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American Iron and Steel Institute

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

2007 EDITION

Approved in Canada by the
Canadian Standards Association
CSA S136-07

Endorsed in Mexico by CANACERO

CANADIAN STANDARDS ASSOCIATION

CANACERO

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

The *North American Specification for the Design of Cold-Formed Steel Structural Members*, as its name implies, is intended for use throughout Canada, Mexico, and the United States. This *Specification* supersedes the 2001 edition of the *North American Cold-Formed Steel Specification,* the previous editions of the *Specification for the Design of Cold-Formed Steel Structural Members* published by the American Iron and Steel Institute, and the previous editions of CSA Standard S136, *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 Standards 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 effort was coordinated through the North American Specification Committee, which was made up of members from the AISI Committee on Specifications and CSA's S136 Committee.

Since the *Specification* is intended for use in Canada, Mexico, and the United States, it was necessary to develop a format that would allow for requirements particular to each country. This resulted in a main document, Chapters A through G and Appendix 1 and 2, that is intended for use in all three countries, and two country-specific appendices (A and B). In this edition of the *Specification*, what was previously Appendix C has been combined with Appendix A. The new Appendix A is for use in both the United States and Mexico, and Appendix B is for use in Canada. A symbol (☞A,B) is used in the main document to point out that additional provisions are provided in the corresponding appendices indicated by the letters.

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

The *Specification* also contains some terminology that is defined differently in Canada, the United States, and Mexico. These differences are set out in Section A1.3, "Definitions".

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 previous editions of the *Specification* developed by AISI and CSA.

The AISI and CSA consensus committees responsible for developing these provisions provide a balanced forum, with representatives of 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 and the United States. AISI, CSA, and CANACERO acknowledge the continuing dedication of the members of the specifications committees and their subcommittees. The membership of these committees follows this Preface.

In this edition of the *Specification*, the terminology jointly used by AISC and AISI is applied. Terms defined in Section A1.3 are italicized when they appear for the first time in each section. A new standard numbering system has been introduced for standards developed by AISI: for example, this *Specification* will be referred as AISI S100-07, where the last two digits represent the year that this standard is updated. All AISI test procedures are referenced by a number with the format "S9xx-yy", where "xx" is the sequence number, starting from "01", and "yy" is the year the test standard is developed or updated.

In addition, design provisions are reorganized according to their applicability to wall studs and wall stud assemblies (Section D4), floor, roof, or wall steel diaphragm construction (Section D5), and metal roof and wall systems (Section D6). Accordingly, provisions under Chapters C and D of previous editions are relocated.

The other major technical changes made in this edition of the *Specification*, compared to the previous edition are summarized below.

Materials

- Provisions for applications of other steels (Section A2.2) have been rewritten.

Strength

- Strength reduction provisions (Section A2.3.2) are introduced for high-strength and low-ductility closed-box section members.

Elements

- The effective width equation (Eq. B2.2-2) for uniformly compressed stiffened elements with circular holes has been revised.
- New provisions for unstiffened elements and edge stiffeners with stress gradient (Section B3.2) are introduced.
- The provisions for determining the effective width of uniformly compressed elements with one intermediate stiffener (previously in Section B4.1) have been replaced by the provisions of B5.1.

Members

- Provisions for distortional buckling for beams (Section C3.1.4) and columns (C4.2) are introduced.
- The design provisions for bearing stiffeners (previously termed "transverse stiffeners") have been revised.
- Provisions for web crippling strength for C- or Z-members with an overhang are added in Section C3.4.1.
- The equations for members subjected to combined bending and web crippling have been recalibrated.
- Provisions for considering combined bending and torsional loading (Section C3.6) are added.

Member Bracing

- Explicit equations for determining the required bracing force for members having neither flange connected to sheathing are provided.

- Provisions for determining the required bracing force and stiffness of a compression member are introduced.

Wall Stud and Wall Stud Assemblies

- The sheathing braced design provisions have been removed.
- New framing standards are referenced.

Floor, Roof, or Wall Steel Diaphragm Construction

- The safety factors and the resistance factors for diaphragms (Section D5) have been revised.

Metal Roof and Wall System

- New provisions for Z-section compression members having one flange fastened to a standing seam roof (Section D6.1.4) are added for the United States and Mexico.
- For standing seam roof panel systems, a load reduction is permitted in the United States and Mexico for load combinations that include wind uplift.
- The provisions for determining the anchorage forces and required stiffness for a purlin roof system under gravity load with the top flange connected to metal sheathing have been revised.

Connections

- Provisions for shear strength determination of welded sheet-to-sheet connections are added.
- An interaction check for screws subjected to combined shear and pull-over is added.
- The design provisions for block shear rupture (Section E5.3) have been revised.

Appendix B

- The section for delivered minimum thickness for Canada is deleted.
- The specified loads (Section A3.1) and the load factors and load combinations for LSD (Section A6.1.2) for Canada have been revised.

New Appendices

- Appendix 1, Design of Cold-Formed Steel Structural Members Using the Direct Strength Method, is added. The Direct Strength Method provides alternative design provisions for several sections of Chapters C and D.
- Appendix 2, Second Order Analysis, is added. Appendix 2 provides alternative method for considering the second order effect in members subjected to compression and bending.

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

American Iron and Steel Institute
Canadian Standards Association
Camara Nacional de la Industria del Hierro y del Acero
July 2007

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SYMBOLS AND DEFINITIONS

Symbol	Definition	Section
A	Full unreduced cross-sectional area of member	A1.3, C3.1.2.1, C4.1.2, C5.2.1, C5.2.2, C4.1.5, D6.1.3, D6.1.4, 2.2.3
A	Area of directly connected elements or gross area	E2.7
A_b	$b_1t + A_s$, for bearing stiffener at interior support and or under concentrated load, and $b_2t + A_s$, for bearing stiffeners at end support	C3.7.1
A_b	Gross cross-sectional area of bolt	E3.4
A_c	$18t^2 + A_s$, for bearing stiffener at interior support or under concentrated load, and $10t^2 + A_s$, for bearing stiffeners at end support	C3.7.1
A_e	Effective area at stress F_n	A1.3, C3.7.1, C3.7.2, C4.1, C4.1.2, C5.2.1, C5.2.2, C4.1.5
A_e	Effective net area	E2.7, E3.2
A_f	Cross-sectional area of compression flange plus edge stiffener	C3.1.4
A_g	Gross area of element including stiffeners	B5.1
A_g	Gross area of section	A1.3, C2, C2.1, C4.2, E2.7, E3.2, 1.2.1.1
A_{gv}	Gross area subject to shear	E5.3
A_{nt}	Net area subject to tension	E5.3
A_{nv}	Net area subject to shear	E5.3
A_n	Net area of cross-section	A1.3, C2, C2.2, E3.2
A_o	Reduced area due to local buckling	C4.1.5
A_p	Gross cross-sectional area of roof panel per unit width	D6.3.1
A_s	Cross-sectional area of bearing stiffener	C3.7.1
A_s	Gross area of stiffener	B5.1
A_{st}	Gross area of shear stiffener	C3.7.3
A_t	Net tensile area	G4
A_w	Area of web	C3.2.1
A_{wn}	Net web area	E5.1
a	Shear panel length of unreinforced web element, or distance between shear stiffeners of reinforced web elements	C3.2.1, C3.7.3
a	Intermediate fastener or spot weld spacing	D1.2
a	Fastener distance from outside web edge	D6.1.3
a	Length of bracing interval	D3.2.1

SYMBOLS AND DEFINITIONS

Symbol	Definition	Section
B_c	Term for determining tensile yield point of corners	A7.2
b	Effective design width of compression element	B2.1, B2.2, B3.1, B3.2, B4
b	Flange width	D6.1.3, D6.3.1
b_d	Effective width for deflection calculation	B2.1, B2.2, B3.1, B3.2, B4, B5.2
b_e	Effective width of elements, located at centroid of element including stiffeners	B5.1
b_e	Effective width	B2.3
b_e	Effective width determined either by Section B4 or Section B5.1 depending on stiffness of stiffeners	B5.2
b_o	Out-to-out width of compression flange as defined in Figure B2.3-2	B2.3, C3.1.4, C4.2
b_o	Overall width of unstiffened element as defined in Figure B3.2-3	B3.2
b_o	Total flat width of stiffened element	B5.1
b_o	Total flat width of edge stiffened element	B5.2, 1.1.1.1, 1.1.1.2
b_p	Largest sub-element flat width	B5.1
b_1, b_2	Effective widths	B2.3, B2.4
b_1, b_2	Effective widths of bearing stiffeners	C3.7.1
C	For compression members, ratio of total corner cross-sectional area to total cross-sectional area of full section; for flexural members, ratio of total corner cross-sectional area of controlling flange to full cross-sectional area of controlling flange	A7.2
C	Coefficient	C3.4.1
C	Bearing factor	E3.3.1
C_b	Bending coefficient dependent on moment gradient	C3.1.2.1, C3.1.2.2
C_f	Constant from Table G1	G1, G3, G4
C_h	Web slenderness coefficient	C3.4.1
C_m	End moment coefficient in interaction formula	C5.2.1, C5.2.2
C_{mx}	End moment coefficient in interaction formula	C5.2.1, C5.2.2, 2.1
C_{my}	End moment coefficient in interaction formula	C5.2.1, C5.2.2, 2.1
C_N	Bearing length coefficient	C3.4.1
C_p	Correction factor	F1.1
C_R	Inside bend radius coefficient	C3.4.1
C_s	Coefficient for lateral-torsional buckling	C3.1.2.1
C_{TF}	End moment coefficient in interaction formula	C3.1.2.1
C_v	Shear stiffener coefficient	C3.7.3

SYMBOLS AND DEFINITIONS

Symbol	Definition	Section
C_w	Torsional warping constant of cross-section	C3.1.2.1
C_{wf}	Torsional warping constant of flange	C3.1.4, C4.2
C_y	Compression strain factor	C3.1.1
C_1, C_2, C_3	Axial buckling coefficients	D6.1.3
C1 to C6	Coefficients tabulated in Tables D6.3.1-1 to D6.3.1-3	D6.3.1
C_ϕ	Calibration coefficient	F1.1
c	Strip of flat width adjacent to hole	B2.2
c	Distance	C3.2.2
c_f	Amount of curling displacement	B1.1
c_i	Horizontal distance from edge of element to centerline of stiffener	B5.1, B5.1.2
D	Outside diameter of cylindrical tube	C6, C3.1.3, C4.1.5
D	Overall depth of lip	B1.1, B4, C3.1.4, C4.2, 1.1.1.1, 1.1.1.2
D	Shear stiffener coefficient	C3.7.3
D	Dead load	A3.1, A6.1.2
D_2, D_3	Lip dimension	1.1.1.1, 1.1.1.2
d	Depth of section	B1.1, C3.1.2.1, C3.4.2, C3.7.2, D3.2.1, D6.1.1, D6.1.3, D6.1.4, D6.3.1
d	Nominal screw diameter	E4, E4.1, E4.2, E4.3.1, E4.4.1, E4.5.1, E4.5.2
d	Flat depth of lip defined in Figure B4-1	B4
d	Width of arc seam weld	E2.3
d	Visible diameter of outer surface of arc spot weld	E2.2.1.1, E2.2.1.2, E2.2.1.3, E2.2.2
d	Diameter of bolt	E3a, E3.2, E3.3.1, E3.3.2, E3.4
d_a	Average diameter of arc spot weld at mid-thickness of t	E2.2.1.2, E2.2.1.3, E2.2.2
d_a	Average width of seam weld	E2.3
d_b	Nominal diameter (body or shank diameter)	G4
d_e	Effective diameter of fused area	E2.2, E2.2.1.2, E2.2.1.3, E2.2.2
d_e	Effective width of arc seam weld at fused surfaces	E2.3
d_h	Diameter of hole	B2.2
d_h	Depth of hole	B2.2, B2.4, C3.2.2, C3.4.2

SYMBOLS AND DEFINITIONS

Symbol	Definition	Section
d_h	Diameter of standard hole	E3a, E3.1, E3.2, E5.1
$d_{p_{i,j}}$	Distance along roof slope between the ith purlin line and the jth anchorage device	D6.3.1
d_s	Reduced effective width of stiffener	B4
d_s	Depth of stiffener	1.1.1.2
d'_s	Effective width of stiffener calculated according to B3.1	B4
d_{wx}	Screw head or washer diameter	E4.4
d_w	Larger value of screw head or washer diameter	E4, E4.4, E4.4.2, E4.5.1, E4.5.2
E	Modulus of elasticity of steel, 29,500 ksi (203,000 MPa, or 2,070,000 kg/cm^2)	A2.3.2, B1.1, B2.1, B4, B5.1, C3.1.1, C3.1.2.1, C3.1.2.2, C3.1.4, C3.2.1, C3.5.1, C3.5.2, C3.7.1, C3.7.3, C4.1.1, C4.2, C5.2.1, C5.2.2, C3.1.3, C4.1.5, D1.3, D6.1.3, D6.3.1, E2.2.1.2, 1.1.1.1, 1.1.1.2, 2.2.3
E	Live load due to earthquake twist of stud from initial, ideal, unbuckled shape	A3.1, A6.1.2, A6.1.2.1
E*	Reduced modulus of elasticity for flexural and axial stiffness in second-order analysis	2.2.3
e	Distance measured in line of force from center of a standard hole to nearest edge of an adjacent hole or to end of connected part toward which force is directed	E3.1, E3.1a
e	Distance measured in line of force from center of a standard hole to nearest end of connected part	E4.3.2
e_{min}	Minimum allowable distance measured in line of force from centerline of a weld to nearest edge of an adjacent weld or to end of connected part toward which the force is directed	E2.2.1.1, E2.2.2
e_{sx}, e_{sy}	Eccentricities of load components measured from the shear center and in the x- and y- directions, respectively	D3.2.1
e_y	Yield strain = F_y/E	C3.1.1
F	Fabrication factor	F1.1
F_{SR}	Design stress range	G3
F_{TH}	Threshold fatigue stress range	G1, G3, G4
F_c	Critical buckling stress	B2.1, C3.1.2.1, C3.1.3

