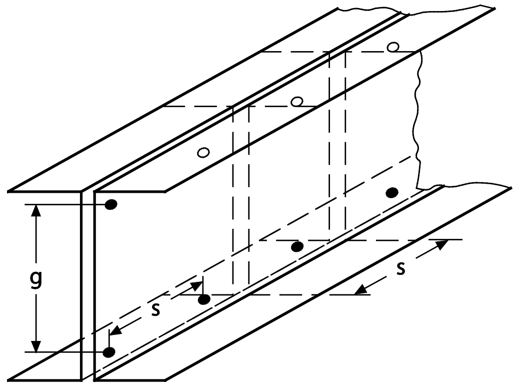


Figure C-I1.1-2 Spacing of Connectors

For simple C-sections without stiffening lips at the outer edges,

$$m = \frac{w_f^2}{2w_f + d/3} \tag{C-I1.1-3}$$

For C-sections with stiffening lips at the outer edges,

$$m = \frac{w_f dt}{4I_x}\left[w_f d + 2D\left(d - \frac{4D^2}{3d}\right)\right] \tag{C-I1.1-4}$$

where

w_f = Projection of *flanges* from the inside face of the *web* (for C-sections with *flanges* of unequal width, w_f should be taken as the width of the wider *flange*)

d = Depth of C-section or beam

D = Overall depth of lip

I_x = Moment of inertia of one C-section about its centroidal axis normal to the *web*

In addition to the above considerations on the *required strength* [force due to *factored loads*] of *connections*, the spacing of connectors should not be so great as to cause excessive distortion between connectors by separation along the top *flange*. In view of the fact that C-sections are connected back-to-back and are continuously in contact along the bottom *flange*, a maximum spacing of L/3 may be used. Considering the possibility that one connection may be defective, a maximum spacing of s_{max} = L/6 is the first requirement in *Specification* Equation I1.1-1.

I1.2 Compression Members Composed of Multiple Cold-Formed Steel Members

I1.2.1 General Requirements

Where multiple cold-formed steel members participate in carrying a compressive *load*, the total axial strength can be increased based on composite action with sufficient connectivity. The *connections* between members can influence global, *local*, and *distortional buckling* behavior of the members.

The *connection* requirements to achieve composite action for these *buckling* modes are covered in *Specification* Sections I1.2.2, I1.2.3, and I1.2.4, respectively. If these *connection* requirements are not met, or are not considered in design, the strength of the combined members is limited to the sum of the strengths of the separate members acting individually.

I1.2.2 Yielding and Global Buckling

Compression members composed of two or more shapes joined together at discrete points have a reduced shear rigidity. The influence of this reduced shear rigidity is taken into account by reducing the *stiffness* used to calculate the elastic critical *buckling stress.* This *stiffness* reduction was previously handled by increasing the *effective length* (Bleich, 1952), but in 2022 the reduction was changed to a reduced moment of inertia. The overall slenderness and the local slenderness between connected points both influence the compressive resistance. The combined action is expressed by the reduced moment of inertia given by *Specification* Equation I1.2.2.1-1.

Note that the reduced moment of inertia, I_r, is for the same axis of *flexural buckling* as the *effective length*, KL, and the radius of gyration, r. However, the minimum radius of gyration, r_i, is for the minor principal axis of the individual shape. The reduced moment of inertia should be utilized for both *flexural* and *flexural-torsional buckling*. This *stiffness* reduction is similar to the modified slenderness used in other steel standards, including AISC (AISC, 1999, 2005, 2010a) and CAN/CSA S16.1 (CSA Group, 1994a).

In addition to this modification for reduced shear rigidity, the *Specification* includes requirements for the *connections*.

(a) Intermediate connection spacing

To prevent the *flexural buckling* of the individual shapes between intermediate connectors, the spacing of intermediate fasteners, a, is limited such that a/r_i does not exceed one-half the governing slenderness ratio of the built-up member (i.e., $a/r_i \leq 0.5(KL/r)$). This fastener spacing requirement is consistent with earlier editions of the AISI *Specification*, with the one-half factor included to account for any one of the connectors becoming loose or ineffective.

(b) Intermediate connection strength

The intermediate fastener(s) or weld(s) at any longitudinal member tie location is required, as a group, to transmit a force equal to 2.5 percent of the *nominal axial strength* [*resistance*] of the built-up member. A longitudinal member tie is defined as a location of interconnection of the two members in contact. In the 2001 edition of the *Specification*, a 2.5 percent total force determined in accordance with appropriate *load* combinations was used for design of the intermediate fastener(s) or weld(s). This requirement was adopted from CSA S136-94. In 2004, the requirement was changed to be a function of the *nominal axial strength* [*resistance*]. This change ensures that the *nominal axial strength* [*resistance*] of the built-up member is valid and is not compromised by the strength of the member interconnections. To avoid confusion for different design methods, the minimum *required strength* [force due to *factored loads*] of the interconnection changed to 2.5 percent of the *available strength* [*factored resistance*] of the built-up member.

(c) End connection strength

The shear forces produced when a built-up column undergoes *flexural buckling* are greatest at the ends of the *effective length.* Preventing end slip is important to maintaining composite action. The shear force resisted by the *connections* at these locations is conservatively based on the total shear force developed along half the *effective length*, and determined at the point where the buckled state reaches the *available flexural strength* [*factored resistance*] at the midpoint of the *effective length* (see Glauz, 2023b).

The shear *stiffness* of the end *connections*, and to some extent the intermediate *connections*, impacts the effectiveness of composite action. The mechanics of this behavior was investigated by Rasmussen et al. (2020) where complex *stiffness* expressions were derived and evaluated. The strength-based provisions for these *connections* in the *Specification* generally result in *connections* which produce composite *stiffness* exceeding the reduced shear rigidity associated with the reduced moment of inertia (see Glauz, 2023b).

The ends of the *effective length* are inflection points of the buckled shape, which often occur at the top and bottom of a column. However, where columns are continuous or end conditions have rotational restraint, these inflection points may occur within a column span. It is important to recognize these cases and locate the required end *connections* accordingly. For these cases, the end *connections* are required both above and below the inflection points, resisting shear for their respective *effective lengths*.

Each possible axis of *flexural buckling* must be considered when determining the required *connection* strength. This typically includes the major and minor principal axes, and may include others if bracing directions do not align with these axes. The variables $M_{a\ell o}$, Q and I_g in *Specification* Equation I1.2.2.1-2 are specific to the axis of *buckling* evaluated, and the first moment of area, Q, is associated with the connected shape or shapes. Examples are shown in Figure C-I1.2.2-1, where the axis of *buckling*, connected shape(s), and participating end *connection* locations are identified.

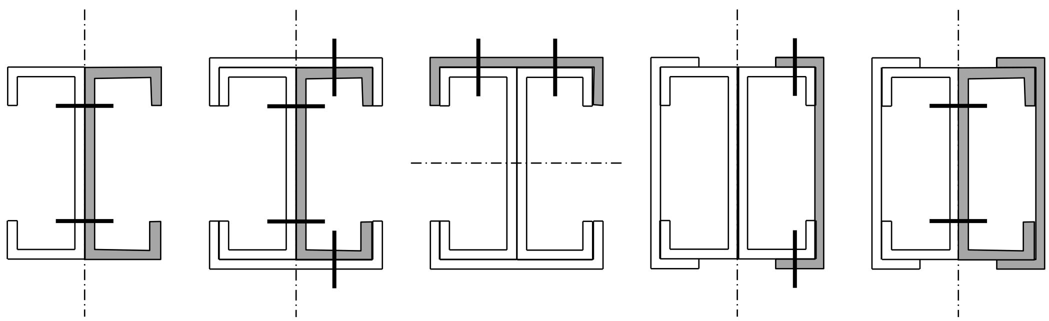


Figure C-I1.2.2-1 Examples of End Connections Participating in Shear

Research by Stone and LaBoube (2005) demonstrated that when a built-up stud is seated in a track section and bears on a firm surface, end slip is precluded, and thus the need for the additional fasteners at the end of the *effective length* is not required. In order to justify use of the exception to end *connections* given in the *Specification*, the end of the built-up member must bear on a surface that prevents relative slip between the two cross-sections. This requires full support over the entire area of the built-up member by steel or concrete components and that bearing stresses on the steel or concrete are within allowable limits.

The intermediate *connection* requirements in (a) and (b) have been substantially taken from research on hot-rolled built-up members connected with bolts or welds. These hot-rolled provisions have been extended to include other fastener types common in cold-formed steel construction (such as screws) provided they meet the 2.5 percent requirement for shear strength and the conservative spacing requirement $a/r_i \leq 0.5(KL/r)$.

I1.2.3 Local Buckling Interacting With Yielding and Global Buckling

The wavelength for *local buckling* is relatively short, thus intermittent *connections* are ineffective in increasing strength. But elements connected with continuous longitudinal welds can behave as stiffened elements, where welded edges can be treated as simply supported such as the edges at fold lines. For slender elements, the stiffening provided at locations of continuous welds can significantly increase *local buckling* strength.

It is important to recognize that a weld line acts only as a simply supported edge of the element. Where an element lays flat against another element and the edges are welded, the elements cannot be treated as having a combined thickness.

I1.2.4 Distortional Buckling

Research studies have shown that typical *connections* in built-up members do not have significant impact on *distortional buckling* strength. Fratamico, et al. (2018) observed that *distortional buckling* is modestly influenced by end conditions and *web* interconnection, but meaningfully improved by the presence of sheathing. The specific improvement in *distortional buckling* strength is affected by the location and spacing of *connections*, and the geometric characteristics of the connected parts.

The *Specification* permits the use of a rational elastic analysis to determine any potential increase in *distortional buckling* strength. Such an analysis must consider the specific geometry utilized, and meticulously account for *connections* and attached components by modeling them with appropriate elastic *stiffnesses*.

I1.3 Spacing of Connections in Cover-Plated Sections

When compression elements are joined to other parts of built-up members by intermittent *connections*, these connectors must be closely spaced to develop the required strength of the connected element. Figure C-I1.3-1 shows a box-shaped beam made by connecting a flat sheet to an inverted hat section. If the connectors are appropriately placed, this flat sheet will act as a stiffened compression element with a width, w, equal to the distance between rows of connectors, and the sectional properties can be calculated accordingly. This is the intent of the provisions in Section I1.3 of the *Specification*.

Section I1.3(a) of the *Specification* requires that the necessary shear strength be provided by the same standard structural design procedure that is used in calculating *flange connections* in bolted or welded plate girders or similar structures.

Section I1.3(b) of the *Specification* ensures that the part of the flat sheet between two adjacent connectors will not buckle as a column (see Figure C-I1.3-1) at a *stress* less than $1.67f_c$ for *ASD* and f_c for *LRFD* and *LSD*, where f_c is the *compressive stress* in the connected compression element (Winter, 1970; Yu and LaBoube, 2010). The AISI requirement is based on the following Euler equation for column *buckling*:

$$\sigma_{cr} = \frac{\pi^2 E}{(KL/r)^2}$$

by substituting $\sigma_{cr} = \alpha f_c$, where α=1.67 for *ASD* and α=1.0 for *LRFD* or *LSD,* K = 0.6, L = s, and $r = t/\sqrt{12}$. This provision is conservative because the length is taken as the center distance instead of the clear distance between connectors, and the coefficient K is taken as 0.6 instead of 0.5, which is the theoretical value for a column with fixed end supports.

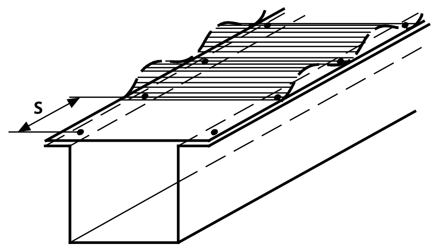


Figure C-I1.3-1 Spacing of Connectors in Composite Section

Section I1.3(c) ensures satisfactory spacing to make a row of connectors act as a continuous line of stiffening for the flat sheet under most conditions (Winter, 1970; Yu and LaBoube, 2010).

Specification Section 1.1.4 extends the limits of this section and uses the post-*buckling* strength of the edge-stiffened compression plate. *Specification* Section 1.1.4 specifies the parameter ranges that are validated by the research (Luttrell and Balaji, 1992; Snow and Easterling, 2008).

I2 Floor, Roof, or Wall Steel Diaphragm Construction

In building construction, it has been a common practice to provide a separate bracing system to resist horizontal *loads* due to wind load, blast force, or earthquake. However, steel floor and roof panels, with or without concrete fill, are capable of resisting horizontal *loads* in addition to the bending strength for gravity loads if they are adequately interconnected to each other and to the supporting frame. The effective use of steel floor and roof decks can therefore eliminate separate bracing systems and result in a reduction of building costs. For the same reason, wall panels can not only provide enclosure surface and support normal *loads*, but they can also provide *diaphragm* action in their own planes.

With the publication of AISI S310, *North American Standard for the Design of Profiled Steel Diaphragm Panels*, the provisions in *Specification* Section I2 have moved to AISI S310. See AISI S310-C for background information on floor, roof and wall steel *diaphragm* construction. See AISI S240 and AISI S400 for information on the design and construction of cold-formed steel framing with diagonal bracing or covered with sheathings other than fluted panels or cellular deck.

In 2020, because they are independently adopted by the *applicable building code*, references to AISI S240 and AISI S400 were deemed unnecessary in the U.S. and Mexico, and were removed from this section of the *Specification*. Each of those standards adopts AISI S100 as appropriate.

I3 Mixed Systems

When cold-formed steel members are used in conjunction with other construction materials, the design requirements of the other material specifications must also be satisfied.

I3.1 Composite Design

Steel-concrete composite construction is very popular owing to the efficient use of the two constituent materials. However, the *Specification* does not provide provisions for cold-formed steel-concrete composite design. The *Specification* permits the use of rational analysis, as well as testing. Other design standards exist for specific structural members incorporating cold-formed steel. For example, the design of composite steel deck-slabs is covered by the applicable Steel Deck Institute Standard (2022) in the United States and the applicable Canadian Sheet

Steel Building Institute Standard (2023) in Canada.

Consideration should be given to the impact of any loss of *stiffness* of the shear *connections* because of the thin nature of typical cold-formed steel sections/shapes. The *Specification* permits to the use of tests to determine the *stiffness* of the shear *connections*.

I4 Cold-Formed Steel Light-Frame Construction

In 2007, the scope of Section I4 on "Wall Studs and Wall Stud Assemblies" of the 2001 edition of the *Specification* with 2004 Supplement was broadened to include light-frame construction. This was done in order to recognize the growing use of cold-formed steel framing in a broader range of residential and light commercial framing applications and to provide a means for either requiring or accepting use of the various ANSI-approved standards that have been developed by the AISI Committee on Framing Standards.

In 2012, the reference to nonstructural members was removed from Section I4 because the provisions for nonstructural members were moved from AISI S200, *North American Standard for Cold-Formed Steel Framing - General Provisions,* to the newly developed AISI S220, *North American Standard for Cold-Formed Steel Framing – Nonstructural Members*.

In 2016, the provisions for the design and installation of *structural members* and *connections* utilized in cold-formed steel light-frame construction applications were consolidated in AISI S240, *North American Standard for Cold-Formed Steel Structural Framing,* from the following previously referenced standards:

(a) AISI S200, *North American Standard for Cold-Formed Steel Framing – General Provisions*
(b) AISI S210, *North American Standard for Cold-Formed Steel Framing – Floor and Roof System Design*
(c) AISI S211, *North American Standard for Cold-Formed Steel Framing – Wall Stud Design*
(d) AISI S212, *North American Standard for Cold-Formed Steel Framing – Header Design*
(e) AISI S213, *North American Standard for Cold-Formed Steel Framing – Lateral Design*
(f) AISI S214, *North American Standard for Cold-Formed Steel Framing – Truss Design*

In 2016, AISI S400 was developed to address the design and construction of *cold-formed steel structural members* and *connections* in seismic force-resisting systems and *diaphragms* in buildings and other structures. AISI S400 is applicable in the United States and Mexico in *Seismic Design Categories* (SDC) D, E, or F, or in SDC B or C with seismic response modification coefficient, R, used to determine the seismic design forces is taken as other than 3; and in Canada where the design spectral response acceleration S(0.2) as specified in the NBCC is greater than 0.12 and the seismic force modification factors, R_dR_o, used to determine the seismic design forces, are taken as greater than or equal to 1.56.

AISI S220, AISI S240 and AISI S400 are available for adoption and use in the United States, Canada and Mexico, and provide an integrated treatment of *Allowable Strength Design* (*ASD*), *Load and Resistance Factor Design* (*LRFD*), and *Limit States Design* (*LSD*). These framing standards do not preclude the use of other materials, assemblies, structures or designs not meeting the criteria herein when the other materials, assemblies, structures or designs demonstrate equivalent performance for the intended use to those specified in the standards.

In 2020, because they are independently adopted by the *applicable building code*, references to AISI S240 and AISI S400 were deemed unnecessary in the U.S. and Mexico, and were removed from this section of the *Specification*. Each of those standards adopts AISI S100 as appropriate.

In 2024, Section I4.1 All-Steel Design of Wall Stud Assemblies was removed from the

Specification. The pertinent commentary of Section I4.1 was moved to the Commentary of Section B1.2.2 of AISI S240. All-steel design is still a valid method and is fully enabled and detailed in AISI S240 and thus a specific callout in AISI S100 was no longer required.

I5 Special Bolted Moment Frame Systems

In 2015, AISI S110, *Standard for Seismic Design of Cold-Formed Steel Structural Systems - Special Bolted Moment Frames*, was incorporated into AISI S400.

In 2020, because it is independently adopted by the *applicable building code*, reference to AISI S400 was deemed unnecessary and removed from this section of the *Specification*. AISI S400 adopts AISI S100 as appropriate for the design of the CFS-SBMF seismic-force resisting system.

I6 Metal Roof and Wall Systems

For members connected to deck or metal sheathing, the member flexural and compression strengths as well as bracing requirements are provided in *Specification* Section I6. Two strength prediction methods are provided—one for general cross-sections and system connectivity (Section I6.1), and one for specific cross-sections and system connectivity (Section I6.2). The provisions in *Specification* Section I6.1 directly calculate member capacity, including stiffness from connected roof or wall panels, bridging and bracing, span continuity, and torsion from loading eccentric to the shear center and from roof slope. The provisions in *Specification* Section I6.2 define wall and roof system capacity based on past experiments for variables within defined limits.

I6.1 Member Strength: General Cross-Sections and System Connectivity

This method provides a means for directly calculating the axial and flexural capacity of members (such as *purlins* and *girts*) connected to deck, sheathing, or through-fastened or standing seam panels. The approach employs the *Direct Strength Method* and available computational tools; for example, the finite strip elastic *buckling* program CUFSM (Li and Schafer, 2010).

An elastic *buckling* analysis is performed that includes the test-derived rotational, translational, and composite stiffness provided by the panel or sheathing *connection* to the members (Schafer, 2013; Gao and Moen, 2013a). The member critical elastic *local*, *distortional*, and global *buckling* loads or moments are calculated considering wall or roof *connection* stiffness, end support conditions, *span continuity*, and bridging and bracing. Member slenderness, including the wall or roof system influence, is determined within the *Direct Strength Method* to predict axial or flexural capacity.

Panel, deck, and sheathing rotational and translational stiffnesses are available for bare deck through-fastened to members (Gao and Moen, 2012; Pham et al., 2016), deck with rigid board insulation (Gao, 2012), and for through-fastened and standing seam insulated metal panels (IMPs) (Wu and Moen, 2015). Composite stiffness developed by the *connection* between the panel and a member can also be approximated (Vieira, 2011).

In many cases, the applied load on a member is eccentric to its shear center from forces applied through the *flange connection* or because of a sloped roof. In these cases, the combined effects of bending and torsion are checked in accordance with *Specification* Section H4.

The method described above can be applied to members with, generally, any cross-section and system connectivity. Supporting documentation for this method applied to metal building wall and roof systems comes from experimental, computational, and analytical studies conducted between 2009 and 2015, including Gao and Moen (2013a and 2013b). Example calculations are available for many of these systems (Moen, 2015), including standing seam roofs (Moen, et al., 2012).

The design methodology for general cross-sections and system connectivity has been thoroughly validated. The strength predictions were compared to a database of 62 through-fastened roof and wall tests containing the same experiments that form the basis for the provisions of Section I6.2. The test-to-predicted mean and coefficient of variance (COV) for this database comparison is 1.05 and 0.18, respectively, corresponding to an *LRFD resistance factor* of 0.90. Extensive validation also exists for sheathed cold-formed steel framing (Vieira, 2011).

I6.2 Member Strength: Specific Cross-Sections and System Connectivity

I6.2.1 Flexural Members Having One Flange Through-Fastened to Deck or Sheathing

For beams having the tension *flange* attached to deck or sheathing and the compression *flange* unbraced, e.g., a roof *purlin* or wall *girt* subjected to wind suction, the bending capacity is less than a fully braced member, but greater than an unbraced member. This partial restraint is a function of the rotational stiffness provided by the panel-to-*purlin connection*. The *Specification* contains factors that represent the reduction in capacity from a fully braced condition. These factors are based on experimental results obtained for both simple and continuous span *purlins* (Peköz and Soroushian, 1981 and 1982; LaBoube, 1986; Haussler and Pahers, 1973; LaBoube, et al., 1988; Haussler, 1988; Fisher, 1996).

The R factors for simple span C-sections and Z-sections up to 8.5 inches (216 mm) in depth have been increased from the 1986 *Specification*, and a member design *yield stress* limit added based on the work by Fisher (1996).

As indicated by LaBoube (1986), the rotational *stiffness* of the panel-to-*purlin connection* is primarily a function of the member *thickness*, sheet *thickness*, fastener type and fastener location. To ensure adequate rotational *stiffness* of the roof and wall systems designed using the AISI provisions, *Specification* Section I6.2.1 explicitly states the acceptable panel and fastener types.

Continuous beam tests were made on three equal spans and the R values were calculated from the failure loads using a maximum positive moment, $M = 0.08\ wL^2$.

The provisions of *Specification* Section I6.2.1 apply to beams for which the tension *flange* is attached to deck or sheathing and the compression *flange* is completely unbraced. Beams with discrete point braces on the compression *flange* may have a bending capacity greater than those completely unbraced. Available data from simple span tests (Peköz and Soroushian, 1981 and 1982; LaBoube and Thompson, 1982a; LaBoube, et al., 1988; LaBoube and Golovin, 1990) indicate that for members having a lip edge stiffener at an angle of 75 degrees or greater with the plane of the compression *flange* and braces to the compression *flange* located at third points or more frequently, member capacities may be increased over those without discrete braces.

For the *LRFD* method, the use of the reduced *nominal flexural strength* [*resistance*] (*Specification* Equation I6.2.1-1) with a *resistance factor* of $\phi_b = 0.90$ provides the β values varying from 1.5 to 1.60, which are satisfactory for the target value of 1.5. This analysis was based on the *load* combination of 1.17 W - 0.9D using a reduction factor of 0.9 applied to the *load factor* for the *nominal* wind *load*, where W and D are nominal wind and dead *loads*, respectively (Hsiao, Yu and Galambos, 1988a; AISI, 1991).

In 2007, the panel depth was reduced from 1-1/4 inch (32 mm) to 1-1/8 inch (29 mm). This reduction in depth was justified because the behavior during full-scale tests indicated that the panel deformation was restricted to a relatively small area around the screw attachment of the panel to the *purlin*. Also, tests by LaBoube (1986) demonstrated that the panel depth did not influence the rotational stiffness of the panel-to-*purlin* attachment.

Prior to the 2007 edition, the *Specification* specifically limited the applicability of these provisions to continuous *purlin* and *girt* systems in which "the longest member span length shall not be more than 20 percent greater than the shortest span length". This limitation was included in recognition of the fact that the research was based on systems with equal bay spacing. In 2007, the *Specification* was revised to permit *purlin* and *girt* systems with adjacent span lengths varying more than 20 percent to use the reduction factor, R, for the simply supported condition. The revision allows a row of continuous *purlins* or *girts* to be treated with a continuous beam condition R-factor in some bays and a simple span beam condition R-factor in others. The 20 percent span variation rule is a local effect and as such, only variation in adjacent spans is relevant. CCFSS Technical Bulletin, Vol. 15, No. 2 (CCFSS, 2006) provided the technical justification for this revision.

In 2012, based on tests reported by Wibbenmeyer (2009), the limitation on the member depth was increased to 12 in. (305 mm), the ratio of depth-to-*flange* width was increased to 5.5, and a minimum *flange* width of 2.125 in. (54.0 mm) was added. The ratio of *tensile strength* to *yield stress* of 1.08 was added based on research at the University of Sydney (Pham and Hancock, 2009), which is also consistent with the applicable steels listed in *Specification* Section A2. The average depth-to-*flange* width ratio based on measured properties in the research by Wibbenmeyer (2009) was 5.3. However, the limit was increased to 5.5 in the *Specification*. This increased value was justified because the smallest measured *purlin flange* width for any of the members tested by Wibbenmeyer (2009) was 2.1875 in. (71.56 mm), which resulted in a ratio of depth-to-*flange* width of 5.5. Also, the reported value of R for the 12-in. (305-mm) deep *purlins* significantly exceeded those previously stipulated for 11.5-in. (292-mm) deep members.

The provisions of *Specification* Section H4, Combined Bending and Torsion, should not be used in combination with the bending provisions in *Specification* Section I6.2.1 since these provisions are based on tests in which torsional effects are present.

I6.2.2 Flexural Members Having One Flange Fastened to a Standing Seam Roof System

The design provision of this section is only applicable to the United States and Mexico. The discussion for this section is provided in the *Commentary* Appendix A. ☜**A**

I6.2.3 Compression Members Having One Flange Through-Fastened to Deck or Sheathing

For axially loaded C- or Z-sections having one *flange* attached to deck or sheathing and the other *flange* unbraced, e.g., a roof *purlin* or wall *girt* subjected to wind- or seismic-generated compression forces, the axial load capacity is less than a fully braced member, but greater than an unbraced member. The partial restraint relative to weak axis *buckling* is a function of the rotational stiffness provided by the panel-to-*purlin connection*. *Specification* Equation I6.2.3-1 is used to calculate the weak axis capacity. This equation is not valid for sections attached to standing seam roofs. The equation was developed by Glaser, Kaehler and Fisher (1994) and is also based on the work contained in the reports of Hatch, Easterling and Murray (1990), and Simaan (1973).

A limitation on the maximum *yield stress* of the C- or Z-section is not given in the *Specification* since *Specification* Equation I6.2.3-1 is based on elastic *buckling* criteria. A limitation on minimum length is not contained in the *Specification* because Equation I6.2.3-1 is conservative for spans less than 15 feet. The *gross area*, A, has been used rather than the *effective area*, A_e, because the ultimate axial *stress* is generally not large enough to result in a significant reduction in the *effective area* for common cross-section geometries.

As indicated in the *Specification*, the strong axis axial load capacity is determined by assuming that the weak axis of the strut is braced.

The controlling axial capacity (weak or strong axis) is suitable for usage in the combined axial *load* and bending equations in Section H1 of the *Specification* (Hatch, Easterling, and Murray, 1990).

Note: As stated in the *Specification*, when a member is designed in accordance with Section I6.2.3, Compression Members Having One Flange Through-Fastened to Deck or Sheathing, the provisions of Section E4.1, Distortional Buckling Strength [Resistance], need not be applied since *distortional buckling* is inherently included as a limit state in Section I6.2.3 on strength prediction equations.

I6.2.4 Z-Section Compression Members Having One Flange Fastened to a Standing Seam Roof

The design provision of this section is only applicable to the United States and Mexico. The discussion for this section is provided in the *Commentary* Appendix A. ☜A

I6.3 Standing Seam Roof Panel Systems

I6.3.1 Strength [Resistance] of Standing Seam Roof Panel Systems

Under gravity loading, the *nominal strength* [*resistance*] of many panels can be calculated accurately. Under uplift loading, *nominal strength* [*resistance*] of standing seam roof panels and their attachments or anchors cannot be calculated with accuracy. Therefore, it is necessary to determine the *nominal strength* [*resistance*] by testing. Three test protocols have been used in this effort: FM 4471 developed by Factory Mutual, CEGS 07416 by the U.S. Army Corps of Engineers and ASTM E1592. In Supplement No. 1 to the 1996 edition of the *Specification*, (AISI, 1999), only the ASTM E1592-95 procedure was approved. In 2004, the Factory Mutual and Corps of Engineers protocols were also approved, provided that testing was in accordance with the AISI test procedure defined in S906 (AISI, 2002). While these test procedures have a common base, none define a *design strength* [*factored resistance*]. *Specification* Section I6.3.1 and AISI S906, *Standard Procedures for Panel and Anchor Structural Tests*, adopted in 1999, added closure to the question by defining appropriate *resistance* and *safety factors*. The *safety factors* determined in Section I6.3.1 will vary depending on the characteristics of the test data. In 2006, limits were placed on the *safety factor* and *resistance factor* determined in this section to require a minimum *safety factor* of 1.67 and a maximum *resistance factor* of 0.9.

The *Specification* permits end conditions other than those prescribed by ASTM E1592-01. Areas of the roof plane that are sufficiently far enough away from crosswise restraint can be simulated by testing the open/open condition that was permitted in the 1995 edition of

ASTM E1592. In addition, eave and ridge configurations that do not provide crosswise restraint can be evaluated.

The relationship of strength to serviceability limits may be taken as strength limit/serviceability limit = 1.25, or

$$\Omega_{serviceability} = \Omega_{strength}/1.25 \qquad \text{(C-I6.3.1-1)}$$

It should be noted that the purpose of the test procedure specified in *Specification* Section I6.3.1 is not to set up guidelines to establish the serviceability limit. The purpose is to define the method of determining the *available strength* [*factored resistance*] whether based on the serviceability limit or on the *nominal strength* [*resistance*]. The Corps of Engineers Procedure CEGS 07416 (1991) requires a *safety factor* of 1.65 on strength and 1.3 on serviceability. A *buckling* or crease does not have the same consequences as a failure of a clip. In the latter case, the roof panel itself may become detached and expose the contents of a building to the elements of the environment. Further, Galambos (1988a) recommended a value of 2.0 for the target reliability index, β_o, when slight damage is expected and a value of 2.5 when moderate damage is expected. The resulting ratio is 1.25.

In *Specification* Section I6.3.1, a target reliability index of 2.5 is used for *connection* limits. It is used because the consequences of a panel fastener failure (β_o = 2.5) are not nearly as severe as the consequences of a primary frame *connection* failure (β_o = 3.5). The intermittent nature of wind *load* as compared to the relatively long duration of snow *load* further justifies the use of β_o = 2.5 for panel anchors. In *Specification* Section I6.3.1, the coefficient of variation of the material factor, V_M, is recommended to be 0.08 for failure limited by anchor or *connection* failure, and 0.10 for limits caused by flexural or other modes of failure. *Specification* Section I6.3.1 also eliminates the limit on coefficient of variation of the test results, V_p, because consistent test results often lead to V_p values lower than the 6.5 percent value set in *Specification* Section K2.1. The elimination of the limit will be beneficial when test results are consistent.

The value for the number of tests for fasteners is set as the number of anchors tested with the same tributary area as the anchor that failed. This is consistent with design practice where anchors are checked using a *load* calculated based on tributary area. Actual anchor *loads* are not calculated from a stiffness analysis of the panel in ordinary design practice.

Commentary for *load* combinations including wind uplift is provided in Appendix A.

A

I6.4 Roof System Bracing and Anchorage

I6.4.1 Anchorage of Bracing for Purlin Roof Systems Under Gravity Load With Top Flange Connected to Metal Sheathing

In metal roof systems utilizing C- or Z-*purlins*, the application of gravity *loads* will cause torsion in the *purlin* and lateral displacements of the roof system. These effects are due to the slope of the roof, the loading of the member eccentric to its shear center, and for Z-*purlins*, the inclination of the principal axes. The torsional effects are not accounted for in the design provisions of Chapter F, Sections I6.1 and I6.2; and lateral displacements may create *instability* in the system. Lateral restraint is typically provided by the roof sheathing and lateral anchorage devices to minimize the lateral movement and the torsional effects. The anchorage devices are designed to resist the lateral anchorage force and provide the appropriate level of stiffness to ensure the overall *stability* of the *purlins*.

The calculation procedure in *Specification* Equations I6.4.1-1 through I6.4.1-6 determines the anchorage force by first calculating an upper bound force for each *purlin*, P_i, at the line of anchorage. This upper bound force is then distributed to anchorage devices and reduced due to the system stiffness based on the relative effective stiffness of each component. For the calculation procedure, the anchorage devices are modeled as linear springs located at the top of the *purlin web*. The stiffness of anchorage devices that do not attach at this location must be adjusted, through analysis or testing, to an equivalent lateral stiffness at the top of the *web*. This adjustment must include the influence of the attached *purlin* but not include any reduction due to the flexibility of the sheathing to *purlin connection*. *Specification* Equation I6.4.1-4 establishes an effective lateral stiffness for each anchorage device, relative to each *purlin*, that has been adjusted for the flexibility of the roof system between the *purlin* location and the anchorage location. It is important to note that the units of A_p are area per unit width. Therefore, the bay length, L, in this equation must have units consistent with the unit width used for establishing A_p. The resulting product, LA_p, has units of area. The total effective stiffness for a given *purlin* is then calculated with *Specification* Equation I6.4.1-5 by summarizing the effective stiffness relative to each anchorage device and the system stiffness from *Specification* Equation I6.4.1-6. The force generated by an individual *purlin* is calculated by Equation I6.4.1-2, and then distributed to an anchorage device based on the relative stiffness ratio in *Specification* Equation I6.4.1-1.

Lateral bracing forces will accumulate within the roof sheathing and must be transferred into the anchorage devices. The strength of the elements in this *load* path must be verified. AISI S912, *Test Procedures for Determining a Strength Value for a Roof Panel-to-Purlin-to-Anchorage Device Connection*, provides a means to determine a lower bound strength for the complete *load* path. For through-fastened roof systems, this strength value can be reasonably estimated by rational analysis by assuming that the roof fasteners within 12 inches (305 mm) of the anchorage device participate in the force transfer.

The 1986 through 2001 *Specifications* included brace force equations that were based on the work by Murray and Elhouar (1985) with various extensions from subsequent work. The original work assumed the applied loading was parallel to the *purlin webs*. The later addition of the "cosθ" and "sinθ" terms attempted to account for the roof slope, but it failed to correctly model the system effect for higher-sloped roofs. Tests by Lee and Murray (2001) and Seek and Murray (2004) showed generally that the brace force equations conservatively predicted the lateral anchorage forces at slopes less than 1:12, but predicted unconservative lateral anchorage forces at steeper slopes. The new procedure outlined in *Specification* Section I6.4.1 was formulated to correlate better with test results. Also, the original work was based on the application of one anchorage device to a group of *purlins*. Until the work of Sears and Murray (2007), a generally accepted manual technique to extend this procedure to roofs with multiple anchors was not available.

Prior to the work by Seek and Murray (2006, 2007) and Sears and Murray (2007), the anchorage devices were assumed to have a constant and relatively high lateral stiffness. The current provisions recognize the finite stiffness of the anchorage device, and the corresponding decrease in anchorage forces for more flexible anchorage devices. *Specification* Equation I6.4.1-7 establishes a minimum effective stiffness that must be provided to limit the lateral displacement at the anchorage device to d/20. This required stiffness does not represent the required stiffness of each anchorage device, but instead the total stiffness provided by the stiffness of the *purlin* system (K_{sys}) and the anchorage devices relative to the most remote *purlin*.

Several alternative rational analysis methods have been developed to predict lateral anchorage forces for Z-section roof systems. A method for calculating lateral anchorage forces is presented by Seek and Murray (2006, 2007). The method is similar to the procedure outlined in *Specification* Section I6.4.1 but uses a more complex method derived from mechanics to determine the lateral force introduced into the system at each Z-section, P_i, and distributes the force to the components of the system according to the relative lateral stiffness of each of the components. The method is more computationally intensive, but allows for analysis of more complex bracing configurations such as supports plus third points lateral anchorage and supports plus third points torsional braces.

A method to predict lateral anchorage forces using the finite element method is presented in Seek and Murray (2004). The model uses shell finite elements to model the Z-sections and sheathing in the roof system. The model accurately represents Z-section behavior and is capable of handling configurations other than lateral anchorage applied at the top *flange*. However, the computational complexity limits the size of the roof system that can be modeled by this method.

Rational analysis may also be performed using the elastic stiffness model developed by Sears and Murray (2007) upon which the provisions of *Specification* Section I6.4.1 are based. The model uses frame finite elements to represent the Z-sections and a truss system to represent the *diaphragm*. The model is computationally efficient, allowing for analysis of large systems.

Anchorage is most commonly applied along the frame lines due to the effectiveness and ease in which the forces are transferred out of the system. In the absence of substantial *diaphragm* stiffness, anchorage may be required along the interior of the span to prevent large lateral displacements. Torsional braces applied along the span of a Z- or C-section provide an alternative to interior anchorage.

I6.4.2 Alternative Lateral and Stability Bracing for Purlin Roof Systems

Tests (Shadravan and Ramseyer, 2007) have shown that C- and Z-sections can reach the capacity determined by *Specification* Chapter F through the application of torsional braces along the span of the member. Torsional braces applied between pairs of *purlins* prevent twist of the section at a discrete location. The moments developed due to the torsional brace can be resolved by forces in the plane of the *web* of each section and do not require external anchorage at the location of the brace. The vertical forces should, however, be accounted for when determining the applied *load* on the section.

Torsional braces should be applied at or near each *flange* of the Z- or C-section to prevent deformation of the *web* of the section and ensure the effectiveness of the brace. When twist of the section is thus prevented, a section may deflect laterally and retain its strength. Second-order moments can be resisted by the rotational restraints. Therefore, a more liberal lateral deflection of L/180 between the supports is permitted for a C- or Z- section with torsional braces. Anchorage is required at the frame line to prevent excessive deformation at the support location that undermines the strength of the section. A lateral displacement limit, therefore, is imposed along the frame lines to ensure that adequate restraint is provided.

I7 Storage Rack Systems

Steel storage rack systems are designed and constructed in accordance with ANSI MH16.1 and ANSI MH16.3.

In 2020, because they are independently adopted by the *applicable building code*, references to MH16.1 and MH16.3 were deemed unnecessary and removed from this section of the *Specification*. They both adopt AISI S100 as appropriate for the design of steel storage rack systems.

J. CONNECTIONS AND JOINTS

J1 General Provisions

Welds, bolts, screws, rivets, and other special devices such as metal stitching and adhesives are generally used for cold-formed steel *connections* (Brockenbrough, 1995). The 2016 edition of the *Specification* contains provisions in Chapter J for welded *connections*, bolted *connections*, screw *connections*, and *power–actuated fastener* (*PAF*) *connections*. Among these commonly used types of *connections*, the design provisions for using screws were developed in 1993 and were included in the 1996 *Specification* for the first time, and the design provisions for *power-actuated fasteners* were added in the 2012 *Specification*.

The *available connection strength* [*factored resistance*] of members or components is the result of several design considerations that limit the strength. Section J2 provides the strength of a weldment, this would be the strength of the weld filler metal or the interface strength of the weld filler metal to the connected components. Sections J3, J4, and J5 provide the dowel strength, the shear, or tension of the bolt, screw or *PAF*, and the bearing strength of the steel against the connector. Section J6 provides the provisions for rupture, including shear lag of the connected members through the connector or weldment pattern, or to the end or edge of the connected member. The minimum edge distance and minimum spacing for connectors or weldments for constructability does not ensure that the effect of rupture will not limit the strength of the connected members. The geometry of the *connection* will influence any eccentricity that will affect the combination of shear and tension forces applied on the bolt, screw, *PAF*, or weldment. The governing *available strength* [*factored resistance*] of the *connection* is the result of all applicable *limit states*.

The following brief discussions deal with the application of rivets and other special devices:

(a) Rivets

While hot rivets have little application in cold-formed steel construction, cold rivets find considerable use, particularly in special forms such as blind rivets (for application from one side only), tubular rivets (to increase bearing area), high shear rivets, and explosive rivets. For the design of connections using cold rivets, the provisions for bolted connections may be used as a general guide, except that the shear strength of rivets may be quite different from that of bolts. Additional design information on the strength of rivets should be obtained from manufacturers or from tests.

(b) Special Devices

Special devices include: (1) metal stitching, achieved by tools that are special developments of the common office stapler, and (2) connecting by means of special clinching tools that draw the sheets into interlocking projections.

Most of these *connections* are proprietary devices for which information on strength of *connections* must be obtained from manufacturers or from tests carried out by or for the user. Guidelines provided in *Specification* Section K2 are to be used in these tests.

The plans or specifications are to contain information and design requirement data for the adequate detailing of each *connection* if the *connection* is not detailed on the engineering design drawings.

In the 2001 edition of the *Specification*, the *ASD*, *LRFD* and *LSD* design provisions for welded and bolted *connections* were based on the 1996 edition of the *Specification*, with some revisions and additions which will be discussed in subsequent sections. Most of those design provisions were kept in this edition of the *Specification*. Some content reorganization was made in 2010, where shear rupture check for welds and fasteners was moved to Section J6.

J2 Welded Connections

For welded cold-formed steel *connections*, the weldability of the steels (ASTM, 2017) should be considered and a welding procedure suitable for the steels used is to be utilized.

Welds used for cold-formed steel construction may be classified as fusion welds (or arc welds) and resistance welds. Fusion welding is used for connecting cold-formed steel members to each other as well as connecting such members to heavy, hot-rolled steel framing (such as floor panels to beams of the steel frame). It is used in groove welds, arc spot welds, arc seam welds, fillet welds, and flare-groove welds.

The design provisions contained in this *Specification* section for fusion welds have been based primarily on experimental evidence obtained from an extensive test program conducted at Cornell University. The results of this program are reported by Peköz and McGuire (1979) and summarized by Yu and LaBoube (2010). In addition, the Cornell research provided the experimental basis for the AWS *Structural Welding Code for Sheet Steel* (AWS, 1998). In most cases, the provisions of the AWS code are in agreement with this *Specification* section. All possible failure modes are covered in the *Specification* since 1996, whereas the earlier *Specifications* mainly dealt with shear failure.

For most of the *connection* tests reported by Peköz and McGuire (1979), the onset of yielding was either poorly defined or followed closely by failure. Therefore, in the provisions of this section, rupture rather than yielding is used as a more reliable criterion of failure.

The welded *connection* tests, which served as the basis of the provisions given in *Specification* Sections J2.1 through J2.8, were conducted on sections with single and double sheets (see *Specification* Figures J2.2-1 and J2.2-2). The largest total sheet *thickness* of the cover plates was approximately 0.15 inch (3.81 mm). However, within this *Specification*, the validity of the equations was extended to welded *connections* in which the *thickness* of the thinnest connected part is 3/16 inch (4.76 mm) or less. For arc spot welds, the maximum *thickness* of a single sheet (*Specification* Figure J2.2.2.1-1) and the combined *thickness* of double sheets (*Specification* Figure J2.2.2.1-2) are set at 0.15 inch (3.81 mm). In 2022, this upper limit was extended from 0.15 in. (3.81 mm) to 0.19 in. (4.83 mm).

The upper limit of the *Specification* applicability was revised in 2004 from 0.18 in. (4.57 mm) to 3/16 in. (4.76 mm). This change was made to be consistent with the limit given in AWS D1.3 (1998). In 2022, this limit was changed to 0.19 in. (4.83 mm), to restore the limit to a decimal value that is closest to the AWS D1.3 limit of 3/16 in. (4.76 mm).

In 2001, the *safety factors* and *resistance factors* in this section were modified for consistency based on the research work by Tangorra, Schuster, and LaBoube (2001). In 2022, the *safety* and *resistance factors* for arc spot welds and arc plug welds were modified based on work by Blackburn, Sputo, and Meyer (2016).

For design tables and example problems on welded *connections*, see Part IV of the *Cold-Formed Steel Design Manual* (AISI, 2017).

J2.1 Groove Welds in Butt Joints

The design equations for determining *nominal strength* [*resistance*] for groove welds in butt *joints* have been taken from the AISC *LRFD* Specification (AISC, 1993). Therefore, the AISC definition for the effective throat thickness, t_e, is equally applicable to this section of the *Specification*. Prequalified *joint* details are given in AWS D1.3-98 (AWS, 1998) or other equivalent weld standards.

In 2010, *Specification* Section J2.1(a) was revised to delete the case for tension or compression parallel to the axis of the weld, so that *Specification* Equation J2.1-1 is applicable only to tension or compression normal to the *effective area* of the weld. For tension or compression parallel to the weld axis, the computation of the weld strength is not required (AISC, 2005 and 2010a).

J2.2 Arc Spot Welds

Arc spot welds (commonly referred to as "puddle welds") used for connecting thin sheets are made by burning through the top sheet(s) and then filling with weld metal to fuse the layer(s) to the bottom sheet or supporting member. No prepunched holes are required for arc spot welds.

AWS D1.3 does not consider arc spot welds as prequalified welds. Therefore, all arc spot welding must be qualified for the particular application. A Welding Procedure Specification (WPS) and Procedure Qualification Record (PQR) must be developed and followed for the weld process. The WPS provides in detail the required welding variables for the specific application to assure repeatability by properly trained welders. The welding variables would include, but are not limited to, thickness of the welded sheets and the support being welded to, electrodes, machine settings, and arc time for a given weld size. The PQR is a record of the welding variables used to produce an acceptable test weldment and the results of tests conducted on the weldment to qualify a WPS by an individual welder.

In Canada, all arc spot welds should be qualified in accordance with CSA W47.1. A Welding Procedure Data Sheet (WPDS) and a Procedure Qualification Record (PQR) must be developed and followed for each welding process. The PQR should list all essential variables recorded during the testing process. The WPDS can list a range of essential variables as permitted by CSA W47.1.

The titles for Figures J2.2.1-2 and J2.2.2.1-2 were changed from double thicknesses to multiple thicknesses to reflect the intent of the *Specification* to not limit the number of welded sheets to two sheets. In 2022, AWS made a parallel change to the AWS D1.3 Standard.

J2.2.1 Minimum Edge and End Distance

In the 2001 and 2007 editions of the *Specification*, the distance measured in the line of force from the centerline of weld to the nearest edge of an adjacent weld or to the end of the connected part toward which the force is directed was required to not be less than e_{min}, which is equal to *required strength* [forces due to *factored loads*] divided by (tF_u). In 2010, an equivalent resistance was determined by the use of Section J6.1.

In 2024, the *Specification* was updated to specifically require that rupture be checked in addition to minimum end or edge distance. The minimum edge distance was never intended to prevent rupture. Therefore, this change requires users to check both the minimum edge or end distance and for rupture.

J2.2.2 Shear

J2.2.2.1 Shear Strength for Sheet(s) Welded to a Thicker Supporting Member

The Cornell tests (Peköz and McGuire, 1979) identified four modes of failure for arc spot welds, which are addressed in this *Specification* section. They are: (1) shear failure of welds in the fused area, (2) tearing of the sheet along the contour of the weld with the tearing spreading the sheet at the leading edge of the weld, (3) sheet tearing combined with *buckling* near the trailing edge of the weld, and (4) shearing of the sheet behind the weld. It should be noted that many failures, particularly those of the plate tearing type, may be preceded or accompanied by considerable inelastic out-of-plane deformation of the type indicated in Figure C-J2.2.2.1-1. This form of behavior is similar to that observed in wide, pin-connected plates. Such behavior should be avoided by closer spacing of welds. When arc spot welds are used to connect two sheets to a framing member as shown in *Specification* Figure J2.2.2.1-2, consideration should also be given to possible shear failure between thin sheets.

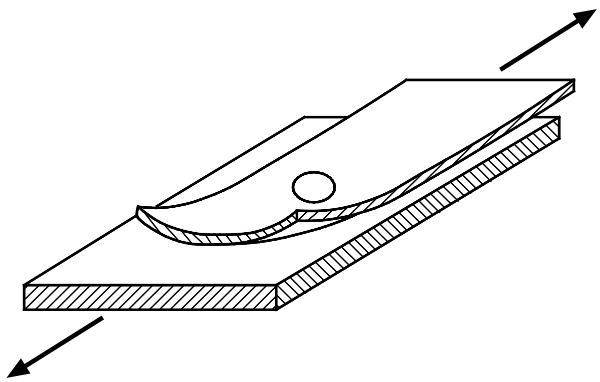


Figure C-J2.2.2.1-1 Out-of-Plane Distortion of Welded Connection

Prior to 2022, the *thickness* limitation of 0.15 inch (3.81 mm) was due to the range of the test program that served as the basis of these provisions. An evaluation of additional testing (Blackbum, Sputo and Meyer, 2016) extended this *thickness* limitation to 0.19 in. (4.83 mm). On sheets below 0.028-inch (0.711-mm) thick, weld washers are required to avoid excessive burning of the sheets and, therefore, inferior quality welds.

Specification Equation J2.2.2.1-1 shows that the *nominal shear strength* [*resistance*] of arc spot welds is proportional to the square of effective diameter, d_e, of fused area at plane of maximum shear transfer. Since d_e is a function of sheet *thickness* in accordance with *Specification* Equation J2.2.2.1-5, a larger visible diameter, d, may be needed to maintain the same weld strength if the welded sheet *thickness*, t, is increased. The equation for d_e was revised in 2022 (Blackburn, Sputo and Meyer, 2016).

The *safety* and *resistance factors* were revised in 2022 based on a review and recalibration of new and existing test data (Blackburn, Sputo and Meyer, 2016).

J2.2.2.2 Shear Strength for Sheet-to-Sheet Connections

The Steel Deck Institute (SDI) *Diaphragm Design Manual* (SDI, 1987 and 2004) stipulates that the shear strength for a sheet-to-sheet arc spot weld *connection* be taken as 75 percent of the strength of a sheet-to-structural *connection*. SDI further stipulates that the sheet-to-structural *connection* strength be defined by *Specification* Equation J2.2.2.1-2. This design provision was adopted by the *Specification* in 2004. Prior to accepting the SDI design recommendation, a review of the pertinent research by Luttrell (SDI, 1987) was performed by LaBoube (2001). *Safety* and *resistance factors* were revised in 2022 to match the corresponding factors for sheet to support welds, based on a review and re-evaluation of additional test data (Blackburn, Sputo, and Meyer, 2016). The tested sheet *thickness* range that is reflected in the *Specification* documents is based on the scope of Luttrell's test program. SDI suggests that sheet-to-sheet welds are problematic for *thicknesses* of less than 0.0295 in. (0.75 mm). Such welds result in "blowholes," but the perimeter must be fused to be effective.

Quality control for sheet–to-sheet *connections* is not within the purview of AWS D1.3. However, using AWS D1.3 as a guide, the following quality control/assurance guidelines are suggested:

(1) Measure the visible diameter of the weld face,

(2) Ensure no cracks in the welds,

(3) Maximum undercut = 1/8 of the weld circumference, and

(4) Sheets are to be in contact with each other.

J2.2.3 Tension

For tensile capacity of arc spot welds, the design provisions in the 1989 *Specification* Addendum were based on the tests reported by Fung (1978) and the study made by Albrecht (1988). Those provisions were limited to sheet failure with restrictive limitations on material properties and sheet *thickness*. These design criteria were revised in 1996 because the tests conducted at the University of Missouri-Rolla (LaBoube and Yu, 1991 and 1993) have shown that two potential *limit states* may occur. The most common failure mode is that of sheet tearing around the perimeter of the weld. This failure condition was found to be influenced by the sheet *thickness*, the average weld diameter, and the material *tensile strength*. In some cases, it was found that tensile failure of the weld can occur. The strength of the weld was determined to be a function of the cross-section of the fused area and *tensile strength* of the weld material. Based on analysis by LaBoube (2001), the *nominal strength* [*resistance*] equation was changed in 2001 to reflect the ductility of the sheet, F_u/F_y, and the sheet *thickness*, the average weld diameter, and the material *tensile strength*.

