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

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The material contained herein has been developed by a joint effort of the American Iron and Steel Institute Committee on Specifications, the Canadian Standards Association Technical Committee on Cold-Formed Steel Structural Members (S136), and Camara Nacional de la Industria del Hierro y del Acero (CANACERO) in Mexico. The organizations and the Committees have made a diligent effort to present accurate, reliable, and useful information on cold-formed steel design. The Committees acknowledge and are grateful for the contributions of the numerous researchers, engineers, and others who have contributed to the body of knowledge on the subject. Specific references are included in the *Commentary* on the *Specification*.

With anticipated improvements in understanding of the behavior of cold-formed steel and the continuing development of new technology, this material may eventually become dated. It is anticipated that future editions of this specification will update this material as new information becomes available, but this cannot be guaranteed.

The materials set forth herein are for general information only. They are not a substitute for competent professional advice. Application of this information to a specific project should be reviewed by a registered professional engineer. Indeed, in most jurisdictions, such review is required by law. Anyone making use of the information set forth herein does so at their own risk and assumes any and all resulting liability arising therefrom.

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PREFACE

This is the premier edition of the *North American Specification for the Design of Cold-Formed Steel Structural Members* and, as its name implies, is intended for use throughout Canada, Mexico and the United States. This *Specification* supercedes the previous editions of the *Specification for the Design of Cold-Formed Steel Structural Members* published by American Iron and Steel Institute and the *S136-94 Standard for Cold-Formed Steel Structural Members* published by the Canadian Standards Association.

The *Specification* was developed by a joint effort of the American Iron and Steel Institute's Committee on Specifications, the Canadian Standard Association's Technical Committee on Cold-Formed Steel Structural Members (S136), and Camara Nacional de la Industria del Hierro y del Acero (CANACERO) in Mexico. This was coordinated through the North American Specifications Committee which contained three members each from the AISI Committee on Specifications, CSA's S136 Committee, and CANACERO.

Since the *Specification* is intended for use in Canada, Mexico and the United States, it was necessary to develop a format that would facilitate the allowance of unique requirements in each country. This resulted in a format that contained a basic document, Chapters A through G, that is intended for use in all three countries, and three country specific appendices (A to C). Appendix A is for use in the United States, Appendix B is for use in Canada and Appendix C is for use in Mexico.

This *Specification* provides an integrated treatment of Load and Resistance Factor Design (LRFD) and Allowable Stress Design (ASD) and Limit States Design (LSD). This is accomplished by including the appropriate resistance factors (ϕ) for use with LRFD and LSD and the appropriate factors of safety (Ω) for use with ASD It should be noted that Limit States Design (LSD) is limited to Canada and Load and Resistance Factor Design (LRFD) and Allowable Stress Design (ASD) are limited to use in Mexico and the United States.

The basic document also contains some terminology that is defined differently between Canada and the Unites States and Mexico. These differences are set out in the Glossary.

The *Specification* provides well defined procedures for the design of load carrying cold-formed steel members in buildings, as well as other applications provided that proper allowances are made for dynamic effects. The provisions reflect the results of continuing research to develop new and improved information on the structural behavior of cold-formed steel members. The success of these efforts is evident in the wide acceptance of the predecessor documents to these *Specifications*.

The AISI and CSA consensus committees responsible for developing these provisions provide a balanced forum with representatives from steel producers, fabricators, users, educators, researchers, and building code regulators. They are composed of engineers with a wide range of experience and high professional standing from throughout Canada, Mexico and the United States. AISI, CANACERO and CSA acknowledge the continuing dedication by the members of the specifications committees and their subcommittees. The current membership of these committees follows this Preface.

Because this is the first Edition of the *North American Specification*, no attempt will be made here to list provisions that represent changes to the documents that it supercedes. Such changes are numerous and are distributed throughout.

Users of the *Specification* are encouraged to offer comments and suggestion for improvement.

American Iron and Steel Institute
Canadian Standards Association
Camara Nacional de la Industria del Hierro y del Acero

October 2001

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A_{gv}	Gross area subject to shear	E5.3
A_{nt}	Net area subject to tension	E5.3
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A_n	Net area of cross section	C2, E3.2
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A_s	Gross area of stiffener	B5.1
A'_s	Effective area of stiffener	B4, B4.1, B4.2
A_{st}	Gross area of shear stiffener	C3.6.2
A_t	Net tensile area	G4
Aw	Area of web area	C3.2.1
A_{wn}	Net web area	E5.1
a	Shear panel length of unreinforced web element, or the distance between	C3.2.1, C3.6.2

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Symbol	Definition	Section
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Symbol	Definition	Section
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C_s	Coefficient for lateral torsional buckling	C3.1.2.1
C_{TF}	End moment coefficient in interaction formula	C3.1.2.1
C_{th}	Coefficient for lateral bracing of Z-sections	D3.2.1
C_{tr}	Coefficient for lateral bracing of Z-sections	D3.2.1
C_v	Shear stiffener coefficient	C3.6.2
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C_y	Compression strain factor	C3.1.1
C_1	Term used to compute shear strain in wall board	D4.1
C	Bearing factor	E3.3.1
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C_h	Web slenderness coefficient	C3.4.1
C_N	Bearing length coefficient	C3.4.1
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Symbol	Definition	Section
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E	Live load due to earthquake	A3.1, A6.1.2
E_o	Initial column imperfection; a measure of the initial twist of the stud from the initial, ideal, unbuckled location	D4.1
E_1	Term used to compute shear strain in wallboard	D4.1
E'	Inelastic modulus of elasticity	D4.1

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Symbol	Definition	Section
e	The distance measured in the line of force from the center of a standard hole to the nearest edge of an adjacent hole or to the end of the connected part toward which the force is directed	E3.1, E3.1a
e	Distance measured in the line of force from the center of a standard hole to the nearest end of the connected part.	E4.3.2
e_{min}	Minimum allowable distance measured in the line of force from the centerline of a weld to the nearest edge of an adjacent weld or to the end of the connected part toward which the force is directed	E2.2.1, E2.2.2
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F	Nominal tensile or shear strength	E3.4
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F_t	Nominal tensile stress in flat sheet	E3.2

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Symbol	Definition	Section
F_u	Tensile strength as specified in Sections A2.1 or A2.3.2	A2.3.2, C2, E2.2.1, E2.2.2, E2.3, E2.4, E2.5, E2.7, E3.1, E3.2, E3.3.1, E3.3.2, E4.3.2, E5.1, E5.3
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F_{u2}	Tensile strength of member not in contact with the screw head	E4, E4.3.1, E4.4.1
F_v	Nominal shear stress	E3.2.1
F_y	Yield point used for design, not to exceed the specified yield point or established in accordance with Section F3, or as increased for cold work of forming in Section A7.2 or as reduced for low ductility steels in Section	A1.2, A2.3.2, A7.1, A8.1, A7.2, B2.1, C2, C3.1.1, C3.1.2.1, C3.1.2.2, C3.1.3, C3.2.1, C3.1.4, C3.4.1, C3.5.1, C3.5.2, C3.6.1, C3.6.2, C4, C4.2, C5.1.1, C5.1.2, C5.2.1, C5.2.2, C6, C6.1, C6.2, D1.2, D4.1, E2.1, E2.2.2, E5.2, G1
F_{ya}	Average yield point of section	A7.2
F_{yc}	Tensile yield point of corners	A7.2
F_{yf}	Weighted average tensile yield point of the flat portions	A7.2, F3.2
F_{ys}	Yield point of stiffener steel	C3.6.1
F_{yv}	Tensile yield point of virgin steel specified by Section A2 or established in accordance with Section F3.3	A7.2
f	Stress in the compression element computed on the basis of the effective design width	B2.1, B2.2, B2.4, B3.1, B3.2, B4, B4.1, B4.2, B5.2
f_{av}	Average computed stress in the full, unreduced flange width	B1.1

SYMBOLS AND DEFINITIONS

Symbol	Definition	Section
f_c	Stress at service load in the cover plate or sheet	D1.2
f_d	Computed compressive stress in the element being considered. Calculations are based on the effective section at the load for which deflections are determined.	B2.1, B2.2, B3.1, B4.1, B4.2 B5.1.1, B5.1.2, B5.2
f_{d1}, f_{d2}	Computed stresses f_1 and f_2 as shown in Figure B2.3-1. Calculations are based on the effective section at the load for which serviceability is determined	B2.3
f_{d3}	Computed stress f_3 in edge stiffener, as shown in Figure B4-2. Calculations are based on the effective section at the load for which serviceability is determined	B3.2
f_v	Computed shear stress on a bolt	E3.4
f_1, f_2	Web stresses defined by Figure B2.3-1	B2.3, B2.4
f_1	Uniform compressive stress acting on the flat element	B5.1, B5.1.1, B5.1.2, B5.2
f_3	Edge stiffener stress defined by Figure B4-2	B3.2
G	Shear modulus of steel, 11,300 ksi (78,000MPa or 795,000 kg/cm^2)	C3.1.2.1, C3.1.2.2, D4.1
G′	Inelastic shear modulus	D4.1
g	Vertical distance between two rows of connections nearest to the top and bottom flanges	D1.1
g	Transverse center-to-center spacing between fastener gage lines	E3.2
h	Depth of flat portion of web measured along the plane of web	B1.2, B2.4, C3.1.1, C3.2.1, C3.2.2, C3.4.1, C3.4.2, C3.5.1, C3.5.2, C3.6.2
h	Width of elements adjoining stiffened element	B5.1
h	Lip height as defined in Figures E2.5D to E2.5G	E2.5
h_o	Out-to-out width of the web as defined in Figure B2.3-2	B2.3
h_o	Depth of web hole	B2.4
h_{wc}	Coped flat web depth	E5.1

SYMBOLS AND DEFINITIONS

Symbol	Definition	Section
I_a	Adequate moment of inertia of stiffener so that each component element will behave as a stiffened element	B1.1, B4, B4.1, B4.2
I_s	Actual moment of inertia of the full stiffener about its own centroidal axis parallel to the element to be stiffened	B1.1, B4, B4.1, B4.2, C3.6.2
I_{smin}	Minimum moment of inertia of shear stiffener(s) with respect to an axis in the plane of web	C3.6.2
I_{sp}	Moment of inertia of stiffener about centerline of flat portion of element	B5.1, B5.1.1, B5.1.2
I_x, I_y	Moment of inertia of full section about principal axis	C3.1.2.1, C3.1.2.2, C5.2.1, C5.2.2, D3.2.2
I_{xy}	Product of inertia of full section about major and minor centroidal axes	D3.2.2, D4.1
I_{yc}	Moment of inertia of the compression portion of a section about the centroidal axis of the entire section parallel to the web, using the full unreduced section	C3.1.2.1
i	Index of stiffener	B5.1, B5.1.2
J	Saint-Venant torsion constant	C3.1.2.1, C3.1.2.2, D4.1
j	Section property for torsional-flexural buckling	C3.1.2.1
K	Effective length factor	C4.1, C4.5, D4.1
K′	A constant	D3.2.2
K_t	Effective length factor for torsion	C3.1.2.1
K_x	Effective length factor for buckling about x-axis	C3.1.2.1, C5.2.1, C5.2.2
K_y	Effective length factor for buckling about y-axis	C3.1.2.1, C3.1.2.2, C5.2.1, C5.2.2
k	Plate buckling coefficient	B2.1, B2.3, B3.1, B3.2, B4, B4.1, B4.2, B5.1, B5.2
k_d	Plate buckling coefficient for distortional buckling	B5.1, B5.1.1, B5.1.2
k_{loc}	Plate buckling coefficient for local sub-element buckling	B5.1, B5.1.1, B5.1.2
k_v	Shear buckling coefficient	C3.2.1, C3.6.2,

SYMBOLS AND DEFINITIONS

Symbol	Definition	Section
L	Full span for simple beams, distance between inflection points for continuous beams, twice the length of cantilever beams	B1.1
L	Span length	D3.2.1, D1.1
L	Length of weld	E2.1, E2.5
L	Length of longitudinal welds	E2.7
L	Length of seam weld not including the circular ends	E2.3
L	Length of fillet weld	E2.4
L	Length of the connection	E3.2
L	Unbraced length of member	C4.1, C4.5, C5.2.1, C5.2.2 D4.1
L	Overall length	D4.1
L	Live load	A3.1, A6.1.2, A6.1.2.2
L_{br}	Unsupported length between brace point or other restraint which restricts distortional buckling of element	B5.1, B5.1.1, B5.1.2
L_{st}	Length of transverse stiffener	C3.6.1
L_t	Unbraced length of compression member for torsion	C3.1.2.1
L_x	Unbraced length of compression member for bending about x-axis	C3.1.2.1, C5.2.1, C5.2.2
L_y	Unbraced length of compression member for bending about y-axis	C3.1.2.1, C3.1.2.2, C5.2.1, C5.2.2
L_u	Limit of unbraced length by which lateral-torsional buckling will not be considered	C3.1.2.2
M_{max}, M_A, M_B, M_C	Absolute value of moments in an unbraced segment, used for determining C_b	C3.1.2.1
M_m	Mean value of the material factor	C3.1.5, F1.1
M_n	Nominal flexural strength [resistance]	B2.1, C3.1, C3.1.1, C3.1.2.1, C3.1.2.2, C3.1.3, C3.1.4, C3.3.1, C3.3.2, C6.1

SYMBOLS AND DEFINITIONS

Symbol	Definition	Section
M	Required allowable flexural strength, ASD	C3.3.1, C3.5.1
M_{nx}, M_{ny}	Nominal flexural strengths [resistances] about the centroidal axes determined in accordance with Section C3	C5.1.1, C5.1.2, C5.2.1 C5.2.2, D4.3
M_{nxo}, M_{nyo}	Nominal flexural strengths [resistance] about the centroidal axes determined in accordance with Section C3.1 excluding the provisions of Section C3.1.2	C3.3.1, C3.3.2, C3.5.1, C3.5.2, D4.2, D4.3
M_{no}	Nominal yield moment for nested Z-sections	C3.5.1, C3.5.2
M_{nxt}, M_{nyt}	Nominal flexural strengths [resistances] about the centroidal axes determined using the gross, unreduced cross-section properties	C5.1.1, C5.1.2
M_f	Factored moment	C3.3.2
M_{fx}, M_{fy}	Moments due to factored loads with respect to the centroidal axes	C4, C5.1.2, C5.2.2
M_x, M_y	Required allowable flexural strength with respect to the centroidal axes, for ASD	C4, C5.1.1, C5.2.1
M_u	Required flexural strength, for LRFD	C3.3.2, C3.5.2
M_{ux}, M_{uy}	Required flexural strength with respect to centroidal axes, for LRFD	C4, C5.1.2, C5.2.2
M_*	Required flexural strength [factored moment]	C3.3.2, C3.5.2
M_{*x}, M_{*y}	Required flexural strengths [factored moments]	C4, C5.1.2
M_y	Moment causing a maximum strain e_y	B2.1, C3.1.2
M_1	Smaller end moment	C3.1.2.1, C5.2.1, C5.2.2
M_2	Larger end moment	C3.1.2.1, C5.2.1, C5.2.2
m	Degrees of freedom	F1.1
m	Term for determining the tensile yield point of corners	A7.2

SYMBOLS AND DEFINITIONS

Symbol	Definition	Section
m	Distance from the shear center of one C-section to the mid-plane of its web	D1.1, D3.2.2
N	Actual length of bearing	C3.4.1, C3.4.2, C3.5.1, C3.5.2
N	Number of stress range fluctuations in design life	G3
n	Coefficient	B4.1, B4.2
n	Number of stiffeners	B5.1, B5.1.1, B5.1.2
n	Number of holes	E5.1
n	Number of tests	F1.1
n	Number of anchors in the test assembly with same tributary area (for anchor failure), or number of panels with identical spans and loading to the failed span (for non-anchor failure)	C3.1.5
n	Number of threads per inch	G4
n_b	Number of bolt holes	E3.2
n_p	Number of parallel purlin lines	D3.2.1
P	Required allowable strength for the concentrated load Reaction in the presence of bending moment, for ASD	C3.5.1
P	Required allowable strength (nominal force) transmitted By weld, for ASD	E2.2.1
P	Required allowable compressive axial strength, for ASD	A2.3.1, C5.2.1
P	Professional factor	F1.1
P	Pitch (mm per thread)	G4
P_{Ex}, P_{Ey}	Elastic buckling strengths [resistances]	C5.2.1, C5.2.2
P_f	Axial force due to factored loads	A2.3.1, C5.2.2
P_f	Concentrated load or reaction due to factored loads	C3.5.2
P_f	Factored shear force transmitted by welding	E2.2.1
P_L	Force to be resisted by intermediate beam brace	D3.2.1, D3.2.2
P_m	Mean value of the tested-to-predicted load ratios	F1.1
P_n	Nominal web crippling strength [resistance] of member	C3.4.1, C3.5.1, C3.5.2

SYMBOLS AND DEFINITIONS

Symbol	Definition	Section
P_n	Nominal axial strength [resistance] of member	A2.3.1, C4, C4.6, C5.2.1, C5.2.2, C6.2, D4.1, D4.3
P_n	Nominal axial strength [resistance] of transverse stiffener	C3.6.1
P_n	Nominal strength [resistance] of connection component	E2.1, E2.2.1, E2.2.2, E2.3, E2.4, E2.5, E2.6, E3.1, E3.2, E3.4
P_n	Nominal bearing strength [resistance]	E3.3.1, E3.3.2
P_n	Nominal tensile strength of welded member	E2.7
P_{no}	Nominal axial strength [resistance] of member determined in accordance with Section C4 with $F_n = F_y$	C5.2.1, C5.2.2
P_{not}	Nominal pull-out strength [resistance] per screw	E4, E4.4.1, E4.4.3
P_{nov}	Nominal pull-over strength [resistance] per screw	E4, E4.4.2, E4.4.3
P_{ns}	Nominal shear strength [resistance] per screw	E4, E4.2, E4.3.1, E4.3.2, E4.3.3
P_{nt}	Nominal tension strength [resistance] per screw	E4, E4.4.3
P_s	Concentrated load or reaction	D1.1
P_{ss}	Nominal shear strength [resistance] of screw as reported by manufacturer	E4, E4,3,3
P_{ts}	Nominal tension strength [resistance] of screws as reported by manufacturer or determined by independent laboratory testing	E4, E4.4.3
P_u	Required axial strength [resistance], for LRFD	A2.3.1, C5.2.2
P_u	Factored force (required strength) transmitted by weld, For LRFD	E2.2.1
P_u	Required strength for the concentrated load or reaction in the presence of bending moment, for LRFD	C3.5.2
P_*	Required strength for concentrated load or reaction [concentrated load or reaction due to factored loads] in the presence of bending moment.	C3.5.2
P_*	Required compressive axial strength [factored compressive force]	C5.2.2
$\overline{Q}$	Design shear rigidity for sheathing	D4.1
$\overline{Q}_a$	$\overline{Q}$ / A	D4.1

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Symbol	Definition	Section
$\overline{Q}_t$	$(\overline{Q}\, d^2)/(4Ar_o^2)$	D4.1
$\overline{Q}_o$	Sheathing parameter	D4.1
Q_i	Load effect	F1.1
q	Design load in the plane of the web	D1.1
q_s	Reduction factor	C3.2.2
R	Required allowable strength, for ASD	A4.1.1
R	Modification factor	B5.1
R	Reduction factor	C3.1.3, C3.1.4
R	Coefficient	C6.2
R	Inside bend radius	A7.2, C3.4.1, C3.5.1, C3.5.2
R	Radius of outside bend surface	E2.5
R_I	I_s/I_a	B4.1, B4.2
R_a	Allowable design strength	F1.2
R_b	Reduction factor	A2.3.2
R_c	Reduction factor	C3.4.2
R_f	Effect of factored loads	A6.1.1
R_n	Nominal strength [resistance]	A1.2, A4.1.1, A5.1.1, A6.1.1, F2
R_n	Nominal blockshear rupture strength [resistance]	E5.3
R_n	Average value of all test results	F1.1, F1.2
R_u	Required strength, for LRFD	A5.1.1
r	Correction factor	C3.1.3
r	Least radius of gyration of full unreduced cross section	C4.1, C4.2, C4.5
r_i	Minimum radius of gyration of full unreduced cross section	C4.5
r_o	Polar radius of gyration of cross section about the shear center	C3.1.2.1, C4.2, D4.1
r_x, r_y	Radius of gyration of cross section about centroidal principal axis	C3.1.2.1, D4.1
S	$1.28\sqrt{E/f}$	B4, B4.1, B4.2, B5.2

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Symbol	Definition	Section
S_c	Elastic section modulus of the effective section calculated at a stress F_c in the extreme compression fiber	B2.1, C3.1.2.1
S_e	Elastic section modulus of the effective section calculated with extreme compression or tension fiber at F_y	C3.1.1, C3.1.3, C3.1.4
S_f	Elastic section modulus of full, unreduced section for extreme compression fiber	B2.1, C3.1.2.1, C3.1.2.2, C6.1
S_{ft}	Section modulus of the full section for the extreme tension the fiber	C5.1.1, C5.1.2
S_n	In-plane diaphragm nominal shear strength [resistance]	D5
s	Fastener spacing	D4.1
s	Spacing in line of stress of welds, rivets, or bolts connecting a compression coverplate or sheet to a non-integral stiffener or other element	D1.2
s	Sheet width divided by the number of bolts holes in the cross section being analyzed	E3.2
s	Weld spacing	D1.1
s′	Longitudinal center-to-center spacing of any consecutive holes	E3.2
s′	Fastener spacing for which $\overline{Q}_o$ is tabulated	D4.1
s_{max}	Maximum permissible longitudinal spacing of welds or other connectors joining two C-sections to form an I-section	D1.1
T	Required allowable tensile axial strength, for ASD	C5.1.1
T	Load due to contraction or expansion caused by temperature changes	A3.1, A3.1, A6.1.2, A6.1.2.2
T_f	Tension due to factored loads	C5.1.2
T_n	Nominal tensile strength [resistance]	C2, C5.1.1, C5.1.2
T_s	Design strength [factored resistance] of connection in tension	D1.1
T_u	Required tensile axial strength, for LRFD	C5.1.2

SYMBOLS AND DEFINITIONS

Symbol	Definition	Section
T_*	Required tensile axial strength [factored tension] with respect to the centroid	C5.1.2
t	Base steel thickness of any element or section	A1.2, A2.3.2, A2.4, A7.2, B1.1, B1.2, B2.1, B2.2, B2.4, B4, B4.1, B4.2, B5.1, B5.1.1, B5.1.2, B5.2, C3.1.1, C3.2.1, C3.2.2, C3.4.1, C3.4.2, C3.5.1, C3.5.2, C3.6.1, C3.6.2, C4.6, C6, C6.1, C6.2, D1.2, D3.2.1, D4.1, E3.3.1, E3.3.2, E4.3.2, D4,
t	Thickness of coped web	E5.1
t	Total thickness of the two welded sheets	E2.2.1, E2.2.2, E2.3
t	Thickness of thinnest connected part	E2.4, E2.5, E2.6, E3.1, E3.2, E3.3.2
t_1, t_2	Based thicknesses connected with fillet weld	E2.4
t_1	Thickness of member in contact with the screw head	E4, E4.3.1, E4.4.2
t_2	Thickness of member not in contact with the screw head	E4, E4.3.1,
t_c	Lesser of the depth of the penetration and t_2	E4, E4.4.1
t_e	Effective throat dimension for groove weld	E2.1
t_i	Thickness of uncompressed glass fiber blanket insulation	C3.1.3
t_s	Thickness of stiffener	C3.6.1
t_w	Effective throat of weld	E2.4, E2.5
U	Reduction coefficient	E2.7, E3.2
V	Required allowable shear strength, for ASD	C3.3.1
V_F	Coefficient of variation of the fabrication factor	C3.1.5, F1.1
V_M	Coefficient of variation of the material factor	C3.1.5, F1.1
V_f	Shear force due to factored loads, LSD	C3.3.2
V_n	Nominal shear strength [resistance]	C3.2.1, C3.3.1, C3.3.2, C3.6.2, E5.1
V_P	Coefficient of variation of the tested-to-predicted load ratios	C3.1.5, F1.1
V_Q	Coefficient of variation of the load effect	C3.1.5, F1.1
V_u	Required shear strength, for LRFD	C3.3.2

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Symbol	Definition	Section
V_*	Required shear strength [factored shear]	C3.3.2
W	Design load supported by all purlin lines being restrained	D3.2.1
W	Live load due to wind	A3.1, A6.1.2, A6.1.2.2
w	Flat width of element exclusive of radii	A2.3.2, B1.1, B2.1, B2.2, B3.1, B4, B4.1, B4.2, C3.1.1, C3.6.1, D1.2
w	Flat width of the beam flange which contacts the bearing plate	C3.5.1, C3.5.2
w	Flat width of the narrowest unstiffened compression element tributary to the connections	D1.2
w_f	Width of flange projection beyond the web or half the distance between webs for box- or U-type sections	B1.1
w_1	Leg on weld	E2.4, E2.5
w_2	Leg on weld	E2.4, E2.5
x	Distance from concentrated load to brace	D3.2.2
x	Non-dimensional fastener location	C4.6
x	Nearest distance between web hole and edge of bearing	C3.4.2
x_o	Distance from shear center to centroid along the principal x-axis	C3.1.2.1, C4.2, D4.1
$\bar{x}$	Distance from shear plane to centroid of the cross section	E2.7, E3.2
Y	Yield point of web steel divided by yield point of stiffener steel	C3.6.2
α	Coefficient for purlin directions	D3.2.1
α	Modification factor for type of bearing connection	E3.3.1
α	Coefficient for conversion of units	C4.6, E3.3.2, G3
α	Load factor	A1.2a
α_D	Dead load factor	A6.1.2, A6.1.2.1
α_E	Load factor of live load due to earthquake	A6.1.2, A6.1.2.1
α_L	Live load factor	A6.1.2, A6.1.2.1
α_T	Load factor due to contraction or expansion caused by by temperature changes	A6.1.2, A6.1.2.1

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Symbol	Definition	Section
α_W	Wind load factor	A6.1.2, A.6.1.2.1
$1/\alpha_x$, $1/\alpha_y$	Magnification factors	C5.2.1, C5.2.2
β	Coefficient	B5.1.1, B5.1.2, C4.2, D4.1
β_o	Target reliability index	C3.1.5, F1.1
δ, δ_i, γ, γ_i, ω, ω_i	Coefficients	B5.1.1, B5.1.2
γ	Actual shear strain in the sheathing	D4.1
$\bar{\gamma}$	Permissible shear strain of the sheathing	D4.1
γ	Importance factor	A1.2a, A6.1.2, A6.1.2.3
γ_i	Load factor	F1.1
θ	Angle between web and bearing surface >45° but no more than 90°	C3.4.1
θ	Angle between the vertical and the plane of the web of the Z-section, degrees	D3.2.1
θ	Angle between an element and its edge stiffener	B4, B4.2
λ, λ_c	Slenderness factors	B2.1, B2.2, B5.1, C3.5.1, C3.5.2, C4, C6.2
λ_1, λ_2	Parameters used in determining compression strain factor	C3.1.1
μ	Poisson's ratio for steel = 0.30	B2.1, C3.2.1
ρ	Reduction factor	A7.2, B2.1, B5.1, F3.1
σ_{CR}	Theoretical elastic buckling stress	D4.1
σ_{ex}	$(\pi^2E)/(K_xL_x/r_x)^2$	C3.1.2.1, C4.2
	$(\pi^2E)/(L/r_x)^2$	D4.1
σ_{exy}	$(\pi^2EI_{xy})/(AL^2)$	D4.1

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Symbol	Definition	Section
σ_{ey}	$(\pi^2 E)/(K_y L_y / r_y)^2$	C3.1.2.1
	$(\pi^2 E)/(L / r_y)^2$	D4.1
σ_{tQ}	$\sigma_t + Q_t$	D4.1
σ_t	Torsional buckling stress	C3.1.2.1, C4.2, C4.3, D4.1
ϕ	Resistance factor	A1.2, A5.1.1, A6.1.1, C3.1.5, C3.5.2, , C4.6, E2.1, E2.2.1, E2.2.2, E2.3, E2.4, E2.5, E2.6, E2.7, E3.1, E3.2, E3.3.1, E3.3.2, E3.4, E4, E4.3.2, E4.4, E4.4.3, E5.1, F1.1, F1.2
ϕ_b	Resistance factor for bending strength	C3.1.1, C3.1.2, C3.1.3, C3.1.4, C3.3.2, C3.5.2, C5.1.2, C5.2.2, C6.1, D4.2
ϕ_c	Resistance factor for concentrically loaded compression member	A2.3.1, C3.6.1, C4, C5.2.2, C6.2, D4.1
ϕ_d	Resistance factor for diaphragms	D5
ϕ_t	Resistance factor for tension member	C2, C5.1.2
ϕ_v	Resistance factor for shear strength	C3.2.1, C3.3.2
ϕ_w	Resistance factor for web crippling strength	C3.4.1, C3.5.2
ψ	$\lvert f_2 / f_1 \rvert$	B2.3
ψ	Load combination factor	A6.1.2.3
Ω	Factor of safety	A1.2, A4.1.1, C3.1.5, C3.5.1, C4.6, E2.1, E2.2.1, E2.2.2, E2.3, E2.4, E2.5, E2.6, E2.7 E3.1, E3.2, E3.3.1, E3.3.2, E3.4, E4, E4.3.2, E4.4, E4.4.3, E5.1, F.1.2
Ω_b	Factor of safety for bending strength	C3.1.1, C3.1.2, C3.1.3, C3.1.4, C3.3.1, C3.5.1, C5.1.1, C5.2.1, C6.1, D4.2

SYMBOLS AND DEFINITIONS

Symbol	Definition	Section
Ω_c	Factor of safety for concentrically loaded compression member	A2.3.1, C4, C5.2.1, C6.2, D4.1
Ω_c	Factor of safety for bearing strength	C3.6.1
Ω_d	Factor of safety for diaphragms	D5
Ω_t	Factor of safety for tension member	C2, C5.1.1
Ω_v	Factor of safety for shear strength	C3.2.1, C3.3.1
Ω_w	Factor of safety for web crippling strength	C3.4.1, C3.5.1

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

A. GENERAL PROVISIONS

A1 Limits of Applicability and Terms

A1.1 Scope and Limits of Applicability

This *Specification* shall apply to the design of structural members cold-formed to shape from carbon or low-alloy steel sheet, strip, plate or bar not more than one in. (25.4 mm) in thickness and used for load-carrying purposes in buildings. It shall be permitted to be used for structures other than buildings provided appropriate allowances are made for dynamic effects.

This *Specification* includes Symbols and Definitions, Chapters A through G, and Appendices A through C which shall apply as follows:

- Appendix A shall apply only in the United States,
- Appendix B shall apply only in Canada, and
- Appendix C shall apply only in Mexico

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:

- The use of ASD and LRFD shall be limited to the United States and Mexico, and
- The use of LSD shall be limited to Canada

The nominal strength [nominal resistance]***** and stiffness of cold-formed steel elements, members, assemblies, connections, and details shall be determined in accordance with the provisions in Chapters B through G of the *Specification* and Appendices A through C. Where the composition or configuration of such components is such that calculation of strength [resistance] and/or stiffness cannot be made in accordance with those provisions, structural performance shall be established from either of the following:

(a) Determine design strength [factored resistance] or stiffness by tests, undertaken and evaluated in accordance with Chapter F.

(b) Determine design strength [factored resistance] or stiffness by rational engineering analysis based on appropriate theory, related testing if data is available, and engineering judgment. Specifically, the design strength [factored resistance] shall be determined from the calculated nominal strength [resistance] by applying the following factors of safety or resistance factors:

Members		
USA and Mexico		Canada
Ω (ASD)	ϕ (LRFD)	ϕ(LSD)
2.00	0.80	0.75

Connections		
USA and Mexico		Canada
Ω (ASD)	ϕ (LRFD)	ϕ(LSD)
2.50	0.65	0.60

Note:
*Bracketed terms are equivalent terms that apply particularly to LSD.