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Symbol	Definition	Section
F_{cr}	Plate elastic buckling stress	A2.3.2, B2.1, B5.1
F_d	Elastic distortional buckling stress	C3.1.4, C4.2
F_e	Elastic buckling stress	C3.1.2.1, C3.1.2.2, C4.1, C4.1.1, C4.1.2, C4.1.3, C4.1.4, C4.1.5
F_m	Mean value of fabrication factor	D6.2.1, F1.1
F_n	Nominal buckling stress	B2.1, C4.1, C5.2.1, C5.2.2
F_n	Nominal strength [resistance] of bolts	E3.4
F_{nt}	Nominal tensile strength [resistance] of bolts	E3.4
F_{nv}	Nominal shear strength [resistance] of bolts	E3.4
F'_{nt}	Nominal tensile strength [resistance] for bolts subject to combination of shear and tension	E3.4
F_{sy}	Yield stress as specified in Section A2.1, A2.2, or A2.3.2	A2.3.2, E2.2.1.1, E3.1
F_t	Nominal tensile stress in flat sheet	E3.2
F_u	Tensile strength as specified in Section A2.1, A2.2, or A2.3.2	A2.3.2, C2, C2.2, E2.2.1.1, E2.2.1.2, E2.2.1.3, 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
F_{uv}	Tensile strength of virgin steel specified by Section A2 or established in accordance with Section F3.3	A7.2
F_{wy}	Lower value of F_y for beam web or F_{ys} for bearing stiffeners	C3.7.1
F_{xx}	Tensile strength of electrode classification	E2.1, E2.2.1.2, E2.2.1.3, E2.2.2, E2.3, E2.4, E2.5
F_{u1}	Tensile strength of member in contact with screw head	E4, E4.3.1, E4.4.2, E4.5.1, E4.5.2
F_{u2}	Tensile strength of member not in contact with screw head	E4, E4.3.1, E4.4.1
F_v	Nominal shear stress	E3.2.1
F_y	Yield stress used for design, not to exceed specified yield stress 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	A2.3.2, A7.1, A7.2, B2.1, C2, C2.1, C3.1.1, C3.1.2.1, C3.1.2.2, D6.1.1, C3.2.1, C3.4.1, C3.5.1, C3.5.2, A2.3.2, C3.7.1, C3.7.2, C3.7.3, C4.1, C4.1.2, C4.2, C5.1.1,C5.2.1, C5.2.2, C6, C3.1.3, C4.1.5, C5.1.2, D1.3, D6.1.2, D6.1.4, E2.1, E2.2.2, E5.2, G1, 1.1.1.1, 1.1.1.2, 1.2.1.1, 2.2.3
F_{ya}	Average yield stress of section	A7.2
F_{yc}	Tensile yield stress of corners	A7.2
F_{yf}	Weighted average tensile yield stress of flat portions	A7.2, F3.2

SYMBOLS AND DEFINITIONS

Symbol	Definition	Section
F_{ys}	Yield stress of stiffener steel	C3.7.1
F_{yv}	Tensile yield stress of virgin steel specified by Section A2 or established in accordance with Section F3.3	A7.2
f	Stress in compression element computed on basis of effective design width	B2.1, B2.2, B2.4, B3.1, B3.2, B4, B5.1, B5.1.1, B5.1.2, B5.2
f_{av}	Average computed stress in full unreduced flange width	B1.1
f_c	Stress at service load in cover plate or sheet	D1.3
$f_{bending}$	Normal stress due to bending alone at the maximum normal stress on the cross section due to combined bending and torsion	C3.6
$f_{torsion}$	Normal stress due to torsion alone at the maximum normal stress on the cross section due to combined bending and torsion	C3.6
f_d	Computed compressive stress in element being considered. Calculations are based on effective section at load for which deflections are determined.	B2.1, B2.2, B3.1, B4, 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 effective section at load for which serviceability is determined.	B2.3
f_{d1}, f_{d2}	Computed stresses f_1 and f_2 in unstiffened element, as defined in Figures B3.2-1 to B3.2-3. Calculations are based on effective section at load for which serviceability is determined.	B3.2
f_v	Required 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, f_2	Stresses on unstiffened element defined by Figures B3.2-1 to B3.2-3	B3.2
f_1, f_2	Stresses at the opposite ends of web	C3.1.4
G	Shear modulus of steel, 11,300 ksi (78,000 MPa or 795,000 kg/cm^2)	C3.1.2.1, C3.1.2.2, C3.1.4
g	Vertical distance between two rows of connections nearest to top and bottom flanges	D1.1
g	Transverse center-to-center spacing between fastener gage lines	E3.2
g	Gauge, spacing of fastener perpendicular to force	C2.2

SYMBOLS AND DEFINITIONS

Symbol	Definition	Section
H	A permanent load due to lateral earth pressure, including groundwater	A3.1, A3.2
h	Depth of flat portion of web measured along 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, C3.7.3
h	Width of elements adjoining stiffened element	B5.1
h	Lip height as defined in Figures E2.5-4 to E2.5-7	E2.5
h_o	Out-to-out depth of web	B2.3, C3.1.4, C4.2, 1.1.1.1, 1.1.1.2
h_o	Overall depth of unstiffened C-section member as defined in Figure B3.2-3	B3.2
h_s	Depth of soil supported by the structure	A6.1.2
h_{wc}	Coped flat web depth	E5.1
h_x	x distance from the centroid of flange to the shear center of the flange	C3.1.4
I_E	Importance factor for earthquake	A6.1.2.2
I_S	Importance factor for snow	A6.1.2.2
I_W	Importance factor for wind	A6.1.2.2
I_a	Adequate moment of inertia of stiffener, so that each component element will behave as a stiffened element	B1.1, B4
I_{eff}	Effective moment of inertia	1.1.3
I_g	Gross moment of inertia	1.1.3
I_s	Actual moment of inertia of full stiffener about its own centroidal axis parallel to element to be stiffened	B1.1, B4, C3.7.3
I_{smin}	Minimum moment of inertia of shear stiffener(s) with respect to an axis in plane of web	C3.7.3
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 unreduced section about principal axis	C3.1.2.1, C3.1.2.2, C5.2.1, C5.2.2, D3.2.1, D6.3.1
I_{xf}	x-axis moment of inertia of the flange	C3.1.4, C4.2
I_{xy}	Product of inertia of full unreduced section about major and minor centroidal axes	D3.2.1, D6.3.1
I_{xyf}	Product of inertia of flange about major and minor centroidal axes	C3.1.4, C4.2
I_{yc}	Moment of inertia of compression portion of section about centroidal axis of entire section parallel to web,	C3.1.2.1

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Symbol	Definition	Section
	using full unreduced section	
I_{yf}	y-axis moment of inertia of flange	C3.1.4, C4.2
i	Index of stiffener	B5.1, B5.1.2
i	Index of each purlin line	D6.3.1
J	Saint-Venant torsion constant	C3.1.2.1, C3.1.2.2
J_f	Saint-Venant torsion constant of compression flange, plus edge stiffener about an x-y axis located at the centroid of the flange	C3.1.4
j	Section property for torsional-flexural buckling	C3.1.2.1
j	Index for each anchorage device	D6.3.1
K	Effective length factor	A2.3.2, C4.1.1, D1.2
K′	A constant	D3.2.1
K_a	Lateral stiffness of anchorage device	D6.3.1
K_{af}	Parameter for determining axial strength of Z-Section member having one flange fastened to sheathing	D6.1.4
$K_{eff_{i,j}}$	Effective lateral stiffness of jth anchorage device with respect to ith purlin	D6.3.1
K_{req}	Required stiffness	D6.3.1
K_{sys}	Lateral stiffness of roof system, neglecting anchorage devices	D6.3.1
K_t	Effective length factor for torsion	C3.1.2.1
K_{total_i}	Effective lateral stiffness of all elements resisting force P_i	D6.3.1
K_x	Effective length factor for buckling about x-axis	C3.1.2.1, C5.2.1, C5.2.2, 2.1
K_y	Effective length factor for buckling about y-axis	C3.1.2.1, C3.1.2.2, C5.2.1, C5.2.2, 2.1
k	Plate buckling coefficient	B2.1, B2.2, B2.3, B3.1, B3.2, B4, B5.1, B5.2
k_d	Plate buckling coefficient for distortional buckling	B5.1, B5.1.1, B5.1.2, C3.1.4, C4.2
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, C3.7.3,
k_ϕ	Rotational stiffness	C3.1.4, C4.2
$k_{\phi fe}$	Elastic rotational stiffness provided by the flange to the flange/web juncture	C3.1.4, C4.2

SYMBOLS AND DEFINITIONS

Symbol	Definition	Section
$\tilde{k}_{\phi fg}$	Geometric rotational stiffness demanded by the flange from the flange/web juncture	C3.1.4, C4.2
$k_{\phi we}$	Elastic rotational stiffness provided by the web to the flange/web juncture	C3.1.4, C4.2
$\tilde{k}_{\phi wg}$	Geometric rotational stiffness demanded by the web from the flange/web juncture	C3.1.4, C4.2
L	Full span for simple beams, distance between inflection point for continuous beams, twice member length for cantilever beams	B1.1
L	Span length	D6.3.1, D1.1
L	Length of weld	E2.1, E2.5
L	Length of longitudinal welds	E2.7
L	Length of seam weld not including circular ends	E2.3
L	Length of fillet weld	E2.4
L	Length of connection	E3.2
L	Unbraced length of member	C4.1.1, D1.2, C5.2.1, C5.2.2
L	Overall length	A2.3.2
L	Live load	A3.1, A6.1.2, A6.1.2.1
L	Minimum of L_{cr} and L_m	C3.1.4, C4.2
L_b	Distance between braces on one compression member	D3.3
L_{br}	Unsupported length between brace points or other restraints which restrict distortional buckling of element	B5.1, B5.1.1, B5.1.2
L_c	Summation of critical path lengths of each segment	C2.2
L_{cr}	Critical unbraced length of distortional buckling	C3.1.4, C4.2
L_{gv}	Gross failure path length parallel to force	C2.2
L_h	Length of hole	B2.2, B2.4, C3.2.2, C3.4.2
L_m	Distance between discrete restraints that restrict distortional buckling	C3.1.4, C4.2
L_{nv}	Net failure path length parallel to force	C2.2
L_o	Overhang length measured from the edge of bearing to the end of member	C3.4.1
L_s	Net failure path length inclined to force	C2.2
L_{st}	Length of bearing stiffener	C3.7.1
L_t	Unbraced length of compression member for torsion	C3.1.2.1
L_t	Net failure path length normal to force due to direct tension	C2.2

SYMBOLS AND DEFINITIONS

Symbol	Definition	Section
L_u	Limit of unbraced length below which lateral-torsional buckling is not considered	C3.1.2.2
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_0	Length at which local buckling stress equals flexural buckling stress	A2.3.2
l	Distance from concentrated load to a brace	D3.2.1
M	Required allowable flexural strength, ASD	C3.3.1, C3.5.1
M	Bending moment	1.1.3
M_{crd}	Distortional buckling moment	C3.1.4, 1.1.2, 1.2.2.3
M_{cre}	Overall buckling moment	1.1.2, 1.2.2.1
$M_{cr\ell}$	Local buckling moment	1.1.2, 1.2.2.2
M_d	Nominal moment with consideration of deflection	1.1.3
M_f	Factored moment	C3.3.2
M_{fx}, M_{fy}	Moments due to factored loads with respect to centroidal axes	C4.1, C5.1.2, C5.2.2
M_m	Mean value of material factor	D6.2.1, F1.1
M_{max}, M_A, M_B, M_C	Absolute value of moments in unbraced segment, used for determining C_b	C3.1.2.1
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, D6.1.1, D6.1.2, 1.1.1, 1.1.3, 1.2.2
M_{nd}	Nominal flexural strength for distortional buckling	1.2.2, 1.2.2.3
M_{ne}	Nominal flexural strength for overall buckling	1.2.2, 1.2.2.1, 1.2.2.2
$M_{n\ell}$	Nominal flexural strength for local buckling	1.2.2, 1.2.2.2
M_{nx}, M_{ny}	Nominal flexural strengths [resistances] about centroidal axes determined in accordance with Section C3	C5.1.1, C5.1.2, C5.2.1, C5.2.2
M_{nxo}, M_{nyo}	Nominal flexural strengths [resistances] about centroidal axes determined in accordance with Section C3.1 excluding provisions of Section C3.1.2	C3.3.1, C3.3.2, C3.5.1, C3.5.2
M_{nxt},	Nominal flexural strengths [resistances] about	C5.1.1, C5.1.2

SYMBOLS AND DEFINITIONS

Symbol	Definition	Section
M_{nyt}	centroidal axes determined using gross, unreduced cross-section properties	
M_x, M_y	Required allowable flexural strength with respect to centroidal axes for ASD	C4.1, 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	C4.1, C5.1.2, C5.2.2
M_y	Moment causing maximum strain e_y	B2.1, C3.1.2
M_y	Yield moment ($=S_fF_y$)	C3.1.4, 1.1.3, 1.2.2.1, 1.2.2.3
M_1	Smaller end moment in an unbraced segment	C3.1.2.1, C3.1.4, C5.2.1, C5.2.2
M_2	Larger end moment in an unbraced segment	C3.1.2.1, C3.1.4, C5.2.1, C5.2.2
$\overline{M}$	Required flexural strength [factored moment]	C3.3.2, C3.5.2
$\overline{M}_x$, $\overline{M}_y$	Required flexural strengths [factored moments]	C4.1, C5.1.2
M_z	Torsional moment of required load P about shear center	D3.2.1
m	Degrees of freedom	F1.1
m	Term for determining tensile yield point of corners	A7.2
m	Distance from shear center of one C-section to mid-plane of web	D1.1, D3.2.1, D6.3.1
m_f	Modification factor for type of bearing connection	E3.3.1
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_a	Number of anchorage devices along a line of anchorage	D6.3.1
N_i	Notional lateral load applied at level i	2.2.4
N_p	Number of purlin lines on roof slope	D6.3.1
n	Coefficient	B4
n	Number of stiffeners	B5.1, B5.1.1, B5.1.2, 1.1.1.2
n	Number of holes	E5.1
n	Number of tests	F1.1
n	Number of equally spaced intermediate brace locations	D3.3
n	Number of anchors in test assembly with same tributary area (for anchor failure), or number of panels with identical spans and loading to failed span (for non-anchor failure)	D6.2.1
n	Number of threads per inch	G4