The multiple *safety factors* and *resistance factors* recognize the behavior of a panel system with many *connections* versus the behavior of a member *connection* and the potential for a catastrophic failure in each application. In *Specification* Section J2.2.3, a target reliability index of 3.0 for the United States and Mexico and 3.5 for Canada is used for the panel connection limit, whereas a target reliability index of 3.5 for the United States and Mexico and 4 for Canada is used for the other *connection* limit. Precedence for the use of a smaller target reliability index for systems was established in Section I6.3.1 of the *Specification*.

Tests (LaBoube and Yu, 1991 and 1993) have also shown that when reinforced by a weld washer, thin sheet weld *connections* can achieve the *nominal strength* [*resistance*] given by *Specification* Equation J2.2.3-2 using the *thickness* of the thinner sheet.

The equations given in the *Specification* were derived from the tests for which the applied tension *load* imposed a concentric *load* on the weld, as would be the case, for example, for the interior welds on a roof system subjected to wind uplift. Welds on the perimeter of a roof or floor system would experience an eccentric tensile loading due to wind uplift. Tests have shown that as much as a 50 percent reduction in *nominal connection strength* [*resistance*] could occur because of the eccentric *load* application (LaBoube and Yu, 1991 and 1993). Eccentric conditions may also occur at *connection* laps as depicted by Figure C-J2.2.3-1.

At a lap *connection* between two deck sections as shown in Figure C-J2.2.3-1, the length of the unstiffened *flange* and the extent of the encroachment of the weld into the unstiffened *flange* have a measurable influence on the strength of the welded *connection* (LaBoube and Yu, 1991). The *Specification* recognizes the reduced capacity of this *connection* detail by imposing a 30 percent reduction on the calculated *nominal strength* [*resistance*].

Safety and *resistance factors* were revised, and a weld effectiveness reduction factor to account for eccentric loading was added in 2022, based on a review and re-evaluation of additional test data (Blackburn, Sputo, and Meyer, 2016).

In 2022, the requirement that F_{xx} must be greater than F_u was deleted because the requirement for matching weld metal was removed from AWS D1.3 in 2017, and the *nominal strength* [*resistance*] equations consider both the ultimate strength of the sheet and the electrode.

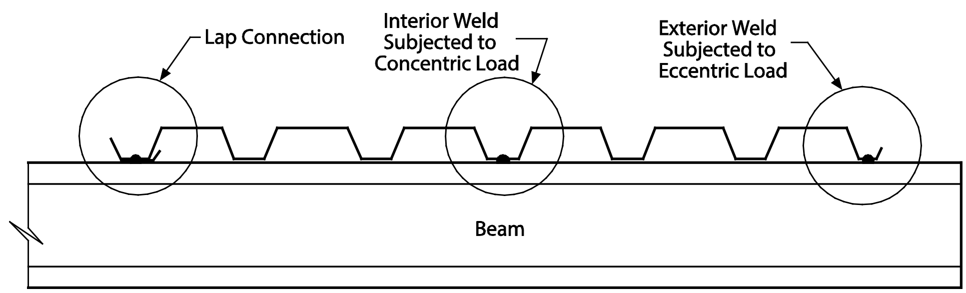


Figure C-J2.2.3-1 Interior Weld, Exterior Weld and Lap Connection

J2.2.4 Combined Shear and Tension on an Arc Spot Weld

The Steel Deck Institute *Diaphragm Design Manual* (2004) provides a design equation for evaluating the strength of an arc spot weld *connection* subject to combined shear and tension forces. An experimental investigation was conducted at the University of Missouri–Rolla to study the behavior and to develop design recommendations for the relationship (interaction) of the tension and shear forces on an arc spot weld *connection* (Stirnemann and LaBoube, 2007).

The experimental study focused on six variables that were deemed to be the key parameters that could influence the strength of the arc spot weld *connection*. These variables were the sheet *thickness*; sheet material properties including *yield stress*, *tensile strength* and ductility of the sheet; visible diameter of the arc spot weld; and the relationship between the magnitude of the shear force and tension force. Based on an analysis of the test results, the Steel Deck Institute's interaction equation was found to provide an acceptable estimate of the strength of the arc spot weld *connection*.

J2.3 Arc Seam Welds

The general behavior of arc seam welds is similar to that of arc spot welds. In 2010, Section

J2.3 was reorganized to be consistent with provisions provided for arc spot welds.

J2.3.1 Minimum Edge and End Distance

In 2024, the *Specification* was updated to specifically require that rupture be checked in addition to minimum end or edge distance. The minimum edge distance was never intended to prevent rupture. Therefore, this change requires users to check both the minimum edge or end distance and for rupture.

J2.3.2 Shear

J2.3.2.1 Shear Strength for Sheet(s) Welded to a Thicker Supporting Member

No simple shear failures of arc seam welds were observed in the Cornell tests (Peköz and McGuire, 1979). Therefore, *Specification* Equation J2.3.2.1-1, which accounts for shear failure of welds, is adopted from the AWS welding provisions for sheet steel (AWS, 1998).

Specification Equation J2.3.2.1-2 is intended to prevent failure through a combination of tensile tearing plus shearing of the cover plates.

Safety and *resistance factors* were revised in 2022 to match the corresponding factors for arc spot welds (Blackburn, Sputo, and Meyer, 2016).

J2.3.2.2 Shear Strength for Sheet-to-Sheet Connections

In 2010, the provisions for determining the shear strength of sheet-to-sheet arc spot weld *connections* were adopted for arc seam weld *connections*. This is conservative because the length of the seam weld is not considered. *Safety* and *resistance factors* were revised in 2022 to match the corresponding factors for arc spot welds (Blackburn, Sputo, and Meyer, 2016).

J2.4 Top Arc Seam Sidelap Welds

Top arc seam sidelap welds (often referred to as TSWs) have commonly been used to attach the edges of standing seam steel roof and floor deck panels, particularly those used for *diaphragms*. The *top arc seam sidelap weld connection* is formed by a vertical sheet leg (edge stiffener of deck) inside an overlapping sheet hem, or by two vertical sheet legs back-to-back. *Top arc seam welds* have been referenced in some historical *diaphragm* design standards as part of a system without defining the strength of individual *connections*. Similarly, AWS D1.3 has shown the weld as a possible variation of an arc seam weld, without clear provisions to determine weld strength. The research to develop the design provisions for the *top arc seam welds* is presented in the S. B. Barnes Associates (Nunna and Pinkham, 2012; Nunna, et al., 2012) report.

J2.4.1 Shear Strength of Top Arc Seam Sidelap Welds

The design limitations are due to the scope of the test program that served as the basis for these provisions. The tests included typical weld spacing of approximately 12 in. (305 mm) o.c. and this established the strength of the welds with the stated limits. All testing was performed on *joints* with a vertical sheet leg inside an overlapping sheet hem configuration, but the behavior of *connections* with back-to-back vertical sheet legs is assumed to be similar.

Testing was performed in general accordance with AISI S905 (AISI, 2008), with the specimen dimensions in S905 Table 2 modified as required to address the described deck edge configuration. The ductility of the tested steels ranged from F_u/F_{sy} = 1.01 to F_u/F_{sy} = 1.52. The limits were extended to permit the use of the full range of recognized steels. Application should be based on the specified F_u/F_{sy} for steels recognized in Section A3 of the *Specification*. The exclusion of the *connection* design restrictions for *top arc seam welds* used in *diaphragms* considers that the shear in the side lap welds is flowing from the sheet into each weld such that each weld is loaded as if it were a singular weld by its tributary length. This mitigates the concern over *load* sharing in brittle *connections*, and the strength reduction of lower ductility steels is based on the tests and built into *Specification* Equation J2.4.1-1.

The impact of shear rupture in the sheet can be calculated based on *Specification* Section J6 and this can be used to determine minimum acceptable weld spacing. The distance from the centerline of any weld and the centerline of adjacent weld can be checked by using Equation C-J2.4.1-1. Equation C-J2.4.1-1 is derived by equating the *nominal shear strength* [*resistance*] expression from *Specification* Section J6 (*Eq*. J6.1-1 with A_{nv} = st) to the *nominal shear strength* [*resistance*] expression from *Specification* Section J2.4.1.

$$s = [6.67(F_u/F_{sy})-2.53]L_w(t/L_w)^{0.33} \tag{C-J2.4.1-1}$$

where

s = Minimum distance from centerline of any weld to centerline of adjacent weld

$s/2$ = Minimum distance from centerline of weld to end of connected member

L_w = Specified weld length

t = Base steel *thickness* (exclusive of coatings) of the thinner connected sheet

F_u = Minimum *tensile strength* of connected sheets as determined in accordance with *Specification* Section A3.1.1, A3.1.2 or A3.1.3

F_{sy} = *Minimum specified yield stress* of connected sheets as determined in accordance with *Specification* Section A3.1.1, A3.1.2 or A3.1.3

The steel deck sheets at the sidelap need to be tightly interlocked by crimping or pinching the sidelap prior to welding. When using the *joint* variation shown in *Specification* Figure J2.4.1-1(b), contact must be maintained between the two vertical legs while welding. For sidelaps with overlapping hem, *Specification* Figure J2.4.1-1(a) illustrates a crimped area nominally longer than the length of fusion, and the top of the overlapping hem sidelap must be burned through to allow fusion with the top of the inner vertical leg. Holes are commonly present at either or both ends of the completed welds. The holes do not necessarily indicate deficient welds or poor workmanship provided the specified length of fusion is obtained. Holes may aid in determining proper fusion with the inner vertical leg.

J2.5 Arc Plug Welds

Arc plug welds are used for connecting relatively thicker sheets as compared to arc spot welds. For arc plug welds, it is required to make holes in the top sheet(s), and then fill with weld metal to fuse the top layer(s) to the bottom sheet or a framing member. The previous *Commentary* Section J2.2 permitted arc plug welds to be designed using the arc spot weld provisions. In 2022, the arc plug weld design was added into the *Specification*.

J2.6 Fillet Welds

For fillet welds on the lap *joint* specimens tested in the Cornell research (Peköz and McGuire, 1979), the dimension, w_1, of the leg on the sheet edge generally was equal to the sheet *thickness*; the other leg, w_2, often was two or three times longer than w_1 (see *Specification* Figure J2.6-1). In *connections* of this type, the fillet weld throat is commonly larger than the throat of conventional fillet welds of the same size. Usually, ultimate failure of fillet-welded *joints* has been found to occur by the tearing of the plate adjacent to the weld (see Figure C-J2.6-1).

In most cases, the higher strength of the weld material prevents weld shear failure; therefore, the provisions of this *Specification* section are based on sheet tearing. Because specimens up to 0.15 inch (3.81 mm) *thickness* were tested in the Cornell research (Peköz and McGuire, 1979), the last provision in this section covers the possibility that for sections thicker than 0.15 inch (3.81 mm), the throat dimension may be less than the *thickness* of the cover plate and the tear may occur in the weld rather than in the plate material. Additional research at the University of Sydney (Zhao and Hancock, 1995) has further indicated that weld throat failure may even occur between the *thicknesses* of 0.10 in. (2.54 mm) to 0.15 in. (3.81 mm). Accordingly, the *Specification* was revised in 2001 to require weld strength check when the plate *thickness* is greater than 0.10 in. (2.54 mm). For high-strength materials with *yield stress* of 65 ksi (448 MPa) or higher, research at the University of Sydney (Teh and Hancock, 2000) has shown that weld throat failure does not occur in materials less than 0.10-in. (2.54-mm) thick and that the *Specification* provisions based on sheet strength are satisfactory for high-strength material less than 0.10-in. (2.54-mm) thick. Prequalified fillet welds are given in AWS D1.3-98 (AWS, 1998) or other equivalent weld standards.

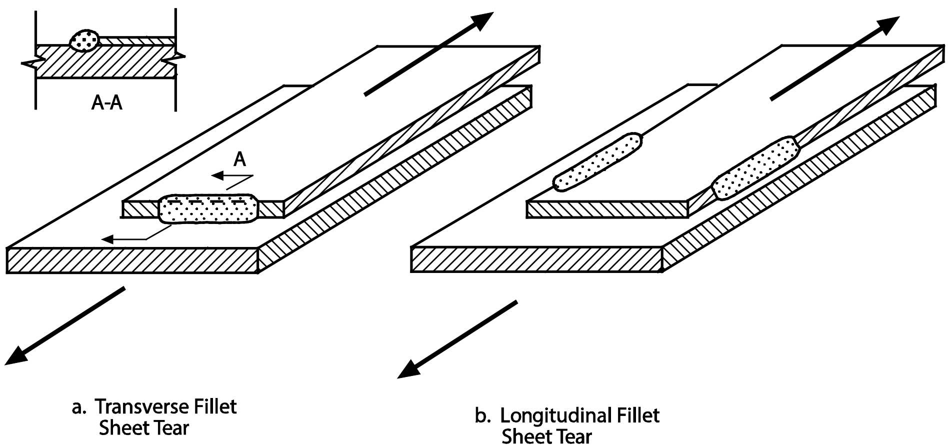


Figure C-J2.6-1 Fillet Weld Failure Modes

In 2012, the design provisions were modified to take into consideration that the connected parts may have different *tensile strengths*.

J2.7 Flare Groove Welds

The primary mode of failure in cold-formed steel sections welded by flare groove welds, loaded transversely or longitudinally, was found to be sheet tearing along the contour of the weld (see Figure C-J2.7-1).

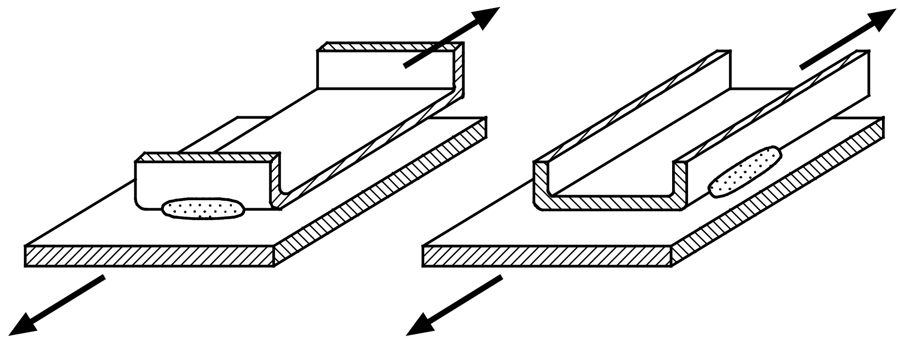


Figure C-J2.7-1 Flare Groove Weld Failure Modes

Except for *Specification* Equation J2.7-4, the provisions of this *Specification* section are intended to prevent shear tear failure. *Specification* Equation J2.7-4 covers the possibility that thicker sections may have effective throats less than the *thickness* of the channel and weld failure may become critical.

In 2001, the *Specification* was revised to require that weld strength be checked when the plate *thickness* is greater than 0.10 in. (2.54 mm) based on the research by Zhao and Hancock (1995).

In 2010, two figures were added showing reference dimensions for flare-bevel groove welds and flare V-groove welds, respectively, which replaced the figures for these welds in the previous editions of the *Specification*. *Specification* Equations J2.7-5 and J2.7-7 were added to more accurately define the effective throat of these welds. Filled flush throat depths were modified to match those specified in AWS D1.1-2006 Section 2.3.1.4 and Table 2.1. Welding process designations in *Specification* Tables J2.7-1 and J2.7-2 were based on AWS D1.1 Annex K, where SMAW stands for "shielded metal arc welding," FCAW-S stands for "flux cored arc welding-self shielded," GMAW stands for "gas metal arc welding," FCAW-G stands for "flux cored arc welding-gas shielded," and SAW stands for "submerged arc welding." No change was needed in the *Specification* requirements from previous editions except in the definitions of the effective throat for use in *Specification* Equation J2.7-4.

In 2021 a limit on w_1 was placed on *Specification* Equation J2.7-5 to clarify a limit consistent with the original derivation of the equation. In addition, $w_2 = 0$ was specified to clarify the intent of the filled flush weld condition.

Equation J2.7-5 in the *Specification* provides the effective throat, t_w, for a flare-bevel groove weld when the leg dimensions, w_1 and w_2, of the finished weld are provided. To solve for the face dimensions when a given effective throat, t_w, is specified or determined, the following equation (C-J2.7-1) may be solved. The desired face angle, α, of the weld must be specified to reduce the solution to a single result. Solve the following equation for cos (β):

$$\begin{aligned}&\cos^2(\beta)(1+\tan^2(\alpha))+\cos(\beta)\{2\tan(\alpha)[\sin(\theta)-\cos(\theta)\tan(\alpha)+t_w/(R\cos(\alpha))]\}\\&+[\sin(\theta)-\cos(\theta)\tan(\alpha)+t_w/(R\cos(\alpha))]^2-1=0\end{aligned} \tag{C-J2.7-1}$$

For the above quadratic equation solution:

$$a=1+\tan^2(\alpha) \tag{C-J2.7-2}$$

$$b=2\tan(\alpha)[\sin(\theta)-\cos(\theta)\tan(\alpha)+t_w/(R\cos(\alpha))] \tag{C-J2.7-3}$$

$$c = [\sin(\theta) - \cos(\theta)\tan(\alpha) + t_w / (R\cos(\alpha))]^2 - 1 \quad \text{(C-J2.7-4)}$$

$$\beta = \cos^{-1}\left(\frac{-b \pm \sqrt{b^2 - 4ac}}{2a}\right) \quad \text{(C-J2.7-5)}$$

$$w_1 = R(1 - \cos(\beta)) \quad \text{(C-J2.7-6)}$$

$$w_2 = w_1 \tan(\alpha) \quad \text{(C-J2.7-7)}$$

$$w_f = w_1 / \cos(\alpha) = \sqrt{w_1^2 + w_2^2} \quad \text{(C-J2.7-8)}$$

where

θ = Angle from the center of the radius of the curved surface to the point of effective throat depth
= 43.433 degrees (SMAW, FCAW-S, SAW weld types)
= 22.024 degrees (GMAW, FCAW-G weld types)

α = Angle of the weld face

β = Angle from the center of the radius of the curved surface to the point of contact of the weld face to the curved surface

t_w = Effective weld throat

Angles α, θ and β are measured perpendicular to the flat surface (See Figure C-J2.7-2 for more information.)

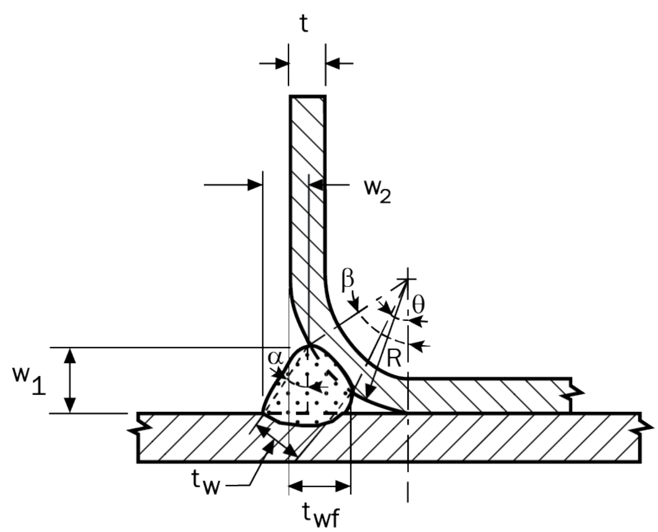


Figure C-J2.7-2 Illustration of Weld Geometry

Alternatively, basing the weld strength calculation only on the leg dimensions (w_1, w_2) of the weld is conservative. This simplifying assumption is particularly relevant when the radius of the curved surface is very small.

J2.8 Resistance Welds

The shear values for outside sheets of 0.125 inch (3.18 mm) or less in *thickness* are based on "Recommended Practice for Resistance Welding Coated Low-Carbon Steels," AWS C1.3-70 (Table 2.1 - Spot Welding Galvanized Low-Carbon Steel). Shear values for outside sheets thicker than 0.125 inch (3.18 mm) are based upon "Recommended Practices for Resistance

Welding," AWS C1.1-66 (Table 1.3 - Pulsation Welding Low-Carbon Steel) and apply to pulsation welding as well as spot welding. They are applicable for all structural grades of low-carbon steel, uncoated or galvanized with 0.90 oz/ft^2 (275 g/m^2) of sheet or less, and are based on values selected from AWS C1.3-70 (Table 2.1), and AWS C1.1-66 (Table 1.3). These values may also be applied to medium carbon and low-alloy steels. Spot welds in such steels give somewhat higher shear strengths than those upon which these values are based; however, they may require special welding conditions. In view of the fact that AWS C1.1-66 and AWS C1.3-70 Standards were incorporated in AWS C1.1-2000, resistance welds should be performed in accordance with AWS C1.1-2000 (AWS, 2000).

In the 2001 edition of the *Specification*, a design equation is used to determine the *nominal shear strength* [*resistance*] that replaces the tabulated values given in previous editions of the *Specification*. The upper limit of *Specification* Equations J2.8-1, J2.8-3 and J2.8-5 is selected to best fit the data provided in AWS C1.3-70, Table 2.1 and AWS C1.1-66, Table 1.3. Shear strength values for welds with the *thickness* of the thinnest outside sheet greater than 0.180 in. (4.57 mm) have been excluded in *Specification* Equations J2.8-2, J2.8-4 and J2.8-6 due to the *thickness* limit set forth in *Specification* Section J2.

J3 Bolted Connections

The structural behavior of bolted *connections* in cold-formed steel construction is somewhat different from that in hot-rolled heavy construction, mainly because of the thinness of the connected parts. Prior to 1980, the provisions included in the *Specification* for the design of bolted *connections* were developed on the basis of the Cornell tests (Winter, 1956a, 1956b). These provisions were updated in 1980 to reflect the results of additional research performed in the United States (Yu, 1982) and to provide better coordination with the specifications of the Research Council on Structural Connections (RCSC, 1980) and AISC (1978). In 1986, design provisions for the maximum size of bolt holes and the allowable tension *stress* for bolts were added to the *Specification* (AISI, 1986). In the 1996 edition of the *Specification*, minor changes to the *safety factors* were made for computing the *allowable* and *design tensile* and *shear strengths* [*factored resistances*] of bolts. The allowable tensile *stress* for the bolts subject to the combination of shear and tension was determined by the equations provided in *Specification* Table J3.4-2 with the applicable *safety factor*. In 2022, SAE J429 bolts were added to the *Specification* based on comparisons with ASTM A307 and ASTM F3125 Grade A325/A325M and A490/A490M bolts. Comparisons were made between the bolt geometry, nut and washer, bearing strength of *connections*, shear strength of the SAE bolt, and respective quality assurance requirements (Bodwell and Green, 2020).

(a) Scope

Previous studies and practical experiences have indicated that the structural behavior of bolted *connections* used for joining *relatively thick* cold-formed steel members is similar to that for connecting hot-rolled shapes and built-up members. The *Specification* criteria are applicable only to cold-formed steel members or elements 3/16 inch (4.76 mm) or less in *thickness*. For materials greater than 3/16 inch (4.76 mm), ANSI/AISC 360 (AISC, 2016a) should be used for the United States and Mexico and CSA S16 (CSA, 2014) should be used for Canada.

Because of the lack of appropriate test data and the use of numerous surface conditions, this *Specification* does not provide design criteria for slip-critical (also called friction-type) *connections*. When such *connections* are used with cold-formed steel members where the *thickness* of the thinnest connected part is 3/16 inch (4.76 mm) or less, it is recommended that tests be conducted to confirm their design capacity. The test data should verify that the specified design capacity for the *connection* provides sufficient safety against initial slip at least equal to that implied by the provisions of ANSI/AISC 360 and CSA S16. In addition, the safety against ultimate capacity should be at least equal to that implied by this *Specification* for bearing-type connections.

The *Specification* provisions apply only when there are no gaps between plies. The designer should recognize that the *connection* of a rectangular tubular member by means of bolt(s) through such members may have less strength than if no gap existed. Structural performance of *connections* containing unavoidable gaps between plies would require tests in accordance with *Specification* Section K2.1.

(b) *Materials*

This section lists different types of fasteners which are normally used for cold-formed steel construction. In view of the fact that ASTM F3125 Grades A325/A325M and A490/A490M bolts are available only for diameters of 1/2 inch (12 mm) and larger, A449 and A354 Grade BD bolts should be used as an equivalent of ASTM F3125 Grades A325/A325M and A490/A490M bolts, respectively, whenever smaller bolts (less than 1/2 inch (12 mm) in diameter) are required.

In addition to the ASTM fasteners, SAE J429 bolts along with appropriate nuts and washers have been added to the *Specification*. These fasteners provide practitioners of cold-formed steel design more available options with bolt sizes. The SAE bolts have been added based on a comparative study of fastener geometry, grade (strength) and manufacturing quality assurance standards. It should be noted that SAE bolts are required to be inspected in accordance with ASME B18.18 (ASME, 2017). The *nominal tension* and *shear strengths* [*tension* and *shear resistance*] presented in *Specification* Table J3.4-1(b) of Appendices A and B for SAE J429 bolts are the result of the *tensile strength*, F_u, of the steel specified in the SAE J429 standard multiplied by the appropriate factor for threaded parts in *Specification* Table J3.4-1(b) of Appendices A and B.

During recent years, other types of fasteners, with or without special washers, have been widely used in steel structures using cold-formed steel members. The design of these fasteners should be determined by tests in accordance with Section K2 of this *Specification*.

(c) *Bolt Installation*

Bolted *connections* in cold-formed steel structures use either mild or high-strength steel bolts and are designed as a bearing-type *connection*. Bolt pre-tensioning is not required because the ultimate strength of a bolted *connection* is independent of the level of bolt preload. Installation must ensure that the bolted assembly will not come apart during service. Experience has shown that bolts installed to a snug tight condition do not loosen or "back-off" under normal building conditions and are not subject to vibration or *fatigue*.

Bolts in slip-critical *connections*, however, must be tightened in a manner which ensures the development of the fastener tension forces required by the Research Council on Structural Connections (1985 and 2000) for the particular size and type of bolts. Turn-of-nut rotations specified by the Research Council on Structural Connections may not be applicable because such rotations are based on larger grip lengths than those encountered in usual cold-formed steel construction. Reduced turn-of-the-nut values would have to be established for the actual combination of grip and bolt. A similar test program (RCSC, 1985 and 1988) could establish a cut-off value for calibrated wrenches. Direct tension indicators (ASTM F959), whose published clamping forces are independent of grip, can be used for tightening slip-critical *connections*.

The nuts specified, SAE J995, J2486, or J2655, should be appropriately selected to match the J429 bolt selected. Selection of the appropriate nut for the specified SAE bolt is critical. Unlike the ASTM specifications, the SAE bolt specifications do not specify the appropriate nut for the grade of the bolt. This leaves it to the discretion of the designer to select the appropriate grade of nut to be compatible with the grade of the bolt. Typically, a nut with equal or greater proof *stress* is selected to ensure compatibility.

(d) Hole Sizes

For bolts having diameters less than 1/2 inch (12 mm), the diameter of a standard hole is the diameter of bolt plus 1/32 inch (1 mm). In 2014, metric hole sizes were adjusted to whole millimeters. Hole sizes for 1 inch (24 mm) and larger bolts were increased in line with AISC practices (AISC, 2016a).

An alternative short-slotted hole size was added to Table J3 as a result of a research project undertaken by Yu and Xu (2010), who investigated bolted *connections* having various hole dimensions.

When using oversized holes or short-slotted holes, care must be exercised by the designer to ensure that excessive deformation due to slip will not occur at working *loads*. Excessive deformations, which can occur in the direction of the slots, may be prevented by requiring bolt pretensioning.

Short-slotted holes are usually treated in the same manner as oversized holes. Washers or back-up plates should be used over oversized or short-slotted holes in an outer ply when the bolt hole deformation is considered in design. For *connections* using long-slotted holes, *Specification* Section J3 requires that the washers or back-up plates be used and that the shear capacity of bolts be determined by tests because a reduction in strength may be encountered.

Design information for oversized and slotted holes is included in Section J3.3.1 because such holes are often used in practice to meet dimensional tolerances during erection.

When the bolt hole deformation is considered in design, standard holes should be used in bolted *connections*. Oversized holes and slotted holes are only permitted as approved by the designer. An exception to the provisions for slotted holes is made in the case of slotted holes in lapped and nested zees. Resistance is provided in this situation partially by the nested components, rather than direct bolt shear and bearing. An oversized or slotted hole is required for proper fit-up due to offsets inherent in nested parts. Research (Bryant and Murray, 2001) has shown that lapped and nested zee members with 1/2-in. (12-mm) diameter bolts without washers and 9/16 in. × 7/8 in. (15 mm x 23 mm) slotted holes can develop the full moment in the lap.

J3.2 Minimum Edge and End Distance

In 2024, the *Specification* was updated to specifically require that rupture be checked in addition to minimum end or edge distance. The minimum edge distance was never intended to prevent rupture. Therefore, this change requires users to check both the minimum edge or end distance and for rupture.

J3.3 Bearing

Previous bolted *connection* tests have shown that *bearing* strength of bolted *connections* depends on: (1) the *tensile strength*, F_u, of the connected parts, (2) the *thickness* of connected parts, (3) the diameter of bolt, (4) *joints* with single shear and double shear conditions, (5) the F_u/F_y ratio, and (6) the use of washers (Winter, 1956a and 1956b; Chong and Matlock, 1974; Yu, 1982 and 2000). These design parameters were used in the 1996 and earlier editions of the *Specification* for determining the *bearing* strength between bolt and connected parts (AISI, 1996).

In the Canadian Standard (CSA, 1994), the d/t ratio was also used in the design equation for determining the *bearing* strength of bolted *connections*.

J3.3.1 Bearing Strength Without Consideration of Bolt Hole Deformation

Rogers and Hancock (1998) developed the design equation for bearing of bolted *connections* with washers (*Specification* Table J3.3.1-1). Based on research at the University of Waterloo (Wallace, Schuster, and LaBoube, 2001a), the Rogers and Hancock equation was extended to bolted *connections* without washers and to the inside sheet of double shear *connections* with or without washers (*Specification* Table J3.3.1-2). In *Specification* Table J3.3.1-1, the bearing factor, C, depends on the ratio of bolt diameter to member *thickness*, d/t. The design equations in *Specification* Section J3.3.1 are based on available test data. Thus, for sheets thinner than 0.024 in. (0.61 mm), tests must be performed to determine the structural performance.

The *safety factor* and *resistance factors* are based on calibration of available test data (Wallace, Schuster, and LaBoube, 2001b).

Yu and Xu (2010) conducted testing of bolted *connections* without washers on oversized and short-slotted holes. Based on the test data, Yu and Xu developed new equations for bearing factor, C, and new values for modification factor, m_f. The hole dimensions investigated in Yu and Xu (2010) are consistent with those in Table J3. The added provisions for oversized and short-slotted holes do not apply to the slotted holes in lapped and nested zees. The *safety factor* and *resistance factors* are verified by Yu and Xu (2010) to be applicable for bolted *connections* using oversized and short-slotted holes.

J3.3.2 Bearing Strength With Consideration of Bolt Hole Deformation

Based on research at the University of Missouri-Rolla (LaBoube and Yu, 1995), design equations have been developed that recognize the presence of hole elongation prior to reaching the limited *bearing* strength of a bolted *connection*. The researchers adopted an elongation of 0.25 in. (6.4 mm) as the acceptable deformation limit. This limit is consistent with the permitted elongation prescribed for hot-rolled steel.

Since the *nominal strength* [*resistance*] value with consideration of bolt hole deformation should not exceed the *nominal strength* [*resistance*] without consideration of the hole deformation, this limit was added in 2004.

J3.4 Shear and Tension in Bolts

The design provisions of this section are given in Section J3.4 of Appendix A or B. In Appendix A, the commentary is provided for Section J3.4. ☞A

J3.5 Shear Strength of Lap Connections with a Single Bolt

In 2022, provisions were added to *Specification* Section J3.5 for the shear strength of bolted lap *connections* with a single bolt. As shown by Ding et al. (2020), *Specification* Equation J6.1-1 for *shear rupture strength* underpredicted the bolt *rupture* capacity when the bolt was close to the edge. It was found that the end *rupture* equation in the 2007 edition of the *Specification* was predicting the capacity more accurately than those of the 2012 and 2016 editions of *Specification*. Accordingly, the end *rupture* equation was brought back to the *Specification* as in Equation J3.5-1. Based on the experimental observations, an additional limit state of tilting-bearing was considered for bolted lap *connections*, as Ding et al. (2020) and Teh and Uz (2017) discussed.

As shown by Ding et al. (2020), for the end *rupture*, the typical shear *rupture* path, 2 (0.6 t e F_u) or (1.2t e F_u), is unconservative even if the clear distance is used for variable (e), as discussed by Ding et al. (2020) and Teh and Uz (2015). Consequently, (t e F_u) is employed.

The proposed method is generally applicable to bolted *connections* prone to tilting including lap *connections* with very thin plates, large bolt spacings, and multiple bolts but further studies are required to quantify the *connection* capacity.

J4 Screw Connections

The results of over 3500 tests worldwide were analyzed to formulate screw *connection* provisions (Peköz, 1990). European Recommendations (1987) and British Standards (1992) were considered and modified as appropriate. Since the provisions apply to many different screw *connections* and fastener details, a greater degree of conservatism is implied than is otherwise typical within this *Specification*. These provisions are intended for use when a sufficient number of test results are not available for the particular application. A higher degree of accuracy can be obtained by testing any particular *connection* geometry (AISI, 1992).

Over 450 elemental *connection* tests and eight *diaphragm* tests were conducted in which compressible fiberglass insulation, typical of that used in metal building roof systems (MBMA, 2002), was placed between steel sheet samples in the elemental *connection* tests and between the deck and *purlin* in the *diaphragm* tests (Lease and Easterling, 2006a, 2006b). The results indicate that the equations in Section J4 of the *Specification* are valid for applications that incorporate 6-3/8 in. (162 mm) or less of compressible fiberglass insulation.

Screw *connection* tests used to formulate the provisions included single fastener specimens as well as multiple fastener specimens. However, it is recommended that at least two screws should be used to connect individual elements. This provides redundancy against under-torquing, over-torquing, etc., and limits lap shear *connection* distortion of flat unformed members such as straps.

In 2020, a research study (Stevens, Sputo, and Bridge, 2020) was undertaken to review the data available at that time on *connections* loaded in both shear and tension. The result of this research permitted the revision of *safety factors* and *resistance factors.*

Proper installation of screws is important to achieve satisfactory performance. Power tools with adjustable torque controls and driving depth limitations are usually used.

For the convenience of designers, Table C-J4-1 gives the correlation between the common number designation and the nominal diameter for screws. See Figure C-J4-1 for the measurement of nominal diameters.

Equations for pullout and shear (tilting and bearing) assumed that the proper fastener is chosen for the application and the thickness of the joined materials.

Table C-J4-1 Nominal Diameter for Screws

Number Designation	Nominal Diameter, d in.	mm
0	0.060	1.52
1	0.073	1.85
2	0.086	2.18
3	0.099	2.51
4	0.112	2.84
5	0.125	3.18
6	0.138	3.51
7	0.151	3.84
8	0.164	4.17
10	0.190	4.83
12	0.216	5.49
1/4	0.250	6.35

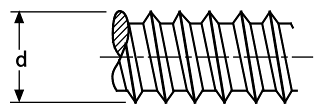


Figure C-J4-1 Nominal Diameter for Screws

In 2024, reference to Chapter D was deleted to eliminate conflicting requirements.

J4.1 Minimum Spacing

Minimum spacing is the same as specified for bolts.

J4.2 Minimum Edge and End Distances

In 2001, the minimum edge distance was decreased from 3d to 1.5d.

In 2024, the *Specification* was updated to specifically require that rupture be checked in addition to minimum end or edge distance. The minimum edge distance was never intended to prevent rupture. Therefore, this change requires users to check both the minimum edge or end distance and for rupture.

J4.3 Shear

J4.3.1 Single Shear Connection Strength [Resistance] Limited by Tilting and Bearing

Screw *connections* loaded in shear can fail in one mode or in combination of several modes. These modes are screw shear, edge tearing, tilting and subsequent pull-out of the screw, and bearing of the joined materials.

Tilting of the screw followed by threads tearing out of the lower sheet reduces the *connection* shear capacity from that of the typical *connection bearing* strength (Figure C-J4.3.1-1).

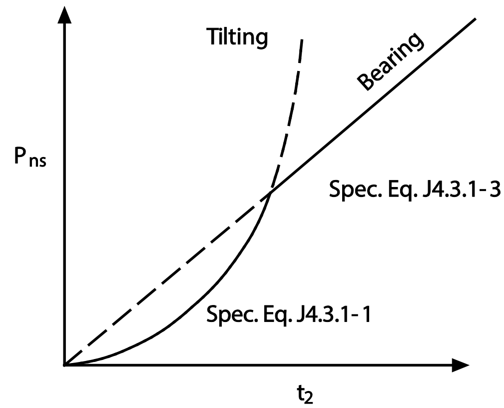
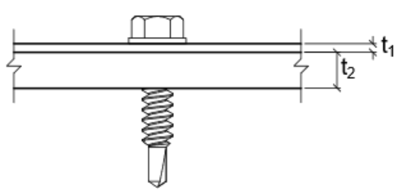


Figure C-J4.3.1-1 Comparison of Tilting and Bearing

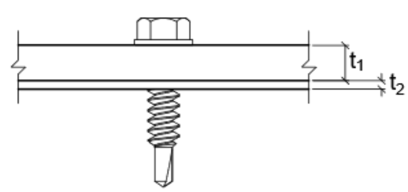
These provisions are focused on the tilting and *bearing* failure modes. Two cases are given depending on the ratio of *thicknesses* of the connected members. Normally, the head of the screw will be in contact with the thinner material as shown in Figure C-J4.3.1-2. However, when both members are the same *thickness*, or when the thicker member is in contact with the screw head, tilting must also be considered as shown in Figure C-J4.3.1-3.

It is necessary to determine the lower *bearing* capacity of the two members based on the product of their respective *thicknesses* and *tensile strengths*.



Tilting: N/A
Bearing: $P_{nv} = 2.7\ t_1 dF_{u1}$ or
Bearing: $P_{nv} = 2.7\ t_2 dF_{u2}$

Figure C-J4.3.1-2, Design Equations for t2/t1 ≥ 2.5



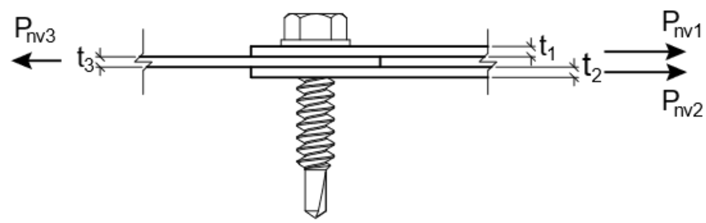
Tilting: $P_{nv} = 4.2(t_2^3)^{1/2}\ F_{u2}$ or
Bearing: $P_{nv} = 2.7\ t_1 dF_{u1}$ or
Bearing: $P_{nv} = 2.7\ t_2 dF_{u2}$

Figure C-J4.3.1-3 Design Equations for $t_2/t_1 \le 1.0$

J4.3.2 Double Shear Connection Strength Limited by Bearing

The *Specification* provisions for determining the double shear *connection* strength represent an extension of the bearing strength of screw *connections* where tilting of screws is restrained by the double shear *connection* geometry. The limiting strength of the *connection* is based on the bearing strength of the plies in the double shear *connection*. This provides a path to design of *connections* with three or more sheets.

The total *connection* strength is the sum of the forces in the individual sheets when the weakest of the sheets has reached its limiting strength, but not to exceed the dowel shear of the screws as defined in *Specification* Section J4.3.3. For a three-layer *connection* as shown in Figure C-J4.3.2-1, the limiting strength occurs when the first of the three bearing strengths is exceeded.



$P_{nv1} = 2.7\ t_1\ d\ F_{u1}$
$P_{nv2} = 2.7\ t_2\ d\ F_{u2}$
$P_{nv3} = 2.7\ t_3\ d\ F_{u3}$

Figure C-J4.3.2-1 Three-Layer Double Shear Connection

When extending the double shear *connection* to assemblies with varying sheet steel *thickness* or with four or more layers, consideration of screw tilting effects due to eccentricity may be appropriate. For these conditions additional analysis is recommended.

J4.3.3 Shear in Screws

Shear strength of the screw fastener itself should be known and documented from testing. Screw strength should be established and published by the manufacturer. In order to prevent the brittle and sudden shear fracture of the screw, the *Specification* applies a 25 percent adjustment to the *safety factor* or the *resistance factor* where determined in accordance with *Specification* Section K2.1.

J4.4 Tension

Screw *connections* loaded in tension can fail either by the screw pulled out from the plate (pull-out); material pulled over the screw head and the washer, if a washer is present (pull-over); or by tensile fracture of the screw. The serviceability concerns of gross distortion are not covered by the equations given in *Specification* Section J4.4.

Diameter and rigidity of the fastener head assembly as well as sheet *thickness* and *tensile strength* have a significant effect on the pull-over failure load of a *connection*.

There are a variety of washers and head styles in use. Washers must be sufficiently thick to withstand bending forces with little or no deformation. In 2010, the minimum washer *thickness* requirement of 0.050 in. (1.27 mm) was relaxed for the washers in *connections* where t_1 does not exceed 0.027 in. (0.686 mm), with the evidence that the washer *thickness* of as low as 0.024 in. (0.610 mm) does not adversely impact the pull-over strength of the *connection* for such top substrate *thicknesses* (Mujagic, 2008). In 2012, the washer dimension requirements were modified to harmonize the limitations of *Specification* Sections J4.5 with J4.4, given similar pull-over models in the two sections. Based on the findings of Zwick and LaBoube (2002), washers with outside diameter of 5/8 to 3/4 in. (15.9 mm to 19.1 mm) and a minimum *thickness* of 0.063 in. (1.60 mm) were included in the scope of *Specification* Section J4.4. Designers should include minimum required washer *thickness* in project documents.

J4.4.1 Pull-Out Strength

For the *limit state* of pull-out, *Specification* Equation J4.4.1-1 was derived on the basis of the modified European Recommendations and the results of a large number of tests. The statistical data on pull-out design considerations were presented by Peköz (1990).

In 2020, a research study (Stevens, Sputo, and Bridge, 2020) indicated that an empirical modifier ($1.63t_c^{0.18}$) could be added to the resistance equation to better fit the available experimental data. Pull-out strength is related to the ratio of sheet *thickness* to thread pitch, but in the interest of practicality, instead of thread pitch, the sheet *thickness* alone was used in the calibration.

J4.4.2 Pull-Over Strength

For the *limit state* of pull-over, *Specification* Equation J4.4.2-1 was derived on the basis of the modified British Standard and the results of a series of tests as reported by Peköz (1990). In 2007, a rational allowance was included to cover the contribution of steel washers beneath screw heads. For the special case of screws with domed washers (washers that are not solid or do not seat flatly against the sheet metal in contact with the washer), the calculated *nominal pull-over strength* [*resistance*] should not exceed $1.5t_1d'_wF_{u1}$ with d'_w = 5/8 in. (15.9 mm). The 5/8 in. (15.9 mm) limit does not apply to solid steel washers in full contact with the sheet metal. In accordance with *Specification* Section J4, testing is allowed as an alternative method to determine fastener capacity. To use test data in design, the tested material should be consistent with the design. When a polygon-shaped washer is used and capacity is determined using *Specification* Equation J4.4.2-1, the washer should have rounded corners to prevent premature tearing.

In 2010, the pancake head washer screws and domed washers integral with the screw head were added and defined to assist the designer in proper determination of computational variables.

In 2020, a research study (Stevens, Sputo, and Bridge, 2020) indicated that the existing pull-over equation adequately predicted the pullover strength, except for low ductility sheets which are covered by *Specification* Section A3.1.3 (Elongation ≤ 3%) with *thickness* less than 0.023 inches (0.58 mm), where a reduction of 40 percent was warranted. For these thin sheets, it appears that the magnitude of the clamping force, and the geometry of the *connection* (including distance from the screw to adjacent stiffeners) is a factor (Kreiner and Ellifritt, 1998).

J4.4.3 Tension in Screws

Tensile strength of the screw fastener itself should be known and documented from testing. Screw strength should be established and published by the manufacturer. In order to prevent the brittle and sudden tensile fracture of the screw, the *Specification* applies a 25 percent adjustment to the *safety factor* or the *resistance factor* where determined in accordance with Section K2.1.

J4.5 Combined Shear and Tension

Section J4.5 checks three failure modes where shear and tension are present at a *connection*: *connection* failures due to combined shear and pull-over, and combined shear and pull-out, as well as screw failure in the shank due to combined shear and tension.

J4.5.1 Combined Shear and Pull-Over

Research pertaining to the behavior of a screw *connection* has been conducted at West Virginia University (Luttrell, 1999). Based on the review and analysis of West Virginia University's data for the behavior of a screw *connection* subject to combined shear and tension (Zwick and LaBoube, 2002), equations were derived that enable the evaluation of the strength of a screw *connection* when subjected to combined shear and tension. The tests indicated that at failure, the sheet beneath the screw head pulled over the head of the screw or the washer. Therefore, the *nominal tensile strength* [*resistance*] is based solely on P_{nov}. Although both nonlinear and linear equations were developed for ease of computation and because the linear equation provides regions of $\overline{V}/P_{nv}$ and $\overline{T}/P_{nov}$ equal to unity, the linear equation was adopted for the *Specification*. The proposed equation is based on the following test program limits:

0.0285 in. (0.724 mm) $\le t_1 \le 0.0445$ in. (1.13 mm)

No. 12 and No. 14 self-drilling screws with or without washers

$d_w \le 0.75$ in. (19.1 mm)

62 ksi (427 MPa or 4360 kg/cm^2) $\le F_{u1} \le$ 70.7 ksi (487 MPa or 4970 kg/cm^2)

$t_2 / t_1 \ge 2.5$

The limit $t_2 / t_1 \ge 2.5$ reflects the fact that the test program (Luttrell, 1999) focused on *connections* having sheet *thicknesses* that precluded the tilting *limit state* from occurring. Thus, this limit ensures that the design equations will only be used when tilting *limit state* is not the controlling *limit state*.

The standard washer with outside diameter of 3/4 in. (19.1 mm) has a minimum *thickness* of 0.063 in. (1.60 mm). In 2011, the washer dimension limitations of *Specification* Sections J4.4 and J4.5 were harmonized, given similar pull-over models in the two sections.

The linear form of the equation as adopted by the *Specification* is similar to the following more conservative linear design equation that has been used by engineers:

$$\overline{V}/P_{nv} + \overline{T}/P_{nov} \le 1.0 \qquad \text{(C-J4.5.1-1)}$$

See *Specification* Section J4.5.1 for the definitions of the variables.

An eccentric *load* on a clip *connection* may create a nonuniform *stress* distribution around the fastener. For example, tension tests on roof panel welded *connections* have shown that under an eccentrically applied tension force, the resulting *connection* capacity is 50 percent of the tension capacity under a uniformly applied tension force. Thus, the *Specification* stipulates that the *nominal pull-over strength* [*resistance*] shall be taken as 50 percent of P_{nov}. If the eccentric *load* is applied by a rigid member such as a clip, the resulting tension force on the screw may be uniform; thus, the force in the screw can be determined by mechanics, and the capacity of the fastener should be reliably estimated by P_{nov}. Based on the field performance of screw-attached panels, the 30 percent reduction associated with welds at sidelaps need not be applied when evaluating the strength of sidelap screw *connections* at supports or for sheet-to-sheet. The reduction is due to transverse prying or peeling. It is acceptable to apply the 50 percent reduction at panel ends due to longitudinal prying.

J4.5.2 Combined Shear and Pull-Out

Research pertaining to the behavior of a screw *connection* has been conducted at the Missouri University of Science and Technology (Francka and LaBoube, 2010). Based on the findings of this research, equations were derived that enable the evaluation of the strength of a screw *connection* when subjected to combined shear and tension. The tests indicated that at failure, the screw pulled out of the bottom sheet of the *connection*. Therefore, the *nominal tensile strength* [*resistance*] is based solely on the tilting and tearing failure mode, *Specification* Equation J4.5.2-2. Although both nonlinear and linear equations were developed, the reliability of the nonlinear and linear equations was comparable. Therefore, for ease of computation, the linear equation was adopted for the *Specification*. The proposed equation is based on the test program limits as defined in the *Specification*. Evaluation of the *connection* for the combined shear and pull-out does not negate the need to evaluate the shear alone and pull-out alone *limit states*.

J4.5.3 Combined Shear and Tension in Screws

In 2012, new provisions were added to account for shear and tension interaction in screws. Based on the *rational engineering analysis*, the same strength interaction as that used for bolts, *Specification* Equations J3.4-2 (*ASD*) and J3.4-3 (*LRFD* and *LSD*) (but in a different form) are used for screws.

J5 Power-Actuated Fastener (PAF) Connections

In 2012, Section J5 was added to address *connections* with *power-actuated fasteners* (*PAFs*) connecting steel elements in non-*diaphragm* applications. These provisions do not preclude evaluation of any *limit state* on any *power-actuated fastener* through manufacturer or independent laboratory testing. The *safety* and *resistance factors* for any *nominal strength* [*resistance*] established through testing should be determined using provisions of Section K2 of the *Specification*.

In *Specification* Section J5, the provisions for determining the *available strengths* [*factored resistances*] were developed based on the study by Mujagic, et al. (2010). Applicability constraints of these provisions correspond to the limitations of data available in the study (Mujagic, et al., 2010).

In the provisions, the term "near side of the embedment material" refers to the surface of the embedment material from which the *PAF* is driven. The term "far side of the embedment material" refers to the embedment material surface from which the driven fastener exits.

PAFs are produced from heat-treated steels, resulting in a material that is hardened with a certain level of ductility. Such properties are needed for *PAFs* to penetrate steel, concrete, or masonry and compress within these base materials without fracturing. Some *PAFs* are knurled. Knurling is a manufacturing process where a pattern of straight, angled or crossed fine grooves are cut or rolled into the shank or tip of a *power-actuated fastener*. In many cases, knurling of carbon steel *PAF* shanks and, for thicker base materials, tips, has been demonstrated to improve the holding power of the *PAF* to steel *connections* and reduce the variability of anchorage strengths. Stainless steel *PAFs* exhibit reliable *connection* strengths without knurling.

J5.1 Minimum Spacing, Edge and End Distances

The minimum center-to-center spacing of the *PAFs* and the edge distances in the *Specification* are those stipulated by Table 2 of ASTM E1190 (ASTM, 2008) for placement of test equipment supports on the steel embedment material. While larger spacing and edge distances are frequently found in test reports, the minimum distances given in ASTM E1190 (ASTM, 2008) are also deemed sufficient for fastener placement in steel in avoiding the detrimental effects of inadequate edge distance or fastener grouping.

In 2024, the *Specification* was updated to specifically require that rupture be checked in addition to minimum end or edge distance. The minimum edge distance was never intended to prevent rupture. Therefore, this change requires users to check both the minimum edge or end distance and for rupture.

J5.2 Power-Actuated Fasteners (PAFs) in Tension

Applicable *limit states* in tension include tension fracture, pull-out, and pull-over. The determination of *available strength* [*factored resistance*] due to any particular *limit state* for the fasteners depicted in *Specification* Figure J5-1 should be accomplished through appropriate testing. Alternatively, the *available strength* [*factored resistance*] should be determined using Sections J5.2.1 through J5.2.3 of the *Specification*.

J5.2.1 Tension Strength of Power-Actuated Fasteners (PAFs)

Power-actuated fasteners (*PAFs*) typically possess the Rockwell hardness (HRC) values of 49 to 61. Adequate HRC values represent one of the most critical design, installation and behavioral features of *PAFs*. HRC_p values may be determined by tests using currently available test methods such as ASTM E18 (ASTM, 2016) or ISO 6508-1 (ISO, 2016). The HRC_p values can be properly related to *tensile strength* in most ranges of HRC_p. The study by Mujagic, et al. (2010) showed that the *nominal tensile fracture strength* [*resistance*] can be determined using the value of 260,000 psi (1790 MPa) for the HRC_p range in excess of 52. The user is cautioned to distinguish between the strength properties and HRC_p of pre-hardened steel from which a fastener is made and those of the hardened steel representing the final fastener product.

Specification Equation J5.2.1-1 was developed with the *PAF* driven such that no part of the length, ℓ_{dp}, as illustrated in *Specification* Figure J5-1, is located above the near side of the embedment material.