A1.2 Terms

Where the following terms appear in this *Specification* they shall have the meaning herein indicated:

General Terms

Cold-Formed Steel Structural Members. Shapes 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.

Confirmatory Test. Test made, when desired, on members, connections, and assemblies designed according to the provisions of Chapters A through G of this Specification or its specific references, in order to compare actual versus calculated performance.

Cross-Sectional Area:

Effective Area. Effective area, A_e, calculated using the effective widths of component elements in accordance with Chapter B. It can be a gross area or a net area, as applicable, if the effective widths of all component elements, determined in accordance with Chapter B, are equal to the actual flat widths.

Full, Unreduced Area. Full, unreduced area, A, calculated without reducing the widths of component element to their effective widths. It can be an unreduced gross area or an unreduced net area, as applicable.

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

Net Area. Net area, A_n, equal to gross area less the area of holes, openings, and cutouts.

Distortional Buckling. Buckling mode in which the angle between elements of the cross section does not stay constant.

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

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

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

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.

Girt. Horizontal structural member which supports wall panel and is subjected to principally bending under applied loads.

Local Buckling. Buckling of elements only within a section, where the line junctions between elements remain straight and angles between elements do not change.

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

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.

Performance Test. Test made on structural members, connections, and assemblies whose performance cannot be determined by the provisions of Chapters A through G of this *Specification* or its specific references.

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

Purlin. Horizontal structural member which supports roof deck and is subjected to principally bending under applied loads.

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

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

Specified Minimum Yield Point. Lower limit of yield point in a test specified to qualify a lot of steel for use in a cold-formed steel structural member designed at that yield point.

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.

SS. ASTM designation for certain sheet steels intended for structural applications.

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

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: Maximum stress reached in a tension test.

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

Torsional-Flexural Buckling. Buckling mode in which compression members bend and twist simultaneously without change in cross sectional shape.

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

Unsymmetric Section. Section not symmetric either about an axis or a point.

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 point, tensile strength, and elongation.

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.

Yield Point. Yield point, F_y or F_{sy}, as used in this *Specification* shall mean yield point or yield strength.

ASD and LRFD Terms (USA and Mexico):

ASD (Allowable Stress Design, herein referred as Allowable Strength Design). A method of proportioning structural components (members, connectors, connecting elements and assemblages) such that the allowable stress, allowable force or allowable moment is not exceeded by the required allowable strength of the component determined by the load effects of all appropriate combinations of nominal loads.

Allowable Design Strength. Allowable strength, R_n/Ω, (force, moment, as appropriate), provided by the structural component.

Design Strength. Factored resistance, ϕR_n (force, moment, as appropriate), provided by the structural component.

LRFD (Load and Resistance Factor Design). 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 combinations of factored loads.

Nominal loads. The magnitudes of the loads specified by the applicable code not including load factors.

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

Required Allowable Strength. Load effect (force, moment, as appropriate) acting on the structural component determined by structural analysis from the nominal loads for ASD (using all appropriate load combinations).

Required Strength. Load effect (force, moment, as appropriate) acting on the structural component determined by structural analysis from the factored loads for LRFD or nominal loads for ASD (using all appropriate load combinations).

Resistance. See the definition of Nominal Strength.

Resistance Factor. A factor that accounts for unavoidable deviations of the actual strength from the nominal value and the manner and consequences of failure.

LSD Terms (Canada):

Limit States. Those conditions in which a structural member ceases to fulfill the function for which it was designed. Those states concerning safety are called the ultimate limit states. The ultimate limit state for strength is the maximum load-carrying capacity. Limit states that restrict the intended use of a member for reasons other than safety, such as deflection and vibration, are called serviceability limit states.

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.

Factored Load. Product of a specified load and appropriate load factor.

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

Nominal 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.

Resistance Factor. A factor that accounts for unavoidable deviations of the actual strength from the nominal value and the manner and consequences of failure.

Specified loads. The magnitudes of the loads specified by the applicable code not including load factors.

A1.3 Units of Symbols and Terms

The *Specification* is written so that any compatible system of units may be used except where explicitly stated otherwise in the text of these provisions. The unit systems considered in those sections are 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 Material

A2.1 Applicable Steels

This *Specification* requires the use of steels intended for structural applications as defined in general by the specifications of the American Society for Testing and Materials listed below. Such steels are identified in many ASTM specifications for sheet material as SS. Other steels for structural applications that are applicable to specific countries as listed in Section A2.1a of Appendix A, B or C shall also be permitted.

ASTM A36/A36M, Carbon Structural Steel

ASTM A242/A242M, High-Strength Low-Alloy Structural Steel

ASTM A283/A283M, Low and Intermediate Tensile Strength Carbon Steel Plates

ASTM A500, Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes

ASTM A529/A529M, High-Strength Carbon-Manganese Steel of Structural Quality

ASTM A572/A572M, High-Strength Low-Alloy Columbium-Vanadium Structural Steel

ASTM A588/A588M, High-Strength Low-Alloy Structural Steel with 50 ksi (345 MPa) Minimum Yield Point to 4 in. (100 mm) Thick

ASTM A606, Steel, Sheet and Strip, High Strength, Low Alloy, Hot-Rolled and Cold-Rolled, with Improved Atmospheric Corrosion Resistance

ASTM A653/A653M (SS Grades 33 (230), 37 (255), 40 (275), and 50 (340) Class 1 and Class 3; HSLAS Types A and B, Grades 40 (275), 50 (340), 60 (410), 70 (480) and 80 (550)), Steel Sheet, Zinc-Coated (Galvanized) or Zinc-Iron Alloy-Coated (Galvannealed) by the Hot-Dip Process

ASTM A792/A792M (Grades 33 (230), 37 (255), 40 (275), and 50 Class 1 (340 Class 1)), Steel Sheet, 55% Aluminum-Zinc Alloy-Coated by the Hot-Dip Process

ASTM A847 Cold-Formed Welded and Seamless High Strength, Low Alloy Structural Tubing with Improved Atmospheric Corrosion Resistance

ASTM A875/A875M (SS Grades 33 (230), 37 (255), 40 (275), and 50 (340) Class 1 and Class 3; HSLAS Types A and B, Grades 50 (340), 60 (410), 70 (480), and 80 (550)), Steel Sheet, Zinc-5% Aluminum Alloy-Coated by the Hot-Dip Process

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

ASTM A1008/A1008M (SS Grades 25 (170), 30 (205), 33 (230) Types 1 and 2, and 40 (275) Types 1 and 2; HSLAS Classes 1 and 2, Grades 45 (310), 50 (340), 55 (380), 60 (450), 65 (450), and 70 (480); HSLAS-F Grades 50 (340), 60 (410), 70 (480), and 80 (550)), Steel, Sheet, Cold-Rolled, Carbon, Structural, High-Strength Low-Alloy and High-Strength Low-Alloy with Improved Formability

ASTM A1011/A1011M (SS Grades 30 (205), 33 (230), 36 (250) Types 1 and 2, 40 (275), 45 (310), 50 (340), and 55 (380); HSLAS Classes 1 and 2, Grades 45 (310), 50 (340), 55 (380), 60 (410), 65 (450), and 70 (480); HSLAS-F Grades 50 (340), 60 (410), 70 (480), and 80(550)), Steel, Sheet and Strip, Hot-Rolled, Carbon, Structural, High-Strength Low-Alloy and High-Strength Low-Alloy with Improved Formability

A2.2 Other Steels

The listing in Section A2.1 does not exclude the use of steel up to and including one in. (25.4 mm) in thickness ordered or produced to other than the listed specifications provided such steel conforms to the chemical and mechanical requirements of one of the listed specifications or other published specification which establishes its properties and suitability, and provided it is subjected by either the producer or the purchaser to analyses, tests and other controls to the extent and in the manner prescribed by one of the listed specifications and Section A2.3.

A2.3 Ductility

Steels not listed in Section A2.1 and used for structural members and connections in accordance with Section A2.2 shall comply with one of the following ductility requirements:

A2.3.1 The ratio of tensile strength to yield point shall not be less than 1.08, and the total elongation shall not be less than 10 percent for a two-inch gage (50 mm) length or 7 percent for an eight-inch (200 mm) gage length standard specimen tested in accordance with ASTM A370. If these requirements cannot be met, the following criteria shall be satisfied: (1) local elongation in a 1/2 in. (12.7 mm) gage length across the fracture shall not be less than 20%, (2) uniform elongation outside the fracture shall not be less than 3%. When material ductility is determined on the basis of the local and uniform elongation criteria, the use of such material is restricted to the design of purlins and girts in accordance with Sections C3.1.1(a), C3.1.2, C3.1.3, and C3.1.4. For purlins and girts subject to combined axial load and bending moment (Section C5), $\frac{\Omega_c P}{P_n}$ 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.

A2.3.2 Steels conforming to ASTM A653/A653M SS Grade 80 (550), A1008/A1008M SS Grade 80 (550), A792/A792M Grade 80 (550), A875/A875M SS Grade 80 (550) and other steels which do not meet the provisions of Section A2.3.1 shall be permitted for multiple-web configurations such as roofing, siding and floor decking provided that:

(1) the yield point, F_y, used for determining nominal strength [resistance] in Chapters B, C, and D is taken as 75 percent of the specified minimum yield point or 60 ksi (414 MPa or 4220 kg/cm^2), whichever is less, and

(2) the tensile strength, F_u, used for determining nominal strength [resistance] in Chapter E is taken as 75 percent of the specified minimum tensile strength or 62 ksi (427 MPa or 4360 kg/cm^2), whichever is less.

Alternatively, the suitability of such steels for any multi-web configuration shall be demonstrated by load tests according to the provisions of Section F1. Design strengths [factored resistances] based on these tests shall not exceed the design strengths [factored resistances] calculated according to Chapters B through G, using the specified minimum yield point, F_y, and the specified minimum tensile strength, F_u.

Exception: For multiple-web configurations, a reduced yield point, R_bF_y, shall be permitted for determining the nominal flexural strength [moment resistance] in Section C3.1.1(a), for which the reduction factor, R_b, shall be determined as follows:

(a) Stiffened and Partially Stiffened Compression Flanges

For $w/t \leq E/F_y$

$R_b = 1.0$

For $0.067E/F_y < w/t < 0.974E/F_y$

$$R_b = 1-0.26[wF_y/(tE) - 0.067]^{0.4} \quad (Eq. \text{ A2.3.2-1})$$

For $0.974E/F_y \leq w/t \leq 500$

$R_b = 0.75$

(b) Unstiffened Compression Flanges

For $w/t \le 0.0173E/F_y$

$R_b = 1.0$

For $0.0173E/F_y < w/t \le 60$

$$R_b = 1.079 - 0.6\sqrt{wF_y/(tE)} \qquad (Eq.\ A2.3.2\text{-}2)$$

where

E = Modulus of elasticity

F_y = Yield point as specified in Section A7.1 ≤ 80 ksi (550 MPa, or 5620 kg/cm^2)

t = Thickness of section

w = Flat width of compression flange

The above Exception Clause does not apply to the use of steel deck for composite slabs, for which the steel deck acts as the tensile reinforcement of the slab.

A2.4 Delivered Minimum Thickness

The uncoated minimum steel thickness of the cold-formed 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 thicknesses shall be permitted at bends, such as corners, due to cold-forming effects.

A3 Loads

Loads and load combinations shall be as stipulated by the applicable country specific provisions, Section A3 of Appendix A, B, or C.

A4 Allowable Strength Design

A4.1 Design Basis

Design under this Section of the *Specification* shall be based on Allowable Strength Design (ASD) principles. All provisions of this *Specification*, except for those in Sections A5, A6 and in Chapters C and F designated for LRFD and LSD, shall apply.

A4.1.1 ASD Requirements

A design satisfies the requirements of this *Specification* when the allowable strength of each structural component equals or exceeds the required allowable strength, determined on the basis of the nominal loads, for all applicable load combinations.

The design shall be performed in accordance with Equation (A4.1.1-1):

$$R \le R_n/\Omega \qquad (Eq.\ A4.1.1\text{-}1)$$

where

R = Required allowable strength

R_n = Nominal strength specified in Chapters B through G

Ω = Factor of safety specified in Chapters B through G

R_n / Ω = Allowable design strength

A4.1.2 Load Combinations for ASD

Load combinations for ASD shall be as stipulated by Section A4.1.2 of Appendix A or C.

A5 Load and Resistance Factor Design

A5.1 Design Basis

Design under this Section of the *Specification* shall be based on Load and Resistance Factor Design (LRFD) principles. All provisions of this *Specification*, except for those in Sections A4, A6 and in Chapters C and F designated for ASD and LSD, shall apply.

A5.1.1 LRFD Requirements

A design satisfies the requirements of this *Specification* when the design strength of each structural component equals or exceeds the required strength determined on the basis of the nominal loads, multiplied by the appropriate load factors, for all applicable load combinations.

The design shall be performed in accordance with Equation (A5.1.1-1):

$$R_u \le \phi R_n \qquad (Eq.\ A5.1.1\text{-}1)$$

where

R_u = Required strength

R_n = Nominal strength specified in Chapters B through G

ϕ = Resistance factor specified in Chapters B through G

ϕR_n = Design strength

A5.1.2 Load Factors and Load Combinations for LRFD

Load factors and load combinations for LRFD shall be as stipulated by Section A5.1.2 of Appendix A or C.

A6 Limit States Design

A6.1 Design Basis

Design under this Section of the *Specification* shall be based on Limit States Design (LSD) principles. All provisions of this *Specification*, except for those in Sections A4, A5 and Chapters C and F designated for ASD and LRFD, shall apply.

A6.1.1 LSD Requirements

Structural members and their connections shall be designed to have resistance such that the factored resistance equals or exceeds the effect of factored loads. The design shall be performed in accordance with Equation (A6.1.1-1):

$$\phi R_n \geq R_f \qquad (Eq.\ A6.1.1\text{-}1)$$

where

R_f = Effect of factored loads

R_n = Nominal resistance specified in Chapters B through G

ϕ = Resistance factor specified in Chapters B through G

ϕR_n = Factored resistance

A6.1.2 Load Factors and Load Combinations for LSD

Load factors and load combinations for LSD shall be as stipulated by Section A6.1.2 of Appendix B.

A7 Yield Point and Strength Increase from Cold Work of Forming

A7.1 Yield Point

The yield point used in design, F_y, shall not exceed the specified minimum yield point of steels as listed in Section A2.1 or A2.3.2, as established in accordance with Chapter F, or as increased for cold work of forming in Section A7.2.

A7.2 Strength Increase from Cold Work of Forming

Strength increase from cold work of forming shall be permitted by substituting F_{ya} for F_y, where F_{ya} is the average yield point of the full section. Such increase shall be limited to Sections C2, C3.1 (excluding Section C3.1.1(b)), C4, C5, C6 and D4. The limitations and methods for determining F_{ya} are as follows:

(a) For axially loaded compression members and flexural members whose proportions are such that the quantity ρ for strength determination is unity as calculated according to Section B2 for each of the component elements of the section, 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 F3.1]

(2) stub column tests [see paragraph (b) of Section F3.1]

(3) computed as follows:

$$F_{ya} = CF_{yc} + (1 - C)\,F_{yf} \qquad (Eq.\ A7.2\text{-}1)$$

where

F_{ya} = Average yield point of the steel in the full section of compression members or full flange sections of flexural members

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

F_{yf} = Weighted average tensile yield point of the flat portions established in accordance with Section F3.2 or virgin steel yield point if tests are not made

F_{yc} = $B_cF_{yv}/(R/t)^m$, tensile yield point of corners. This equation is applicable only when $F_{uv}/F_{yv} \geq 1.2$, $R/t \leq 7$, and the included angle $\leq 120^o$ (*Eq.* A7.2-2)

B_c = $3.69\,(F_{uv}/F_{yv}) - 0.819\,(F_{uv}/F_{yv})^2 - 1.79$ (*Eq.* A7.2-3)

m = $0.192\,(F_{uv}/F_{yv}) - 0.068$ (*Eq.* A7.2-4)

R = Inside bend radius

F_{yv} = Tensile yield point of virgin steel specified by Section A2 or established in accordance with Section F3.3

F_{uv} = Tensile strength of virgin steel specified by Section A2 or established in accordance with Section F3.3

(b) For axially loaded tension members the yield point 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.

A8 Serviceability

A structure shall be designed to perform its required functions during its expected life. Serviceability limits shall be chosen based on the intended function of the structure, and shall be evaluated using realistic loads and load combinations.

A9 Referenced Documents

The following documents are referenced in this *Specification*. Refer to Section A9a of Appendix A, B, or C for documents applicable to the corresponding country.

1. American Society of Mechanical Engineers, ASME B46.1-85, “Surface Texture, Surface Roughness, Waviness, and Lay”, American Society of Mechanical Engineers, 1828 L Street, NW, Washington, DC 20036.
2. American Society for Testing and Materials (ASTM), 100 Barr Harbor Drive, West Conshohocken, Pennsylvania 19428-2959:

   ASTM A36/A36M-00a, Carbon Structural Steel

   ASTM A194/A194M-00b, Carbon and Alloy Steel Nuts for Bolts for High-Pressure and High-Temperature Service

   ASTM A242/A242M-00a, High-Strength Low-Alloy Structural Steel

   ASTM A283/A283M-00, Low and Intermediate Tensile Strength Carbon Steel Plates

   ASTM A307-00, Carbon Steel Bolts and Studs, 60,000 PSI Tensile Strength

   ASTM A325-00, Structural Bolts, Steel, Heat Treated, 120/105 ksi Minimum Tensile Strength

   ASTM A325M-00, High Strength Bolts for Structural Steel Joints [Metric]

   ASTM A354-00a, Quenched and Tempered Alloy Steel Bolts, Studs, and Other Externally Threaded Fasteners

   ASTM A370-97a, Standard Test Methods and Definitions for Mechanical Testing of Steel Products

   ASTM A449-00, Quenched and Tempered Steel Bolts and Studs

   ASTM A490-00, Heat-Treated Steel Structural Bolts, 150ksi Minimum Tensile Strength

   ASTM A490M-00, High Strength Steel Bolts, Classes 10.9 and 10.9.3, for Structural Steel Joints [Metric]

   ASTM A500-99, Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes

   ASTM A529/A529M-00, High-Strength Carbon-Manganese Steel of Structural Quality

   ASTM A563-00, Carbon and Alloy Steel Nuts

   ASTM A563M-00, Carbon and Alloy Steel Nuts [Metric]

   ASTM A572/A572M-00a, High-Strength Low-Alloy Columbium-Vanadium Structural Steel

   ASTM A588/A588M-00a, High-Strength Low-Alloy Structural Steel with 50 ksi [345 MPa] Minimum Yield Point to 4 in. [100 mm] Thick

   ASTM A606-98, Steel, Sheet and Strip, High-Strength, Low-Alloy, Hot-Rolled and Cold-Rolled, with Improved Atmospheric Corrosion Resistance

   ASTM A653/A653M-00, Steel Sheet, Zinc-Coated (Galvanized) or Zinc-Iron Alloy-Coated (Galvannealed) by the Hot-Dip Process

   ASTM A792/A792M-99, Steel Sheet, 55% Aluminum-Zinc Alloy-Coated by the Hot-Dip Process

   ASTM A847-99a, Cold-Formed Welded and Seamless High Strength, Low Alloy Structural Tubing with Improved Atmospheric Corrosion Resistance

   ASTM A875/A875M-99 Steel Sheet, Zinc-5% Aluminum Alloy-Coated by the hot-Dip Process

   ASTM A1003/A1003M-00, Steel Sheet, Carbon, Metallic- and

Nonmetallic-Coated for Cold-Formed Framing Members

ASTM A1008/A1008M-00 Steel, Sheet, Cold-Rolled, Carbon, Structural, High-Strength Low-Alloy and High-Strength Low-Alloy with Improved Formability

ASTM A1011/A1011M-00 Steel, Sheet and Strip, Hot-Rolled, Carbon, Structural, High-Strength Low-Alloy and High-Strength Low-Alloy with Improved Formability

ASTM F436-00, Hardened Steel Washers

ASTM F436M-00, Hardened Steel Washers [Metric]

ASTM F844-00, Washers, Steel, Plain (Flat), Unhardened for General Use

ASTM F959-99a, Compressible Washer-Type Direct Tension Indicators for Use with Structural Fasteners

ASTM F959M-99a, Compressible Washer-Type Direct Tension Indicators for Use with Structural Fasteners [Metric]

B. ELEMENTS

B1 Dimensional Limits and Considerations

B1.1 Flange Flat-Width-to-Thickness Considerations

(a) Maximum Flat-Width-to-Thickness Ratios

Maximum allowable overall flat-width-to-thickness ratios, w/t, disregarding intermediate stiffeners and taking as t the actual thickness of the element, shall be as follows:

(3) Stiffened compression element having *one* longitudinal edge connected to a web or flange element, the other stiffened by:

Simple lip	60
Any other kind of stiffener	
i) when $I_s < I_a$	60
ii) when $I_s \geq I_a$	90

(2) Stiffened compression element with *both* longitudinal edges connected to other stiffened elements — 500

(3) Unstiffened compression element — 60

It shall be noted that unstiffened compression elements that have w/t ratios exceeding approximately 30 and stiffened compression elements that have w/t ratios exceeding approximately 250 are likely to develop noticeable deformation at the full design strength, without affecting the ability of the member to develop the required strength.

Stiffened elements having w/t ratios larger than 500 can be used with adequate design strength [factored resistance] to sustain the required loads; however, substantial deformations of such elements usually will invalidate the design equations of this *Specification*.

(b) 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, the following equation applies to compression and tension flanges, either stiffened or unstiffened:

$$w_f = \sqrt{0.061tdE / f_{av}}\sqrt[4]{(100c_f / d)} \qquad (Eq.\ B1.1\text{-}1)$$

where

w_f = Width of flange projecting beyond the web; or half of the distance between webs for box- or U-type beams

t = Flange thickness

d = Depth of beam

c_f = Amount of curling displacement

f_{av} = Average stress in the 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) 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 it 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 to the following:

Table B1.1(c)
Short Span, Wide Flanges
Maximum Allowable Ratio of Effective Design Width to Actual Width

L / w_f	Ratio	L / w_f	Ratio
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 the web for I-beam and similar sections; or half the distance between webs of 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.

B1.2 Maximum Web Depth-to-Thickness Ratio

The ratio, h / t, of the webs of flexural members shall not exceed the following limitations:

(a) For unreinforced webs: $(h / t)_{max} = 200$

(b) For webs which are provided with transverse stiffeners satisfying the requirements of Section C3.6.1:

(1) When using bearing stiffeners only, $(h / t)_{max} = 260$

(2) When using bearing stiffeners and intermediate stiffeners,
$(h/t)_{max} = 300$

In the above,

h = Depth of flat portion of web measured along the plane of web
t = Web thickness
Where a web consists of two or more sheets, the h/t ratio shall be computed for the individual sheets.

B2 Effective Widths of Stiffened Elements

B2.1 Uniformly Compressed Stiffened Elements

(a) Strength Determination

The effective width, b, shall be determined from the following equations:

$b = w$ when $\lambda \leq 0.673$ (*Eq.* B2.1-1)

$b = \rho w$ when $\lambda > 0.673$ (*Eq.* B2.1-2)

where

w = Flat width as shown in Figure B2.1-1

$\rho = (1 - 0.22/\lambda)/\lambda$ (*Eq.* B2.1-3)

λ is a slenderness factor determined as follows:

$$\lambda = \sqrt{\frac{f}{F_{cr}}} \quad (Eq.\ B2.1\text{-}4)$$

$$F_{cr} = k\frac{\pi^2 E}{12(1-\mu^2)}\left(\frac{t}{w}\right)^2 \quad (Eq.\ B2.1\text{-}5)$$

where

t = Thickness of the uniformly compressed stiffened elements
μ = Poisson's ratio of steel, and
f is as follows:

For flexural members:

(l) If Procedure I of Section C3.1.1 is used:
When the initial yielding is in compression in the element considered, $f = F_y$.
When the initial yielding is in tension, the compressive stress, f, in the element considered shall be determined on the basis of the effective section at M_y (moment causing initial yield).

(2) If Procedure II of Section C3.1.1 is used, f is the stress in the element considered at M_n determined on the basis of the effective section.

(3) If Section C3.1.2.1 is used, f is the stress F_c as described in that Section in determining S_c.

For compression members, f is taken equal to F_n as determined in Section C4 or D4.1 as applicable.

E = Modulus of elasticity

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.

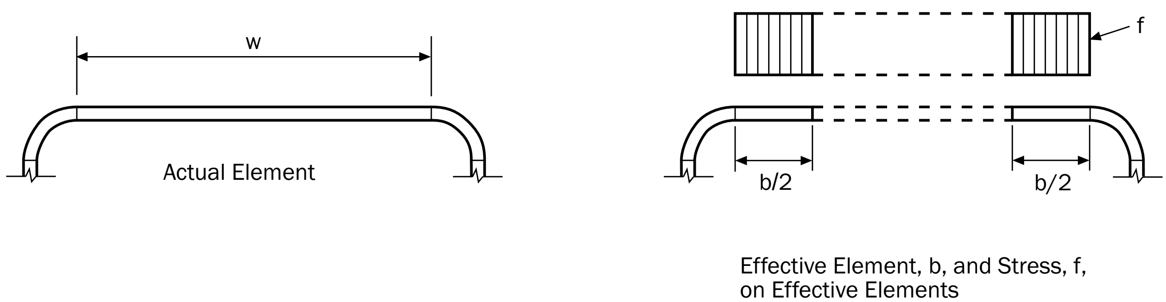


Figure B2.1-1 Stiffened Elements

(b) Serviceability Determination

The effective width, b_d, used in determining serviceability shall be calculated from the following equations:

$b_d = w$ when $\lambda \leq 0.673$ (*Eq*. B2.1-6)

$b_d = \rho w$ when $\lambda > 0.673$ (*Eq*. B2.1-7)

where

w = Flat width

ρ = Reduction factor determined by either of the following two procedures:

(1) Procedure I.

A low estimate of the effective width may be obtained from Eqs. B2.1-3 and B2.1-4 except that f_d is substituted 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 can be obtained by calculating ρ as follows:

$\rho = 1$ when $\lambda \leq 0.673$ (*Eq*. B2.1-8)

$\rho = (1.358 - 0.461/\lambda)/\lambda$ when $0.673 < \lambda < \lambda_c$ (*Eq*. B2.1-9)

$\rho = (0.41 + 0.59\sqrt{F_y / f_d} - 0.22/\lambda)/\lambda$ when $\lambda \geq \lambda_c$ (*Eq*. B2.1-10)

ρ shall not exceed 1.0 for all cases.

where

$\lambda_c = 0.256 + 0.328\,(w/t)\sqrt{F_y / E}$ (*Eq*. B2.1-11)

and λ is as defined by Eq. B2.1-4, except that f_d is substituted for f.

B2.2 Uniformly Compressed Stiffened Elements with Circular Holes

(a) Strength Determination

The effective width, b, shall be determined as follows:

for $0.50 \geq \frac{d_h}{w} \geq 0$, and $\frac{w}{t} \leq 70$ and

the distance between centers of holes $\geq 0.50w$ and $\geq 3d_h$,

$$b = w - d_h \quad \text{when } \lambda \leq 0.673 \qquad (Eq.\ B2.2\text{-}1)$$

$$b = \frac{w\left[1 - \frac{(0.22)}{\lambda} - \frac{(0.8d_h)}{w}\right]}{\lambda} \quad \text{when } \lambda > 0.673 \qquad (Eq.\ B2.2\text{-}2)$$

b shall not exceed $w - d_h$

where

w = Flat width

d_h = Diameter of holes

λ is as defined in Section B2.1.

(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 B2.1(b), except that f_d is substituted for f, where f_d is the computed compressive stress in the element being considered.

B2.3 Webs and other Stiffened Elements under Stress Gradient

The following notation is used in this section:

b_1 = Effective width, dimension defined in Figure B2.3-1

b_2 = Effective width, dimension defined in Figure B2.3-1

b_e = Effective width b determined in accordance with Section B2.1 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 B2.3-2

f_1, f_2 = Stresses shown in Figure B2.3-1 calculated on the basis of effective section. Where f_1 and f_2 are both compression, $f_1 \geq f_2$

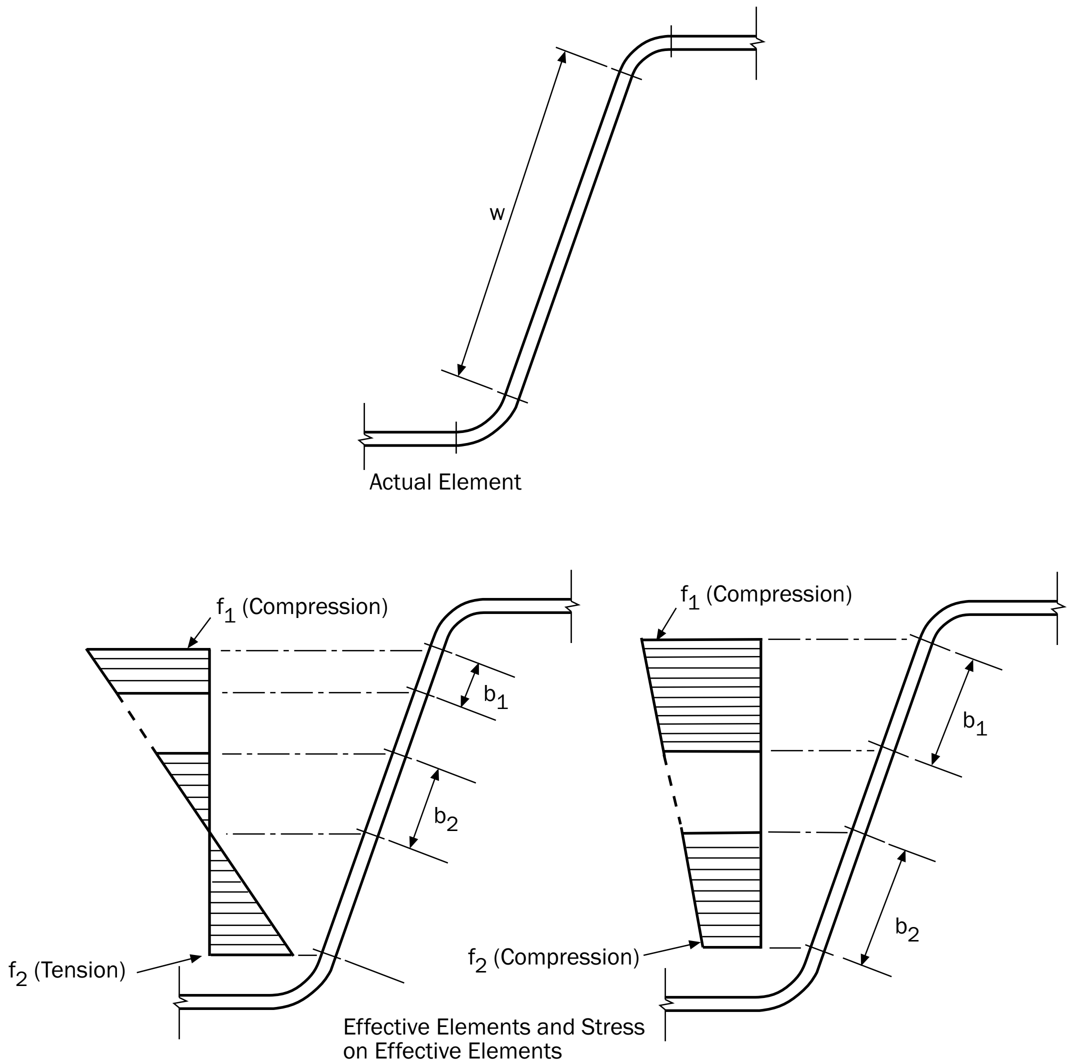
h_o = Out-to-out width of the web as defined in Figure B2.3-2

k = Plate buckling coefficient

$\psi = |f_2/f_1|$ (absolute value) (*Eq.* B2.3-1)

(a) Strength Determination

(i) For webs under stress gradient (f_1 in compression and f_2 in tension as shown in Figure B2.3-1)



(a) Webs under Stress Gradient (b) Other Stiffened Elements under Stress Gradient

Figure B2.3-1 Webs and other Stiffened Elements under Stress Gradient

$k = 4 + 2(1 + \psi)^3 + 2(1 + \psi)$ (*Eq*. B2.3-2)

For $h_o / b_o \le 4$

$b_1 = b_e / (3 + \psi)$ (*Eq*. B2.3-3)

$b_2 = b_e / 2$ when $\psi > 0.236$ (*Eq*. B2.3-4)

$b_2 = b_e - b_1$ when $\psi \le 0.236$ (*Eq*. B2.3-5)

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*. B2.3-6)

$b_2 = b_e / (1 + \psi) - b_1$ (*Eq*. B2.3-7)

(ii)For other stiffened elements under stress gradient (f_1 and f_2 in compression as shown in Figure B2.3-1)

$$k = 4 + 2(1 - \psi)^3 + 2(1 - \psi) \quad (Eq.\ B2.3\text{-}8)$$

$$b_1 = b_e / (3 - \psi) \quad (Eq.\ B2.3\text{-}9)$$

$$b_2 = b_e - b_1 \quad (Eq.\ B2.3\text{-}10)$$

(b) Serviceability Determination

The effective widths used in determining serviceability shall be calculated in accordance with Section B2.3(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.