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Symbol	Definition	Section
n_b	Number of bolt holes	E3.2
n_c	Number of compression flange stiffeners	1.1.1.2
n_w	Number of web stiffeners and/or folds	1.1.1.2
n_t	Number of tension flange stiffeners	1.1.1.2
P	Required allowable strength for concentrated load reaction in presence of bending moment for ASD	C3.5.1
P	Required allowable strength (nominal force) transmitted by weld for ASD	E2.2.1.1
P	Required allowable compressive axial strength for ASD	A2.3.1, C5.2.1
P	Professional factor	F1.1
P	Required concentrated load [factored load] within a distance of 0.3a on each side of a brace, plus 1.4(1-l/a) times each required concentrated load located farther than 0.3a but not farther than 1.0a from the brace	D3.2.1
P	Required nominal brace strength [resistance] for a single compression member	D3.3
P_{Ex}, P_{Ey}	Elastic buckling strengths [resistances]	C5.2.1, C5.2.2
P_{L1}, P_{L2}	Lateral bracing forces	D3.2.1
P_{L_j}	Lateral force to be resisted by the jth anchorage device	D6.3.1
P_{crd}	Distortional buckling load	C4.2, 1.1.2, 1.2.1.3
P_{cre}	Overall buckling load	1.1.2, 1.2.1.1
$P_{cr\ell}$	Local buckling load	1.1.2, 1.2.1.2
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	E2.2.1.1
P_i	Lateral force introduced into the system at the ith purlin	D6.3.1
P_m	Mean value of the tested-to-predicted load ratios	F1.1
P_n	Nominal web crippling strength [resistance]	C3.4.1, C3.5.1, C3.5.2, A2.3.1,
P_n	Nominal axial strength [resistance] of member	C4.1, C4.2, C5.2.1, C5.2.2, D3.3, D6.1.3, D6.1.4, 1.1.1, 1.2.1, 2.1
P_n	Nominal axial strength [resistance] of bearing stiffener	C3.7.1, C3.7.2
P_n	Nominal strength [resistance] of connection component	E2.1, E2.2.1.2, E2.2.1.3, E2.2.2, E2.3, E2.4, E2.5, E2.6, E3.1, E3.2
P_n	Nominal bearing strength [resistance]	E3.3.1, E3.3.2
P_n	Nominal tensile strength [resistance] of welded member	E2.7

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Symbol	Definition	Section
P_n	Nominal bolt strength [resistance]	E3.4
P_{nc}	Nominal web crippling strength [resistance] of C- or Z-Section with overhang(s)	C3.4.1
P_{nd}	Nominal axial strength for distortional buckling	1.2.1, 1.2.1.3
P_{ne}	Nominal axial strength for overall buckling	1.2.1, 1.2.1.1, 1.2.1.2
$P_{n\ell}$	Nominal axial strength for local buckling	1.2.1, 1.2.1.2
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, E4.5.1, E4.5.2
P_{ns}	Nominal shear strength [resistance] per screw	E4, E4.2, E4.3.1, E4.3.2, E4.3.3, E4.5.1, E4.5.2
P_{nt}	Nominal tension strength [resistance] per screw	E4, E4.4.3
P_r	Required axial compressive strength [resistance]	2.2.3
P_s	Concentrated load or reaction	D1.1
P_{ss}	Nominal shear strength [resistance] of screw as reported by manufacturer or determined by independent laboratory testing	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 for LRFD	A2.3.1, C5.2.2
P_u	Factored force (required strength) transmitted by weld, for LRFD	E2.2.1.1
P_u	Required strength for concentrated load or reaction in presence of bending moment for LRFD	C3.5.2
P_{wc}	Nominal web crippling strength [resistance] for C-Section flexural member	C3.7.2
P_x, P_y	Components of required load P parallel to x and y axis, respectively	D3.2.1
P_y	Member yield strength	C4.2, 1.2.1.1, 1.2.1.3, 2.2.3
$\overline{P}$	Required strength for concentrated load or reaction [concentrated load or reaction due to factored loads] in presence of bending moment.	C3.5.2
$\overline{P}$	Required compressive axial strength [factored compressive force]	C5.2.2
p	Pitch (mm per thread for SI units and cm per thread for MKS units)	G4

SYMBOLS AND DEFINITIONS

Symbol	Definition	Section
Q	Required allowable shear strength of connection	E4.5.1
$\overline{Q}$	Required shear strength [factored shear force] of connection	E4.5.2
Q_i	Load effect	F1.1
q	Design load in plane of 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	C3.6
R	Reduction factor	D6.1.1
R	Reduction factor determined in accordance with AISI S908	D6.1.2
R	Reduction factor determined from uplift tests in accordance with AISI S908	D6.1.4
R	Coefficient	C4.1.5
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
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]	A4.1.1, A5.1.1, A6.1.1, F2
R_n	Nominal block shear rupture strength [resistance]	E5.3
R_n	Average value of all test results	F1.1, F1.2
R_r	Reduction factor	A2.3.2
R_u	Required strength for LRFD	A5.1.1
r	Correction factor	D6.1.1
r	Least radius of gyration of full unreduced cross-section	A2.3.2, C4.1.1, C4.1.2, D1.2
r	Centerline bend radius	1.1.1.1, 1.1.1.2
r_i	Minimum radius of gyration of full unreduced cross-section	D1.2
r_o	Polar radius of gyration of cross-section about shear center	C3.1.2.1, C4.1.2

SYMBOLS AND DEFINITIONS

Symbol	Definition	Section
r_x, r_y	Radius of gyration of cross-section about centroidal principal axis	C3.1.2.1
S	$1.28\sqrt{E/f}$	B4, B5.2
S	Variable load due to snow, including ice and associated rain or rain	A3.1, A6.1.2, A6.1.2.1
S_c	Elastic section modulus of effective section calculated relative to extreme compression fiber at F_c	B2.1, C3.1.2.1
S_e	Elastic section modulus of effective section calculated relative to extreme compression or tension fiber at F_y	C3.1.1, D6.1.1, D6.1.2
S_f	Elastic section modulus of full unreduced section relative to extreme compression fiber	B2.1, C3.1.2.1, C3.1.2.2, C3.1.3, C3.1.4
S_{ft}	Section modulus of full unreduced section relative to extreme tension fiber about appropriate axis	C5.1.1, C5.1.2
S_{fy}	Elastic section modulus of full unreduced section relative to extreme fiber in first yield	C3.1.4
S_n	In-plane diaphragm nominal shear strength [resistance]	D5
s	Center-to-center hole spacing	B2.2
s	Spacing in line of stress of welds, rivets, or bolts connecting a compression cover plate or sheet to a non-integral stiffener or other element	D1.3
s	Sheet width divided by number of bolt holes in cross-section being analyzed	E3.2
s	Weld spacing	D1.1
s	Pitch, spacing of fastener parallel to force	C2.2
s′	Longitudinal center-to-center spacing of any consecutive holes	E3.2
s_{end}	Clear distance from the hole at ends of member	B2.2
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	Required allowable tension strength of connection	E4.5.1
T	Load due to contraction or expansion caused by temperature changes	A3.1, A3.2
T_f	Tension due to factored loads for LSD	C5.1.2
T_f	Factored tensile force of connection for LSD	E4.5.2

SYMBOLS AND DEFINITIONS

Symbol	Definition	Section
T_n	Nominal tensile strength [resistance]	C2, C2.1, C2.2, 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
T_u	Required tension strength of connection for LRFD	E4.5.2
$\overline{T}$	Required tensile axial strength [factored tensile force]	C5.1.2
$\overline{T}$	Required tension strength [factored tensile force] of connection	E4.5.2
t	Base steel thickness of any element or section	A1.3, A2.3.2, A2.4, A7.2, B1.1, B1.2, B2.1, B2.2, B2.4, B3.2, B4, B5.1, B5.1.1, B5.1.2, B5.2, C2.2, C3.1.1, C3.1.3, C3.1.4, C3.2.1, C3.2.2, C3.4.1, C3.4.2, C3.5.1, C3.5.2, C3.7.1, C3.7.3, C4.2, C6, C4.1.5, D1.3, D6.1.3, D6.1.4, D6.3.1, E3.3.1, E3.3.2, E4.3.2, 1.1.1.1, 1.1.1.2
t	Thickness of coped web	E5.1
t	Total thickness of two welded sheets	E2.2.1.1, E2.2.1.2, E2.2.1.3, 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_c	Lesser of depth of penetration and t_2	E4, E4.4.1
t_e	Effective throat dimension of groove weld	E2.1
t_i	Thickness of uncompressed glass fiber blanket insulation	D6.1.1
t_s	Thickness of stiffener	C3.7.1
t_w	Effective throat of weld	E2.4, E2.5
t_1, t_2	Based thicknesses connected with fillet weld	E2.4
t_1	Thickness of member in contact with screw head	E4, E4.3.1, E4.4.2, E4.5.1, E4.5.2
t_2	Thickness of member not in contact with screw head	E4, E4.3.1, E4.5.1, E4.5.2
U	Reduction coefficient	E2.7, E3.2
V	Required allowable shear strength for ASD	C3.3.1
V_F	Coefficient of variation of fabrication factor	D6.2.1, F1.1
V_f	Shear force due to factored loads for LSD	C3.3.2
V_f	factored shear force of connection for LSD	E4.5.2
V_M	Coefficient of variation of material factor	D6.2.1, F1.1

SYMBOLS AND DEFINITIONS

Symbol	Definition	Section
V_n	Nominal shear strength [resistance]	C3.2.1, C3.3.1, C3.3.2, E5.1
V_P	Coefficient of variation of tested-to-predicted load ratios	D6.2.1, F1.1
V_Q	Coefficient of variation of load effect	D6.2.1, F1.1
V_u	Required shear strength for LRFD	C3.3.2
V_u	Required shear strength of connection for LRFD	E4.5.2
$\overline{V}$	Required shear strength [factored shear]	C3.3.2
W	Wind load, a variable load due to wind	A3.1, A6.1.2, A6.1.2.1
W	Required strength from critical load combinations for ASD, LRFD, or LSD	D3.2.1
W_{pi}	Total required vertical load supported by ith purlin in a single bay	D6.3.1
W_x, W_y	Components of required strength W	D3.2.1
w	Flat width of element exclusive of radii	A2.3.2, B1.1, B2.1, B2.2, B3.1, B3.2, B4, C3.1.1, C3.7.1
w	Flat width of beam flange which contacts bearing plate	C3.5.1, C3.5.2
w	Flat width of narrowest unstiffened compression element tributary to connections	D1.3
w_f	Width of flange projection beyond web for I-beams and similar sections; or half distance between webs for box- or U-type sections	B1.1
w_i	Required distributed gravity load supported by the ith purlin per unit length	D6.3.1
w_o	Out-to-out width	B2.2
w_1	Leg of weld	E2.4, E2.5
w_2	Leg of weld	E2.4, E2.5
x	Non-dimensional fastener location	D6.1.3
x	Nearest distance between web hole and edge of bearing	C3.4.2
x_o	Distance from shear center to centroid along principal x-axis	C3.1.2.1, C4.1.2
x_o	Distance from flange/web junction to the centroid of the flange	C3.1.4, C4.2
$\bar{x}$	Distance from shear plane to centroid of cross-section	E2.7, E3.2

SYMBOLS AND DEFINITIONS

Symbol	Definition	Section
Y	Yield point of web steel divided by yield point of stiffener steel	C3.7.3
Y_i	Gravity load from the LRFD or LSD load combinations or 1.6 times the ASD load combinations applied at level i	2.2.3, 2.2.4
y_o	y distance from flange/web junction to the centroid of the flange	C3.1.4
α	Coefficient for purlin directions	D6.3.1
α	Coefficient for conversion of units	D6.1.3, E3.3.2, G3
α	Load factor	A1.2a
α	Coefficient for strength [resistance] increase due to due to overhang	C3.4.1
α	Coefficient accounts for the benefit of an unbraced length, L_m, shorter than L_{cr}.	C4.2
α	Second-order amplification coefficient	2.2.3
$1/\alpha_x$, $1/\alpha_y$	Magnification factors	C5.2.1, C5.2.2, 2.1
β	Coefficient	B5.1.1, B5.1.2, C4.1.2
β	A value accounting for moment gradient	C3.1.4
$\beta_{br,1}$	Required brace stiffness for a single compression member	D3.3
β_o	Target reliability index	D6.2.1, F1.1
Δ_{tf}	Lateral displacement of purlin top flange at the line of restraint	D6.3.1
δ, δ_i, γ, γ_i, ω, ω_i	Coefficients	B5.1.1, B5.1.2
ξ_{web}	Stress gradient in web	C3.1.4
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 vertical and plane of web of Z-section,	D6.3.1

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Symbol	Definition	Section
	degrees	
θ	Angle between an element and its edge stiffener	B4, C3.1.4, C4.2, 1.1.1.1, 1.1.1.2
θ_2, θ_3	Angle of segment of complex lip	1.1.1.1, 1.1.1.2
λ, λ_c	Slenderness factors	B2.1, B2.2, B3.2, B5.1, C3.5.1, C3.5.2, C4.1, 1.2.1.1
λ_1, λ_2, λ_3, λ_4	Parameters used in determining compression strain factor	C3.1.1
λ_ℓ	Slenderness factor	1.2.1.2, 1.2.2.2
λ_d	Slenderness factor	C3.1.4, C4.2, 1.2.1.3, 1.2.2.3
μ	Poisson's ratio for steel = 0.30	B2.1, C3.2.1, C3.1.4, C4.2
ρ	Reduction factor	A7.2, B2.1, B3.2, B5.1, F3.1
σ_{ex}	$(\pi^2E)/(K_xL_x/r_x)^2$ $(\pi^2E)/(L/r_x)^2$	C3.1.2.1
σ_{ey}	$(\pi^2E)/(K_yL_y/r_y)^2$ $(\pi^2E)/(L/r_y)^2$	C3.1.2.1
σ_t	Torsional buckling stress	C3.1.2.1, C4.1.2, C4.1.3
ϕ	Resistance factor	A1.2, A1.3, A5.1.1, A6.1.1, D6.2.1, C3.5.2, C3.7.2, D6.1.3, D6.3.1, E2.1, E2.2.2, E2.3, E2.4, E2.5, E2.6, E2.2.1.1, E2.2.1.2, E2.2.1.3, E2.7, E3.1, E3.2, 3.3.1, E3.3.2, E3.4, E4, E4.3.2, E4.4, E4.4.3, E4.5.2, E5.1, E5.3, F1.1, F1.2, 1.1.1, 1.1.1.1, 1.1.1.2, 1.2.1, 1.2.2
ϕ_b	Resistance factor for bending strength	C3.1.1, C3.1.2, C3.1.3, C3.1.4, C3.3.2, C4.2, C3.5.2, C5.1.2, C5.2.2, D6.1.1, D6.1.2, 1.2.2
ϕ_c	Resistance factor for concentrically loaded compression strength	A2.3.1, C3.7.1, C4.1, C5.2.2 1.2.1
ϕ_d	Resistance factor for diaphragms	D5
ϕ_t	Resistance factor for tension strength	C2, C2.1, C5.1.2
ϕ_u	Resistance factor for fracture on net section	C2.2
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

SYMBOLS AND DEFINITIONS

Symbol	Definition	Section
ψ	$\lvert f_2/f_1 \rvert$	B2.3, B3.2, C3.1.1
τ_b	Parameter for reduced stiffness using second-order analysis	2.2.3
Ω	Safety factor	A1.2, A1.3, A4.1.1, D6.2.1, C3.5.1, C3.7.2, D6.1.3, D6.3.1, E2.1, E2.2.1.1, E2.2.1.2, E2.2.1.3, 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, E4.5.1, E5.1, E5.3, F1.2, 1.1.1, 1.1.1.1, 1.1.1.2, 1.2.1, 1.2.2
Ω_b	Safety factor for bending strength	C3.1.1, C3.1.2, C3.1.3, 3.1.4, C3.3.1, C4.2, C3.5.1, C5.1.1, C5.2.1, D6.1.1, D6.1.2, 1.2.2
Ω_c	Safety factor for concentrically loaded compression strength	A2.3.1, C4.1, C5.2.1, 1.2.1
Ω_c	Safety factor for bearing strength	C3.7.1
Ω_d	Safety factor for diaphragms	D5
Ω_t	Safety factor for tension strength	C2, C5.1.1
Ω_v	Safety factor for shear strength	C3.2.1, C3.3.1
Ω_w	Safety factor for web crippling strength	C3.4.1, C3.5.1

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NORTH AMERICAN SPECIFICATION FOR THE DESIGN OF COLD-FORMED STEEL STRUCTURAL MEMBERS 2007 EDTION

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

A. GENERAL PROVISIONS

A1 Scope, Applicability, and Definitions

A1.1 Scope

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

(a) buildings; and

(b) structures other than buildings provided allowances are made for dynamic effects.