J5.2.2 Pull-Out Strength

The *nominal pull-out strength* [*resistance*] of *power-actuated fasteners* (*PAFs)* greatly depends on minute metallurgical, geometric, installation, and other design (often proprietary) features. *PAFs* develop their pull-out strength through partial fusion to the embedment material and friction resulting from the confinement *stresses* imposed by the displaced embedment material. Mechanical interlock or keying with *PAF* shank knurling and brazing effects due to zinc plating of the *PAF* also contribute to strength. According to Beck, Englehardt, and Glaser (AISC, 2003), a number of factors affect the pull-out strength of *PAFs* in steel. These include the embedment depth of the fastener, the base steel *thickness*, the base steel strength, the diameter of the fastener, knurling of the fastener, fastener zinc coating thickness, base steel *stress*, and others. While various behavioral trends can be established, it is not possible to develop a generic prediction model for *PAFs* which captures the above-mentioned, often proprietary, specific design features. Consequently, it was decided to stipulate testing as the only viable method of determining the pull-out strength. This approach is similar to how the pull-out strength is addressed in the EN 1993-1-3 (CEN 2006). The currently available testing protocols for determining the pull-out strength are given in AISI S905-17 and ASTM E1190 (ASTM, 2011).

The tabulated *nominal pull-out strengths* [*resistances*] in Table C-J5.2.2-1 are provided for informational purposes. The table is extracted from the study by Mujagic, et al. (2010), and it represents lower bound values from a limited selection of industry fastener and embedment plate combinations available to the study. Table C-J5.2.2-1 is only applicable to fasteners embedded in steel plate for which manufacturer applicability guidelines stipulate embedment condition whereby no part of the length, ℓ_{dp}, of *PAF point*, as illustrated in *Specification* Figure J5-1, is located above the near side of the embedment material. The values in Table C-J5.2.2-1 were scaled such that a *safety factor* of 3.0 computed in accordance with Section K2 of the *Specification* can be justified for the *nominal strength* [*resistance*] value of each of the considered fasteners. Since these are lower bound solutions, the actual *safety factor* for some of the fasteners would be higher than 3.0. The table is only applicable to fastener types and geometries depicted in *Specification* Figure J5-1. The current design practice generally involves reliance on tested capacities established per International Code Council Evaluation Service (ICC-ES) Acceptance Criteria 70 (AC70) (ICC-ES, 2010). The AC70 stipulates a minimum *safety factor* of 5.0, thus in many cases resulting in lower *allowable strength* values than those implied by Table C-J5.2.2-1. The approaches for establishing the *safety factor* stipulated by Section K2 of the *Specification* and by ICC-ES AC70 are not consistent. However, the values in Table C-J5.2.2-1 can be conservatively related to the current practice by reducing the *nominal strength* [*resistance*] values given therein by a factor of 0.6 (i.e., 3/5).

Table C-J5.2.2-1
Nominal Tensile Pull-Out Strength of PAFs in Steel, P_{not}, lbs (N)

	Embedment Thickness, in. (mm)		
PAF Shank Diameter, d_s, in. (mm)	1/8 (3.18)	3/16 (4.76)	1/4 (6.35)
0.106 (2.69) ≤ d_s < 0.146 (3.71)	450 (2000)	915 (4070)	1230 (5470)
0.177 (4.50) ≤ d_s < 0.206 (5.23)	-	-	1970 (8760)

Where statistical indices required to compute the *safety* and *resistance factors* in accordance with *Specification* Section K2 are not given for a pull-out strength provided by a manufacturer, a *safety factor* of 4.0 and *a resistance factor* of 0.40 (0.30 for *LSD*) can be applied to the *nominal strengths* [*resistance*] provided in Table C-J5.2.2-1, or to test results where the mean is known. The number of data points should be 10 or more in accordance with ASTM E1190 (ASTM, 2011). This option was provided based on the study by Mujagic, et al. (2010) which shows that 4.0 represents a conservative lower bound value of *safety factor* for a variety of fastener types and models when the computed *safety factor* or data required for its computation is not available to the user.

J5.2.3 Pull-Over Strength

The pull-over *limit state* in *PAF connections* is fundamentally the same as that in screw *connections*. The *Specification* addresses the screw-like *PAFs* in an identical manner that screw *connections* are dealt with in *Specification* Section J4. The two notable exceptions represent *connections* with tapered-head fasteners that consistently yield about 20 percent lower pull-over strength than screw-like *PAF connections*, and *connections* with collapsible spring washers that consistently yield about 30 percent higher strength than screw-like *PAF connections*. The *Specification* addresses the two special cases by varying the constant multiplier of the pull-over equation.

J5.3 Power-Actuated Fasteners (PAFs) in Shear

Applicable *limit states* in shear are shear fracture, bearing and tilting, pull-out, net section checks, and *nominal shear strength* [*resistance*] limited by edge distance.

J5.3.1 Shear Strength of Power-Actuated Fasteners (PAFs)

Nominal shear strength [*resistance*] of *PAFs* is determined by relating the ultimate *tensile strength* in tension to that in shear by a factor of 0.6.

J5.3.2 Bearing and Tilting Strength

The *nominal bearing strength* [*resistance*] is based on the equation proposed in the study by Mujagic, et al. (2010) based on the data for which $t_2/t_1 \geq 2.0$ and $t_2 \geq 1/8$ in. (3.2 mm). While some decrease in calculated strength was observed with decreasing t_2/t_1 ratio, thus suggesting the presence of tilting at lower ratios of t_2/t_1, it was noted that the bearing and tilting strength can be predicted by setting the constant multiplier in the bearing equation to 3.7. Since the study by Mujagic, et al. (2010) was based only on the types of fasteners shown in *Specification* Figures J5(c) and J5(d), the ENV 1993-1-3 (ECS, 2006) equation constant of 3.2 is conservatively adopted for other types of *PAFs*.

J5.3.3 Pull-Out Strength in Shear

Pull-out in shear is essentially a derivative of fastener tilting in steel. The pull-out failures were reported at wide range of t_2/t_1 ratios. The *bearing* strength equation of *Specification* Section J5.3.2 considers the effect of tilting deformation on *bearing* failures at low ratios of t_2/t_1. However, as expected, it does not accurately predict the *connection* strength where tilting is the predicted failure mode. The *Specification*, therefore, stipulates a separate pull-out check over the entire range of t_2/t_1 ratios and *thicknesses* covered by the *Specification*.

J5.3.4 Net Section Rupture Strength

Based on the recommendations of Beck and Engelhardt (2002), the *PAF* hole is required to be calculated based on a width of 1.10 times the *PAF* diameter. The effect of partially driven *PAFs* (i.e., where the *PAF point length*, ℓ_{dp}, is fully or partially located in the embedment material) on net section properties of a *connection* are not presently known. The *Specification*, therefore, stipulates that the *PAF* shank diameter, d_s, be used in determination of net section properties.

J5.3.5 Shear Strength Limited by Edge Distance

The *Specification* presently stipulates the application of the same criteria given for screws in *Specification* Section J6.1, recognizing fundamental similarities in behavior and application of screw and *PAF connections*. Favorable local effects of sheath folding and local hardening of the sheathing near the *PAF* hole may render the screw *connection* criteria slightly conservative when applied to *PAF connections*. The effect of partially driven *PAFs* (i.e., where the *PAF point length*, ℓ_{dp}, is fully or partially located in the embedment material) on edge distance properties of a *connection* are not presently known. The *Specification*, therefore, stipulates that the *PAF* shank diameter, d_s, be used in edge distance checks.

J5.4 Combined Shear and Tension

Combined shear and tension in the *PAF connection* should include the interaction of combined shear and pull-over, combined shear and pull-out, and fracture due to combined shear and tension on the *PAF* fastener itself. Currently available research does not address *PAF connections* subject to combined tension and shear. Consequently, the *Specification* does not at present provide equations for consideration of such *connections*. The ICC-ES AC 70 (ICC-ES, 2010) criteria can be used to consider combined tension and shear through testing. Alternatively, such a condition can be evaluated in accordance with *Specification* Section A1.2.6. Based upon fundamental principles of fastener mechanics, Equation C-J5.4-1 represents an exact interaction between tension and shear when fastener fracture governs. Since the actual interaction curve is not presently known for other combinations of tension and shear *limit states*, the power coefficient of one, rendering the Equation C-J5.4-1 a linear interaction, can be used as a conservative check when both shear and tension are not limited by fracture.

$$\left(\frac{\overline{T}}{P_{at}}\right)^n + \left(\frac{\overline{V}}{P_{av}}\right)^n \leq 1.0 \qquad \text{(C-J5.4-1)}$$

where

$\overline{T}$ = *Required tension strength* [force due to *factored loads*]

P_{at} = *Available tension strength* [*factored resistance*] determined in accordance with *Specification* Section J5.2

$\overline{V}$ = *Required shear strength* [shear force due to *factored loads*]

P_{av} = *Available shear strength* [*factored resistance*] determined in accordance with *Specification* Section J5.3

n = 2 when both tension and shear are governed by the fracture *limit state*
= 1 in all other cases

J6 Rupture

The provisions contained in *Specification* Section J6 and its subsections are applicable only when the thinnest connected part is 3/16 inch (4.76 mm) or less in *thickness*. For materials thicker than 3/16 inch (4.76 mm), the design should follow ANSI/AISC 360 for the United States and Mexico and CSA S16 for Canada.

Significant changes were made to the format of *Specification* Section J6 in 2010. *Connections* may be subject to shear rupture, tension rupture, block failure in tension, block failure, or any combinations of these failures in shear depending upon the relationship of the connectors to the *connection* geometry and loading direction. *Specification* Table J6.2-1 provides adjustment factors consistent with prior editions of the *Specification* to cover shear lag factors. Other adjustment factors provide allowances for staggered connector patterns and nonuniform *stress* distribution on the tensile plane. In 2012, the Committee added a reference to *PAFs* in Table J6-1, permitting the use of the same *safety* and *resistance factors* as for screws. This step was taken recognizing inherent similarities in configurations and behavior of screw and *PAF connections* as they relate to net fracture of connected elements. Furthermore, partial fusion occurring between the embedment steel and *PAF* should result in a conservative design with respect to application of *resistance* and *safety factors* for screw *connections*. In 2024, the requirements for *PAFs* in this Section were aligned with the requirements of Section J5.3. In 2022, the *safety* and *resistance factors* for rupture of bolted *connections* were adjusted to give gains in design strength and were in line with the shear lag factors in *Specification* Table J6.2-1 and the block shear rupture *Specification* Equation J6.3-1 (Teh and Clements, 2012). In 2024, the *safety and resistance factors* for screw and *PAF connections* in Table J6-1 were adjusted to the same *safety and resistance factors* used for bolts for consistency of all dowel type *connections* since *rupture* is a failure of the member material not the fastener. In lieu of definitive research, it would be reasonable to use the nominal screw diameter, d, as shown in Figure C-J4-1, as the hole size in rupture computations.

(a) Shear Lag for Flat Sheet Connections

Earlier tests showed that for flat sheet *connections* using a single bolt or a single row having multiple bolts perpendicular to the force (Chong and Matlock, 1975; Carill, LaBoube and Yu, 1994), the *joint* rotation and out-of-plane deformation of flat sheets are excessive. Consequently, specific shear lag factors were developed. However, it was found by Teh and Gilbert (2014) that, for the *limit state* of net section tension rupture, there is no noticeable difference in the shear lag factors between different types of bolted *connections*. The apparent differences in the shear lag factors "due to joint rotation and out-of-plane deformation of flat sheets" cited in the earlier *Specification* edition were actually due to a different failure mode, namely tilt bearing failure, which is considered separately. For flat sheet *connections* using multiple connectors in the line of force and having less out-of-plane deformations, the strength reduction was not required in the 2012 edition of the *Specification* (Rogers and Hancock, 1998). A single shear lag reduction factor given by *Specification* Equation J6.2-4 (Teh and Gilbert, 2014) now applies to all cases (both single and multiple bolts in the line of the force, and single and double shear *connections*) in the 2016 edition of the *Specification.*

(b) Staggered Holes

The presence of staggered or diagonal hole patterns in a bolted *connection* has long been recognized as increasing the net section area for the *limit state* of rupture. It was first analytically studied by Cochrane (1922), who derived the adjustment term $s'^2/(4g + 2d_h)$ shown in *Specification* Equation J6.2-3. LaBoube and Yu (1995) summarized the findings of a limited study of the behavior of bolted *connections* having staggered hole patterns. The research showed that when a staggered hole pattern is present, the width of a rupture plane could be adjusted by use of $s'^2/4g$ with an additional 10 percent reduction factor. More recent testing on the critical tensile path involving stagger has been carried out by Fox and Schuster (2010), the results of which indicate that the 10 percent reduction is not required. However, the neglect of the variable involving the bolt hole diameter, d_h, in the earlier *Specification* edition was not required, as it did not lead to meaningful simplification while potentially leading to 10 percent overestimation (Teh and Gilbert, 2014). Consequently, d_h was included in Equation J6.2-3 of the 2016 edition of the *Specification.* For staggered hole patterns as may occur in block shear, Cochrane's formula may not be accurate when the staggered plane is close to the line of the force as the force on the staggered plane is mainly in shear and not tension. Consequently, for this situation, a different design approach is required for staggered holes as discussed in (d) below.

(c) Shear Lag for Other Than Flat Sheet Connections

Shear lag has a debilitating effect on the tensile capacity of a cross-section. Based on the University of Missouri-Rolla research (LaBoube and Yu, 1995), design equations have been developed that can be used to estimate the influence of the shear lag. The research demonstrated that the shear lag effect differs for an angle and a channel. For both cross-sections, however, the key parameters that influence shear lag are the distance from the shear plane to the center of gravity of the cross-section and the length of the *connection* (See Figures C-J6-1 and C-J6-2). The research by Teh and Gilbert (2014) has shown that the shear lag factors for bolted connections in angle and channel members should take into account the width ratio between the connected and the unconnected parts, in addition to the traditional ratio between the *connection* eccentricity, $\bar{x}$, and the *connection* length, L. *Specification* Equations J6.2-6 and J6.2-8 developed by Teh and Gilbert (2014) lead to accurate results for bolted *connections* in angle and channel members of various configurations and material properties. Additionally, there are no artificial lower or upper bound values for the computed shear lag factors.

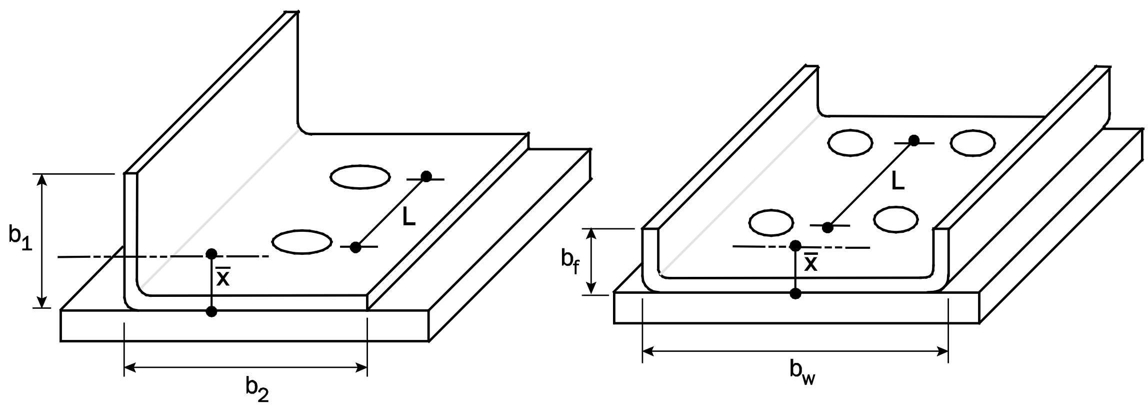


Figure C-J6-1 $\bar{x}$ Definition for Sections With Bolted Connections

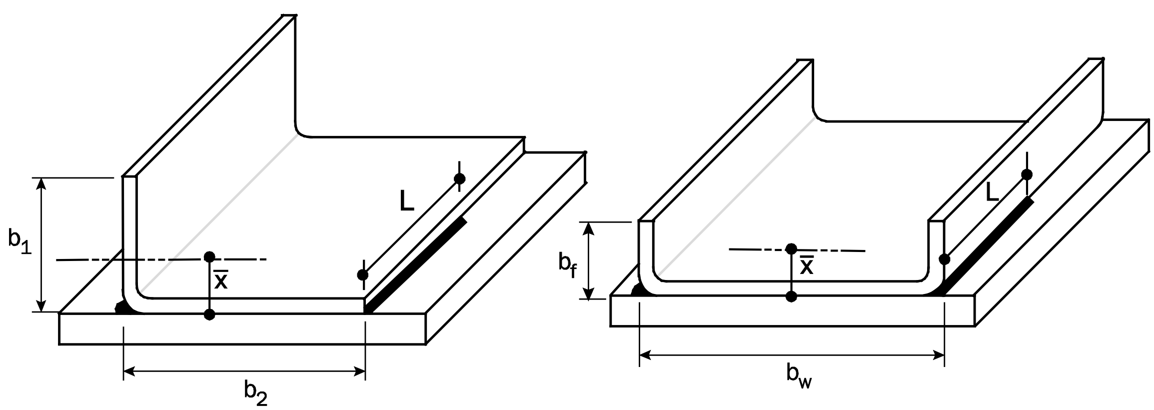


Figure C-J6-2 $\bar{x}$ Definition for Sections With Fillet Welding

Research has also shown that for cold-formed steel sections using single-bolt *connections*, *bearing* or shear rupture usually controlled the *nominal strength* [*resistance*], not rupture in the net section.

(d) Block Shear

Block shear is a *limit state* in which the resistance is determined by the sum of the shear strength on a failure path(s) parallel to the force and the tensile strength on the segment(s) perpendicular to the force. A comprehensive test program does not exist regarding block shear for cold-formed steel members. However, a limited study conducted at the University of Missouri-Rolla indicates that the AISC equations may be applied to cold-formed steel members.

Connection tests conducted by Birkemoe and Gilmor (1978) have shown that on coped beams, a tearing failure mode as shown in Figure C-J6-5 can occur along the perimeter of the holes. Hardash and Bjorhovde (1985) have demonstrated these effects for tension members as illustrated in Figure C-J6-4. The research paper "AISC *LRFD* Rules for Block Shear in Bolted Connections – A Review" (Kulak and Grondin, 2001) provides a summary of test data for block shear *rupture strength*. Teh and Clements (2012) have demonstrated that the shear area to be used is an active shear area between the net and gross shear areas, where the shear *stresses* arising from the bolts *bearing* on the bolt holes are the maximum. Numerically, the active shear plane is equal to the mean between the gross shear plane and the net shear plane, and the active shear area can be defined by $A_{av} = (A_{gv} + A_{nv})/2$ (The and Clements, 2012). The active shear area is depicted in Figure C-J6-4 as Paths 2-3 and 7-8. Further, Teh and Clements (2012) demonstrated that the net section area subject to tension should use the same in-plane shear lag reduction factor given by *Specification* Equation J6.2-4 (Teh and Gilbert, 2014). The previous equation for block shear rupture ($P_{nr} = 0.6F_yA_{gv} + U_{bs}F_uA_{nt}$) based on the gross shear area has therefore been modified in 2022 to be based on the active shear area and the equation ($P_{nr} = 0.6F_uA_{nv} + U_{bs}F_uA_{nt}$) based on the net shear area has been deleted. The term involving net area subject to tension has been modified in 2022 to include the in-plane shear lag reduction factor in the *Specification* Equation J6.2-4. For the case of staggered holes in the block shear rupture, research by Pham, V.B et al. (2022) shows that the use of Cochrane's formula as described in (b) above is not accurate when the angle of the stagger is close to the line of the force since the force on the staggered plane is mainly carried by shear and not tension. Consequently, an additional term was added to *Specification* Equation J6.3-1 in 2024 to account for the separate components in shear and tension on the staggered plane. The area of the staggered plane A_{st} defined by *Specification* Equations J6.3-5 and J6.3-6 varies with the angle of the stagger α to account for the transition from an active shear area A_{av} when the stagger has a zero angle to an angle closer to the net area A_{pt} when the stagger angle is large.

The distribution of tensile *stresses* is not always uniform (Ricles and Yura, 1983; Kulak and Grondin, 2001). For shear forces on coped beams, an additional multiplier, U_{bs}, of 0.5 is used when more than one row of bolts is present. This approach is consistent with the provisions of ANSI/AISC 360 (AISC, 2005 and 2010a).

Tests performed at the University of Missouri-Rolla have indicated that the current design equations for shear and tilting provide a reasonably good estimate of the *connection* performance for multiple screws in a pattern (LaBoube and Sokol, 2002).

Examples of failure paths can be found in Figures C-J6-3 through C-J6-7.

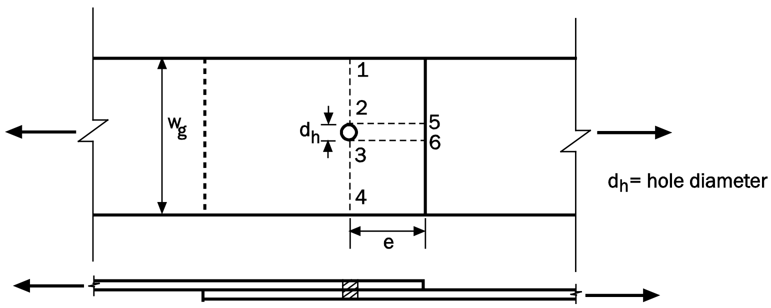


Figure C-J6-3 Potential Failure Paths of Single Lap Joint

(Tension Rupture)

Failure Path 1, 2, 3, 4

Specification Section J6.2 applies:

$A_e = U_{s\ell} A_{nt}$

$U_{s\ell}$ in accordance with *Specification* Equation J6.2-4

$A_{nt} = (w_g - d_h)\ t$

(Shear Rupture)

Failure Path 5, 2, 3, 6

Specification Section J6.1 applies:

$A_{nv} = 2n(e – d_h/2)\ t$

n = 1 as there is only a single fastener

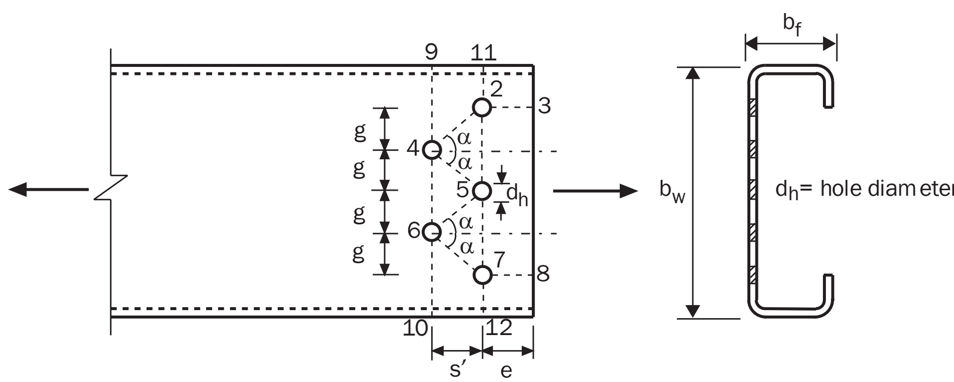


Figure C-J6-4 Potential Failure Paths of Stiffened Channel (Tension or Block Shear Rupture)

Planes 2-4, 4-5, 5-6 and 6-7 in Figure C-J6-4 are commonly called staggered planes.

(Tension Rupture)

Specification J6.2 applies:

$U_{s\ell}$ in accordance with *Specification* Equation J6.2-8

Failure Path 9, 4, 6, 10

$A_{nt} = A_g - 2d_h t$

Failure Path 11, 2, 4, 5, 6, 7, 12

$A_{nt} = A_g – 5d_h t + t(4s'^2)/(4g' + 2d_h)$

(Block Shear Rupture)

Specification Section J6.3 applies:

$A_{av} = n_{sh}L_{av}t$

$n_{sh} = 2$

$L_{av} = e - d_h/4$

$U_{bs} = 1.0$

<u>Failure Path 3, 2, 5, 7, 8</u>

$A_{pt} = (4g' - 2d_h)t$

$U_{s\ell} = 0.9 + 0.1d/(2g')$

<u>Failure Path 3, 2, 4, 5, 6, 7, 8</u>

$A_{pt} = 0$

$$A_{st} = 4t\left(\sqrt{s'^2 + g^2} - d_h + \frac{d_h}{2}10^{-4\sin\alpha}\right)$$

$U_{s\ell} = 1.0$ for staggered bolt pattern

<u>Failure Path 3, 2, 4, 6, 7, 8</u>

$A_{pt} = (2g' - d_h)\ t$

$U_{s\ell} = 0.9 + 0.1d/(2g')$

$$A_{st} = 2t\left(\sqrt{s'^2 + g^2} - d_h + \frac{d_h}{2}10^{-4\sin\alpha}\right)$$

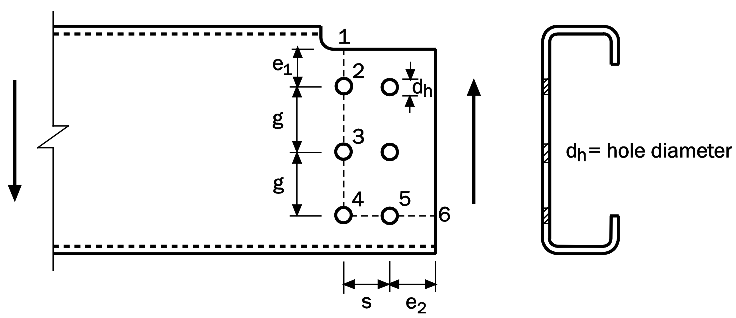


Figure C-J6-5 Potential Failure Path of Coped Stiffened Channel (Block Shear Rupture)

(Block Shear Rupture)

<u>Failure Path 1, 2, 3, 4, 5, 6</u>

Specification Section J6.3 applies:

$A_{av} = n_{sh}L_{av}t$

$n_{sh} = 1$

$L_{av} = (2p + e_1) - 5\ (d_h/4)$

$A_{pt} = [(g + e_2) - 1.5d_h]\ t$

Average value of g and e_2 is used in *Specification* Equation J6.3-4:

$U_{s\ell} = 0.9 + 0.1d/[(g + e_2)/2]$

$U_{bs} = 0.5$

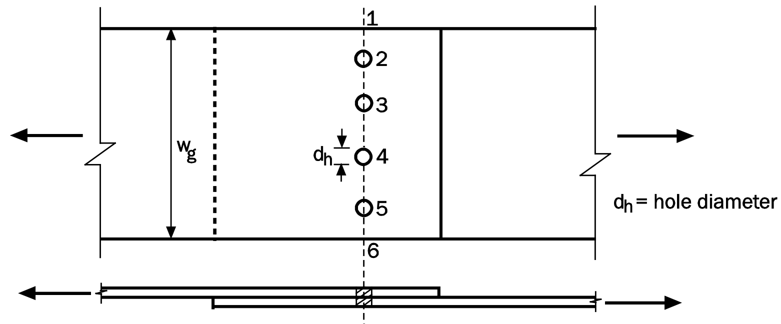


Figure C-J6-6 Potential Failure Path of Multiple-Fastener Lap Joint (Tension Rupture)

(Tension Rupture)

Failure Path 1, 2, 3, 4, 5, 6

Specification Section J6.2 applies:

$A_e = U_{s\ell} A_{nt}$

$U_{s\ell}$ in accordance with *Specification* Eq. J6.2-4

$A_{nt} = (w_g - 4d_h)\ t$

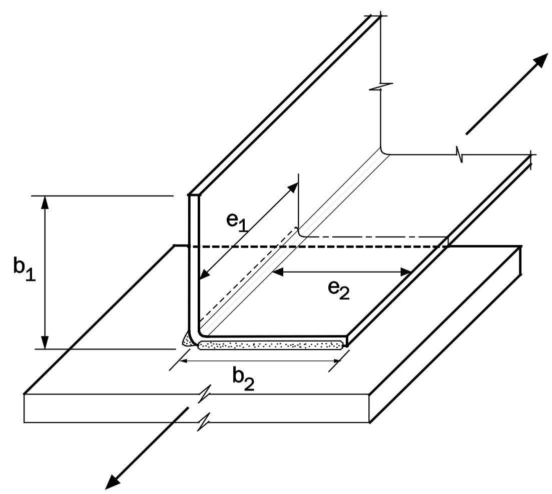


Figure C-J6-7 Potential Failure Path of Fillet-Welded Joint (Tension Rupture)

(Tension Rupture)

Specification Section J6.2 applies

$U_{s\ell}$ in accordance with *Specification Eq*. J6.2-5

J7 Connections to Other Materials

When a *cold-formed steel structural member* is connected to other materials, such as hot-rolled steel, aluminum, concrete, masonry or wood, the *connection* strength should be the smallest of the strength of the fastener, the strength of the fastener attachment to the *cold-formed steel structural member*, or the strength of the fastener attachment to the other material.

In 2016, provisions were added to *Specification* Section J7.2 for *power-actuated fasteners* (*PAFs*) connecting cold-formed steel framing track-to-concrete base materials. These provisions were based on an experimental study where cold-formed steel wall tracks were attached to concrete base materials and subjected to monotonic and cyclic/seismic test loads (AISI, 2013h). In 2018, these provisions were removed to avoid unconservative designs of track and other cold-formed steel structural member attachments to concrete and to avoid unintended interpretation of the validity of these provisions in different applications.

J7.1 Connection Strength to Other Materials

The design of *connections* to other materials should be in accordance with the *applicable building code*, including those referenced standards, as applicable. When the *applicable building code* provides no requirement with respect to consideration of specific *limit states*, other codes and standards and manufacturers' technical reports and catalogues acceptable to the *authority having jurisdiction* may be utilized. The following is a list of suggested references:

(a) Cold-Formed Steel Attached to Steels Over 3/16-Inch (4.76-mm) Thick

(1) For Welded Connections:

In the U.S. and Mexico:

AWS D1.1/D1.1M, *Structural Welding Code – Steel*

AWS D1.3/D1.3M, *Structural Welding Code – Sheet Steel*

In Canada:

CSA W47.1, *Certification of Companies for Fusion Welding of Steel*

CSA W55.3, *Certification of Companies for Resistance Welding of Steel and Aluminum*

CSA W59, *Welded Steel Construction (Metal Arc Welding)*

(2) For Bolted Connections:

In the U.S. and Mexico: ANSI/AISC 360, *Specification for Structural Steel Buildings*

In Canada: CSA S16, *Design of Steel Structures*

(3) For Screw Connections:

Published manufacturers' technical reports and catalogs

(4) For Power-Actuated Fastener Connections:

Published manufacturers' technical reports and catalogs

(b) Cold-Formed Steel Attached to Aluminum

(1) For Bolted or Screw Connections:

In the U.S. and Mexico: ADM1, *Aluminum Design Manual: Part 1—Specification for Aluminum Structures*

In Canada: CSA S157, *Strength Design in Aluminum*

(c) Cold-Formed Steel Attached to Concrete

(1) For Post-Installed Anchors and Cast-in-Place Anchors:

In the U.S. and Mexico: ACI 318, *Building Code Requirements for Structural Concrete*

In Canada: CSA A23.3, *Design of Concrete Structures*

Published manufacturers' technical reports and catalogs

(2) For Power-Actuated Fasteners:

Published manufacturers' technical reports and catalogs

(d) Cold-Formed Steel Attached to Masonry

(1) For Cast-in-Place Bolts:

In the U.S. and Mexico: TMS 402/ACI 530/ASCE 5, *Building Code Requirements for Masonry Structures*

In Canada: CSA S370, *Connectors for Masonry*

(2) For Power-Actuated Fasteners and Other Post-Installed Anchors:

Published manufacturers' technical reports and catalogs

(e) Cold-Formed Steel Attached to Wood

(1) For Bolt or Screw Connections:

In the U.S. and Mexico: ANSI/AWC NDS, *National Design Specification (NDS) for Wood Construction*

In Canada: CSA O86, *Engineering Design in Wood*

Published manufacturers' technical reports and catalogs

(f) Cold-Formed Steel Attached to Plywood

(1) For Screw Connections:

APA Technical Note E830E, *Fastener Loads for Plywood-Screws*

J7.1.1 Bearing

The design provisions for the *nominal bearing strength* [*resistance*] on the other materials should be derived from appropriate material specifications.

J7.1.2 Tension

This section is included in the *Specification* to raise the awareness of the design engineer regarding tension on fasteners and the connected parts.

J7.1.3 Shear

This section is included in the *Specification* to raise the awareness of the design engineer regarding the transfer of shear forces from steel components to adjacent components of other materials.

K. STRENGTH FOR SPECIAL CASES

K1 Test Standards

Specification Section K1 lists standards developed for testing cold-formed steel elements, *connections*, or assemblies. Commentaries are provided along with the test standards as needed.

K2 Tests for Special Cases

All tests for: (1) the determination and confirmation of structural performance, and (2) the determination of mechanical properties must be made by an independent testing laboratory or by a manufacturer's testing laboratory. The design and testing of cold-formed steel *diaphragms* should be in accordance with the standards specified in *Specification* Section I2. Accordingly, the statement that the provisions in *Specification* Section K2 do not apply to cold-formed steel *diaphragms* was deleted in 2016.

K2.1 Tests for Determining Structural Performance

This *Specification* section contains provisions for proof of structural adequacy by *load* tests. This section is restricted to those cases permitted under Section A1.2.6 of the *Specification* or specifically permitted elsewhere in the *Specification*.

K2.1.1 Load and Resistance Factor Design and Limit States Design

The determination of *load*-carrying capacity of the tested elements, assemblies, *connections*, or members is based on the same procedures used to calibrate the *LRFD* design criteria, for which the ϕ factor can be computed from *Specification* Equation K2.1.1-2 as developed in the *Commentary* as Equation C-B3.2.2-15.

The calibration coefficient, C_ϕ, and coefficient of variation of the *load* effect, V_Q, are dependent on the selected *load* combination and *load* ratio (e.g., dead-to-live *load* ratio). Justification for the selected choices is provided in *Commentary* Sections B3.2.2 and B3.2.3. If the special case being considered deviates significantly from the assumed governing *load* combination (1.2D +1.6L in the United States and 1.25D+1.5L in Canada) or dead-to-live *load* ratio (1:5 in the United States and 1:3 in Canada), then updated values such as those provided in Meimand and Schafer (2014) for C_ϕ and V_Q may be considered. With the exception of earthquake *load* combinations, the constant values for C_ϕ and V_Q that the *Specification* provides were shown to result in ϕ factors within 15 percent of more exact approximations (Meimand and Schafer, 2014).

The correction factor, C_P, is used in *Specification* Equation K2.1.1-2 for determining the ϕ factor to account for the influence due to a small number of tests (Peköz and Hall, 1988b and Tsai, 1992). It should be noted that when the number of tests is large enough, the effect of the correction factor is negligible. In the 1996 edition of the *Specification*, Equation K2.1.1-4 was revised because the old formula for C_P could be unconservative for combinations of a high V_P and a small sample size (Tsai, 1992). This revision enables the reduction of the minimum number of tests from four to three identical specimens. Consequently, the ±10 percent deviation limit was relaxed to ±15 percent. The use of C_P with a minimum V_P reduces the need for this restriction. In *Specification* Equation K2.1.1-4, a numerical value of C_P = 5.7 was found for n = 3 by comparison with a two-parameter method developed by Tsai (1992). It is based on the given value of V_Q and other statistics listed in *Specification* Table

K2.1.1-1, assuming that V_P will be no larger than about 0.20. The requirements of *Specification* Section K2.1.1(a) for n = 3 help to ensure this outcome.

The 0.065 minimum value of V_P, when used in *Specification* Equation K2.1.1-2 for the case of three tests, produces *safety factors* similar to those of the 1986 edition of the AISI *ASD Specification*, i.e., approximately 2.0 for members and 2.5 for *connections*. The *LRFD* calibration reported by Hsiao, Yu and Galambos (1988a) indicates that V_P is almost always greater than 0.065 for common cold-formed steel components, and can sometimes reach values of 0.20 or more. The minimum value for V_P helps to prevent potential unconservatism compared to values of V_P implied in *LRFD* design criteria.

In evaluating the coefficient of variation V_P from test data, care must be taken to use the coefficient of variation for a sample. This can be calculated as follows:

$$V_P = \frac{\sqrt{s^2}}{R_n} \qquad \text{C-K2.1.1-1}$$

where

s^2 = Sample variance of all test results

$$= \frac{1}{n-1}\sum_{i=1}^{n}\left(R_i - R_n\right)^2 \qquad \text{C-K2.1.1-2}$$

R_n = Mean of all test results

R_i = Test result i of n total results

Alternatively, V_P can be calculated as the sample standard deviation of n ratios R_i/R_n.

If the *nominal strength* [*resistance*] is determined in accordance with a *rational engineering analysis* while the *safety* and *resistance factors* are calculated based on tests, the coefficient of variation, V_P, is determined in accordance with *Specification* Equation K2.1.1-6 with P_m determined in accordance with *Specification* Equation K2.1.1-3.

For beams having tension *flange* through-fastened to deck or sheathing and with compression *flange* laterally unbraced (subject to wind uplift), the calibration is based on a *load* combination of 1.17W-0.9D with D/W = 0.1 (see Section I6.2.1 of this *Commentary* for detailed discussion).

The additional statistical data needed for the determination of the *resistance factor* are listed in *Specification* Table K2.1.1-1. The original basis for the Table K2.1.1-1 member statistics is the *LRFD* calibration of Hsiao, Yu and Galambos (1988a). *Connection* statistics are based on Rang, Galambos, and Yu (1979b) and Peköz (1990). Values for *power-actuated fasteners* are based on Mujagic, et al. (2010). The statistical data for *connections* to structural concrete and wood are based on those employed in AISI S310. In 2007, the *Specification* more clearly defined the appropriate material properties that are to be used when evaluating test results by specifying that supplier-provided properties are not to be used.

In 2012, statistical data of M_m, V_M, F_m, and V_F were added for *power-actuated fasteners* to accompany the newly created *Specification* Section J5, based on the study by Mujagic et al. (2010).

Specification Table K2.1.1-1 was simplified and updated in 2016 to reflect current *limit states*, to provide clarity in its use for *rational engineering analysis* and test-based methods, and to reflect the actual accuracy of the selected M_m, V_M, F_m, and V_F statistics.

In 2022, the statistical data for screw shear strength limited by tilting and *bearing* was added back. These statistical data are consistent with those in AISI S100-12, where V_M = 0.08 based on the *tensile strength* statistical data provided in the University of Missouri-Rolla research report (Rang, Galambos, and Yu, 1979b). V_f = 0.05 to reflect the tolerance of the cross-sectional area of the screw.

In 2012, Section K2.1.1(c) was revised to permit the use of mill certificates to establish the mechanical properties of small connectors and devices. As a general practice, the *yield stress*, F_y, is determined by testing a tensile specimen that is either cut from the test specimen, or the steel coil or sheet used to produce the test specimen. However, for some cold-formed steel components such as small hurricane ties and clips, it is often impossible to cut a standard size or sub-size tensile specimen that would meet the requirements of ASTM A370 (ASTM, 2015). Since mill certificate tensile specimens are taken from the lead or tail of the *master coil* which may not be representative of the entire coil, and because coiling and uncoiling operations can alter mechanical properties, it is necessary to reduce M_m. When using mill certificates instead of tensile specimens for a range of 21 coils (Stauffer and McEntee, 2012), it has been shown that using M_m = 0.85 will provide corresponding ϕ and Ω values that are on average 15 percent more conservative. In order to use mill certificates to establish material properties, it is important to maintain proper records and procedures that can trace the connector or device to the *master coil*. The use of mill certificates is not permitted for members. In addition, although mill certificates are routinely used to establish the raw material properties for fasteners such as screws or *power-actuated fasteners*, they should not be used to establish the final material properties. This is because the raw steel undergoes secondary operations such as heat treating that alters its final properties.

In 2012, Section A1.2.6(b) and Section K2.1.1(b) were added as an optional method to calibrate *safety* and *resistance factors* for a proposed strength theory using test data. In order to use this optional method, sufficient correlation must exist between the proposed strength theory and the test data. The correlation coefficient, C_c, used in this section is a statistical measure of the agreement between the strength predictions ($R_{n,i}$) and test results ($R_{t,i}$):

$$C_c = \frac{n\sum R_{t,i}R_{n,i} - \left(\sum R_{t,i}\right)\left(\sum R_{n,i}\right)}{\sqrt{n\left(\sum R_{t,i}^2\right) - \left(\sum R_{t,i}\right)^2}\sqrt{n\left(\sum R_{n,i}^2\right) - \left(\sum R_{n,i}\right)^2}} \quad \text{(C-K2.1.1-3)}$$

where

$R_{t,i}$ = Tested *strength* [*resistance*], corresponding to test i

$R_{n,i}$ = Predicted *nominal strength* [*resistance*], corresponding to test i.

The value of the correlation coefficient reveals information about the potential quality of the proposed strength theory, namely:

(1) High or moderately high positive correlation indicates that the theory and tests either agree substantially as they are, or can be brought into good agreement by using a constant factor. This means that bias factor, P_m, will compensate for the bias, as intended, in the calibration procedure to determine the *resistance factor*.

(2) Low or nearly zero correlation is an indicator of independence; in other words, no relationship between the tests and theory can be discerned. Using the theory will produce bad results and it should be rejected.

(3) Negative correlation indicates that the theory and test data not only disagree but actually have opposite relationships. For example, when the theory says the strength increases, it

actually decreases. Using the theory will produce bad results and it should be rejected.

The square of the correlation coefficient is referred to as the coefficient of determination. It gives the proportion of the variance (fluctuation) of one variable (tested *strength* [*resistance*]) that is predicted by the other variable (strength theory). For example, for $C_c^2 = (0.8)^2$, 64 percent of the variance is accounted for by the theory. Alternative values for the minimum correlation coefficient could be used, but values above $C_c = 0.707$ have the desirable characteristic that $C_c^2 \geq 0.5$; that is, more than 50 percent of the variance is explained by the theory.

In general, higher values of the correlation coefficient are desirable, and indicate a better agreement with the theory, lower V_P, and a better result for the product of the *resistance factor* times the *nominal strength* [*resistance*] given by the theory.

Another advantage of a correlation coefficient criterion is that it is less restrictive and easier to satisfy than alternative criteria based on individual deviations, such as a 15 percent deviation restriction. C_c is obtained from the full data set and does not apply to individual values. Also, there are multiple ways to obtain a good correlation coefficient. For example, if the test data and strength theory differ by a constant factor; i.e., they are proportional, one will still get a correlation coefficient of 1.0, as if they had agreed directly. This advantage also holds for moderately high correlation coefficients. As mentioned above, this will improve the effectiveness of bias factor, P_m, and the *resistance factor*.

It is important that users not only test at the upper and lower bounds of the desired parameter range, but that even coverage of tests is provided throughout the range. This is emphasized in the *Specification* in order to ensure that potential minima or maxima within the test range are detected and that the *resistance factor* and *safety factor* calibrated using the test data properly reflect any variation from the minima/maxima.

The *Specification* provides methods for determining the deflection of some members for serviceability consideration, but the *Specification* does not provide *serviceability limits*. Justification is discussed in Section B3.7 of the *Commentary*.

K2.1.2 Allowable Strength Design

The equation for the *safety factor*, Ω (*Specification* Equation K2.1.2-2), converts the *resistance factor*, ϕ, from *LRFD* test procedures in *Specification* Section K2.1.1 to an equivalent *safety factor* for the A*llowable Strength Design*. The average of the test results, R_n, is then divided by the *safety factor* to determine an *allowable strength*. It should be noted that *Specification* Equation K2.1.2-2 is identical with Equation C-B3.2.2-16 for D/L = 0.

K2.2 Tests for Confirming Structural Performance

Members, *connections* and assemblies that can be designed according to the provisions of Chapters A through J, L, and M of the *Specification* need no confirmation of calculated results by test. However, special situations may arise where it is desirable to confirm the results of calculations by test. Tests may be called for by the manufacturer, the engineer, or a third party.

Since design is in accordance with the *Specification*, all that is needed is for the tested specimen or assembly to demonstrate that the strength is not less than the applicable *nominal resistance*, R_n.

K2.3 Tests for Determining Mechanical Properties

K2.3.1 Full Section

Explicit methods for utilizing the effects of cold work are incorporated in Section A3.3.2 of the *Specification*. In that section, it is specified that as-formed mechanical properties, in particular the *yield stress*, can be determined either by full-section tests or by calculating the strength of the corners and computing the weighted average for the strength of corners and flats. The strength of flats can be taken as the virgin strength of the steel before forming, or can be determined by special tension tests on specimens cut from flat portions of the formed cross-section. This *Specification* section spells out in considerable detail the types and methods of these tests, and their number as required for use in *connection* with *Specification* Section A3.3.2. For details of testing procedures which have been used for such purposes, but which in no way should be regarded as mandatory, see *Specification* (1968), Chajes, Britvec and Winter (1963), and Karren (1967). AISI S902, *Stub-Column Test Method for Effective Area of Cold-Formed Steel Columns,* provides testing procedures (AISI, 2013c).

K2.3.2 Flat Elements of Formed Sections

Specification Section K2.3.2 provides the basic requirements for determining the mechanical properties of flat elements of formed sections. These tested properties are to be used in *Specification* Section A3.3.2 for calculating the average *yield stress* of the formed section by considering the strength increase from cold work of forming.

K2.3.3 Virgin Steel

For steels other than the ASTM Specifications listed in *Specification* Section A3.1, the tensile properties of the *virgin steel* used for calculating the increased *yield stress* of the formed section should also be determined in accordance with the Standard Methods of ASTM A370 (2015).

L. DESIGN FOR SERVICEABILITY

Reduced stiffness values used in the *direct analysis method*, described in Chapter C, are not intended for use with the provisions of this chapter.

L1 Serviceability Determination for Effective Width Method

The effective moment of inertia is calculated based on the reduced cross-section at the *service load* level. Examples are provided in the *Cold-Formed Steel Design Manual* (AISI, 2017).

L2 Serviceability Determination for Direct Strength Method

The provisions of this section use a simplified approach to deflection calculations that assume the moment of inertia of the section for deflection calculations is linearly proportional to the strength of the section, determined at the allowable *stress* of interest. This approximation avoids lengthy effective section calculations for deflection determination.

L3 Flange Curling

In beams which have unusually wide and thin, but stable *flanges* (i.e., primarily tension *flanges* with large w/t ratios), there is a tendency for these *flanges* to curl under bending. That is, the portions of these *flanges* most remote from the *web* (edges of I-beams, center portions of *flanges* of box or hat beams) tend to deflect toward the neutral axis. An approximate, analytical treatment of this problem was given by Winter (1948b). Equation L3-1 of the *Specification* permits one to compute the maximum permissible *flange* width, w_f, for a given amount of *flange* curling, c_f. The equation has been shown to be conservative when compared with more recent experimental data and more exact analytical expressions for predicting flange curling are now available (Lecce and Rasmussen, 2008, 2009).

It should be noted that Section L3 does not stipulate the amount of curling which can be regarded as tolerable, but an amount of curling in the order of 5 percent of the depth of the section is not excessive under usual conditions. In general, *flange* curling is not a critical factor to govern the *flange* width. However, when the appearance of the section is important, the out-of-plane distortion should be closely controlled in practice. An example in the AISI *Cold-Formed Steel Design Manual* (AISI, 2017) illustrates the design consideration for *flange* curling.

M. DESIGN FOR FATIGUE

Fatigue in a cold-formed steel member or *connection* is the process of initiation and subsequent growth of a crack under the action of a cyclic or repetitive *load*. The *fatigue* process commonly occurs at a *stress* level less than the static failure condition.

When *fatigue* is a design consideration, its severity is determined primarily by three factors: (1) the number of cycles of loading, (2) the type of member and *connection* detail, and (3) the *stress* range at the detail under consideration (Fisher et al., 1998).

Fluctuation in *stress*, which does not involve tensile *stress*, does not cause crack propagation and is not considered to be a *fatigue* situation.

When fabrication details involving more than one category occur at the same location in a member, the design *stress* range at the location must be limited to that of the most restrictive category. By locating notch-producing fabrication details in regions subject to a small range of *stress*, the need for a member larger than required by static loading will often be eliminated.

For axially stressed angle members, the *Specification* allows the effects of eccentricity on the weld group to be ignored provided the weld lengths L_1 and L_2 are proportional such that the centroid of the weld group falls between "$\bar{x}$" and "b/2" in Figure C-M-1(a). When the weld lengths L_1 and L_2 are so proportioned, the effects of eccentric *loads* causing moment about x-x in Figure C-M-1(b) also need not be considered.

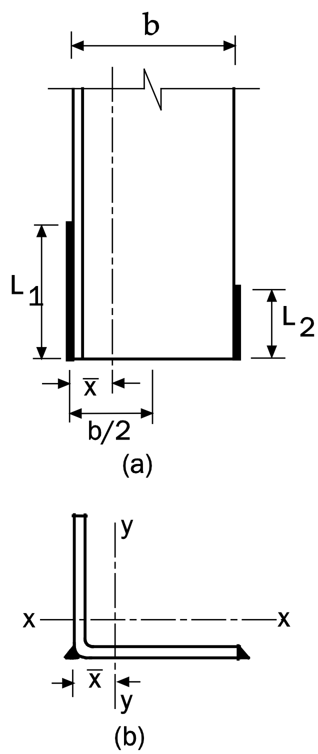


Figure C-M-1 Welded Angle Members

Research by Barsom et al. (1980) and Klippstein (1980, 1981, 1985, 1988) developed *fatigue* information on the behavior of sheet and plate steel weldments and mechanical *connections*. Although research indicates that the values of F_y and F_u do not influence *fatigue* behavior, the *Specification* provisions are based on tests using ASTM A715 (Grade 80), ASTM A607 Grade 60, and SAE 1008 (F_y = 30 ksi). Using regression analysis, mean *fatigue* life curves (S-N curves) with the corresponding standard deviation were developed. The *fatigue* resistance S-N curve has been expressed as an exponential relationship between *stress* range and life cycle (Fisher et al, 1970). The general relationship is often plotted as a linear log-log function, Equation C-M-1.

$$\log N = C_f - m \log F_{SR} \qquad \text{(C-M-1)}$$

$$C_f = B - (n\,s) \qquad \text{(C-M-2)}$$

where

N = number of full *stress* cycles

m = slope of the mean *fatigue* analysis curve

F_{SR} = effective *stress* range

B = intercept of the mean *fatigue* analysis curve from Table C-M-1

n = number of standard deviations to obtain a desired confidence level
= 2 for C_f given in Table M1-1 of the *Specification*

s = approximate standard deviation of the *fatigue* data
= 0.25 (Klippstein, 1988)

The database for these design provisions is based upon cyclic testing of real *joints*; therefore, *stress* concentrations have been accounted for by the categories in Table M1-1 of the *Specification*. It is not intended that the allowable *stress* ranges should be compared to “hot-spot” *stresses* determined by finite element analysis. Also, calculated *stresses* computed by ordinary analysis need not be amplified by stress concentration factors at geometrical discontinuities and changes of cross-section. All categories were found to have a common slope with m = -3. Equation M3-1 of the *Specification* is to be used to calculate the design *stress* range for the chosen design life, N. Table M1 of the *Specification* provides a classification system for the various *stress* categories. This also provides the constant, C_f, that is applicable to the *stress* category that is required for calculating design *stress* range, F_{SR}.

Table C-M-1 Intercept for Mean Fatigue Curves

Stress Category	b
I	11.0
II	10.5
III	10.0
IV	9.5

The provisions for bolts and threaded parts were taken from the AISC Specification (AISC, 1999).

In 2024, the requirement for the length of a longitudinal fillet-weld attachment not to exceed 4 in. (101.6 mm) was added in the last row of *Specification* Table M1-1 to be consistent with the requirements in Figure M1-3(b).