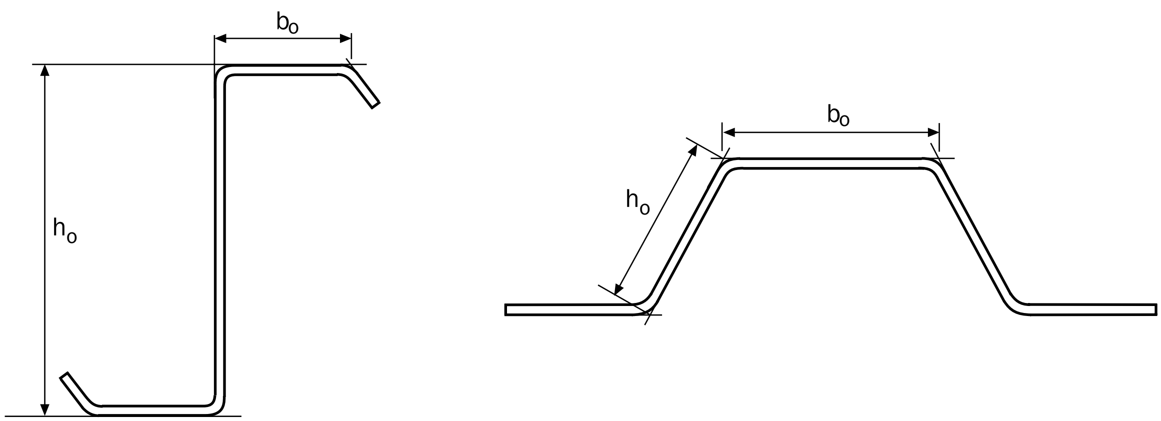


Figure B2.3-2 Out-to-Out Dimensions of Webs and Stiffened Elements under Stress Gradient

B2.4 C-Section Webs with Holes under Stress Gradient

These provisions shall be applicable within the following limits:

(1) d_0 / h < 0.7
(2) h / t ≤ 200
(3) Holes centered at mid-depth of the web
(4) Clear distance between holes ≥ 18 in. (457 mm)
(5) Non-circular holes, corner radii ≥ 2t
(6) Non-circular holes, d_0 ≤ 2.5 in. (64 mm) and b ≤ 4.5 in. (114 mm)
(7) Circular hole diameters ≤ 6 in. (152 mm)
(8) d_0 > 9/16 in. (14 mm)

(a) Strength Determination

When $d_0/h < 0.38$, the effective widths, b_1 and b_2, shall be determined by Section B2.3(a) by assuming no hole exists in the web.

When $d_0/h \geq 0.38$, the effective width shall be determined by Section B3.1(a) assuming the compression portion of the web consists of an unstiffened element adjacent to the hole with $f = f_1$ as shown in Figure B2.3-1.

(b) Serviceability Determination

The effective widths shall be determined by Section B2.3(b) by assuming no hole exists in the web.

where

d_0 = Depth of web hole
b = Length of web hole
b_1, b_2 = Effective widths defined by Figure B2.3-1
h = Depth of flat portion of the web measured along the plane of the web

Other variables are defined in B2.3.

B3 Effective Widths of Unstiffened Elements

B3.1 Uniformly Compressed Unstiffened Elements

(a) Strength Determination

The effective width, b, shall be determined in accordance with Section B2.1(a), except that k shall be taken as 0.43 and w as defined in Figure B3.1-1.

(b) Serviceability Determination

The effective width, b_d, used in determining serviceability shall be calculated in accordance with Procedure I of Section B2.1(b), except that f_d is substituted for f and k = 0.43.

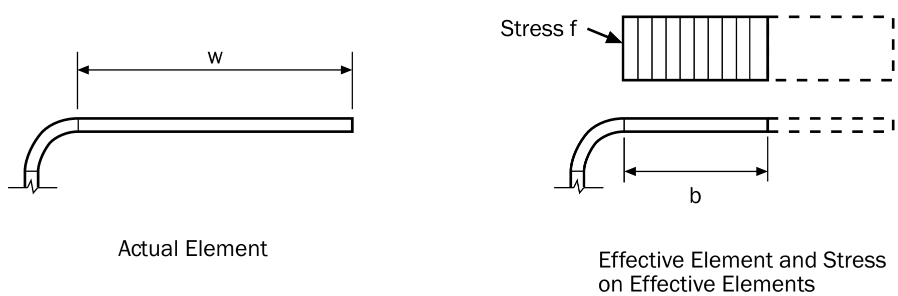


Figure B3.1-1 Unstiffened Element with Uniform Compression

B3.2 Unstiffened Elements and Edge Stiffeners with Stress Gradient

(a) Strength Determination

The effective width, b, shall be determined in accordance with Section B2.1(a) with $f = f_3$ as in Figure B4-2 in the element and k = 0.43.

(b) Serviceability Determination

The effective width, b_d, used in determining serviceability shall be calculated in accordance with Procedure I of Section B2.1(b), except that f_{d3} is substituted for f and k = 0.43, where f_{d3}=computed stress f_3 as shown in Figure B4-2. Calculations are based on the effective section at the load for which the serviceability is determined.

B4 Effective Widths of Elements with One Intermediate Stiffener or an Edge Stiffener

The following notation is used in this section.

$S = 1.28\sqrt{E/f}$ (*Eq*. B4-1)

k = Buckling coefficient

b_o = Dimension defined in Figure B4-1

d, w, D = Dimensions defined in Figure B4-2

d_s = Reduced effective width of the stiffener as specified in this section. d_s, calculated according to Section B4.2, is to be used in computing the overall effective section properties (see Figure B4-2)

d'_s = Effective width of the stiffener calculated according to Section B3.1 (see Figure B4-2)

A_s = Reduced area of the stiffener as specified in this section. A_s is to be used in computing the overall effective section properties. The centroid of the stiffener is to be considered located at the centroid of the full area of the stiffener.

I_a = Adequate moment of inertia of the stiffener, so that each component element will behave as a stiffened element.

I_s, A'_s = Moment of inertia of the full section of the stiffener about its own centroidal axis parallel to the element to be stiffened, and the effective area of the stiffener, respectively. For edge stiffeners, the round corner between the stiffener and the element to be stiffened shall not be considered as a part of the stiffener.

For the stiffener shown in Figure B4-2:

$I_s = (d^3 t \sin^2\theta)/12$ (*Eq*. B4-2)

$A'_s = d'_s t$ (*Eq*. B4-3)

B4.1 Uniformly Compressed Elements with One Intermediate Stiffener

(a) Strength Determination

For $b_o/t \le S$

$I_a = 0$ (no intermediate stiffener required)

$b = w$ (*Eq*. B4.1-1)

$A_s = A'_s$ (*Eq*. B4.1-2)

For $b_o/t > S$

$A_s = A'_s(R_I)$ (*Eq*. B4.1-3)

$$n = \left[0.583 - \frac{b_o/t}{12S}\right] \ge \frac{1}{3}$$ (*Eq*. B4.1-4)

$k = 3(R_I)^n + 1$ (*Eq*. B4.1-5)

$$R_I = I_s / I_a \le 1 \qquad (Eq.\ B4.1\text{-}6)$$

where

i) For $S < b_o / t < 3S$

$$I_a = t^4 \left[50 \frac{b_o / t}{S} - 50 \right] \qquad (Eq.\ B4.1\text{-}7)$$

ii)For $b_o / t \ge 3S$

$$I_a = t^4 \left[128 \frac{b_o / t}{S} - 285 \right] \qquad (Eq.\ B4.1\text{-}8)$$

The effective width, b, is calculated in accordance with Section B2.1(a).

(b) Serviceability Determination

The effective width, b_d, used in determining serviceability shall be calculated as in Section B4.1(a), except that f_d is substituted for f.

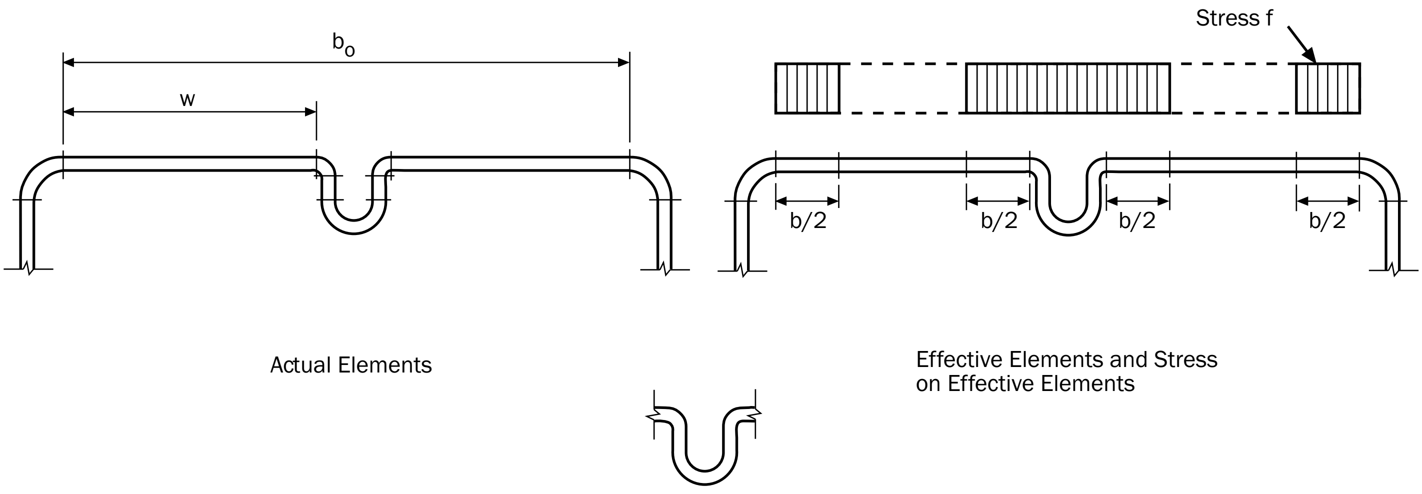


Figure B4-1 Elements with One Intermediate Stiffener

B4.2 Uniformly Compressed Elements with an Edge Stiffener

(a) Strength Determination

For $w / t \le 0.328S$:

$I_a = 0$ (no edge stiffener needed)

$b = w$ (*Eq*. B4.2-1)

$b_1 = b_2 = w / 2$ (see Fig. B4-2) (*Eq*. B4.2-2)

$d_s = d'_s$ for simple lip stiffener (*Eq*. B4.2-3)

$A_s = A'_s$ for other stiffener shapes (*Eq*. B4.2-4)

For $w / t > 0.328S$

$b_1 = b / 2 \, (R_I)$ (see Fig. B4-2) (*Eq*. B4.2-5)

$b_2 = b - b_1$ (see Fig. B4-2) (*Eq*. B4.2-6)

$d_s = d'_s \, (R_I)$ for simple lip stiffener (*Eq*. B4.2-7)

$A_s = A'_s\,(R_I)$ for other stiffener shapes (*Eq*. B4.2-8)

where

S =Term defined in Eq. B4-1.

$(R_I) = I_s / I_a \le 1$ (Eq. B4.2-9)

$$I_a = 399t^4\left[\frac{w/t}{S} - 0.327\right]^3 \le t^4\left[115\frac{w/t}{S} + 5\right] \quad (\textit{Eq}.\ \text{B4.2-10})$$

$$n = \left[0.582 - \frac{w/t}{4S}\right] \ge \frac{1}{3} \quad (\textit{Eq}.\ \text{B4.2-11})$$

The effective width, b, shall be calculated in accordance with Section B2.1 with k as given in Table B4.2.

Table B4.2 Determination of Plate Buckling Coefficient k

Simple Lip Edge Stiffener (140° ≥ θ ≥ 40°)		Other Edge Stiffener Shapes
$D/w \le 0.25$	$0.25 < D/w \le 0.8$	
$3.57(R_I)^n + 0.43 \le 4$	$(4.82 - \frac{5D}{w})(R_I)^n + 0.43 \le 4$	$3.57(R_I)^n + 0.43 \le 4$

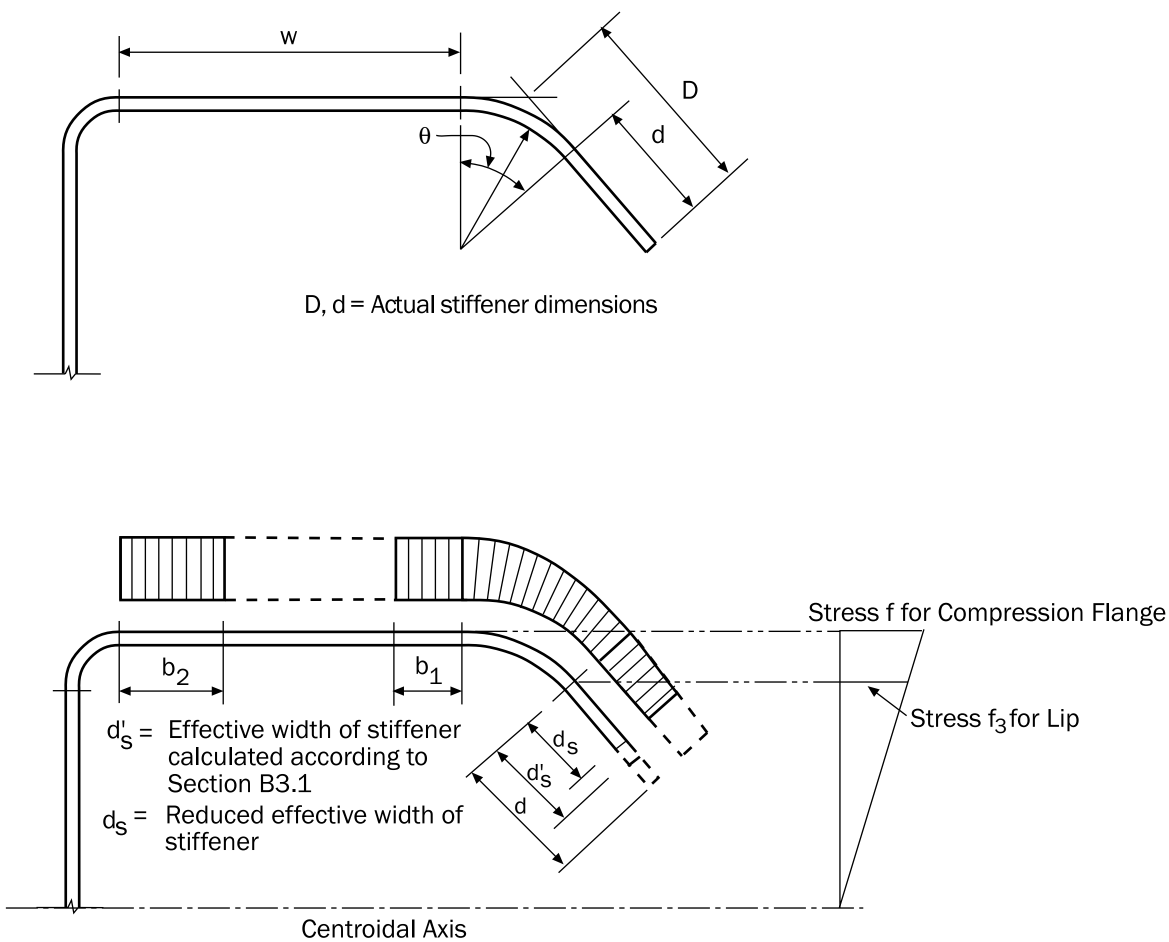


Figure B4-2 Elements with Simple Lip Edge Stiffener

(b) Serviceability Determination

The effective width, b_d, used in determining serviceability shall be calculated as in Section B4.2(a), except that f_d is substituted for f.

B5 Effective Widths of Stiffened Elements with Multiple Intermediate Stiffeners or Edge Stiffened Elements with Intermediate Stiffeners

B5.1 Effective Widths of Uniformly Compressed Stiffened Elements with Multiple Intermediate Stiffeners

The following notation is used in this section.

A_g =Gross area of the element including stiffeners

A_s =Gross area of a stiffener

b_e =Effective width of the element, located at the centroid of the element including stiffeners, see Figure B5.1-2.

b_p =Largest sub-element flat width, see Figure B5.1-1.

b_o =Total flat width of the stiffened element, see Figure B5.1-1.

c_i =Horizontal distance from the edge of the element to centerline(s) of the stiffener(s), see Figure B5.1-1.

f_1 =Uniform compressive stress acting on the flat element

h =Width of elements adjoining the stiffened element (e.g., the depth of the web in a hat section with multiple intermediate stiffeners in the compression flange is equal to h; if adjoining elements have different widths, use the smallest one.)

I_{sp} =Moment of inertia of a stiffener about the centerline of the flat portion of the element, the radii which connect the stiffener to the flat may be included.

k =Plate buckling coefficient of the 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 point or other restraint which restricts distortional buckling of the element.

R =Modification factor for the distortional plate buckling coefficient

n =Number of stiffeners in the element

t =Element thickness

i =Index for stiffener "i"

The effective width shall be determined as follows:

$$b_e = \rho\left(\frac{A_g}{t}\right) \quad (Eq.\ B5.1\text{-}1)$$

$$\rho = 1 \qquad \text{when } \lambda \le 0.673 \quad (Eq.\ B5.1\text{-}2)$$

$\rho = (1 - 0.22/\lambda)/\lambda$ when $\lambda > 0.673$ (*Eq*. B5.1-3)

$$\lambda = \sqrt{\frac{f_1}{F_{cr}}} \quad \text{(Eq. B5.1-4)}$$

$$F_{cr} = k \frac{\pi^2 E}{12(1-\mu^2)} \left(\frac{t}{b_o}\right)^2 \quad \text{(Eq. B5.1-5)}$$

The plate buckling coefficient, k, shall be determined from the minimum of Rk_d and k_{loc}, as determined from section B5.1.1 or B5.1.2, as appropriate.

k = the minimum of Rk_d and k_{loc} (*Eq*. B5.1-6)

$R = 2$ when $b_o/h < 1$ (*Eq*. B5.1-6)

$$R = \frac{11 - b_o/h}{5} \geq \frac{1}{2} \quad \text{when } b_o/h \geq 1 \quad \text{(Eq. B5.1-8)}$$

B5.1.1 Specific Case: ‘n’ Identical Stiffeners, Equally Spaced

(a) Strength Determination

$$k_{loc} = 4(n+1)^2 \quad \text{(Eq. B5.1.1-1)}$$

$$k_d = \frac{(1+\beta^2)^2 + \gamma(1+n)}{\beta^2(1+\delta(n+1))} \quad \text{(Eq. B5.1.1-2)}$$

$$\beta = (1+\gamma(n+1))^{1/4} \quad \text{(Eq. B5.1.1-3)}$$

If $L_{br} < \beta b_o$ then L_{br}/b_o shall be permitted to be substituted for β to account for increased capacity due to bracing.

$$\gamma = \frac{10.92 I_{sp}}{b_o t^3} \quad \text{(Eq. B5.1.1-4)}$$

$$\delta = \frac{A_s}{b_o t} \quad \text{(Eq. B5.1.1-5)}$$

(b) Serviceability Determination

The effective width, b_d, used in determining serviceability shall be calculated as in Section B5.1.1(a), except that f_d shall be substituted for f_1, 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.

B5.1.2 General Case: Arbitrary Stiffener Size, Location and Number

(a) Strength Determination

$$k_{loc} = 4\left(b_o/b_p\right)^2 \quad \text{(Eq. B5.1.2-1)}$$

$$k_d = \frac{(1+\beta^2)^2 + 2\sum_{i=1}^{n}\gamma_i \omega_i}{\beta^2\left(1 + 2\sum_{i=1}^{n}\delta_i \omega_i\right)} \quad (Eq.\ B5.1.2\text{-}2)$$

$$\beta = \left(2\sum_{i=1}^{n}\gamma_i \omega_i + 1\right)^{1/4} \quad (Eq.\ B5.1.2\text{-}3)$$

If $L_{br} < \beta b_o$ then L_{br}/b_o shall be permitted to be substituted for β to account for increased capacity due to bracing.

$$\gamma_i = \frac{10.92(I_{sp})_i}{b_o t^3} \quad (Eq.\ B5.1.2\text{-}4)$$

$$\omega_i = \sin^2\left(\pi \frac{c_i}{b_o}\right) \quad (Eq.\ B5.1.2\text{-}5)$$

$$\delta_i = \frac{(A_s)_i}{b_o t} \quad (Eq.\ B5.1.2\text{-}6)$$

(b) Serviceability Determination

The effective width, b_d, used in determining serviceability shall be calculated as in Section B5.1.2(a), except that f_d shall be substituted for f_1, 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.

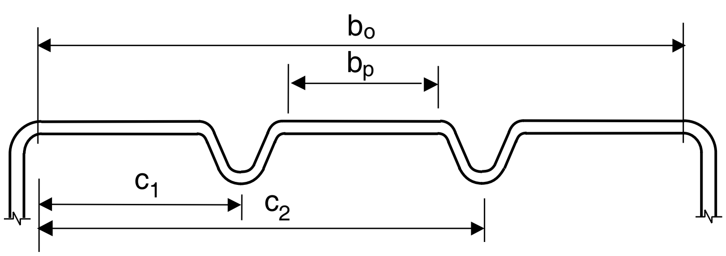


Figure B5.1-1 Plate Widths and Stiffener Location

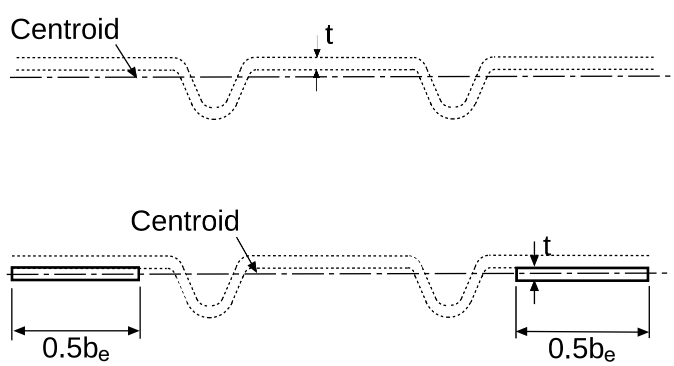


Figure B5.1-2 Effective Width Determination

B5.2 Edge Stiffened Elements with Intermediate Stiffeners

(a) Strength Determination

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 reductions are required.

If $b_o/t > 0.328S$, then the plate buckling coefficient, k, shall be determined from the provisions of Section B4.2, but with b_o replacing w in all expressions.

If k calculated from Section B4.2 is less than 4.0 ($k < 4$) then the intermediate stiffener(s) shall be ignored and the provisions of Section B4.2 should be followed for calculation of the effective width.

If k calculated from Section B4.2 is equal to 4.0 ($k = 4$) then the effective width of the edge stiffened element shall be calculated from the provisions of Section B5.1, with the following exception:

R calculated from equations B5.1-7 and B5.1-8 must be less than or equal to 1.

where

b_o = Total flat width of the edge stiffened element

Other variables are defined in Section B4 and B5.1.

(b) Serviceability Determination

The effective width, b_d, used in determining serviceability shall be calculated as in Section B5.2(a), except that f_d shall be substituted for f and f_1, where f_d is the computed compressive stress in the element being considered.

C. MEMBERS

C1 Properties of Sections

Properties of 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 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, is required.

C2 Tension Members

The provisions of this section are given in Section C2 of the Appendices.

C3 Flexural Members

C3.1 Strength [Resistance] for Bending Only

The nominal flexural strength [moment resistance], M_n, shall be the smallest of the values calculated according to Sections C3.1.1, C3.1.2, and C3.1.3, as well as sections provided under C3.1 of Appendix A, B, or C where applicable.

The provisions of this Section do not consider torsional effects, such as those resulting from loads that do not pass through the shear center of the cross section. See Section D3 for the design of lateral bracing required to restrain lateral bending or twisting.

C3.1.1 Nominal Section Strength [Resistance]

The nominal flexural strength [moment resistance], M_n, shall be calculated either on the basis of initiation of yielding in the effective section (Procedure I) or on the basis of the inelastic reserve capacity (Procedure II) as applicable.

For sections with stiffened or partially stiffened compression flanges:

USA and Mexico		Canada
Ω_b(ASD)	ϕ_b(LRFD)	ϕ_b(LSD)
1.67	0.95	0.90

For sections with unstiffened compression flanges:

USA and Mexico		Canada
Ω_b(ASD)	ϕ_b(LRFD)	ϕ_b(LSD)
1.67	0.90	0.90

(a) Procedure I - Based on Initiation of Yielding

Effective yield moment based on section strength [resistance], M_n, shall be determined as follows:

$$M_n = S_e F_y \qquad (Eq.\ C3.1.1\text{-}1)$$

where

F_y = Design yield stress as determined in Section A7.1

S_e = Elastic section modulus of the effective section calculated with the extreme compression or tension fiber at F_y

(b) Procedure II - Based on Inelastic Reserve Capacity

The inelastic flexural reserve capacity is permitted to be used when the following conditions are met:

(1) The member is not subject to twisting or to lateral, torsional, or torsional-flexural buckling.

(2) The effect of cold forming is not included in determining the yield point F_y.

(3) The ratio of the depth of the compressed portion of the web to its thickness does not exceed λ_1.

(4) The shear force does not exceed $0.35F_y$ times the web area, ht, for ASD, and $0.6F_y ht$ for LRFD and LSD.

(5) The angle between any web and the vertical does not exceed 30 degrees.

The nominal flexural strength [moment resistance], M_n, shall not exceed either 1.25 S_eF_y determined according to Procedure I or that causing a maximum compression strain of $C_y e_y$ (no limit is placed on the maximum tensile strain).

where

e_y =Yield strain = F_y/E

E =Modulus of elasticity

C_y =Compression strain factor determined as follows:

(a) Stiffened compression elements without intermediate stiffeners

$C_y = 3$ for $w/t \le \lambda_1$

$$C_y = 3 - 2\left(\frac{w/t - \lambda_1}{\lambda_2 - \lambda_1}\right) \text{for } \lambda_1 < \frac{w}{t} < \lambda_2$$

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

where

$$\lambda_1 = \frac{1.11}{\sqrt{F_y / E}} \qquad (Eq.\ C3.1.1\text{-}2)$$

$$\lambda_2 = \frac{1.28}{\sqrt{F_y / E}} \qquad (Eq.\ C3.1.1\text{-}3)$$

(b) Unstiffened compression elements

$C_y = 1$

(c) Multiple-stiffened compression elements and compression elements with edge stiffeners
$C_y = 1$

When applicable, effective design widths shall be used in calculating section properties. M_n 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 provisions of Section C3.5.

C3.1.2 Lateral-Torsional Buckling Strength [Resistance]

C3.1.2.1 Lateral-Torsional Buckling Strength [Resistance] of Open Cross Section Members

The provisions of this Section apply to I-, Z-, C- and other singly-symmetric section flexural members (not including multiple-web deck, U- and closed box-type members, and curved or arch members). The provisions of this Section do not apply to laterally unbraced compression flanges of otherwise laterally stable sections. Refer to C3.1.3 for C- and Z-purlins in which the tension flange is attached to sheathing.

For laterally unbraced segments of singly-, doubly-, and point-symmetric sections subject to lateral-torsional buckling, the nominal flexural strength [moment resistance], M_n, shall be calculated as follows:

$$M_n = S_c F_c \qquad (Eq.\ C3.1.2.1\text{-}1)$$

USA and Mexico		Canada
Ω_b(ASD)	ϕ_b(LRFD)	ϕ_b(LSD)
1.67	0.90	0.90

where

S_c = Elastic section modulus of effective section calculated at a stress F_c relative to the extreme compression fiber

F_c is determined as follows:

For $F_e \geq 2.78F_y$

$$F_c = F_y \qquad (Eq.\ C3.1.2.1\text{-}2)$$

For $2.78F_y > F_e > 0.56F_y$

$$F_c = \frac{10}{9}F_y\left(1 - \frac{10F_y}{36F_e}\right) \qquad (Eq.\ C3.1.2.1\text{-}3)$$

For $F_e \leq 0.56F_y$

$$F_c = F_e \qquad (Eq.\ C3.1.2.1\text{-}4)$$

where

F_e = Elastic critical lateral-torsional buckling stress calculated according to (a) or (b) below:

(a) For singly-, doubly-, and point-symmetric sections:

$$F_e = \frac{C_b r_o A}{S_f}\sqrt{\sigma_{ey}\sigma_t} \quad \text{for bending about the symmetry axis.} \qquad (Eq.\ C3.1.2.1\text{-}5)$$

For singly-symmetric sections, x-axis is the axis of symmetry oriented such that the shear center has a negative x-coordinate.

For point-symmetric sections, use 0.5 F_e. X-axis of Z-sections is the centroidal axis perpendicular to the web.

Alternatively, F_e can be calculated using the equation given in (b) for doubly-symmetric I-sections, singly-symmetric C-sections, or point-symmetric Z-sections.

For singly-symmetric sections bending about the centroidal axis perpendicular to the axis of symmetry:

$$F_e = \frac{C_s A \sigma_{ex}}{C_{TF} S_f}\left[j + C_s\sqrt{j^2 + r_o^2(\sigma_t / \sigma_{ex})} \right] \qquad (Eq.\ C3.1.2.1\text{-}6)$$

C_s = +1 for moment causing compression on the shear center side of the centroid

C_s = -1 for moment causing tension on the shear center side of the centroid

$$\sigma_{ex} = \frac{\pi^2 E}{(K_x L_x / r_x)^2} \qquad (Eq.\ C3.1.2.1\text{-}7)$$

$$\sigma_{ey} = \frac{\pi^2 E}{(K_y L_y / r_y)^2} \qquad (Eq.\ C3.1.2.1\text{-}8)$$

$$\sigma_t = \frac{1}{A r_o^2}\left[GJ + \frac{\pi^2 E C_w}{(K_t L_t)^2} \right] \qquad (Eq.\ C3.1.2.1\text{-}9)$$

A =Full unreduced cross-sectional area

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

$$C_b = \frac{12.5 M_{max}}{2.5 M_{max} + 3M_A + 4M_B + 3M_C} \qquad (Eq.\ C3.1.2.1\text{-}10)$$

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 unity for all cases. For cantilevers or overhangs where the free end is unbraced, C_b shall be taken as unity.