☜A

A1.2 Applicability

This *Specification* includes Symbols and Definitions, Chapters A through G, Appendices A and B, and Appendices 1 and 2 that shall apply as follows:

- Appendix A — the United States and Mexico,
- Appendix B — Canada,
- Appendix 1 — alternative design provisions for several sections of Chapter C, and
- Appendix 2 — second-order analysis.

Symbol ☜X is used to point out that additional provisions are provided in the appendices indicated by the letter(s).

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

- ASD and LRFD — United States and Mexico, and
- LSD — Canada

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

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, Appendices A and B, and Appendices 1 and 2 of the *Specification*.

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) A*vailable strength* [*factored resistance*] or stiffness by tests, undertaken and evaluated in accordance with Chapter F,

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

For members
Ω = 2.00 (ASD)
ϕ = 0.80 (LRFD)
= 0.75 (LSD)
For connections
Ω = 2.50 (ASD)
ϕ = 0.65 (LRFD)
= 0.60 (LSD)

When rational engineering analysis is used to determine the nominal strength [nominal resistance] for a *limit state* already provided in this *Specification*, the safety factor shall not be less than the applicable safety factor (Ω) nor shall the resistance factor exceed the applicable resistance factor (ϕ) for the prescribed limit state.

A1.3 Definitions

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

The following terms are italicized when they appear for the first time in a sub-section of the *Specification*. Terms listed under the ASD and LRFD Terms sections shall apply to the USA and Mexico, while definitions listed under the LSD Terms section shall apply in Canada.

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

General Terms

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

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

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

Block Shear Rupture✦. In a connection, limit state of tension rupture along one path and shear yielding or shear rupture along another path.

Braced Frame✦. Essentially vertical truss system that provides resistance to lateral *loads* and provides stability for the structural system.

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

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

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

Confirmatory Test. Test made, when desired, on members, *connections*, and assemblies designed in accordance with the provisions of Chapters A through G, Appendices A and B, and Appendices 1 and 2 of this *Specification* or its specific references, in order to compare actual to calculated performance.

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

Cross-Sectional Area:

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

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

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

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

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

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

Direct Strength Method. An alternative design method detailed in Appendix 1 that provides predictions of member strengths [resistances] without the use of effective widths.

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

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

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

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

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

Flange of a Section in Bending (Flange). 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.

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

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

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

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

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

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

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

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

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

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

Load Factor✦. Factor that accounts for deviations of the nominal load from the actual load, for uncertainties in the analysis that transforms the load into a *load effect*, and for the probability that more than one extreme load will occur simultaneously.

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

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

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

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

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

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

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

Out-of-Plane Buckling✦. Limit state of a beam, column or beam-column involving lateral or *lateral-torsional buckling*.

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

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

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

Published Specification. Requirements for a steel listed by a manufacturer, processor, producer, purchaser, or other body, which (1) are generally available in the public domain or are available to the public upon request, (2) are established before the steel is ordered, and (3) as a minimum, specify minimum mechanical properties, chemical composition limits, and, if coated sheet, coating properties.

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

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

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

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

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

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

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

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

Serviceability Limit State✦. Limiting condition affecting the ability of a structure to preserve its appearance, maintainability, durability, or the comfort of its occupants or function of machinery, under normal usage.

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

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

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

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

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 (Structural Steel). ASTM designation for certain sheet steels intended for structural applications.

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

Structural Analysis✦. Determination of load effects on members and connections based on principles of structural mechanics.

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

Structural Component✦. Member, connector, connecting element, or assemblage.

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

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

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

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

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

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.

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

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

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

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

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

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

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

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

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

Yielding+. Limit state of inelastic deformation that occurs when the *yield stress* is reached.

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

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

ASD and LRFD Terms (USA and Mexico):

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

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

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

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

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

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

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

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

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

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

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

Resistance. See the definition of Nominal Strength.

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

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

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

LSD Terms (Canada):

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 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.

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

A1.4 Units of Symbols and Terms

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

A2 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 in this Section. The term *SS* shall designate sheet material and the terms HSLAS and HSLAS-F shall designate high-strength low-alloy steels.

ASTM A36/A36M, Standard Specification for Carbon Structural Steel

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

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

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

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

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

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

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

ASTM A653/A653M (SS Grades 33 (230), 37 (255), 40 (275), 50 (340) Class 1, Class 3 and Class 4, and 55 (380); HSLAS and HSLAS-F, Grades 40 (275), 50 (340), 55 (380) Class 1 and 2, 60 (410), 70 (480) and 80 (550)), Standard Specification for 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 (340) Class 1 and Class 4)), Standard Specification for Steel Sheet, 55% Aluminum-Zinc Alloy-Coated by the Hot-Dip Process

ASTM A847/A847M, Standard Specification for 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 and HSLAS-F, Grades 50 (340), 60 (410), 70 (480), and 80 (550)), Standard Specification for Steel Sheet, Zinc-5% Aluminum Alloy-Coated by the Hot-Dip Process

ASTM A1003/A1003M (ST Grades 50 (340) H, 40 (275) H, 37 (255) H, 33 (230) H), Standard Specification for 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 (410), 65 (450), and 70 (480); HSLAS-F Grades 50 (340), 60 (410), 70 (480), and 80 (550)), Standard Specification for Steel, Sheet, Cold-Rolled, Carbon, Structural, High-Strength Low-Alloy, High-Strength Low-Alloy with Improved Formability, Solution Hardened, and Bake Hardenable

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)), Standard Specification for Steel, Sheet and Strip, Hot-Rolled, Carbon, Structural, High-Strength Low-Alloy and High-Strength Low-Alloy with Improved Formability

ASTM A1039/A1039M (SS Grades 40 (275), 50 (340), 55 (380), 60 (410), 70 (480), and 80 (550)), Standard Specification for Steel, Sheet, Hot Rolled, Carbon, Commercial and Structural, Produced by the Twin-Roll Casting Process. Thicknesses of Grades 55 (380) and higher that do not meet the minimum 10% elongation requirement are limited per Section A2.3.2.

☜ B

A2.2 Other Steels

See Section A2.2 of Appendix A or B. ☜ A,B

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 ductility requirements in either Section A2.3.1 or Section A2.3.2:

A2.3.1 The ratio of *tensile strength* to *yield stress* shall not be less than 1.08, and the total elongation shall not be less than 10 percent for a two-inch (50 mm) gage 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 percent, and (2) uniform elongation outside the fracture shall not be less than 3 percent. When material ductility is determined on the basis of the local and uniform elongation criteria, the use of such material shall be restricted to the design of *purlins, girts*, and *curtain wall studs* in accordance with Sections C3.1.1(a), C3.1.2, D6.1.1, D6.1.2, D6.2.1, and country-specific requirements given in A2.3.1a of the Appendix A or B. For purlins, girts, and curtain wall studs 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*. ☜ A,B

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), *thicknesses* of ASTM A1039 Grades 55 (380), 60 (410), 70 (480), and 80 (550) that do not meet the minimum 10 percent elongation requirement in Section A2.3.1, and other steels that do not meet the provisions of Section A2.3.1 shall be permitted for concentrically loaded closed box section compression members as given in Exception 2 below and for multiple-web configurations such as roofing, siding, and floor decking as given in Exception 1 provided that:

(1) the *yield stress*, F_y, used for determining *nominal strength* [*nominal resistance*] in Chapters B, C, D, and E is taken as 75 percent of the *specified minimum yield stress* or 60 ksi (410 MPa or 4220 kg/cm^2), whichever is less, and

(2) the *tensile strength*, F_u, used for determining nominal strength [nominal 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 in accordance with the provisions of Section F1. *Available strengths* [*factored resistances*] based on these tests shall not exceed the available strengths [factored resistances] calculated in accordance with Chapters B through G, Appendices A and B, and Appendices 1 and 2, using the specified minimum yield stress, F_{sy}, and the specified minimum tensile strength, F_u.

Exception 1: For multiple-web configurations, a reduced specified minimum yield stress, $R_b F_{sy}$, 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 in accordance with (a) or (b):

(a) For stiffened and partially stiffened compression flanges

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

$R_b = 1.0$

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

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

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

$R_b = 0.75$

(b) For unstiffened compression flanges

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

$R_b = 1.0$

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

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

where

w = *Flat width* of compression flange
t = *Thickness* of section
E = Modulus of elasticity of steel
F_{sy} = Specified minimum yield stress determined in accordance with Section A7.1 $\leq$ 80 ksi (550 MPa, or 5620 kg/cm^2)

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

Exception 2: For concentrically loaded compression members with a closed box section, a reduced yield stress, $0.9F_{sy}$, shall be permitted to be used in place of F_y in Eqs. C4.1-2, C4.1-3, and C4.1-4 for determining the axial strength in Section C4. A reduced radius of gyration $(R_r)(r)$ shall be used in Eq. C4.1.1-1 when the value of the effective length KL is less than $1.1L_0$, where L_0 is given by Eq. A2.3.2-3, and R_r is given by Eq. A2.3.2-4.

$$L_0 = \pi r\sqrt{\frac{E}{F_{cr}}} \quad (Eq.\ A2.3.2\text{-}3)$$

$$R_r = 0.65 + \frac{0.35(KL)}{1.1L_0} \quad (Eq.\ A2.3.2\text{-}4)$$

where

L_0 = Length at which *local buckling* stress equals *flexural buckling* stress
r = Radius of gyration of full unreduced cross section
F_{cr} = Minimum critical *buckling* stress for section calculated by Eq. B2.1-5
R_r = Reduction factor
KL = Effective length

A2.4 Delivered Minimum Thickness

The uncoated minimum steel *thickness* of the cold-formed steel product as delivered to the job site shall not at any location be less than 95 percent of the thickness, t, used in its design; however, lesser 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 in Section A3 of Appendix A or B.

☜A,B

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* shall apply, except for those in Sections A5 and A6 and in Chapters C and F designated for *LRFD* and *LSD*.

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 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:

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

where

R = Required strength

R_n = *Nominal strength* specified in Chapters B through G and Appendix 1

Ω = *Safety factor* specified in Chapters B through G and Appendix 1

R_n/Ω = *Allowable strength*

A4.1.2 Load Combinations for ASD

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

☜A

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* shall apply, except for those in Sections A4 and A6 and in Chapters C and F designated for *ASD* and *LSD*.

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 applicable *load factors*, for all applicable *load combinations*.

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

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

where

R_u = Required strength

ϕ = *Resistance factor* specified in Chapters B through G and Appendix 1

R_n = *Nominal strength* specified in Chapters B through G and Appendix 1

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.

☜A

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* shall apply, except for those in Sections A4 and

A5 and Chapters C and F designated for *ASD* and *LRFD*.

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

ϕ = *Resistance factor* specified in Chapters B through G and Appendix 1

R_n = *Nominal resistance* specified in Chapters B through G and Appendix 1

ϕR_n = Factored resistance

R_f = Effect of factored loads

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.

☜B

A7 Yield Stress and Strength Increase from Cold Work of Forming

A7.1 Yield Stress

The *yield stress* used in design, F_y, shall not exceed the *specified minimum yield stress* 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 stress* of the full section. Such increase shall be limited to Sections C2, C3.1 (excluding Section C3.1.1(b)), C4, C5, D4, and D6.1. The limits and methods for determining F_{ya} shall be in accordance with (a), (b) and (c).

(a) For axially loaded compression members and flexural members whose proportions are such that the quantity ρ for strength determination is unity as determined in accordance with 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 in accordance with Eq. A7.2-1.

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

where

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

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

F_{yc} = $B_c F_{yv}/(R/t)^m$, tensile yield stress of corners. (*Eq.* A7.2-2)

Eq. A7.2-2 applies only when $F_{uv}/F_{yv} \geq 1.2$, $R/t \leq 7$, and the included angle ≤ 120°.

where

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

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

R = Inside bend radius

t = *Thickness* of section

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

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

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

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

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

A8 Serviceability

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

A9 Referenced Documents

The following documents or portions thereof are referenced in this *Specification* and shall be considered part of the requirements of this *Specification*. Refer to Section A9a of Appendix A or B for documents applicable to the corresponding country. A,B

1. American Iron and Steel Institute (AISI), 1140 Connecticut Avenue, NW, Washington, DC 20036:

   AISI S200-07, North American Standard for Cold-Formed Steel Framing - General Provisions

   AISI S210-07, North American Standard for Cold-Formed Steel Framing – Floor and Roof System Design

   AISI S211-07, North American Standard for Cold-Formed Steel Framing – Wall Stud Design

   AISI S212-07, North American Standard for Cold-Formed Steel Framing – Header Design

   AISI S214-07, North American Standard for Cold-Formed Steel Framing – Truss Design

   AISI S901-02*, Rotational-Lateral Stiffness Test Method for Beam-to-Panel Assemblies

   AISI S902-02, Stub-Column Test Method for Effective Area of Cold-Formed Steel Columns

   AISI S906-04, Standard Procedures for Panel and Anchor Structural Tests

Note: * AISI test procedures previously designated as AISI TSn-xx are re-designated to AISI S9n-xx, where "n" is the test procedure sequence number and "xx" is the year the standard was developed or updated.