APPENDIX 1, EFFECTIVE WIDTH OF ELEMENTS

In cold-formed steel construction, individual elements of steel structural members are thin and the width-to-*thickness* ratios are large as compared to hot-rolled steel shapes. These thin elements may buckle locally at a *stress* level lower than the *yield stress* of steel when they are subjected to compression in flexural bending, axial compression, shear, or *bearing*. Figure C-1-1 illustrates some *local buckling* patterns of certain beams and columns (Yu and LaBoube, 2010).

Because *local buckling* of individual elements of cold-formed steel sections is a major design criterion, the design of such members should provide sufficient safety against the failure by local *instability* with due consideration given to the post-*buckling* strength of *structural components*. Section B4.1 and Appendix 1 of the *Specification* contain the design requirements for width-to-*thickness* ratios and the design equations for determining the *effective widths* of stiffened compression elements, *unstiffened compression elements*, elements with edge stiffeners or intermediate stiffeners, and beam *webs*. The design provisions are provided for the use of stiffeners in *Specification* Section G7 for flexural members.

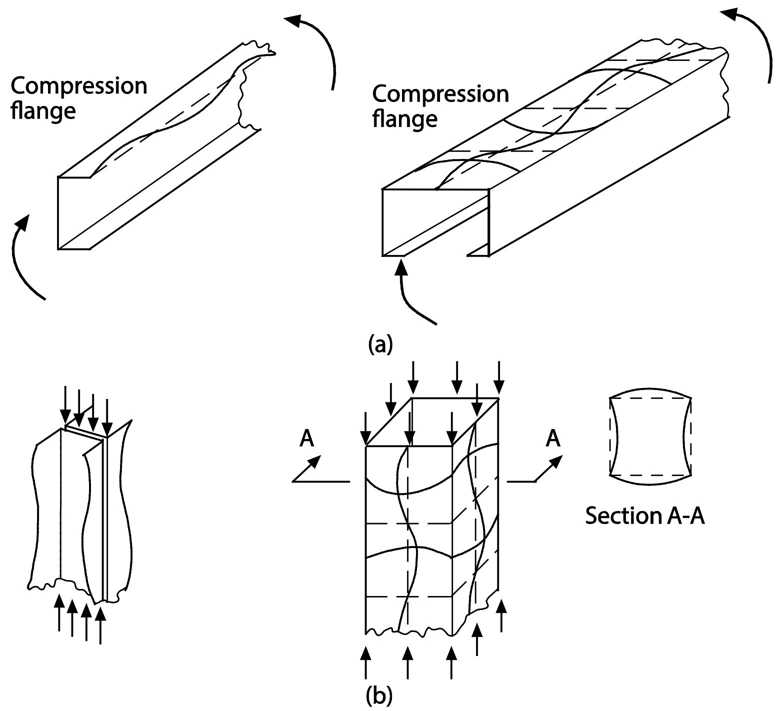


Figure C-1-1 Local Buckling of Compression Elements:
(a) Beams, (b) Columns

It is well known that the structural behavior and the *load*-carrying capacity of a stiffened compression element such as the compression *flange* of a hat section depend on the w/t ratio and the supporting condition along both longitudinal edges. If the w/t ratio is small, the *stress* in the compression *flange* can reach the *yield stress* of steel and the strength of the compression element is governed by yielding. For the compression *flange* with large w/t ratios, *local buckling* (Figure C-1-2) will occur at the following elastic critical *buckling stress*:

$$f_{cr} = \frac{k\pi^2 E}{12(1-\mu^2)(w/t)^2} \tag{C-1-1}$$

where
k = Plate *buckling* coefficient (Table C-1-1)
 = 4 for stiffened compression elements supported by a *web* on each longitudinal edge
E = Modulus of elasticity of steel
μ = Poisson's ratio = 0.3 for steel in the elastic range
w = *Flat width* of the compression element
t = *Thickness* of the compression element

When the elastic critical *buckling stress* computed according to Equation C-1-1 exceeds the proportional limit of the steel, the compression element will buckle in the inelastic range (Yu and LaBoube, 2010).

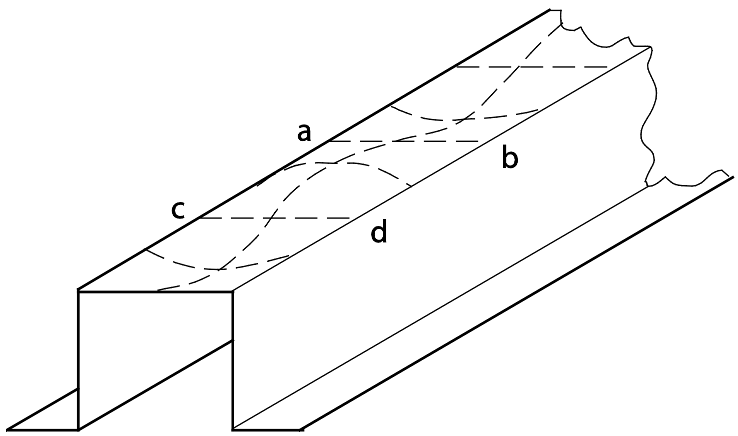


Figure C-1-2 Local Buckling of Stiffened Compression Flange of Hat-Shaped Beam

Unlike one-dimensional structural members such as columns, stiffened compression elements will not collapse when the *buckling stress* is reached. An additional *load* can be carried by the element after *buckling* by means of a redistribution of *stress*. This phenomenon is known as post-*buckling* strength of the compression elements and is most pronounced for stiffened compression elements with large w/t ratios. The mechanism of the post-*buckling* action of compression elements was discussed by Winter in previous editions of the *Commentary* (Winter, 1970).

Imagine for simplicity a square plate uniformly compressed in one direction, with the unloaded edges simply supported. Since it is difficult to visualize the performance of such two-dimensional elements, the plate will be replaced by a model which is shown in Figure C-1-3. It consists of a grid of longitudinal and transverse bars in which the material of the actual plate is thought to be concentrated. Since the plate is uniformly compressed, each of the longitudinal struts represents a column loaded by P/5, if P is the total *load* on the plate. As the *load* is gradually increased, the compression *stress* in each of these struts will reach the critical column *buckling* value and all five struts will tend to buckle simultaneously. If these struts were simple columns, unsupported except at the ends, they would simultaneously collapse through unrestrained increasing lateral deflection. It is evident that this cannot occur in the grid model of the plate. Indeed, as soon as the longitudinal struts start deflecting at their *buckling stresses*, the transverse bars, which are connected to them, must stretch like ties in order to accommodate the imposed deflection. Like any structural material, they resist stretch and, thereby, have a restraining effect on the deflections of the longitudinal struts.

Table C-1-1
Values of Plate Buckling Coefficients

Case	Boundary Condition	Type of Stress	Value of k for Long Plate
(a)	s.s. s.s. s.s. s.s.	Compression	4.0
(b)	Fixed s.s. s.s. Fixed	Compression	6.97
(c)	s.s. s.s. s.s. Free	Compression	0.425
(d)	Fixed s.s. s.s. Free	Compression	1.277
(e)	Fixed s.s. s.s. s.s.	Compression	5.42
(f)	s.s. s.s. s.s. s.s.	Shear	5.34
(g)	Fixed Fixed Fixed Fixed	Shear	8.98
(h)	s.s. s.s. s.s. s.s.	Bending	23.9
(i)	Fixed Fixed Fixed Fixed	Bending	41.8

The tension forces in the horizontal bars of the grid model correspond to the so-called membrane *stresses* in a real plate. These *stresses*, just as in the grid model, come into play as soon as the compression *stresses* begin to cause *buckling* waves. They consist mostly of transverse tension, but also of some shear *stresses*, and they counteract increasing wave deflections, i.e., they tend to stabilize the plate against further *buckling* under the applied increasing longitudinal compression. Hence, the resulting behavior of the model is as follows: (a) there is no collapse by unrestrained deflections, as in unsupported columns, and (b) the various struts will deflect unequal amounts—those nearest the supported edges being held almost straight by the ties, and those nearest the center being able to deflect most.

In consequence of (a), the model will not collapse and fail when its *buckling stress* (Equation C-1-1) is reached; in contrast to columns, it will merely develop slight deflections but will continue to carry increasing *load*. In consequence of (b), the struts (strips of the plate) closest to the center, which deflect most, "get away from the *load*," and hardly participate in carrying any further *load* increases. These center strips may, in fact, even transfer part of their pre-*buckling* load to their neighbors. The struts (or strips) closest to the edges, held straight by the ties, continue to resist increasing *load* with hardly any increasing deflection. For the plate, this means that the hitherto uniformly distributed compression *stress* redistributes itself in a manner shown in Figure C-1-4, with the *stresses* being largest at the edges and smallest in the center. With further increase in *load,* this nonuniformity increases, as also shown in Figure C-1-4. The plate fails, i.e., refuses to carry any further *load* increases, only when the most highly stressed strips near the supported edges begin to yield, i.e., when the compression *stress* f_{max} reaches the *yield stress* F_y.

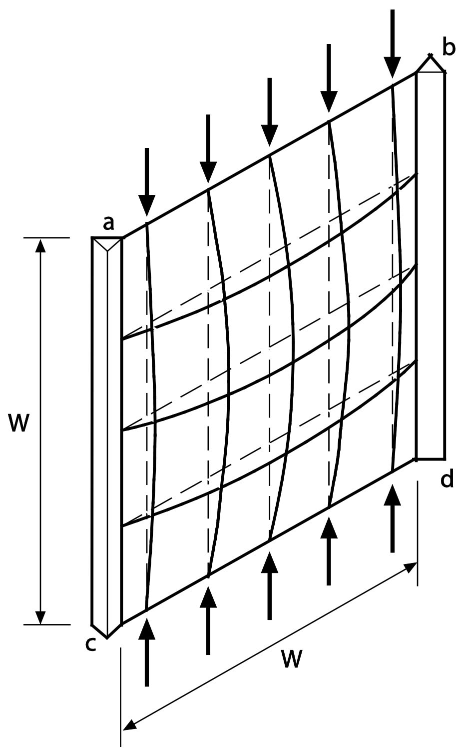


Figure C-1-3 Post-Buckling Strength [Resistance] Model

This post-*buckling* strength of plates was discovered experimentally in 1928, and an approximate theory of it was first given by Th. v. Karman in 1932 (Bleich, 1952). It has been used in aircraft design ever since. A graphic illustration of the phenomenon of *post-buckling* strength can be found in the series of photographs in Figure 7 of Winter (1959b).

The model of Figure C-1-3 is representative of the behavior of a compression element supported along both longitudinal edges, as the *flange* in Figure C-1-2. In fact, such elements buckle into approximately square waves.

In order to utilize the post-*buckling* strength of the stiffened compression element for design purposes, the *Specification* has used the *effective design width* approach to determine the sectional properties since 1946. In Appendix 1 of the *Specification*, design equations for computing the *effective widths* are provided for the following cases: (1) uniformly compressed stiffened elements, (2) uniformly compressed stiffened elements with circular or noncircular holes, (3) *webs* and other stiffened elements with *stress* gradient, (4) unstiffened elements with uniform or gradient *stress*, and (5) C-section *webs* with holes under *stress* gradient. The background information on various design requirements is discussed in subsequent sections.

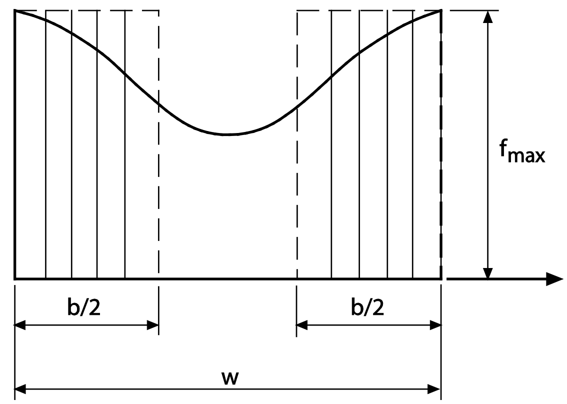


Figure C-1-4 Stress Distribution in Stiffened Compression Elements

1.1 Effective Width of Uniformly Compressed Stiffened Elements

(a) Strength Determination

In the *effective design width* approach, instead of considering the nonuniform distribution of *stress* over the entire width of the plate w, it is assumed that the total *load* is carried by a fictitious *effective width b*, subject to a uniformly distributed *stress* equal to the edge *stress* f_{max}, as shown in Figure C-1-4. The width b is selected so that the area under the curve of the actual nonuniform *stress* distribution is equal to the sum of the two parts of the equivalent rectangular shaded area with a total width b and an intensity of *stress* equal to the edge *stress* f_{max}.

Based on the concept of *effective width* introduced by von Karman et al. (von Karman, Sechler and Donnell, 1932) and the extensive investigation on light-gage, cold-formed steel sections at Cornell University, the following equation was developed by Winter in 1946 for determining the *effective width* b for stiffened compression elements simply supported along both longitudinal edges:

$$b = 1.9t\sqrt{\frac{E}{f_{max}}}\left[1 - 0.475\left(\frac{t}{w}\right)\sqrt{\frac{E}{f_{max}}}\right] \qquad \text{(C-1.1-1)}$$

The above equation can be written in terms of the ratio of F_{cr}/f_{max} as follows:

$$\frac{b}{w} = \sqrt{\frac{F_{cr}}{f_{max}}}\left(1 - 0.25\sqrt{\frac{F_{cr}}{f_{max}}}\right) \qquad \text{(C-1.1-2)}$$

where F_{cr} is the critical elastic *buckling stress* of a plate, and is expressed in Equation C-1-1.

Thus, the *effective width* expression (e.g., Equation C-1.1-1) provides a prediction of the *nominal strength* [*resistance*] based only on the critical elastic *buckling stress* and the applied *stress* of the plate. During the period from 1946 to 1968, the *Specification* design provision for the determination of the *effective design width* was based on Equation C-1.1-1. Accumulated experience has demonstrated that a more realistic equation as shown below may be used for the determination of the *effective width* b (Winter, 1970):

$$b = 1.9t\sqrt{\frac{E}{f_{max}}}\left[1 - 0.415\left(\frac{t}{w}\right)\sqrt{\frac{E}{f_{max}}}\right] \qquad \text{(C-1.1-3)}$$

The correlation between the test data on stiffened compression elements and Equation C-1.1-3 is illustrated by Yu and LaBoube (2010).

It should be noted that Equation C-1.1-3 may also be rewritten in terms of the F_{cr}/f_{max} ratio as follows:

$$\frac{b}{w} = \sqrt{\frac{F_{cr}}{f_{max}}}\left(1 - 0.22\sqrt{\frac{F_{cr}}{f_{max}}}\right) \quad \text{(C-1.1-4)}$$

Therefore, the *effective width*, b, can be determined as

$$b = \rho w \quad \text{(C-1.1-5)}$$

where ρ = reduction factor

$$= (1 - 0.22/\sqrt{f_{max}/F_{cr}})/\sqrt{f_{max}/F_{cr}} = (1 - 0.22/\lambda)/\lambda \le 1 \quad \text{(C-1.1-6)}$$

In Equation C-1.1-6, λ is a slenderness factor determined below.

$$\lambda = \sqrt{f_{max}/F_{cr}} \quad \text{(C-1.1-7)}$$

Figure C-1.1-1 shows the relationship between ρ and λ. It can be seen that when $\lambda \le 0.673$, $\rho = 1.0$.

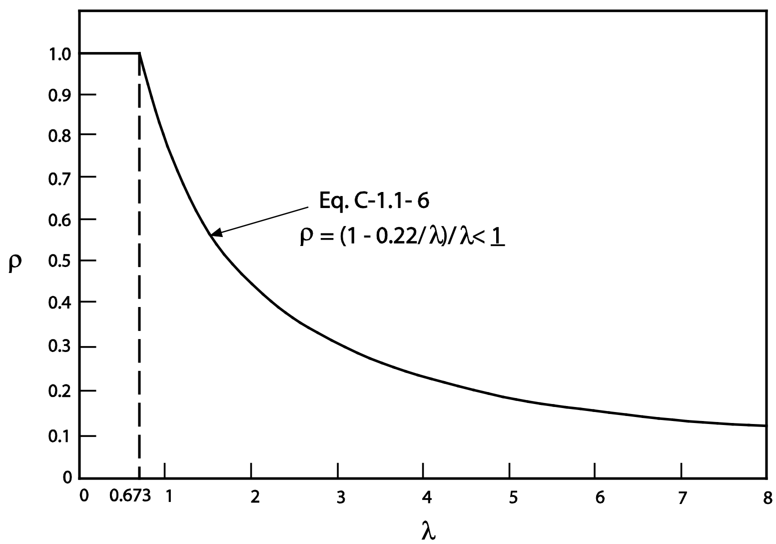


Figure C-1.1-1 Reduction Factor, ρ, vs. Slenderness Factor, λ

Based on Equations C-1.1-5 through C-1.1-7 and the unified approach proposed by Peköz (1986b and 1986c), the 1986 edition of the *Specification* adopted the non-dimensional format in Section 1.1 for determining the *effective design width*, b, for uniformly compressed stiffened elements. The same design equations were used in the 1996 edition of the *Specification* and were retained in this edition of the *Specification*. For design examples, see Part I of the AISI *Cold-Formed Steel Design Manual* (AISI, 2017).

(b) Serviceability Determination

The *effective design width* equations discussed above for strength determination can also be used to obtain a conservative *effective width*, b_d, for serviceability determination. It is included in Section 1.1(b) of the *Specification* as Procedure I.

For stiffened compression elements supported by a *web* on each longitudinal edge, a study conducted by Weng and Peköz (1986) indicated that Equations 1.1-6 through 1.1-8 of the

Specification can yield a *more accurate estimate* of the *effective width*, b_d, for serviceability. These equations are given in Procedure II for additional design information. The design engineer has the option of using either of the two procedures for determining the *effective width* to be used for serviceability determination.

1.1.1 Uniformly Compressed Stiffened Elements With Circular or Noncircular Holes

In *cold-formed steel structural members*, holes are sometimes provided in *webs* and/or *flanges* of beams and columns for duct work, piping, and other construction purposes. The presence of such holes may result in a reduction of the strength of individual component elements and the overall strength and stiffness of the members depending on the size, shape, and arrangement of holes, the geometric configuration of the cross-section, and the mechanical properties of the material.

The exact analysis and the design of steel sections having perforations are complex, particularly when the shapes and the arrangement of holes are unusual. The limited design provisions included in Section 1.1.1 of the *Specification* for uniformly compressed stiffened elements with circular holes are based on a study conducted by Ortiz-Colberg and Peköz at Cornell University (Ortiz-Colberg and Peköz, 1981). For additional information on the structural behavior of perforated elements, see Yu and Davis (1973a) and Yu and LaBoube (2010).

In 2004, *Specification* Equation 1.1.1-2 was revised to provide continuity at $\lambda = 0.673$.

Within the limitations stated for the size and spacing of perforations and section depth, the provisions were deemed appropriate for members with uniformly compressed stiffened elements, not just wall studs. The validity of this approach for C-section wall studs was verified in a Cornell University project on wall studs reported by Miller and Peköz (1989 and 1994). The limitations included in *Specification* Section 1.1.1 for the size and spacing of perforations and the depth of studs are based on the parameters used in the test program. Although Figure 1.1.1-1 in the *Specification* shows a hole centered within the *flat width*, w, holes not centered within w are allowed. For such a case, the unstiffened strip, c, and resulting *effective width*, b, must be calculated separately for the strips on each side of the hole. For sections with perforations which do not meet these limits, the *effective area*, A_e, can be determined by stub column tests.

The geometric limitations (w/t, etc.) and hole size for the circular and noncircular hole provisions in *Specification* Section 1.1.1 are not consistent with one another. This anomaly in the limitations is due to the differing scopes of the test programs that serve as the basis for these *effective width* equations. The provisions for noncircular holes generally give a more conservative prediction of the *effective width* than the provisions for circular holes, as long as $d_h/w < 0.4$. Provisions for designing perforated members using the *Direct Strength Method* (*DSM*) can be found in *Specification* Chapters E and F, and Appendix 2.

1.1.2 Webs and Other Stiffened Elements Under Stress Gradient

When a beam is subjected to bending moment, the compression portion of the *web* may buckle due to the compressive *stress* caused by bending. The theoretical critical *buckling stress* for a flat rectangular plate under pure bending can be determined by Equation C-1-1, except that the depth-to-*thickness* ratio, h/t, is substituted for the width-to-*thickness* ratio, w/t, and the plate *buckling* coefficient, k, is equal to 23.9 for simple supports as listed in Table C-1-1.

Prior to 1986, the design of cold-formed steel beam *webs* was based on the full *web* depth with the allowable bending *stress* specified in the *Specification*. In order to unify the design methods for *web* elements and compression *flanges*, the effective design depth approach was adopted in the 1986 edition of the *Specification* on the basis of the studies made by Peköz (1986b), Cohen and Peköz (1987). This is a different approach as compared with the past practice of using a full area of the *web* element in conjunction with a reduced *stress* to account for *local buckling* and post-*buckling* strength (LaBoube and Yu, 1982b; Yu, 1985).

Prior to 2001, the b_1 and b_2 expressions used in the *Specification* for the *effective width* of *webs* (Equations 1.1.2-3 through 1.1.2-5) implicitly assumed that the *flange* provided beneficial restraint to the *web*. Collected data (Cohen and Peköz (1987), Elhouar and Murray (1985), Ellifritt et al. (1997), Hancock et al (1996), LaBoube and Yu (1978), Moreyra and Peköz (1993), Rogers and Schuster (1995), Schardt and Schrade (1982), Schuster (1992), Shan et al. (1994), and Willis and Wallace (1990) as summarized in Schafer and Peköz (1999)) on flexural tests of C- and Z-sections indicate that *Specification* Equations 1.1.2-3 through 1.1.2-5 can be unconservative if the overall *web* width (h_o) to overall *flange* width (b_o) ratio exceeds 4. Consequently, in 2001, in the absence of a comprehensive method for handling local *web* and *flange* interaction, the *Specification* adopted a two-part approach for the *effective width* of *webs*: an additional set of alternative expressions (Equations 1.1.2-6 and 1.1.2-7), originally developed by Cohen and Peköz (1987), were adopted for $h_o/b_o > 4$; while the expressions adopted in the 1986 edition of the *Specification* (Equations 1.1.2-3 through 1.1.2-5) remain for $h_o/b_o \le 4$. For flexural members with *local buckling* in the *web*, the effect of these changes is that the strengths will be somewhat lower when $h_o/b_o > 4$ compared with the 1996 *Specification* (AISI, 1996). When compared with the CSA S136 (CSA, 1994) *Standard*, there are only minor changes for members with $h_o/b_o > 4$, but an increase in strength will be experienced when $h_o/b_o \le 4$.

It should be noted that in the *Specification*, the *stress* ratio, ψ, is defined as an absolute value. As a result, some signs for ψ have been changed in *Specification* Equations 1.1.2-2, 1.1.2-3, 1.1.2-6 and 1.1.2-7 as compared with the 1996 edition of the *Specification* (AISI, 1996).

1.1.3 C-Section Webs With Holes Under Stress Gradient

Studies of the behavior of *web* elements with holes conducted at the University of Missouri-Rolla (UMR) serve as the basis for the design recommendations for bending alone, shear, *web crippling*, combinations of bending and shear, and bending and *web crippling* (Shan et al., 1994; Langan et al., 1994; Uphoff, 1996; Deshmukh, 1996). The *Specification* considers a hole to be any flat-punched opening in the *web* without any edge-stiffened openings.

The UMR design recommendations for a perforated *web* with *stress* gradient are based on the tests of full-scale C-section beams having h/t ratios as large as 200 and d_h/h ratios as large as 0.74. The test program considered only stud and joist industry standard *web* holes. These holes were rectangular with fillet corners, punched during the rolling process. For noncircular holes, the corner radii recommendation was adopted to avoid the potential of high *stress* concentration at the corners of a hole. *Webs* with circular holes and a *stress* gradient were not tested; however, the provisions are conservatively extended to cover this case. Other shaped holes must be evaluated by the virtual hole method described below, by test, or by other provisions of the *Specification*. The *Specification* is not intended to cover cross-sections having repetitive 1/2-in. diameter holes.

Based on the study by Shan et al. (1994), it was determined that the *nominal bending strength* [*resistance*] of a C-section with a *web* hole is unaffected when $d_h/h < 0.38$. For situations where the $d_h/h \geq 0.38$, the effective depth of the *web* can be determined by treating the flat portion of the remaining *web* that is in compression as an unstiffened compression element.

Although these provisions are based on tests of singly-symmetric C-sections having the *web* hole centered at mid-depth of the section, the provisions may be conservatively applied to sections for which the full unreduced compression region of the *web* is less than the tension region. However, for cross-sections having a compression region greater than the tension region, the *web* strength must be determined by test in accordance with Section K2.1.

The provisions for circular and noncircular holes also apply to any hole pattern that fits within an equivalent virtual hole. For example, Figure C-1.1.3-1 illustrates the L_h and d_h that may be used for a multiple-hole pattern that fits within a noncircular virtual hole. Figure C-1.1.3-2 illustrates the d_h that may be used for a rectangular hole that exceeds the 2.5 in. (64 mm) by 4.5 in. (114 mm) limit but still fits within an allowed circular virtual hole. For each case, the design provisions apply to the geometry of the virtual hole, not the actual hole or holes.

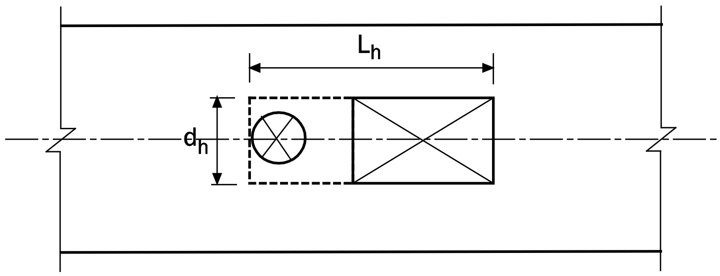


Figure C-1.1.3-1 Virtual Hole Method for Multiple Openings

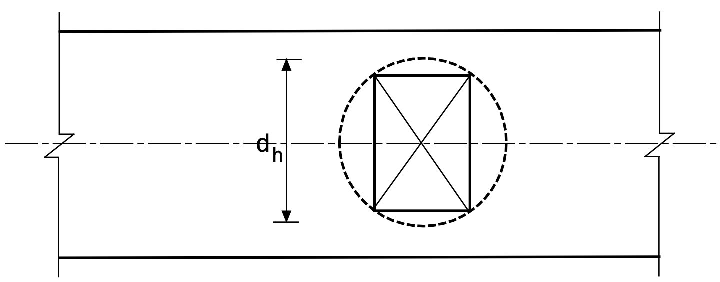


Figure C-1.1.3-2 Virtual Hole Method for Opening Exceeding Limit

The effects of holes on shear strength and *web crippling* strength of C-section *webs* are discussed in Sections G3 and G6 of the *Commentary*, respectively.

To consider the *effective width* of the compression element adjacent to the hole, the *Specification* ignored the *stress* variation and considered the compression element under a uniform compression with the *stress* level of f_1 as shown in *Specification* Figure 1.1.2-1. In 2022, the design provisions in *Specification* Section 1.2.2 were adopted to consider the *web stress* variation. For common C-sections, this resulted in up to a two percent strength increase.

1.1.4 Uniformly Compressed Elements Restrained by Intermittent Connections

Specification Section I1.3 limits the spacing of *connections* in compression elements so that the strength of the section is fully developed before *buckling* occurs between *connections*. In practice this limitation is often exceeded. Luttrell and Balaji (1992) and Snow and Easterling

(2008) developed a method to determine the *effective width* of compression elements when greater *connection* spacing exists. The design provisions in *Specification* Section 1.1.4 were based on the research work by Snow and Easterling (2008) with 82 standard roof deck tests. All test specimens had multiple flutes and the depth range was between 1-½ in. (38.1 mm) and 7-½ in. (191 mm). As shown in Figures C-1.1.4-1 and C-1.1.4-2, all test compression plates had edge stiffeners.

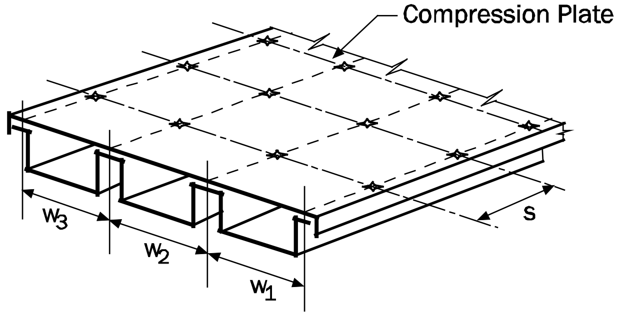


Figure C-1.1.4-1 Built-Up Deck

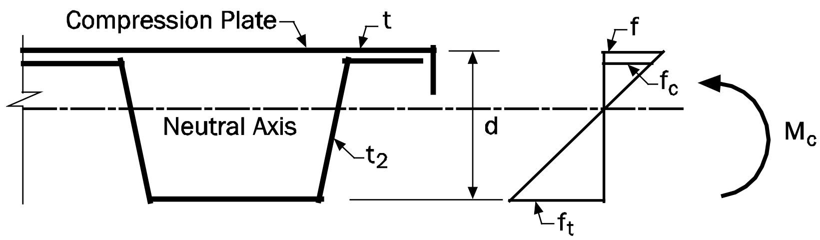


Figure C-1.1.4-2 Built-Up Deck in Bending

The full *stress* potential of the "built-up" section is determined by recognizing the post-*buckling* strength of the compression plate after local waves form between *connections*. The method models an equivalent composite transformed section and maintains the classical assumption of linear strain distribution. The critical compression *stress*, F_c, is based on "column-like" *buckling* in the plate. The *connections* provide fixed-end column restraint and K = 0.5. Before such *buckling* occurs ($f < F_c$), the *effective width* of the section is calculated using Section 1.1 with the *connection* lines treated as *webs*. When the critical *stress* is reached and exceeded ($f \geq F_c$), the compression plate might not resist the same *stress*, f_c, as the adjacent element that theoretically has slightly less strain. An equivalent width is determined to provide the approximate true force contribution of the buckled plate in resisting bending. This equivalent width is assumed to have an artificially high *stress*, f, which is compatible with both a constant "E" and linear strain distribution across the "built-up" section; however, the actual *stress* might be between F_c and f. ρ_t provides the *effective width* at F_c, and ρ_m allows further *effective width* reduction to provide the equivalent force. The equivalent transformed section properties cannot be greater than the section calculated using *Specification* Section 1.1 at the *stress* level, f. The moment of inertia for deflection is determined by substituting the maximum *stress* at *service load* for F_y and the compression *stress* at *service load*, f_d, for f in *Specification*

Section 1.1.4.

Figure C-1.1.4-2 shows the built-up deck section in bending. In Figures C-1.1.4-1 and C-1.1.4-2, s is the center-to-center *connection* spacing along the plate, w is the center-to-center *connection* spacing across the plate, t is the *thickness* of cover plate, t_2 is the *thickness* of the member connected to the cover plate, f is the compression *stress* in the cover plate, f_c is the compression *stress* in the element connected to the cover plate, f_t is the maximum tension *stress* in the member connected to the cover plate, and d is the overall depth of the built-up member.

In 2012, provisions for determining the *effective width* between the first line of fasteners and the edge stiffener and the effective length of the stiffener were added. The post-*buckling stress* at the first interior line of *connections* is applied across the first interior width, w_1 or w_3, as illustrated in Figure C-1.1.4-1, and at the edge stiffener. *Specification* Equation 1.1.4-7 is based on the approximate shape of the half sine wave restrained by the connectors in the compression element and by the edge stiffener. w′ given in *Specification* Equation 1.1.4-7 is twice the distance from the stiffener to the apex of the wave and models w in *Specification* Section 1.3 for the same wavelength. Equation 1.1.4-6 sets w as e before "column-like" *buckling* occurs. *Specification* Equations 1.3-7 to 1.3-10 are then applied based on w and f. When f reaches or exceeds F_c, *Specification* Equations 1.3-7 to 1.3-10 are applied based on w′ and f′ to evaluate the stiffener. $\rho_m f$ approximates the post-*buckling stress* that cannot be less than F_c since the stiffener must resist F_c as *buckling* begins.

Jones (Jones et al., 1997) validated Luttrell's method (1992), but the researchers cautioned its use for single-fluted members having compression plates with edge stiffeners. Luttrell and Balaji (1992) tested built-up deck with compression plate *thickness* between 0.045 in. (1.14 mm) and 0.06 in. (1.52 mm). Jones (1997) investigated unstiffened cover plates to 0.017 in. (0.432 mm) in *thickness*. The research work at the University of Missouri-Rolla (UMR) indicated that the method worked reasonably well for single-fluted members having unstiffened compression plates when the plate *thickness* exceeded 0.045 in. (1.14 mm). See the illustrative example in the AISI *Cold-Formed Steel Design Manual* (AISI, 2017).

1.2 Effective Widths of Unstiffened Elements

Similar to stiffened compression elements, the *stress* in the *unstiffened compression elements* can reach to the *yield stress* of steel if the w/t ratio is small. Because the unstiffened element has one longitudinal edge supported by the *web* and the other edge is free, the limiting width-to-*thickness* ratio of unstiffened elements is much less than that for stiffened elements.

When the w/t ratio of the unstiffened element is large, *local buckling* (Figure C-1.2-1) will occur at the elastic critical *stress* determined by Equation C-1-1 with a value of k = 0.43. This *buckling* coefficient is listed in Table C-1-1 for case (c). For the intermediate range of w/t ratios, the unstiffened element will buckle in the inelastic range. Figure C-1.2-2 shows the relationship between the maximum *stress* for *unstiffened compression elements* and the w/t ratio, in which Line

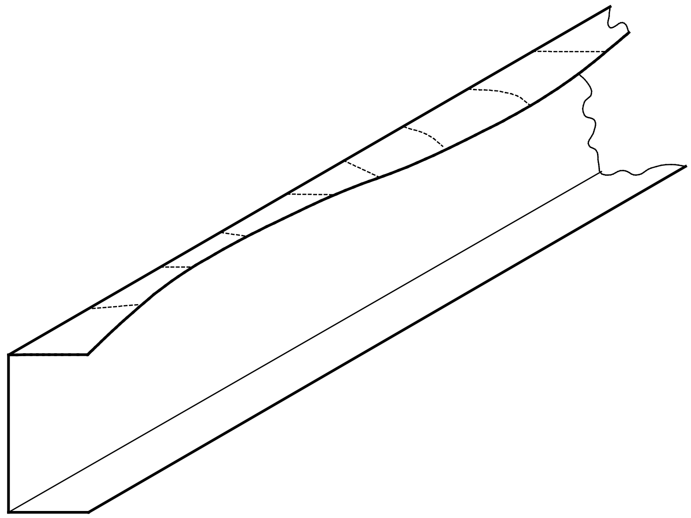


Figure C-1.2-1 Local Buckling of Unstiffened Compression Flange

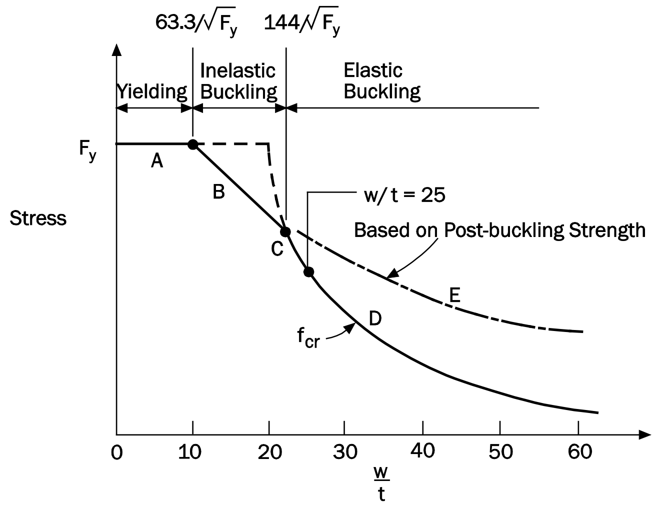


Figure C-1.2-2 Maximum Stress for Unstiffened Compression Elements

A is the *yield stress* of steel, Line B represents the inelastic *buckling stress*, and Curves C and D illustrate the elastic *buckling stress*. The equations for Curves A, B, C, and D have been developed from previous experimental and analytical investigations and used for determining the allowable *stresses* in the *Specification* up to 1986 (Winter, 1970; Yu and LaBoube, 2010). Also shown in Figure C-1.2-2 is Curve E, which represents the maximum *stress* on the basis of the post-*buckling* strength of the unstiffened element. The correlation between some test data on unstiffened elements and the predicted maximum *stresses* is shown in Figure C-1.2-3 (Yu and LaBoube, 2010).

Prior to 1986, it had been a general practice to design cold-formed steel members with unstiffened *flanges* by using the A*llowable Stress Design* approach. The *effective width* equation was not used in earlier editions of the *Specification* due to lack of extensive experimental verification and the concern for excessive out-of-plane distortions under *service loads*.

In the 1970s, the applicability of the *effective width* concept to unstiffened elements under uniform compression was studied in detail by Kalyanaraman, Peköz, and Winter at Cornell University (Kalyanaraman, Peköz, and Winter, 1977; Kalyanaraman and Peköz, 1978). The

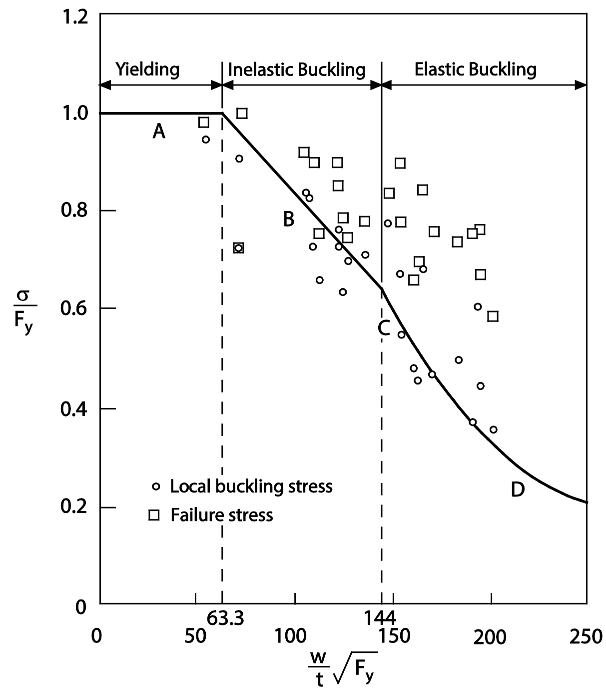


Figure C-1.2-3 Correlation Between Test Data and Predicted Maximum Stress

evaluation of the test data using k = 0.43 was presented and summarized by Peköz in the AISI report (Peköz, 1986b), which indicates that Equation C-1.1-6 developed for stiffened compression elements gives a conservative lower bound to the test results of *unstiffened compression elements*. In addition to the strength determination, the same study also investigated the out-of-plane deformations in unstiffened elements. The results of theoretical calculations and the test results on the sections having unstiffened elements with w/t = 60 were presented by Peköz in the same report. It was found that the maximum amplitude of the out-of-plane deformation at failure can be twice the *thickness* as the w/t ratio approaches 60. However, the deformations are significantly less under the *service loads*. Based on the above reasons and justifications, the *effective design width* approach was adopted for the first time in the 1986 *Specification* for the design of cold-formed steel members having *unstiffened compression elements*.

1.2.1 Uniformly Compressed Unstiffened Elements

In the *Specification*, it is specified that the *effective widths*, b, of uniformly compressed unstiffened elements can be determined in accordance with Section 1.1(a) of the *Specification* with the exception that the *buckling* coefficient, k, is taken as 0.43. This is a theoretical value for long plates. See case (c) in Table C-1-1. For serviceability determination, the *effective widths* of uniformly compressed unstiffened elements can only be determined according to Procedure I of Section 1.1(b) of the *Specification*, because Procedure II was developed only for stiffened compression elements. See Part I of the AISI *Cold-Formed Steel Design Manual* for design examples (AISI, 2017).

1.2.2 Unstiffened Elements and Edge Stiffeners With Stress Gradient

In concentrically loaded compression members and in flexural members where the unstiffened compression element is parallel to the neutral axis, the *stress* distribution is uniform prior to *local buckling*. However, when edge stiffeners of the compression element are present, the compressive *stress* in the edge stiffener is not uniform but varies in proportion to the distance from the neutral axis in flexural members. The unstiffened element (the edge stiffener) in this case has compressive *stress* applied at both longitudinal edges. The unstiffened element of a section may also be subjected to *stress* gradients causing tension at one longitudinal edge and compression at the other longitudinal edge. This can occur in I-sections, plain channel sections and angle sections in minor axis bending.

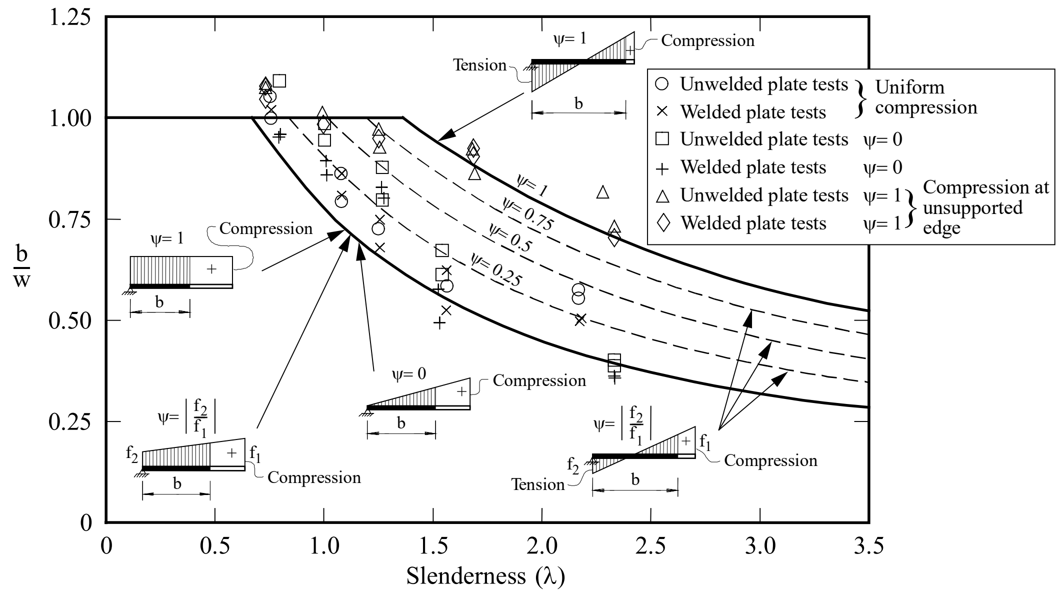


Figure C-1.2.2-1 Effective Width vs. Plate Slenderness

Prior to the 2001 edition of the *Specification*, unstiffened elements with *stress* gradient were designed using the Winter *effective width* equation (Equation C-1.1-4) and k = 0.43. In 2004, Section 1.2.2 of the *Specification* adopted the *effective width* method for unstiffened elements with *stress* gradient proposed by Bambach and Rasmussen (2002a, 2002b and 2002c), based on an extensive experimental investigation of unstiffened plates tested as isolated elements in combined compression and bending. The *effective width*, b, (measured from the supported edge) of unstiffened elements with *stress* gradient causing compression at both longitudinal edges, is calculated using the Winter equation. For unstiffened elements with *stress* gradients causing tension at one longitudinal edge and compression at the other longitudinal edge, modified Winter equations are specified when tension exists at either the supported or the unsupported edges. The *effective width* equations apply to any unstiffened element under *stress* gradient, and are not restricted to particular cross-sections. Figure C-1.2.2-1 demonstrates how the *effective width* of an unstiffened element increases as the *stress* at the supported edge changes from compression to tension. As shown in the figure, the *effective width* curve is independent of the *stress* ratio, ψ, when both edges are in compression. In this case, the effect of *stress* ratio is accounted for by the plate *buckling* coefficient, k, which varies with *stress* ratio and affects the slenderness, λ. When the supported edge is in tension and the unsupported edge is in compression, both the *effective width* curve and the plate *buckling* coefficient depend on the *stress* ratio, as per Equations 1.2.2-4 and 1.2.2-5 of the *Specification*.

Equations are provided for k, determined from the *stress* ratio, ψ, applied to the full element width, and k will usually be higher than 0.43. The equations for k are theoretical solutions for long plates assuming simple support along the longitudinal edge. A more accurate determination of k by accounting for interaction between adjoining elements is permitted for plain channels in minor axis bending (causing compression at the unsupported edge of the unstiffened element), based on research of plain channels in compression and bending by Yiu and Peköz (2001).

In the 2016 edition of the *Specification*, the definition of the *stresses* f_1 and f_2 was revised to reflect that the *effective width* calculations for unstiffened elements should be determined iteratively due to a shift of the neutral axis location, as with other elements in the cross-section. However, if all other elements are fully effective, the *stresses* f_1 and f_2 may be based on the gross cross-section such that iteration is not required.

The *effective width* is located adjacent to the supported edge for all *stress* ratios, including those producing tension at the unsupported edge. Research has found (Bambach and Rasmussen, 2002a) that for the unsupported edge to be effective, tension must be applied over at least half of the width of the element starting at the unsupported edge. For less tension, the unsupported edge will buckle, and the effective part of the element is located adjacent to the supported edge. Further, when tension is applied over half of the element or more starting at the unsupported edge, the compressed part of the element will remain effective for elements with w/t ratios less than the limits set out in Section B4.1 of the *Specification*.

The method for serviceability determination is based on the method used for stiffened elements with *stress* gradient in Section 1.1.2(b) of the *Specification*.

1.3 Effective Width of Uniformly Compressed Elements With a Simple Lip Edge Stiffener

An edge stiffener is used to provide continuous support along a longitudinal edge of the compression *flange* to improve the *buckling stress*. In most cases, the edge stiffener takes the form of a simple lip. Other types of edge stiffeners can be beneficial and are also used for cold-formed steel members, but are not covered in *Specification* Section 1.3.

In order to provide necessary support for the compression element, the edge stiffener must possess sufficient rigidity. Otherwise, it may buckle perpendicular to the plane of the element to be stiffened. As far as the design provisions are concerned, the 1980 and earlier editions of the AISI *Specification* included the requirements for the minimum moment of inertia of stiffeners to provide sufficient rigidity. When the size of the actual stiffener does not satisfy the required moment of inertia, the *load*-carrying capacity of the beam has to be determined either on the basis of a flat element disregarding the stiffener or through tests.

Both theoretical and experimental studies on the local stability of compression *flanges* stiffened by edge stiffeners have been carried out in the past. The design requirements included in the 1986 *Specification* were based on the investigations of adequately stiffened and partially stiffened elements conducted by Desmond, Peköz and Winter (1981a), with additional research work by Peköz and Cohen (Peköz, 1986b). These design provisions were developed on the basis of the critical *buckling* criterion and the post-*buckling* strength criterion.

Specification Section 1.3 recognizes that the necessary stiffener rigidity depends upon the slenderness (w/t) of the plate element being stiffened. The interaction of the plate elements, as well as the degree of edge support, full or partial, is compensated for in the expressions for k, d_s, and A_s (Peköz, 1986b).

In the 1996 edition of the *Specification* (AISI, 1996), the design equations for *buckling* coefficient were changed for further clarity. The requirement of 140° ≥ θ ≥ 40° for the applicability of these provisions was decided on an intuitive basis. For design examples, see Part I of the AISI *Cold-Formed Steel Manual* (AISI, 2013).

Test data to verify the accuracy of the simple lip stiffener design was collected from a number of sources, both university and industry. These tests showed good correlation with the equations in *Specification* Section 1.3.

In 2001, Dinovitzer's expressions (Dinovitzer, et al., 1992) for n (*Specification* Equation 1.3-11) were adopted, which eliminated a discontinuity that existed in the previous design expressions. The revised equation gives n =1/2 for w/t = 0.328S and n = 1/3 for w/t = S, in which S is also the maximum w/t ratio for a stiffened element to be fully effective.

In 2007, the expressions were limited to cover only simple lip edge stiffeners, as the previously employed expressions for complex lip stiffeners were found to be unconservative in comparison with rigorous nonlinear finite element analysis (Schafer, et al., 2006). Design of members with complex lips may be handled via the *Direct Strength Method* provided in Chapters E and F, as applicable. In addition, the design provisions for the uniformly compressed elements with one intermediate stiffener were deleted in the 2007 edition of the *Specification* due to the fact that the *effective width* of such members can be determined in accordance with *Specification* Section 1.4.1.

1.4 Effective Widths of Stiffened Elements With Single or Multiple Intermediate Stiffeners or Edge-Stiffened Elements With Intermediate Stiffener(s)

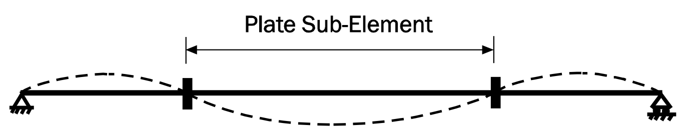
1.4.1 Effective Width of Uniformly Compressed Stiffened Elements With Single or Multiple Intermediate Stiffeners

The structural efficiency of a stiffened element always exceeds that of an unstiffened element with the same w/t ratio by a sizeable margin, except for low w/t ratios, for which the compression element is fully effective. When stiffened elements with large w/t ratios are used, the material is not employed economically inasmuch as an increasing proportion of the width of the compression element becomes ineffective. On the other hand, in many applications of cold-formed steel construction, such as panels and decks, maximum coverage is desired and, therefore, large w/t ratios are called for. In such cases, structural economy can be improved by providing intermediate stiffeners between *webs*.

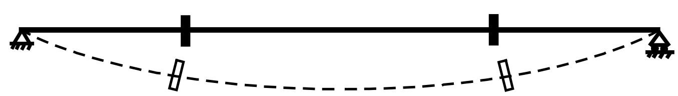
The *buckling* behavior of rectangular plates with central stiffeners is discussed by Bulson (1969). For the design of cold-formed steel beams using intermediate stiffeners, the 1980 *Specification* contained provisions for the minimum required moment of inertia, which was based on the assumption that an intermediate stiffener needed to be twice as rigid as an edge stiffener. In view of the fact that for some cases the design requirements for intermediate stiffeners included in the 1980 *Specification* could be unduly conservative (Peköz, 1986b), the design provisions were revised in 1986 according to Peköz's research findings (Peköz, 1986b and 1986c). In 2007, the design of uniformly compressed elements with multiple or single intermediate stiffeners was merged. The multiple intermediate stiffener provisions were developed based on Peköz's continuing research on intermediate stiffeners (Schafer and Peköz, 1998) and the finding that the method developed in Section 1.4.1 of the *Specification* for multiple intermediate stiffeners could provide the same reliability as the method for single intermediate stiffeners (Yang and Schafer, 2006) in the previous edition of the *Specification* (AISI, 2001).

Prior to 2001, the AISI *Specification* and the Canadian *Standard* provided different design provisions for determination of the *effective widths* of uniformly compressed stiffened elements with multiple intermediate stiffeners or edge-stiffened elements with intermediate stiffeners. In the *Specification*, the design requirements of Section 1.4 dealt with: (1) the minimum moment of inertia of the intermediate stiffener, (2) the number of intermediate stiffeners considered to be effective, (3) the "equivalent element" of *multiple-stiffened element* having closely spaced intermediate stiffeners, (4) the *effective width* of sub-element with $w/t > 60$, and (5) the reduced area of stiffeners. In the Canadian *Standard*, a different design equation was used to determine the equivalent *thickness*.

In 2001, *Specification* Section 1.4.1 was revised to reflect recent research findings for flexural members with multiple intermediate stiffeners in the compression *flange* (Papazian et al., 1994; Schafer and Peköz, 1998; Acharya and Schuster, 1998). The method is based on determining the plate *buckling* coefficient for the two competing modes of *buckling*: *local buckling*, in which the stiffener does not move; and *distortional buckling*, in which the stiffener buckles with the entire plate. See Figure C-1.4.1-1. Experimental research shows that the distortional mode is prevalent for members with multiple intermediate stiffeners.