E =Modulus of elasticity

$$C_{TF} = 0.6 - 0.4\,(M_1 / M_2) \qquad (Eq.\ C3.1.2.1\text{-}11)$$

where

M_1 is the smaller and M_2 the larger bending moment at the ends of the unbraced length in the plane of bending, and where M_1/M_2, the ratio of end moments, is positive when M_1 and M_2 have the same sign (reverse curvature bending) and negative when they are of opposite sign (single curvature bending). When the bending moment at any point within an unbraced length is larger than that at both ends of this length, C_{TF} shall be taken as unity.

r_o = Polar radius of gyration of the cross section about the shear center

$$= \sqrt{r_x^2 + r_y^2 + x_o^2} \qquad (Eq.\ C3.1.2.1\text{-}12)$$

r_x, r_y = Radii of gyration of the cross section about the centroidal principal axes

G = Shear modulus

K_x, K_y, K_t = Effective length factors for bending about the x- and y-axes, and for twisting

L_x, L_y, L_t = Unbraced length of the member for bending about the x- and y-axes, and for twisting

x_o = Distance from the shear center to the centroid along the principal x-axis, taken as negative

J = Saint-Venant torsion constant of the cross section

C_w = Torsional warping constant of the cross section

$$j = \frac{1}{2I_y}\left[\int_A x^3 dA + \int_A xy^2 dA\right] - x_o \qquad (Eq.\ C3.1.2.1\text{-}13)$$

(b) For I-sections, singly-symmetric C-sections, or Z-sections bent about the centroidal axis perpendicular to the web (x-axis), the following equations are permitted to be used in lieu of (a) to calculate F_e:

$$F_e = \frac{C_b \pi^2 E d I_{yc}}{S_f (K_y L_y)^2} \quad \text{for doubly-symmetric I-sections and singly-symmetric C-sections} \qquad (Eq.\ C3.1.2.1\text{-}14)$$

$$= \frac{C_b \pi^2 E d I_{yc}}{2 S_f (K_y L_y)^2} \quad \text{for point-symmetric Z-sections} \qquad (Eq.\ C3.1.2.1\text{-}15)$$

where

d = Depth of section

I_{yc} = Moment of inertia of the compression portion of a section about the centroidal axis of the entire section parallel to the web, using the full unreduced section

Other terms are defined in (a).

C3.1.2.2 Lateral-Torsional Buckling Strength [Resistance] of Closed Box Members

For closed box members, the nominal flexural strength [moment resistance], M_n, shall be determined as follows:

If the laterally unbraced length of the member is less than or equal to L_u, the nominal flexural strength [moment resistance] shall be determined by using Section C3.1.1.

where

$$L_u = \frac{0.36 C_b \pi}{F_y S_f} \sqrt{EGJI_y} \qquad (Eq.\ C3.1.2.2\text{-}1)$$

If the laterally unbraced length of a member is larger than L_u, the nominal flexural strength [moment resistance] shall be determined in accordance with C3.1.2.1, where the critical lateral buckling stress, F_e, is calculated as follows:

$$F_e = \frac{C_b \pi}{K_y L_y S_f} \sqrt{EGJI_y} \qquad (Eq.\ C3.1.2.2\text{-}2)$$

where

I_y = Moment of inertia of full unreduced section about its centroidal axis parallel to web

J = Torsional constant of box section

Other variables are defined in Section C3.1.2.1.

C3.1.3 Beams Having One Flange Through-Fastened to Deck or Sheathing

This section does 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 [moment 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 as follows:

$$M_n = RS_eF_y \qquad (Eq.\ C3.1.3\text{-}1)$$

USA and Mexico		Canada
Ω_b(ASD)	ϕ_b(LRFD)	ϕ_b(LSD)
1.67	0.90	0.80

where R is obtained from Table C3.1.3-1 for simple span C- or Z-sections, and

R = 0.60 for continuous span C-sections

= 0.70 for continuous span Z-sections

S_e and F_y are defined in Section C3.1.1.

The reduction factor, R, shall be limited to roof and wall systems meeting the following conditions:

(1) Member depth less than 11.5 in. (292 mm)

(2) Member flanges shall have edge stiffeners

(3) $60 \le$ depth / thickness ≤ 170

(4) $2.8 \le$ depth / flange width ≤ 4.5

(5) $16 \le$ flat width / thickness of flange ≤ 43

(6) For continuous span systems, the lap length at each interior support

in each direction (distance from center of support to end of lap) shall not be less than 1.5d

(7) Member span length shall be no greater than 33 feet (10 m)

(8) For continuous span systems, the longest member span length shall not be more than 20% greater than the shortest span length

(9) Both flanges shall be prevented from moving laterally at the supports

(10) Roof or wall panels shall be steel sheets with 50 ksi (340 MPa or 3520 kg/cm^2) minimum yield strength, and a minimum of 0.018 in. (0.46 mm) base metal thickness, having a minimum rib depth of 1-1/4 in. (32 mm), spaced a maximum of 12 in. (305 mm) on centers and attached in a manner to effectively inhibit relative movement between the panel and purlin flange

(11) Insulation shall be 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

(12) Fastener type: minimum No. 12 self-drilling or self-tapping sheet metal screws or 3/16 in. (4.76 mm) rivets, having washers 1/2 in. (12.7 mm) diameter

(13) Fasteners shall not be standoff type screws

(14) Fasteners shall be spaced not greater than 12 in. (305 mm) on centers and placed near the center of the beam flange, and adjacent to the panel high rib

(15) The design yield strength of the member shall not exceed 60 ksi (410 MPa or 4220 kg/cm^2)

If variables fall outside any of the above stated limits, the user must perform full scale tests in accordance with Section F1 of the *Specification*, or apply a rational analysis procedure. In any case, the user is permitted to perform tests, in accordance with Section F1, as an alternate to the procedure described in this section.

TABLE C3.1.3-1
Simple Span C- or Z-Section R Values

Depth Range, in. (mm)	Profile	R
$d \le 6.5$ (165)	C or Z	0.70
6.5 (165) $< d \le$ 8.5 (216)	C or Z	0.65
8.5 (216) $< d \le$ 11.5 (292)	Z	0.50
8.5 (216) $< d \le$ 11.5 (292)	C	0.40

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 C3.1.3-1 by the following correction factor, r:

$r = 1.00 - 0.01\, t_i$ when t_i is in inches (*Eq*. C3.1.3-2)

$r = 1.00 - 0.0004\, t_i$ when t_i is in millimeters (*Eq*. C3.1.3-3)

t_i = Thickness of uncompressed glass fiber blanket insulation

C3.1.4 Beams Having One Flange Fastened to a Standing Seam Roof System

The provisions of this section are given in Section C3.1.4 of the Appendices.

C3.1.5 Strength [Resistance] of Standing Seam Roof Panel Systems

When results of tests on standing seam roof panel systems conducted according to ASTM E1592-95 are to be evaluated, the “Standard Procedures for Panel and Anchor Structural Tests” of Part VIII of the AISI *Cold-Formed Steel Design Manual* shall be followed. Strength [Resistance] under uplift loading shall be evaluated by this procedure.

When the number of physical test assemblies is 3 or more, factors of safety and resistance factors shall be determined in accordance with the procedures of Section F1.1(b) with the following definition for the variables:

β_o = Target reliability index
= 2.0 for panel flexural limits
= 2.5 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 same tributary area (for anchor failure), or number of panels with identical spans and loading to the failed span (for non-anchor failures)

When the number of physical test assemblies is less than 3, a factor of safety, Ω, of 2.0 and a resistance factor, ϕ, of 0.8 (LRFD) and 0.70 (LSD) shall be used.

C3.2 Strength [Resistance] for Shear Only

C3.2.1 Shear Strength [Resistance] of Webs without Holes

The nominal shear strength [resistance], V_n, shall be calculated as follows:

$$V_n = A_w F_v \qquad (Eq.\ C3.2.1\text{-}1)$$

(a) For $h/t \le \sqrt{Ek_v/F_y}$

$$F_v = 0.60F_y \qquad (Eq.\ C3.2.1\text{-}2)$$

(b) For $\sqrt{Ek_v/F_y} < h/t \le 1.51\sqrt{Ek_v/F_y}$

$$F_v = \frac{0.60\sqrt{Ek_v F_y}}{(h/t)} \qquad (Eq.\ C3.2.1\text{-}3)$$

(c) For $h/t > 1.51\sqrt{Ek_v/F_y}$

$$F_v = \frac{\pi^2 Ek_v}{12(1-\mu^2)(h/t)^2} = 0.904\,Ek_v/(h/t)^2 \qquad (Eq.\ C3.2.1\text{-}4)$$

USA and Mexico		Canada
Ω_v(ASD)	ϕ_v(LRFD)	ϕ_v(LSD)
1.60	0.95	0.80

where

A_w = Area of web element = ht

E = Modulus of elasticity of steel

F_v = Nominal shear stress

V_n = Nominal shear strength [resistance]

t = Web thickness

h = Depth of flat portion of ~~the~~ web measured along plane of web

μ = Poisson's ratio = 0.3

k_v = Shear buckling coefficient determined as follows:

1. For unreinforced webs, $k_v = 5.34$
2. For webs with transverse stiffeners satisfying the requirements of Section C3.6

   when $a/h \le 1.0$

$$k_v = 4.00 + \frac{5.34}{(a/h)^2} \qquad (Eq.\ C3.2.1\text{-}5)$$

   when $a/h > 1.0$

$$k_v = 5.34 + \frac{4.00}{(a/h)^2} \qquad (Eq.\ C3.2.1\text{-}6)$$

   where

   a = Shear panel length of unreinforced web element
   = Clear distance between transverse stiffeners of reinforced web elements.

For a web consisting of two or more sheets, each sheet shall be considered as a separate element carrying its share of the shear force.

C3.2.2 Shear Strength [Resistance] of C-Section Webs with Holes

These provisions shall be applicable within the following limits:

(1) $d_0 / h < 0.7$
(2) $h / t \leq 200$
(3) Holes centered at mid-depth of the web
(4) Clear distance between holes ≥ 18 in. (457 mm)
(5) Non-circular holes corner radii ≥ 2t
(6) Non-circular holes, $d_0 \leq 2.5$ in. (64 mm) and $b \leq 4.5$ in. (114 mm)
(7) Circular hole diameters ≤ 6 in. (152 mm)
(8) $d_0 > 9/16$ in. (14 mm)

The nominal shear strength [resistance], V_n, determined by Section C3.2.1 shall be multiplied by q_s:

When $c/t \geq 54$

$$q_s = 1.0 \qquad \text{(Eq. C3.2.2-1)}$$

When $5 \leq c/t < 54$

$$q_s = c/(54t) \qquad \text{(Eq. C3.2.2-2)}$$

where

$c = h/2 - d_0/2.83$ for circular holes (Eq. C3.2.2-3)

$ = h/2 - d_0/2$ for non-circular holes (Eq. C3.2.2-4)

d_0 = Depth of web hole
b = Length of web hole
h = Depth of flat portion of the web measured along the plane of the web

C3.3 Strength [Resistance] for Combined Bending and Shear

C3.3.1 ASD Method

For beams subjected to combined bending and shear, the required allowable flexural strength, M, and required allowable shear strength, V, shall not exceed M_n/Ω_b and V_n/Ω_v, respectively.

For beams with unreinforced webs, the required allowable flexural strength, M, and required allowable shear strength, V, shall also satisfy the following interaction equation:

$$\left(\frac{\Omega_b M}{M_{nxo}}\right)^2 + \left(\frac{\Omega_v V}{V_n}\right)^2 \leq 1.0 \qquad \text{(Eq. C3.3.1-1)}$$

For beams with transverse web stiffeners, when $\Omega_b M/M_{nxo} > 0.5$ and $\Omega_v V/V_n > 0.7$, M and V shall also satisfy the following interaction equation:

$$0.6\left(\frac{\Omega_b M}{M_{nxo}}\right) + \left(\frac{\Omega_v V}{V_n}\right) \leq 1.3 \qquad \text{(Eq. C3.3.1-2)}$$

where:

Ω_b =Factor of safety for bending (See Section C3.1.1)

Ω_v =Factor of safety for shear (See Section C3.2)

M_n =Nominal flexural strength when bending alone is considered

M_{nxo}= Nominal flexural strength about the centroidal x-axis determined in accordance with Section C3.1.1

V_n =Nominal shear strength when shear alone is considered

C3.3.2 LRFD and LSD Methods

For beams subjected to combined bending and shear, the required flexural strength [factored moment], M_*, and the required shear strength [factored shear], V_*, shall not exceed $\phi_b M_n$ and $\phi_v V_n$, respectively.

For beams with unreinforced webs, the required flexural strength [factored moment], M_*, and the required shear strength [factored shear], V_*, shall also satisfy the following interaction equation:

$$\left(\frac{M_*}{\phi_b M_{nxo}}\right)^2 + \left(\frac{V_*}{\phi_v V_n}\right)^2 \leq 1.0 \qquad (Eq.\ C3.3.2\text{-}1)$$

For beams with transverse web stiffeners, when $M_* / (\phi_b M_{nxo}) > 0.5$ and $V_* / (\phi_v V_n) > 0.7$, M_* and V_* shall also satisfy the following interaction equation:

$$0.6\left(\frac{M_*}{\phi_b M_{nxo}}\right) + \left(\frac{V_*}{\phi_v V_n}\right) \leq 1.3 \qquad (Eq.\ C3.3.2\text{-}2)$$

where:

ϕ_b =Resistance factor for bending (See Section C3.1.1)

ϕ_v =Resistance factor for shear (See Section C3.2)

M_n =Nominal flexural strength [moment resistance] when bending alone is considered

M_{nxo}=Nominal flexural strength [moment resistance] about the centroidal x-axis determined in accordance with Section C3.1.1

M_* =Required flexural strength [factored moment]

$M_* = M_u$ (LRFD)

$M_* = M_f$ (LSD)

V_n =Nominal shear strength [resistance] when shear alone is considered

V_* =Required shear strength [factored shear]

$V_* = V_u$ (LRFD)

$V_* = V_f$ (LSD)

C3.4 Web Crippling Strength [Resistance]

C3.4.1 Web Crippling Strength [Resistance] of Webs without Holes

The nominal web crippling strength [resistance], P_n, shall be determined as follows:

$$P_n = Ct^2 F_y \sin\theta \left(1 - C_R\sqrt{\frac{R}{t}}\right)\left(1 + C_N\sqrt{\frac{N}{t}}\right)\left(1 - C_h\sqrt{\frac{h}{t}}\right) \qquad (Eq.\ C3.4.1\text{-}1)$$

where:

P_n = Nominal web crippling strength [resistance]

C = Coefficient from Table C3.4.1-1, C3.4.1-2, C3.4.1-3, C3.4.1-4 or C3.4.1-5

C_h = Web slenderness coefficient from Table C3.4.1-1, C3.4.1-2, C3.4.1-3, C3.4.1-4 or C3.4.1-5

C_N = Bearing length coefficient from Table C3.4.1-1, C3.4.1-2, C3.4.1-3, C3.4.1-4 or C3.4.1-5

C_R = Inside bend radius coefficient from Table C3.4.1-1, C3.4.1-2, C3.4.1-3, C3.4.1-4 or C3.4.1-5

h = Flat dimension of web measured in plane of web

N = Bearing length [¾ in. (19 mm) minimum]

R = Inside bend radius

t = Web thickness

θ = Angle between plane of web and plane of bearing surface, $45° < \theta \leq 90°$

Webs of members in bending for which h/t is greater than 200 shall be provided with adequate means of transmitting concentrated loads or reactions directly into the web(s).

P_n represents the nominal strength [resistance] for load or reaction for one solid web connecting top and bottom flanges. For webs consisting of two or more such sheets, P_n, shall be calculated for each individual sheet and the results added to obtain the nominal strength for the full section.

One-flange loading or reaction occurs when the clear distance between the bearing edges of adjacent opposite concentrated loads or reactions is greater than 1.5h.

Two-flange loading or reaction occurs when the clear distance between the bearing edges of adjacent opposite concentrated loads or reactions is equal to or less than 1.5h.

End loading or reaction occurs when 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 occurs when the distance from the edge of the bearing to the end of the member is greater than 1.5h, except that otherwise noted herein.

The factors of safety and resistance factors are provided in the Tables C3.4.1-1 to C3.4.1-5.

TABLE C3.4.1-1
BUILT-UP 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	10	0.14	0.28	0.001	2.00	0.75	0.60	R / t ≤ 5
			Interior	20	0.15	0.05	0.003	1.65	0.90	0.80	R / t ≤ 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 ≤ 5
			Interior	20.5	0.17	0.11	0.001	1.75	0.85	0.75	R / t ≤ 3
		Two-Flange Loading or Reaction	End	15.5	0.09	0.08	0.04	2.00	0.75	0.65	R / t ≤ 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 ≤ 5
			Interior	20.5	0.17	0.11	0.001	1.75	0.85	0.75	R / t ≤ 3

Notes:

This Table applies to I-beams made from two channels connected back to back. See Section C3.4.1 of *Commentary* for explanation.

The above coefficients apply when $h/t \le 200$, $N/t \le 210$, $N/h \le 1.0$ and $\theta = 90°$.

TABLE C3.4.1-2
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 ≤ 9
			Interior	13	0.23	0.14	0.01	1.65	0.90	0.80	R / t ≤ 5
		Two-Flange Loading or Reaction	End	7.5	0.08	0.12	0.048	1.75	0.85	0.75	R / t ≤ 12
			Interior	20	0.10	0.08	0.031	1.75	0.85	0.75	R / t ≤ 12
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 ≤ 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 ≤ 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 ≤ 2
			Interior	13	0.32	0.10	0.01	1.80	0.85	0.70	R / t ≤ 1
		Two-Flange Loading or Reaction	End	2	0.11	0.37	0.01	2.00	0.75	0.65	R / t ≤ 1
			Interior	13	0.47	0.25	0.04	1.90	0.80	0.65	

Note:

(1) The above coefficients apply when $h / t \leq 200$, $N / t \leq 210$, $N / h \leq 2.0$ and $\theta = 90°$.

(2) For interior two-flange loading or reaction of members having flanges fastened to the support, the distance from the edge of bearing to the end of the member shall be extended at least 2.5h. Otherwise, values for the unfastened condition shall apply.

TABLE C3.4.1-3
SINGLE WEB Z-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	$R / t \leq 9$
			Interior	13	0.23	0.14	0.01	1.65	0.90	0.80	$R / t \leq 5$
		Two-Flange Loading or Reaction	End	9	0.05	0.16	0.052	1.75	0.85	0.75	$R / t \leq 12$
			Interior	24	0.07	0.07	0.04	1.85	0.80	0.70	$R / t \leq 12$
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	$R / t \leq 5$
			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	$R / t \leq 3$
			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	$R / t \leq 2$
			Interior	13	0.32	0.10	0.01	1.80	0.85	0.70	$R / t \leq 1$
		Two-Flange Loading or Reaction	End	2	0.11	0.37	0.01	2.00	0.75	0.65	$R / t \leq 1$
			Interior	13	0.47	0.25	0.04	1.90	0.80	0.65	

Note:

(1) The above coefficients apply when $h / t \leq 200$, $N / t \leq 210$, $N / h \leq 2.0$ and $\theta = 90°$.

(2) For interior two-flange loading or reaction of members having flanges fastened to the support, the distance from the edge of bearing to the end of the member shall be extended at least 2.5h. Otherwise, values for the unfastened condition shall apply.

TABLE C3.4.1-4
SINGLE HAT SECTIONS

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	$R/t \le 5$
		Interior	17	0.13	0.13	0.04	1.90	0.80	0.70	$R/t \le 10$
	Two-Flange Loading or Reaction	End	9	0.10	0.07	0.03	1.75	0.85	0.75	$R/t \le 10$
		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	$R/t \le 4$
		Interior	17	0.13	0.13	0.04	1.70	0.90	0.75	$R/t \le 4$

Note:

The above coefficients apply when $h/t \le 200$, $N/t \le 200$, $N/h \le 2$ and $\theta = 90°$.

TABLE C3.4.1-5
MULTI-WEB DECK SECTIONS

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	3	0.08	0.70	0.055	2.25	0.65	0.55	$R/t \le 7$
		Interior	8	0.10	0.17	0.004	1.75	0.85	0.75	$R/t \le 10$
	Two-Flange Loading or Reaction	End	9	0.12	0.14	0.040	1.80	0.85	0.70	$R/t \le 10$
		Interior	10	0.11	0.21	0.020	1.75	0.85	0.75	
Unfastened	One-Flange Loading or Reaction	End	3	0.08	0.70	0.055	2.25	0.65	0.55	$R/t \le 7$
		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	$R/t \le 5$
		Interior	17	0.10	0.10	0.046	1.65	0.90	0.80	

Notes:

(1) The above coefficients apply when $h/t \le 200$, $N/t \le 210$, $N/h \le 3$.

(2) $45° < \theta \le 90°$

C3.4.2 Web Crippling Strength [Resistance] of C-Section Webs with Holes

When a web hole is within the bearing length, a bearing stiffener shall be used.

For beam webs with holes, the web crippling strength [resistance] shall be computed by using Section C3.4.1 multiplied by the reduction factor, R_c, given in this section.

These provisions shall be applicable within the following limits:

(1) $d_0 / h \leq 0.7$

(2) $h / t \leq 200$

(3) Hole centered at mid-depth of the web

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

(5) Distance between the end of the member and the edge of the hole ≥ d

(6) Non-circular holes, corner radii ≤ 2t

(7) Non-circular holes, $d_0 \leq 2.5$ in. (64 mm) and $b \leq 4.5$ in. (114 mm)

(8) Circular hole diameters ≤ 6 in. (152 mm)

(9) $d_0 > 9/16$ in. (14 mm)

For end-one flange reaction (Equation C3.4.1-1 with Table C3.4.1-2) when a web hole is not within the bearing length:

$$R_c = 1.01 - 0.325 d_0/h + 0.083\, x/h \leq 1.0 \qquad (Eq.\ C3.4.2\text{-}1)$$

$N \geq 1$ in. (25 mm)

For interior-one flange reaction (Equation C3.4.1-1 with Table C3.4.1-2) when any portion of a web hole is not within the bearing length:

$$R_c = 0.90 - 0.047\, d_0/h + 0.053\, x/h \leq 1.0 \qquad (Eq.\ C3.4.2\text{-}2)$$

$N \geq 3$ in. (76 mm)

where

b = Length of web hole

d = Depth of cross section

d_0 = Depth of web hole

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

x = Nearest distance between the web hole and the edge of bearing

N = Bearing length

C3.5 Combined Bending and Web Crippling Strength [Resistance]

C3.5.1 ASD Method

Unreinforced flat webs of shapes subjected to a combination of bending and concentrated load or reaction shall be designed to meet the following requirements:

(a) For shapes having single unreinforced webs:

$$1.2\left(\frac{\Omega_w P}{P_n}\right) + \left(\frac{\Omega_b M}{M_{nxo}}\right) \leq 1.5 \qquad (Eq.\ C3.5.1\text{-}1)$$

Exception: At the interior supports of continuous spans, the above equation is not applicable 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 which provide a high degree of restraint against rotation of the web (such as I-sections made by welding two angles to a C-section);

$$1.1\left(\frac{\Omega_w P}{P_n}\right)+\left(\frac{\Omega_b M}{M_{nxo}}\right)\leq 1.5 \qquad (Eq.\ C3.5.1\text{-}2)$$

Exception: In lieu of equation C3.5.1-2, when $h/t \leq 2.33/\sqrt{F_y/E}$ and $\lambda \leq 0.673$, it shall be permitted to determine the allowable concentrated load or reaction by using $\frac{P_n}{\Omega_w}$ from Section C3.4.

In the above equations:

Ω_b = Factor of safety for bending (See Section C3.1.1)

Ω_w = Factor of safety for web crippling (See Section C3.4)

P = Required allowable strength for the concentrated load or reaction in the presence of bending moment

P_n = Nominal strength for concentrated load or reaction in the absence of bending moment determined in accordance with Section C3.4

M = Required allowable flexural strength at, or immediately adjacent to, the point of application of the concentrated load or reaction, P

M_{nxo} = Nominal flexural strength about the centroidal x-axis determined in accordance with Section C3.1.1

w = Flat width of the beam flange which contacts the bearing plate

t = Thickness of the web or flange

λ = Slenderness factor given by Section B2.1

(c) For the support point of two nested Z-shapes:

$$\frac{M}{M_{no}}+0.85\frac{P}{P_n}\leq\frac{1.65}{\Omega} \qquad (Eq.\ C3.5.1\text{-}3)$$

In addition, the moment, M, and the concentrated load or reaction, P, shall satisfy $M \leq M_{no}/\Omega_b$, and $P \leq P_n/\Omega_w$.

where

M = Required allowable flexural strength at the section under consideration

M_{no} = Nominal flexural strength for the nested Z-sections, i.e. sum of the two sections evaluated individually, determined in accordance with Section C3.1.1

P = Required allowable strength for the concentrated load or reaction in the presence of bending moment

P_n = Nominal web crippling strength assuming single web interior one-flange loading for the nested Z-sections, i.e., sum of the two webs evaluated individually

Ω = Factor of safety for combined bending and web crippling = 1.75

The above equation is valid for shapes that meet the following limits:

$h/t \leq 150$

$N/t \leq 140$

$F_y \leq$ 70 ksi (480 MPa or 4910 kg/cm^2)

$R/t \leq 5.5$

The following conditions shall also be satisfied:

(1) The ends of each section shall be connected to the other section by a minimum of two 1/2 in. (12.7 mm) diameter A307 bolts through the web.

(2) The combined section shall be connected to the support by a minimum of two 1/2 in. (12.7 mm) diameter A307 bolts through the flanges.

(3) The webs of the two sections shall be in contact.

(4) The ratio of the thicker to the thinner part shall not exceed 1.3.

C3.5.2 LRFD and LSD Methods

Unreinforced flat webs of shapes subjected to a combination of bending and concentrated load or reaction shall be designed to meet the following requirements:

(a) For shapes having single unreinforced webs:

$$1.07\left(\frac{P_*}{\phi_w P_n}\right)+\left(\frac{M_*}{\phi_b M_{nxo}}\right)\leq 1.42 \qquad (Eq.\ C3.5.2\text{-}1)$$

Exception: At the interior supports of continuous spans, the above equation is not applicable 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 which provide a high degree of restraint against rotation of the web (such as I-sections made by welding two angles to a C-section);

$$0.82\left(\frac{P_*}{\phi_w P_n}\right)+\left(\frac{M_*}{\phi_b M_{nxo}}\right)\leq 1.32 \qquad (Eq.\ C3.5.2\text{-}2)$$

Exception: In lieu of equation C3.5.2-2, when $h/t \le 2.33/\sqrt{F_y/E}$ and $\lambda \le$ 0.673, it shall be permitted to determine the design strength for a concentrated load or reaction by using $\phi_w P_n$ from Section C3.4.

In the above equations:

ϕ_b =Resistance factor for bending (See Section C3.1.1)

ϕ_w =Resistance factor for web crippling (See Section C3.4)

P_* =Required strength for the concentrated load or reaction [factored concentrated load or reaction] in the presence of bending moment.

$P_* = P_u$ (LRFD)

$P_* = P_f$ (LSD)

P_n =Nominal strength [resistance] for concentrated load or reaction in the absence of bending moment determined in accordance with Section C3.4

M_* =Required flexural strength [factored moment] at, or immediately adjacent to, the point of application of the concentrated load or reaction P_*

$M_* = M_u$ (LRFD)

$M_* = M_f$ (LSD)

M_{nxo}= Nominal flexural strength [moment resistance] about the centroidal x-axis determined in accordance with Section C3.1.1

w =Flat width of the beam flange which contacts the bearing plate

t =Thickness of the web or flange

λ =Slenderness factor given by Section B2.1

(c) For two nested Z-shapes

$$\frac{M_*}{M_{no}} + 0.85\frac{P_*}{P_n} \le 1.65\phi \qquad (Eq.\ C3.5.2\text{-}3)$$

In addition, the moment, M_*, and the concentrated load or reaction, P_*, shall satisfy $M_* \le \phi_b M_{no}$, and $P_* \le \phi_w P_n$.

where

M_* =Required flexural strength [factored moment] at the section under consideration.

$M_* = M_u$ (LRFD)

$M_* = M_f$ (LSD)

M_{no}= Nominal flexural strength for the two nested Z-sections, i.e., sum of the two sections evaluated individually, determined in accordance with Section C3.1.1

P_* =Required strength for the concentrated load or reaction [factored concentrated load or reaction] in the presence of bending moment.

$P_* = P_u$ (LRFD)

$P_* = P_f$ (LSD)

P_n =Nominal web crippling strength [resistance] assuming single web interior one-flange loading for the nested Z-sections, i.e., sum of the two webs evaluated individually

ϕ =0.90 (LRFD)

=0.80 (LSD)

The above equation is valid for shapes that meet the following limits:

$h/t \leq 150$

$N/t \leq 140$

$F_y \leq$ 70 ksi (480 MPa or 4910 kg/cm^2)

$R/t \leq 5.5$

The following conditions shall also be satisfied:

(1) The ends of each section shall be connected to the other section by a minimum of two 1/2 in. (12.7 mm) diameter A307 bolts through the web.

(2) The combined section shall be connected to the support by a minimum of two 1/2 in. (12.7 mm) diameter A307 bolts through the flanges.

(3) The webs of the two sections shall be in contact.

(4) The ratio of the thicker to the thinner part shall not exceed 1.3.

C3.6 Stiffeners

C3.6.1 Transverse Stiffeners

Transverse 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 according to Chapter E. For concentrated loads or reactions the nominal strength [resistance] equals P_n, where P_n is the smaller value given by (a) and (b) as follows:

(a) $P_n = F_{wy}A_c$ (*Eq.* C3.6.1-1)

(b) P_n =Nominal axial strength [resistance] evaluated according to Section C4(a), with A_e replaced by A_b

USA and Mexico		Canada
Ω_c(ASD)	ϕ_c(LRFD)	ϕ_c(LSD)
2.00	0.85	0.80

where

A_c $=18t^2 + A_s$, for transverse stiffeners at interior support and under concentrated load (*Eq.* C3.6.1-2)

A_c $=10t^2 + A_s$, for transverse stiffeners at end support (*Eq.* C3.6.1-3)

F_{wy} =Lower value of F_y for the beam web, or F_{ys} for the stiffener section

A_b $=b_1 t + A_s$, for transverse stiffeners at interior support and under concentrated load (*Eq.* C3.6.1-4)

A_b $=b_2 t + A_s$, for transverse stiffeners at end support (*Eq.* C3.6.1-5)

A_s =Cross sectional area of transverse stiffeners

b_1 $=25t\,[0.0024(L_{st}/t) + 0.72] \le 25t$ (*Eq.* C3.6.1-6)

b_2 $=12t\,[0.0044(L_{st}/t) + 0.83] \le 12t$ (*Eq.* C3.6.1-7)

L_{st} =Length of transverse stiffener

t =Base thickness of beam web

The w/t_s ratio for the stiffened and unstiffened elements of cold-formed steel transverse stiffeners shall not exceed $1.28\sqrt{E/F_{ys}}$ and $0.42\sqrt{E/F_{ys}}$, respectively, where F_{ys} is the yield stress, and t_s is the thickness of the stiffener steel.

C3.6.2 Shear Stiffeners

Where shear stiffeners are required, the spacing shall be based on the nominal shear strength [resistance], V_n, permitted by Section C3.2, and the ratio a/h shall not exceed $[260/(h/t)]^2$ nor 3.0.

The actual moment of inertia, I_s, of a pair of attached shear stiffeners, or of a single shear stiffener, with reference to an axis in the plane of the web, shall have a minimum value of

$$I_{smin} = 5ht^3[h/a - 0.7(a/h)] \ge (h/50)^4 \quad (Eq.\ C3.6.2\text{-}1)$$

The gross area of shear stiffeners shall be not less than

$$A_{st} = \frac{1-C_v}{2}\left[\frac{a}{h} - \frac{(a/h)^2}{(a/h)+\sqrt{1+(a/h)^2}}\right]YDht \quad (Eq.\ C3.6.2\text{-}2)$$

where

$$C_v = \frac{1.53Ek_v}{F_y(h/t)^2} \text{ when } C_v \le 0.8 \quad (Eq.\ C3.6.2\text{-}3)$$

$$C_v = \frac{1.11}{h/t}\sqrt{\frac{Ek_v}{F_y}} \text{ when } C_v > 0.8 \quad (Eq.\ C3.6.2\text{-}4)$$

$$k_v = 4.00 + \frac{5.34}{(a/h)^2} \text{ when } a/h \le 1.0 \quad (Eq.\ C3.6.2\text{-}5)$$

$$k_v = 5.34 + \frac{4.00}{(a/h)^2} \text{ when } a/h > 1.0 \quad (Eq.\ C3.6.2\text{-}6)$$

a =Distance between transverse stiffeners

$$Y = \frac{\text{Yield point of web steel}}{\text{Yield point of stiffener steel}}$$

D =1.0 for stiffeners furnished in pairs
D =1.8 for single-angle stiffeners
D =2.4 for single-plate stiffeners
t and h are as defined in Section B1.2

C3.6.3 Non-Conforming Stiffeners

The design strength [factored resistance] of members with transverse stiffeners that do not meet the requirements of Section C3.6.1 or C3.6.2, such as stamped or rolled-in transverse stiffeners, shall be determined by tests in accordance with Chapter F or rational engineering analysis in accordance with A1.1(b).