2. American Society of Mechanical Engineers (ASME), 1828 L Street, NW, Washington, DC 20036:

   ASME B46.1-2000, Surface Texture, Surface Roughness, Waviness, and Lay

3. American Society for Testing and Materials (ASTM), 100 Barr Harbor Drive, West Conshohocken, Pennsylvania 19428-2959:

   ASTM A36/A36M-05, Standard Specification for Carbon Structural Steel

   ASTM A194/A194M-06, Standard Specification for Carbon and Alloy Steel Nuts for Bolts for High-Pressure and High-Temperature Service, or Both

   ASTM A242/A242M-04e1, Standard Specification for High-Strength Low-Alloy Structural Steel

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

   ASTM A307-04, Standard Specification for Carbon Steel Bolts and Studs, 60,000 PSI Tensile Strength

   ASTM A325-06, Standard Specification for Structural Bolts, Steel, Heat Treated, 120/105 ksi Minimum Tensile Strength

   ASTM A325M-05, Standard Specification for Structural Bolts, Steel, Heat Treated, 830 MPa Minimum Tensile Strength [Metric]

   ASTM A354-04, Standard Specification for Quenched and Tempered Alloy Steel Bolts, Studs, and Other Externally Threaded Fasteners

   ASTM A370-05, Standard Specification for Standard Test Methods and Definitions for Mechanical Testing of Steel Products

   ASTM A449-04b, Standard Specification for Hex Cap Screws, Bolts, and Studs, Steel, Heat Treated, 120/105/90 ksi Minimum Tensile Strength, General Use

   ASTM A490-06, Standard Specification for Structural Bolts, Alloy Steel, Heat Treated, 150 ksi Minimum Tensile Strength

   ASTM A490M-04a, Standard Specification for High Strength Steel Bolts, Classes 10.9 and 10.9.3, for Structural Steel Joints [Metric]

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

   ASTM A529/A529M-05, Standard Specification for High-Strength Carbon-Manganese Steel of Structural Quality

   ASTM A563-04, Standard Specification for Carbon and Alloy Steel Nuts

   ASTM A563M-04, Standard Specification for Carbon and Alloy Steel Nuts [Metric]

   ASTM A572/A572M-06, Standard Specification for High-Strength Low-Alloy Columbium-Vanadium Structural Steel

   ASTM A588/A588M-05, Standard Specification for High-Strength Low-Alloy Structural Steel with 50 ksi [345 MPa] Minimum Yield Point to 4-in. [100 mm] Thick

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

ASTM A653/A653M-06, Standard Specification for Steel Sheet, Zinc-Coated (Galvanized) or Zinc-Iron Alloy-Coated (Galvannealed) by the Hot-Dip Process

ASTM A792/A792M-05, Standard Specification for Steel Sheet, 55% Aluminum-Zinc Alloy-Coated by the Hot-Dip Process

ASTM A847/A847M-05, Standard Specification for Cold-Formed Welded and Seamless High Strength, Low Alloy Structural Tubing with Improved Atmospheric Corrosion Resistance

ASTM A875/A875M-05, Standard Specification for Steel Sheet, Zinc-5% Aluminum Alloy-Coated by the Hot-Dip Process

ASTM A1003/A1003M-05, Standard Specification for Steel Sheet, Carbon, Metallic- and Nonmetallic-Coated for Cold-Formed Framing Members

ASTM A1008/A1008M-05b, Standard Specification for Steel, Sheet, Cold-Rolled, Carbon, Structural, High-Strength Low-Alloy, High-Strength Low-Alloy with Improved Formability, Solution Hardened, and Bake Hardenable

ASTM A1011/A1011M-05a, Standard Specification for Steel, Sheet and Strip, Hot-Rolled, Carbon, Structural, High-Strength Low-Alloy and High-Strength Low-Alloy with Improved Formability

ASTM A1039/A1039M-04, Standard Specification for Steel, Sheet, Hot Rolled, Carbon, Commercial and Structural, Produced by the Twin-Roll Casting Process

ASTM E1592-01, Standard Test Method for Structural Performance of Sheet Metal Roof and Siding Systems by Uniform Static Air Pressure Difference

ASTM F436-04, Standard Specification for Hardened Steel Washers

ASTM F436M-04, Standard Specification for Hardened Steel Washers [Metric]

ASTM F844-04, Standard Specification for Washers, Steel, Plain (Flat), Unhardened for General Use

ASTM F959-05a, Standard Specification for Compressible Washer-Type Direct Tension Indicators for Use with Structural Fasteners

ASTM F959M-04, Standard Specification for Compressible Washer-Type Direct Tension Indicators for Use with Structural Fasteners [Metric]

4. U. S. Army Corps of Engineers:

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

5. Factory Mutual, Corporate Offices, 1301 Atwood Avenue, P.O. Box 7500, Johnston, RI 02919:

FM 4471, Approval Standard for Class 1 Metal Roofs, 1995

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 t as the actual *thickness* of the element, shall be determined in accordance with this section as follows:

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

Simple lip, $w/t \le 60$

Any other kind of stiffener

i) when $I_s < I_a$, $w/t \le 60$

ii) when $I_s \ge I_a$, $w/t \le 90$

where

I_s = Actual moment of inertia of full stiffener about its own centroidal axis parallel to element to be stiffened

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

(2) *Stiffened compression element* with both longitudinal edges connected to other stiffened elements, $w/t \le 500$

(3) *Unstiffened compression element*, $w/t \le 60$

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 *available strength* [*factored resistance*], without affecting the ability of the member to develop the *required strength* [effect of factored loads].

Stiffened elements having w/t ratios greater than 500 provide adequate available 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, Eq. B1.1-1 shall be permitted to be applied to compression and tension flanges, either stiffened or unstiffened as follows:

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

where

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

t = Flange thickness

d = Depth of beam

f_{av} = Average *stress* in full unreduced flange width. (Where members are designed by the *effective design width* procedure, the average stress equals the maximum stress multiplied by the ratio of the effective design width to the actual width.)

c_f = Amount of curling displacement

(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 by the values in Table B1.1(c).

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

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

where

L = Full span for simple beams; or the distance between inflection points for continuous beams; or twice the length for cantilever beams

w_f = Width of flange projection beyond web for I-beam and similar sections; or half the distance between webs for box- or U-type sections

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

B1.2 Maximum Web Depth-to-Thickness Ratios

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

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

(b) For webs which are provided with bearing stiffeners satisfying the requirements of Section C3.7.1:

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

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

where

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

t = Web *thickness.* Where a web consists of two or more sheets, the h/t ratio is 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 calculated from either Eq. B2.1-1 or Eq. B2.1-2 as follows:

$b = w$ 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

ρ = Local reduction factor

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

λ = Slenderness factor

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

where

f = *Stress* in compression element computed as follows:

For flexural members:

(1) 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 is 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 effective section modulus, S_c.

For compression members, f is taken equal to F_n as determined in accordance with Section C4.

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

where

k = Plate *buckling* coefficient

= 4 for stiffened elements supported by a *web* on each longitudinal edge. Values for different types of elements are given in the applicable sections.

E = Modulus of elasticity of steel

t = *Thickness* of uniformly compressed *stiffened element*

μ = Poisson's ratio of steel

(b) Serviceability Determination

The effective width, b_d, used in determining serviceability shall be calculated from either Eq. B2.1-6 or Eq. B2.1-7 as follows:

$b_d = w$ 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 conservative estimate of the effective width is obtained from Eqs. B2.1-3 and

B2.1-4 by substituting f_d for f, where f_d is the computed compressive stress in the element being considered.

(2) Procedure II:

For stiffened elements supported by a web on each longitudinal edge, an improved estimate of the effective width is obtained by calculating ρ as follows:

$\rho = 1$ when $\lambda \leq 0.673$

$$\rho = (1.358 - 0.461/\lambda)/\lambda \quad \text{when } 0.673 < \lambda < \lambda_c \qquad (Eq.\ B2.1\text{-}8)$$

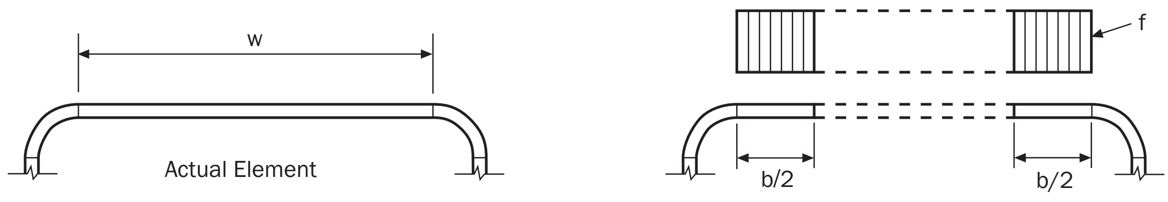
$$\rho = (0.41 + 0.59\sqrt{F_y / f_d} - 0.22/\lambda)/\lambda \quad \text{when } \lambda \geq \lambda_c \qquad (Eq.\ B2.1\text{-}9)$$

$\rho \leq 1$ for all cases.

where

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

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



Effective Element, b, and Stress, f, on Effective Elements

Figure B2.1-1 Stiffened Elements

B2.2 Uniformly Compressed Stiffened Elements with Circular or Non-Circular Holes

(a) Strength Determination

For circular holes:

The *effective width*, b, shall be calculated by either Eq. B2.2-1 or Eq. B2.2-2 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 $\geq 3d_h$

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

$$b = \frac{w\left[1 - \frac{(0.22)}{\lambda} - \frac{(0.8d_h)}{w} + \frac{(0.085d_h)}{w\lambda}\right]}{\lambda} \qquad \text{when } \lambda > 0.673 \qquad (Eq.\ B2.2\text{-}2)$$

In all cases, $b \leq w - d_h$

where

w = *Flat width*

t = *Thickness* of element

d_h = Diameter of holes

λ = A value as defined in Section B2.1

For non-circular holes:

A uniformly compressed stiffened element with non-circular holes shall be assumed to consist of two unstiffened strips of flat width, c, adjacent to the holes (see Figure B2.2-1). The effective width, b, of each unstiffened strip adjacent to the hole shall be determined in accordance with B2.1(a), except that plate buckling coefficient, k, shall be taken as 0.43 and w as c. These provisions shall be applicable within the following limits:

(1) Center-to-center hole spacing, $s \geq 24$ in. (610 mm),
(2) Clear distance from the hole at ends, $s_{end} \geq 10$ in. (254 mm),
(3) Depth of hole, $d_h \leq 2.5$ in. (63.5 mm),
(4) Length of hole, $L_h \leq 4.5$ in. (114 mm), and
(5) Ratio of the depth of hole, d_h, to the out-to-out width, w_o, $d_h/w_o \leq 0.5$.

Alternatively, the effective width, b, shall be permitted to be determined by stub-column tests in accordance with the test procedure, AISI S902.

(b) Serviceability Determination

The effective width, b_d, used in determining serviceability shall be equal to b calculated in accordance with Procedure I of Section B2.1(b), except that f_d is substituted for f, where f_d is the computed compressive *stress* in the element being considered.

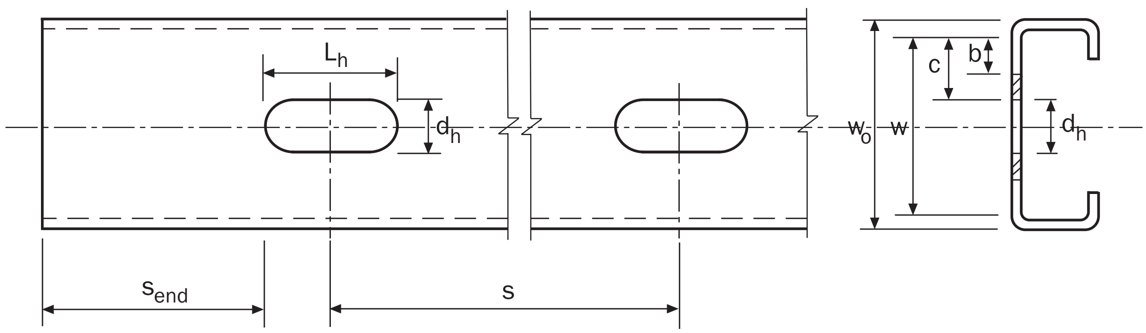


Figure B2.2-1 Uniformly Compressed Stiffened Elements with Non-Circular Holes

B2.3 Webs and Other Stiffened Elements under Stress Gradient

The following notation shall apply 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 depth of *web* as defined in Figure B2.3-2
k = Plate *buckling* coefficient
ψ = $|f_2/f_1|$ (absolute value) (*Eq.* B2.3-1)