(a) Local Buckling



(a) Distortional Buckling

Figure C-1.4.1-1 Local and Distortional Buckling of a Uniformly Compressed Element With Multiple Intermediate Stiffeners

The reduction factor, ρ, is applied to the entire element (*gross area* of the element/*thickness*) instead of only the flat portions. Reducing the entire element to an *effective width*, which ignores the geometry of the stiffeners, for effective section property calculation allows *distortional buckling* to be treated consistently with the rest of the *Specification*, rather than as an "*effective area*" or other method. The resulting *effective width* must act at the centroid of the original element including the stiffeners. This ensures that the neutral axis location for the member is unaffected by the use of the simple *effective width*, which replaces the more complicated geometry of the element with multiple intermediate stiffeners. One possible result of this approach is that the calculated *effective width* (b_e) may be greater than b_o. This may occur when ρ is near 1, and is due to the fact that b_e includes contributions from the stiffener area and b_o does not. As long as the calculated b_e is placed at the centroid of the entire element, the use of $b_e > b_o$ is correct.

In 2010, *Specification* Equation 1.4.1.1-1 was replaced by

$$k_{loc} = 4(b_o/b_p)^2 \qquad \text{(C-1.4.1-1)}$$

where

k_{loc} = Plate *buckling* coefficient of element

b_o = Total *flat width* of stiffened element

b_p = Sub-element *flat width* for *flange* with equally spaced stiffeners

This replacement ensures that *Specification* Sections 1.4.1.1 and 1.4.1.2 provide the same answer for sub-element *local buckling*, and replaces the overly conservative estimate of the 2007 edition of the *Specification* Equation 1.4.1.1-1, which ignored the stiffener width (Schafer, 2009).

1.4.2 Edge-Stiffened Elements With Intermediate Stiffener(s)

The *buckling* modes for edge-stiffened elements with one or more intermediate stiffeners include *local* sub-element *buckling*, *distortional buckling* of the intermediate stiffener, and *distortional buckling* of the edge stiffener, as shown in Figure C-1.4.2-1. If the edge-stiffened element is stocky ($b_o/t < 0.328S$) or the stiffener is large enough ($I_s > I_a$ and thus $k = 4$, per the rules of *Specification* Section 1.3), then the edge-stiffened element performs as a stiffened element. In this case, *effective width* for *local* sub-element *buckling* and *distortional buckling* of the intermediate stiffener may be predicted by the rules of *Specification* Section 1.4.1. However, an edge-stiffened element does not have the same *web* rotational restraint as a stiffened element; therefore, the constant R of *Specification* Section 1.4.1 is conservatively limited to be less than or equal to 1.0.

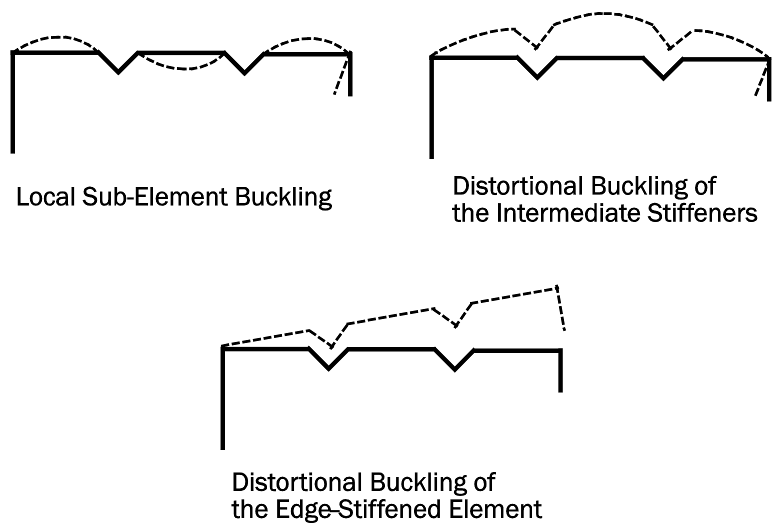


Figure C-1.4.2-1 Buckling Modes in an Edge-Stiffened Element With Intermediate Stiffeners

If the edge-stiffened element is partially effective ($b_o/t > 0.328S$ and $I_s < I_a$ and thus $k < 4$, per the rules of *Specification* Section 1.3), then the intermediate stiffener(s) should be ignored and the provisions of *Specification* Section 1.3 followed. Elastic *buckling* analysis of the distortional mode for an edge-stiffened element with intermediate stiffener(s) indicates that the effect of intermediate stiffener(s) on the *distortional buckling stress* is ±10 percent for practical intermediate and edge stiffener sizes.

When applying *Specification* Section 1.4.2 for *effective width* determination of edge-stiffened elements with intermediate stiffeners, the *effective width* of the intermediately stiffened *flange*, b_e, is replaced by an equivalent flat section (as shown in *Specification* Figure 1.4.1-2). The edge stiffener should not be used in determining the centroid location of the equivalent flat *effective width*, b_e, for the intermediately stiffened *flange*.

Stub compression testing performed in 2003 demonstrates the adequacy of this approach (Yang and Hancock, 2003).

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APPENDIX 2, ELASTIC BUCKLING ANALYSIS OF MEMBERS

Elastic *buckling stress*, or *stress* resultants (axial force, shear force, bending moment, etc.) are used extensively in the *Specification* for the determination of strength. The *buckling* of cold-formed steel members includes traditional global *buckling* modes such as *flexural buckling* and *lateral-torsional buckling*, as well as *buckling* modes that include cross-sectional deformation such as *local buckling* and *distortional buckling*.

It is important to realize that elastic *buckling* itself is not a *limit state*. Elastic *buckling stress* or *stress* resultants are instead used as inputs in various strength equations throughout the *Specification*. For example, in determining the *nominal strength* [*resistance*] of a column, Section E2 requires the global *buckling stress*, and Section E3 requires the *local buckling stress* either implicitly in determining the effective width in Section E3.1 or explicitly after conversion to a *local buckling* force in the *Direct Strength Method* of Section E3.2. Section E4 requires the input of *distortional buckling* force. In each case, the elastic *buckling stress* (or its resultant) is employed in strength expressions that provide varying degrees of post-*buckling* reserve and interaction with *yielding* and other *buckling* modes in determining the *nominal strength* [*resistance*] in a given *limit state*.

2.1 General Provisions

The *Specification* does not place a preference for what methods are used to determine elastic *buckling stress* or *stress* resultants. Conversion between *stress* and *stress* resultants is provided.

2.2 Numerical Solutions

2.2.1 Elastic Buckling of Cold-Formed Steel Members

The fundamental *buckling* modes in a cold-formed steel member include: *local buckling*, *distortional buckling*, and global *buckling* modes: *flexural buckling*, *torsional buckling*, and *flexural-torsional buckling* for compression members, and *lateral-torsional buckling* for bending members. The fundamental *buckling* modes are illustrated in Figure C-2.2.1-1.

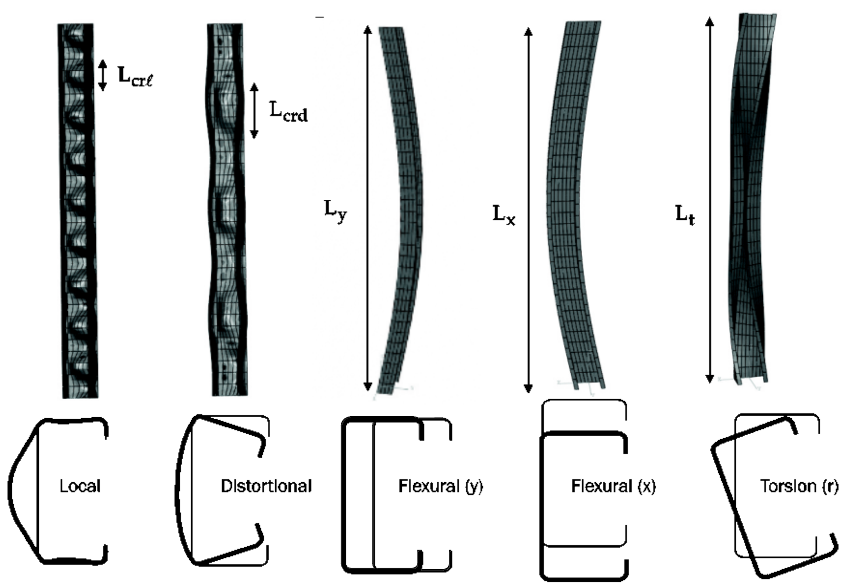


Figure C-2.2.1-1 Illustration of Fundamental Elastic Buckling Modes for a Lipped Channel in Compression

The elastic *buckling* load (force) is the load in which the equilibrium of the member is neutral between two alternative states: buckled and straight. Thin-walled cold-formed steel members have at least three relevant elastic *buckling* modes: local, distortional, and global (Figure C-2.2.2-2). The global *buckling* mode includes *flexural*, *torsional*, or *flexural-torsional buckling* for columns, and *lateral-torsional buckling* for beams.

The *Effective Width Method* traditionally addressed *local buckling* and its interaction with global *buckling*. The *distortional buckling* consideration was added in 2004. Further, the *Effective Width* approach to *local buckling* is to conceptualize the member as a collection of "elements" and investigate *local buckling* of each element separately.

The *Direct Strength Method*, introduced in 2004, provides a means to incorporate all three relevant *buckling* modes into the design process. Further, all *buckling* modes are determined for the member as a whole rather than element by element. This ensures that compatibility and equilibrium are maintained at element junctures.

Local Buckling. *Limit state* of *buckling* of a compression element where the line junctions between elements remain straight and angles between elements do not change.

Local buckling involves significant distortion of the cross-section, but this distortion involves only rotation, not translation, at the fold lines of the member, as shown in Figure C-2.2.1-1. The *buckling* half-wavelength ($L_{cr\ell}$) for *local buckling* is less than the largest characteristic dimension of the member under compressive *stress* (this length is demarcated with a short vertical dashed line in the examples of Figure C-2.2.2-2). Since the *local buckling* half-wavelength is short, *local buckling* is difficult to retard, and in general must always be considered. Changes to the geometry of the member (stiffeners, change of *thickness*, etc.) are the most effective means for changing *local buckling* loads or moments.

Distortional Buckling. A mode of *buckling* involving change in cross-sectional shape, excluding *local buckling*.

Distortional buckling involves both translation and potentially rotation at the fold line of a member. *Distortional buckling* involves distortion of one portion of the cross-section and predominantly rigid response of a second portion. For instance, the edge-stiffened *flanges* of the lipped C-section in Figure C-2.2.1-1 are primarily responding as a rigid cross-section while the *web* is distorting. *Distortional buckling* occurs at a *buckling* half-wavelength (L_{crd}) intermediate to *local* and global *buckling* modes. The half-wavelength is typically several times larger than the largest characteristic dimension of the member; however, L_{crd} is highly dependent on both the loading and the geometry. For some members, *distortional buckling* may not occur. Bracing can be effective in retarding *distortional buckling* and boosting the strength of a member.

Global Buckling. A mode of *buckling* that does not involve distortion of the cross-section. The global *buckling* includes the following *buckling* modes:

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

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

Flexural-Torsional Buckling. *Buckling* mode in which a compression member bends and twists simultaneously without change in cross-sectional shape.

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

For columns, global *buckling* modes include *flexural*, *torsional* and *flexural-torsional buckling*. For beams bent about their strong-axis, *lateral-torsional buckling* is the global *buckling* mode of interest. Figure C-2.2.1-1 illustrates the uncoupled global *buckling* modes; but for the *singly-symmetric section* illustrated, the strong-axis flexure (x) and torsion (t) are coupled as two *flexural-torsional buckling* modes. Global *buckling* modes involve translation (flexure) and/or rotation (torsion) of the entire cross-section. No distortion exists in any of the elements in the cross-section. The global *buckling* half-wavelength is equal to the unbraced length (L_x, L_y or L_t). Bracing can be effective in retarding global *buckling* and boosting the member strength.

2.2.2 Summary of Available Numerical Solution Methods

The analytical solutions for elastic global, *local*, and *distortional buckling* in *Specification* Section 2.3 are convenient for many common cases, however, there are important limitations to consider. The *local* and *distortional buckling* provisions are approximations that may sometimes be overly conservative. The global *buckling* solutions are accurate but only for specific boundary conditions and points of load application. Members stiffened by attachments to other components such as deck or sheathing may exhibit higher *buckling* capacities. The numerical methods discussed in this section provide solutions to address some of the limitations of the analytical provisions.

Finite Strip Analysis

The semi-analytical Finite Strip Method is a numerical solution utilizing plate bending strips to discretize a cold-formed steel cross-section. For a model with simply supported end boundary conditions, a finite strip *buckling* analysis leads to the member's signature curve which provides the *local*, *distortional*, and *global* elastic *buckling* loads or moments as needed in the *Specification*. Each *buckling* mode is associated with a particular cross-section shape and a *buckling* half-wavelength that together provide a complete description of the *buckling* mode. An example signature curve for a lipped channel in pure compression is provided in Figure C-2.2.2-1, and additional examples are provided in Figure C-2.2.2-2.

Finite strip analysis is a specialized variant of the Finite Element Method. For elastic stability of cold-formed steel structures, it is one of the most efficient and popular methods. Cheung and Tham (1998) explain the basic theory while Hancock et al. (2001) and Ádány and Schafer (2006) provide specific details for stability analysis with this method. Hancock and his researchers pioneered the use of finite strip analysis for stability of cold-formed steel members and convincingly demonstrated the important potential of finite strip analysis in both cold-formed steel design and behavior.

AISI has sponsored research that, in part, has led to the development of the freely available program, CUFSM, which employs the Finite Strip Method for elastic *buckling* determination of any cold-formed steel cross-section. The program is available at www.ce.jhu.edu/bschafer/cufsm and runs on both Windows and Mac platforms. Tutorials and examples are available online at the same address. Other programs that provide similar solutions include THIN-WALL (Hancock, 1995), and CFS. Steel Smart System uses an embedded version of CUFSM.

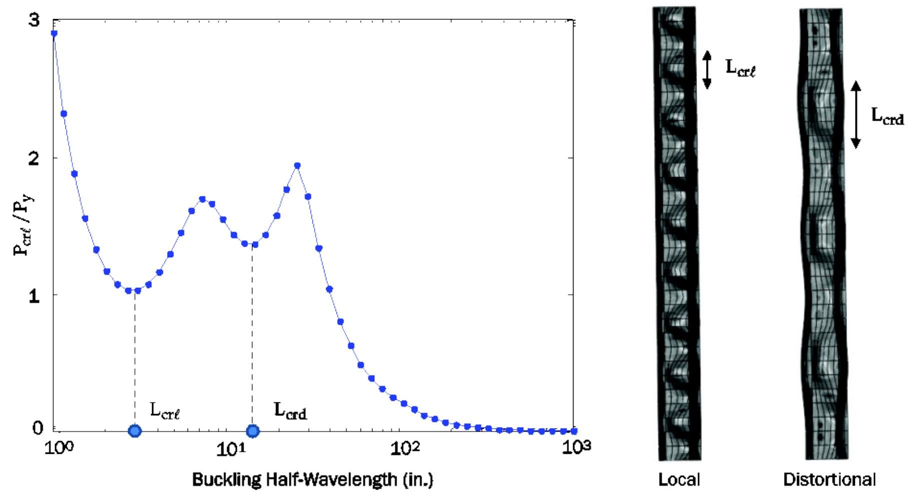
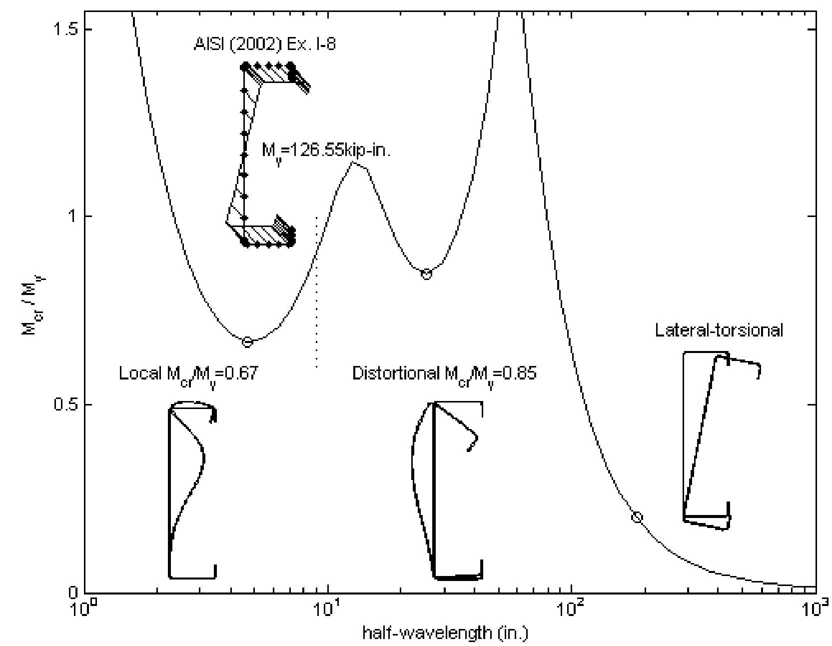
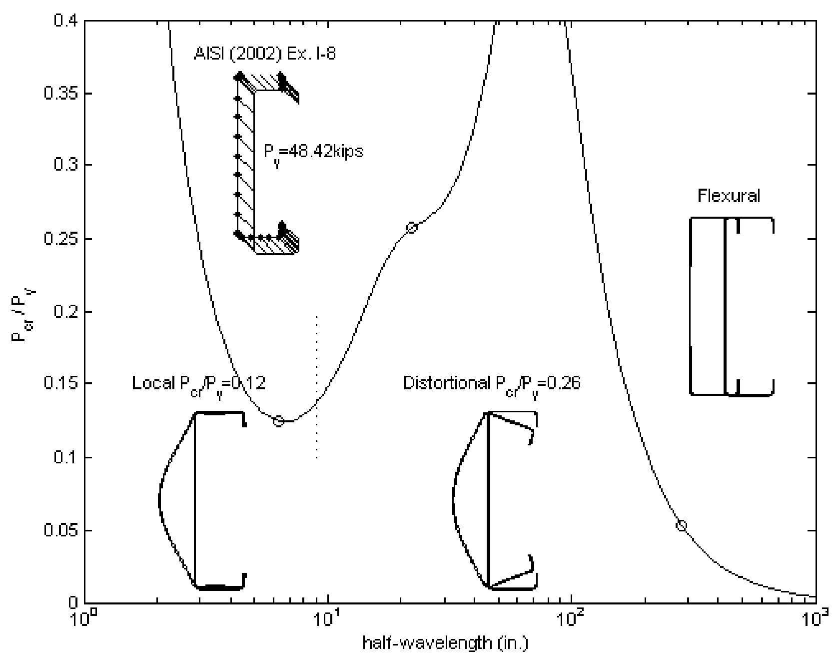


Figure C-2.2.2-1 Semi-Analytical Finite Strip Analysis Signature Curve Results for Lipped Channel in Compression

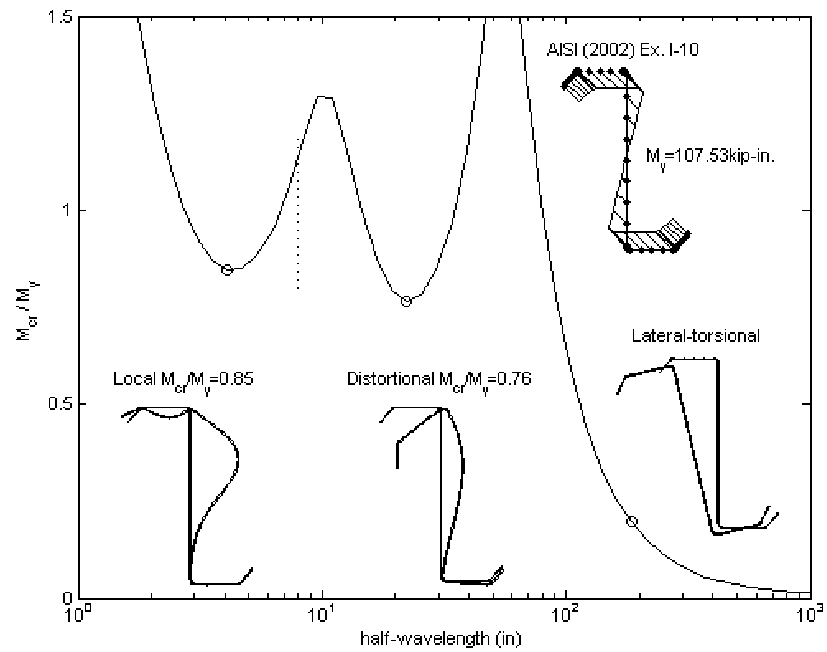
As detailed in *Commentary* Sections 2.2.3 to 2.2.11, specialized variants of the Finite Strip Method exist for shear, general end boundary conditions, members with holes, members with attachments, and for numerically (and automatically) identifying the *local*, *distortional*, and global *buckling* modes, and other special cases.

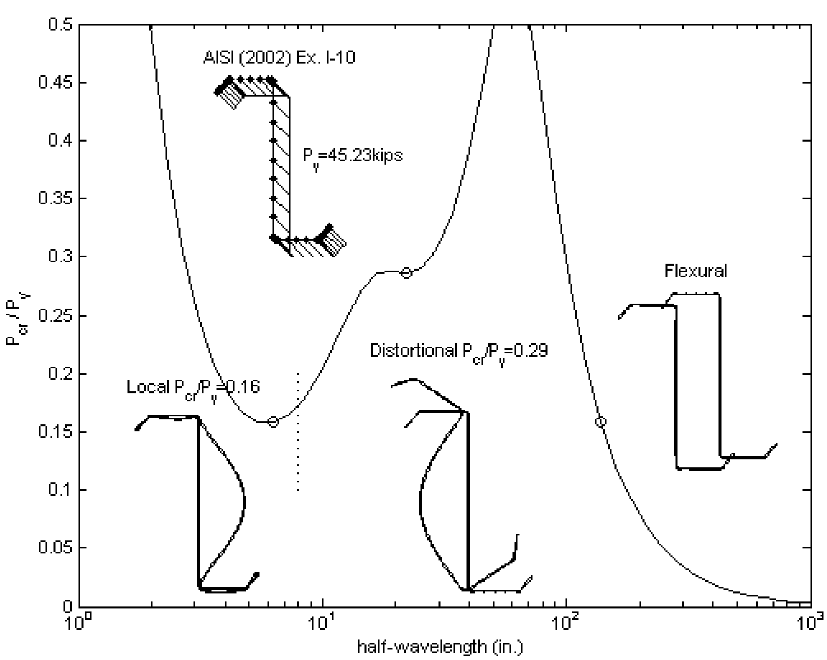




(a) 9CS2.5x059 of AISI *Cold-Formed Steel Design Manual* (2002), Example I-8

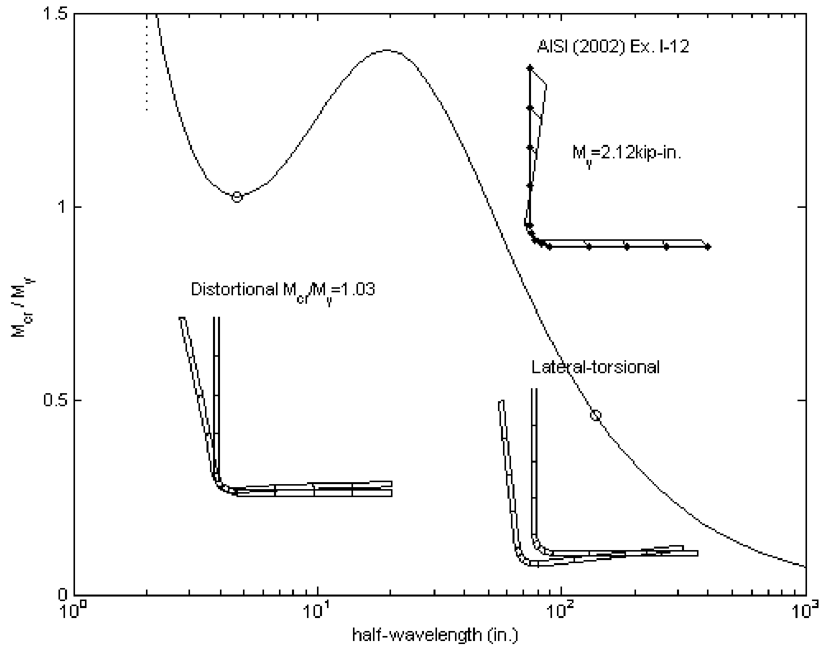
Figure C-2.2.2-2 Examples of Bending and Compression Elastic Buckling Analysis With Finite Strip Method

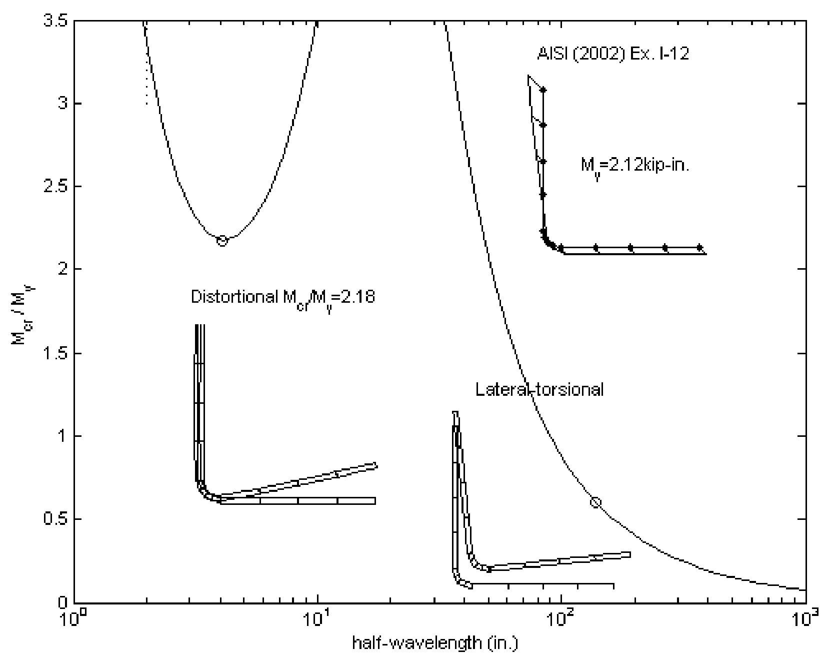




(b) 8ZS2.25x059 of AISI *Cold-Formed Steel Design Manual* (2002), Example I-10

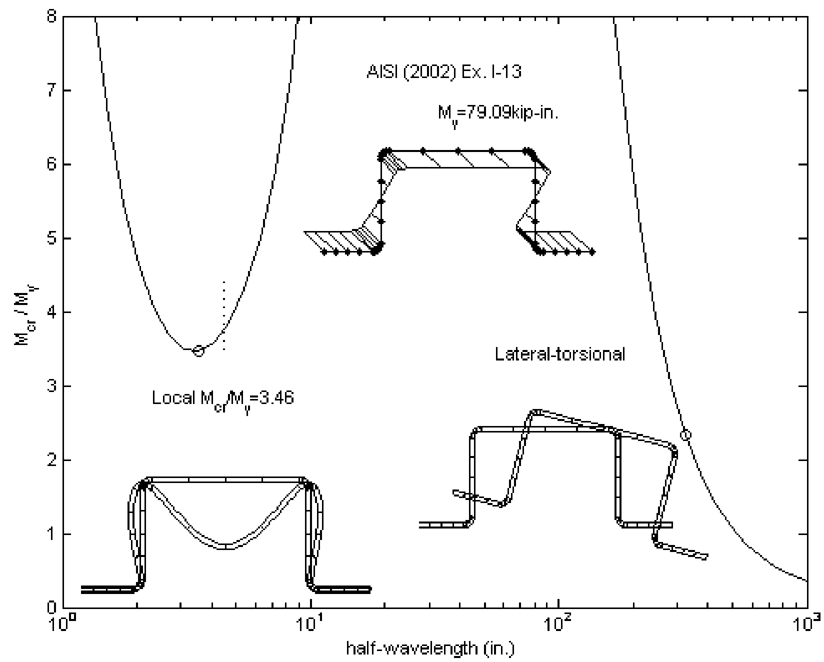
Figure C-2.2.2-2 Examples of Bending and Compression Elastic Buckling Analysis With Finite Strip Method (cont.)

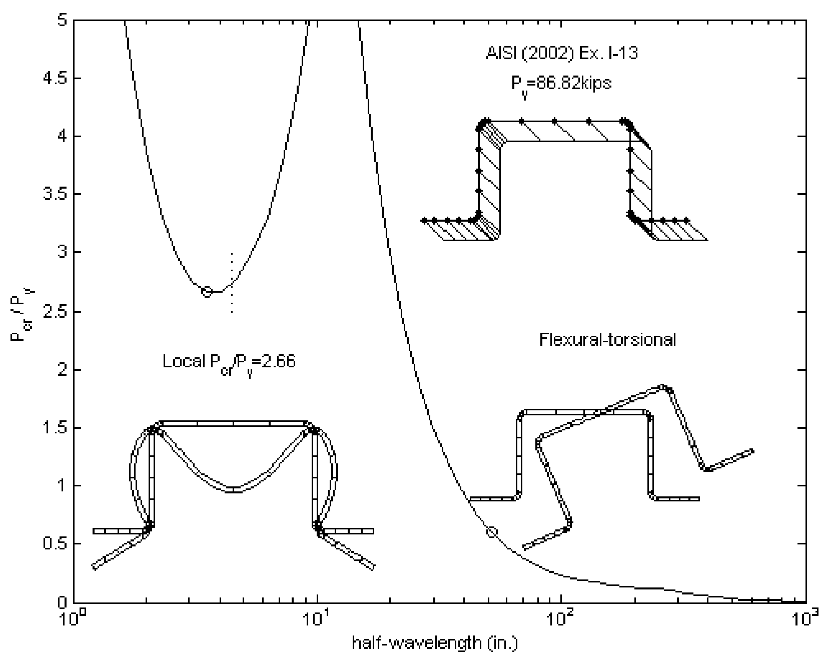




(c) 2LU2x060 of AISI *Cold-Formed Steel Design Manual* (2002), Example I-12

Figure C-2.2.2-2 Examples of Bending and Compression Elastic Buckling Analysis With Finite Strip Method (cont.)





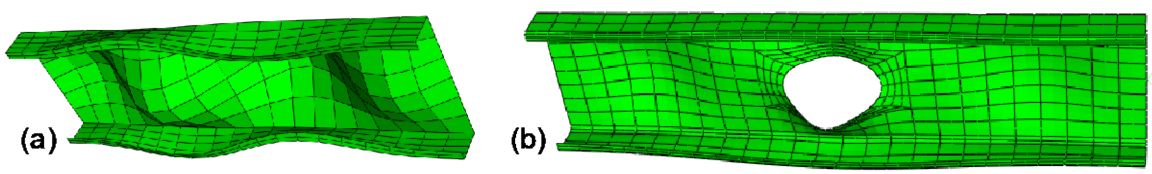
(d) 3HU4.5x135 of AISI *Cold-Formed Steel Design Manual* (2002), Example I-13

Figure C-2.2.2-2 Examples of Bending and Compression Elastic Buckling Analysis With Finite Strip Method (cont.)

Shell Finite Element Methods

Finite element models of cold-formed steel members developed from plate or shell finite elements are capable of providing appropriate *buckling* solutions for *local*, *distortional*, and global *buckling*. The *buckling* modes illustrated in Figure C-2.2.1-1 were generated from an eigen-*buckling* analysis using shell finite elements. Incorporation of specialized details of the section, including holes (as illustrated in Figure C-2.2.2-3(b)), or any other variation along the length, as well as unique end boundary conditions, attachments, etc. are all possible using shell finite element models. In general, the more complicated the situation, the greater the preference for the use of shell finite element-based models.

However, categorization of the numerically determined *buckling* solutions into *local*, *distortional*, and global *buckling* for use in the *Specification* often requires significant engineering judgment. A typical shell finite element model may require visual evaluation of as many as 100 modes to find the fundamental *buckling* modes. *Buckling* modes often appear as coupled, such as in Figure C2.2.2-3(a), further complicating the identification effort. No direct equivalent to the finite strip analysis signature curve exists for shell finite element models. Additional discussion of identification is provided in *Commentary* Section 2.2.3.



(a) Local-Distortional Interacted Elastic Bucking Mode

(b) Buckling in a Member With a Hole

Figure C-2.2.2-3 Shell Finite Element Elastic Buckling Results for a Lipped Channel in Compression

Most basic finite element texts for solid mechanics include the full details of thin-plate and thin-shell finite elements appropriate for modeling thin-walled cold-formed steel members (e.g., see Cook et al. (1989), or Zienkiewicz and Taylor (1989, 1991)). Due to the common practice of using linear or polynomial shape functions in the Finite Element Method, the number of elements required for reasonable accuracy can be significant and mesh convergence studies may need to be performed to ensure adequate accuracy, particularly for *local buckling* modes.

A large variety of commercial software provides plate or shell finite elements capable of accurately predicting the elastic *buckling* modes of cold-formed steel members, including (but not limited to): ABAQUS, ANSYS, MARC, and MSC NASTRAN.

Generalized Beam Theory

Generalized Beam Theory enriches a standard beam finite element with additional cross-section deformation modes consistent with *local* and *distortional buckling* and can provide elastic *buckling* solutions appropriate for use in the *Specification*. Generalized Beam Theory is capable of generating a member's signature curve for stability as in Figure C-2.2.2-1. In common implementations, the method is directly comparable to the Finite Strip Method, though generally utilizing less degrees of freedom. Specialized variants of Generalized Beam Theory exist for a variety of member conditions, loading conditions, shear deformations, etc.

Generalized Beam Theory originally was developed by Schardt (1989), disseminated by

Davies et al. (1994), implemented by Davies and Jiang (1996, 1998), and further expanded by Silvestre and Camotim (2002a, 2002b), Bebiano et al. (2007, 2015), Camotim et al. (2008) and Basaglia and Camotim (2013). Research on Generalized Beam Theory remains active. The method provides an explicit ability to separate the different *buckling* modes, making the approach especially amenable in design. Professor Camotim's group at University of Lisbon developed the program GBTUL and made it available for Generalized Beam Theory based *buckling* analysis. Version 2 can be used to analyze members with arbitrary flat-walled cross-sections and handles general loading and support conditions (Bebiano et al., 2014).

Other Solutions

Any numerical method that incorporates plate theory has the potential to provide an accurate elastic *buckling* solution for cold-formed steel members. For example, beyond finite strip analysis, finite element analysis, and Generalized Beam Theory, finite differences and boundary elements have both been successfully used in related stability problems (e.g., Harik et al. (1991), Elzein, 1991). In addition, many of the analytical solutions provided in *Specification* Section 2.3 can be generalized and applied as numerical solutions.

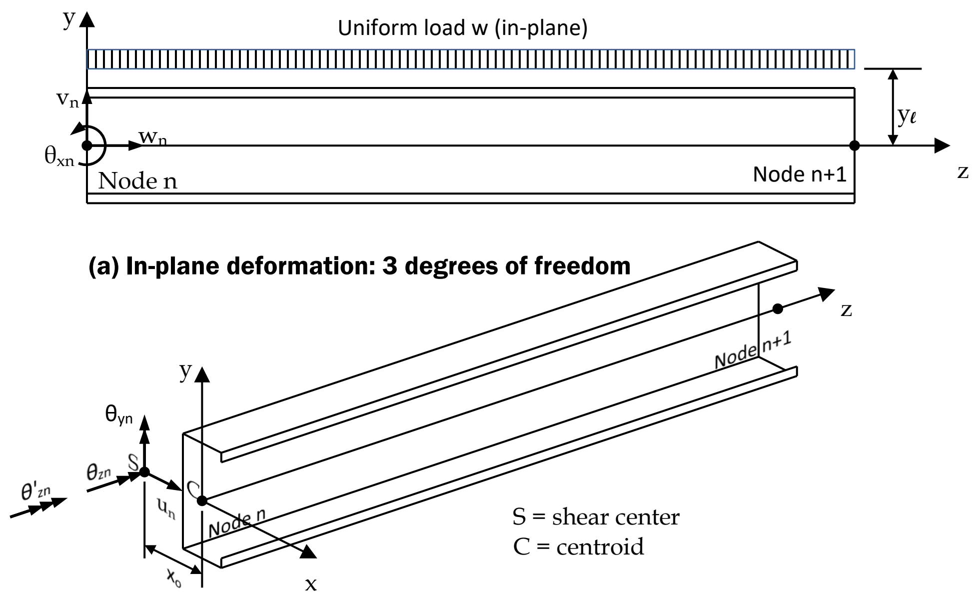
Beam elements used in typical structural analysis software are not capable of including cross-sectional distortion and thus do not include *local buckling* or *distortional buckling*. Beam elements used in typical structural analysis software do not explicitly include warping torsion and thus do not accurately model *torsional*, *flexural-torsional*, or *lateral-torsional buckling*. Beam elements used in typical structural analysis software do not account for torsion demands inherent in sections where the shear center and centroid do not coincide, and thus should be used with care for singly- and un-symmetric sections.

Beam-Column Global Buckling Analysis

The general numerical analysis of undistorted thin-walled members in three dimensions involves subdividing the members into linear elements with seven degrees of freedom (DOF) at each end. These DOF include the three displacements in the x, y, z orthogonal directions, and the three rotations about the x, y, z orthogonal axes. The 7^{th} DOF is a twist per unit length at each end of the element about its longitudinal axis, corresponding to warping displacement from non-uniform torsion (Timoshenko and Gere, 1961; Vlasov, 1961). Torsion is therefore represented by two degrees of freedom at each end: a twist rotation and a rate of change of twist rotation. Geometric nonlinear analysis of thin-walled structures in three dimensions with general cross-sections is described in detail in Liu et al. (2018), Rinchen et al. (2020), Chen et al. (2022) and Abdelrahman et al. (2022). In the case of compression members, the global *buckling* mode is generally called *flexural-torsional buckling* as in Section E2, and for flexural members, the global *buckling* mode is generally called *lateral-torsional buckling* as in Section F2.

To consider *lateral-torsional buckling* out of the plane of a thin-walled member or planar structure composed of thin-walled members, the seven-DOF problem at each end of each element is segregated to three DOF corresponding to in-plane deformations at each end, and four DOF corresponding to out-of-plane deformations at each end. If the longitudinal axis of the element is taken as the z-axis as shown in Figure C-2.2.2-4, and the y-axis is in the plane of the element containing the load (w) acting in the y-direction, so that the x-axis can be considered as the out-of-plane direction, then the three in-plane DOF are the y- and z-displacements (v_n, w_n) at node n and the rotation about the x-axis (θ_{xn}) perpendicular to the plane at node n as shown in Figure C-2.2.2-4. The four out-of-plane DOF are the x-displacement (u_n), the y- and z-axis rotations (θ_{yn}, θ_{zn}) and twist per unit length about the z-axis (θ'_{zn}) at node n as shown in Figure C-2.2.2-4.

The full FEM formulation of the *lateral-torsional buckling* analysis is given in Trahair (1993). A computer program (called PRFELB) which performs the in-plane stiffness analysis and the out-of-plane *buckling* analysis of a plane frame can be found in Papangelis et al. (1998). Full implementation for global *buckling* of beam-columns is provided in the 3D structural analysis software MASTAN2 (Ziemian et al., 2021) with the web address www.mastan2.com.



(a) In-plane deformation: 3 degrees of freedom

(b) Out-of-plane deformation: 4 degrees of freedom

Figure C-2.2.2-4 Thin-Walled Linear Element With Nodal Deformations

2.2.3 Numerical Solutions – Identifying Buckling Modes

Once a model is constructed in any of the available methods, the appropriate *local*, *distortional*, and global *buckling* modes must be identified. In some cases, this can be a challenge; however, it is often easy to identify that a particular *buckling* mode is higher than a certain value due to the nature of most analyses which report results from the smallest *buckling* load (moment) to the largest. For all *buckling* modes—local, distortional, and global—if the elastic *buckling* value is large enough, then the cross-section will develop its full capacity (e.g., the yield moment in bending, M_y, or the squash load in compression, P_y). Using the strength prediction equations of the *Specification*, the following limits can be generated:

Flexural Members (not considering inelastic reserve)

if $M_{cr\ell} > 1.66M_y$, then no reduction will occur due to *local buckling*

if $M_{crd} > 2.21M_y$, then no reduction will occur due to *distortional buckling*

if $M_{cre} > 2.78M_y$, then no reduction will occur due to global *buckling*

Compression Members

if $P_{cr\ell} > 1.66P_y$, then no reduction will occur due to *local buckling*

if $P_{crd} > 3.18P_y$, then no reduction will occur due to *distortional buckling*

if $P_{cre} \geq 3.97P_y$, a 10% or less reduction will occur due to global *buckling*
if $P_{cre} \geq 8.16P_y$, a 5% or less reduction will occur due to global *buckling*
if $P_{cre} \geq 41.64P_y$, a 1% or less reduction will occur due to global *buckling*

When considering the limits for *local buckling*, the given values are conservative, since *local buckling* interacts with global *buckling*, M_y and P_y can be replaced by M_{ne} and P_{ne} for the *local buckling* upper-bounds, where M_{ne} and P_{ne} are the *nominal strengths* [*resistances*] determined in the *Specification* for global *buckling limit states*.

Identification in Finite Strip Analysis

Finite strip analysis is generally the preferred tool for predicting elastic *buckling*, and in some cases identification of the *buckling* modes is readily apparent. For example, in Figure C-2.2.2-1, *local buckling* is the first minimum in the signature curve, *distortional buckling* is the second minimum in the signature curve, and global *buckling* is the final descending branch of the signature curve and can be read directly at the global *buckling effective length*, KL. This is the ideal scenario. Study of the examples of Figure C-2.2.2-2 indicates that immediate identification from the signature curve is often, but not always, possible. If any *buckling* mode can be identified to be at a *buckling* value greater than the preceding limits (e.g., $P_{crd} > 3.18P_y$), then further identification of that mode need not be pursued.

Finite strip analysis may have indistinct minima in the signature curve. For example, *distortional buckling* in the Z-section in compression of Figure C-2.2.2-2 is difficult to identify. The basic definitions in *Commentary* Section 2.2.1 may be used to identify appropriate half-wavelengths and cross-section deformations for manual identification of the modes; however, this can be fairly subjective. In some cases ($K_xL_x \neq K_yL_y \neq K_tL_t$, or $KL < L_{crd}$), it may be easier to use finite strip analysis for *local* and *distortional buckling* determination, but use analytical solutions for global *buckling*. An extension of the Finite Strip Method has been developed that allows for automatic identification and full separation of each mode, termed the constrained Finite Strip Method (Ádány and Schafer, 2008). The method is applied to practical identification of cold-formed steel members in Li and Schafer (2010), is the basis for tabular solutions for lipped channels in CFSEI Tech Note G103-11 (Li and Schafer, 2011), and is provided within the freely available finite strip program CUFSM (Li and Schafer, 2010b). The method is not without its own limitations, and is under active development (Li et al., 2013).

Another study has shown that numerical evaluation of mode shape displacements can be used to identify *buckling* modes (Glauz, 2016). This study separates section and axial deformations, and quantifies mode shape deformation work to categorize the *buckling* mode.

Identification in Shell Finite Element Models

Shell finite element models provide the greatest power and flexibility with respect to construction of a model and calculation of the elastic *buckling* modes and associated loads (moments). However, shell finite element models provide no tools for identification of the modes, and the process can be subjective, time consuming, and difficult to automate. In general, the modes are ordered from smallest to largest and the analyst must visually investigate each mode. Visual identification proceeds using the basic definitions of *Commentary* Section 2.2.1, but the process can be somewhat subjective.

A conservative approach to identification in shell finite element models is to find the smallest *buckling* mode that has characteristics similar to a basic definition; for example, *flange*/lip translation associated with *distortional buckling*, and assign the *buckling* load (or moment) to that mode. In some cases, no deformations will be present in the initial results that match a given mode (e.g., *local buckling* in a thicker member). The limits of the preceding section are useful in this process; if any *buckling* mode can be identified to be at a *buckling* value greater than the preceding limits (e.g., $P_{cr\ell} > 1.66P_y$), then further identification of that mode need not be pursued.

Numerical tools that augment shell finite element models and allow for automatic identification are under development (Li et al., 2013). The deformation work method (Glauz, 2016) described in *Identification in Finite Strip Analysis* could be adapted to finite element models for selected cross-sections of the member.

Identification in Generalized Beam Theory Models

The identification of *buckling* modes in models using Generalized Beam Theory is relatively direct. The analyst determines which deformation modes are to be employed in the model and for any *buckling* mode can assess to what extent *local*, *distortional*, or global *buckling* modes are engaged based on what deformation modes were included. Models must use sharp corners (no corner radius).

2.2.4 Numerical Solutions – Generalized Loading Conditions

The elastic *buckling* response may be found under any combination of applied loads. In the most general condition, all loads of interest are applied to the member and a linear static analysis is used to determine the internal *stresses*. These *stresses* are used to determine the geometric stiffness matrix. Subsequently, an eigen-*buckling* analysis is performed to determine the *buckling* magnitude. Appendix 3 provides a means to utilize such analyses in design for combinations of axial load and bending.

Consider any loading combination that creates axial loading (P_r) and bending demands (M_{1r}, M_{2r}) on a cross-section. Per Appendix 3, the demand is generalized as β_r, following *Specification* Equation 3.1.1-1. The *buckling* solution, under the combined loads, provides a load factor (LF) that is a multiple of the applied loads, i.e.

$$P_{cr} = (LF) \times P_r \tag{C-2.2.4-1a}$$

$$M_{1cr} = (LF) \times M_{1r} \tag{C-2.2.4-1b}$$

$$M_{2cr} = (LF) \times M_{2r} \tag{C-2.2.4-1c}$$

Normalized consistent with β_r, the critical elastic *buckling* magnitude is

$$\beta_{cr} = \sqrt{\left(\frac{P_{cr}}{P_y}\right)^2 + \left(\frac{M_{1cr}}{M_{1y}}\right)^2 + \left(\frac{M_{2cr}}{M_{2y}}\right)^2} = \sqrt{\left(\frac{(LF)P_r}{P_y}\right)^2 + \left(\frac{(LF)M_{1r}}{M_{1y}}\right)^2 + \left(\frac{(LF)M_{2r}}{M_{2y}}\right)^2}$$

$$= (LF) \times \beta_r \tag{C-2.2.4-2}$$

Global, l*ocal*, and *distortional buckling* will each have different (LF) values, thus providing β_{cre}, β_{crd}, $\beta_{cr\ell}$ as required in Appendix 3.

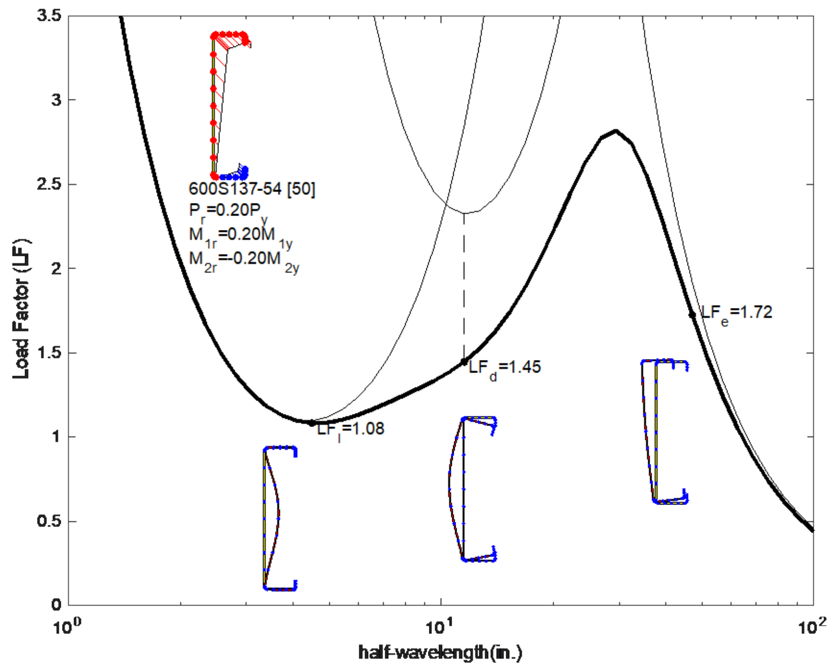


Figure C-2.2.4-1 Example of Finite Strip Analysis Results From Generalized Loading (Thin Lines Provide Decomposed Solutions per cFSM and are used to Identify Critical Lengths, see Li and Schafer (2010)).

A typical Finite Strip elastic *buckling* analysis example of a 600S137-54 [with 50 ksi *yield stress* (345 MPa)] lipped channel member under axial compression P_r = 0.20 P_y, major-axis bending M_{1r} = 0.20 M_{1y}, and minor-axis bending M_{2r} = 0.20 M_{2y} (β_r = 0.346) is provided in Figure C-2.2.4-1. The horizontal axis is the *buckling* half-wavelength, and the vertical axis is the *buckling* load factor (LF). The first minimum in the curve is *local buckling*, LF_ℓ = 1.08, *distortional buckling* is identified per the two-step procedure of Li and Schafer (2010) resulting in, LF_d = 1.45, and at an unbraced length L_b = 47.1 in. (120 cm) global *buckling*, LF_e = 1.72. Equation C-2.2.4-2 is applied to convert from LF to β_{cr} resulting in $\beta_{cr\ell}$ =1.08 × 0.346 = 0.37, β_{crd} = 1.45 × 0.346 = 0.50, β_{cre} = 1.72 × 0.346 = 0.60. These values would be used in Appendix 3.

For shell finite element models or generalized beam theory, the generalization of the elastic *buckling* response is the same as demonstrated for the Finite Strip Method. It is worth noting that identification of the *buckling* modes may be more challenging under generalized loading and the guidance provided in commentary Section 2.2.3 and depicted in Figure C-2.2.4-1 may be needed. CUFSM provides interface features to apply multiple load conditions and to understand the results in terms of β as well as formally separate the *buckling* modes.

2.2.5 Numerical Solutions - End Boundary Conditions

The semi-analytical Finite Strip Method, which is used to generate the signature curve of Figures C-2.2.2-1 and C-2.2.2-2, is based on ends that are simply supported and warping free. This is consistent with all of the plate *buckling* solutions traditionally used in the *Specification* and now provided in Appendix 1. In addition, this is consistent with the boundary conditions used for deriving global *buckling* modes in Chapter E, Chapter F, and the Analytical Solutions of Appendix 2 in the *Specification*. Global *buckling* modes can be modified to account for different end conditions using *effective length*, KL; a similar method is not available for *local* and *distortional buckling*. This is because even in a fixed end member, if the length is great enough, *local* and *distortional buckling* will be free to form in the interior of the specimen and will converge to the pinned end (warping free) solution. For *local buckling*, the length where a fixed-end solution converges to the simply supported value is only three to five times the largest characteristic dimension of the member; however, for *distortional buckling* the length is greater (see Li and Schafer, 2009). For *distortional buckling*, an approximate solution to correct the simply supported boundary conditions to account for fixed ends, developed by Moen (2008), is recommended:

$$(P_{crd})_{fixed} = D_{boost}(P_{crd})_{pinned} \qquad \text{(C-2.2.5-1)}$$

$$D_{boost} = 1 + \frac{1}{2}\left(\frac{L_{crd}}{L}\right)^2 \qquad \text{(C-2.2.5-2)}$$

where

L_{crd} = *Buckling* half-wavelength for *distortional buckling* with pinned ends

L = Unbraced length of the member with respect to *distortional buckling*

Generally, most available methods can directly model a variety of end boundary conditions. However, if the end conditions are not simply supported, the signature curve cannot be constructed, and identification can be more complicated. For finite strip analysis, CUFSM provides a solution for general end boundary conditions (Li and Schafer, 2010b), GBTUL provides a similar solution for generalized beam theory, and, of course, arbitrary end boundary conditions may be included in shell finite element models.

2.2.6 Numerical Solutions – Shear Buckling

Elastic *shear buckling* is treated as a separate *buckling* mode (despite being inextricably tied to moment gradient) and the related shear flow is provided for a lipped channel in Figure C-2.2.6-1. Conventional finite strip analysis, Generalized Beam Theory, and even some plate finite element formulations only include the destabilizing effect of longitudinal *stresses*. Therefore, the Finite Strip Method utilized in CUFSM and the conventional Generalized Beam Theory of GBTUL cannot provide a prediction for *shear buckling*.