C4 Concentrically Loaded Compression Members

This section applies to members in which the resultant of all loads acting on the member is an axial load passing through the centroid of the effective section calculated at the stress, F_n, defined in this section.

(a) The nominal axial strength [compressive resistance], P_n, shall be calculated as follows:

$$P_n = A_eF_n \tag{Eq. C4-1}$$

USA and Mexico		Canada
Ω_c(ASD)	ϕ_c(LRFD)	ϕ_c(LSD)
1.80	0.85	0.80

where

A_e = Effective area at the stress F_n. For sections with circular holes, A_e shall be determined according to Section B2.2(a), subject to the limitations of that section. If the number of holes in the effective length region times the hole diameter divided by the effective length does not exceed 0.015, A_e can be determined ignoring the holes.

F_n is determined as follows:

For $\lambda_c \le 1.5$

$$F_n = \left(0.658^{\lambda_c^2}\right)F_y \tag{Eq. C4-2}$$

For $\lambda_c > 1.5$

$$F_n = \left[\frac{0.877}{\lambda_c^2}\right]F_y \tag{Eq. C4-3}$$

where

$$\lambda_c = \sqrt{\frac{F_y}{F_e}} \tag{Eq. C4-4}$$

F_e =The least of the elastic flexural, torsional and torsional-flexural buckling stress determined according to Sections C4.1 through C4.4.

(b) Concentrically loaded angle sections shall be designed for an additional bending moment as specified in the definitions of M_x, M_y (ASD) or M_{*x}, M_{*y} (LRFD or LSD) in Section C5.2.

C4.1 Sections Not Subject to Torsional or Torsional-Flexural Buckling

For doubly-symmetric sections, closed cross sections and any other sections which can be shown not to be subject to torsional or torsional-flexural buckling, the elastic flexural buckling stress, F_e, shall be determined as follows:

$$F_e = \frac{\pi^2 E}{(KL / r)^2} \qquad (Eq.\ C4.1\text{-}1)$$

where

E = Modulus of elasticity
K = Effective length factor
L = Laterally unbraced length of member
r = Radius of gyration of the full, unreduced cross section about the axis of buckling

In frames where lateral stability is provided by diagonal bracing, shear walls, attachment to an adjacent structure having adequate lateral stability, or floor slabs or roof decks secured horizontally by walls or bracing systems parallel to the plane of the frame, and in trusses, the effective length factor, K, for compression members which do not depend upon their own bending stiffness for lateral stability of the frame or truss, shall be taken as unity, unless analysis shows that a smaller value may used. In a frame which depends upon its own bending stiffness for lateral stability, the effective length, KL, of the compression members shall be determined by a rational method and shall not be less than the actual unbraced length.

C4.2 Doubly- or Singly-Symmetric Sections Subject to Torsional or Torsional-Flexural Buckling

For singly-symmetric sections subject to torsional-flexural buckling, F_e shall be taken as the smaller of F_e calculated according to Section C4.1 and F_e calculated as follows:

$$F_e = \frac{1}{2\beta}\left[(\sigma_{ex} + \sigma_t) - \sqrt{(\sigma_{ex} + \sigma_t)^2 - 4\beta\sigma_{ex}\sigma_t}\right] \qquad (Eq.\ C4.2\text{-}1)$$

Alternatively, a conservative estimate of F_e can be obtained using the following equation:

$$F_e = \frac{\sigma_t \sigma_{ex}}{\sigma_t + \sigma_{ex}} \qquad (Eq.\ C4.2\text{-}2)$$

where σ_t and σ_{ex} are as defined in Section C3.1.2.1:

$$\beta = 1 - (x_o / r_o)^2 \qquad (Eq.\ C4.2\text{-}3)$$

For singly-symmetric sections, the x-axis is assumed to be the axis of symmetry.

For doubly-symmetric sections subject to torsional buckling, F_e shall be taken as the smaller of F_e calculated according to Section C4.1 and $F_e=\sigma_t$, where σ_t is defined in Section C3.1.2.1.

For singly-symmetric unstiffened angle sections for which the effective area (A_e) at stress F_y is equal to the full unreduced cross-sectional area (A), then F_e shall be computed using Eq C4.1-1 where r is the least radius of gyration thus ignoring torsional-flexural buckling.

C4.3 Point-Symmetric Sections

For point-symmetric sections, F_e shall be taken as the lesser of σ_t as defined in Section C3.1.2.1 and F_e as calculated in Section C4.1 using the minor principal axis of the section.

C4.4 Nonsymmetric Sections

For shapes whose cross sections do not have any symmetry, either about an axis or about a point, F_e shall be determined by rational analysis. Alternatively, compression members composed of such shapes may be tested in accordance with Chapter F.

C4.5 Built-Up Members

For compression members composed of two sections in contact, the nominal axial strength [compressive resistance] shall be determined in accordance with Section C4(a) subject to the following modification. If the buckling mode involves relative deformations that produce shear forces in the connectors between individual shapes, KL/r is replaced by $(KL/r)_m$ determined as follows:

$$\left(\frac{KL}{r}\right)_m = \sqrt{\left(\frac{KL}{r}\right)_o^2 + \left(\frac{a}{r_i}\right)^2} \qquad (Eq.\ C4.5\text{-}1)$$

where:

$(KL/r)_o$ = Overall slenderness ratio of entire section about the built-up member axis

a = Intermediate fastener or spot weld spacing

r_i = Minimum radius of gyration of full unreduced cross-sectional area of an individual shape in a built-up member

Other symbols are defined in C4.1.

In addition, the fastener strength [resistance] and spacing shall satisfy the following:

(1) The intermediate fastener or spot weld spacing, a, shall be limited such that a/r_i does not exceed one half the governing slenderness ratio of the

built-up member.

(2) The ends of a built-up compression member shall be connected by a weld having a length not less than the maximum width of the member or by connectors spaced longitudinally not more than 4 diameters apart for a distance equal to 1.5 times the maximum width of the member.

(3) Each discrete connector shall be capable of transmitting a longitudinal shear force of 2.5% of the total force (unfactored force for ASD and factored force for LRFD and LSD) in the built-up member.

C4.6 Compression Members Having One Flange Through-Fastened to Deck or Sheathing

These provisions are applicable 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 as follows:

(a) For weak axis nominal strength [resistance]

$$P_n = C_1 C_2 C_3 AE / 29500 \quad \text{kips (Newtons)} \qquad (Eq.\ C4.6\text{-}1)$$

USA and Mexico		Canada
Ω(ASD)	ϕ(LRFD)	ϕ(LSD)
1.80	0.85	0.80

where:

$$C_1 = (0.79x + 0.54) \qquad (Eq.\ C4.6\text{-}2)$$

$$C_2 = (1.17\alpha t + 0.93) \qquad (Eq.\ C4.6\text{-}3)$$

$$C_3 = \alpha(2.5b - 1.63d) + 22.8 \qquad (Eq.\ C4.6\text{-}4)$$

For Z-sections:

x = The fastener distance from the outside web edge divided by the flange width, as shown in Figure C4.6.

For C-sections:

x = the flange width minus the fastener distance from the outside web edge divided by the flange width, as shown in Figure C4.6.

t = C- or Z-section thickness

b = C- or Z-section flange width

d = C- or Z-section depth

A = The full unreduced cross-sectional area of the C- or Z-section

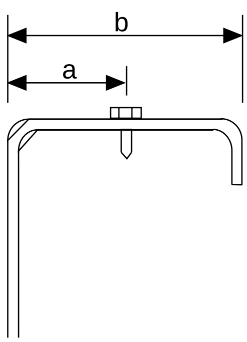
E = Modulus of elasticity of steel
= 29,500 ksi for U.S. customary units
= 203,000 MPa for SI units
= 2,070,000 kg/cm^2 for MKS units

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

Eq. C4.6-1 shall be limited to roof and wall systems meeting the following conditions:

(1) t not exceeding 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) $2.8 \le d/b < 5$
(6) $16 \le$ flange flat width / $t < 50$
(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 (10,300N/m/m) (fastener at mid-flange width) as determined by the AISI test procedure*
(9) C- and Z-sections having a minimum yield point of 33 ksi (230 MPa, or 2320 kg/cm^2)
(10) Span length not exceeding 33 feet (10 m)

(b) For strong axis nominal strength [resistance], the equations contained in Sections C4 and C4.1 of the *Specification* shall be used.



For Z-Section $x = \dfrac{a}{b}$ (*Eq*. C4.6-7)

For C-Section $x = \dfrac{b-a}{b}$ (*Eq*. C4.6-8)

Figure C4.6 Definition of x

Note:

* Further information on the test procedure should be obtained from "Rotational-Lateral Stiffness Test Method for Beam-to-Panel Assemblies", AISI *Cold-Formed Steel Design Manual*, Part VIII.

C5 Combined Axial Load and Bending

C5.1 Combined Tensile Axial Load and Bending

C5.1.1 ASD Method

The required allowable strengths T, M_x, and M_y shall satisfy the following interaction equations:

$$\frac{\Omega_b M_x}{M_{nxt}} + \frac{\Omega_b M_y}{M_{nyt}} + \frac{\Omega_t T}{T_n} \leq 1.0 \qquad (Eq.\ C5.1.1\text{-}1)$$

and

$$\frac{\Omega_b M_x}{M_{nx}} + \frac{\Omega_b M_y}{M_{ny}} - \frac{\Omega_t T}{T_n} \leq 1.0 \qquad (Eq.\ C5.1.1\text{-}2)$$

where

T = Required allowable tensile axial strength

M_x, M_y = Required allowable flexural strengths with respect to the centroidal axes of the section

T_n = Nominal tensile axial strength determined in accordance with Section C2

M_{nx}, M_{ny} = Nominal flexural strengths about the centroidal axes determined in accordance with Section C3

M_{nxt}, M_{nyt} = $S_{ft}F_y$

S_{ft} = Section modulus of the full section for the extreme tension fiber about the appropriate axis

Ω_b = 1.67 for bending strength (Section C3.1.1) or for laterally unbraced beams (Section C3.1.2)

Ω_t = 1.67

C5.1.2 LRFD and LSD Methods

The required strengths [factored tension and moments] T_*, M_{*x}, and M_{*y} shall satisfy the following interaction equations:

$$\frac{M_{*x}}{\phi_b M_{nxt}} + \frac{M_{*y}}{\phi_b M_{nyt}} + \frac{T_*}{\phi_t T_n} \leq 1.0 \qquad (Eq.\ C5.1.2\text{-}1)$$

$$\frac{M_{*x}}{\phi_b M_{nx}} + \frac{M_{*y}}{\phi_b M_{ny}} - \frac{T_*}{\phi_t T_n} \leq 1.0 \qquad (Eq.\ C5.1.2\text{-}2)$$

where

T_* = Required tensile axial strength [factored tension]

$T_* = T_u$ (LRFD)

$T_* = T_f$ (LSD)

M_{*x}, M_{*y} = Required flexural strengths [factored moments] with respect to the centroidal axes.

$M_{*x} = M_{ux}$, $M_{*y} = M_{uy}$ (LRFD)

$M_{*x} = M_{fx}$, $M_{*y} = M_{fy}$ (LSD)

T_n = Nominal axial strength determined in accordance with Section C2

M_{nx}, M_{ny} = Nominal flexural strengths about the centroidal axes determined in accordance with Section C3.1

M_{nxt}, M_{nyt} = $S_{ft}F_y$

S_{ft} = Section modulus of the full section for the extreme tension fiber about the appropriate axis

ϕ_b = For bending strength [resistance] (Section C3.1.1), ϕ_b = 0.90 or 0.95 (LRFD) and 0.90 (LSD). For laterally unbraced beams (Section C3.1.2), ϕ_b = 0.90 (LRFD and LSD)

ϕ_t = 0.95 (LRFD)
= 0.90 (LSD)

C5.2 Combined Compressive Axial Load and Bending

C5.2.1 ASD Method

The required allowable strengths P, M_x, and M_y shall satisfy the following interaction equations. In addition, each individual ratio in Eqs. C5.2.1-1 to C5.2.1-3 shall not exceed unity.

$$\frac{\Omega_c P}{P_n} + \frac{\Omega_b C_{mx} M_x}{M_{nx}\alpha_x} + \frac{\Omega_b C_{my} M_y}{M_{ny}\alpha_y} \leq 1.0 \qquad (Eq.\ C5.2.1\text{-}1)$$

$$\frac{\Omega_c P}{P_{no}} + \frac{\Omega_b M_x}{M_{nx}} + \frac{\Omega_b M_y}{M_{ny}} \leq 1.0 \qquad (Eq.\ C5.2.1\text{-}2)$$

When $\Omega_c P / P_n \leq 0.15$, the following equation may be used in lieu of the above two equations:

$$\frac{\Omega_c P}{P_n} + \frac{\Omega_b M_x}{M_{nx}} + \frac{\Omega_b M_y}{M_{ny}} \leq 1.0 \qquad (Eq.\ C5.2.1\text{-}3)$$

where

P = Required allowable compressive axial strength

M_x, M_y = Required allowable flexural strengths with respect to the centroidal axes of the effective section determined for the required compressive axial strength alone. For singly-symmetric unstiffened angle sections with unreduced effective area, M_y shall be permitted to be taken as the required flexural strength only. For other angle sections or singly-symmetric unstiffened angles for which the effective area (A_e) at stress F_y is less than the full unreduced cross-sectional area (A), M_y shall be taken either as the required flexural strength or the required flexural strength plus PL / 1000, whichever results in a lower permissible value of P.

P_n = Nominal axial strength determined in accordance with Section C4

P_{no} = Nominal axial strength determined in accordance with Section C4, with $F_n = F_y$

M_{nx}, M_{ny} = Nominal flexural strengths about the centroidal axes determined in accordance with Section C3.1

$$\alpha_x = 1 - \frac{\Omega_c P}{P_{Ex}} \quad \text{(Eq. C5.2.1-4)}$$

$$\alpha_y = 1 - \frac{\Omega_c P}{P_{Ey}} \quad \text{(Eq. C5.2.1-5)}$$

$$P_{Ex} = \frac{\pi^2 EI_x}{(K_x L_x)^2} \quad \text{(Eq. C5.2.1-6)}$$

$$P_{Ey} = \frac{\pi^2 EI_y}{(K_y L_y)^2} \quad \text{(Eq. C5.2.1-7)}$$

Ω_b = 1.67 for bending strength (Section C3.1.1) or for laterally unbraced beams (Section C3.1.2)

Ω_c = 1.80

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

I_y = Moment of inertia of the full, unreduced cross section about the y-axis

L_x = Actual unbraced length for bending about the x-axis

L_y = Actual unbraced length for bending about the y-axis

K_x = Effective length factor for buckling about the x-axis

K_y = Effective length factor for buckling about the y-axis

C_{mx}, C_{my} = Coefficients whose value shall be taken as follows:

1. For compression members in frames subject to joint translation (sidesway)
   $C_m = 0.85$
2. For restrained compression members in frames braced against joint translation and not subject to transverse loading between their supports in the plane of bending
   $$C_m = 0.6 - 0.4\,(M_1/M_2) \quad \text{(Eq. C5.2.1-8)}$$
   where
   M_1/M_2 is the ratio of the smaller to the larger moment at the ends of that portion of the member under consideration which is unbraced in the plane of bending. M_1/M_2 is positive when the member is bent in reverse curvature and negative when it is bent in single curvature

3. For compression members in frames braced against joint translation in the plane of loading and subject to transverse loading between their supports, the value of C_m may be determined by rational analysis. However, in lieu of such analysis, the following values may be used:
(a) for members whose ends are restrained, $C_m = 0.85$
(b) for members whose ends are unrestrained, $C_m = 1.0$

C5.2.2 LRFD and LSD Methods

The required strengths [factored compressive force and moments] P_*, M_{*x}, and M_{*y} shall satisfy the following interaction equations. In addition, each individual ratio in Eqs. C5.2.2-1 to C5.2.2-3 shall not exceed unity.

$$\frac{P_*}{\phi_c P_n} + \frac{C_{mx} M_{*x}}{\phi_b M_{nx} \alpha_x} + \frac{C_{my} M_{*y}}{\phi_b M_{ny} \alpha_y} \le 1.0 \qquad (Eq.\ C5.2.2\text{-}1)$$

$$\frac{P_*}{\phi_c P_{no}} + \frac{M_{*x}}{\phi_b M_{nx}} + \frac{M_{*y}}{\phi_b M_{ny}} \le 1.0 \qquad (Eq.\ C5.2.2\text{-}2)$$

When $P_* / \phi_c P_n \le 0.15$, the following equation may be used in lieu of the above two equations:

$$\frac{P_*}{\phi_c P_n} + \frac{M_{*x}}{\phi_b M_{nx}} + \frac{M_{*y}}{\phi_b M_{ny}} \le 1.0 \qquad (Eq.\ C5.2.2\text{-}3)$$

where

P_* = Required compressive axial strength [factored compressive force]
$P_* = P_u$ (LRFD)
$P_* = P_f$ (LSD)

M_{*x}, M_{*y} = Required flexural strengths [factored moments] with respect to the centroidal axes of the effective section determined for the required compressive axial strength alone. For singly-symmetric unstiffened angle sections with un-reduced effective area, M_{*y} shall be permitted to be taken as the required flexural strength [factored moment] only. For other angle sections or singly-symmetric unstiffened angles for which the effective area (A_e) at stress F_y is less than the full unreduced cross-sectional area (A), M_{*y}, shall be taken either as the required flexural strength [factored moment] or the required flexural strength [factored moment] plus

$(P_*)L/1000$, whichever results in a lower permissible value of P_*.

$M_{*x} = M_{ux}$, $M_{*y} = M_{uy}$ (LRFD)
$M_{*x} = M_{fx}$, $M_{*y} = M_{fy}$ (LSD)

P_n = Nominal axial strength determined in accordance with Section C4

P_{no} = Nominal axial strength determined in accordance with Section C4, with $F_n = F_y$

M_{nx}, M_{ny} = Nominal flexural strengths about the centroidal axes determined in accordance with Section C3

$$\alpha_x = 1 - \frac{P_u}{P_{Ex}} \quad (Eq.\ C5.2.2\text{-}4)$$

$$\alpha_y = 1 - \frac{P_u}{P_{Ey}} \quad (Eq.\ C5.2.2\text{-}5)$$

$$P_{Ex} = \frac{\pi^2 EI_x}{(K_x L_x)^2} \quad (Eq.\ C5.2.2\text{-}6)$$

$$P_{Ey} = \frac{\pi^2 EI_y}{(K_y L_y)^2} \quad (Eq.\ C5.2.2\text{-}7)$$

ϕ_b = For bending strength [resistance] (Section C3.1.1), ϕ_b = 0.90 or 0.95 (LRFD) and 0.90 (LSD). For laterally unbraced beams (Section C3.1.2), ϕ_b = 0.90 (LRFD and LSD)

ϕ_c = 0.85 (LRFD)
= 0.80 (LSD)

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

I_y = Moment of inertia of the full, unreduced cross section about the y-axis

L_x = Actual unbraced length for bending about the x-axis

L_y = Actual unbraced length for bending about the y-axis

K_x = Effective length factor for buckling about the x-axis

K_y = Effective length factor for buckling about the y-axis

C_{mx}, C_{my} = Coefficients whose value shall be taken as follows:

1. For compression members in frames subject to joint translation (sidesway)
   $C_m = 0.85$
2. For restrained compression members in frames braced against joint translation and not subject to transverse loading between their supports in the plane of bending

$C_m = 0.6 - 0.4\,(M_1/M_2)$ (*Eq.* C5.2.2-8)

where

M_1/M_2 is the ratio of the smaller to the larger moment at the ends of that portion of the member under consideration which is unbraced in the plane of bending. M_1/M_2 is positive when the member is bent in reverse curvature and negative when it is bent in single curvature.

3. For compression members in frames braced against joint translation in the plane of loading and subject to transverse loading between their supports, the value of C_m may be determined by rational analysis. However, in lieu of such analysis, the following values may be used:
   (a) for members whose ends are restrained, $C_m = 0.85$,
   (b) for members whose ends are unrestrained, $C_m = 1.0$.

C6 Closed Cylindrical Tubular Members

The requirements of this Section apply to closed cylindrical tubular members having a ratio of outside diameter to wall thickness, D/t, not greater than $0.441\,E/F_y$.

C6.1 Bending

For flexural members, the nominal flexural strength [moment resistance], M_n, shall be calculated as follows:

$M_n = F_c S_f$ (*Eq.* C6.1-1)

USA and Mexico		Canada
Ω_b(ASD)	ϕ_b(LRFD)	ϕ_b(LSD)
1.67	0.95	0.90

For $D/t \le 0.0714\,E/F_y$

$F_c = 1.25\,F_y$ (*Eq.* C6.1-2)

For $0.0714\,E/F_y < D/t \le 0.318\,E/F_y$

$$F_c = \left[0.970 + 0.020\left(\frac{E/F_y}{D/t}\right)\right]F_y \quad (Eq.\ C6.1\text{-}3)$$

For $0.318\,E/F_y < D/t \le 0.441\,E/F_y$

$F_c = 0.328E/(D/t)$ (*Eq.* C6.1-4)

where

F_c = Critical flexural stress

S_f = Elastic section modulus of the full, unreduced cross section

C6.2 Compression

The requirements of this Section apply to members in which the resultant of all loads and moments acting on the member is equivalent to a single force in the direction of the member axis passing through the centroid of the section.

The nominal axial strength [compressive resistance], P_n, shall be calculated as follows:

$$P_n = F_n A_e \qquad (Eq.\ C6.2\text{-}1)$$

USA and Mexico		Canada
Ω_c(ASD)	ϕ_c(LRFD)	ϕ_c(LSD)
1.80	0.85	0.80

F_n is determined as follows:

For $\lambda_c \leq 1.5$

$$F_n = \left(0.658^{\lambda_c^2}\right) F_y \qquad (Eq.\ C6.2\text{-}2)$$

For $\lambda_c > 1.5$

$$F_n = \left[\frac{0.877}{\lambda_c^2}\right] F_y \qquad (Eq.\ C6.2\text{-}3)$$

where

$$\lambda_c = \sqrt{\frac{F_y}{F_e}} \qquad (Eq.\ C6.2\text{-}4)$$

In the above equations:

F_e = The elastic flexural buckling stress determined according to Section C4.1

$A_e = A_o + R(A - A_o)$ (*Eq.* C6.2–5)

$R = F_y / 2F_e \leq 1.0$ (*Eq.* C6.2–6)

$$A_o = \left[\frac{0.037}{(DF_y)/(tE)} + 0.667\right] A \leq A \quad \text{for } \frac{D}{t} \leq 0.441 \frac{E}{F_y} \qquad (Eq.\ C6.2\text{-}7)$$

A = Area of the unreduced cross section

C6.3 Combined Bending and Compression

Combined bending and compression shall satisfy the provisions of Section C5.

D. STRUCTURAL ASSEMBLIES

D1 Built-Up Sections

D1.1 I-Sections Composed of Two C-Sections

(a) For compression members:

Refer to Section C4.5.

(b) For flexural members:

The maximum permissible longitudinal spacing of welds or other connectors, s_{max}, joining two C-sections to form an I-section shall be:

$$s_{max} = L / 6 \leq \frac{2gT_s}{mq} \qquad (Eq.\ D1.1\text{-}1)$$

where

L =Span of beam

T_s =Design strength [factored resistance] of connection in tension (Chapter E)

g =Vertical distance between the two rows of connections nearest to the top and bottom flanges

q =Design load on the beam for spacing of connectors (Use nominal loads for ASD, factored loads for LRFD and LSD. For methods of determination, see below)

m =Distance from the shear center of one C-section to the mid-plane of its web.

The load, q, is 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 equal to three times the uniformly distributed load, based on nominal loads for ASD, factored loads for LRFD and LSD. If the length of bearing of a concentrated load or reaction is smaller than the weld spacing, s, the required design strength [factored resistance] of the welds or connections closest to the load or reaction is

$$T_s = P_s m / 2g \qquad (Eq.\ D1.1\text{-}2)$$

where P_s is a concentrated load or reaction based on nominal loads for ASD, factored loads for LRFD and LSD.

The allowable maximum spacing of connections, s_{max}, depends 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 may be adopted: (a) the connection spacing may be varied along the beam according to the variation of the load intensity; or (b) reinforcing cover plates may be welded to the flanges at points where concentrated loads occur. The design shear strength of the connections joining these plates to the flanges shall then be used for T_s, and g shall be taken as the depth of the beam.

D1.2 Spacing of Connections in Compression Elements

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) that which is required to transmit the shear between the connected parts on the basis of the design strength [factored resistance] per connection specified elsewhere herein; nor

(b) $1.16t\sqrt{E/f_c}$, where t is the thickness of the cover plate or sheet, and f_c is the stress at service load in the cover plate or sheet; nor

(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 one-half inch (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 which act only as sheathing material and are not considered as load-carrying elements.

D2 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.

D3 Lateral Bracing

Braces shall be designed to restrain lateral bending or twisting of a loaded beam or column, and to avoid local crippling at the points of attachment.

D3.1 Symmetrical Beams and Columns

Braces and bracing systems, including connections, shall be designed considering strength and stiffness requirements.

D3.2 C-Section and Z-Section Beams

The following provisions for bracing to restrain twisting of C-sections and Z-sections used as beams loaded in the plane of the web, apply only when (a) the top flange is connected to deck or sheathing material in such a manner as to effectively restrain lateral deflection of the connected flange, or (b) neither flange is so connected. When both flanges are so connected, no further bracing is required. When the *Specification* does not provide an explicit method for design, further information should be obtained from the *Commentary*.

D3.2.1 Anchorage of Bracing for Roof Systems Under Gravity Load With Top Flange Connected to Sheathing

For C-sections and Z-sections designed according to Section C3.1.1, and having deck or sheathing fastened to the top flanges (through fastened or standing seam systems), provisions shall be made to restrain the flanges so that the maximum top flange lateral displacements with respect to the purlin reaction points do not exceed the span length divided by 360. If the top flanges of all purlins face in the same direction, anchorage of the restraint shall satisfy the requirements of Sections D3.2.1(a) and D3.2.1(b). If the top flanges of adjacent lines of purlins face in opposite directions, a restraint system shall be provided to resist the down-slope component of the total gravity load.

Anchored braces need to be connected to only one line of purlins in each purlin bay of each roof slope if provision is made to transmit forces from other purlin lines through the roof deck and its fastening system. Anchored braces shall be as close as possible to the flange which is connected to the deck or sheathing. Anchored braces shall be provided for each purlin bay.

For bracing arrangements other than those covered in Sections D3.2.1(a) and D3.2.1(b), tests in accordance with Chapter F shall be performed so that the type and/or spacing of braces selected are such that the test strength [resistance] of the purlin assembly is equal to or greater than its nominal flexural strength [moment resistance], instead of that required by Chapter F.

(a) C-Sections

For roof systems using C-sections for purlins with all compression flanges facing in the same direction, a system possessing restraint force, P_L, in addition to resisting other loading, shall be provided:

$$P_L = (0.05\alpha\cos\theta - \sin\theta)W \qquad (Eq.\ D.3.2.1\text{-}1)$$

where

W = Total vertical load (nominal load for ASD, factored load for LRFD and LSD) supported by all purlin lines being restrained. Where more than one brace is used at a purlin line, the restraint force P_L shall be divided equally between all braces.

α = +1 for purlin facing upward direction, and -1 for purlin facing down slope direction.

θ = Angle between the vertical and the plane of the web of the C-section, degrees.

A positive value for the force, P_L, means that restraint is required to prevent movement of the purlin flanges in the upward roof slope direction, and a negative value means that restraint is required to prevent movement of purlin flanges in the downward slope direction.

(b) Z-Sections

For roof systems having four to twenty Z-purlin lines with all top flanges facing in the direction of the upward roof slope, and with restraint braces at the purlin supports, midspan or one-third points, each brace shall be designed to resist a force determined as follows:

(1) Single-Span System with Restraints at the Supports:

$$P_L = 0.5\left[\frac{0.220b^{1.50}}{n_p^{0.72}d^{0.90}t^{0.60}}\cos\theta - \sin\theta\right]W \qquad (Eq.\ D3.2.1\text{-}2)$$

(2) Single-Span System with Third-Point Restraints:

$$P_L = 0.5\left[\frac{0.474b^{1.22}}{n_p^{0.57}d^{0.89}t^{0.33}}\cos\theta - \sin\theta\right]W \qquad (Eq.\ D3.2.1\text{-}3)$$

(3) Single-Span System with Midspan Restraint:

$$P_L = \left[\frac{0.224b^{1.32}}{n_p^{0.65}d^{0.83}t^{0.50}}\cos\theta - \sin\theta\right]W \qquad (Eq.\ D3.2.1\text{-}4)$$

(4) Multiple-Span System with Restraints at the Supports:

$$P_L = C_{tr}\left[\frac{0.053b^{1.88}L^{0.13}}{n_p^{0.95}d^{1.07}t^{0.94}}\cos\theta - \sin\theta\right]W \qquad (Eq.\ D.3.2.1\text{-}5)$$

With

C_{tr} =0.63 for braces at end supports of multiple-span systems

C_{tr} = 0.87 for braces at the first interior supports

C_{tr} = 0.81 for all other braces

(5) Multiple-Span System with Third-Point Restraints:

$$P_L = C_{th}\left[\frac{0.181b^{1.15}L^{0.25}}{n_p^{0.54}d^{1.11}t^{0.29}}\cos\theta - \sin\theta\right]W \qquad (Eq.\ D3.2.1\text{-}6)$$

With

C_{th} =0.57 for outer braces in exterior spans

C_{th} =0.48 for all other braces

(6) Multiple-Span System with Midspan Restraints:

$$P_L = C_{ms}\left[\frac{0.116b^{1.32}L^{0.18}}{n_p^{0.70}dt^{0.50}}\cos\theta - \sin\theta\right]W \qquad (Eq.\ D3.2.1\text{-}7)$$

with

C_{ms} =1.05 for braces in exterior spans

C_{ms} =0.90 for all other braces

where

b =Flange width

d =Depth of section

t =Thickness
L =Span length
θ =Angle between the vertical and the plane of the web of the Z-section, degrees
n_p =Number of parallel purlin lines
W =Total vertical load supported by the purlin lines between adjacent supports (Use nominal loads for ASD, factored loads for LRFD and LSD)

The force, P_L, is positive when restraint is required to prevent movement of the purlin flanges in the upward roof slope direction.

For systems having less than four purlin lines, the brace force shall be determined by taking 1.1 times the force found from Equations D3.2.1-2 through D3.2.1-7, with n_p = 4. For systems having more than twenty purlin lines, the brace force shall be determined from Equations D3.2.1-2 through D3.2.1-7, with n_p = 20 and W based on the total number of purlins.

D3.2.2 Neither Flange Connected to Sheathing

Each intermediate brace, at the top and bottom flange, shall be designed to resist a required lateral force, P_L, determined as follows:

(a) For uniform loads, P_L = 1.5K′ times the design load (nominal loads for ASD, factored loads for LRFD and LSD) within a distance 0.5a each side of the brace.

(b) For concentrated loads, P_L = 1.0K′ times each design concentrated load within a distance 0.3a each side of the brace, plus 1.4K′ (1-x/a) times each design concentrated load located farther than 0.3a but not farther than 1.0a from the brace. The design concentrated load is the nominal load for ASD or the factored load for LRFD and LSD.