(a) Strength Determination

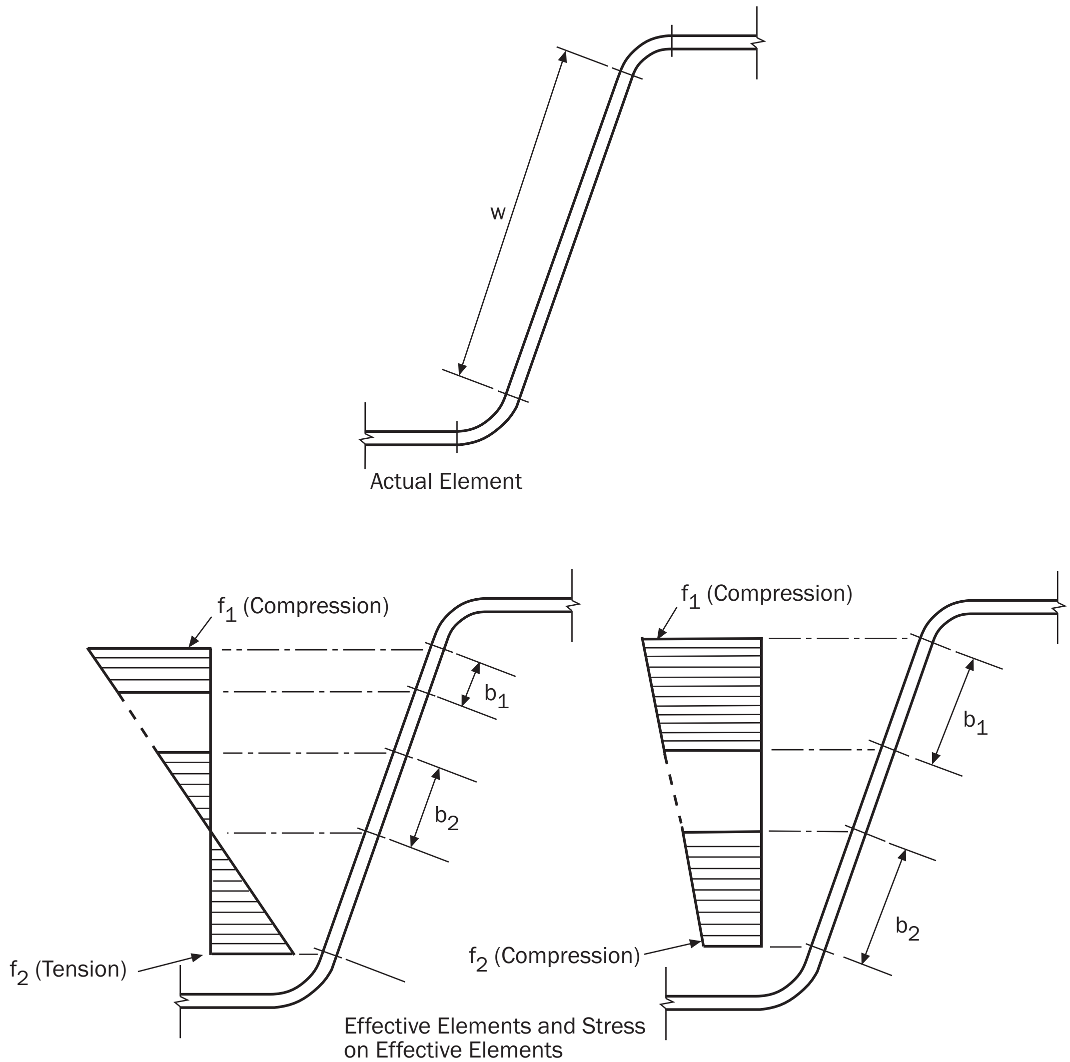
(i) For webs under stress gradient (f_1 in compression and f_2 in tension as shown in Figure B2.3-1(a)), the effective widths and plate buckling coefficient shall be calculated as follows:

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

For $h_o/b_o \leq 4$

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

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



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

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

$b_2 = b_e - b_1$ when $\psi \leq 0.236$ (*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(b))

$k = 4 + 2(1 - \psi)^3 + 2(1 - \psi)$ (*Eq.* B2.3-8)

$b_1 = b_e/(3 - \psi)$ (*Eq.* B2.3-9)

$b_2 = b_e - b_1$ (*Eq.* B2.3-10)

(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.

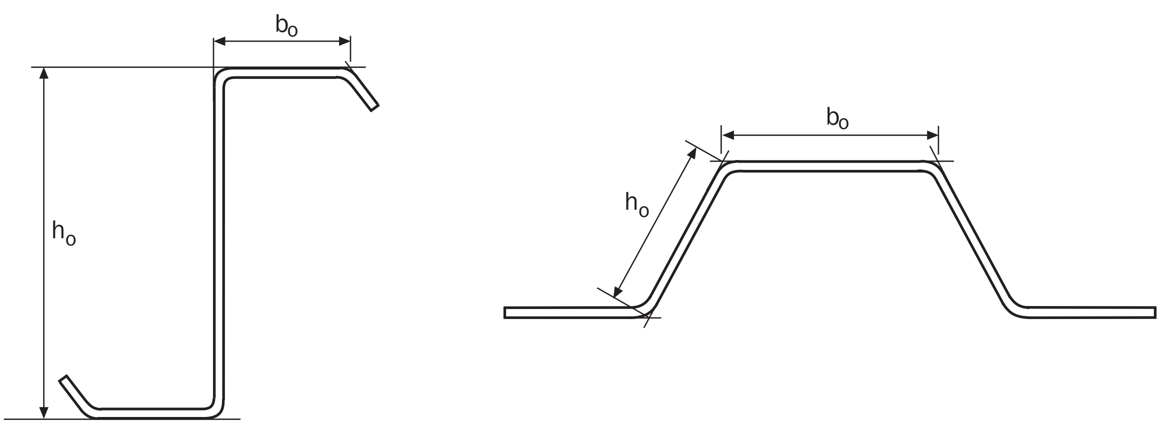


Figure B2.3-2 Out-to-Out Dimensions of Webs and Stiffened Elements under Stress Gradient

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

The provisions of Section B2.4 shall apply within the following limits:

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

where

d_h = Depth of *web* hole
h = Depth of flat portion of web measured along plane of web
t = *Thickness* of web
L_h = Length of web hole

b_1, b_2 = *Effective widths* defined by Figure B2.3-1

(a) *Strength Determination*

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

When $d_h/h \geq 0.38$, the effective width shall be determined in accordance with Section 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 in accordance with Section B2.3(b) by assuming no hole exists in the web.

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 plate buckling coefficient, 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.

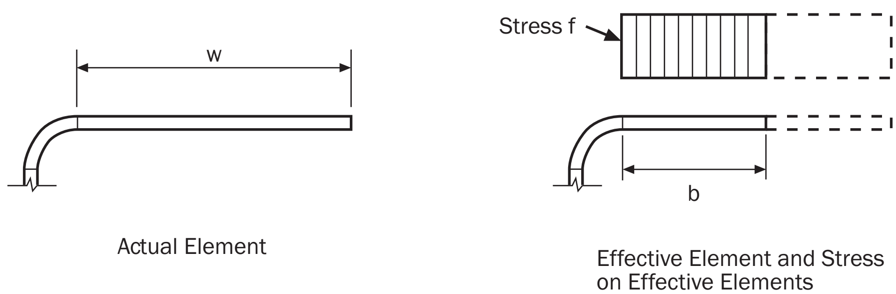


Figure B3.1-1 Unstiffened Element with Uniform Compression

B3.2 Unstiffened Elements and Edge Stiffeners with Stress Gradient

The following notation shall apply in this section:

b = *Effective width* measured from the supported edge, determined in accordance with Section B2.1(a), with f equal to f_1 and with k and ρ being determined in accordance with this section

b_o = Overall width of unstiffened element of unstiffened C-section member as defined in Fig. B3.2-3

f_1, f_2 = *Stresses* shown in Figures B3.2-1, B3.2-2, and B3.2-3 calculated on the basis of the gross section. Where f_1 and f_2 are both compression, $f_1 \geq f_2$.

h_o = Overall depth of unstiffened C-section member as defined in Fig. B3.2-3

k = Plate *buckling* coefficient defined in this section or, otherwise, as defined in Section

B2.1(a)

t = *Thickness* of element

w = *Flat width* of unstiffened element, where $w/t \le 60$

ψ = $|f_2/\ f_1|$ (absolute value) (*Eq*. B3.2-1)

λ = Slenderness factor defined in Section B2.1(a) with $f = f_1$

ρ = Reduction factor defined in this section or, otherwise, as defined in Section B2.1(a)

(a) Strength Determination

The effective width, b, of an unstiffened element under stress gradient shall be determined in accordance with Section B2.1(a) with f equal to f_1 and the plate buckling coefficient, k, determined in accordance with this section, unless otherwise noted. For the cases where f_1 is in compression and f_2 is in tension, ρ in Section B2.1(a) shall be determined in accordance with this section.

(1) When both f_1 and f_2 are in compression (Fig. B3.2-1), the plate buckling coefficient shall be calculated in accordance with either Eq. B3.2-2 or Eq. B3.2-3 as follows:

If the stress decreases toward the unsupported edge (Figure B3.2-1(a)):

$$k = \frac{0.578}{\psi + 0.34} \qquad (Eq.\ B3.2\text{-}2)$$

If the stress increases toward the unsupported edge (Figure B3.2-1(b)):

$$k = 0.57 - 0.21\psi + 0.07\psi^2 \qquad (Eq.\ B3.2\text{-}3)$$

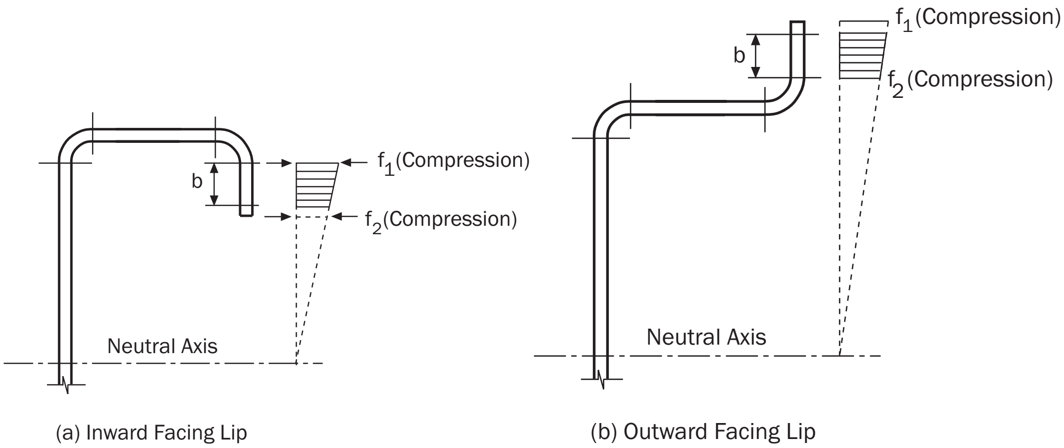


Figure B3.2-1 Unstiffened Elements under Stress Gradient, Both Longitudinal Edges in Compression

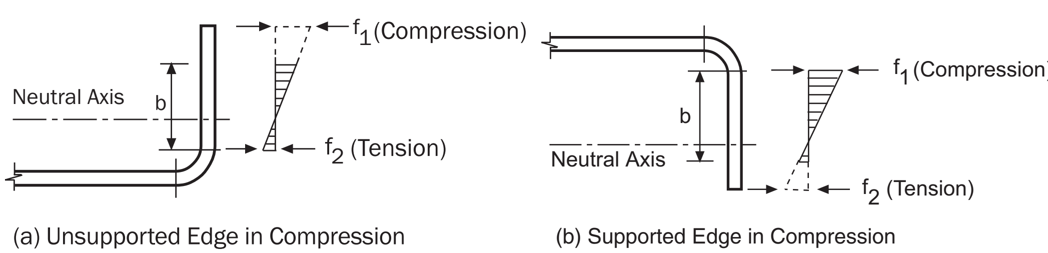


Figure B3.2-2 Unstiffened Elements under Stress Gradient, One Longitudinal Edge in Compression and the Other Longitudinal Edge in Tension

(2) When f_1 is in compression and f_2 in tension (Fig. B3.2-2), the reduction factor and plate buckling coefficient shall be calculated as follows:

(i) If the unsupported edge is in compression (Figure B3.2-2(a)):

$\rho = 1$ when $\lambda \leq 0.673(1 + \psi)$

$$\rho = (1+\psi)\frac{\left(1-\frac{0.22(1+\psi)}{\lambda}\right)}{\lambda} \quad \text{when } \lambda > 0.673(1+\psi) \qquad (Eq.\ B3.2\text{-}4)$$

$$k = 0.57 + 0.21\psi + 0.07\psi^2 \qquad (Eq.\ B3.2\text{-}5)$$

(ii) If the supported edge is in compression (Fig. B3.2-2(b)):

For $\psi < 1$

$\rho = 1$ when $\lambda \leq 0.673$

$$\rho = (1-\psi)\frac{\left(1-\frac{0.22}{\lambda}\right)}{\lambda} + \psi \quad \text{when } \lambda > 0.673 \qquad (Eq.\ B3.2\text{-}6)$$

$$k = 1.70 + 5\psi + 17.1\psi^2 \qquad (Eq.\ B3.2\text{-}7)$$

For $\psi \geq 1$,

$\rho = 1$

The effective width, b, of the unstiffened elements of an unstiffened C-section member shall be permitted to be determined using the following alternative methods, as applicable:

Alternative 1 for unstiffened C-sections: When the unsupported edge is in compression and the supported edge is in tension (Figure B3.2-3 (a)):

$$b = w \quad \text{when } \lambda \leq 0.856 \qquad (Eq.\ B3.2\text{-}8)$$

$$b = \rho w \quad \text{when } \lambda > 0.856 \qquad (Eq.\ B3.2\text{-}9)$$

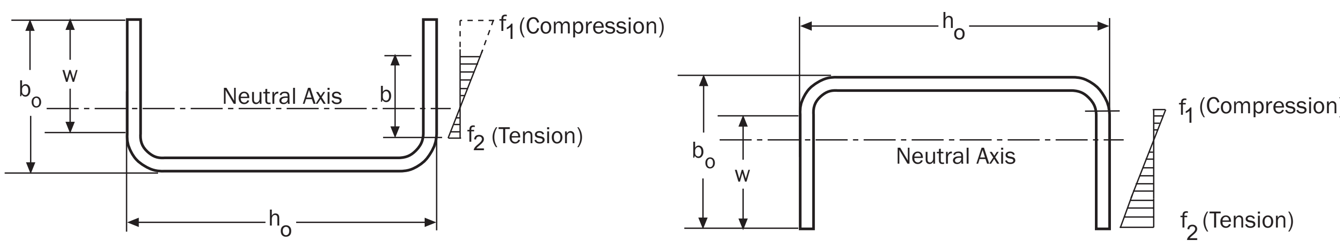
where

$$\rho = 0.925/\sqrt{\lambda} \qquad (Eq.\ B3.2\text{-}10)$$

$$k = 0.145(b_o/h_o) + 1.256 \qquad (Eq.\ B3.2\text{-}11)$$

$$0.1 \leq b_o/h_o \leq 1.0$$

Alternative 2 for unstiffened C-sections: When the supported edge is in compression and the unsupported edge in tension (Figure B3.2-3(b)), the effective width is determined in accordance with Section B2.3.