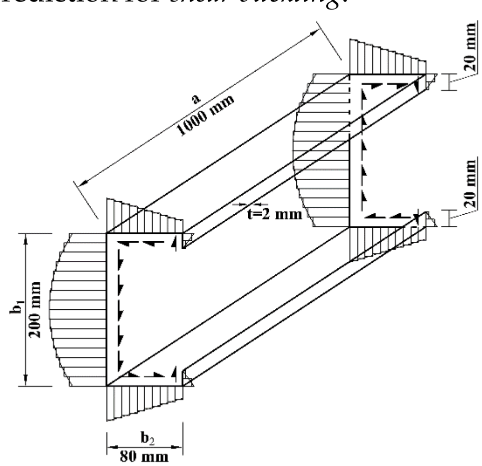
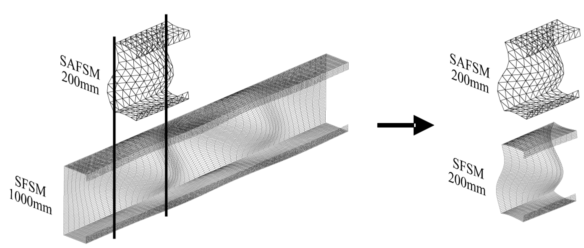


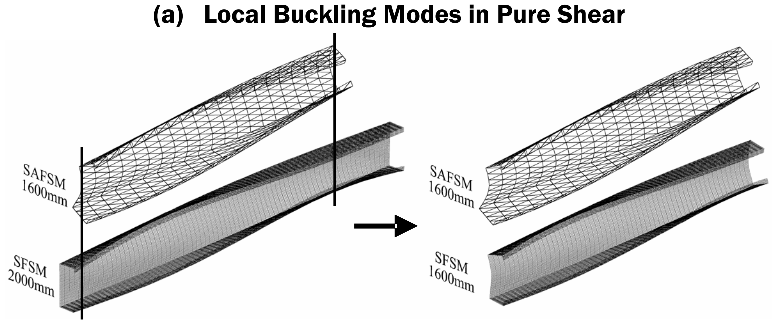
Figure C-2.2.6-1 Shear Flow Distributions in a Lipped Channel

Available numerical solutions include: (1) a generalized version of the semi-analytical Finite Strip Method (SAFSM) developed by Plank and Wittrick (1974) and implemented in Hancock and Pham (2011), (2) a new version of SAFSM which accounts for the restraint from simply supported ends in the shear mode developed by Hancock and Pham (2013), (3) the Spline Finite Strip Method (SFSM) as developed by Lau and Hancock (1986) and implemented in Pham and Hancock (2009a), or (4) shell finite element models as previously discussed.

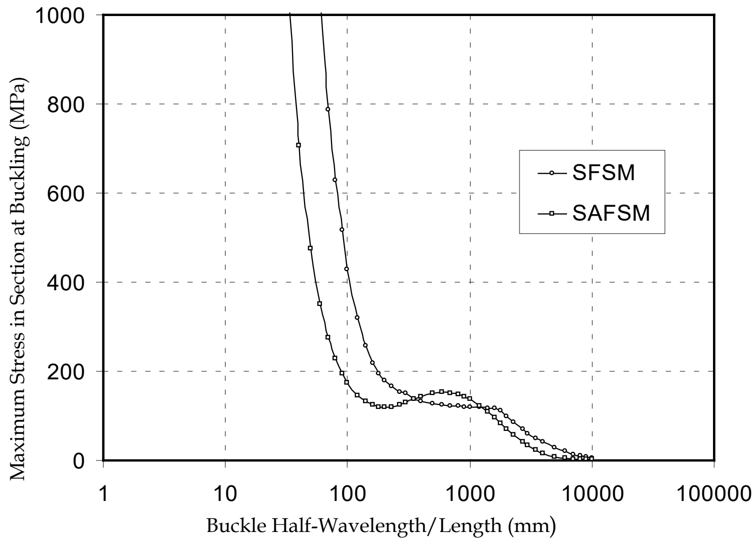
Members in pure shear can also cause *buckling* of the whole section in the form of shear *local buckling* as shown in Figure C-2.2.6-2(a) or shear *distortional buckling* as shown in Figure C-2.2.6-2(b) depending on the geometry of the section, loading, and restraint. *Shear buckling* is different from that for compression or bending in that the nodal lines are not perpendicular to the axis of the section as shown for the shear *local buckling* mode in Figure C-2.2.6-2(a). The modes shown as Semi-Analytical Finite Strip Method (SAFSM) apply to a single half-wavelength of an infinitely long section, and those designated as Spline Finite Strip Method (SFSM) apply to a section of finite length with simply supported ends. SFSM results are directly comparable to shell finite element method results. Typically, the local mode dominates at short half-wavelengths, and shear *distortional buckling* is evident at longer half-wavelengths in some instances. The *buckling stress* versus half-wavelength curves from Hancock and Pham (2011) are shown in Figure C-2.2.6-2(c). The minimum on the SAFSM curve corresponds to the value on the SFSM curve at longer half-wavelengths where end conditions do not affect the *buckling*.



(a) Local Buckling Modes in Pure Shear



(b) Distortional Buckling Modes in Pure Shear



(c) Buckling Stress vs. Half-Wavelength/Length for Plain Lipped Channel

Figure C-2.2.6-2 Examples of Shear Elastic Buckling Analysis by Spline Finite Strip Method (SFSM) Similar to Shell Finite Element Solution, and a Generalized Version of the Semi-Analytical Finite Strip Method (SAFSM)

2.2.7 Numerical Solutions – Members With Holes

Members with holes may be directly modeled using shell finite elements. Identification can be challenging, but model creation and analysis is straightforward (See Figure C-2.2.2-3(b)). Generalized Beam Theory is not well suited for handling holes in members, nor is finite strip analysis. Spline finite strip analysis has been extended to members with holes (Yao and Rasmussen, 2012), but is not generally available, nor markedly more efficient than shell finite element models.

Given the popularity of finite strip analysis, approximate numerical methods have been developed for finding the *local*, *distortional*, and global *buckling* modes of members with holes using finite strip analysis. The methods generally apply to isolated perforations/holes as found in cold-formed steel framing and related applications. Members with flanged or stiffened holes and members with patterned holes (storage racks) currently require a shell finite element model to establish the elastic *buckling* values. Work is ongoing to provide general simplified methods for these cases in the near future (Grey and Moen, 2011; Casafont et al., 2012; Smith and Moen, 2014). In general, the provided methods are complementary to the analytical methods for members with holes provided in Appendix 2 of the *Specification*.

Local Buckling of Members With Holes Using Finite Strip Analysis

Researchers have observed that holes can change the *local buckling* mode shapes of thin plates and cold-formed steel columns and beams (Kumai, 1952; Schlack, Jr., 1964; Kawai and Ohtsubo, 1968; Vann, 1971; Kesti, 2000; El-Sawy and Nazmy, 2001; Sarawit, 2003; and Moen and Schafer, 2009b). A finite strip approximate method for predicting $P_{cr\ell}$ and $M_{cr\ell}$ including the influence of holes is described in Moen and Schafer (2009c). The method assumes that *local buckling* occurs as either *buckling* of the unstiffened strip(s) adjacent to a hole at the net section or as *local buckling* of the gross section between holes. This approach is an improvement over element-based methods because the interaction between the unstiffened strip and the connected cross-section is explicitly considered. For a column with holes:

$$P_{cr\ell} = \min(P_{cr\ell nh}, P_{cr\ell h}) \qquad \text{(C-2.2.7-1)}$$

where

$P_{cr\ell nh}$ = *Local buckling* load of the gross section by a finite strip analysis

$P_{cr\ell h}$ = *Local buckling* load of the net section by a finite strip analysis (e.g., in CUFSM), but restraining the deformations to *local buckling* and examining only those *buckling* half-wavelengths shorter than the length of the hole

To calculate $P_{cr\ell h}$, a finite strip analysis of the net section is performed as shown in Figure C-2.2.7-1. To ensure a consistent comparison of $P_{cr\ell h}$ and $P_{cr\ell nh}$, the reference stress used in the net section and gross section finite strip analyses should be calculated with the same reference load (e.g., 1 kip (4.45 kN) on the net section, 1 kip (4.45 kN) on the gross section).

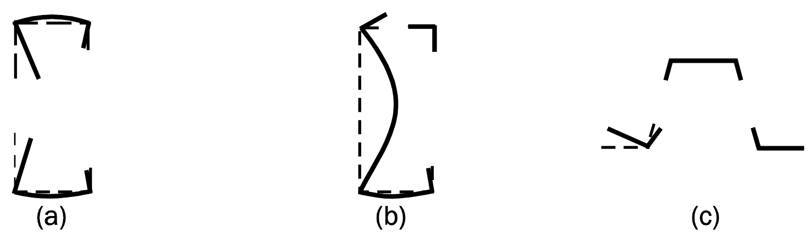
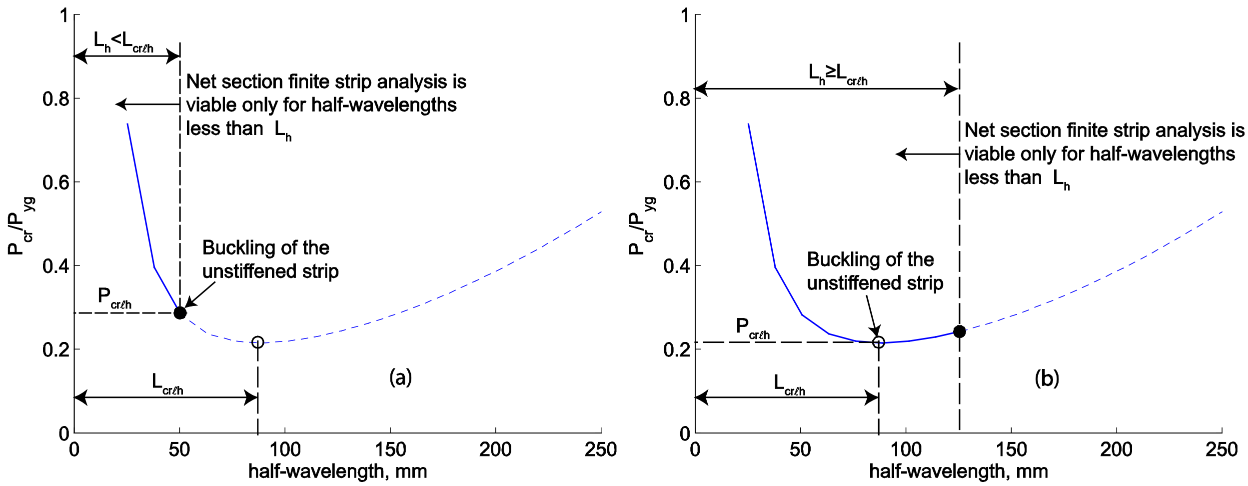


Figure C-2.2.7-1 Modeling a Column Net Cross-Section in the Finite Strip Method (e.g., CUFSM): (a) C-Section With a Web Hole, (b) C-Section With a Flange Hole, (c) Hat Section With Web Holes

Eigen-*buckling* analysis of the restrained cross-section results in an elastic *buckling* curve similar to Figure C-2.2.7-2, where the buckled half-wavelength at the minimum *buckling* load is $L_{cr\ell h}$. When the hole length, L_h, is less than $L_{cr\ell h}$, as shown in Figure C-2.2.7-2(a), $P_{cr\ell h}$ is equal to the *buckling* load for a single half-wave forming over the length of the hole. (This case is common for circular and square holes, where L_h is less than the width of the cross-sectional element containing the hole.) If $L_h \geq L_{cr\ell h}$ (Figure C-2.2.7-2(b)), $P_{cr\ell h}$ is the minimum on the *buckling* curve, corresponding to a single half-wave forming within the length of the hole. Note that use of the net cross-section for *buckling* half-wavelengths greater than L_h is conservative by failing to reflect the stiffness contributions of the gross section. Knowledge of the specific *buckling* half-wavelength of interest allows the Finite Strip Method to be extended by utilizing the net section, but only for half-waves less than the length of the hole, L_h.



(a) Hole Length Less Than $L_{cr\ell h}$ **(b) Hole Length Greater Than or Equal to $L_{cr\ell h}$**

Figure C-2.2.7-2 Local Elastic Buckling Curve of Net Cross-Section

Note: $P_{gy} = A_gF_y$ in Figure C-2.2.7-2.

The same approach described previously for columns is also applicable to beams, i.e., $M_{cr\ell} = \min(M_{cr\ell nh}, M_{cr\ell h})$. In this case, the applied reference *stress* in the finite strip analysis should be represented as a moment, i.e., 1 kip-in. (113 kN-mm) on the net section and 1 kip-in. (113 kN-mm) on the gross cross-section. See Moen and Schafer (2010b).

A similar approach is recommended for patterned type hole patterns. See Smith and Moen (2014) for additional examples and complete details.

Distortional Buckling of Members With Holes Using Finite Strip Analysis

The *distortional buckling* loads P_{crd} and M_{crd} are, at least in part, dictated by the bending *stiffness* provided by the *web* of an open cross-section as it restrains the attached *flange* from rotating (see Figures C-2.2.1-1, C-2.2.2-1 and C-2.2.2-2). If a hole with length L_h is introduced into the *web* of an open cross-section, the rotational restraint provided by the *web* is decreased, resulting in a lower critical *distortional buckling* load (Kesti, 2000; Moen and Schafer, 2009a). An approximate method for calculating P_{crd} and M_{crd} including the influence of flat-punched unstiffened *web* holes has been developed by Moen and Schafer (2009c). To implement the method, applicable for a single hole in the *web* or a single line of *web* holes longitudinally, a finite strip analysis is performed with the gross cross-section to identify the *distortional buckling* half-wavelength, L_{crd}. Then, the *web thickness* is modified from t to t_r to simulate the reduction in bending *stiffness* caused by the presence of a *web* hole, where:

$$t_r = t\left(1 - \frac{L_h}{L_{crd}}\right)^{1/3} \geq 0 \qquad \text{(C-2.2.7-2)}$$

and L_h is the length of the hole. Note that the cross-sectional *thickness* is modified over the full depth of the *web*, not just at the location of the hole in the cross-section. The *buckling* load P_{crd} or M_{crd} (including the influence of holes) is obtained with another finite strip analysis of the modified cross-section performed just at L_{crd} of the gross cross-section with the reduced thickness. The second analysis is only conducted at L_{crd} as this is the only length for which the reduced *thickness* t_r has any relevance. This finite strip elastic *buckling* simplified method is only appropriate for the case of flat-punched discrete holes in the *web* or *flange* (or both). For $L_h \geq L_{crd}$, a long hole stretches across one or more *distortional buckling* half-wavelengths. In this case it is assumed that t_r=0 because the *web* loses its ability to restrain *flange distortional buckling* deformation.

Specification Equation 2.3.3.3-1 uses L_{dh} in place of L_{crd}, where L_{dh} is the minimum of L_{crd}, L_m, and the longitudinal center-to-center hole spacing. If the distance L_m between *distortional buckling* restraints is less than L_{crd}, this half-wavelength is used instead of L_{crd}. If the hole spacing is less than the half-wavelength, more than one hole participates in the reduction of *web* stiffness.

Other *web* hole patterns are identified by Smith and Moen (2014) as different from discrete perforations in that they are smaller in size than typical punched service holes, they are more tightly spaced, they may contain more than one row of holes across the *web* in a cross-section, and they are arranged in a pattern over the length of the member. These hole patterns are common in storage rack members and they are also sometimes found in cold-formed steel framing to mitigate acoustic or thermal affects. The hole patterns often have two or more holes within a *local buckling* half-wavelength, a square region of the *web*, and *web* depth by *web* depth.

For hole patterns different from a single hole in the *web* or a single line of *web* holes longitudinally, a different *web thickness* reduction is required, specifically:

$$t_r = t\left(\frac{A_{web,net}}{A_{web,gross}}\right)^{1/3} \geq 0 \tag{C-2.2.7-3}$$

where t is the *thickness* of the *web*, $A_{web,net}$ is the net area of the *web*, and $A_{web,gross}$ is the gross area of the *web*. Since the reduction is along the full length of the member, the model, with modified *thickness*, should be completed along the full length of the member. A new finite strip analysis is conducted to find the new L_{crd} and resulting *buckling* load, P_{crd}. The model should be loaded with a reference force to account for the reduced area due to the holes. This method has been validated for compressive members and is recommended for use with flexural members as well.

Global Buckling of Members With Holes Using Finite Strip Analysis

A general approach to including the influence of holes for global *buckling* in a finite strip analysis is not available. Bending rigidities EI_x and EI_y, torsion rigidity GJ, and warping rigidity EC_w each require different reductions in the section to provide the appropriate reduced properties to account for the holes. For example, the reduced thickness needed to provide $I_{x,avg}$, J_{avg}, and $C_{w,net}$ as discussed in the *Specification* Section 2.3.1 for analytical solutions are all different—since these rigidities are typically coupled, one finite strip model cannot have two different thickness reductions. As a result, the analytical solutions of *Specification* Section 2.3.1, as developed by Moen and Schafer (2009c), are preferred, or shell finite element models may be used directly. Note that the section property calculator in CUFSM does provide a convenient means to calculate the necessary average and net properties.

2.2.8 Numerical Solutions – Bracing, Sheathing Bracing, and Attachments

Bracing and other attachments to a member (sheathing, sheeting, etc.) can have a significant impact on the elastic *buckling* force (moment, etc.) of a member. Thus, it is often desirable to include such additional elements in the elastic *buckling* analysis. The most common method is the inclusion of a spring in the member model.

For the specific case of cold-formed steel light-frame construction, e.g., a wall stud braced by sheathing, research has been conducted to determine: (a) how to characterize the semi-rigid restraints developed at the fastener-sheathing *connection* to the cold-formed steel member, and (b) how to model the elastic stability of the resulting member (Vieira and Schafer, 2013; Schafer, 2013; Peterman and Schafer, 2014). This research has resulted in the design provisions in Appendix 1 of AISI S240 (AISI, 2020), and test standards AISI S917 and S918. Central to the method is the determination of the spring *stiffnesses*, as illustrated in Figure C-2.2.8-1, to account for the restraint provided at the fastener locations. The k_x, k_y, and k_ϕ springs reflect the *stiffness* developed through deformations in the flange-fastener-sheathing system. Spring k_x represents the lateral restraint developed through bearing and tilting of the fastener against the sheathing acting as a shear *diaphragm*. Spring k_y is the out-of-plane restraint developed as composite action between the member and sheathing in major-axis bending. Spring k_ϕ is the rotational restraint developed as the flange attempts to rotate against the face of the sheathing. Due to its prominent role in restricting *distortional buckling,* k_ϕ is also discussed in *Commentary* Section 2.3.3.2.

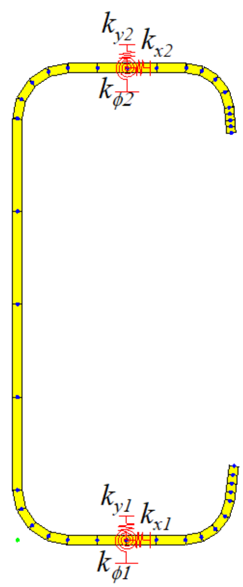


Figure C-2.2.8-1 Cold-Formed Steel Cross-Section Braced by Sheathing, Introduced as Springs at Fastener Attachment Points

Shell Finite Element Models With Bracing

It is possible to construct shell finite element models of members, fasteners, and sheathing and use these models to determine the elastic *buckling* forces and moments. However, since most of the deformations are developed locally at the fastener locations (often through damage in the sheathing material), it can be difficult to properly capture the stiffness and interactions between the components (Vieira and Schafer, 2012). However, modeling and experimentation have shown that the complex member-fastener-sheathing interaction can be simplified to a series of springs at the fastener locations as indicated in Figure C-2.2.8-1.

Shell finite element models of the member with springs added at the fastener locations can provide an accurate prediction of the elastic critical loads and moments; see Vieira (2011) or Post (2012). In addition, such models readily allow for a mixture of discrete bracing (springs) and sheathing-based springs. In that sense, this approach is the most general approach.

However, startup time for developing and analyzing such models is relatively significant. Further, identification of the individual *local*, *distortional*, and global *buckling* modes must be done visually, and can be time-consuming. As a result, the finite strip method or other methods are generally preferred, where applicable.

Finite Strip Models With Bracing

Finite strip models of members with springs may be completed in CUFSM (Vieira and Schafer, 2013; Peterman and Schafer, 2014). However, it is important to note that the springs in CUFSM are continuous springs (also called foundation springs), i.e., continuous along the length of the member, not discrete at the fastener locations. Conversion of discrete k_x, k_y, and k_ϕ springs to continuous $\bar{k}_x$, $\bar{k}_y$, and $\bar{k}_\phi$ springs only requires dividing the discrete springs by the fastener spacing. For practical fastener spacing, the approach has been found to work well.

Finite Strip Models With Bracing – Local Buckling

Local buckling has a short wavelength, thus sheathing bracing typically has little impact on *local buckling,* and it is recommended to ignore any bracing restraint for *local buckling*. Theoretically, k_x and k_ϕ (if located at the exact mid-width of the flange) have no influence on *local buckling*, only k_y. However, if the procedures of AISI S240 Appendix 1 (AISI, 2020) are applied, the out-of-plane stiffness, k_y, is derived consistent with global bending resistance and not localized resistance and, therefore, should not be included in a *local buckling analysis*.

Finite Strip Models With Bracing – Distortional Buckling

Sheathing provides beneficial rotational restraint against *distortional buckling*, and k_ϕ should be included when determining the elastic *distortional buckling* force or moment. For studs with deep *webs* (and narrow flanges), the additional restraint supplied by k_x may also be beneficial and optionally may be included. Stiffness k_y should not be included when determining *distortional buckling*. According to AISI S240 Appendix 1, k_y's deformations are consistent with strong-axis member flexure, not rotation of the flange. Further, k_ϕ already accounts for the moment couple that develops between k_y at the fastener and bearing between the flange and sheathing. The use of the smeared continuous stiffness ($\bar{k}_\phi$ as opposed to k_ϕ) in the prediction of *distortional buckling* has been shown to be adequate for a large variety of members with fastener spacing of 12 in. (305 mm) (Schafer, 2013). In general, the fastener spacing (s_f) should be less than the *distortional buckling* half-wavelength (L_{crd}), and it is recommended that $s_f/L_{crd} \le 0.5$ for the use of the smeared continuous stiffness. Otherwise, the bracing should be ignored, or a model capable of accounting for the discrete spacing (e.g., shell finite element model) should be employed.

Finite Strip Models With Bracing – Global Buckling

Sheathing greatly influences the global *buckling* force or moment of a member. For determining P_{cre} or M_{cre} inclusion of all available fastener-sheathing springs (k_x k_y k_ϕ) is recommended, but k_x is critical as it provides the primary fastener-sheathing restraint for both weak-axis flexure and torsion (when present on both flanges). The use of smeared continuous stiffness as opposed to discrete springs in the elastic *buckling* prediction has been shown to be adequate when the fastener spacing (s_f) is less than the global *buckling* half-wavelength (L_{cre}). Specifically, it is recommended that $s_f/L_{cre} \leq 0.25$ in Post (2012). Otherwise, the bracing should be ignored, or a model capable of accounting for the discrete spacing (e.g., shell finite element model, or beam model with discrete springs) should be employed.

2.2.9 Numerical Solutions – Moment Gradient or Stress Gradient

Moment gradient influences the elastic *buckling* of a section. For shell finite element models, it is possible to explicitly model the loading conditions and include moment gradient. For Generalized Beam Theory, inclusion of moment gradient is also possible and is available in Version 2 of GBTUL.

Finite strip analysis typically does not include moment gradient (a constant moment is assumed). For *local buckling*, due to the short half-wavelength of the *buckling* mode, moment gradient only has a minor influence, and no correction needs to be made. For *distortional buckling*, the moment gradient will increase the *buckling* moment, and β of *Specification* Equation 2.3.3.2-2 may be applied to increase the result from a finite strip analysis. For global *buckling*, the moment gradient is also beneficial, and C_b of *Specification* Equation 2.3.1-8 may be applied.

2.2.10 Numerical Solutions—Members With Variation Along Length

Shell finite element models are best suited for handling unusual members with significant variation along the length. In some cases, conservative simplifications using finite strip analysis or Generalized Beam Theory are possible.

2.2.11 Numerical Solutions – Built-Up Sections and Assemblages

Elastic *buckling* of built-up sections may be explicitly considered with shell finite element models. Care must be taken to ensure the end boundary conditions are realistic and that appropriate stiffness is selected for the attachments between members. Finite strip analysis may be used if it is appropriate to smear the attachments along the length of the member—see Schafer (2013) for a related discussion. Research is underway to develop improved elastic *buckling* prediction methods for built-up sections.

In some cases, it is both possible and desirable to treat an assemblage as a member—such as trusses, wall panels, and floor systems—for elastic *buckling* determination. Common practice is to model such assemblages with traditional beam finite elements. Care must be taken with this approach, since *local*, *distortional*, and often *flexural-torsional buckling* are not present in typical beam element models. Secondary models will be required to capture these *buckling* modes. Shell finite element models do provide a means to include complete assemblage information, but with added complexity.

2.3 Analytical Solutions

Specification Section 2.3 provides analytical solutions for elastic *buckling* of cold-formed steel members. In 2022, analytical solutions for typical cross-sections were consolidated into this section from *Specification* Sections E2 and F2.1. Additional analytical solutions may be found in the SSRC Guide (Ziemian, 2010), the *Direct Strength Method Design Guide* (AISI, 2006), as well as other reference texts (Allen and Bulson, 1980; Chajes, 1974; and Timoshenko and Gere, 1961). The use of alternative analytical formulae for elastic *buckling* determination falls under the *rational engineering analysis* clause of Chapter A.

Many of the analytical solutions provided are relatively complex due to the lack of symmetry and the thin-walled nature of typical cold-formed steel members. In general, numerical solutions, as detailed in *Specification* Section 2.2, can provide efficient predictions for arbitrary cross-sections, boundary conditions, and loading conditions, and thus are recommended whenever practical. Also, for common sections, elastic *buckling* solutions are tabulated in the AISI *Cold-Formed Steel Design Manual* (AISI, 2017) and in CFSEI Tech Note G103-11 (Li and Schafer, 2011).

2.3.1 Global Buckling

A. General Solution

The global equilibrium of a member subjected to compression and/or bending must consider three types of *buckling*: flexure about the x-axis, flexure about the y-axis, and torsion. Derivation of the *buckling* loads for equal unbraced lengths using principal axes is provided in Timoshenko and Gere (1961), and other common reference texts (e.g., Yu and LaBoube, 2010). This derivation involves three simultaneous equations of equilibrium. The solution for a general beam-column can be represented as a 3x3 matrix where the determinant is equated to zero.

$$\begin{vmatrix} P_{ey}-P & 0 & -Py_o+M_x \\ 0 & P_{ex}-P & Px_o-M_y \\ -Py_o+M_x & Px_o-M_y & (P_t-P)r_o^2-2\beta_x M_x-2\beta_y M_y \end{vmatrix}=0 \qquad \text{(C-2.3.1-1)}$$

In the above equation, P_{ex}, P_{ey}, and P_t are axial forces for *buckling* about the x-axis, y-axis, and torsion, respectively, x_o and y_o are the coordinates of the shear center, r_o is the radius of gyration about the shear center, β_x and β_y are asymmetry properties of the cross-section, and P, M_x, and M_y are the axial force and moments applied to the member.

The *Specification* treats compression and bending separately, which are special cases of the general solution for global *buckling*. Section 2.3.1.1 addresses compression, where M_x = M_y = 0, and Section 2.3.1.2 addresses bending about one geometric axis, where P = 0 and either M_x = 0 or M_y = 0.

B. Influence of Holes

Specification Section 2.3.1 also provides a method of calculating reduced global *buckling* forces and moments using modified section properties for members with holes. Other methods which properly account for the effect of holes may be used when determining P_{cre} or M_{cre} by numerical methods.

The global *flexural buckling* load decreases when holes are present (Sarawit, 2003; Moen and Schafer, 2009a). This is due to a reduction in the bending rigidity, EI, with the presence of the holes. The weighted average property approach to determination of the moment of inertia as used in *Specification* Section 2.3.1 has been shown to provide sufficient accuracy when compared with numerical solutions (Moen and Schafer, 2009c). The calculation of average moment of inertia was refined to accurately handle uniform hole spacing, based on a study by Glauz (2018). This form uses the ratio of hole length to hole spacing, which easily accommodates non-uniform hole lengths and spacings by conservatively using the worst-case scenario.

If the holes are not equal size or uniformly spaced, a more precise approximation of I_{avg}

can be employed where:

$$I_{avg} = I_g - \sum_{j=1}^{n}(I_g - I_{net,j})\left(1-\cos\frac{2\pi c_j}{L}\right)\frac{L_{h,j}}{L} \quad \text{(C-2.3.1-2)}$$

where

$L_{h,j}$ = Length of hole or net section region, j

c_j = Distance from end of *unbraced length* to hole centerline or net section region; see Figure C-2.3.1-1

All other variables are defined in *Specification* Section 2.3.1.

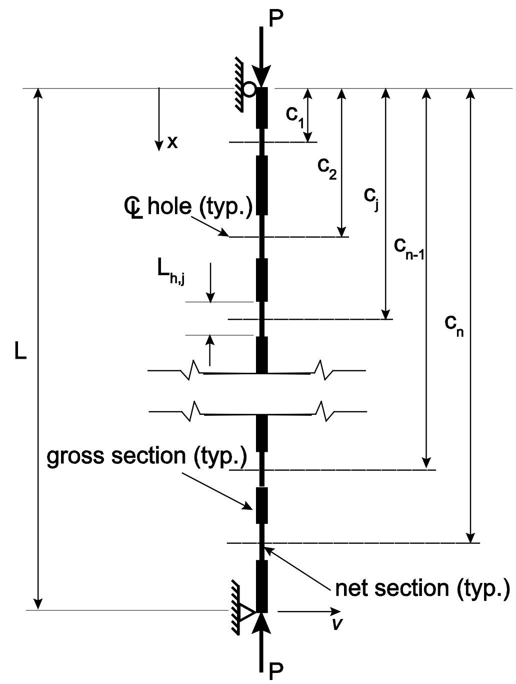


Figure C-2.3.1-1 A Column With j = 1, 2, ..., n Holes or Net Section Regions

The weighted average property approach for *flexural buckling* can be extended to *flexural-torsional buckling* and *lateral-torsional buckling* as described in Moen and Schafer (2009c). The key extensions are the determination of the influence of holes on torsion rigidities: GJ and EC_w and the distances between the shear center and centroid, x_o and y_o along the corresponding principal axes, and related polar radius of gyration r_o. The form of the weighted average property employed for flexural rigidity EI is found to also work for GJ, and a J_{avg} approximation is provided in *Specification* Section 2.3.1 as well. Similar weighted average property approximations are provided for x_o, y_o, and r_o. The warping torsion rigidity, EC_w, does not follow the weighted average property approximation, as the presence of holes prevents warping resistance from fully developing (Moen and Schafer, 2009c). A viable approximation for warping *stiffness* at the net section is $EC_{w,net}$.

Calculation of the torsional properties J, C_w, x_o, and y_o is explained in *Commentary* Section 2.3.1.1.4. To determine these properties for net sections, the same approach is used but with the thickness, t, set to 0 at the hole locations. The total area of the cross-section, A, used in the calculation of $C_{w,net}$ should be the net area of the cross-section.

Some care must be exercised in the use of average vs. *gross area*. The *buckling* load is derived based on the cross-section rigidities: EI, GJ, and EC_w and the *buckling* load is independent of the *cross-sectional area*. Therefore, conversion to *buckling stress* uses the gross *cross-sectional area* if the rigidities have been properly reduced to account for holes. Average *cross-sectional area* is only necessary for calculating the radius of gyration since this quantity is directly tied to the rigidities.

If holes are not of equal size or uniformly spaced, a more precise approximation of A_{avg}, $x_{o,avg}$, $y_{o,avg}$, and $I_{xy,avg}$ can use the same general form as Equation C-2.3.1-2. However, J_{avg} should use a slightly different form due to the relationship between angle of twist and strain energy for pure torsion:

$$J_{avg} = J_g - \sum_{j=1}^{n}(J_g - J_{net,j})\left(1 + \cos\frac{2\pi c_j}{L}\right)\frac{L_{h,j}}{L} \qquad \text{(C-2.3.1-3)}$$

Note that all net section properties, i.e., $I_{x,net}$, $I_{y,net}$, A_{net}, $x_{o,net}$, $y_{o,net}$, J_{net}, and $C_{w,net}$, can be readily calculated with the built-in section property calculator in the freely available open source program CUFSM (Schafer and Ádány, 2006) by setting the element thicknesses to zero at the holes. See Moen and Schafer (2010a).

Within the limitations of the hole size given in Appendix 1 Sections 1.1.1 and 1.1.3, the hole influence on the global *buckling stress* is negligible when using the *Effective Width Method*; therefore, an exception is provided to exclude these cases from the additional requirements of Appendix 2.

2.3.1.1 Global Buckling for Compression Members (F_{cre}, P_{cre})

A. General Solution

The general form of the *buckling* solution shown in Equation C-2.3.1-1 assumes equal unbraced lengths (K_xL_x=K_yL_y=K_tL_t) and uses principal x and y axes. It is common for compression members to have different unbraced lengths, and sometimes the bracing directions do not align with the principal axes. A more general form of the solution for a non-symmetric compression member with unequal unbraced lengths oriented to non-principal axes is provided in Glauz (2017), where the 3×3 matrix is given as:

$$\begin{vmatrix} P_{fy} - P & P_{fxy} & -Py_o \\ P_{fxy} & P_{fx} - P & Px_o \\ -Py_o\left(\dfrac{K_tL_t}{K_fL_f}\right)^2 & Px_o\left(\dfrac{K_tL_t}{K_fL_f}\right)^2 & (P_t - P)r_o^2 \end{vmatrix} = 0 \qquad \text{(C-2.3.1.1-1)}$$

The subscript, f, indicates a coupled flexural mode, where *buckling* about the x and y axes occurs together in one direction with a common half-wavelength, K_fL_f. P_{fx} and P_{fy} represent the axial forces for *flexural buckling* about the x and y axes, respectively. A new term, P_{fxy}, is required for handling non-principal axes. Determining the axial compression *buckling* force, P, requires solving a cubic equation, which has potentially three different roots. The lowest root represents the controlling *buckling* force. This general form is discussed in more detail in *Commentary* Section 2.3.1.1.4 for *non-symmetric sections*.

Cross-sections with symmetry are special cases where the flexural modes are uncoupled and the roots of the cubic equation are more directly solved. These are the formulae for global *flexural*, *torsional*, and *flexural-torsional buckling* provided in *Specification* Sections 2.3.1.1.1 to 2.3.1.1.3, and are discussed further in the corresponding sections of the *Commentary*.

B. Effective Length Factor, K

The *effective length factor*, K, accounts for the influence of restraint against rotation and translation at the ends of a column on its *load*-carrying capacity. For the simplest case, a column with both ends hinged and braced against lateral translation, *buckling* occurs in a single half-wave and the *effective length* KL, being the length of this half-wave, is equal to the actual physical length of the column (Figure C-2.3.1.1-1); correspondingly, for this case, K = 1. This situation is approached if a given compression member is part of a structure which is braced in such a manner that no lateral translation (sidesway) of one end of the column relative to the other can occur. This is so for columns or studs in a structure with diagonal bracing, diaphragm bracing, shear-wall construction or any other provision which prevents horizontal displacement of the upper relative to the lower column ends. In these situations, it is safe and only slightly, if at all, conservative to take K = 1.

If translation is prevented and abutting members (including foundations) at one or both ends of the member are rigidly connected to the column in a manner which provides substantial restraint against rotation, K-values smaller than 1 (one) are sometimes justified. Table C-2.3.1.1-1 provides the theoretical K-values for six idealized conditions in which *joint* rotation and translation are either fully realized or nonexistent. The same table also includes the K-values recommended by the Structural Stability Research Council for design use (Galambos, 1998).

In trusses, the intersection of members provides rotational restraint to the compression members at *service loads*. As the collapse *load* is approached, the member *stresses* approach the *yield stress*, which greatly reduces the restraint they can provide. For this reason, K-value is usually taken as unity regardless of whether they are welded, bolted, or connected by screws. However, when sheathing is attached directly to the top *flange* of a continuous compression chord, research (Harper, LaBoube and Yu, 1995) has shown that the K-values may be taken as 0.75 (AISI, 1995).

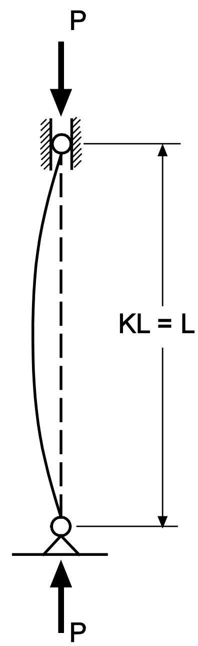
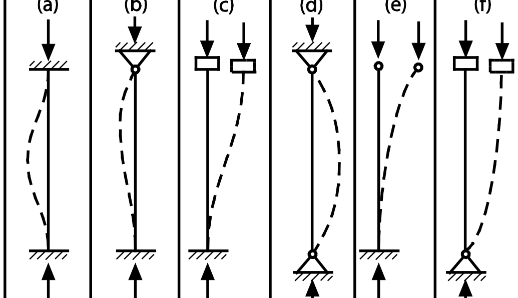
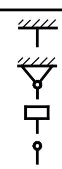


Figure C-2.3.1.1-1 Overall Column Buckling

Table C-2.3.1.1-1
Effective Length Factors K for Concentrically Loaded Compression Members

	(a)	(b)	(c)	(d)	(e)	(f)
Buckled shape of column is shown by dashed line						
Theoretical K value	0.5	0.7	1.0	1.0	2.0	2.0
Recommended K value when ideal conditions are approximated	0.65	0.80	1.2	1.0	2.10	2.0
End condition code		Rotation fixed, Translation fixed Rotation free, Translation fixed Rotation fixed, Translation free Rotation free, Translation free				

On the other hand, when no lateral bracing against sidesway is present, such as in the portal frame of Figure C-2.3.1.1-2, the structure depends on its own bending *stiffness* for lateral stability. In this case, when failure occurs by *buckling* of the columns, it invariably takes place by the sidesway motion shown. This occurs at a lower *load* than the columns would be able to carry if they were braced against sidesway, and the figure shows that the half-wavelength into which the columns buckle is longer than the actual column length. Hence, in this case K is larger than 1 (one) and its value can be read from the graph of Figure C-2.3.1.1-3 (Winter, et al., 1948a and Winter, 1970). Since column bases are rarely either actually hinged or completely fixed, K-values between the two curves should be estimated depending on actual base fixity.

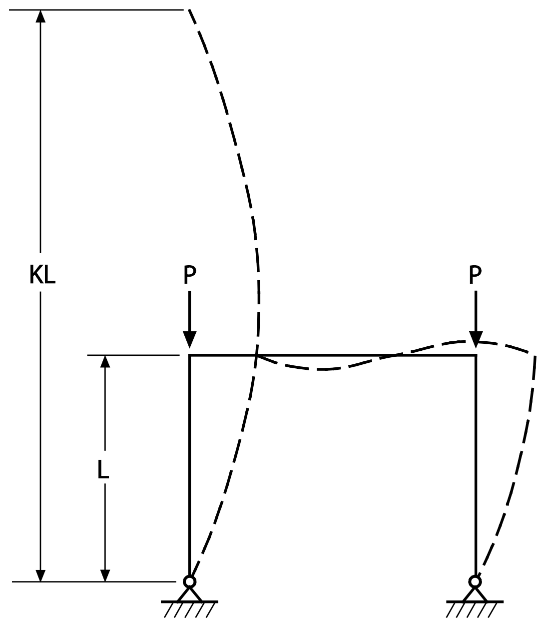


Figure C-2.3.1.1-2 Laterally Unbraced Portal Frame

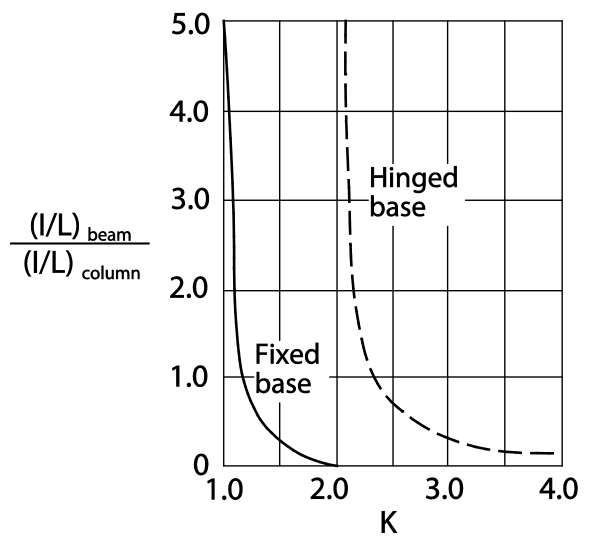


Figure C-2.3.1.1-3 Effective Length Factor K in Laterally Unbraced Portal Frames

Figure C-2.3.1.1-3 can also serve as a guide for estimating K for other simple situations. For multi-bay and/or multi-story frames, simple alignment charts for determining K are given in the AISC Commentaries (AISC, 1989, 1999, 2005). For additional information on frame *stability* and *second-order effects*, see SSRC *Guide to Stability Design Criteria for Metal Structures* (Galambos, 1998) and the AISC Specifications and Commentaries.

If roof or floor slabs, anchored to *shear walls* or vertical plane bracing systems, are counted upon to provide lateral support for individual columns in a building system, their stiffness must be considered when functioning as horizontal *diaphragms* (Winter, 1958a).

2.3.1.1.1 Sections Not Subject to Torsional or Flexural-Torsional Buckling

A slender, axially loaded column may fail by overall *flexural buckling* if the cross-section of the column is a *doubly-symmetric* shape, closed shape (square or rectangular tube), cylindrical shape, or point-symmetric shape. For *singly-symmetric* shapes, *flexural buckling* is one of the possible failure modes. Wall studs connected with sheathing material can also fail by *flexural buckling*.

The elastic critical *buckling* load for a long column can be determined by the following Euler equation:

$$P_{cre} = \frac{\pi^2 EI}{(KL)^2} \qquad \text{(C-2.3.1.1.1-1)}$$

where P_{cre} is the column *buckling* load in the elastic range, E is the modulus of elasticity, I is the moment of inertia, K is the *effective length factor*, and L is the *unbraced length*. Accordingly, the elastic column *buckling stress* is

$$F_{cre} = \frac{P_{cre}}{A_g} = \frac{\pi^2 E}{(KL/r)^2} \qquad \text{(C-2.3.1.1.1-2)}$$

in which r is the radius of gyration of the full cross-section, and KL/r is the effective slenderness ratio.

If the *unbraced length* is the same for *buckling* about any axis, *buckling* will occur about the minor principal axis. If the *unbraced lengths* are different for *buckling* about different axes, all possible axes of *buckling* must be considered, and the one with the largest slenderness ratio controls. If bracing directions do not align with the principal axes of the cross-section, *buckling* may occur about the minor principal axis, where the controlling span has the largest distance between adjacent brace locations, regardless of brace direction.

2.3.1.1.2 Singly-Symmetric Sections Subject to Flexural-Torsional Buckling

For *singly-symmetric* shapes such as channels, hat sections, angles, T-sections, and I-sections with unequal *flanges*, for which the shear center and centroid do not coincide, *flexural-torsional buckling* is one of the possible *buckling* modes as shown in Figure C-2.3.1.1.2-1. *Non-symmetric sections* will always buckle in the flexural-torsional mode.

It should be emphasized that one needs to design for *flexural-torsional buckling* only when it is physically possible for such *buckling* to occur. This means that if a member is so connected to other parts of the structure, such as wall sheathing, that it can only bend but cannot twist, it needs to be designed for *flexural buckling* only. This may hold for the entire member or for individual parts. For instance, a channel member in a wall or the chord of a roof truss is easily connected to *girts* or *purlins* in a manner which prevents twisting at these connection points. In this case, *flexural-torsional buckling* needs to be checked only for the *unbraced lengths* between such *connections*. Likewise, a *doubly-symmetric* compression member can be made up by connecting two spaced channels at intervals by batten plates. In this case, each channel constitutes an "intermittently fastened component of a built-up shape." Here the entire member, being *doubly-symmetric*, is not subject to *flexural-torsional buckling* so this mode needs to be checked only for the individual component channels between batten *connections* (Winter, 1970).

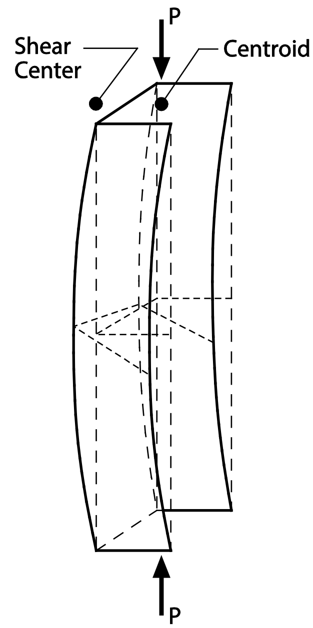


Figure C-2.3.1.1.2-1 Flexural-Torsional Buckling of a Channel in Axial Compression

The governing elastic *flexural-torsional buckling* load of a column can be found from the following equation (Chajes and Winter, 1965; Chajes, Fang and Winter, 1966; Yu and LaBoube, 2010; Glauz, 2017), which can be derived from the general form in Equation C-2.3.1.1-1 for flexure about the x-axis combined with torsion where $y_o = 0$, $P_{fxy} = 0$, and $P_{fx} = P_{ex}$:

$$P_{cre} = \frac{1}{2\beta}\left[\left(P_{ex} + P_t\right) - \sqrt{\left(P_{ex} + P_t\right)^2 - 4\beta P_{ex} P_t}\right] \qquad \text{(C-2.3.1.1.2-1)}$$

For this equation, the x-axis is the axis of symmetry; $P_{ex} = \pi^2 EI_x/(K_xL_x)^2$ is the axial force for Euler *buckling* about the x-axis; P_t is the axial force for *torsional buckling* (Equation C-2.3.1.1.3-1); and $\beta=1-(x_o/r_o)^2(K_tL_t/K_xL_x)^2$. It is worth noting that the *flexural-torsional buckling* force is always lower than the Euler *buckling* force P_{ex} for *flexural buckling* about the symmetry axis. Hence, for these *singly-symmetric sections*, *flexural buckling* can only occur, if at all, about the y-axis, which is the principal axis perpendicular to the axis of symmetry.

Intermediate braces can be used to reduce the torsional *unbraced length*. As investigated in Glauz (2017), *flexural-torsional buckling* modes with different *unbraced lengths* experience more complex interaction between flexure and torsion. If a member is restrained against twisting at a point within the *flexural buckling* half-wavelength, *torsional buckling* undergoes reverse curvature such that warping is also restrained at the brace. An *effective length factor*, K_t, equal to 0.7 has been shown to properly account for this condition. Conversely, if the *unbraced length* for twisting is greater than the *effective length* for flexure, the direction of flexure dictates the direction of twist, and therefore the *effective length* for twisting is equal to the *effective length* for flexure. In both cases, the result is a shorter *effective length* for twisting, which increases P_t and decreases the impact of the shear center offset. These reductions in the *effective length* for twisting should be used with care to ensure applicability, and a separate check must be made for pure *torsional buckling* without this reduction in *effective length* using the following general equation:

$$\left(\frac{y_o^2}{r_o^2}I_x+\frac{x_o^2}{r_o^2}I_y+2\frac{x_o y_o}{r_o^2}I_{xy}\right)P_{cre}^2+\frac{\pi^2 E}{(K_t L_t)^2}\left(I_x I_y-I_{xy}^2\right)\left(P_{cre}-P_t\right)=0 \qquad \text{(C-2.3.1.1.2-2)}$$

An inspection of Equation C-2.3.1.1.2-1 will show that in order to calculate β and P_t, it is necessary to determine x_o = distance between shear center and centroid, J = Saint-Venant torsion constant, and C_w = warping constant, in addition to several other, more familiar cross-sectional properties. Because of these complexities, the calculation of the *flexural-torsional buckling stress* cannot be made as simple as that for *flexural buckling*. Formulas for typical C-sections, Z-sections, angle and hat sections are provided in Part I of the AISI *Design Manual* (AISI, 2017).

The following simplified equation provides a conservative approximation for the elastic *flexural-torsional buckling* force:

$$P_{cre}=\frac{P_{ex}P_t}{P_{ex}+P_t} \qquad \text{(C-2.3.1.1.2-3)}$$

The above equation is based on the following interaction relationship given by Peköz and Winter (1969a), which is equivalent to using β = 0:

$$\frac{1}{P_{cre}}=\frac{1}{P_{ex}}+\frac{1}{P_t} \qquad \text{(C-2.3.1.1.2-4)}$$

2.3.1.1.3 Doubly- or Point-Symmetric Sections Subject to Torsional Buckling

Pure *torsional buckling*, i.e., failure by sudden twist without concurrent bending, is also possible for certain cold-formed open shapes. These are all point-symmetric shapes, such as *doubly-symmetric* I-shapes, point-symmetric Z-shapes, and such unusual sections as cruciforms, swastikas, and the like. Since the shear center coincides with the centroid, *flexural buckling* and *torsional buckling* occur independently. Under concentric *load*, *torsional buckling* of such shapes very rarely governs design. This is so because such members of realistic slenderness will buckle flexurally or by a combination of flexural and *local buckling* at *loads* smaller than those which would produce *torsional buckling*. However, for relatively short members of this type, carefully dimensioned to minimize *local buckling*, such *torsional buckling* cannot be completely ruled out. If such *buckling* is elastic, it occurs at the critical *stress*, σ_t, calculated as follows (Winter, 1970):

$$P_t=\frac{1}{r_o^2}\left[GJ+\frac{\pi^2 EC_w}{(K_t L_t)^2}\right] \qquad \text{(C-2.3.1.1.3-1)}$$

The above equation is the same as *Specification* Equation 2.3.1-3, in which r_o is the polar radius of gyration of the cross-section about the shear center, G is the shear modulus, J is

Saint-Venant torsion constant of the cross-section, E is the modulus of elasticity, C_w is the torsional warping constant of the cross-section, and K_tL_t is the *effective length* for twisting.

The *effective length factor* for twisting, K_t, accounts for the influence of restraints similar to flexural *effective length factors*. Figure C-2.3.1.1.3-1 illustrates that torsion restraints may be a combination of twisting and warping fixity. The torsional end conditions affect the *torsional buckling* half-wavelength as demonstrated in Timoshenko and Gere (1961). These investigations reveal that twisting restraints for torsion behave similar to translation restraints for flexure (simple supports), and warping restraints for torsion behave similar to rotation restraints for flexure. Therefore, Table C-2.3.1.1-1 can be used to determine K_t by associating twisting end restraints with translation, and warping end restraints with rotation.

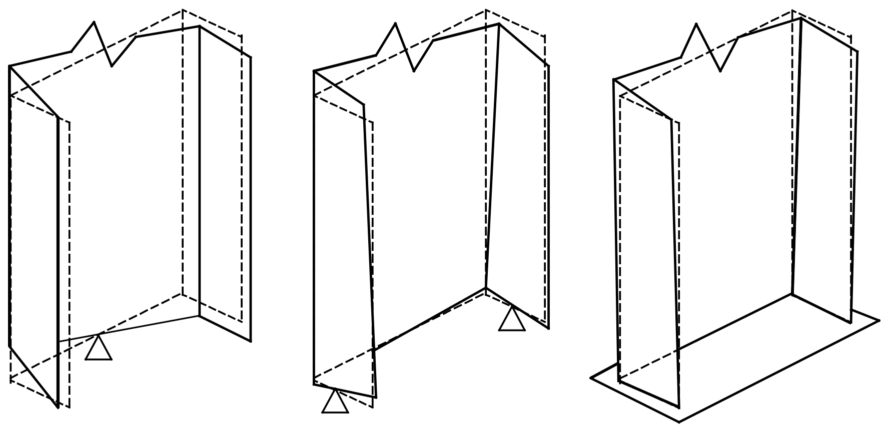


Figure C-2.3.1.1.3-1 Torsional End Conditions

For example, a column which has twisting fixed at both ends and warping free, is similar to a pinned end column for flexure; therefore, $K_t = 1.0$. If this column has warping fixed at one end, the theoretical K_t is 0.7 and the recommended K_t is 0.8. If this column has warping fixed at both ends, the theoretical K_t is 0.5 and the recommended K_t is 0.65.