In the above equations:

For C-sections:

$$K' = m/d \qquad (Eq.\ D3.2.2\text{-}1)$$

where
m =Distance from the shear center to the mid-plane of the web
d =Depth of C-section

Brace force P_L, shall be applied to both flanges in opposite directions in order to resist the twist caused by the load.

For Z-sections:

$$K' = I_{xy}/(2I_x) \qquad (Eq.\ D3.2.2\text{-}2)$$

where
I_{xy} =Product of inertia of the full section about centroidal axes parallel and perpendicular to the web
I_x =Moment of inertia of the full section about the centroidal axis perpendicular to the web

Brace force, P_L, shall be applied to both flanges in the same direction in order to constrain bending of the section about the axis perpendicular to its web.

For C-sections and Z-sections:

x =Distance from the concentrated load to the brace

a =Distance between center line of braces

When braces are provided, they shall be attached in such a manner 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 which 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 according to Section C3.1.2.

D4 Wall Studs and Wall Stud Assemblies

Wall studs shall be designed either on the basis of an all steel system in accordance with Chapter C or on the basis of sheathing in accordance with Section D4.1 through D4.3. Both solid and perforated webs shall be permitted. Both ends of the stud shall be connected to restrain rotation about the longitudinal stud axis and horizontal displacement perpendicular to the stud axis.

(a) All Steel Design

Wall stud assemblies using an all steel design shall be designed neglecting the structural contribution of the attached sheathings and shall comply with the requirements of Chapter C. For compression members with circular web perforations, see Section B2.2, and for non-circular web perforations, the effective area shall be determined as follows:

The effective area, A_e at a stress F_n, shall be determined in accordance with Chapter B, assuming the web to consist of two unstiffened elements, one on each side of the perforation, or the effective area, A_e, shall be determined from stub-column tests.

When A_e is determined in accordance with Chapter B, the following limitations related to the size and spacing of perforations and the depth of the stud shall apply:

(1) The center-to-center spacing of web perforations shall not be less than 24 in. (610 mm).
(2) The maximum width of web perforations shall be the lesser of 0.5 times the depth, d, of the section or 2-1/2 in. (63.5 mm).
(3) The length of web perforations shall not exceed 4-1/2 in. (114 mm).
(4) The section depth-to-thickness ratio, d/t, shall not be less than 20.
(5) The distance between the end of the stud and the near edge of a perforation shall not be less than 10 in. (254 mm).

(b) Sheathing Braced Design

Wall stud assemblies using a sheathing braced design shall be designed in accordance with Sections D4.1 through D4.3 and in addition shall comply with the following requirements:

In the case of perforated webs, the effective area, A_e, shall be determined as in (a) above.

Sheathing shall be attached to both sides of the stud and connected to the bottom and top horizontal members of the wall to provide lateral and torsional support to the stud in the plane of the wall.

Sheathing shall conform to the limitations specified under Table D4. Additional bracing shall be provided during construction, if required.

D4.1 Wall Studs in Compression

For studs having identical sheathing attached to both flanges, and neglecting any rotational restraint provided by the sheathing, the nominal axial strength [compressive resistance], P_n, shall be calculated as follows:

$$P_n = A_e F_n \qquad (Eq.\ D4.1\text{-}1)$$

USA and Mexico		Canada
Ω_c(ASD)	ϕ_c(LRFD)	ϕ_c(LSD)
1.80	0.85	0.80

where

A_e = Effective area determined at F_n

F_n = The lowest value determined by three conditions (a), (b), and (c) given below. The equations provided in these three conditions are applicable within the following limits:

Yield strength, $F_y \leq$ 50 ksi (340 MPa or 3520 kg/cm^2)
Section depth, $d \leq 6.0$ in. (152 mm)
Section thickness, $t \leq 0.075$ in. (1.91 mm)
Overall length, $L \leq 16$ ft. (4.88 m)
Stud spacing, 12 in. (305 mm) minimum; 24 in. (610 mm) maximum
Fastener spacing, 6 in. (152 mm) $\leq s \leq$ 12 in. (305 mm)

(a) To prevent column buckling between fasteners in the plane of the wall, F_n shall be calculated according to Section C4 with KL equal to two times the distance between fasteners.

(b) To prevent flexural and/or torsional overall column buckling, F_n shall be calculated in accordance with Section C4 with F_e taken as the smaller of the two σ_{CR} values specified for the following section types, where σ_{CR} is the theoretical elastic buckling stress under concentric loading.

(1) Singly-symmetric C-Sections

$$\sigma_{CR} = \sigma_{ey} + \overline{Q}_a \qquad (Eq.\ D4.1\text{-}2)$$

$$\sigma_{CR} = \frac{1}{2\beta}\left[(\sigma_{ex} + \sigma_{tQ}) - \sqrt{(\sigma_{ex} + \sigma_{tQ})^2 - 4\beta\sigma_{ex}\sigma_{tQ}}\right] \qquad (Eq.\ D4.1\text{-}3)$$

(2) Z-Sections

$$\sigma_{CR} = \sigma_t + \overline{Q}_t \quad (Eq.\ D4.1\text{-}4)$$

$$\sigma_{CR} = \frac{1}{2}\left\{(\sigma_{ex} + \sigma_{ey} + \overline{Q}_a) - \sqrt{[(\sigma_{ex} + \sigma_{ey} + \overline{Q}_a)^2 - 4(\sigma_{ex}\sigma_{ey} + \sigma_{ex}\overline{Q}_a - \sigma_{exy}^2)]}\right\} \quad (Eq.\ D4.1\text{-}5)$$

(3) I-Sections (doubly-symmetric)

$$\sigma_{CR} = \sigma_{ey} + \overline{Q}_a \quad (Eq.\ D4.1\text{-}6)$$

$$\sigma_{CR} = \sigma_{ex} \quad (Eq.\ D4.1\text{-}7)$$

In the above equations:

$$\sigma_{ex} = \frac{\pi^2 E}{(L / r_x)^2} \quad (Eq.\ D4.1\text{-}8)$$

$$\sigma_{exy} = (\pi^2 EI_{xy}) / (AL^2) \quad (Eq.\ D4.1\text{-}9)$$

$$\sigma_{ey} = \frac{\pi^2 E}{(L / r_y)^2} \quad (Eq.\ D4.1\text{-}10)$$

$$\sigma_t = \frac{1}{Ar_o^2}\left[GJ + \frac{\pi^2 EC_w}{L^2}\right] \quad (Eq.\ D4.1\text{-}11)$$

$$\sigma_{tQ} = \sigma_t + \overline{Q}_t \quad (Eq.\ D4.1\text{-}12)$$

$$\overline{Q} = \overline{Q}_o (2 - s / s') \quad (Eq.\ D4.1\text{-}13)$$

where:

s = fastener spacing, in. (mm)

s' = 12 in. (305 mm)

$\overline{Q}_o$ = See Table D4

$$\overline{Q}_a = \overline{Q} / A \quad (Eq.\ D4.1\text{-}14)$$

A = Area of full unreduced cross section

L = Length of stud

$$\overline{Q}_t = (\overline{Q}d^2) / (4Ar_o^2) \quad (Eq.\ D4.1\text{-}15)$$

d = Depth of section

I_{xy} = Product of inertia

(c) To prevent shear failure of the sheathing, a value of F_n shall be used in the following equations so that the shear strain of the sheathing, γ, does not exceed the permissible shear strain, $\overline{\gamma}$. The shear strain, γ, shall be determined as follows:

$$\gamma = (\pi / L)\,[C_1 + (E_1\, d / 2)] \quad (Eq.\ D4.1\text{-}16)$$

where

C_1 and E_1 are the absolute values of C_1 and E_1 specified below for each section type:

(1) Singly-Symmetric C-sections

$$C_1 = (F_n\, C_o) / (\sigma_{ey} - F_n + \overline{Q}_a) \quad (Eq.\ D4.1\text{-}17)$$

$$E_1 = \frac{F_n[(\sigma_{ex} - F_n)(r_o^2 E_o - x_o D_o) - F_n x_o (D_o - x_o E_o)]}{(\sigma_{ex} - F_n) r_o^2 (\sigma_{tQ} - F_n) - (F_n x_o)^2} \quad (Eq.\ D4.1\text{-}18)$$

(2) Z-Sections

$$C_1 = \frac{F_n[C_o(\sigma_{ex} - F_n) - D_o \sigma_{exy}]}{(\sigma_{ey} - F_n + \overline{Q}_a)(\sigma_{ex} - F_n) - \sigma_{exy}^2} \quad (Eq.\ D4.1\text{-}19)$$

$$E_1 = (F_n E_o) / (\sigma_{tQ} - F_n) \quad (Eq.\ D4.1\text{-}20)$$

(3) I-Sections

$$C_1 = (F_n C_o) / (\sigma_{ey} - F_n + \overline{Q}_a) \quad (Eq.\ D4.1\text{-}21)$$

$$E_1 = 0$$

where

x_o = Distance from shear center to centroid along principal x-axis, (absolute value)

C_o, E_o, and D_o are initial column imperfections which shall be assumed to be at least

C_o = L / 350 in a direction parallel to the wall (*Eq.* D4.1-22)

D_o = L / 700 in a direction perpendicular to the wall (*Eq.* D4.1-23)

E_o = L / (d × 10,000), rad., a measure of the initial twist of the stud from the initial, ideal, unbuckled shape (*Eq.* D4.1-24)

If $F_n > 0.5 F_y$, then in the definitions for σ_{ey}, σ_{ex}, σ_{exy} and σ_{tQ}, the parameters E and G shall be replaced by E′ and G′, respectively, as defined below

$$E' = 4EF_n (F_y - F_n) / F_y^2 \quad (Eq.\ D4.1\text{-}25)$$

$$G' = G (E' / E) \quad (Eq.\ D4.1\text{-}26)$$

Sheathing parameters $\overline{Q}_o$ and $\overline{\gamma}$ shall be permitted to be determined from representative full-scale tests, conducted and evaluated as described by published documented methods (see *Commentary*), or from the small scale test values given in Table D4.

TABLE D4
Sheathing Parameters(1)

Sheathing(2)	$\overline{Q}_o$ kip	$\overline{Q}_o$ kN	$\overline{Q}_o$ kg	$\overline{\gamma}$ length / length
3/8 in. (9.5 mm) to 5/8 in. (15.9 mm) thick gypsum	24.0	107.0	10,900	0.008
Lignocellulosic board	12.0	53.4	5440	0.009
Fiberboard (regular or impregnated)	7.2	32.0	3270	0.007
Fiberboard (heavy impregnated)	14.4	64.1	6530	0.010

(1) The values given are subject to the following limitations:
All values are for sheathing on both sides of the wall assembly.

All fasteners are No. 6, type S-12, self-drilling drywall screws with pan or bugle head, or equivalent.

(2) All sheathing is 1/2 in. (12.7 mm) thick except as noted.

For other types of sheathing, $\overline{Q}_o$ and $\overline{\gamma}$ shall be permitted to be determined conservatively from representative small-specimen tests as described by published documented methods (see *Commentary*).

D4.2 Wall Studs in Bending

For studs having identical sheathing attached to both flanges, and neglecting any rotational restraint provided by the sheathing, the nominal flexural strengths [moment resistances] are M_{nxo} and M_{nyo} where:

For sections with stiffened or partially stiffened compression flanges:

USA and Mexico		Canada
Ω_b(ASD)	ϕ_b(LRFD)	ϕ_b(LSD)
1.67	0.95	0.90

For sections with unstiffened compression flanges:

USA and Mexico		Canada
Ω_b(ASD)	ϕ_b(LRFD)	ϕ_b(LSD)
1.67	0.90	0.90

M_{nxo} and M_{nyo} = Nominal flexural strengths [moment resistances] about the centroidal axes determined in accordance with Section C3.1, excluding the provisions of Section C3.1.2 (lateral-torsional buckling).

D4.3 Wall Studs with Combined Axial Load and Bending

The required axial strength [resistance] and flexural strength [moment resistance] shall satisfy the interaction equations of Section C5 with the following redefined terms:

P_n = Nominal axial strength [resistance] determined according to Section D4.1

M_{nx} and M_{ny} in Equations C5.2.1-1, C5.2.1-2 and C5.2.1-3 for ASD or C5.2.2-1, C5.2.2-2 and C5.2.2-3 for LRFD or LSD shall be replaced by nominal flexural strengths [moment resistances], M_{nxo} and M_{nyo}, respectively.

D5 Floor, Roof or Wall Steel Diaphragm Construction

The in-plane diaphragm nominal shear strength [resistance], S_n shall be established by calculation or test.

Ω_d =As specified in Table D5 (ASD)

ϕ_d =As specified in Table D5 (LRFD and LSD)

TABLE D5
Factors of Safety and Resistance Factors for Diaphragms

USA and Mexico		Canada	Diaphragm Condition
Ω_d (ASD)	ϕ_d (LRFD)	ϕ_d (LSD)	
2.65	0.60	0.50	for diaphragms for which the failure mode is that of buckling, otherwise;
3.0	0.50	0.50	for diaphragms welded to the structure subjected to earthquake loads, or subjected to load combinations which include earthquake loads.
2.35	0.55	0.50	for diaphragms welded to the structure subjected to wind loads, or subjected to load combinations which include wind loads
2.5	0.60	0.50	for diaphragms mechanically connected to the structure subjected to earthquake loads, or subjected to load combinations which include earthquake loads.
2.0	0.65	0.50	for diaphragms mechanically connected to the structure subjected to wind loads, or subjected to load combinations which include wind loads.
2.45	0.65	0.50	for diaphragms connected to the structure by either mechanical fastening or welding subjected to load combinations not involving wind or earthquake loads.

E. CONNECTIONS AND JOINTS

E1 General Provisions

Connections shall be designed to transmit the maximum design forces acting on the connected members. Proper regard shall be given to eccentricity.

E2 Welded Connections

The following design criteria govern welded connections used for cold-formed steel structural members in which the thickness of the thinnest connected part is 0.18 in. (4.57 mm) or less. For the design of welded connections in which the thickness of the thinnest connected part is greater than 0.18 in. (4.57 mm), refer to the specifications or standards stipulated in the corresponding Section E2a of Appendix A, B or C.

Welds shall follow the requirements of the weld standards also stipulated in Section E2a of Appendix A, B, or C.

E2.1 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 as follows:

(a) Tension or compression normal to the effective area or parallel to the axis of the weld

$$P_n = Lt_eF_y \quad (Eq.\ E2.1\text{-}1)$$

USA and Mexico		Canada
Ω(ASD)	ϕ(LRFD)	ϕ(LSD)
1.70	0.90	0.80

(b) Shear on the effective area, the smaller of either Eq. E2.1-2 or E2.1-3

$$P_n = Lt_e\,0.6F_{xx} \quad (Eq.\ E2.1\text{-}2)$$

USA and Mexico		Canada
Ω(ASD)	ϕ(LRFD)	ϕ(LSD)
1.90	0.80	0.70

$$P_n = Lt_eF_y\ /\sqrt{3} \quad (Eq.\ E2.1\text{-}3)$$

USA and Mexico		Canada
Ω(ASD)	ϕ(LRFD)	ϕ(LSD)
1.70	0.90	0.80

where

P_n = Nominal strength [resistance] of a groove weld
F_{xx} = Tensile strength of the electrode classification
F_y = Specified minimum yield point of the lowest strength base steel
L = Length of weld
t_e = Effective throat dimension for groove weld

E2.2 Arc Spot Welds

Arc spot welds permitted by this *Specification* are for welding sheet steel to thicker supporting members in the flat position. Arc spot welds (puddle welds) shall not be made on steel where the thinnest connected part is over 0.15 in. (3.81 mm) thick, nor through a combination of steel sheets having a total thickness over 0.15 in. (3.81 mm).

Weld washers, Figures E2.2A and E2.2B, shall be used when the thickness of the sheet is less than 0.028 in. (0.711 mm). Weld washers shall have a thickness between 0.05 (1.27 mm) and 0.08 in. (2.03 mm) with a minimum prepunched hole of 3/8 in. (9.53 mm) diameter.

Arc spot welds shall be specified by minimum effective diameter of fused area, d_e. Minimum allowable effective diameter is 3/8 in. (9.5 mm).

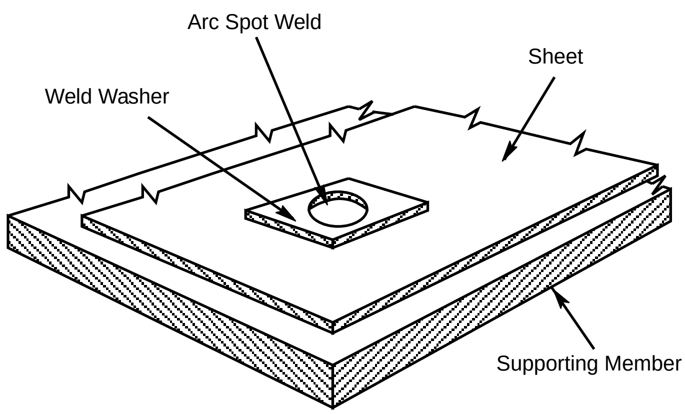


Figure E2.2A Typical Weld Washer

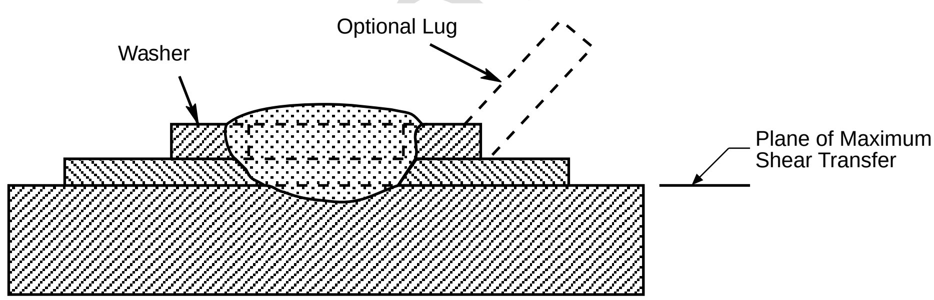


Figure E2.2B Arc Spot Weld Using Washer

E2.2.1 Shear

The nominal shear strength [resistance], P_n, of each arc spot weld between sheet or sheets and supporting member shall be determined by using the smaller of either

(a) $P_n = \dfrac{\pi d_e^2}{4} 0.75 F_{xx}$ (*Eq*. E2.2.1-1)

USA and Mexico		Canada
Ω(ASD)	ϕ(LRFD)	ϕ(LSD)
2.55	0.60	0.50

(b) For $(d_a / t) \le 0.815\sqrt{(E / F_u)}$

$$P_n = 2.20\, t\, d_a\, F_u \qquad (Eq.\ E2.2.1\text{-}2)$$

USA and Mexico		Canada
Ω(ASD)	ϕ(LRFD)	ϕ(LSD)
2.20	0.70	0.60

For $0.815\sqrt{(E / F_u)} < (d_a / t) < 1.397\sqrt{(E / F_u)}$

$$P_n = 0.280\left[1 + 5.59\frac{\sqrt{E / F_u}}{d_a / t}\right] t d_a F_u \qquad (Eq.\ E2.2.1\text{-}3)$$

USA and Mexico		Canada
Ω(ASD)	ϕ(LRFD)	ϕ(LSD)
2.80	0.55	0.45

For $(d_a / t) \ge 1.397\sqrt{(E / F_u)}$

$$P_n = 1.40\, t\, d_a\, F_u \qquad (Eq.\ E2.2.1\text{-}4)$$

USA and Mexico		Canada
Ω(ASD)	ϕ(LRFD)	ϕ(LSD)
3.05	0.50	0.40

where

P_n = Nominal shear strength [resistance] of an arc spot weld

d = Visible diameter of outer surface of arc spot weld

d_a = Average diameter of the arc spot weld at mid-thickness of t where $d_a = (d - t)$ for a single sheet and multiple sheets not more than four lapped sheets over a supporting member

d_e = Effective diameter of fused area at plane of maximum shear transfer

$= 0.7d - 1.5t$ but $\le 0.55d$ (*Eq.* E2.2.1-5)

t = Total combined base steel thickness (exclusive of coatings) of sheets involved in shear transfer above the plane of maximum shear transfer

F_{xx} = Tensile strength of the electrode classification

F_u = Tensile strength as specified in Section A2.1 or A2.2

Note: See Figures E2.2C and E2.2D for diameter definitions.

The distance measured in the line of force from the centerline of a weld to the nearest edge of an adjacent weld or to the end of the connected part toward which the force is directed shall not be less than the value of e_{min} as given below:

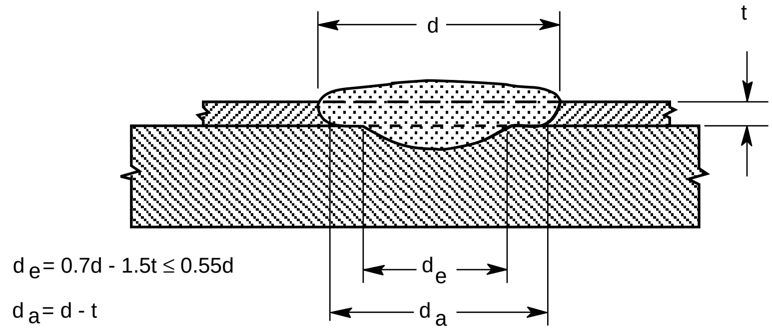


Figure E2.2C Arc Spot Weld – Single Thickness of Sheet

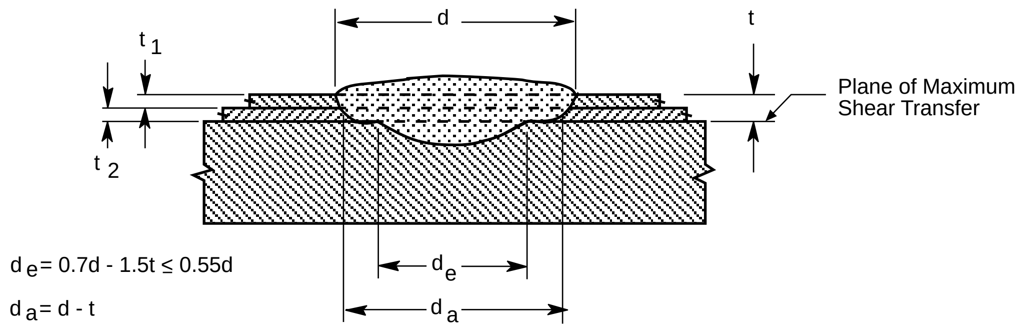


Figure E2.2D Arc Spot Weld – Double Thickness of Sheet

$$e_{min} = \frac{P\Omega}{F_u t} \quad \text{For ASD} \qquad (Eq.\ E2.2.1\text{-}6a)$$

$$e_{min} = \frac{P_u}{\phi F_u t} \quad \text{For LRFD} \qquad (Eq.\ E2.2.1\text{-}6b)$$

$$e_{min} = \frac{P_f}{\phi F_u t} \quad \text{For LSD} \qquad (Eq.\ E2.2.1\text{-}6c)$$

When $F_u / F_{sy} \geq 1.08$

USA and Mexico		Canada
Ω(ASD)	ϕ(LRFD)	ϕ(LSD)
2.20	0.70	0.60

When $F_u / F_{sy} < 1.08$

USA and Mexico		Canada
Ω(ASD)	ϕ(LRFD)	ϕ(LSD)
2.55	0.60	0.50

where

P = Required strength (nominal force) transmitted by the weld (ASD)
P_u = Required strength (factored force) transmitted by the weld (LRFD)
P_f = Shear force due to factored loads transmitted by the weld (LSD)
t = Total combined base steel thickness (exclusive of coatings) of sheets involved in shear transfer above the plane of maximum shear transfer
F_{sy} = Yield point as specified in Sections A2.1 or A2.2

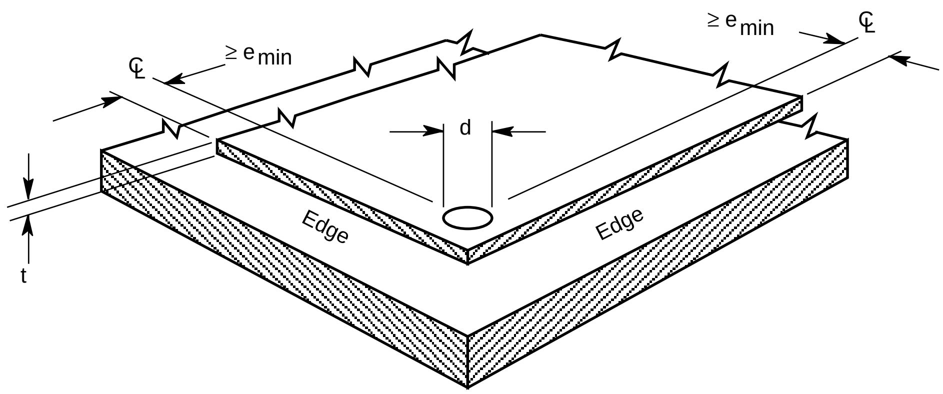


Figure E2.2E Edge Distance for Arc Spot Welds – Single Sheet

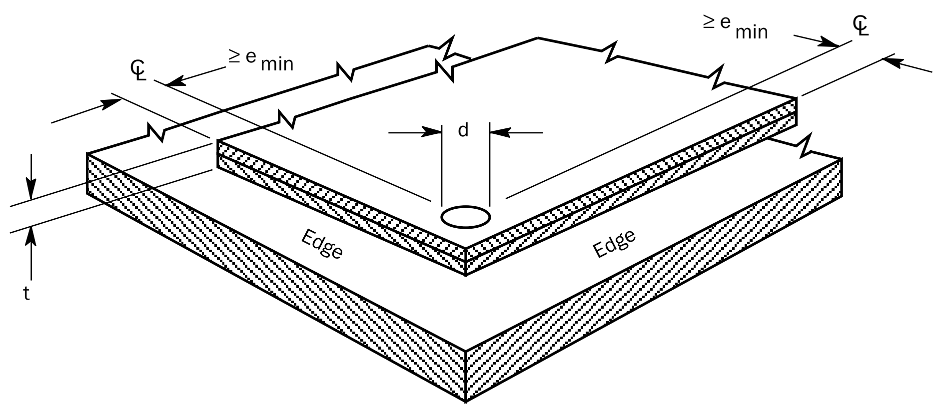


Figure E2.2F Edge Distance for Arc Spot Welds – Double Sheet

Note: See Figures E2.2E and E2.2F for edge distances of arc welds.

In addition, the distance from the centerline of any weld to the end or boundary of the connected member shall not be less than 1.5d. In no case shall the clear distance between welds and the end of member be less than 1.0d.

E2.2.2 Tension

The uplift nominal tensile strength [resistance], P_n, of each concentrically loaded arc spot weld connecting sheets and supporting member, shall be computed as the smaller of either:

$$P_n = \frac{\pi d_e^2}{4} F_{xx} \qquad (Eq.\ E2.2.2\text{-}1)$$

or

$$P_n = 0.8(F_u / F_y)^2 t d_a F_u \qquad (Eq.\ E2.2.2\text{-}2)$$

For panel and deck applications:

USA and Mexico		Canada
Ω(ASD)	φ(LRFD)	φ(LSD)
2.50	0.60	0.50

For all other applications

USA and Mexico		Canada
Ω(ASD)	φ(LRFD)	φ(LSD)
3.00	0.50	0.40

The following limitations shall apply:

$t\ d_a\ F_u \leq 3$

$e_{min} \geq d$

$F_{xx} \geq$ 60 ksi (410 MPa or 4220 kg/cm^2)

$F_u \leq$ 82 ksi (565 MPa or 5770 kg/cm^2) (of connecting sheets)

$F_{xx} > F_u$

where all other parameters are as defined in Section E2.2.1

For eccentrically loaded arc spot welds subjected to an uplift tension load, the nominal tensile strength [resistance] shall be taken as 50 percent of the above value.

For connections having multiple sheets, the strength [resistance] shall be determined by using the sum of the sheet thicknesses as given by Equation E2.2.2-2.

At the side lap connection within a deck system, the nominal tensile strength [resistance] of the weld connection shall be 70 percent of the above values.

If it can be shown by measurement that a given weld procedure will consistently give a larger effective diameter, d_e, or average diameter, d_a, as applicable, this larger diameter may be used providing the particular welding procedure used for making those welds is followed.

E2.3 Arc Seam Welds

Arc seam welds (Figure E2.3A) covered by this *Specification* apply only to the following joints:

(a) Sheet to thicker supporting member in the flat position.

(b) Sheet to sheet in the horizontal or flat position.

The nominal shear strength [resistance], P_n, of arc seam welds shall be determined by using the smaller of either:

$$\text{(a)}\ P_n = \left[\frac{\pi d_e^2}{4} + L d_e\right] 0.75 F_{xx} \qquad (Eq.\ E2.3\text{-}1)$$

$$\text{(b)}\ P_n = 2.5 t F_u (0.25L + 0.96 d_a) \qquad (Eq.\ E2.3\text{-}2)$$

USA and Mexico		Canada
Ω(ASD)	ϕ(LRFD)	ϕ(LSD)
2.55	0.60	0.50

where

P_n = Nominal shear strength [resistance] of an arc seam weld

d = Width of arc seam weld

L = Length of seam weld not including the circular ends (For computation purposes, L shall not exceed 3d)

d_a = Average width of seam weld

= (d - t) for a single and a double sheet (*Eq*. E2.3-3)

d_e = Effective width of arc seam weld at fused surfaces

d_e = 0.7d - 1.5t (*Eq*. E2.3-4)

and F_u, F_{xx}, and t are defined in Section E2.2.1. The minimum edge distance shall be as determined for the arc spot weld, Section E2.2.1. See Figure E2.3B.

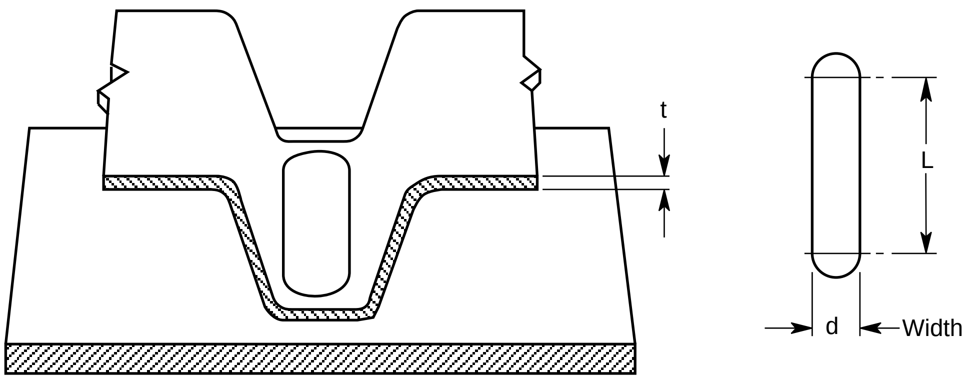


Figure E2.3A Arc Seam Welds - Sheet to Supporting Member in Flat Position

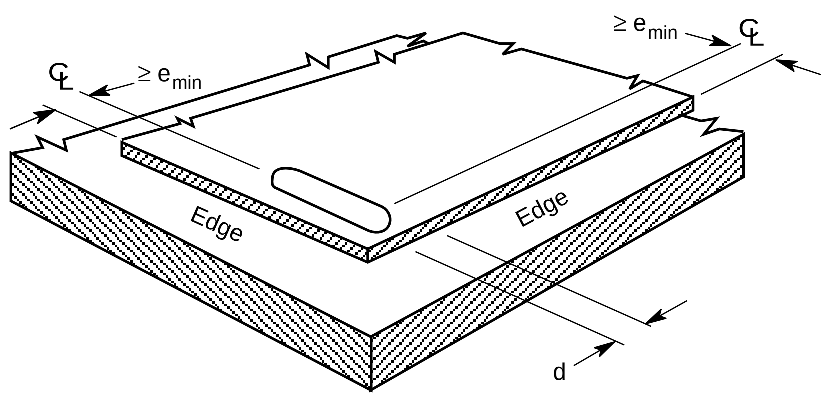


Figure E2.3B Edge Distances for Arc Seam Welds

E2.4 Fillet Welds

Fillet welds covered by this *Specification* apply to the welding of joints in any position, either

(a) Sheet to sheet, or
(b) Sheet to thicker steel member.