(a) Unsupported Edge in Compression

(b) Supported Edge in Compression

Figure B3.2-3 Unstiffened Elements of C-Section under Stress Gradient for Alternative Methods

In calculating the effective section modulus S_e in Section C3.1.1 or S_c in Section C3.1.2.1, the extreme compression fiber in Figures B3.2-1(b), B3.2-2(a), and B3.2-3(a) shall be taken as the edge of the effective section closer to the unsupported edge. In calculating the effective section modulus S_e in Section C3.1.1, the extreme tension fiber in Figures B3.2-2(b) and B3.2-3(b) shall be taken as the edge of the effective section closer to the unsupported edge.

(b) Serviceability Determination

The effective width b_d used in determining serviceability shall be calculated in accordance with Section B3.2(a), except that f_{d1} and f_{d2} are substituted for f_1 and f_2, respectively, where f_{d1} and f_{d2} are the computed stresses f_1 and f_2 as shown in Figures B3.2-1, B3.2-2, and B3.2-3, respectively, based on the gross section at the *load* for which serviceability is determined.

B4 Effective Width of Uniformly Compressed Elements with a Simple Lip Edge Stiffener

The *effective widths* of uniformly compressed elements with a simple edge stiffener shall be calculated in accordance with (a) for strength determination and (b) for serviceability determination.

(a) Strength Determination

For $w/t \le 0.328S$:

$I_a = 0$ (no edge stiffener needed)

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

$b_1 = b_2 = w/2$ (see Figure B4-1) (*Eq.* B4-2)

$d_s = d'_s$ (*Eq.* B4-3)

For $w/t > 0.328S$

$b_1 = (b/2)(R_I)$ (see Figure B4-1) (*Eq.* B4-4)

$b_2 = b - b_1$ (see Figure B4-1) (*Eq.* B4-5)

$d_s = d'_s (R_I)$ (*Eq.* B4-6)

where

$S = 1.28\sqrt{E/f}$ (*Eq.* B4-7)

w = Flat dimension defined in Figure B4-1

t = *Thickness* of section

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

$$= 399t^4\left[\frac{w/t}{S} - 0.328\right]^3 \le t^4\left[115\frac{w/t}{S} + 5\right] \quad (Eq.\ B4\text{-}8)$$

b = Effective design width

b_1, b_2 = Portions of effective design width as defined in Figure B4-1

d_s = Reduced effective width of stiffener as defined in Figure B4-1, and used in computing overall effective section properties

d'_s = Effective width of stiffener calculated in accordance with Section B3.2 (see Figure B4-1)

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

where

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

$$= (d^3 t \sin^2\theta)/12 \qquad (Eq.\ B4\text{-}10)$$

See Figure B4-1 for definitions of other dimensional variables.

The effective width, b, in Eqs. B4-4 and B4-5 shall be calculated in accordance with Section B2.1 with the plate *buckling* coefficient, k, as given in Table B4-1 below:

Table B4-1
Determination of Plate Buckling Coefficient k

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

where

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

(b) Serviceability Determination

The effective width, b_d, used in determining serviceability shall be calculated as in Section B4(a), except that f_d is substituted for f, where f_d is computed compressive *stress* in the effective section at the *load* for which serviceability is determined.

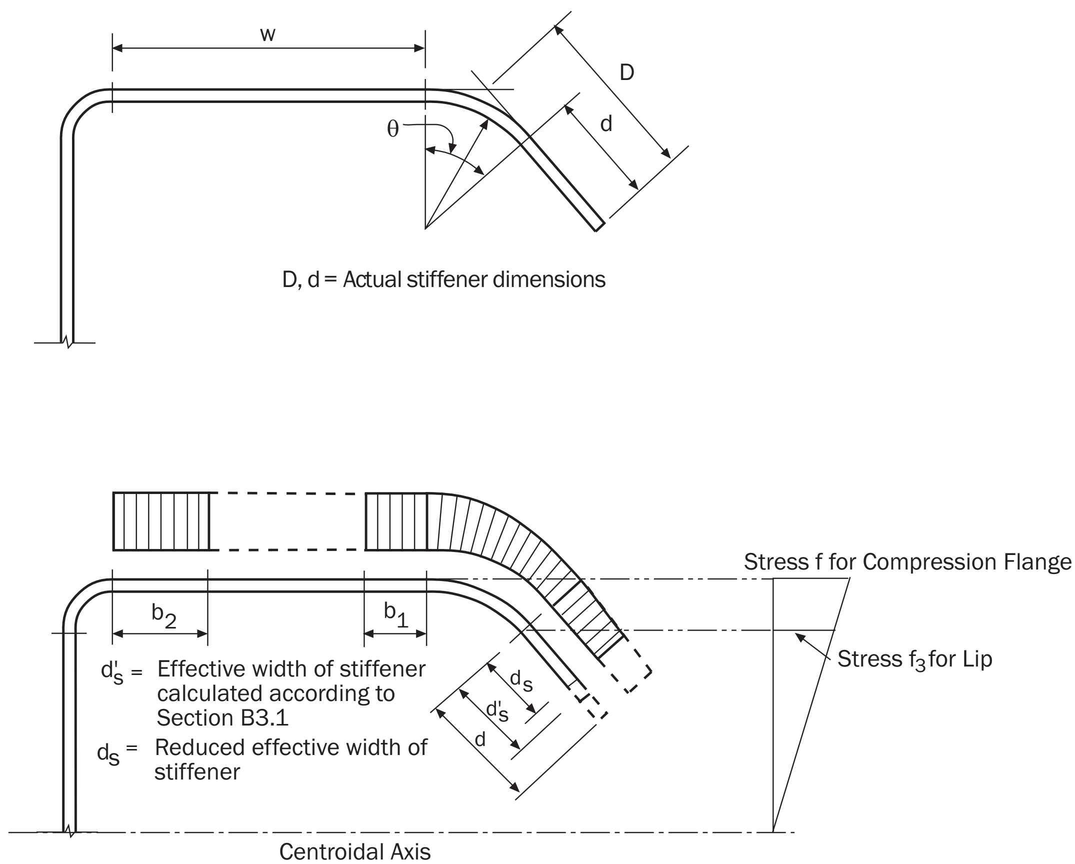


Figure B4-1 Elements with Simple Lip Edge Stiffener

B5 Effective Widths of Stiffened Elements with Single or Multiple Intermediate Stiffeners or Edge Stiffened Elements with Intermediate Stiffener(s)

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

The following notation shall apply as used in this section.

A_g = *Gross area* of element including stiffeners
A_s = Gross area of stiffener
b_e = *Effective width* of element, located at centroid of element including stiffeners; see Figure B5.1-2
b_o = Total flat width of stiffened element; see Figure B5.1-1
b_p = Largest sub-element *flat width*; see Figure B5.1-1
c_i = Horizontal distance from edge of element to centerline(s) of stiffener(s); see Figure B5.1-1
F_{cr} = Plate elastic *buckling stress*
f = Uniform compressive stress acting on flat element
h = Width of elements adjoining stiffened element (e.g., depth of *web* in hat section with multiple intermediate stiffeners in compression flange is equal to h; if adjoining elements have different widths, use smallest one)
I_{sp} = Moment of inertia of stiffener about centerline of flat portion of element. The radii that connect the stiffener to the flat can be included.
k = Plate buckling coefficient of element
k_d = Plate buckling coefficient for *distortional buckling*
k_{loc}= Plate buckling coefficient for local sub-element buckling
L_{br} = Unsupported length between brace points or other restraints which restrict distortional buckling of element
R = Modification factor for distortional plate buckling coefficient
n = Number of stiffeners in element
t = Element *thickness*
i = Index for stiffener "i"
λ = Slenderness factor
ρ = Reduction factor

The effective width shall be calculated in accordance with Eq. B5.1-1 as follows:

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

where

$\rho = 1$ when $\lambda \le 0.673$

$$\rho = (1-0.22/\lambda)/\lambda \quad \text{when } \lambda > 0.673 \qquad (Eq.\ B5.1\text{-}2)$$

where

$$\lambda = \sqrt{\frac{f}{F_{cr}}} \quad (Eq.\ B5.1\text{-}3)$$

where

$$F_{cr} = k \frac{\pi^2 E}{12(1-\mu^2)} \left(\frac{t}{b_o}\right)^2 \quad (Eq.\ B5.1\text{-}4)$$

The plate buckling coefficient, k, shall be determined from the minimum of Rk_d and k_{loc}, as determined in accordance with Section B5.1.1 or B5.1.2, as applicable.

k = the minimum of Rk_d and k_{loc} (*Eq*. B5.1-5)

R = 2 when $b_o/h < 1$

$$R = \frac{11 - b_o/h}{5} \geq \frac{1}{2} \quad \text{when } b_o/h \geq 1 \quad (Eq.\ B5.1\text{-}6)$$

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

For uniformly compressed elements with multiple identical and equally spaced stiffeners, the plate *buckling* coefficients and *effective widths* shall be calculated as follows:

(a) Strength Determination

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

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

where

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

where

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

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

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

(b) Serviceability Determination

The effective width, b_d, used in determining serviceability shall be calculated as in Section B5.1.1(a), except that f_d is substituted for f, where f_d is the computed compressive *stress* in the element being considered based on the effective section at the *load* for which serviceability is determined.

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

For uniformly compressed stiffened elements with multiple stiffeners of arbitrary size, location and number, the plate *buckling* coefficients and *effective widths* shall be calculated as follows:

(a) Strength Determination

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

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

where

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

where

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

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

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

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

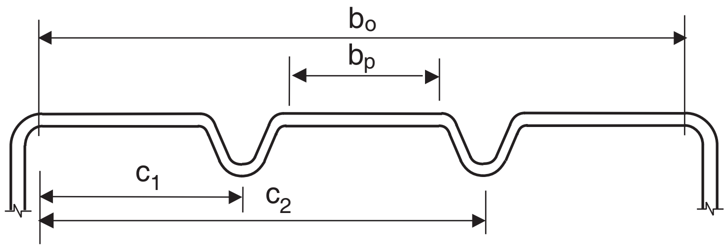


Figure B5.1-1 Plate Widths and Stiffener Locations

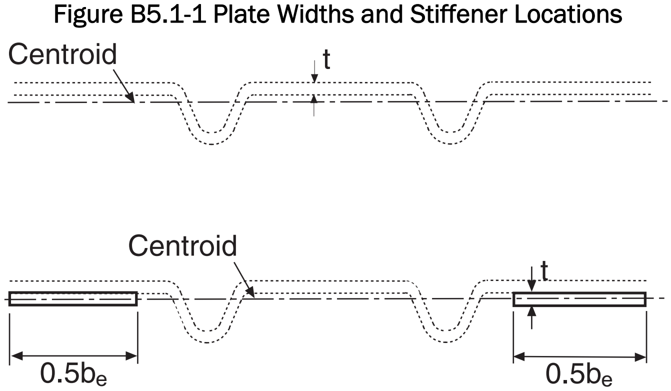


Figure B5.1-2 Effective Width Locations

(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 is substituted for f, where f_d is the computed compressive *stress* in the element being considered based on the effective section at the *load* for which serviceability is determined.

B5.2 Edge Stiffened Elements with Intermediate Stiffener(s)

(a) *Strength Determination*

For edge stiffened elements with intermediate stiffener(s), the *effective width*, b_e, shall be determined as follows:

If $b_o/t \leq 0.328S$, the element is fully effective and no *local buckling* reduction is required.

If $b_o/t > 0.328S$, then the plate *buckling* coefficient, k, is determined in accordance with Section B4, but with b_o replacing w in all expressions:

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

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

R calculated in accordance with Section B5.1 is less than or equal to 1.

where

b_o= Total *flat width* of edge stiffened element

See Sections B4 and B5.1 for definitions of other variables.

(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 is substituted for f, where f_d is the computed compressive *stress* in the element being considered based on the effective section at the *load* for which serviceability is determined.

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

See Section C2 of Appendix A or B for the provisions of this section. ☜A,B

C3 Flexural Members

C3.1 Bending

The nominal flexural strength [moment resistance], M_n, shall be the smallest of the values calculated in accordance with sections C3.1.1, C3.1.2, C3.1.3, C3.1.4, D6.1.1, D6.1.2, and D6.2.1, where applicable.

See Section C3.6, as applicable, for laterally unrestrained flexural members subjected to both bending and torsional loading, such as *loads* that do not pass through the shear center of the cross-section, a condition which is not considered in the provision of this section.

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* of the effective section (Procedure I) or on the basis of the inelastic reserve capacity (Procedure II), as applicable. The applicable *safety factors* and the *resistance factors* given in this section shall be used to determine the *allowable strength* or *design strength* [*factored resistance*] in accordance with the applicable design method in Section A4, A5 or A6.

For sections with stiffened or partially stiffened compression flanges:

Ω_b = 1.67 (*ASD*)

ϕ_b = 0.95 (*LRFD*)

= 0.90 (*LSD*)

For sections with unstiffened compression flanges:

Ω_b = 1.67 (ASD)

ϕ_b = 0.90 (LRFD)

= 0.90 (LSD)

(a) Procedure I – Based on Initiation of Yielding

The nominal flexural strength [moment resistance], M_n, for the effective *yield moment* shall be calculated in accordance with Eq. C3.1.1-1 as follows:

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

where

S_e = Elastic section modulus of effective section calculated relative to extreme compression or tension fiber at F_y

F_y = Design *yield stress* determined in accordance with Section A7.1

(b) Procedure II — Based on Inelastic Reserve Capacity

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

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

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

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

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

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

The nominal flexural strength [moment resistance], M_n, shall not exceed either $1.25\,S_eF_y$, as determined in accordance with Procedure I of Section C3.1.1 (a) or that causing a maximum compression strain of C_ye_y (no limit is placed on the maximum tensile strain).

where

h = Flat depth of web

t = Base steel *thickness* of element

e_y = Yield strain

= F_y/E

w = Element *flat width*

E = Modulus of elasticity

C_y = Compression strain factor calculated as follows:

(i) *Stiffened compression elements* without intermediate stiffeners

For compression elements without intermediate stiffeners, C_y shall be calculated as follows:

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

$$C_y = 3 - 2\left(\frac{w/t - \lambda_1}{\lambda_2 - \lambda_1}\right) \text{ when } \lambda_1 < \frac{w}{t} < \lambda_2 \qquad (Eq.\ C3.1.1\text{-}2)$$

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

where

$$\lambda_1 = \frac{1.11}{\sqrt{F_y/E}} \qquad (Eq.\ C3.1.1\text{-}3)$$

$$\lambda_2 = \frac{1.28}{\sqrt{F_y / E}} \quad (Eq.\ C3.1.1\text{-}4)$$

(ii) *Unstiffened compression elements*

For unstiffened compression elements, C_y shall be calculated as follows:

(ii-1) Unstiffened compression elements under *stress* gradient causing compression at one longitudinal edge and tension at the other longitudinal edge:

$C_y = 3.0$ when $\lambda \leq \lambda_3$

$C_y = 3 - 2[(\lambda - \lambda_3)/(\lambda_4 - \lambda_3)]$ when $\lambda_3 < \lambda < \lambda_4$ (*Eq*. C3.1.1-5)

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

where

$\lambda_3 = 0.43$

$\lambda_4 = 0.673(1+\psi)$ (*Eq*. C3.1.1-6)

ψ = A value defined in Section B3.2

(ii-2) Unstiffened compression elements under stress gradient causing compression at both longitudinal edges:

$C_y = 1$

(ii-3) Unstiffened compression elements under uniform compression:

$C_y = 1$

(iii) Multiple-stiffened compression elements and compression elements with edge stiffeners

For multiple-stiffened compression elements and compression elements with edge stiffeners, C_y shall be taken as follows:

$C_y = 1$

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 the provisions of Section C3.5.