2.3.1.1.4 Non-Symmetric Sections

For a general *non-symmetric section*, *flexural-torsional buckling* may either occur with flexure perpendicular to the x- or y-bracing directions, or with coupled flexure about a rotated axis, combined with torsion. Therefore, three different equations are required to capture all possible *buckling* modes.

Specification Equations 2.3.1.1.4-1 and 2.3.1.1.4-2 represent *flexural-torsional buckling* about the axes perpendicular to the bracing directions, and are equivalent to the *flexural-torsional buckling* equation provided in *Specification* Section 2.3.1.1.2. *Specification* Equation 2.3.1.1.4-3 is the cubic equation resulting from expansion of the determinant in Equation C-2.3.1.1-1. The lowest root of this equation represents the *flexural-torsional buckling* mode involving flexure about both the x and y axes.

The advantage of the provided formulae is that they are applicable to any cross-section including those covered in *Specification* Sections 2.3.1.1.1 to 2.3.1.1.3. Therefore, if programmed, they provide a general solution. The disadvantage of the formulae is that they are complex. Roots of a cubic equation are required as are torsional cross-section properties that may not be commonly available. The AISI *Cold-Formed Steel Design Manual* (AISI, 2017) provides examples for calculation of these cross-section properties. In general, the torsion-related cross-section properties may be found from the following:

J = Saint Venant torsion constant of the cross-section, in.^{4} (mm^4)

$$= \frac{1}{3}(\ell_1 t_1^3 + \ell_2 t_2^3 + \ldots + \ell_n t_n^3) \quad \text{(C-2.3.1.1.4-1)}$$

C_w = Warping constant of torsion of the cross section, in.^{6} (mm^6)

$$= \int_0^{\ell} \omega_o^2 t\,ds - \frac{1}{A}\left(\int_0^{\ell} \omega_o t\,ds\right)^2 \quad \text{(C-2.3.1.1.4-2)}$$

x_o = Distance from centroid to shear center along the principal x-axis, in. (mm)

$$= \frac{1}{I_x}\int_0^{\ell} \omega_c y t\,ds \quad \text{(C-2.3.1.1.4-3)}$$

y_o = Distance from centroid to shear center along the principal y-axis, in. (mm)

$$= -\frac{1}{I_y}\int_0^{\ell} \omega_c x t\,ds \quad \text{(C-2.3.1.1.4-4)}$$

ω_c = Sectorial coordinate based on the centroid, in.^{2} (mm^2)

$$= \int_0^{s} R_c\,ds \quad \text{(C-2.3.1.1.4-5)}$$

ω_o = Sectorial coordinate based on the shear center, in.^{2} (mm^2)

$$= \int_0^{s} R_o\,ds \quad \text{(C-2.3.1.1.4-6)}$$

where

ℓ_i = Length of cross-section middle line of segment i, in. (mm)

t_i = Wall thickness of segment i, in. (mm)

ℓ = Total length of middle line of cross-section, in. (mm)

$$= \sum_0^{n} \ell_i \quad \text{(C-2.3.1.1.4-7)}$$

s = Distance measured along middle line of cross-section from one end to Point P (See Figure C-2.3.1.1.4-1), in. (mm)

A = Total area of cross-section, in.^{2} (mm^2)

x, y = Coordinates of principal coordinate system, measured from centroid of any point P along middle line of cross-section, in. (mm)

I_x, I_y = Centroidal moment of inertia of cross-section about principal x- and y-axes, in.^{4} (mm^4)

R_c, R_o = Perpendicular distances from centroid (C.G.) and shear center (S.C.), respectively, to middle line at Point P, in. R_c or R_o is positive if a vector tangent to the middle line at P in the direction of increasing s has a counter-clockwise moment about C.G. or S.C. as shown in Figure C-2.3.1.1.4-1, in. (mm)

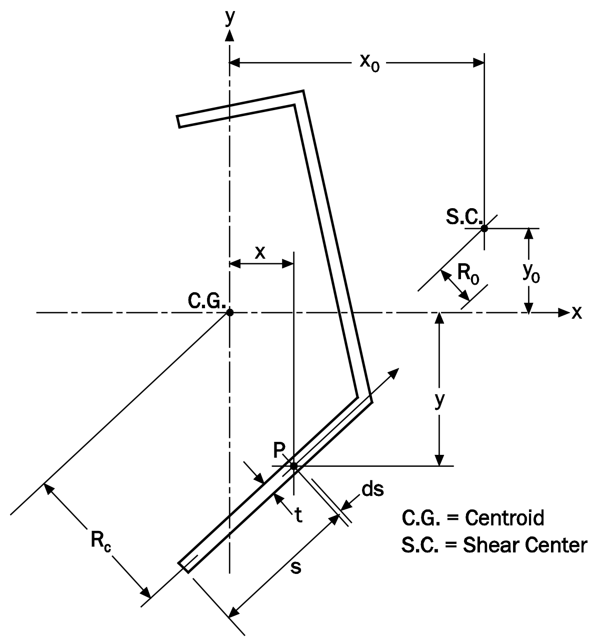


Figure C-2.3.1.1.4-1 Non-Symmetric Cross-Section

2.3.1.2 Global Buckling for Flexural Members (F_{cre}, M_{cre})

A. General Solution

The global (*lateral-torsional*) *buckling* moment about a principal axis can be determined from the general form of the *buckling* solution in Equation C-2.3.1-1. For bending about the principal x-axis, P = 0, M_y = 0, and the *buckling* solution reduces to the following quadratic equation:

$$M_{cre}^2 + 2\beta_x P_{ey} M_{cre} - r_o^2 P_{ey} P_t = 0 \qquad \text{(C-2.3.1.2-1)}$$

For sections with symmetry and other special cases, the equations for *lateral-torsional buckling* are provided in *Specification* Sections 2.3.1.2.1 to 2.3.1.2.5 and are discussed further in the corresponding sections of the *Commentary*.

B. Bending Coefficient, C_b

Bending coefficient, C_b, is applied to the critical elastic *buckling* moment, M_{cre}, to account for nonuniform bending. C_b can be determined as follows:

$$C_b = 1.75 + 1.05\,(M_1/M_2) + 0.3\,(M_1/M_2)^2 \le 2.3 \qquad \text{(C-2.3.1.2-2)}$$

in which M_1 is the smaller and M_2 the larger bending moment at the ends of the unbraced length.

The above equation was used in the 1968, 1980, 1986, and 1991 editions of the *Specification*. Because it is valid only for straight-line moment diagrams, Equation C-2.3.1.2-

2 was replaced by the following equation for C_b in the 1996 edition of the *Specification* and is retained in this edition of the *Specification*:

$$C_b = \frac{12.5M_{max}}{2.5M_{max} + 3M_A + 4M_B + 3M_C} \tag{C-2.3.1.2-3}$$

where

M_{max} = Absolute value of maximum moment in the unbraced segment

M_A = Absolute value of moment at quarter point of unbraced segment

M_B = Absolute value of moment at centerline of unbraced segment

M_C = Absolute value of moment at three-quarter point of unbraced segment

Equation C-2.3.1.2-3, derived from Kirby and Nethercot (1979), can be used for various shapes of moment diagrams within the unbraced segment. It gives more accurate solutions for fixed-end members in bending and moment diagrams which are not straight lines. This equation is the same as that being used in ANSI/AISC S360 (AISC, 2010a).

Figure C-2.3.1.2-1 shows the differences between Equations C-2.3.1.2-2 and C-2.3.1.2-3 for a straight-line moment diagram.

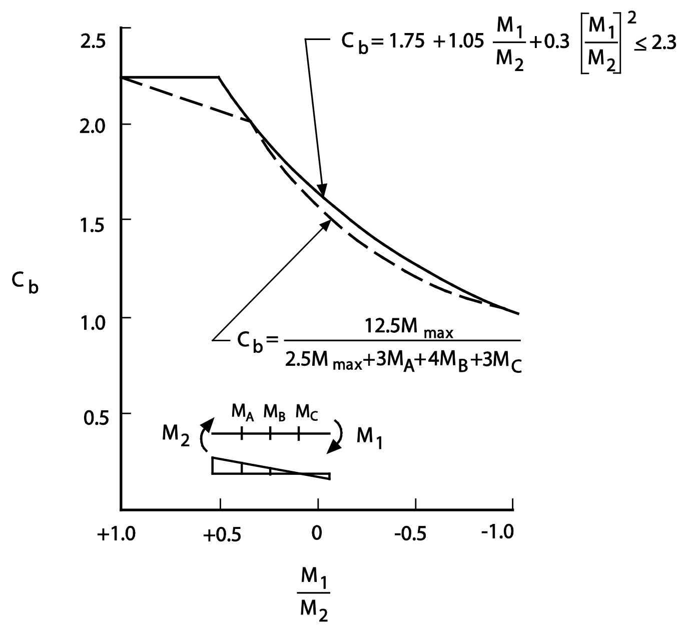


Figure C-2.3.1.2-1 C_b for Straight Line Moment Diagram

In the 1986 through 2016 editions of the *Specification*, C_{TF} was the bending coefficient used for *singly-symmetric sections* bending about the centroidal axis perpendicular to the axis of symmetry, where C_{TF} was used as a divisor rather than a multiplier. For straight-line moment diagrams with end moment ratios between -1 and +1/3, $1/C_{TF}$ was equal to C_b as defined by equation C-2.3.1.2-3. For end moment ratios larger than 1/3, $1/C_{TF}$ was greater than C_b. In 2022, $1/C_{TF}$ was replaced by C_b to account for various shapes of moment diagrams within the unbraced segment.

C. Limit of Unbraced Length

The elastic and inelastic critical *stresses* for the *lateral-torsional buckling* strength are shown in Figure C-2.3.1.2-2. For any unbraced length, L, less than L_u, *lateral-torsional buckling* does not need to be considered. L_u is determined by setting $M_{cre} = 2.78F_yS_f$ and $L_u = L_y = L_t$. L_u may then be calculated using the expression given below (AISI, 1996):

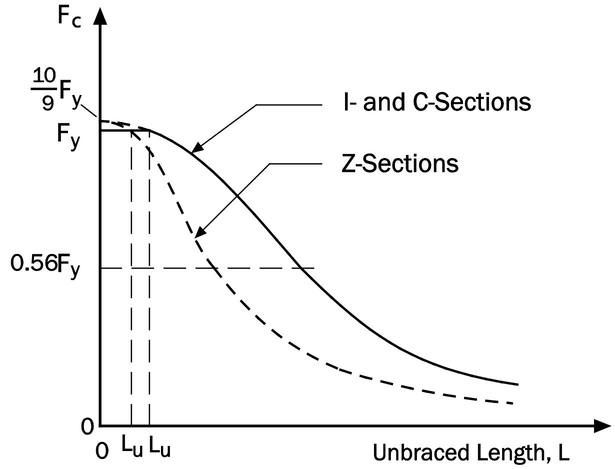


Figure C-2.3.1.2-2 Lateral-Torsional Buckling Stress

(1) For Singly-, Doubly- and Point-Symmetric Sections:

$$L_u = \left\{ \frac{GJ}{2C_1} + \left[\frac{C_2}{C_1} + \left(\frac{GJ}{2C_1} \right)^2 \right]^{0.5} \right\}^{0.5} \quad \text{(C-2.3.1.2-4)}$$

where

$$C_1 = \frac{7.72}{AE} \left(\frac{K_y F_y S_f}{C_b \pi r_y} \right)^2 \text{ for } \textit{singly-} \text{ and } \textit{doubly-symmetric sections} \quad \text{(C-2.3.1.2-5)}$$

$$C_1 = \frac{30.9}{AE} \left(\frac{K_y F_y S_f}{C_b \pi r_y} \right)^2 \text{ for } \textit{point-symmetric sections} \quad \text{(C-2.3.1.2-6)}$$

$$C_2 = \frac{\pi^2 E C_w}{(K_t)^2} \quad \text{(C-2.3.1.2-7)}$$

(2) For I-Sections or Z-Sections Bent About the Centroidal Axis Perpendicular to the Web The following equations may be used in lieu of (1) (AISI, 1996):

For *doubly-symmetric* I-sections:

$$L_u = \frac{1}{K_y} \left(\frac{0.18 C_b \pi^2 E d I_y}{F_y S_f} \right)^{0.5} \quad \text{(C-2.3.1.2-8)}$$

For *point-symmetric* Z-sections:

$$L_u = \frac{1}{K_y} \left(\frac{0.09 C_b \pi^2 E d I_y}{F_y S_f} \right)^{0.5} \quad \text{(C-2.3.1.2-9)}$$

(3) For Closed-Box Sections:

$$L_u = \frac{0.36C_b\pi}{K_yF_yS_f}\sqrt{EI_yGJ} \tag{C-2.3.1.2-10}$$

For members with unbraced length, $L \le L_u$, or elastic *lateral-torsional buckling* moment, $M_{cre} \ge 2.78F_yS_f$, the *nominal flexural strength* [*resistance*] M_{ne} (without considering *local buckling*) is determined by *Specification* Equation F2.1-1 with $F_n = F_y$.

The research work (Ellifritt, Sputo, and Haynes, 1992) and the study of Kavanagh and Ellifritt (1993 and 1994) have shown that a discretely braced beam, not attached to deck and sheathing, may fail either by *lateral-torsional buckling* between braces, or by *distortional buckling* at or near the braced point. See Section F4 for commentary on *distortional buckling* strength.

D. Laterally Unbraced Flanges

The problems discussed above dealt with the type of *lateral-torsional buckling* of I-members, C-sections, and Z-shaped sections for which the entire cross-section rotates and deflects in the lateral direction as a unit. But this is not the case for U-shaped beams and the combined sheet-stiffener sections as shown in Figure C-2.3.1.2-3. For this case, when the section is loaded in such a manner that the brims and the *flanges* of stiffeners are in compression, the tension *flange* of the beam remains straight and does not displace laterally. However, when the *distortional buckling* may occur, the compression *flange* tends to buckle separately in the lateral direction, accompanied by out-of-plane bending of the *web*, as shown in Figure C-2.3.1.2-4. This *distortional buckling* strength can be determined using the design provisions provided in *Specification* Section F4. It should, however, be noted that for laterally unstable U-shaped beams, *lateral-torsional buckling* may still occur. Therefore, *lateral-torsional buckling* should still be considered for U-shaped members.

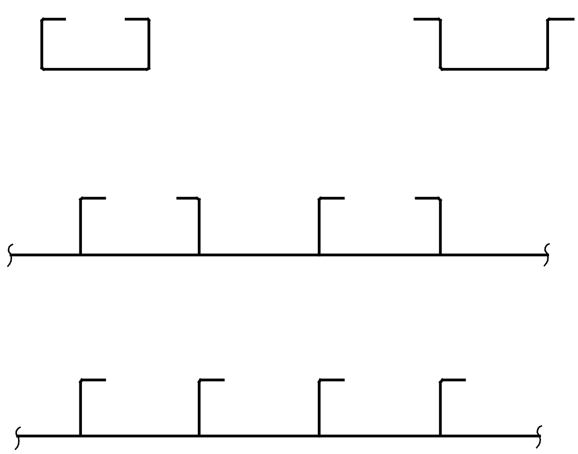


Figure C-2.3.1.2-3 Combined Sheet-Stiffener Sections

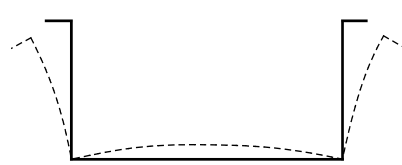


Figure C-2.3.1.2-4 Distortional Buckling of U-Shaped Beam

2.3.1.2.1 Sections Bending About Symmetric Axis

For *lateral-torsional buckling* of a *doubly-* or *singly-symmetric section*, the general equation C-2.3.1.2-1 is simplified because the asymmetry property $\beta_x = 0$ for a section symmetric about the x-axis. The resulting form of the equation is:

$$M_{cre} = r_o\sqrt{P_{ey}P_t} \quad \text{(C-2.3.1.2.1-1)}$$

Earlier editions of the Specification used an approximate *lateral-torsional buckling* moment derived by Winter (1943) for I-sections:

$$M_{cre} = \frac{\pi^2 EdI_{yc}}{(K_yL_y)^2} \quad \text{(C-2.3.1.2.1-2)}$$

In the above equation, K_yL_y is the *effective length* for *buckling* about the y-axis, E is the modulus of elasticity, d is the depth of the section, and I_{yc} is the moment of inertia of the compression portion of the section about the centroidal axis parallel to the *web*. The advantage of this equation is that the torsion properties C_w and J are not required. Therefore, this equation has been retained in the current *Specification* as an alternate for *doubly-symmetric* I-sections, where $I_{yc} = I_y/2$.

2.3.1.2.2 Sections Bending About Non-Symmetric Principal Axis

The *lateral-torsional buckling* moment for any section bending about a non-symmetric principal axis is determined from the general Equation C-2.3.1.2-1. The *Specification* presents this as a moment about the y-axis, so β_y and P_{ex} are used in place of β_x and P_{ey}, and the quadratic formula gives the following equation:

$$M_{cre} = -\beta_yP_{ex} \pm \sqrt{(\beta_yP_{ex})^2 + r_o^2P_{ex}P_t} \quad \text{(C-2.3.1.2.2-1)}$$

The asymmetry property β_y is the same as the property j defined in *Specification* Section 2.3.1 and used in *Specification* Section 2.3.1.2.2. The above equation provides two results corresponding to positive and negative bending. *Specification* Equation 2.3.1.2.2-1 uses a sign convention that ensures the moment is a positive number and is the larger result for moments causing compression on the shear center side of the centroid.

2.3.1.2.3 Point-Symmetric Sections

It should be noted that *point-symmetric sections* such as Z-sections with equal *flanges* will buckle laterally at lower strengths than *doubly-* and *singly-symmetric sections*. A conservative design approach is used in the *Specification*, in which the elastic critical *buckling stress* is taken to be one-half of that for *doubly-symmetric sections*.

2.3.1.2.4 Closed-Box Sections

In computing the *lateral-torsional buckling stress* of closed-box sections, the warping constant, C_w, may be neglected since the effect of nonuniform warping of box sections is small. The critical *buckling stress* is

$$M_{cre} = \frac{\pi}{(K_yL_y)}\sqrt{EI_yGJ} \quad \text{(C-2.3.1.2.4-1)}$$

The Saint-Venant torsional constant, J, of a box section, neglecting the corner radii, may be conservatively determined as follows:

$$J = \frac{2(ab)^2}{(a/t_1) + (b/t_2)} \tag{C-2.3.1.2.4-2}$$

where

a = Distance between *web* centerlines

b = Distance between *flange* centerlines

t_1 = *Thickness* of *flanges*

t_2 = *Thickness* of *webs*

2.3.1.2.5 Biaxial Bending

For biaxial bending, it is also possible to derive the global (*lateral-torsional*) *buckling* response about the resultant axis of bending. For elastic *lateral-torsional buckling* without axial *load*, general solutions for any section bending about a principal axis have been derived (Peköz and Winter, 1969a; Peköz and Celebi, 1969b, Yu and LaBoube, 2010). These expressions were generalized further for bending about any axis by Glauz (2017b) and is the basis for the provisions in this section.

A member subject to biaxial bending does not exhibit bifurcation because elastic displacement and rotation commence upon initial loading. But the so-called *lateral-torsional buckling* moment determined according to these provisions is a limiting moment shown to provide reliable ultimate strength predictions by Wang et al. (2020).

The *effective length*, KL, used in *Specification* Eq. 2.3.1.2.5-2 corresponds to the *flexural buckling* half-wavelength. For bending about the x-axis this is K_yL_y, and for bending about the y-axis it is K_xL_x. For biaxial bending, *flexural buckling* occurs about both the x- and y-axes with a common half-wavelength, thus KL is generally the smaller of K_xL_x and K_yL_y similar to the coupled *flexural buckling* of columns handled in *Specification* Section 2.3.1.1.4. The *effective length factor*, K, is discussed in detail in *Commentary* Section 2.3.1.1B, and its application to *lateral-torsional buckling* is subject to the same *effective length factor* stipulations in *Specification* Section C1 for the type of analysis used.

The *lateral-torsional buckling* calculations require determination of the asymmetry point (x_a, y_a). For any cross-section comprised of straight elements, the coordinates of the asymmetry point may be calculated as follows when using the principal axes:

$$x_a = \frac{1}{24I_y}\sum A_i \begin{bmatrix} y_{1i}^2(3x_{1i} + x_{2i}) + y_{2i}^2(x_{1i} + 3x_{2i}) \\ +(2y_{1i}y_{2i} + 3x_{1i}^2 + 3x_{2i}^2)(x_{1i} + x_{2i}) \end{bmatrix} \tag{C-2.3.1.2.5-1}$$

$$y_a = \frac{1}{24I_x}\sum A_i \begin{bmatrix} x_{1i}^2(3y_{1i} + y_{2i}) + x_{2i}^2(y_{1i} + 3y_{2i}) \\ +(2x_{1i}x_{2i} + 3y_{1i}^2 + 3y_{2i}^2)(y_{1i} + y_{2i}) \end{bmatrix} \tag{C-2.3.1.2.5-2}$$

where

A_i = Cross-sectional area of line element i (length times thickness)

x_{1i}, y_{1i} = Centroidal coordinates at one end of line element i

x_{2i}, y_{2i} = Centroidal coordinates at other end of line element i

2.3.2 Local Buckling

Local buckling is synonymous with plate *buckling*, and the classic plate *buckling* expression is:

$$F_{cr\ell} = k\frac{\pi^2 E}{12(1-\mu^2)}\left(\frac{t}{w}\right)^2 \tag{C-2.3.2-1}$$

This equation is used extensively in Appendix 1 of the *Specification* for the *Effective Width Method*. Equation 1.1-4 uses $F_{cr\ell}$ directly to determine the slenderness of an element, which in turn is used to find the *effective width* of the element. For every type of element and for different *stress* gradients on the elements, different solutions are provided for the plate *buckling* coefficient, k, in Appendix 1.

2.3.2.1 Local Buckling for Compression Members ($F_{cr\ell}$, $P_{cr\ell}$)

For a compression member, all elements have the same uniform compressive stress, so the controlling element is simply the one with the lowest *local buckling stress*. Consider an example of a lipped channel in compression with *web* depth, h = 8.94 in. (227.1 mm), *flange* width, b = 2.44 in. (62.00 mm), lip length d = 0.744 in. (18.88 mm), and t = 0.059 in. (1.499 mm) (and ignoring corner radius for this example). In this case:

Lip: $k = 0.43$, $F_{cr\ell\text{-lip}} = 0.43[\pi^2E/(12(1-\mu^2))](t/d)^2 =$ 72.1 ksi (497 MPa)

Flange: $k \approx 4$, $F_{cr\ell\text{-flange}} = 4.0[\pi^2E/(12(1-\mu^2))](t/b)^2 =$ 62.4 ksi (430 MPa)

Web: $k = 4$, $F_{cr\ell\text{-web}} = 4.0[\pi^2E/(12(1-\mu^2))](t/h)^2 =$ 4.6 ksi (32.0 MPa)

Each separate *local buckling stress* is used for determining the element effective width.

However, if the *Direct Strength Method* given in *Specification* Section E3.2 is used for finding the *local buckling strength*, the *local buckling* load, $P_{cr\ell}$, not *stress*, $F_{cr\ell}$ is required. Obviously, the three separate element (plate) solutions predict three separate $P_{cr\ell}$. The *Specification* requires using the minimum $F_{cr\ell}$, thus the *web local buckling stress* would be used in the preceding example.

In this example, the *web local buckling stress* is significantly lower than the other elements. The User Note in *Specification* Section 2.3.2.1 warns that in this case, prediction of $P_{cr\ell}$ based on the minimum $F_{cr\ell}$ may be very conservative. In this example, the *flange* provides beneficial restraint to the *web* that can be accounted for. The *DSM Design Guide* (Schafer, 2006) provides additional discussion, and improved analytical formulas are available (Schafer, 2001 and 2002; and Schafer and Peköz, 1999). However, for direct numerical solutions or tabulated numerical solutions, the AISI *Cold-Formed Steel Design Manual* (AISI, 2017) and CFSEI Tech Note G103-11 (Li and Schafer, 2011) are preferred since they can readily account for the interaction of the elements.

As an extension to this example, consider the same lipped channel in compression with a 4-in. (102-mm) deep hole located at the mid-depth of the *web*. The unstiffened elements at the hole net section have width a = (h – 4 in.)/2 = 2.47 in. (62.7 mm), and the A_{net} = 0.66 in.^{2} (430 mm^2) and A_g = 0.90 in.^{2} (583 mm^2). The $F_{cr\ell}$ are:

Lip: k = 0.43, $f_{cr\ell\text{-lip}} = 0.43[\pi^2E/(12(1-\mu^2))](t/d)^2 =$ 72.1 ksi (497 MPa)

Flange: $k \approx 4$, $f_{cr\ell\text{-flange}} = 4.0[\pi^2E/(12(1-\mu^2))](t/b)^2 =$ 62.4 ksi (430 MPa)

Web: k = 4, $f_{cr\ell\text{-web}} = 4.0[\pi^2E/(12(1-\mu^2))](t/h)^2 =$ 4.6 ksi (32.0 MPa)

Web at Hole: k = 0.43, $f_{cr\ell\text{-web}} = 0.43[\pi^2E/(12(1-\mu^2))](t/a)^2(A_{net}/A_g) =$ 4.8 ksi (33.3 MPa)

In this case, the net section does not control at the hole and the *web local buckling stress* away from the hole would still be multiplied by A_g to determine $P_{cr\ell}$. In this case, a smaller hole would have actually reduced the *buckling stress* at the hole location, e.g., a 2-in. (50.8-mm) hole yields a net section *local buckling stress* lower than away from the hole. Mitigating this circumstance is the fact that the net section squash load changes as well, and for the net section squash load, smaller holes are always better. Numerical methods may provide superior solutions since they can account for the beneficial restraint provided by the attached elements and can account for details such as edge-stiffened holes, etc.

2.3.2.2 Local Buckling for Flexural Members ($F_{cr\ell}$, $M_{cr\ell}$)

The *local buckling* moment, $M_{cr\ell}$, is determined using the same–minimum of the elements–approach as used for columns in *Specification* Section 2.3.2.1. $M_{cr\ell}$ is required for the *Direct Strength Method* of *Specification* Section F3.2 and may be approximated from the element *local buckling stress*. Note that the *Effective Width Method* of *Specification* Section F3.1 and Appendix 1 utilizes the element *local bucking stress* directly and ignores interaction amongst the elements.

Since it is a common practice to determine element *local buckling stress* utilizing the flat portion of a cross-section, for a member under a *stress* gradient, elements do not have a common reference location. Consider major-axis bending of a braced lipped channel with a 7.8-in. (198-mm) deep *web* (or overall 8-in. (203-mm) deep), 2.3-in. (58.4-mm) wide *flange*, 0.068-in. (1.73-mm) thick, and an outer corner radius of 0.10 in. (2.54 mm). Consider only the *flange* and *web* for this example (ignore the lip). From *Specification* Appendix 1, the plate *buckling* coefficients, k, would be found and are approximated here to be 23.9 for the *web* and 4.0 for the *flange*.

Flange: k = 4, $F_{cr\ell\text{-flange}} = 4.0[\pi^2E/(12(1-\mu^2))](0.068/2.3)^2 =$ 93.2 ksi (643 MPa)

$F_{cr\ell\text{-flange-ext}} = (F_{cr\ell\text{-flange}})(4/(4 - 0.068/2)) =$ 94.0 ksi (648 MPa)

Web: k = 23.9, $F_{cr\ell\text{-web}} = 23.9[\pi^2E/(12(1-\mu^2))](0.068/7.8)^2 =$ 48.4 ksi (334 MPa)

$F_{cr\ell\text{-web-ext}} = (F_{cr\ell\text{-web}})(4/(4 - 0.10)) =$ 49.7 ksi (342 MPa)

where $F_{cr\ell\text{-flange}}$ and $F_{cr\ell\text{-web}}$ are the *local buckling stresses* of the *flange* and the *web*, respectively; and $F_{cr\ell\text{-flange-ext}}$ and $F_{cr\ell\text{-web-ext}}$ are the corresponding stresses referenced to the extreme compression fiber, respectively.

The results show that the *web* controls, as 49.7 ksi (342 MPa) is less than 94.0 ksi (648 ksi). Therefore, the *web local buckling stress*, referenced to the extreme compression fiber is the governing $F_{cr\ell}$, and may be multiplied by the gross section modulus to estimate $M_{cr\ell}$.

Local buckling for flexural members with hole(s) in the *web* follows the same approach as for compression members, as detailed in the *Commentary* Section 2.3.2.1. When the net section with the hole is checked for *local buckling*, the unstiffened element *buckling stress* should be multiplied by the net section modulus and divided by the gross section modulus to develop the approximate *stress* on the gross cross-section. This *stress* can then be referenced to the extreme compression fiber and compared with all other elements.

2.3.3 Distortional Buckling

This section provides methods to directly calculate the *stress* at which a stiffened *flange* in compression buckles by rotating about the *flange/web* juncture. The methods were developed using principles of mechanics where the elastic rotational *stiffness* is equated to the stress-dependent geometric rotational *stiffness*. The geometric stiffness is assumed to be proportional to *stress*, but this is an approximation. Therefore, these methods are not as accurate as numerical analyses and often give lower *distortional buckling stresses*. Numerical methods are recommended, but the provisions of this section provide an acceptable alternative.

2.3.3.1 Distortional Buckling for Compression Members (F_{crd}, P_{crd})

The expressions employed in *Specification* Section 2.3.3.1 are derived in Schafer (2002) and verified for complex stiffeners in Schafer, et al. (2006). The equations used for the *distortional buckling stress* in AS/NZS 4600 (1996) are similar, except that when the *web* is very slender and is restrained by the *flange*, AS/NZS 4600 formulae use a simpler, conservative treatment. Since the provided expressions can be complicated, solutions for the geometric properties of C- and Z-section *flanges* based on centerline dimensions are provided in *Specification* Table 2.3.3-1. More refined values including corner radius are possible and permitted.

In many cases, the *flange* will have full or partial rotational restraint due to attachment to a brace, panel, or sheeting. In this case the appropriate rotational *stiffness*, $\bar{k}_{\phi}$, from the restraining elements may be added to the solution. The *Commentary* Section 2.3.3.2 provides additional details on $\bar{k}_{\phi}$ and its determination.

Application of the method is involved and examples are provided in the AISI *Cold-Formed Steel Design Manual* (AISI, 2017). While numerical methods or tabulated solutions from numerical methods (CFSEI Tech Note G103-11 by Li and Schafer, 2011) are generally preferred, a simplified method was provided until 2010 in the *Specification*. In 2010, the simplified approach was moved to the *Commentary* (as shown below), reflecting the intent that the method be used in preliminary design only, as it intentionally provides a lower bound solution.

Simplified Method for Unrestrained C- and Z-Sections With Simple Lip Stiffeners

For C- and Z-sections that have no rotational restraint of the *flange* and that are within the dimensional limits provided in this section, Equation C-2.3.3.1-1 can be used to calculate a conservative prediction of *distortional buckling stress*, F_{crd}, provided the following dimensional limits are met:

(1) $50 \le h_o/t \le 200$,

(2) $25 \le b_o/t \le 100$,

(3) $6.25 < D/t \le 50$,

(4) $45° \le \theta \le 90°$,

(5) $2 \le h_o/b_o \le 8$, and

(6) $0.04 \le D\sin\theta/b_o \le 0.5$

where

h_o = Out-to-out *web* depth as defined in *Specification* Figure 1.1.2-2

b_o = Out-to-out flange width as defined in *Specification* Figure 1.1.2-2

D = Out-to-out lip dimension as defined in *Specification* Figure 1.3-1

t = Base steel *thickness*

θ = Lip angle as defined in *Specification* Figure 1.3-1

$$F_{crd} = \alpha k_d \frac{\pi^2 E}{12(1-\mu^2)}\left(\frac{t}{b_o}\right)^2 \quad \text{(C-2.3.3.1-1)}$$

where

α = A value that accounts for the benefit of an unbraced length, L_m, shorter than L_{cr}, but can be conservatively taken as 1.0

= 1.0 for $L_m \ge L_{cr}$

= $(L_m/L_{cr})^{\ell n(L_m/L_{cr})}$ for $L_m < L_{cr}$ (C-2.3.3.1-2)

L_m = Distance between discrete restraints that restrict *distortional buckling*

$$L_{cr} = 1.2 h_o\left(\frac{b_o D\sin\theta}{h_o t}\right)^{0.6} \le 10 h_o \quad \text{(C-2.3.3.1-3)}$$

$$k_d = 0.05 \le 0.1\left(\frac{b_o D\sin\theta}{h_o t}\right)^{1.4} \le 8.0 \quad \text{(C-2.3.3.1-4)}$$

E = Modulus of elasticity of steel

μ = Poisson's ratio of steel

2.3.3.2 Distortional Buckling for Flexural Members (F_{crd}, M_{crd})

The expressions employed here for bending about the axis perpendicular to the *web* are derived in Schafer (2002), verified for complex stiffeners in Schafer, et al. (2006), and refined in Glauz (2018). The equations used for the *distortional buckling stress* in AS/NZS 4600 (1996) are similar, except that when the *web* is very slender and is restrained by the *flange*, AS/NZS 4600 uses a simpler, conservative treatment. Since the provided expressions can be complicated, solutions for the geometric properties of C- and Z-sections based on centerline dimensions are provided in *Specification* Appendix 2 Table 2.3.3-1; more refined values including corner radius are possible and permitted.

The expressions for bending about the axis parallel to the *web* were derived in Glauz (2018). These are applicable to C-sections, Hat-sections, and other shapes where the *flange* edge stiffeners are in compression, the *flange* has a *stress* gradient, and the *web* is in tension. For Hat-sections, the *web* is considered the middle element, where *buckling* occurs as shown in Figure C-2.3.1.2-4. These provisions make use of an effective *web* depth, h_e (*Specification* Equation 2.3.3.2-11), determined from properties of the *flange* because the actual *web* depth has almost no influence on the rotational *stiffness* when in tension.

Application of the method is involved and examples are provided in the AISI *Cold-Formed Steel Design Manual* (AISI, 2017). Numerical methods or tabulated solutions from numerical methods in CFSEI Tech Note G103-11 (Li and Schafer, 2011) are often preferred.

(a) $\bar{k}_{\phi}$ *Determination*

In many cases, the *flange* will have full or partial rotational restraint due to attachment to a brace, panel, or sheeting. In this case the appropriate rotational *stiffness*, $\bar{k}_{\phi}$, from the restraining element(s) may be added to the solution. While it is always conservative to ignore the rotational restraint, $\bar{k}_{\phi}$, in most cases it is beneficial to include this effect. Due to the large variety of possible conditions, no specific method is provided for determining the rotational restraint.

For framing applications, such as, studs, joists, girts, etc. sheathed with plywood, OSB, or gypsum board, AISI S240 (AISI, 2020) provides provisions for determining $\bar{k}_{\phi}$ developed based on mechanics and testing (Schafer, Sangree and Guan 2007 and 2008; Schafer, et al. 2010). For metal building applications, such as, *purlins* and *girts* with through-fastened sheathing (both with and without insulation), Gao and Moen (2012) provide a method for determining $\bar{k}_{\phi}$ confirmed by testing. As reference, past testing on 8-in. and 9.5-in. (203-mm and 241-mm) deep Z-sections with a *thickness* between 0.069 in. (1.75 mm) and 0.118 in. (3.00 mm), through-fastened 12 in. (205 mm) o.c., to a 36-in. (914 mm) wide, 1-in. (25.4 mm) and 1.5-in. (38.1 mm) high steel panels, with up to 6 in. (152 mm) of blanket insulation between the panel and the Z-section, results in a $\bar{k}_{\phi}$ between 0.15 to 0.44 kip-in./rad./in. (0.667 to 1.96 kN-mm/rad./mm) (MRI, 1981).

Additional testing on C- and Z-sections with pairs of through-fasteners provides considerably higher rotational *stiffness*: For 6-in. and 8-in. (152-mm and 203-mm) deep C-sections with a *thickness* between 0.054 and 0.097 in. (1.27 and 2.46 mm), fastened with pairs of fasteners on each side of a 1.25-in. (31.8-mm) high steel panel flute at 12 in. (305 mm) o.c., $\bar{k}_{\phi}$ is 0.4 kip-in./rad./in. (1.78 kN-mm/rad./mm); and for 8.5-in. (216-mm) deep Z-sections with a *thickness* between 0.070 in. and 0.120 in. (1.78 mm to 3.05 mm), fastened with pairs of fasteners on each side of 1.25 in. (31.8 mm) high steel panel flute at 12 in. (305 mm) o.c., $\bar{k}_{\phi}$ is 0.8 kip-in./rad./in. (3.56 kN-mm/rad./mm) (Yu and Schafer, 2003; Yu, 2005).

Test determination of $\bar{k}_{\phi}$ may use AISI S901 (AISI, 2013g). K from this method is a lower bound estimate of $\bar{k}_{\phi}$. The member lateral deformation may be removed from the measured lateral deformation to provide a more accurate estimate of $\bar{k}_{\phi}$ as detailed in Schafer, Sangree and Guan, 2008; and Schafer, et al., 2010.

(b) Moment Gradient

The presence of moment gradient can also increase the *distortional buckling* moment. However, this increase is lessened if the moment gradient occurs over a longer length. Thus, in determining the influence of moment gradient, β, the ratio of the end moments, M_1/M_2, and the ratio of the critical *distortional buckling* length to the unbraced length, L/L_m, should both be accounted for. In 2010, the sign convention on the ratio of moments M_1 and M_2 was changed to be consistent with moment gradient expressions for C_m (*Specification* Equation in Section C1). *Specification* Equation 2.3.3.2-3 and *Commentary* Equation C-2.3.3.2-2 were revised accordingly. Yu (2005) performed elastic *buckling* analysis with shell finite element models of C- and Z-sections under different moment gradients to examine this problem.

Significant scatter exists in the results; therefore, a lower bound prediction (*Specification* Equation 2.3.3.2-3) for the increase was selected.

(c) Simplified Method for Unrestrained C- and Z-Sections With Simple Lip Stiffeners

Due to the complexity of the expressions, a simplified method was provided until 2010 in the *Specification*. In 2010, the simplified approach was moved to the *Commentary*, reflecting the intent that the method be used in preliminary design only, as it intentionally provides a lower bound solution. For C- and Z-sections that have no rotational restraint of the compression *flange* and are within the dimensional limits provided in this section, Equation C-2.3.3.2-1 can be used to calculate a conservative prediction of the *distortional buckling stress*, F_{crd}. See *Specification* Section 2.3.3.2 or 2.2 for alternative provisions and for members outside the dimensional limits.

The following dimensional limits apply:

(1) $50 \le h_o/t \le 200$,

(2) $25 \le b_o/t \le 100$,

(3) $6.25 < D/t \le 50$,

(4) $45° \le \theta < 90°$,

(5) $2 \le h_o/b_o \le 8$, and

(6) $0.04 \le D\sin\theta/b_o \le 0.5$.

where

h_o = Out-to-out web depth as defined in *Specification* Figure 1.1.2-2

t = Base steel *thickness*

b_o = Out-to-out *flange* width as defined in *Specification* Figure 1.1.2-2

D = Out-to-out lip dimension as defined in *Specification* Figure 1.3-1

θ = Lip angle as defined in *Specification* Figure 1.3-1

The *distortional buckling stress*, F_{crd}, can be calculated as follows:

$$F_{crd} = \beta k_d \frac{\pi^2 E}{12(1-\mu^2)}\left(\frac{t}{b_o}\right)^2 \quad \text{(C-2.3.3.2-1)}$$

where

β = A value accounting for moment gradient, which is permitted to be conservatively taken as 1.0

$$= 1.0 \le 1 + 0.4(L/L_m)^{0.7}(1 + M_1/M_2)^{0.7} \le 1.3 \quad \text{(C-2.3.3.2-2)}$$

where

L = Minimum of L_{cr} and L_m

$$L_{cr} = 1.2 h_o \left(\frac{b_o D\sin\theta}{h_o t}\right)^{0.6} \le 10 h_o \quad \text{(C-2.3.3.2-3)}$$

L_m = Distance between discrete restraints that restrict *distortional buckling* (for continuously restrained members L_m=L_{cr})

M_1 and M_2= Smaller and larger end moment, respectively, in the unbraced segment (L_m) of the beam; M_1/M_2 is positive when the moments cause reverse curvature and negative when bent in single curvature

$$k_d = 0.5 \le 0.6\left(\frac{b_o D\sin\theta}{h_o t}\right)^{0.7} \le 8.0 \quad \text{(C-2.3.3.2-4)}$$

E = Modulus of elasticity of steel

μ = Poisson's ratio of steel

2.3.3.3 Distortional Buckling for Members With Holes

The *distortional buckling stress* F_{crd} is, at least in part, dictated by the bending *stiffness* provided by the *web* of an open cross-section as it restrains the attached *flange* from rotating (see Figure C-2.2.7-1). If a hole with length L_h is introduced into the *web* of an open cross-section, the rotational restraint provided by the *web* is decreased, resulting in a smaller critical *distortional buckling stress* (Kesti, 2000; Moen and Schafer, 2009a).

An approximate method for calculating F_{crd} including the influence of flat-punched unstiffened *web* holes for finite strip analysis has been developed by Moen and Schafer (2009c) and adapted here for use in *Specification* Sections 2.3.3.1 and 2.3.3.2. The key to the method is the reduction of the bending *stiffness* of the *web*. This is completed by modifying the *web* thickness from t to t_r. This modification is only required for the rotational *stiffness* terms; correction of the *distortional buckling* half-wavelength, L_{crd}, is not required. In 2022, a lower bound of $t_r = 0$ was added to *Specification* Equations 2.3.3.3-1 and 2.3.3.3-3 to avoid applying negative values of modified *thickness*. In 2024, the t_r provisions for discrete *web* holes were clarified for application to a single hole or single line of *web* holes.

A similar reduction may also be applied to members that have other hole patterns (multiple lines of holes) along the full length of the *web* (Smith and Moen, 2014). In this case, the reduced *stiffness* is not only at the hole location but throughout the length of the member. Refer to Section 2.2.7 for more information on hole patterns different from a single hole in the *web* or a single line of *web* holes longitudinally.

2.3.4 Shear Buckling (V_{cr})

Traditionally, the *shear buckling stress* and its resultant (*shear buckling* force) are based on the *web* alone ignoring interaction from the *flanges*, and are consistent with Section G2.3 of the *Specification*. For C- and Z-sections, *Specification* Section 2.3.4 provides a more refined calculation based on the work of Aswegan and Moen (2012). Pham and Hancock (2011) also provide tabulated solutions for a range of lipped channel section geometries calculated using the Spline Finite Strip Method (SFSM).

The *shear buckling* half-wavelength, L_v, for unreinforced *webs* is approximated as 85 percent of the flat *web* depth. This critical half-wavelength actually varies with section geometry and a more accurate value may be appropriate.

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APPENDIX 3, MEMBERS UNDER COMBINED FORCES — ALTERNATIVE PROCEDURE

For members under combined forces, the *Specification* provides interaction expressions, e.g., *Specification* Eq. H1.2-1 for combined compressive axial *load* and bending. In general, interaction expressions are convenient, conservative, and efficient to evaluate. However, interaction expressions assume that the strength of the member under each isolated loading condition is all that is required to assess the impact of *load* interactions. This approach can be significantly conservative for members bent about an axis that is not a cross-section symmetry axis, for members utilizing inelastic reserve, and/or for members with axial tension.

As an alternative to Chapter H, this Appendix provides a method for evaluating interaction on a member where both the elastic *buckling* and the *yielding* of the member are assessed under the actual combined forces. This allows for a more accurate strength prediction and can even be used to replace the general interaction expressions of the *Specification* with cross-section specific interaction expressions if evaluated across a range of relevant interacting loads. The method of this Appendix was developed as an extension of the *Direct Strength Method* of design (Torabian et al.; 2015, 2016a and 2016b, and 2018). It is anticipated that engineers or manufacturers may utilize this approach particularly for unsymmetric and optimized cross-sections where the additional structural efficiency predicted from the expressions outweighs the additional calculation complexity.

3.1 General Requirements

Any interaction can be understood through an appropriate interaction space; for example, Figure C-H2-1 is the interaction space for shear and longitudinal *stress* from bending. The interaction space utilized in this Appendix is three-dimensional and considers normalized axial *load* (P/P_y), and normalized major-axis (M_1/M_{1y}) and minor-axis (M_2/M_{2y}) bending as depicted in Figure C-3.1-1(a). The demand, β_r, is established as the square root of the sum of the squares of the normalized demands for each loading action, as provided in *Specification* Eq. 3.1.1-1. That is, the demand is a vector in the normalized P-M_1-M_2 space with length of β_r, and at angles of φ_{PM} and θ_{12} as shown in Figure C-3.1-1(a). The angles are established by the ratios between the demands, where φ_{PM} is given in *Specification* Eq. 3.1.2-4 and θ_{12} is given in *Specification* Eq. 3.3-7. Note, the normalization in bending is to first yield (not fully plastic) so a demand $\beta_r > 1$ is possible.

The capacity, β_a, is established from the *nominal strength* [*resistance*], β_n, and the application of the *resistance factor* or *safety factor*. As illustrated in Figure C-3.1-1(b) for *distortional buckling*, determination of the *nominal strength* [*resistance*] requires finding the elastic *buckling* value (e.g., β_{crd}) as well as *yielding* values (β_y and β_p) and considering appropriate interactions. All of these calculations are made under *stresses* consistent with the applied actions, i.e., along angles φ_{PM} and θ_{12}. Determination of the *resistance factor* or *safety factor* depends on the extent to which the member is primarily a column (e.g. ϕ_c) or a beam (e.g., ϕ_b) – since these two elements have differing levels of reliabiltiy. The angle φ_{PM} is used to determine extent of the beam-column interaction and provide an appropriate *resistance factor* or *safety factor.*

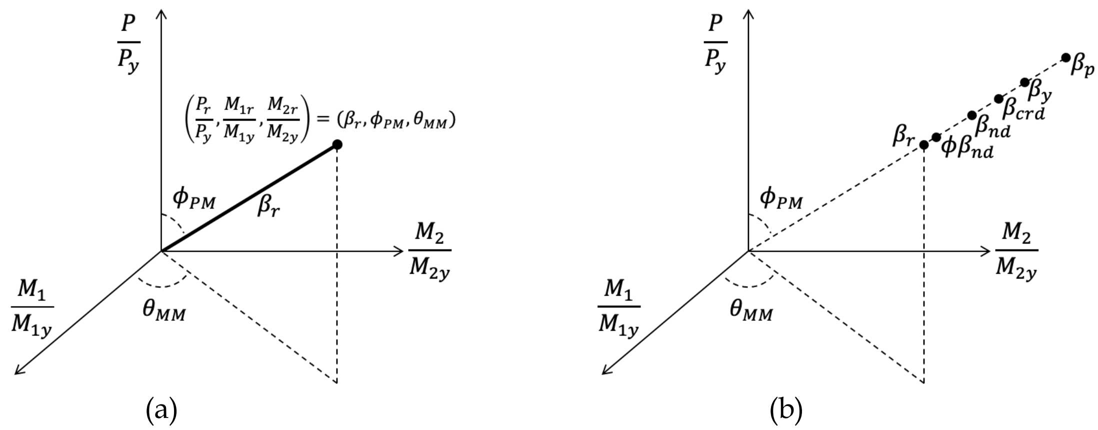

Figure C-3.1-1 Normalized P-M_1-M_2 Interaction Space
(a) Depiction of Generalization of the Demand (i.e., Required Strength [Effect Due to Factored Loads]);
(b) Depiction of Capacity in Distortional Buckling Along With Buckling and Yielding Inputs
(Similarly for Local Buckling and Global Buckling)

3.2 Yielding and Global Buckling , β_{ne}

The strength of a *structural member* which is not subject to *local buckling* or *distortional buckling* is a function of only global *buckling* and *yielding*. While this is true for both beams and columns, the manner in which global *buckling* and *yielding* are considered is different for isolated compression (i.e., Section E2) and bending (i.e., Section F2.2). In a proper beam-column formulation, the isolated beam and column strengths should result when evaluated with only a single action applied. Due to the difference between the strength curves for beams and columns, the beam-column strength uses a weighted interpolation of the two. Determination of the appropriate weighting function is summarized in Torabian et al. (2018) and fully detailed in Torabian et al. (2016). A sine function as provided in *Specification* Eq. 3.2-1 was determined to provide the best combination of accuracy and simplicity, and was further validated by Glauz (2023). Figure C-3.2-1 illustrates how this weighting produces the beam-column strength curve. For members with axial tension, the bending strength is assumed to govern.

Evaluation of the global strength expressions requires three inputs: generalized first yield magnitude (β_y), generalized fully plastic magnitude (β_p), and generalized elastic global *buckling* magnitude (β_{cre}). The simplest of the three is first yield. Practical determination of the *yielding* magnitude, β_y, is detailed in the associated user note in Section 3.2, and depicted in Figure C-3.1-1(b). Conceptually, β_y is the magnitude of the vector along the φ_{PM} and θ_{12} line at which the first fiber in the cross-section reaches yield. The open source software CUFSM (Li and Schafer, 2010b) provides simplified tools for determining β_y for any thin-walled section.

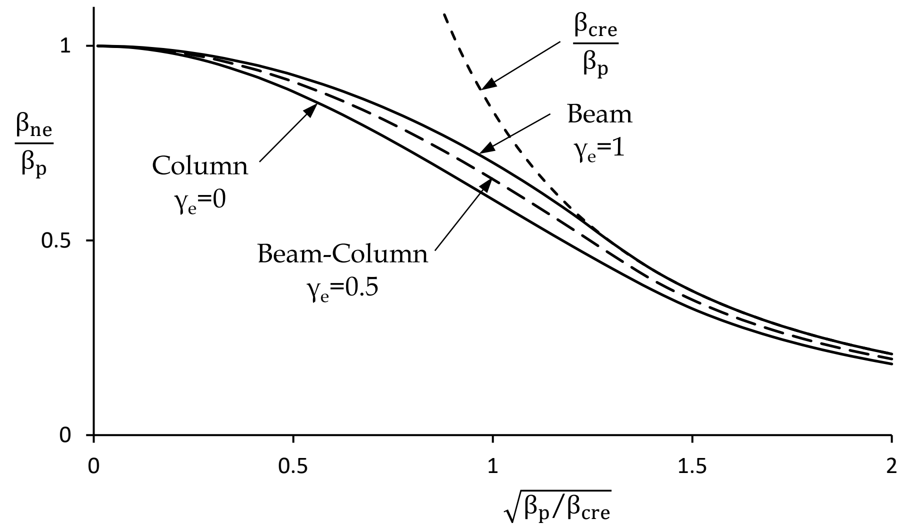

Figure C-3.2-1 Global Buckling Strength Curves for Beam-Columns

Fully plastic magnitude, β_p, is the length along these same angles in which the section is fully plastified and thus every fiber reaches either positive or negative F_y. Analytical expressions are possible for simple cross-section geometries, but in general numerical methods are required. The software CUFSM based on algorithms described in Torabian et al. (2016b) provides tools for determining β_p for any thin-walled section. For sections unable to achieve inelastic reserve strength due to *local* or *distortional buckling* slenderness, *Specification* Eq. 3.2-7 provides a simple β_p calculation to conservatively estimate the strength below the first yield.