The nominal shear strength [resistance], P_n, of a fillet weld shall be determined as follows:

(a) For longitudinal loading:

For $L / t < 25$:

$$P_n = \left(1 - \frac{0.01L}{t}\right) tLF_u \quad (Eq.\ E2.4\text{-}1)$$

USA and Mexico		Canada
Ω(ASD)	ϕ(LRFD)	ϕ(LSD)
2.55	0.60	0.50

For $L / t \geq 25$:

$$P_n = 0.75\ tLF_u \quad (Eq.\ E2.4\text{-}2)$$

USA and Mexico		Canada
Ω(ASD)	ϕ(LRFD)	ϕ(LSD)
3.05	0.50	0.40

(b) For transverse loading:

$$P_n = tLF_u \quad (Eq.\ E2.4\text{-}3)$$

USA and Mexico		Canada
Ω(ASD)	ϕ(LRFD)	ϕ(LSD)
2.35	0.65	0.60

where t=Least value of t_1 or t_2, as shown in Figures E2.4A and E2.4B

In addition, for $t > 0.10$ in. (2.54 mm), the nominal strength [resistance] determined above shall not exceed the following value of P_n:

$$P_n = 0.75\ t_w LF_{xx} \quad (Eq.\ E2.4\text{-}4)$$

USA and Mexico		Canada
Ω(ASD)	ϕ(LRFD)	ϕ(LSD)
2.55	0.60	0.50

where

P_n = Nominal strength [resistance] of a fillet weld

L = Length of fillet weld

t_w = Effective throat = 0.707 w_1 or 0.707 w_2, whichever is smaller. A larger effective throat shall be permitted if measurement shows that the welding procedure to be used consistently yields a larger value of t_w.

w_1 and w_2 = leg on weld (see Figures E2.4A and E2.4B). $w_1 \leq t_1$ in lap joints.

F_u and F_{xx} are defined in Section E2.2.1.

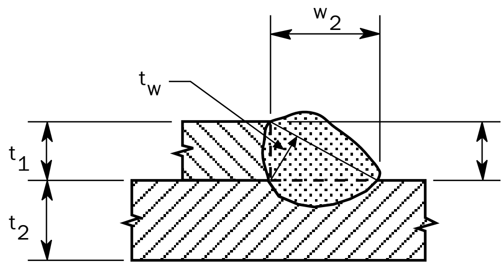


Figure E2.4A Fillet Welds – Lap Joint

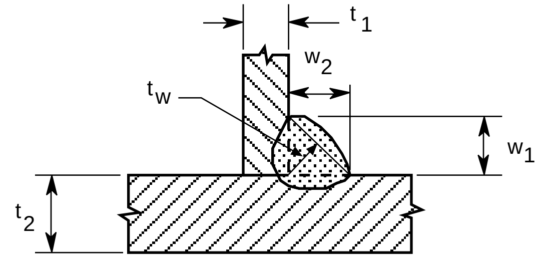


Figure E2.4B Fillet Welds – T Joint

E2.5 Flare Groove Welds

Flare groove welds covered by this *Specification* apply to welding of joints in any position, either:

(a) Sheet to sheet for flare-V groove welds, or
(b) Sheet to sheet for flare-bevel groove welds, or
(c) Sheet to thicker steel member for flare-bevel groove welds.

The nominal shear strength [resistance], P_n, of a flare groove weld shall be determined as follows:

(a) For flare-bevel groove welds, transverse loading (see Figure E2.5A):

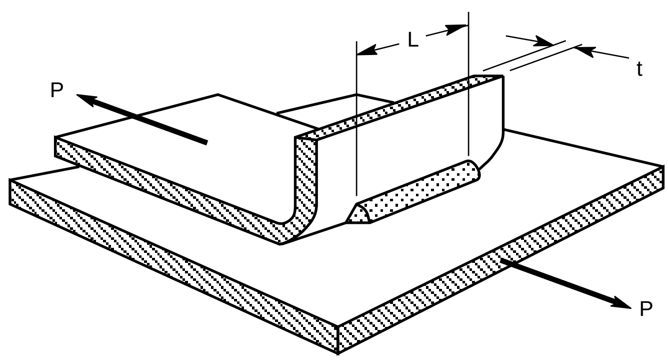


Figure E2.5A Flare-Bevel Groove Weld

$P_n = 0.833tLF_u$ (*Eq*. E2.5-1)

USA and Mexico		Canada
Ω(ASD)	φ(LRFD)	φ(LSD)
2.55	0.60	0.50

(b) For flare groove welds, longitudinal loading (see Figures E2.5B through E2.5G):

(1) For $t \le t_w < 2t$ or if the lip height, h, is less than weld length, L:

$P_n = 0.75tLF_u$ (*Eq*. E2.5-2)

USA and Mexico		Canada
Ω(ASD)	φ(LRFD)	φ(LSD)
2.80	0.55	0.45

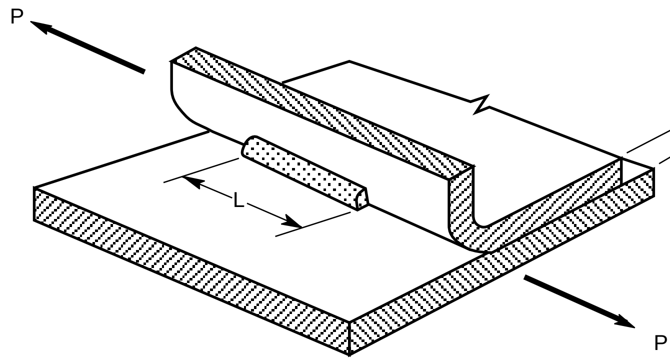


Figure E2.5B Shear in Flare Bevel Groove Weld

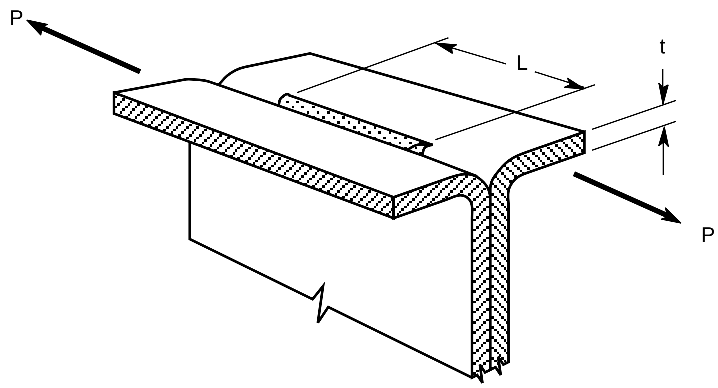


Figure E2.5C Shear in Flare V-Groove Weld

(2) For $t_w \geq 2$ t and the lip height, h, is equal to or greater than weld length L:

$P_n = 1.50tLF_u$ (*Eq*. E2.5-3)

USA and Mexico		Canada
Ω(ASD)	ϕ(LRFD)	ϕ(LSD)
2.80	0.55	0.45

In addition, for t > 0.10 in. (2.54 mm), the nominal strength [resistance] determined above shall not exceed the following value of P_n:

$P_n = 0.75t_wLF_{xx}$ (*Eq*. E2.5-4)

USA and Mexico		Canada
Ω(ASD)	ϕ(LRFD)	ϕ(LSD)
2.55	0.60	0.50

where

P_n = Limiting nominal strength [resistance] of the weld

h = Height of lip

L = Length of the weld

t_w = Effective throat of flare groove weld filled flush to surface (See Figures E2.5D and E2.5E):

For flare bevel groove weld = 5/16R

For flare V-groove weld = 1/2R (3/8R when R > 1/2 in. (12.7mm))

= Effective throat of flare groove weld not filled flush to surface = $0.707w_1$ or $0.707w_2$, whichever is smaller. (See Figures E2.5F and E2.5G.)

= A larger effective throat than those above shall be permitted if measurement shows that the welding procedure to be used consistently yields a larger value of t_w.

R = Radius of outside bend surface.

w_1 and w_2 = Leg on weld (see Figures E2.5F and E2.5G).

F_u and F_{xx} are defined in Section E2.2.1.

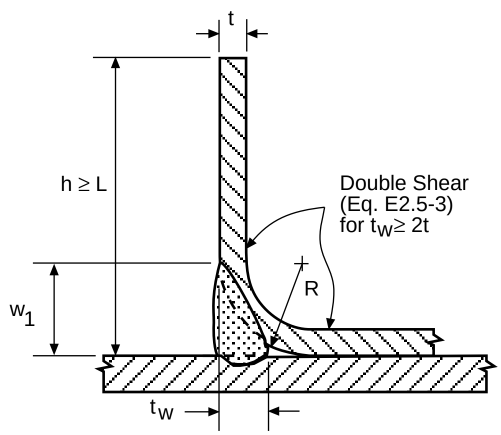


Figure E2.5D Flare Bevel Groove Weld (Filled flush to surface, w_1 = R)

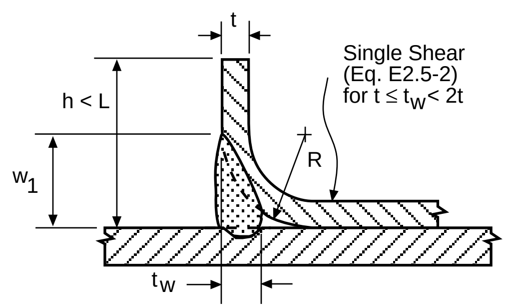


Figure E2.5E Flare Bevel Groove Weld (Filled flush to surface, w_1 = R)

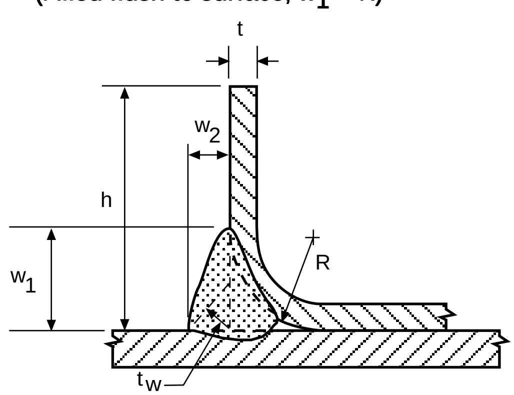


Figure E2.5F Flare Bevel Groove Weld (Not filled flush to surface, w_1 > R)

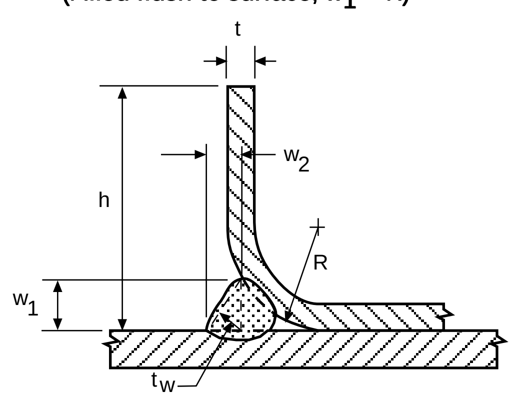


Figure E2.5G Flare Bevel Groove Weld (Not filled flush to surface, w_1 < R)

E2.6 Resistance Welds

The nominal shear strength [resistance], P_n, of spot welds shall be determined as follows:

When t is in inches and P_n is in kips:

For 0.01 in. ≤ t < 0.14 in.:

$$P_n = 144t^{1.47} \quad (Eq. E2.6-1)$$

For 0.14 in. ≤ t ≤ 0.18 in.:

$$P_n = 43.4t + 1.93 \quad (Eq.\ E2.6\text{-}2)$$

When t is in millimeters and P_n is in kN:

For 0.25 mm ≤ t < 3.56 mm:

$$P_n = 5.51t^{1.47} \quad (Eq.\ E2.6\text{-}3)$$

For 3.56 mm ≤ t ≤ 4.57 mm:

$$P_n = 7.6t + 8.57 \quad (Eq.\ E2.6\text{-}4)$$

When t is in centimeters and P_n is in kg:

For 0.025 cm ≤ t < 0.356 cm:

$$P_n = 16600t^{1.47} \quad (Eq.\ E2.6\text{-}5)$$

For 0.356 cm ≤ t ≤ 0.457 cm:

$$P_n = 7750t + 875 \quad (Eq.\ E2.6\text{-}6)$$

where t = Thickness of thinnest outside sheet.

USA and Mexico		Canada
Ω(ASD)	ϕ(LRFD)	ϕ(LSD)
2.35	0.65	0.55

E2.7 Shear Lag Effect in Welded Connections of Members Other Than Flat Sheets

The nominal tensile strength [resistance] of a welded member shall be determined in accordance with Section C2. For fracture and/or yielding in the effective net section of the connected part, the nominal tensile strength [resistance], P_n, shall be determined as follows:

$$P_n = A_e F_u \quad (Eq.\ E2.7\text{-}1)$$

USA and Mexico		Canada
Ω(ASD)	ϕ(LRFD)	ϕ(LSD)
2.50	0.60	0.50

F_u = Tensile strength of the connected part as specified in Section A2.1 or A2.3.2

A_e = AU, effective net area with U defined as follows:

When the load is transmitted only by transverse welds:

A = Area of directly connected elements

U = 1.0

When the load is transmitted only by longitudinal welds or by longitudinal welds in combination with transverse welds:

A = Gross area of member, A_g

U = 1.0 for members when the load is transmitted directly to all of the cross sectional elements. Otherwise the reduction coefficient U is determined as follows:

(a) For angle members:

$$U = 1.0 - 1.20\bar{x}/L < 0.9 \quad (Eq.\ E2.7\text{-}2)$$

but U shall not be less than 0.4.

(b) For channel members

$U = 1.0 - 0.36\ \bar{x}/L < 0.9$ (*Eq*. E2.7-3)

but U shall not be less than 0.5.

$\bar{x}$ = Distance from shear plane to centroid of the cross section

L = Length of longitudinal welds

E3 Bolted Connections

The following design criteria and the requirements stipulated in Section E3a of Appendix A, B, and C govern bolted connections used for cold-formed steel structural members in which the thickness of the thinnest connected part is less than 3/16 in. (4.76 mm). For bolted connections in which the thickness of the thinnest connected part is equal to or greater than 3/16 in. (4.76 mm), refer to the specifications and standards stipulated in Section E3a of Appendix A, B, or C. Bolts, nuts, and washers shall generally conform to one of the following specifications:

ASTM A194/A194M, Carbon and Alloy Steel Nuts for Bolts for High-Pressure and High-Temperature Service

ASTM A307(Type A), Carbon Steel Bolts and Studs, 60 000 PSI Tensile Strength

ASTM A325, Structural Bolts, Steel, Heat Treated, 120/105 ksi Minimum Tensile Strength

ASTM A325M, High Strength Bolts for Structural Steel Joints [Metric]

ASTM A354 (Grade BD), Quenched and Tempered Alloy Steel Bolts, Studs, and Other Externally Threaded Fasteners (for diameter of bolt smaller than $^1/_2$ in.)

ASTM A449, Quenched and Tempered Steel Bolts and Studs (for diameter of bolt smaller than $^1/_2$ in.)

ASTM A490, Heat-Treated Steel Structural Bolts, 150 ksi Minimum Tensile Strength

ASTM A490M, High Strength Steel bolts, Classes 10.9 and 10.9.3, for Structural Steel Joints [Metric]

ASTM A563, Carbon and Alloy Steel Nuts

ASTM A563M, Carbon and Alloy Steel Nuts [Metric]

ASTM F436, Hardened Steel Washers

ASTM F436M, Hardened Steel Washers [Metric]

ASTM F844, Washers, Steel, Plain (Flat), Unhardened for General Use

ASTM F959, Compressible Washer-Type Direct Tension Indicators for Use with Structural Fasteners

ASTM F959M, Compressible Washer-Type Direct Tension Indicators for Use with Structural Fasteners [Metric]

When other than the above are used, drawings shall indicate clearly the type and size of fasteners to be employed and the nominal strength [resistance] assumed in design.

Bolts shall be installed and tightened to achieve satisfactory performance of the connections.

E3.1 Shear, Spacing and Edge Distance

The provisions of this section are given in Section E3.1 of the Appendices.

E3.2 Tension Member Shear Lag Effect in Bolted Connections

The provisions of this section are given in Section E3.2 of the Appendices.

E3.3 Bearing

The design bearing strength [factored resistance] of bolted connections shall be determined according to Sections E3.3.1 and E3.3.2. For conditions not shown, the design bearing strength [factored resistance] of bolted connections shall be determined by tests.

E3.3.1 Strength [Resistance] without Consideration of Bolt Hole Deformation

When deformation around the bolt holes is not a design consideration, the nominal bearing strength [resistance], P_n, of the connected sheet for each loaded bolt shall be determined as follows:

$$P_n = \alpha C d t F_u \qquad (Eq.\ E3.3.1\text{-}1)$$

USA and Mexico		Canada
Ω(ASD)	ϕ(LRFD)	ϕ(LSD)
2.50	0.60	0.50

Where

- C = Bearing factor, which shall be determined according to Table E3.3.1-1.
- d = Nominal bolt diameter
- t = Uncoated sheet thickness
- F_u = Tensile strength of sheet as defined in Section A2.1 or A2.2
- α = Modification factor for type of bearing connection, which shall be determined according to Table E3.3.1-2.

Table E3.3.1-1:
Bearing Factor, C

Thickness of Connected Part, t, in. (mm)	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
	$10 \le d / t \le 22$	$4 - 0.1(d / t)$
	$d / t > 22$	1.8

Table E3.3.1-2
Modification Factor, α, for Type of Bearing Connection

Type of Bearing Connection	α
Single Shear and Outside Sheets of Double Shear Connection With Washers Under Both Bolt Head and Nut	1.00
Single Shear and Outside Sheets of Double Shear Connection Without Washers under both Bolt Head and Nut, Or With only One Washer	0.75
Inside Sheet of Double Shear Connection With or Without Washers	1.33

E3.3.2 Strength [Resistance] with Consideration of Bolt Hole Deformation

When deformation around a bolt hole is a design consideration, the nominal bearing strength [resistance], P_n, shall also be limited by the following values:

$$P_n = (4.64\alpha t + 1.53)dtF_u \qquad (Eq.\ E3.3.2\text{-}1)$$

USA and Mexico		Canada
Ω(ASD)	ϕ(LRFD)	ϕ(LSD)
2.22	0.65	0.55

where

α = Coefficient for conversion of units
= 1 for US customary units (with t in inches)
= 0.0394 for Metric units (with t in mm)
= 0.394 for MKS units (with t in cm)

The other symbols are defined in Section E3.3.1

E3.4 Shear and Tension in Bolts

The provisions under this section are provided in Section E3.4 of the Appendices.

E4 Screw Connections

All E4 requirements 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.

The nominal screw connection strengths [resistances] shall also be limited by Section C2.

For diaphragm applications, Section D5 shall be used.

The following factor of safety or resistance factor shall be used for the sub-sections of Chapter E4.

USA and Mexico		Canada
Ω(ASD)	φ(LRFD)	φ(LSD)
3.00	0.50	0.40

Alternatively, design values for a particular application shall be permitted to be based on tests, with the factor of safety, Ω, and the resistance factor, φ, determined according to Chapter F.

The following notation applies to this section:

d = Nominal screw diameter
d_w = Larger of the screw head diameter or the washer diameter
P_{ns} = Nominal shear strength [resistance] per screw
P_{ss} = Nominal shear strength [resistance] of screw as reported by manufacturer or determined by independent laboratory testing
P_{nt} = Nominal tension strength [resistance] per screw
P_{not} = Nominal pull-out strength [resistance] per screw
P_{nov} = Nominal pull-over strength [resistance] per screw
P_{ts} = Nominal tension strength [resistance] of screw as reported by manufacturer or determined by independent laboratory testing
t_1 = Thickness of member in contact with the screw head
t_2 = Thickness of member not in contact with the screw head
t_c = Lesser of the depth of the penetration and the thickness t_2
F_{u1} = Tensile strength of member in contact with the screw head
F_{u2} = Tensile strength of member not in contact with the screw head

E4.1 Minimum Spacing

The distance between the centers of fasteners shall not be less than 3d.

E4.2 Minimum Edge and End Distance

The distance from the center of a fastener to the edge of any part shall not be less than 1.5d. If the end distance is parallel to the force on the fastener, the nominal shear strength per screw, P_{ns}, shall be limited by Section E4.3.2.

E4.3 Shear

E4.3.1 Connection Shear as Limited by Tilting and Bearing

The nominal shear strength [resistance] per screw, P_{ns}, shall be determined as follows:

For $t_2/t_1 \leq 1.0$, P_{ns} shall be taken as the smallest of

$$P_{ns} = 4.2\,(t_2^3 d)^{1/2} F_{u2} \quad (Eq.\ E4.3.1\text{-}1)$$

$$P_{ns} = 2.7\, t_1\, d\, F_{u1} \quad (Eq.\ E4.3.1\text{-}2)$$

$$P_{ns} = 2.7\, t_2\, d\, F_{u2} \quad (Eq.\ E4.3.1\text{-}3)$$

For $t_2/t_1 \geq 2.5$, P_{ns} shall be taken as the smaller of

$$P_{ns} = 2.7\, t_1\, d\, F_{u1} \quad (Eq.\ E4.3.1\text{-}4)$$

$$P_{ns} = 2.7\, t_2\, d\, F_{u2} \quad (Eq.\ E4.3.1\text{-}5)$$

For $1.0 < t_2/t_1 < 2.5$, P_{ns} shall be determined by linear interpolation between the above two cases.

E4.3.2 Connection Shear as Limited by End Distance

The provisions of this section are given in Section E4.3.2 of the Appendices.

E4.3.3 Shear in Screws

The nominal shear strength [resistance] of the screw shall be calculated as follows:

$$P_{ns} = 0.8 P_{ss} \quad (Eq.\ E4.3.3\text{-}1)$$

E4.4 Tension

For screws which carry tension, the head of the screw or washer, if a washer is provided, shall have a diameter d_w not less than 5/16 in. (7.94 mm). Washers shall be at least 0.050 in. (1.27 mm) thick.

E4.4.1 Pull-Out

The nominal pull-out strength [resistance], P_{not}, shall be calculated as follows:

$$P_{not} = 0.85\, t_c\, d\, F_{u2} \quad (Eq.\ E4.4.1\text{-}1)$$

E4.4.2 Pull-Over

The nominal pull-over strength [resistance], P_{nov}, shall be calculated as follows:

$$P_{nov} = 1.5\, t_1\, d_w\, F_{u1} \quad (Eq.\ E4.4.2.1)$$

where d_w shall be taken not larger than 1/2 in. (12.7 mm).

E4.4.3 Tension in Screws

The nominal tension strength [resistance], P_{nt}, per screw shall be calculated as follows:

$$P_{nt} = 0.8\ P_{ts} \quad (Eq.\ E4.4.3\text{-}1)$$

E5 Rupture

The provisions provided under this section are given in Section E5 of the Appendices.

E6 Connections to Other Materials

E6.1 Bearing

Proper provisions shall be made to transfer bearing forces from steel components covered by the *Specification* to adjacent structural components made of other materials.

E6.2 Tension

The pull-over shear/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, or product specifications and/or product literature.

E6.3 Shear

Proper 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 [resistance] on the steel components shall not exceed that allowed by this *Specification*. The design shear strength [resistance] on the fasteners and other material shall not be exceeded. Embedment requirements are to be met. Proper provision shall also be made for shearing forces in combination with other forces.

F. TESTS FOR SPECIAL CASES

(a) Tests shall be made by an independent testing laboratory or by a testing laboratory of a manufacturer.
(b) The provisions of Chapter F do not apply to cold-formed steel diaphragms. Refer to Section D5.

F1 Tests for Determining Structural Performance

F1.1 Load and Resistance Factor Design and Limit States Design

Any structural performance which is required to be established by tests shall be evaluated in accordance with the following performance procedure:

(a) Evaluation of the test results 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 can be 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) The strength of the tested elements, assemblies, connections, or members shall satisfy Eq. F1.1-1.

$$\Sigma\gamma_i Q_i \leq \phi R_n \qquad (Eq.\ F1.1\text{-}1)$$

where

$\Sigma\gamma_i Q_i$ = Required strength [effect of factored loads] based on the most critical load combination determined in accordance with Section A5.1.2. γ_i and Q_i are load factors and load effects, respectively.

R_n = Average value of all test results

ϕ = Resistance factor

$$= C_\phi (M_m F_m P_m) e^{-\beta_o \sqrt{V_M^2 + V_F^2 + C_P V_P^2 + V_Q^2}}\ ^* \qquad (Eq.\ F1.1\text{-}2)$$

C_ϕ = Calibration coefficient
= 1.52 for United States and Mexico
= 1.42 for Canada

M_m = Mean value of the material factor, M, listed in Table F1 for the type of component involved

F_m = Mean value of the fabrication factor, F, listed in Table F1 for the type of component involved

P_m = Mean value of the professional factor, P, for the tested component
= 1.0

β_o = Target reliability index

= 2.5 for structural members and 3.5 for connections for United States and Mexico
= 3.0 for structural members and 4.0 for connections for Canada

V_M = Coefficient of variation of the material factor listed in Table F1 for the type of component involved

V_F = Coefficient of variation of the fabrication factor listed in Table F1 for the type of component involved

C_P = Correction factor
= $(1+1/n)m/(m-2)$ for $n \geq 4$, and 5.7 for $n = 3$ (*Eq.* F1.1-3)

V_P = Coefficient of variation of the test results, but not less than 6.5%

m = Degrees of freedom
= n-1

n = Number of tests

V_Q = Coefficient of variation of the load effect
= 0.21

e = Natural logarithmic base
= 2.718...

Note:

* For beams having tension flange through-fastened to deck or sheathing and with compression flange laterally unbraced, ϕ shall be determined with a coefficient, C_ϕ, of 1.6 in lieu of 1.52 for the United States and Mexico, $\beta_o = 1.5$, and $V_Q = 0.43$.

The listing in Table F1 does 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 A2.1, 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. F1.1-1, except that the resistance factor ϕ is taken as unity and that the load factor for dead load is taken as 1.0.

(c) If the yield point of the steel from which the tested sections are formed is larger than the specified value, the test results shall be adjusted down to the specified minimum yield point of the steel which the manufacturer intends to use. The test results shall not be adjusted upward if the yield point of the test specimen is less than the minimum specified yield point. Similar adjustments shall be made on the basis of tensile strength instead of yield point where tensile strength is the critical factor.

Consideration must also be given to any variation or differences which may exist between the design thickness and the thickness of the specimens used in the tests.

TABLE F1
Statistical Data for the Determination of Resistance Factor

Type of Component	M_m	V_M	F_m	V_F
Transverse Stiffeners	1.10	0.10	1.00	0.05
Shear Stiffeners	1.00	0.06	1.00	0.05
Tension Members	1.10	0.10	1.00	0.05
Flexural Members				
Bending Strength	1.10	0.10	1.00	0.05
Lateral-Torsional Buckling Strength	1.00	0.06	1.00	0.05
One Flange Through-Fastened to Deck or Sheathing	1.10	0.10	1.00	0.05
Shear Strength	1.10	0.10	1.00	0.05
Combined Bending and Shear	1.10	0.10	1.00	0.05
Web Crippling Strength	1.10	0.10	1.00	0.05
Combined Bending and Web Crippling	1.10	0.10	1.00	0.05
Concentrically Loaded Compression Members	1.10	0.10	1.00	0.05
Combined Axial Load and Bending	1.05	0.10	1.00	0.05
Cylindrical Tubular Members				
Bending Strength	1.10	0.10	1.00	0.05
Axial Compression	1.10	0.10	1.00	0.05
Wall Studs and Wall Stud Assemblies				
Wall Studs in Compression	1.10	0.10	1.00	0.05
Wall Studs in Bending	1.10	0.10	1.00	0.05
Wall Studs with Combined Axial load and Bending	1.05	0.10	1.00	0.05
Structural Members Not Listed Above	1.00	0.10	1.00	0.05

TABLE F1 (Continued)
Statistical Data for the Determination of Resistance Factor

Type of Component	M_m	V_M	F_m	V_F
Welded Connections				
Arc Spot Welds				
Shear Strength of Welds	1.10	0.10	1.00	0.10
Plate Failure	1.10	0.08	1.00	0.15
Arc Seam Welds				
Shear Strength of Welds	1.10	0.10	1.00	0.10
Plate Tearing	1.10	0.10	1.00	0.10
Fillet Welds				
Shear Strength of Welds	1.10	0.10	1.00	0.10
Plate Failure	1.10	0.08	1.00	0.15
Flare Groove Welds				
Shear Strength of Welds	1.10	0.10	1.00	0.10
Plate Failure	1.10	0.10	1.00	0.10
Resistance Welds	1.10	0.10	1.00	0.10
Bolted Connections				
Minimum Spacing and Edge Distance	1.10	0.08	1.00	0.05
Tension Strength on Net Section	1.10	0.08	1.00	0.05
Bearing Strength	1.10	0.08	1.00	0.05
Screw Connections				
Minimum Spacing and Edge Distance	1.10	0.10	1.00	0.10
Tension Strength on Net Section	1.10	0.10	1.00	0.10
Bearing Strength	1.10	0.10	1.00	0.10
Connections Not Listed Above	1.10	0.10	1.00	0.15

F1.2 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 F1.1, except as modified in this section for allowable strength design.

The allowable design strength shall be calculated as:

$$R = R_n / \Omega \qquad (Eq.\ F1.2\text{-}1)$$

where

R_n = Average value of all test results

Ω = Factor of safety to be computed as follows:

$$\Omega = \frac{1.6}{\phi} \qquad (Eq.\ F1.2\text{-}2)$$

in which ϕ is evaluated in accordance with Section F1.1.

The required allowable strength shall be determined from nominal loads and load combinations as described in A4.

F2 Tests for Confirming Structural Performance

For structural members, connections, and assemblies for which the nominal strength [resistance] can be computed according to this *Specification* or its specific references, confirmatory tests may be made to demonstrate the strength is not less than the nominal resistance, R_n, specified in this *Specification* or its specific references for the type of behavior involved.

F3 Tests for Determining Mechanical Properties

F3.1 Full Section

Tests for determination of mechanical properties of full sections to be used in Section A7.2 shall be made as specified below:

(a) Tensile testing procedures shall agree with Standard Methods and Definitions for Mechanical Testing of Steel Products, ASTM A370. Compressive yield point determinations shall be made by means of compression tests of short specimens of the section.

(b) The compressive yield stress shall be taken as the smaller value of either the maximum compressive strength of the sections divided by the cross section area or the stress defined by one of the following methods:

(1) For sharp yielding steel, the yield point shall be determined by the autographic diagram method or by the total strain under load method.

(2) For gradual yielding steel, the yield point shall be 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 point so determined agrees within 5 percent with the yield point which 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 point shall be determined for the flanges only. In determining such yield points, 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 may be used for routine acceptance and control purposes, provided the manufacturer demonstrates that such tests reliably indicate the yield point of the section when subjected to the kind of stress under which the member is to be used.

F3.2 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 A7.2 shall be made in accordance with the following provisions:

The yield point of flats, F_{yf}, shall be established by means of a weighted average of the yield points 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 point for each flat portion times its cross sectional area, divided by the total area of flats in the cross section. 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 point exceeds the specified minimum yield point, the yield point of the flats, F_{yf}, shall be adjusted by multiplying the test values by the ratio of the specified minimum yield point to the actual virgin yield point.

F3.3 Virgin Steel

The following provisions apply to steel produced to other than the ASTM Specifications listed in Section A2.1 when used in sections for which the increased yield point of the steel after cold forming shall be computed from the virgin steel properties according to Section A7.2. 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 point and tensile strength. Specimens shall be taken longitudinally from the quarter points of the width near the outer end of the coil.