For the *buckling* magnitude we again apply P_r, M_{1r}, M_{2r} to the cross-section, determine the elastic *stresses*, and then find the global elastic *buckling* multiplier or load factor (LF) under these *stresses* through an eigen-*buckling* solution. The *buckling* magnitude β_{cre} is simply LF × β_r. Through considering an eccentrically applied axial *load*, analytical expressions for global beam-column *buckling* exist for a variety of cases (Peköz and Winter, 1969a; Peköz and Celebi, 1969b, Yu and LaBoube, 2010); however, it is often simpler to use semi-analytical tools such as the Finite Strip Method (e.g., CUFSM) or even the Finite Element Method. Note, the solutions should converge to known P_{cre} and M_{cre} values when only axial *load* or bending is evaluated. See Appendix 2 Section 2.2.2 commentary for additional discussion.

For the special case of biaxial bending alone, analytical solutions for M_{cre} are also available (Glauz, 2019) and are provided in Appendix 2, Section 2.3.1.2.5. The accuracy of the strength approach of this Appendix for biaxial bending was affirmed by Wang et al. (2020).

3.3 Local Buckling Interacting With Yielding and Global Buckling, $\beta_{n\ell}$

Consideration of *local*-global *buckling* interaction for a beam-column follows the procedures already established for the *Direct Strength Method* for columns in Section E3.2 and beams in Section F3.2. The *local buckling* strength expressions for beams and columns are nearly identical, although the beam equation includes flexural factors α_s and β_s. These equations are merged for beam-column strength prediction as developed and calibrated in Glauz (2023). The equation coefficients are interpolated between the P, M_1, and M_2 coefficients with heavy weighting given to the P curve as seen in the γ_ℓ expression. The flexural factors α_s and β_s can significantly alter the strength curve, but the influence of these factors fades quickly with the introduction of axial compression. For members with axial tension, the flexural curve is retained and the local-global slenderness decreases as tension is increased. The provisions for *local buckling* strength of beam-columns with holes utilize the same interpolation weighting as for members without holes.

The engineer must provide the generalized elastic *local buckling* value $\beta_{cr\ell}$. This is found by applying P_r, M_{1r}, M_{2r} to the cross-section, determining the elastic *stresses*, and then finding the *local* elastic *buckling* multiplier or load factor (LF) under these *stresses* through an eigen-*buckling* solution (See Appendix 2 *Commentary* Section 2.2.2). The *buckling* magnitude $\beta_{cr\ell}$ is LF × β_r. The open source software CUFSM provides tools for determining $\beta_{cr\ell}$ for any thin-walled cross-section. Use of the two-step method and the constrained Finite Strip Method (see Li and Schafer, 2010 and *Commentary* Section 2.2.3) may be necessary to provide a solution for arbitrary sections under arbitrary loading. Use of the element method approximation (e.g. Appendix 2 Section 2.3.2) under generalized stresses for the elements to provide the elastic *buckling* approximation is possible, but generally not recommended as the method tends to be overly conservative and would negate the effort expended for the beam-column solution.

3.4 Distortional Buckling, β_{nd}

Consideration of *distortional buckling* for a beam-column follows the procedures already established for the *Direct Strength Method* for columns in Section E4 and beams in Section F4. Interpolation of the *distortional buckling* strength curves for beams and columns was first developed by Torabian et al. (2016b, 2018) and simplified in Schafer (2019). Improvements to the strength curves for beams and columns by Glauz and Schafer (2022) permitted the use of a single strength equation as a function of slenderness λ_d as developed and calibrated in Glauz (2023) and plotted in Figure C-3.4-1. The equation coefficients are interpolated between the P, M_1, and M_2 coefficients with heavy weighting given to the P curve as seen in the γ_d expression. The flexural factors α_s and β_s can significantly alter the strength curve, but the influence of these factors fades quickly with the introduction of axial compression. For members with axial tension, the flexural curve is retained and the slenderness λ_d decreases as tension is increased. The provisions for *distortional buckling* strength of beam-columns with holes utilize the same interpolation weighting as for members without holes.

The engineer must provide the generalized elastic *distortional buckling* value β_{crd}. This is found by applying P_r, M_{1r}, M_{2r} to the cross-section, determining the elastic *stresses*, and then finding the *distortional* elastic *buckling* multiplier or load factor (LF) under these *stresses* through an eigen-*buckling* solution (See Appendix 2 Commentary Section 2.2.2). The *buckling* magnitude β_{crd} is LF × β_r. The open source software CUFSM provides tools for determining β_{crd} for any thin-walled cross-section. Use of the two-step method and the constrained Finite Strip Method (Li and Schafer 2010, Appendix 2 *Commentary* Section 2.2.3) may be necessary to provide a solution for arbitrary sections under arbitrary loading. Existing analytical *distortional buckling* approximations cannot be readily generalized to a beam-column with accuracy as the *flange* of the section is approximated as a column in *flexural-torsional buckling*, but the portion of the section in which this assumption is appropriate is load-dependent.

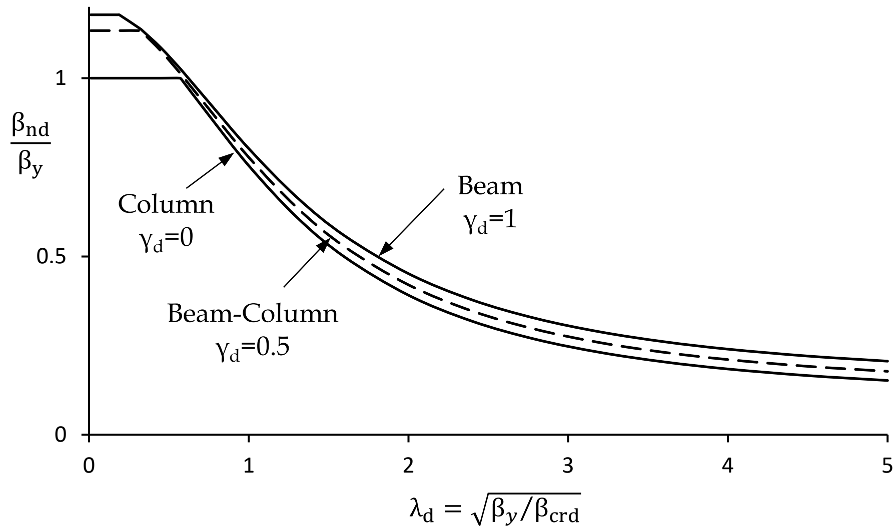

Figure C-3.4-1 Distortional Buckling Strength Curves for Beam-Columns (α_s=1, β_s=1)

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APPENDIX 4, ALTERNATIVE METHOD FOR PERFORMANCE OF COLD-FORMED STEEL SYSTEMS UNDER ELEVATED TEMPERATURES DUE TO FIRE CONDITIONS

4.1 General Provisions

Appendix 4 provides criteria for designing cold-formed steel members and components, including floor and wall assemblies, for fire conditions. In traditional practice, fire design of cold-formed steel members and components is conducted through the use of approved assemblies that have been successfully subjected to standard fire testing. Using Appendix 4, compliance with the performance objective in *Specification* Section 4.1.1 can be demonstrated by either structural fire analysis, component/assembly qualification testing (the traditional approach, which strictly does not require application of this Appendix), or a combination of analysis and testing. Therefore, while traditional prescriptive testing is explicitly allowed as defined in Section 4.3, these provisions provide an alternative path that allows fire performance to be predicted with simulation. Importantly, if small changes (such as new fasteners, new member shapes, or other details) are made to an assembly these provisions provide a potential means to demonstrate the impact of the change without recourse back to full-scale fire testing.

In organization, the provisions in Appendix 4 largely parallel those in AISC 360 (AISC, 2016a). However, fire design for cold-formed steel structures requires specific provisions because the fire response of cold-formed steel construction differs from that of other materials. Notably, cold-formed steel components and systems invariably need fire protection, with gypsum board being the most common material. Therefore, testing and analysis of cold-formed steel in fire necessarily requires the engineer to account for the performance of the fire-protected cold-formed steel assembly, not just the steel alone. Appendix 4 includes provisions for the thermal and mechanical performance of fire protection materials at elevated temperature, connectors, and assemblies. Furthermore, the elevated temperature mechanical properties of cold-formed steels differ from those of hot-rolled steels. Appendix 4 includes retention factors specific to cold-formed steels. The provisions of this appendix rely on fire performance-based design provisions for cold-formed steel recently adopted in AS/NZS 4600 (AS/NZS, 2018) as well as on recent research on cold-formed steel at elevated temperature as referenced in this document.

Methods for design by analysis of cold-formed steel members and components for fire conditions are available for a range of assemblies and conditions, while research is still ongoing for improving methods for some assemblies. For example, modeling of composite assemblies with unprotected cold-formed steel deck with concrete fill is currently difficult due to the fast temperature increase in the steel resulting in differential thermal expansion with the concrete, the modeling complexity from the studs, composite action, and two-way bending with ribbed decks, the sensitivity to boundary conditions, and numerical instability associated with concrete cracking. Numerical issues in modeling composite steel decks have been mentioned in the literature (ASCE, 2020). The limitations of numerical models based on the current state of knowledge must be considered when adopting a design by analysis.

Thermal expansion and progressive decrease in strength and *stiffness* are the primary structural responses to *elevated temperatures* that may occur during fires. An assessment of a design of building components and systems based on structural mechanics that allows designers to address the fire-induced restrained thermal expansions, deformations and material degradation at *elevated temperatures* can lead to a more robust structural design for fire conditions.

Appendix 4 is applicable for ASTM steels referenced in this *Specification*. Appendix 4 applies to plate *thickness* up to 0.14 in. (3.5 mm). The test data that underpins the development of the cold-formed steel material retention factors in this Appendix show that steel *thickness* in the range of 0.016 to 0.14 in. (0.4 to 3.5 mm) does not noticeably influence the retention factors. However, no sufficient test data are available for cold-formed steels at *thickness* greater than 0.14 in. (3.5 mm) to support application of the retention factors to these greater *thicknesses*.

The Appendix is dedicated to design and evaluation under *elevated temperatures* originating from fire conditions. Therefore, although material properties and other provisions may be given from the temperature of 68 °F (20 °C) and up, the Appendix does not need to be applied for "ambient" conditions. For the purpose of this Appendix, "ambient" conditions are defined as a thermal environment that does not generate temperatures higher than 150 °F (65 °C) in the cold-formed steel members or the fire protection materials.

4.1.1 Performance Objective

The performance objective underlying the provisions in this *Specification* is that of life safety. Fire safety levels should depend on the building occupancy, height of the building, the presence of active fire mitigation measures, and the effectiveness of fire-fighting.

Three *limit states* exist for elements serving as fire barriers (compartment walls and floors) (AISC, 2016a): (1) heat transmission leading to unacceptable rise of temperature on the unexposed surface; (2) breach of barrier due to cracking or loss of integrity; and (3) loss of load-bearing capacity. In general, all three must be considered by the engineer to achieve the desired performance. These three *limit states* are interrelated in fire-resistant design. For structural elements that are not part of a separating element, the governing *limit state* is loss of load-bearing capacity.

Specific performance objectives for a building are determined by the stakeholders in the building process, within the context of the general performance objective and *limit states* discussed in the preceding paragraph. The building owner can, in consultation with the stakeholders, add voluntary objectives in addition to the mandatory objectives listed (or implicitly considered) in codes or regulations. Specific performance objectives can include, for example, property protection; continuity of operations; protection of the environment; and preservation of heritage (ISO, 2018).

4.1.4 Load Combinations and Required Strength

The *Specification* Equation (4.1.4-1) for *LSD* is from NBCC, CSA S16-19 Annex K: Cl.K.1.5.

4.2 Design by Analysis

Structural behavior under severe fire conditions is highly nonlinear in nature, because of the constitutive behavior of materials at *elevated temperatures* and the relatively large deformations that may develop in structural systems at sustained *elevated temperatures*. As a result of this behavior, it is difficult to develop design equations to ensure the necessary level of structural performance during severe fires using elastic based *ASD* method. Accordingly, structural design for fire conditions by analysis should be performed using *LRFD* or *LSD* methods, in which the nonlinear structural actions arising during severe fire exposures and the temperature-dependent design strengths can be properly taken into account.

4.2.1 Design-Basis Fire

Published codes and standards provide guidance to determine the *design-basis fire*, including NFPA 557 (NFPA, 2012), SFPE S.01 (SFPE, 2011), and the Eurocode EN1991-1-2 (Eurocode, 2002). In order to determine the appropriate *design-basis fire* for the evaluation of the structure, fuel loads or fuel load densities are needed. These are determined from existing databases, published standards, or from surveys of fuel in the built environment. Fuel load densities are generally expressed in megajoules per unit area. Fuel loads are characterized by the kind of combustible materials (attached to the built environment or its content), their amount and their location. Once a fuel load has been agreed upon for the occupancy, the designer should demonstrate the effect of various fires on the structure by assessing the heat flux and/or temperature-time relationships for various conditions (including ventilation factors). These heating effects may result in different structural responses. The effects of a localized fire should also be assessed to ascertain that local damage is not excessive, and failure of adjacent structural member(s) does not occur. Based on these results, the members and assemblies can be designed to provide a structure that is sufficiently robust.

(a) Localized Fires

Additional information on localized fires is available in the AISC 360, Commentary to Appendix 4 (AISC, 2016b).

(b) Post-Flashover Compartment Fires

Additional information on post-*flashover* compartment fires is available in the AISC 360, Commentary to Appendix 4 (AISC, 2016b).

4.2.2 Temperatures in Cold-formed Steel and Fire-Protected Cold-formed Steel Structural Systems Under Fire Conditions

The heat transfer analysis may range from one-dimensional analyses to three-dimensional analyses. The thermal response of the fire protection material should be taken into account. The heat transfer analyses should consider changes in material properties with increasing temperature for all materials included in the assembly. The boundary conditions for the heat transfer analysis should consider radiation heat transfer in all cases and convection heat transfer if the exposed element is submerged in the smoke or is being subjected to flame impingement. An example of two-dimensional heat transfer analysis of gypsum plasterboard panels is presented by Keerthan and Mahendran (2012); the analysis was conducted by the finite element method using the software SAFIR (Franssen and Gernay, 2017) and using suitable temperature-dependent thermal properties established from tests under standard fire.

4.2.3 Mechanical and Thermal Properties at Elevated Temperatures

4.2.3.1 Steel

4.2.3.1.1 Variation of Strength and Modulus of Elasticity With Temperature

The retention factors are based on a review of experimental data (Lee, et al., 2003; Outinen, 2004; Chen et al., 2007a and 2007b; Ranawaka et al., 2009; Kenanamge et al., 2011; Ye, et al., 2013; McCann, et al., 2015; Imran, et al., 2018; Batista, 2015; Yan et al., 2020), and the background to the equation is given by Yan, et al. (2021). The retention factors relationships are plotted in Figures C-4.2.3.1.1-1 to C-4.2.3.1.1-3 with the experimental data.

The experimental data include measurements from both steady-state and transient-state testing. Analysis of the effect of the testing method on *elevated temperature* properties of cold-formed steel is presented in the background article by Yan, et al. (2021). The retention factors are calibrated on the whole dataset.

It is noted that the *yield stress* provided in the *Specification* Appendix 4 is defined as the 0.2% proof *stress*. The 0.2% proof *stress* is recommended for the design *yield stress* of cold-formed steel under fire conditions, because *local buckling* in slender plates preclude the attainment of high strain levels (Ranby, 1998).

Combination of the retention factors in *Specification* Section 4.2.3.1.1 with the stress-strain relationship in Eq. 4.2.3.1.5-1 yields a stress-strain response that agrees with that of Eurocode EN1993-1-2 (and AISC Appendix 4) up to the 0.2% *proof stress*, then is lower (i.e., has less strain hardening) than that of Eurocode EN1993-1-2 for strains larger than the 0.2% *proof stress* strain. This aligns with experimental evidence that the 2% *yield stress* retention factors recommended for hot-rolled steels would be unconservative for cold-formed steels (Yan, et al., 2021), because temperature reduces the strength hardening effect that was gained from cold-working (Yu, et al., 2019). The equations for retention factors are not applied for temperatures higher than 1472°F (800°C), because data is scarce at temperatures higher than 1472°F (800°C), and properties are so low that measurement errors are likely to be more significant.

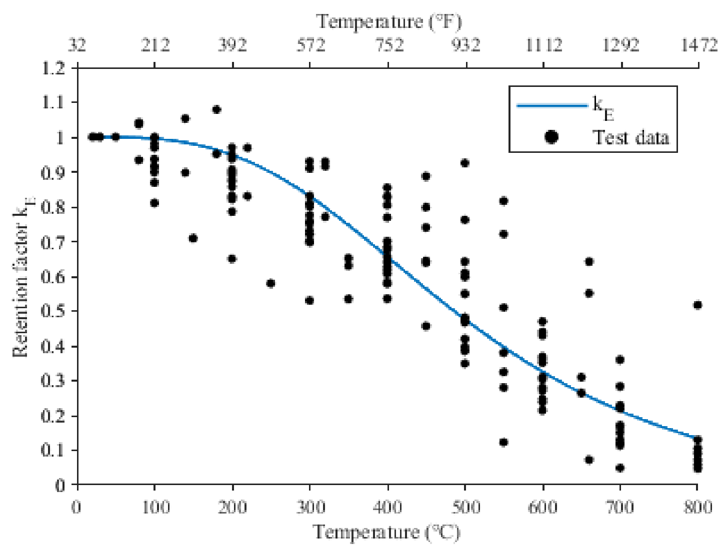

Figure C-4.2.3.1.1-1 Retention Factor for Modulus of Elasticity (k_E) of Cold-Formed Steel at Elevated Temperatures

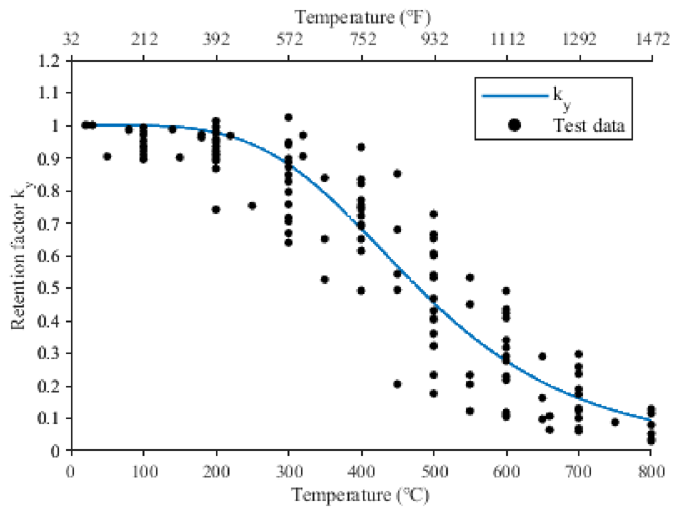

Figure C-4.2.3.1.1-2. Retention Factor for Yield Stress (k_y) of Cold-Formed Steel at Elevated Temperatures

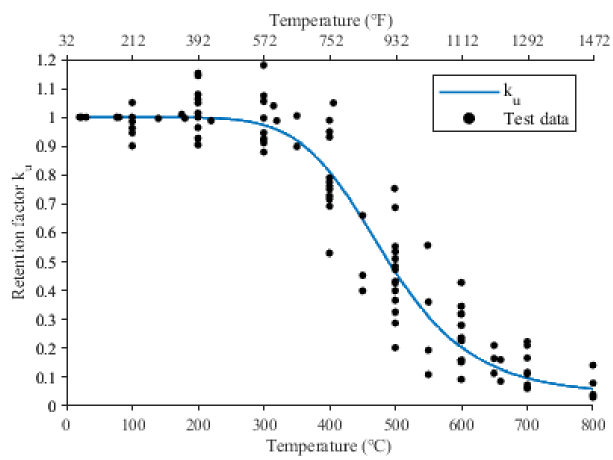

Figure C-4.2.3.1.1-3 Retention Factor for Tensile Strength (k_u) of Cold-Formed Steel at Elevated Temperatures.

4.2.3.1.2 Variation of Specific Heat With Temperature

The equations given in the *Specification* Appendix Section 4.2.3.1.2 are adopted from the AS/NZS 4600 (AS/NZS, 2018), Section 9.

4.2.3.1.3 Variation of Thermal Conductivity With Temperature

The equations given in the *Specification* Appendix Section 4.2.3.1.3 are adopted from the AS/NZS 4600 (AS/NZS, 2018), Section 9.

4.2.3.1.4 Thermal Elongation

The equations given in the *Specification* Appendix Section 4.2.3.1.4 are adopted from the AISC 360, Commentary to Appendix 4 (AISC, 2016b).

4.2.3.1.5 Variation of Stress-Strain Relationship With Temperature

The equations are adopted from the AS/NZS 4600 (AS/NZS, 2018), Section 9. The background to these equations is given by Kankanamge and Mahendran (2011).

An ultimate strain needs to be considered to limit the *stress*-strain relationship. The ultimate strain at *elevated temperature* is generally larger than at ambient temperature. Therefore, it is recommended to conservatively adopt the ultimate strain of the material at ambient temperature as a limit to the *stress*-strain relationship at *elevated temperature*.

4.2.3.2 Fire Protective Boards/Materials

When predicting the heat transfer for the design fire, the thermal performance of fire protection materials needs to be considered. This can be done using Finite Element Analysis (Keerthan and Mahendran, 2012 and 2013a) providing the adequate thermal properties are known for the fire protection materials.

The mechanical performance of fire protection materials at *elevated temperature* needs to be considered if the engineer is relying on the fire protection material to provide resistance to the *cold-formed steel structural members*.

4.2.3.3 Connectors

Retention factors for *stiffness* and strength of *connections* between cold-formed steel members and sheathing at *elevated temperature* can be taken as:

$$k_V, k_T, k_u, k_{k,V}, k_{k,T} = (1-c)\frac{1-x^b}{1+ax^b} + c \qquad \text{(C-4.2.3.3-1)}$$

where

k_V = Retention factor for strength of in-plane lateral resistance of cold-formed steel-to-sheathing *connections* for gypsum board and OSB

k_T = Retention factor for strength of pull-through resistance of cold-formed steel-to-sheathing *connections* for gypsum board and OSB

k_u = Retention factor for strength of in-plane lateral resistance of cold-formed steel-to-sheathing *connections* for steel sheathing

$k_{k,V}$ = Retention factor for stiffness of in-plane lateral resistance of cold-formed steel-to-sheathing *connections* for gypsum board and OSB

$k_{k,T}$ = Retention factor for stiffness of pull-through resistance of cold-formed steel-to-sheathing *connections* for gypsum board and OSB

$$x = \frac{T - T_1}{T_2 - T_1} \qquad \text{(C-4.2.3.3-2)}$$

a, b, c, T_1, and T_2 are coefficients that are established from the test data.

For *connections* between cold-formed steel members and sheathing made of gypsum board or OSB, coefficients for *connections* subjected to in-plane shear and for pull-through/rotation were proposed by Batista Abreu et al. (2021) based on the tests reported by Batista Abreu (2015) and Chen et al. (2016). These coefficients are given in Table C-4.2.3.3-1 and Table C-4.2.3.3-2. Additional related testing is also reported in Abeysiriwardena et al. (2021).

Table C-4.2.3.3-1
Coefficients to Determine Retention Factors for Stiffness ($k_{k,V}$) and Strength (k_V) of in-Plane Lateral Resistance of CFS-to-Sheathing Connections for Gypsum Board and OSB

		a	b	c	T_1 °F (°C)	T_2 °F (°C)
Gypsum	$k_{k,V}$	30	2.8	0.15	68 (20)	932 (500)
	k_V	30	2.5	0.18	68 (20)	932 (500)
OSB	$k_{k,V}$	1	1.5	0.30	68 (20)	662 (350)
	k_V	8	3	0.05	68 (20)	662 (350)

Table C-4.2.3.3-2
Coefficients to Determine Retention Factors for Stiffness ($k_{k,T}$) and Strength (k_T) of Pull-Through Resistance of CFS-to-Sheathing Connections for Gypsum Board and OSB

		a	b	c	T_1 °F (°C)	T_2 °F (°C)
Gypsum	$k_{k,T}$	8	1.5	0	68 (20)	1112 (600)
	k_T	30	2.6	0	68 (20)	1112 (600)
OSB	$k_{k,T}$	1	1.2	0	68 (20)	662 (350)
	k_T	1	1.2	0	68 (20)	662 (350)

For *connections* between cold-formed steel members and steel sheathing, coefficients for in-plane strength were proposed based on the tests reported by Yan, et al. (2012). These coefficients are given in Table C-4.2.3.3-3.

Table C-4.2.3.3-3
Coefficients to Determine Retention Factors for Strength (k_u) of In-Plane Lateral Resistance of CFS-to-Sheathing Connections for Steel Sheathing

		a	b	c	T_1 °F (°C)	T_2 °F (°C)
Steel	k_u	80	5.2	0.03	68 (20)	1832 (1000)

4.2.4 Structural Integrity and Strength at Elevated Temperatures

The resistance of the structural system in the *design-basis fire* may be determined by:

(1) Structural analysis of individual structural components (with their fire protection material as applicable) where the effects of restraint to thermal expansion and restraint to thermal bowing may be ignored but the reduction in strength and *stiffness* with increasing temperature is incorporated.

(2) Structural analysis of assemblies of several components (with their fire protection material as applicable) where the effects of restrained thermal expansion and restrained thermal bowing are considered by incorporating geometric and material nonlinearities.

(3) Global structural analysis where restrained thermal expansion and restrained thermal bowing are considered in addition to material degradation and geometric nonlinearity.

4.2.4.1 Design by Advanced Methods of Analysis

Advanced methods of analysis are required when the designer needs to consider the overall structural system response to fire, the interaction between structural components in fire, the response under non-standard *design-basis fires*, the ability to survive burnout, or the residual strength of the structural system following a fire. ASCE 7 (ASCE, 2016) Appendix E allows the use of advanced analysis for the performance-based design of structures in fire. Gernay and Khorasani (2020) described and applied a systematic procedure to conduct a performance-based fire design of a building using advanced analysis, including the selection of performance objectives, the modeling of *design-basis fires*, thermal response, structural response of the overall structural system with verification of the different *limit states*, and assessment of the numerical results against the predefined performance objectives. Gunalan and Mahendran (2013a and 2013b) developed finite element (FE) models in ABAQUS (2009) to simulate the behavior of load-bearing cold-formed steel walls subjected to the standard ISO 834 fire on one side. Their FE analyses, validated against 10 full scale fire tests in transient and steady-state conditions, confirmed the superior fire performance of externally insulated walls over cavity insulated walls in the studied assembly. The analyses also showed the relation between load ratio versus temperature at failure of the hot flange of the stud, regardless of the number of plasterboard and type of insulation, which supports the validity of the limiting (i.e. critical) temperature concept. Yang and Lu (2016a, 2016b, 2017) studied the effect of web perforations on the behavior of cold-formed steel slender column subjected to non-uniform cross-sectional distribution of *elevated temperature*. They performed a three-dimensional FE analysis of an entire cold-formed steel wall frame. Their modeling showed the importance of accurately accounting for assembly details in the thermal model, such as cavity insulation, *web* perforation and bridging. Arrais et al. (2016) modeled cold-formed steel beams with lipped channel sections using the finite element software SAFIR (Franssen and Gernay, 2017) with shell elements. They conducted parametric numerical analysis on cross-sections chosen to represent a wide range of section slenderness and different susceptibility to *local* and *distortional buckling*. Their results showed that existing simple calculation methods in the Eurocode 3 are often too conservative for these components. Other examples of application of advanced methods for the analysis of cold-formed steel members and systems in the fire situation can be found in the literature (Hancock, 2016; Ni et al., 2022).

4.2.4.2 Design by Simple Methods of Analysis

Simple methods of analysis can be applied for the evaluation of the performance of individual cold-formed steel structural components at *elevated temperatures*. These methods are not able to capture the interaction between components during exposure to fire, notably the forces that can develop due to restrained thermal expansion in statically indeterminate structures. Fundamentally, simple methods in fire design are thus based on the assumption that applied forces and moments at the boundary of the component remain unchanged throughout the fire exposure.

An important distinction has to be made between cold-formed steel components in which the steel temperature can reasonably be assumed to be uniform, and those where a significant thermal gradient develops across the section. Generally, the uniform temperature assumption may be appropriate where a cold-formed steel component is uniformly heated along its entire length and on its two faces (in case of a wall or floor) or around its entire perimeter (in case of a column), and the protection system on the component is uniform. Conversely, a common example of non-uniform temperature is that of a thermally protected cold-formed steel wall (e.g. with gypsum board) heated on one side and not on the other, as is usual when the wall constitutes the boundary of enclosure of a fire compartment. Where non-uniform temperature develops, differential thermal elongation between the two sides of the section (e.g. the *flanges* of a stud member) results in thermal bowing. This thermal bowing may generate large out-of-plane deformations which, in members loaded in their plane, result in second order bending moments.

In members subject to uniform temperature distribution, the demand (*required strength*) is assumed constant during the fire exposure. The capacity (*available strength*) is evaluated using ambient temperature design capacity rules with appropriately reduced mechanical properties in accordance with *Specification* Section 4.2.3, at the temperature developed by the *design-basis fire*. The capacity can be evaluated using the *Direct Strength Method* (*DSM*) of the *Specification* developed for cold-formed steel members at ambient temperature by Schafer (2002, 2008), where retention factors are applied to the material properties. The validity of this approach has been verified by different researchers through comparison with advanced analysis results and *elevated temperature* tests. Shahbazian and Wang (2012) applied the *DSM* for calculating *distortional buckling* capacity of cold-formed thin-walled steel columns with uniform and non-uniform *elevated temperatures*, and concluded that the existing *DSM distortional buckling* curve for ambient temperature application is also applicable for columns with uniform temperature distributions in the cross-section, but is unconservative for columns with non-uniform temperature distributions in the cross-section. Chen and Young (2007a and 2007b) studied the behavior of cold-formed steel lipped channel columns at uniform *elevated temperatures* through finite element analysis and simple analysis; they concluded that the *effective width* and *direct strength methods* with reduced material properties are able to predict the cold-formed steel lipped channel column strengths at *elevated temperatures*, and recommended the *DSM* for its simplicity.

In members subject to non-uniform temperature distributions, care should be taken to account for the effect of thermal bowing and associated second-order response. Accordingly, the demand (*required strength*) must consider the second-order response consistent with the non-uniform temperature distribution. Methods have been proposed to account for the effects of thermal bowing, magnification effects and neutral axis shift for cold-formed steel wall systems (Gunalan and Mahendran, 2013a and 2013b). The capacity (*available strength*) needs to consider *local*, *distortional*, and global *buckling* and *yielding* in the cross-section consistent with the non-uniform temperature distribution. Simple methods of analysis are provided in Appendix G of the AS/NZS 4600:2018 (2018). The methods address the design of *cold-formed steel structural members* (studs) in load-bearing walls, which are subjected to combined axial compression and bending action due to non-uniform temperature distribution across their cross-section. The methods also address the design of *cold-formed steel structural members* (joists) in floors, for which there is a neutral axis shift (as for wall studs) due to varying modulus of elasticity and *local buckling*.

4.3 Design by Qualification Testing

Qualification testing should be made in conformance with ASTM E119 in the United States and Mexico and CAN/ULC-S101 in Canada. Additional information is available in the AISC 360, Commentary to Appendix 4 (AISC, 2016b).

4.4 Design by Combination of Analysis and Testing

Fire testing is expensive, and a fire test only provides performance of a specific assembly against one empirical temperature curve. The provisions of *Specification* Appendix 4 allow qualifying *cold-formed steel structural members* and components based on combinations of both analysis and testing. Notably, these provisions provide a means to demonstrate the impact of small changes (such as new fasteners, new stud shapes, or details) on a qualified assembly without fire testing.

Temperature distributions obtained from a test can be used as thermal inputs to analyses conducted in accordance with *Specification* Sections 4.2.3 and 4.2.4. For example, steel temperatures in a thermally protected assembly can be measured throughout a fire test; then these steel temperatures can be used to evaluate material retention factors for use in the *Direct Strength Method* of the *Specification*. Measured temperatures can also be used in finite element models to evaluate the structural response at *elevated temperature*. This allows analyzing the structural integrity and strength at *elevated temperature* of different structural designs without repeating the fire test.

Alternatively, the structural integrity at *elevated temperature* can be determined by testing with *elevated temperatures* predicted by analysis consistent with *Specification* Sections 4.2.1 to 4.2.3. For example, testing of a cold-formed steel component provides its limiting (critical) failure temperature. The duration of fire endurance can then be evaluated for different designs of thermal protection and different *design-basis fires* by performing heat transfer simulations, provided the thermal properties of the materials are fully characterized. For load-bearing component, it is important to note that the limiting (critical) failure temperature depends on the load ratio.

These provisions thus provide a means to leverage ASTM E119 for similar assemblies by performing analysis instead of additional testing, or to test unique members or assemblages under temperature to establish their strength. Other combinations of analysis and testing are also possible.

APPENDIX 5, EVALUATION OF EXISTING STRUCTURES

Appendix 5 is new in this edition of the *Specification* and provides criteria evaluating existing structures and components under static *loads* by structural analysis, *load* tests, or by a combination of structural analysis and *load* tests.

5.1 General Provisions

The *load* combinations referred to in this Appendix pertain to static loading because it is the most prevalent condition encountered. If other loading conditions are a consideration, the appropriate *load* combination from ASCE/SEI 7 or from the *applicable building code* should be used.

For seismic evaluation of existing buildings, ASCE/SEI 41 (ASCE, 2023) provides a three-tiered process for determination of the design and construction adequacy of existing buildings to resist earthquakes. The standard defines evaluation requirements as well as detailed evaluation procedures. Buildings may be evaluated in accordance with this standard for life safety or immediate occupancy performance levels. Where seismic rehabilitation of existing steel buildings is required, engineering of seismic rehabilitation work may be performed in accordance with the ASCE/SEI 41 (ASCE, 2023) standard or other standards. Use of this standard for seismic evaluation and seismic rehabilitation of existing steel buildings is subject to the approval of the *authority having jurisdiction*.

5.2 Material Properties

5.2.1 Determination of Required Tests

The extent of tests required depends on the nature of the project, the criticality of the structural system or member evaluated, and the availability of records pertinent to the project.

5.2.2 Material Properties

Samples required for tensile tests should be removed from regions of reduced *stress*, such as at *flange* tips at beam ends and external plate edges, to minimize the effects of the reduced area. The number of tests required will depend on whether they are conducted to merely confirm the strength of a known material or to establish the strength of some other material.

It should be recognized that the *yield stress* determined by standard ASTM methods and reported by the certified testing report is somewhat greater than the static *yield stress* because of dynamic effects of testing. Also, the test specimen location may have an effect. These effects have already been accounted for in the *nominal strength* [*resistance*] equations. However, when strength evaluation is done by *load* testing, this effect should be accounted for in test planning because *yielding* will tend to occur earlier than otherwise anticipated.

5.2.5 Fasteners

Because *connections* typically have a greater reliability index than structural members, strength testing of fasteners is not usually necessary. Strength testing of fasteners is required where they cannot properly be identified otherwise.

5.3 Evaluation by Structural Analysis

5.3.1 Dimensional Data

Fabrication imperfections and assembly fit-up issues that cause additional *P-Δ effects* (out-of-plumb columns) could have a significant impact on the strength of the structure. Therefore, particular attention should be given to deviations causing additional *P-Δ effects*.

5.3.2 Strength Evaluation

Resistance and *safety factors* reflect variations in determining strength of members and *connections*, such as uncertainty in theory and variations in material properties and dimensions. If an investigation of an existing structure indicates that there are variations in material properties or dimensions significantly greater than those anticipated in new construction, the use of more conservative values should be considered.

5.4 Evaluation by Load Tests

5.4.1 Determination of Load Rating by Testing

The *load* rating established by testing presumes ϕ = 1.0 for all failure modes. The actual loading during the test, including distribution, should simulate applicable loading and deformation conditions as closely as possible.

It is essential that the necessary precautions be made to ascertain that the structure does not fail catastrophically during testing. A careful assessment of structural conditions before testing is important. This includes accurate measurement and characterization of the size and strength of members, *connections*, and details. Safety regulations of OSHA and other pertinent bodies should be followed. Shoring and scaffolding should be used as required in the proximity of the test area to mitigate against unexpected circumstances. Deformations should be carefully monitored, and structural conditions should be continually evaluated. In some cases, it may be desirable to monitor strains as well.

When deflections become excessive, the tests should be terminated at a safe level even if the desired loading has not been achieved. Incremental loading is specified so that deformations can be accurately monitored, and the performance of the structure carefully observed. Load increments should be small enough initially so that the onset of *local buckling*, *distortional buckling*, global *buckling*, or *yielding* can be determined. The increment can be reduced as the level of inelastic behavior increases. The behavior at this level should be carefully evaluated to determine when to safely terminate the test. Periodic unloading, after the onset of inelastic behavior, will help determine when to terminate the test to avoid excessive permanent deformation or catastrophic failure.

The margin of safety at the maximum *load* level used in the test may be very small, depending on such factors as the original design, the purpose of the tests, and the condition of the structure. Thus, it is imperative that appropriate safety measures be adopted. It is recommended that the maximum live *load* used for *load* tests be selected conservatively. It should be noted that experience in testing more than one bay of a structure is limited.

The provision limiting increases in deformations for a period of one hour provides a means to confirm that the structure is stable at the *loads* evaluated.

5.4.2 Serviceability Evaluation

In certain cases, serviceability performance may be determined by *load* testing. It should be recognized that complete recovery (return to the pre-tested deflected shape) after removal of maximum *load* is unlikely because of phenomena such as local *yielding*, slip at the slab interface in composite construction, creep in concrete slabs, localized crushing or deformation at shear *connections* in slabs, slip in *connections* or bearing deformations at fasteners, and effects of continuity. Because most structures exhibit some slack when *load* is first applied, it is appropriate to project the *load*-deformation curve back to zero *load* to determine the slack and exclude it from the recorded deformations. Where desirable, the applied *load* sequence can be repeated to demonstrate that the structure is essentially elastic under service *loads* and that the permanent set is not detrimental.

5.5 Evaluation Report

Extensive evaluation and *load* testing of existing structures is often performed when appropriate documentation no longer exists or when there is considerable disagreement about the condition of a structure. The resulting evaluation is only effective if well documented, particularly when *load* testing is involved. Furthermore, as time passes, various interpretations of the results can arise unless all parameters of the structural performance, including material properties, strength and *stiffness*, are well documented.

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APPENDIX A, COMMENTARY ON PROVISIONS APPLICABLE TO THE UNITED STATES AND MEXICO

This commentary on Appendix A provides a record of reasoning behind, and justification for, provisions that are applicable to the United States and Mexico. The format used herein is consistent with that used in Appendix A of the *Specification*.

I6.2.2 Flexural Members Having One Flange Fastened to a Standing Seam Roof System

For beams supporting a standing seam roof system, e.g., a roof *purlin* subjected to dead plus live *load*, or uplift from wind *load*, the bending capacity is greater than the bending strength of an unbraced member and may be equal to the bending strength of a fully braced member. The bending capacity is governed by the nature of the loading, gravity or uplift, and the nature of the particular standing seam roof system. Due to the availability of many different types of standing seam roof systems, an analytical method for determining positive and negative bending capacities has not been developed at the present time. However, in order to resolve this issue relative to the gravity loading condition, Section I6.2.2 was added in the 1996 edition of the AISI *Specification* for determining the *nominal flexural strength* [*resistance*] of beams having one *flange* fastened to a standing seam roof system. In *Specification* Equation I6.2.2-1, the reduction factor, R, can be determined by AISI S908. Application of the base test method for uplift loading was subsequently validated after further analysis of the research results.

The provisions of *Specification* Section H4, Combined Bending and Torsion, should not be used in combination with the bending provisions in *Specification* Section I6.2.2 since these provisions are based on tests in which torsional effects are present.

I6.2.4 Z-Section Compression Members Having One Flange Fastened to a Standing Seam Roof

The strength of axially loaded Z-sections having one *flange* attached to a standing seam roof may be limited by either a combination of *torsional buckling* and lateral *buckling* in the plane of the roof, or by *flexural buckling* in a plane perpendicular to the roof. As in the case of Z-sections carrying gravity or wind *loads* as beams, the roof diaphragm and *purlin* clips provide a degree of torsional and lateral bracing restraint that is significant, but not necessarily sufficient, to develop the full strength of the cross-section.

Specification Equation I6.2.4-1 predicts the lateral *buckling* strength using an ultimate axial *buckling stress* ($k_{af}RF_y$) that is a percentage of the ultimate flexural *stress* (RF_y) determined from uplift tests performed using AISI S908, *Base Test Method for Purlins Supporting a Standing Seam Roof System*, as published by AISI (2013f). This equation, developed by Stolarczyk, et al. (2002), was derived empirically from elastic finite element *buckling* studies and calibrated to the results of a series of tests comparing flexural and axial strengths using the uplift "Base Test" setup. The *full unreduced cross-sectional area*, A, has been used rather than the *effective area*, A_e, because the ultimate axial *stress* is generally not large enough to result in a significant reduction in the *effective area* for common cross-section geometries.

Specification Equation I6.2.4-1 may be used with the results of uplift "Base Tests" conducted with and without discrete point bracing. There is no limitation on the minimum length because Equation I6.2.4-1 is conservative for spans that are smaller than those tested under the "Base Test" provisions.

The strength of longer members may be governed by *axial buckling* perpendicular to the roof; consequently, the provisions of *Specification* Section E2 should also be checked for *buckling* about the strong axis.

I6.3.1a Strength of Standing Seam Roof Panel Systems

The introduction of the wind uplift loading *required strength* factor of 0.67 was a result of research conducted to correlate the static uplift capacity represented by tests performed in accordance with AISI S906 and the dynamic behavior of real wind, by Surry, et al. (2007). This research utilized two separate methods of comparison. The first method utilized full-scale tests conducted at Mississippi State University (MSU) using simulated wind l*oads* on a portion of a standing seam metal roof. The second method utilized model-scale wind tunnel tests carried out at the University of Western Ontario of an aeroelastic "failure" model of the same roof system. In spite of these significantly different approaches, the results obtained were very consistent. It was found that the ASTM E1592 uniform pressure test contains conservatism of about 50 percent for the roof system tested by both approaches, and up to about 80 percent for the other roof systems tested only at MSU. This conservatism arises if the roof system is required to withstand the code-recommended pressure applied as uniform pressure in the ASTM E1592 test, without accounting for the reality of the dynamic spatially-varying properties of the wind-induced pressures. The limits of applicability of this factor (panel *thickness* and width) are conservatively listed based on the scope of the research. The failure mode is restricted to those failures associated with the *load* in the clip because this was how the research measured and compared the static and dynamic capacities. Therefore, the 2012 *Specification* was clarified with respect to the strength factor of 0.67 applying to the clips and fasteners as well as the standing seam roof panels. The required strength factor of 0.67 is not permitted to be used with other observed failures. In addition, the research does not support or confirm whether interpolation would be appropriate between ASTM E1592 tests of the same roof system with different spans, where one test meets the requirements, such as a clip failure, and another test does not, such as a panel failure.

It was determined that the strength factor, 0.67, when applied to the corner and edge zones of steeper slope roofs (greater than 27-degree slope) could yield a nominal wind *load* less than that in the field of the roof, based on ASCE 7 (2010). So, the limiting value of the wind *load* in the field of the roof was introduced in the 2012 *Specification*.

An AISI interpretation was issued in 2012 that clarified the strength factor, 0.67, that was based on research that compared the static and dynamic capacities of these types of roof systems, is justified to be used with the *loads* or *load* combinations in the International Building Code (IBC), since this strength factor is based on structural behavior caused by rate or duration of *load*. Therefore, this 0.67 factor is not duplicative of the consideration given for multiple *variable loads* in both the strength design *load* combinations and the allowable *stress load* combinations used in IBC and ASCE/SEI 7 (ASCE, 2010). It would be appropriate to utilize the 0.67 factor on the *nominal* wind *load* for any *load* combination that includes wind uplift as long as all of the conditions stated in *Specification* Section I6.3.1a (Appendix A) are met.

In 2022, the areas of the roof where the 0.67 factor applies were modified to allow application of the provisions independently from the edge and corner zone definitions shown in ASCE/SEI 7, and particularly those introduced in the 2016 edition of ASCE/SEI 7 for low-sloped gable roofs. The editions of ASCE/SEI 7 where use of these provisions are acceptable have been identified in a User Note with the intention that these provisions are examined for continued acceptability under each new edition of the ASCE/SEI 7 Standard.

It is recognized that there are other analytical tools available, especially advanced finite element analyses, that have made strides in replicating the behavior of standing seam roof systems and determining their dynamic uplift capacity. Therefore, alternative means of analysis may be available to compare the dynamic and static behavior that could be used to extend the applicability of this method, provided it was sufficiently calibrated to the existing test data. Any alternative method should also comply with the *rational engineering analysis* requirements of Section A1.2.6, including the appropriate *safety factor* and *resistance factor* for members and connections.

J3.4 Shear and Tension in Bolts

For the design of bolted *connections*, the allowable shear *stresses* for bolts have been provided in the AISI *Specification* for cold-formed steel design since 1956. However, the allowable tension *stresses* were not provided in *Specification* Section J3.4 for bolts subjected to tension until 1986. In *Specification* Table J3.4-1(a), the allowable *stresses* specified for ASTM A307 (d ≥ 1/2 inch (12.7 mm)), A325, and A490 bolts were based on Section 1.5.2.1 of the AISC Specification (AISC, 1978). It should be noted that the same values were also used in Table J3.2 of the AISC *ASD* Specification (AISC, 1989). For ASTM A307, A449, and A354 bolts with diameters less than 1/2 inch (12.7 mm), the allowable tension *stresses* were reduced by 10 percent, as compared with these bolts having diameters not less than 1/2 inch (12.7 mm), because the average ratio of (tensile-*stress* area)/(gross-area) for 1/4-inch (6.35 mm) and 3/8-inch (9.53 mm) diameter bolts is 0.68, which is about 10 percent less than the average area ratio of 0.75 for 1/2-inch (12.7 mm) and 1-inch (25.4 mm) diameter bolts. In the AISI *ASD/LRFD Specification* (AISI, 1996), Table J3.4-1(a) provided *nominal tensile strengths* [*resistance*] for various types of bolts with applicable *safety factors*. The allowable tension *stresses* computed from F_{nt}/Ω were approximately the same as those permitted by the AISI 1986 *ASD Specification*. The same table also gave the *resistance factor* to be used for the *LRFD* method. In 2012, the table values were realigned with the AISC Specification (AISC, 2010a).

The design provisions for bolts subjected to a combination of shear and tension were added in AISI *Specification* Section J3.4 in 1986. Those design equations were based on Section 1.6.3 of the AISC Specification (AISC, 1978) for the design of bolts used for bearing-type *connections*.

In 1996, tables which listed the equations for determining the reduced nominal tension *stress*, F'_{nt}, for bolts subjected to the combination of shear and tension were included in the *Specification* and were retained in the 2001 edition. In 2007, those tables were replaced by *Specification* Equations J3.4-2 and J3.4-3 to determine the reduced tension *stress* of bolts subjected to the combined tension and shear. *Specification* Equations J3.4-2 and J3.4-3 were adopted to be consistent with the AISC Specification (AISC, 2005).

In 2016, Table J3.4-1(a) was brought into agreement with AISC Table J3.2 (AISC, 2010a) in all related respects, both with regard to *safety factors* and F_n *nominal strengths*. As previously stated, the *nominal tensile strength* values have been reduced by 10 percent for all bolts and threaded fasteners less than 1/2-in. (12-mm) diameter. The *nominal shear strength* values have also been reduced by 10 percent when threads are not excluded from the shear planes for all bolts and threaded fasteners less than 1/2-in. (12-mm) diameter.

In 2022, ASTM F3148 bolts were added to *Specification* Table J3.4-1(a). This change aligns with similar changes made in the AISC (2022) and RCSC (2020) Specifications.

Note that when the required *stress*, f, in either shear or tension, is less than or equal to 20 percent of the corresponding available *stress*, the effects of combined *stress* need not be investigated.

For bolted *connection* design, the possibility of pull-over of the connected sheet at the bolt head, nut, or washer should also be considered when bolt tension is involved, especially for thin sheathing material. For *non-symmetric sections*, such as C- and Z-sections used as *purlins* or *girts*, the problem is more severe because of the prying action resulting from rotation of the member which occurs as a consequence of loading normal to the sheathing. The designer should refer to applicable product code approvals, product specifications, other literature, or tests.

For design tables and example problems on bolted connections, see Part IV of the AISI *Cold-Formed Steel Design Manual* (AISI, 2017).

APPENDIX B, COMMENTARY ON PROVISIONS APPLICABLE TO CANADA

This commentary on Appendix B of the *Specification* provides a record of reasoning behind, and justification for, provisions that are applicable only to Canada. The format used herein is consistent with that used in Appendix B of the *Specification*.

C2a Lateral and Stability Bracing

The provisions of this section cover members loaded in the plane of the *web*. Conditions may occur that cause a lateral component of the *load* to be transferred through the bracing member to supporting structural members. In such a case, these lateral forces shall be additive to the requirements of this section. The provisions in the *Specification* recognize the distinctly different behavior of the members to be braced, as defined in Sections C2.1 and C2.2 of this Appendix. The term "discrete braces" is used to identify those braces that are only connected to the member to be braced for this express purpose.

C2.1a Symmetrical Beams and Columns

C2.1.1 Discrete Bracing for Beams

This section was revised to retain the two percent requirement for the compressive force in the compressive *flange* of a flexural member at the braced location only. The discrete bracing provisions for columns are provided in *Specification* Section C2.3.

C2.2a C-Section and Z-Section Beams

This section covers bracing requirements of channel and Z-sections and any other section in which the applied *load* in the plane of the *web* induces twist.

C2.2.2 Discrete Bracing

This section provides for brace intervals to prevent the member from rotating about the shear centre for channels or from rotating about the point of symmetry for Z-sections. The spacing must be such that any *stresses* due to the rotation tendency are small enough so that they will not significantly reduce the *load*-carrying capacity of the member. The rotation must also be small enough (in the order of 2°) to be not objectionable as a service requirement.

Based on tests and the study by Winter, et al. (1949b), it was found that these requirements are satisfied for any type of *load* if braces are provided at intervals of one-quarter of the span, with the exception of concentrated *loads* requiring braces near the point of application.

Fewer brace points may be used if it can be shown to be acceptable by rational analysis or testing in accordance with Section K2 of the *Specification*, recognizing the variety of conditions, including the case where *loads* are applied out of the plane of the *web*.

For sections used as *purlins* with a standing seam roof, the number of braces per bay is often determined by rational analysis and/or testing. The requirement for a minimum number of braces per bay is to recognize that predictability of the lateral support and rotational restraint is limited on account of the many variables such as fasteners, insulation, friction coefficients, and distortion of roof panels under *load*.

C2.2.3 One Flange Braced by Deck, Slab, or Sheathing

Forces generated by the tendency for lateral movement and/or twist of the beams, whether cumulative or not, must be transferred to a sufficiently stiff part of the framing system. There are several ways in which this transfer may be accomplished:

(a) By the deck, slab, or sheathing providing a rigid diaphragm capable of transferring the forces to the supporting structure;

(b) By arranging equally loaded pairs of members facing each other;

(c) By direct axial force in the covering material that can be transferred to the supporting structure or balanced by opposing forces;

(d) By a system of sag members such as rods, angles, or channels that transfer the forces to the supporting structure; or

(e) By any other method that designers may select to transfer forces to the supporting structure.

For all types of single *web* beams, the *flange* that is not attached to the deck or sheathing material may be subject to compressive *stresses* under certain loading arrangements, such as beams continuous over supports or under wind *load*. The elastic lateral support to this *flange* provided through the *web* may allow an increase in limit *stress* over that calculated by assuming that the compressive *flange* is a column, with pinned ends at points of lateral bracing. Research indicates that the compressive limit *stress* is also sensitive to the rotational flexibility of the joint between the beam and the deck or sheathing material.

This section is intended to apply even when the *flange* that is not attached to the sheathing material is in tension.

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