G. DESIGN OF COLD-FORMED STEEL STRUCTURAL MEMBERS AND CONNECTIONS FOR CYCLIC LOADING (FATIGUE)

This design procedure shall apply to cold-formed steel members and connections subject to cyclic loading within the elastic range of stresses of frequency and magnitude sufficient to initiate cracking and progressive failure (fatigue).

G1 General

When cyclic loading is a design consideration, the provisions of this Chapter apply to stresses calculated on the basis of unfactored loads. The maximum permitted tensile stress due to unfactored loads is 0.6 F_y.

Stress range is defined as the magnitude of the change in stress due to the application or removal of the unfactored 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.

The occurrence of full design wind or earthquake loads is too infrequent to warrant consideration in fatigue design. Therefore, evaluation of fatigue resistance is not required for wind load applications in buildings. If the live load stress range is less than the threshold stress range, F_{TH}, given in Table G1, evaluation of fatigue resistance is also not required.

Table G1: 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	3.2×10^{10}	25 (172) [1760]	G1
As-received base metal and weld metal in members connected by continuous longitudinal welds.	II	1.0×10^{10}	15 (103) [1050]	G2
Welded attachments to a plate or a beam, transverse fillet welds, and continuous longitudinal fillet welds less than and equal to 2 inches. Bolt and screw connections and spot welds.	III	3.2×10^{9}	16 (110) [1120]	G3, G4
Longitudinal fillet welded attachments greater than 2 inches parall of the applied stress, and intermittent welds parall l to the direction of the applied force.	IV	1.0×10^{9}	9 (62) [633]	G4

Evaluation of fatigue resistance is not required if the number of cycles of application of live load is less than 20,000.

The cyclic load resistance determined by the provisions of this Chapter is applicable to structures with suitable corrosion protection or subject only to non-aggressive atmospheres.

The cyclic load resistance determined by the provisions of this Chapter is applicable only to structures subject to temperatures not exceeding 300°F (149°C).

The contract documents shall provide, either complete details including weld sizes, or shall specify the planned cycle life and the maximum range of moments, shears, and reactions for the connections.

G2 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.

G3 Design Stress Range

The range of stress at service loads shall not exceed the design stress range computed using Equation G3-1.

For all stress categories,

$$F_{SR} = (\alpha C_f / N)^{0.333} \geq F_{TH} \qquad (Eq.\ G3\text{-}1)$$

where:

F_{SR} = Design stress range

C_f = Constant from Table G1

N = Number of stress range fluctuations in design life
= Number of stress range fluctuations per day x 365 x years of design life

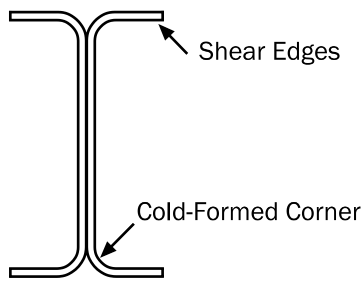
F_{TH} = Threshold fatigue stress range, maximum stress range for indefinite design life from Table G1

α = Coefficient for conversion of units

= 1 for US customary units
= 327 for SI units
= 352,000 for MKS units

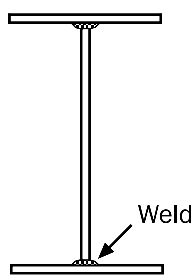
G4 Bolts and Threaded Parts

For mechanically fastened connections loaded in shear, the maximum range of stress in the connected material at service loads shall not exceed the design stress range computed using Equation G3-1. 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).



Cold-Formed Steel Channels, Category I

Figure G1 Typical Detail for Category I



Welded I Beam, Category II

Figure G2 Typical Detail for Category II

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 Equation G3-1. 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 is given by Equation G4-1.

$$A_t = (\pi/4)\,[d_b - (0.9743/n)]^2 \qquad (Eq.\ G4\text{-}1)$$

For Metric or MKS Units:

$$A_t = (\pi/4)\,[d_b - (0.9382P)]^2 \qquad (Eq.\ G4\text{-}1a)$$

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 Metric Units and cm per thread for MKS Units)

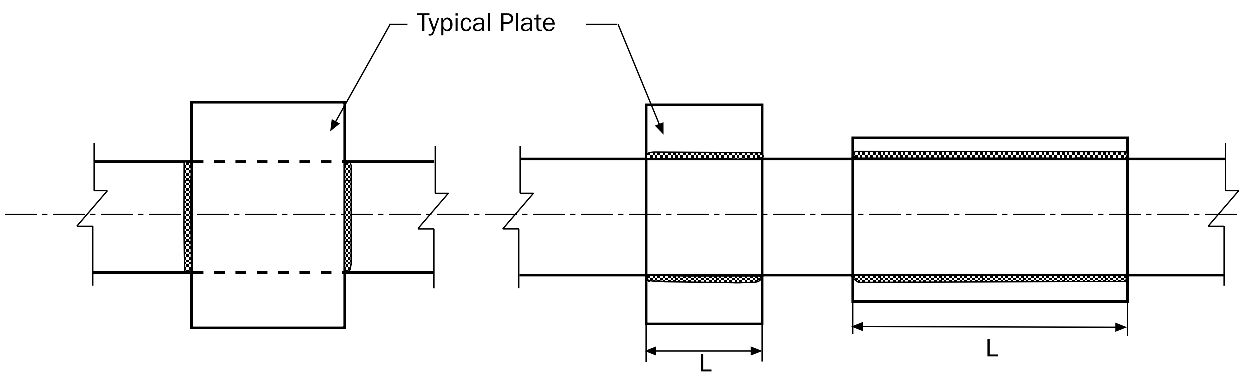
G5 Special Fabrication Requirements

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), where the reference standard is ASME B46.1.

Re-entrant corners at cuts, copes and weld access holes shall form a radius of not less than 3/8 in. (10 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), where the reference standard is ASME B46.1 or other equivalent standards shall be referenced.

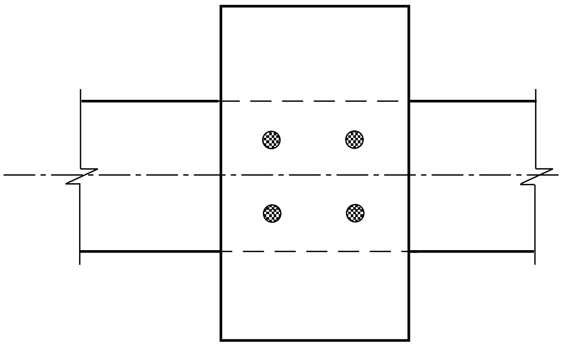


(a) Transverse Welds, Category III

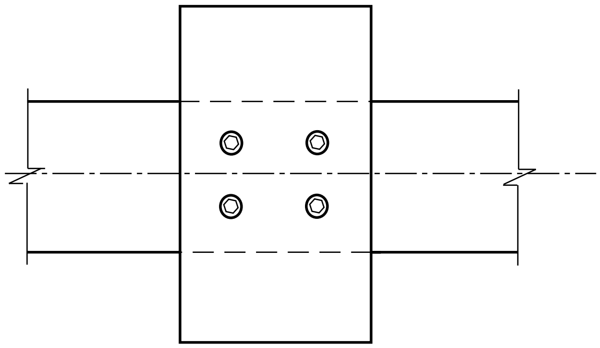
(b) Longitudinal Welds
For Category III , L≤ 2 in. (50.8 mm)
For Category IV, 2 in. (50.8 mm)< L ≤ 4 in. (101.6 mm)

Figure G3 Typical Attachments for Categories III and IV

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 are not required for sheet material if the welding procedures used result in smooth, flush edges.



(c) Arc Spot or Plug Weld



(c) Screws

Figure G4 Typical Attachments for Category III

PREFACE TO APPENDIX A

Appendix A provides specification provisions that are only applicable to the United States. Included are items of a broad nature such as provisions for the design method to be used, ASD or LRFD, and provisions to use ASCE 7 for loads and load combinations where there is not an applicable building code. Reference documents that are not used by all three countries are listed here as well.

Also included in Appendix A are technical items where full agreement between the three countries was not reached. Such items included certain provisions pertaining to the design of

- beams (C and Z sections) for standing seam roofs,
- welded connections,
- bolted connections, and
- tension members.

Efforts will be made to minimize these differences in future editions of the *Specification*.

APPENDIX A: PROVISIONS APPLICABLE TO THE UNITED STATES

This Appendix provides design provisions or supplements to Chapters A through F that are only applicable to the United States. A section number ending with a letter indicates that the provisions herein supplement the corresponding section in Chapters A through F of the *Specification*. A section number not ending with a letter indicates that the section gives the entire design provision.

A1.1a Scope and Limits of Applicability

Designs shall be made according to the provisions for Load and Resistance Factor Design, or to the provisions for Allowable Stress Design. Where allowed, both methods are equally acceptable although they may or may not produce identical designs. However, the two methods shall not be mixed in designing the various cold-formed steel components of a structure.

A3 Loads

A3.1 Nominal Loads

The nominal loads shall be as stipulated by the applicable building code under which the structure is designed or as dictated by the conditions involved. In the absence of a building code, the nominal loads shall be those stipulated in the American Society of Civil Engineers Standard, Minimum Design Loads for Buildings and Other Structures, ASCE 7.

A4.1.2 Load Combinations for ASD

The structure and its components shall be designed so that allowable design strengths equal or exceed the effects of the nominal loads and load combinations as stipulated by the applicable building code under which the structure is designed or, in the absence of an applicable building code, as stipulated in the American Society of Civil Engineers Standard, Minimum Design Loads for Buildings and Other Structures, ASCE 7.

The combined effects of two or more loads, excluding dead load, shall be permitted to be multiplied by 0.75. The combined load used in design shall not be less than the sum of the effects of dead load and any single load that produces the largest effect. The above 0.75 load reduction shall not be used where similar load reductions are permitted by the applicable building code or ASCE 7.

Exception: When evaluating diaphragms using the provisions of Section D5, no decrease in forces is permitted for load combinations including wind or earthquake loads.

A5.1.2 Load Factors and Load Combinations for LRFD

The structure and its components shall be designed so that design strengths equal or exceed the effects of the factored nominal loads and load combinations stipulated by the applicable building code under which the structure is designed or, in the absence of an applicable building code, as stipulated in the American Society of Civil Engineers Standard, Minimum Design Loads for Buildings and Other Structures, ASCE 7.

A9a Referenced Documents

The following documents are referenced in Appendix A:

1. American Society of Civil Engineers, ASCE 7-98, "Minimum Design Loads in Buildings and Other Structures," American Society of Civil Engineers (ASCE), 1801 Alexander Bell Drive, Reston VA, 20191
2. American Institute of Steel Construction, "Specification for Structural Steel Buildings, Allowable Stress Design and Plastic Design," American Institute of Steel Construction (AISC), One East Wacker Drive, Suite 3100, Chicago, Illinois 60601-2001, June 1, 1989
3. American Institute of Steel Construction, "Load and Resistance Factor Design Specification for Structural Steel Buildings", American Institute of Steel Construction (AISC), One East Wacker Drive, Suite 3100, Chicago, Illinois 60601-2001, December 27, 1999
4. American Welding Society, AWS D1.3-98, "Structural Welding Code - Sheet Steel," American Welding Society (AWS), 550 N.W. LeJeune Road, Miami, Florida 33135
5. American Welding Society, AWS C1.1-66, "Recommended Practices for Resistance Welding," American Welding Society (AWS), 550 N.W. LeJeune Road, Miami, Florida 33135
6. American Welding Society, AWS C1.3-70 (Reaffirmed 1987), "Recommended Practices for Resistance Welding Coated Low Carbon Steels," American Welding Society (AWS), 550 N.W. LeJeune Road, Miami, Florida 33135

C2 Tension Members

For axially loaded tension members, the nominal tensile strength [resistance], T_n, shall be the smallest value obtained according to the limit states of (a) yielding in the gross section, (b) fracture in the net section away from connections, and (c) fracture in the effective net section at the connection:

(a) For yielding:

$$T_n = A_g F_y \qquad (Eq.\ C2\text{-}1)$$

$\Omega_t = 1.67$ (ASD)

$\phi_t = 0.90$ (LRFD)

(b) For fracture away from the connection:

$$T_n = A_n F_u \qquad (Eq.\ C2\text{-}2)$$

$\Omega_t = 2.00$ (ASD)

$\phi_t = 0.75$ (LRFD)

where

T_n = Nominal strength [resistance] of member when loaded in tension

A_g = Gross area of cross section

A_n = Net area of the cross section

F_y = Design yield stress as determined in Section A7.1

F_u = Tensile strength as specified in Section A2.1 or A2.3.2

(c) For fracture at the connection:

The nominal tensile strength [resistance] shall also be limited by Sections E2.7, E3, and E5 for tension members using welded connections, bolted connections, and screw connections.

C3.1.4 Beams Having One Flange Fastened to a Standing Seam Roof System

The nominal flexural strength, M_n, 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 C3.1.2.1 or shall be calculated as follows:

$$M_n = RS_eF_y \qquad (Eq.\ C3.1.4\text{-}1)$$

$$\Omega_b = 1.67 \text{ (ASD)}$$

$$\phi_b = 0.90 \text{ (LRFD)}$$

where

R = Reduction factor determined by the "Base Test Method for Purlins Supporting a Standing Seam Roof System" of Part VIII of the AISI *Cold-Formed Steel Design Manual*.

S_e and F_y are defined in Section C3.1.1.

E2a Welded Connections

For welded connections in which the thickness of the thinnest connected part is greater than 0.18 in. (4.57 mm), refer to the AISC “Specification for Structural Steel Buildings, Allowable Stress Design and Plastic Design”, or the “Load and Resistance Factor Design Specification for Structural Steel Buildings”.

Except as modified herein, arc welds on steel where at least one of the connected parts is 0.18 in. (4.57 mm) or less in thickness shall be made in accordance with the AWS D1.3 and its Commentary. Welders and welding procedures shall be qualified as specified in AWS D1.3. These provisions are intended to cover the welding positions as shown in Table E2a.

Resistance welds shall be made in conformance with the procedures given in AWS C1.1 or AWS C1.3.

TABLE E2a
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)

E3a Bolted Connections

In addition to the design criteria given in Section E3 of the *Specification*, the following design requirements shall also be followed for bolted connections used for cold-formed steel structural members in which the thickness of the thinnest connected part is less than 3/16 in. (4.76 mm). For bolted connections in which the thickness of the thinnest connected part is equal to or greater than 3/16 in. (4.76 mm), refer to AISC "Specification for Structural Steel Buildings, Allowable Stress Design and Plastic Design", or the "Load and Resistance Factor Design Specification for Structural Steel Buildings".

The holes for bolts shall not exceed the sizes specified in Table E3a, except that larger holes may be used in column base details or structural systems connected to concrete walls.

Standard holes shall be used in bolted connections, except that oversized and slotted holes may be used as approved by the designer. The length of

TABLE E3a
Maximum Size of Bolt Holes, 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.
$< 1/2$	$d + 1/32$	$d + 1/16$	$(d + 1/32)$ by $(d + 1/4)$	$(d + 1/32)$ by $(2\,1/2\ d)$
$\geq 1/2$	$d + 1/16$	$d + 1/8$	$(d + 1/16)$ by $(d + 1/4)$	$(d + 1/16)$ by $(2\,1/2\ d)$

TABLE E3a
Maximum Size of Bolt Holes, 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
< 12.7	d + 0.8	d + 1.6	(d + 0.8) by (d + 6.4)	(d + 0.8) by ($2^1/_2$ d)
≥ 12.7	d + 1.6	d + 3.2	(d + 1.6) by (d + 6.4)	(d + 1.6) by ($2^1/_2$ d)

slotted holes shall be normal to the direction of the shear load. Washers or backup plates shall be installed over oversized or slotted holes in an outer ply unless suitable performance is demonstrated by tests in accordance with Chapter F.

E3.1 Shear, Spacing and Edge Distance

The nominal shear strength [resistance], P_n, of the connected part as affected by spacing and edge distance in the direction of applied force shall be calculated as follows:

$$P_n = teF_u \qquad (Eq.\ E3.1\text{-}1)$$

(a) When $F_u / F_{sy} \geq 1.08$:

$\Omega = 2.00$ (ASD)

$\phi = 0.70$ (LRFD)

(b) When $F_u / F_{sy} < 1.08$:

$\Omega = 2.22$ (ASD)

$\phi = 0.60$ (LRFD)

where

P_n = Nominal strength [resistance] per bolt

e = The distance measured in the line of force from the center of a standard hole to the nearest edge of an adjacent hole or to the end of the connected part

t = Thickness of thinnest connected part

F_u = Tensile strength of the connected part as specified in Section A2.1 or A2.2

F_{sy} = Yield point of the connected part as specified in Section A2.1 or A2.2

In addition, the minimum distance between centers of bolt holes shall provide sufficient clearance for bolt heads, nuts, washers and the wrench. The minimum distance between centers of bolt holes shall provide sufficient clearance for bolt heads, nuts, washers and the wrench but shall not be less than 3 times the nominal bolt diameter, d. Also, the distance from the center of any standard hole to the end or other boundary of the connecting member shall not be less than $1^1/_2$ d.

For oversized and slotted holes, the distance between edges of two adjacent holes and the distance measured from the edge of the hole to the end or other boundary of the connecting member in the line of stress shall not be less than the value of $e-(d_h/2)$, in which e is the required distance computed from the applicable equation given above, and d_h is the diameter of a standard hole defined in Table E3a. In no case shall the clear distance between edges of two adjacent holes be less than 2d and the distance between the edge of the hole and the end of the member be less than d.

E3.2 Tension Member Shear Lag Effect in Bolted Connections

The nominal tensile strength [resistance] of a bolted member shall be determined in accordance with Section C2. For fracture in the effective net section of the connected part, the nominal tensile strength [resistance], P_n, shall be determined as follows:

(1) For flat sheet connections not having staggered hole patterns:

$$P_n = A_n F_t \quad (Eq.\ E3.2\text{-}1)$$

(a) When washers are provided under both the bolt head and the nut:

For a single bolt, or a single row of bolts perpendicular to the force

$$F_t = (0.1 + 3d/s)\,F_u \le F_u \quad (Eq.\ E3.2\text{-}2)$$

For multiple bolts in the line parallel to the force

$$F_t = F_u \quad (Eq.\ E3.2\text{-}3)$$

For double shear:

Ω =2.00 (ASD)

ϕ =0.65 (LRFD)

For single shear:

Ω =2.22 (ASD)

ϕ =0.55 (LRFD)

(b) When either washers are not provided under the bolt head and the nut, or only one washer is provided under either the bolt head or the nut:

$$F_t = (2.5d/s)\,F_u \le F_u \quad (Eq.\ E3.2\text{-}4)$$

For multiple bolts in the line parallel to the force

$$F_t = F_u \quad (Eq.\ E3.2\text{-}5)$$

Ω =2.22 (ASD)

ϕ =0.65 (LRFD)

where

A_n = Net area of the connected part

s = Sheet width divided by the number of bolt holes in the cross section being analyzed (when evaluating F_t)

F_u = Tensile strength of the connected part as specified in Section A2.1 or A2.2

d = Nominal bolt diameter

(2) For flat sheet connections having staggered hole patterns:

$P_n = A_nF_t$ (*Eq*. E3.2-6)

Ω =2.22 (ASD)

ϕ =0.65 (LRFD)

where

F_t is determined in accordance with Eqs. E3.2-2 to E3.2-5.

A_n $=0.90\,[A_g - n_b d_h t + (\Sigma s'^2/4g)t]$ (*Eq*. E3.2-7)

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 bolt holes in the cross section being analyzed

d_h =Diameter of a standard hole

t is defined in Section E3.1.

(3) For other than flat sheet:

$P_n = A_eF_u$ (*Eq*. E3.2-8)

Ω =2.22 (ASD)

ϕ =0.65 (LRFD)

where

F_u = Tensile strength of the connected part as specified in Section A2.1 or A2.3.2

A_e = A_nU, effective net area with U defined as follows:

U = 1.0 for members when the load is transmitted directly to all of the cross-sectional elements. Otherwise, the reduction coefficient U is determined as follows:

(a) For angle members having two or more bolts in the line of force

$U = 1.0 - 1.20\,\bar{x}/L < 0.9$ (*Eq*. E3.2-9)

but U shall not be less than 0.4.

(b) For Channel members having two or more bolts in the line of force

$U = 1.0 - 0.36\,\bar{x}/L < 0.9$ (*Eq*. E3.2-10)

but U shall not be less than 0.5.

$\bar{x}$ = Distance from shear plane to centroid of the cross section

L = Length of the connection

E3.4 Shear and Tension in Bolts

The nominal bolt strength [resistance], P_n, resulting from shear, tension or a combination of shear and tension shall be calculated as follows:

$P_n = A_b F_n$ (*Eq*. E3.4-1)

where

A_b = Gross cross-sectional area of bolt

When bolts are subject to shear or tension:

F_n is given by F_{nv} or F_{nt} in Table E3.4-1.

Ω and ϕ are given in Table E3.4-1.

The pullover strength [resistance] of the connected sheet at the bolt head, nut or washer shall be considered where bolt tension is involved, see Section E6.2.

When bolts are subject to a combination of shear and tension:

For ASD

F_n is given by F'_{nt} in Table E3.4-2 or E3.4-4 (metric units) or E3.4-6 (MKS units)

Ω is given in Table E3.4-2 or E3.4-4 (metric units) or E3.4-6 (MKS units)

For LRFD

F_n is given by F'_{nt} in Table E3.4-3 or E3.4-5 (metric units) or E3.4-7 (MKS units)

ϕ is given in Table E3.4-3 or E3.4-5 (metric units) or E3.4-7 (MKS units)

TABLE E3.4-1
Nominal Tensile and Shear Strength [Resistance] for Bolts

	Tensile Strength [Resistance]			Shear Strength [Resistance]*		
	Ω (ASD)	ϕ (LRFD)	Nominal Stress F_{nt}, ksi (MPa) [kg/cm^2]	Ω (ASD)	ϕ (LRFD)	Nominal Stress F_{nv}, ksi (MPa) [kg/cm^2]
A307 Bolts, Grade A 1/4 in. (6.4 mm) ≤ d <1/2 in. (12.7 mm)	2.25	0.75	40.5 (279) [2850]	2.4	0.65	24.0 (165) [1690]
A307 Bolts, Grade A d ≥ 1/2 in.	2.25		45.0 (310) [3160]			27.0 (186) [1900]
A325 bolts, when threads are not excluded from shear planes	2.0		90.0 (621) [6330]			54.0 (372) [3800]
A325 bolts, when threads are excluded from shear planes			90.0 (621) [6330]			72.0 (496) [5060]
A354 Grade BD Bolts 1/4 in. (6.4 mm) ≤ d < 1/2 in. (12.7 mm), when threads are not excluded from shear planes			101.0 (696) [7100]			59.0 (407) [4150]
A354 Grade BD Bolts 1/4 in. (6.4 mm) ≤ d < 1/2 in. (1.7 mm), when threads are excluded from shear planes			101.0 (696) [7100]			90.0 (621) [6330]
A449 Bolts 1/4 in. (6.4 mm) ≤ d < 1/2 in. (12.7 mm), when threads are not excluded from shear planes			81.0 (558) [5700]			47.0 (324) [3300]
A449 Bolts 1/4 in. (6.4 mm) ≤ d < 1/2 in. (12.7 mm), when threads are excluded from shear planes			81.0 (558) [5700]			72.0 (496) [5060]
A490 Bolts, when threads are not excluded from shear planes			112.5 (776) [7910]			67.5 (465) [4750]
A490 Bolts, when threads are excluded from shear planes			112.5 (776) [7910]			90.0 (621) [6330]

* Applies to bolts in holes as limited by Table E3a. 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 Chapter F.

TABLE E3.4-2 (ASD)
Nominal Tension Stress, F'_{nt} (ksi), for Bolts
Subjected to the Combination of Shear and Tension

Description of Bolts	Threads Not Excluded from Shear Planes	Threads Excluded from Shear Planes	Factor of Safety, Ω
A325 Bolts	$110 - 3.6f_V \le 90$	$110 - 2.8f_V \le 90$	2.0
A354 Grade BD Bolts	$122 - 3.6f_V \le 101$	$122 - 2.8f_V \le 101$	
A449 Bolts	$100 - 3.6f_V \le 81$	$100 - 2.8f_V \le 81$	
A490 Bolts	$136 - 3.6f_V \le 112.5$	$136 - 2.8f_V \le 112.5$	
A307 Bolts, Grade A			2.25
When 1/4 in. $\le d <$ 1/2 in.	$52 - 4f_V \le 40.5$		
When $d \ge$ 1/2 in.	$58.5 - 4f_V \le 45$		

The shear stress, f_V, shall also satisfy Table E3.4-1.

TABLE E3.4-3 (LRFD)
Nominal Tension Stress, F'_{nt} (ksi), for Bolts
Subjected to the Combination of Shear and Tension

Description of Bolts	Threads Not Excluded from Shear Planes	Threads Excluded from Shear Planes	Resistance Factor, ϕ
A325 Bolts	$113 - 2.4f_V \le 90$	$113 - 1.9f_V \le 90$	0.75
A354 Grade BD Bolts	$127 - 2.4f_V \le 101$	$127 - 1.9f_V \le 101$	
A449 Bolts	$101 - 2.4f_V \le 81$	$101 - 1.9f_V \le 81$	
A490 Bolts	$141 - 2.4f_V \le 112.5$	$141 - 1.9f_V \le 112.5$	
A307 Bolts, Grade A			0.75
When 1/4 in. $\le d <$ 1/2 in.	$47 – 2.4f_V \le 40.5$		
When $d \ge$ 1/2 in.	$52 – 2.4f_V \le 45$		

The shear stress, f_V, shall also satisfy Table E3.4-1.

TABLE E3.4-4 (ASD)
Nominal Tension Stress, F'_{nt} (MPa), for Bolts
Subjected to the Combination of Shear and Tension

Description of Bolts	Threads Not Excluded from Shear Planes	Threads Excluded from Shear Planes	Factor of Safety, ϕ
A325 Bolts	$758 - 3.6f_v \leq 621$	$758 - 2.8f_v \leq 621$	2.0
A354 Grade BD Bolts	$841 - 3.6f_v \leq 696$	$841 - 2.8f_v \leq 696$	
A449 Bolts	$690 - 3.6f_v \leq 558$	$690 - 2.8f_v \leq 558$	
A490 Bolts	$938 - 3.6f_v \leq 776$	$938 - 2.8f_v \leq 776$	
A307 Bolts, Grade A			2.25
When 6.4 mm ≤ d < 12.7 mm	$359 - 4f_v \leq 279$		
When d ≥ 12.7 mm	$403 - 4f_v \leq 310$		

The shear stress, f_v, shall also satisfy Table E3.4-1.

TABLE E3.4-5 (LRFD)
Nominal Tension Stress, F'_{nt} (MPa), for Bolts
Subjected to the Combination of Shear and Tension

Description of Bolts	Threads Not Excluded from Shear Planes	Threads Excluded from Shear Planes	Resistance Factor, ϕ
A325 Bolts	$779 - 2.4f_v \leq 621$	$779 - 1.9f_v \leq 621$	0.75
A354 Grade BD Bolts	$876 - 2.4f_v \leq 696$	$876 - 1.9f_v \leq 696$	
A449 Bolts	$696 - 2.4f_v \leq 558$	$696 - 1.9f_v \leq 558$	
A490 Bolts	$972 - 2.4f_v \leq 776$	$972 - 1.9f_v \leq 776$	
A307 Bolts, Grade A			0.75
When 6.4 mm ≤ d < 12.7 mm	$324 - 2.4f_v \leq 279$		
When d ≥ 12.7 mm	$359 - 2.4f_v \leq 310$		

The shear stress, f_v, shall also satisfy Table E3.4-1.

TABLE E3.4-6 (ASD)
Nominal Tension Stress, F'_{nt} (kg/cm^2), for Bolts
Subjected to the Combination of Shear and Tension

Description of Bolts	Threads Not Excluded from Shear Planes	Threads Excluded from Shear Planes	Factor of Safety, ϕ
A325 Bolts	$7730 - 3.6f_v \le 6330$	$7730 - 2.8f_v \le 6330$	2.0
A354 Grade BD Bolts	$8580 - 3.6f_v \le 7100$	$8580 - 2.8f_v \le 7100$	
A449 Bolts	$7030 - 3.6f_v \le 5700$	$7030 - 2.8f_v \le 5700$	
A490 Bolts	$9560 - 3.6f_v \le 7910$	$9560 - 2.8f_v \le 7910$	
A307 Bolts, Grade A			2.25
When $0.64\ cm \le d < 1.27\ cm$	$3660 - 4f_v \le 2850$		
When $d \ge 1.27\ cm$	$4110 - 4f_v \le 3160$		

The shear stress, f_v, shall also satisfy Table E3.4-1.

TABLE E3.4-7 (LRFD)
Nominal Tension Stress, F'_{nt} (kg/cm^2), for Bolts
Subjected to the Combination of Shear and Tension

Description of Bolts	Threads Not Excluded from Shear Planes	Threads Excluded from Shear Planes	Resistance Factor, ϕ
A325 Bolts	$7950 - 2.4f_v \le 6330$	$7950 - 1.9f_v \le 6330$	0.75
A354 Grade BD Bolts	$8930 - 2.4f_v \le 7100$	$8930 - 1.9f_v \le 7100$	
A449 Bolts	$7100 - 2.4f_v \le 5700$	$7100 - 1.9f_v \le 5700$	
A490 Bolts	$9910 - 2.4f_v \le 7910$	$9910 - 1.9f_v \le 7910$	
A307 Bolts, Grade A			0.75
When $0.64\ cm \le d < 1.27\ cm$	$3300 - 2.4f_v \le 2850$		
When $d \ge 1.27\ cm$	$3660 - 2.4f_v \le 3160$		

The shear stress, f_v, shall also satisfy Table E3.4-1.

E4.3.2 Connection Shear as Limited by End Distance

The nominal shear strength [resistance] per screw, P_{ns} shall not exceed that calculated as follows when the distance to an end of the connected part is parallel to the line of the applied force.

$$P_{ns} = teF_u \quad (Eq.\ E4.3.2\text{-}1)$$

Ω =3.00 (ASD)

ϕ =0.50 (LRFD)

where

t =Thickness of the part in which the end distance is measured

e =The distance measured in the line of force from the center of a standard hole to the nearest end of the connected part.

F_u =Tensile strength of the part in which the end distance is measured.

E5 Rupture

E5.1 Shear Rupture

At beam-end connections, where one or more flanges are coped and failure might occur along a plane through the fasteners, the nominal shear strength [resistance], V_n, shall be calculated as follows:

$$V_n = 0.6\,F_u A_{wn} \quad (Eq.\ E5.1\text{-}1)$$

Ω =2.00 (ASD)

ϕ =0.75 (LRFD)

where

$$A_{wn} = (h_{wc} - nd_h)t \quad (Eq.\ E5.1\text{-}2)$$

h_{wc} = Coped flat web depth

n = Number of holes in the critical plane

d_h = Hole diameter

F_u = Tensile strength of the connected part as specified in Section A2.1 or A2.2

t = Thickness of coped web

E5.2 Tension Rupture

The nominal tensile rupture strength [resistance] along a path in the affected elements of connected members shall be determined by Section E2.7 or E3.2 for welded or bolted connections, respectively.

E5.3 Block Shear Rupture

The nominal block shear rupture strength [resistance], R_n, shall be determined as follows:

(a) When $F_uA_{nt} \geq 0.6F_uA_{nv}$

$$R_n = 0.6F_yA_{gv} + F_uA_{nt} \quad (Eq.\ E5.3\text{-}1)$$

(b) When $F_uA_{nt} < 0.6F_uA_{nv}$

$$R_n = 0.6F_uA_{nv} + F_yA_{gt} \quad (Eq.\ E5.3\text{-}2)$$

For bolted connections:

Ω =2.22 (ASD)

ϕ =0.65 (LRFD)

For welded connections:

Ω =2.50 (ASD)

ϕ =0.60 (LRFD)

where

A_{gv} = Gross area subject to shear

A_{gt} = Gross area subject to tension

A_{nv} = Net area subject to shear

A_{nt} = Net area subject to tension