C3.1.2 Lateral-Torsional Buckling Strength [Resistance]

The provisions of this section shall apply to members with either an open cross-section as specified in Section C3.1.2.1 or closed box sections as specified in Section C3.1.2.2.

Unless otherwise indicated, the following *safety factor* and *resistance factors* and the *nominal strengths* calculated in accordance with Sections C3.1.2.1 and C3.1.2.2 shall be used to determine the allowable flexural strength or design flexural strength [factored moment resistance] in accordance with the applicable design method in Section A4, A5, or A6.

Ω_b = 1.67 (*ASD*)

ϕ_b = 0.90 (*LRFD*)

= 0.90 (*LSD*)

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

The provisions of this section shall 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) subject to *lateral-torsional buckling*. The provisions of this section shall not apply to laterally unbraced compression flanges of otherwise laterally stable sections. See Section D6.1.1 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 in accordance with Eq. C3.1.2.1-1.

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

where

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

F_c shall be determined as follows:

For $F_e \geq 2.78F_y$

The member segment is not subject to lateral-torsional buckling at bending moments less than or equal to M_y. The available flexural strength [moment resistance] shall be determined in accordance with Section C3.1.1(a).

For $2.78F_y > F_e > 0.56F_y$

$$F_c = \frac{10}{9} F_y \left(1 - \frac{10F_y}{36F_e}\right) \quad (Eq.\ C3.1.2.1\text{-}2)$$

For $F_e \leq 0.56F_y$

$$F_c = F_e \quad (Eq.\ C3.1.2.1\text{-}3)$$

where

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

F_e = Elastic critical lateral-torsional buckling stress calculated in accordance with (a) or (b)

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

(i) For bending about the symmetry axis:

$$F_e = \frac{C_b r_o A}{S_f}\sqrt{\sigma_{ey}\sigma_t} \quad \text{for singly- and doubly-symmetric sections} \quad (Eq.\ C3.1.2.1\text{-}4)$$

$$F_e = \frac{C_b r_o A}{2S_f}\sqrt{\sigma_{ey}\sigma_t} \quad \text{for point-symmetric sections} \quad (Eq.\ C3.1.2.1\text{-}5)$$

where

$$C_b = \frac{12.5M_{max}}{2.5M_{max} + 3M_A + 4M_B + 3M_C} \quad (Eq.\ C3.1.2.1\text{-}6)$$

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 shall be 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.

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

$$= \sqrt{r_x^2 + r_y^2 + x_o^2} \qquad (Eq.\ C3.1.2.1\text{-}7)$$

where

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

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

A = *Full unreduced cross-sectional area*

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

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

where

E = Modulus of elasticity of steel

K_y = Effective length factors for bending about y-axis

L_y = Unbraced length of member for bending about y-axis

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

where

G = Shear modulus

J = Saint-Venant torsion constant of cross-section

C_w = Torsional warping constant of cross-section

K_t = Effective length factors for twisting

L_t = Unbraced length of member for twisting

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

For point-symmetric sections, such as Z-sections, x-axis shall be the centroidal axis perpendicular to the web.

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

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

$$F_e = \frac{C_s A \sigma_{ex}}{C_{TF} S_f}\left[j + C_s\sqrt{j^2 + r_o^2(\sigma_t/\sigma_{ex})}\right] \quad (Eq.\ C3.1.2.1\text{-}10)$$

where

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

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

$$\sigma_{ex} = \frac{\pi^2 E}{(K_x L_x / r_x)^2} \quad (Eq.\ C3.1.2.1\text{-}11)$$

where

K_x = Effective length factors for bending about x-axis

L_x = Unbraced length of member for bending about x-axis

$$C_{TF} = 0.6 - 0.4\,(M_1/M_2) \quad (Eq.\ C3.1.2.1\text{-}12)$$

where

M_1 and M_2 = the smaller and the larger bending moment, respectively, at the ends of the unbraced length in the plane of bending; 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

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

(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 shall be permitted to be used in lieu of (a) to calculate F_e:*

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

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

where

d = Depth of section

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

See (a) for definition of other variables.

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 in accordance with this section.

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 in accordance with Section C3.1.1. L_u shall be calculated as follows:

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

See Section C3.1.2.1 for definition of variables.

If the laterally unbraced length of a member is larger than L_u, as calculated in Eq. C3.1.2.2-1, the nominal flexural strength [moment resistance] shall be determined in accordance with Section C3.1.2.1, where the critical *lateral-torsional buckling stress*, F_e, is calculated as follows:

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

where

J = Torsional constant of box section

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

See Section C3.1.2.1 for definition of other variables.

C3.1.3 Flexural Strength [Resistance] of Closed Cylindrical Tubular Members

For closed cylindrical tubular members having a ratio of outside diameter to wall *thickness*, D/t, not greater than 0.441 E/F_y, the nominal flexural strength [moment resistance], M_n, shall be calculated in accordance with Eq. C3.1.3-1. The *safety factor* and *resistance factors* given in this section shall be used to determine the allowable flexural strength or design flexural strength [factored moment resistance] in accordance with the applicable design method in Section A4, A5, or A6.

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

Ω_b = 1.67 (*ASD*)

ϕ_b = 0.95 (*LRFD*)

= 0.90 (*LSD*)

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

$$F_c = 1.25\ F_y \qquad (Eq.\ C3.1.3\text{-}2)$$

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

$$F_c = \left[0.970 + 0.020\left(\frac{E/F_y}{D/t}\right)\right]F_y \qquad (Eq.\ C3.1.3\text{-}3)$$

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

$$F_c = 0.328E/(D/t) \qquad (Eq.\ C3.1.3\text{-}4)$$

where

D = Outside diameter of cylindrical tube

t = *Thickness*

F_c = Critical flexural *buckling stress*

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

See Section C3.1.2.1 for definitions of other variables.

C3.1.4 Distortional Buckling Strength [Resistance]

The provisions of this section shall apply to I-, Z-, C-, and other open cross-section members that employ compression flanges with edge stiffeners, with the exception of members that meet the criteria of Section D6.1.1, D6.1.2 when the R factor of Eq. D6.1.2-1 is employed, or D6.2.1. The nominal flexural strength [moment resistance] shall be calculated in accordance with Eq. C3.1.4-1 or Eq. C3.1.4-2. The *safety factor* and *resistance factors* given in this section shall be used to determine the allowable flexural strength or design flexural strength [factored moment resistance] in accordance with the applicable design method in Section A4, A5, or A6.

Ω_b = 1.67 (*ASD*)
ϕ_b = 0.90 (*LRFD*)
= 0.85 (*LSD*)

For $\lambda_d \le 0.673$

$$M_n = M_y \quad (Eq.\ C3.1.4\text{-}1)$$

For $\lambda_d > 0.673$

$$M_n = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq.\ C3.1.4\text{-}2)$$

where

$$\lambda_d = \sqrt{M_y / M_{crd}} \quad (Eq.\ C3.1.4\text{-}3)$$

$$M_y = S_{fy}F_y \quad (Eq.\ C3.1.4\text{-}4)$$

where

S_{fy} = Elastic section modulus of full unreduced section relative to extreme fiber in first yield

$$M_{crd} = S_fF_d \quad (Eq.\ C3.1.4\text{-}5)$$

where

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

F_d = Elastic *distortional buckling stress* calculated in accordance with either Section C3.1.4(a), (b), or (c)

(a) Simplified Provision for Unrestrained C- and Z-Sections with Simple Lip Stiffeners

For C- and Z-sections that have no rotational restraint of the compression flange and are within the dimensional limits provided in this section, Eq. C3.1.4-6 shall be permitted to be used to calculate a conservative prediction of the distortional buckling stress, F_d. See Section C3.1.4(b) or C3.1.4(c) for alternative provisions and for members outside the dimensional limits of this section.

The following dimensional limits shall apply:

(1) $50 \le h_o/t \le 200$,

(2) $25 \le b_o/t \le 100$,
(3) $6.25 < D/t \le 50$,
(4) $45° \le \theta < 90°$,
(5) $2 \le h_o/b_o \le 8$, and
(6) $0.04 \le D \sin\theta/b_o \le 0.5$.

where
h_o = Out-to-out web depth as defined in Figure B2.3-2
t = Base steel *thickness*
b_o = Out-to-out flange width as defined in Figure B2.3-2
D = Out-to-out lip dimension as defined in Figure B4-1
θ = Lip angle as defined in Figure B4-1

The distortional buckling stress, F_d, shall be calculated as follows:

$$F_d = \beta k_d \frac{\pi^2 E}{12(1-\mu^2)}\left(\frac{t}{b_o}\right)^2 \qquad (Eq.\ C3.1.4\text{-}6)$$

where
β = A value accounting for moment gradient, which is permitted to be conservatively taken as 1.0

$$= 1.0 \le 1 + 0.4(L/L_m)^{0.7}(1 - M_1/M_2)^{0.7} \le 1.3 \qquad (Eq.\ C3.1.4\text{-}7)$$

where
L = Minimum of L_{cr} and L_m

where

$$L_{cr} = 1.2 h_o \left(\frac{b_o D \sin\theta}{h_o t}\right)^{0.6} \le 10 h_o \qquad (Eq.\ C3.1.4\text{-}8)$$

L_m = Distance between discrete restraints that restrict distortional buckling (for continuously restrained members L_m=L_{cr})

M_1 and M_2 = The smaller and the larger end moment, respectively, in the unbraced segment (L_m) of the beam; M_1/M_2 is negative when the moments cause reverse curvature and positive when bent in single curvature

$$k_d = 0.5 \le 0.6\left(\frac{b_o D \sin\theta}{h_o t}\right)^{0.7} \le 8.0 \qquad (Eq.\ C3.1.4\text{-}9)$$

E = Modulus of elasticity
μ = Poisson's ratio

(b) For C- and Z-Sections or any Open Section with a Stiffened Compression Flange Extending to One Side of the Web where the Stiffener is either a Simple Lip or a Complex Edge Stiffener

The provisions of this section shall be permitted to apply to any open section with a single *web* and single edge stiffened compression flange, including those meeting the geometric limits of Section C3.1.4 (a). The distortional buckling stress, F_d, shall be calculated in accordance with Eq. C3.1.4-10 as follows:

$$F_d = \beta \frac{k_{\phi fe} + k_{\phi we} + k_\phi}{\tilde{k}_{\phi fg} + \tilde{k}_{\phi wg}} \qquad (Eq.\ C3.1.4\text{-}10)$$

where

β = A value accounting for moment gradient, which is permitted to be conservatively taken as 1.0

$$= 1.0 \le 1 + 0.4(L/L_m)^{0.7}(1 - M_1/M_2)^{0.7} \le 1.3 \qquad (Eq.\ C3.1.4\text{-}11)$$

where

L = Minimum of L_{cr} and L_m

where

$$L_{cr} = \left(\frac{4\pi^4 h_o (1-\mu^2)}{t^3} \left(I_{xf}(x_o - h_x)^2 + C_{wf} - \frac{I_{xyf}^2}{I_{yf}}(x_o - h_x)^2 \right) + \frac{\pi^4 h_o{}^4}{720} \right)^{1/4}$$

(*Eq.* C3.1.4-12)

where

h_o = Out-to-out web depth as defined in Figure B2.3-2

μ = Poisson's ratio

t = Base steel thickness

I_{xf} = x-axis moment of inertia of the flange

x_o = x distance from the flange/web junction to the centroid of the flange

h_x = x distance from the centroid of the flange to the shear center of the flange

C_{wf} = Warping torsion constant of the flange

I_{xyf} = Product of the moment of inertia of the flange

I_{yf} = y-axis moment of inertia of the flange

In the above, I_{xf}, I_{yf}, I_{xyf}, C_{wf}, x_o, and h_x are properties of the compression flange plus edge stiffener about an x-y axis system located at the centroid of the flange, with the x-axis measured positive to the right from the centroid, and the y-axis positive down from the centroid.

L_m = Distance between discrete restraints that restrict distortional buckling (for continuously restrained members L_m=L_{cr})

M_1 and M_2 = The smaller and the larger end moments, respectively, in the unbraced segment (L_m) of the beam; M_1/M_2 is negative when the moments cause reverse curvature and positive when bent in single curvature

$k_{\phi fe}$ = Elastic rotational stiffness provided by the flange to the flange/web juncture

$$= \left(\frac{\pi}{L}\right)^4 \left(EI_{xf}(x_o - h_x)^2 + EC_{wf} - E\frac{I_{xyf}^2}{I_{yf}}(x_o - h_x)^2 \right) + \left(\frac{\pi}{L}\right)^2 GJ_f \qquad (Eq.\ C3.1.4\text{-}13)$$

where

E = Modulus of elasticity of steel

G = Shear modulus

J_f = St. Venant torsion constant of the compression flange, plus edge stiffener about an x-y axis located at the centroid of the flange, with the x-axis measured positive to the right from the centroid, and the y-axis positive down from the centroid

$k_{\phi we}$ = Elastic rotational stiffness provided by the web to the flange/web juncture

$$= \frac{Et^3}{12(1-\mu^2)}\left(\frac{3}{h_o}+\left(\frac{\pi}{L}\right)^2\frac{19h_o}{60}+\left(\frac{\pi}{L}\right)^4\frac{h_o^3}{240}\right) \quad (Eq.\ C3.1.4\text{-}14)$$

k_ϕ = Rotational stiffness provided by a restraining element (brace, panel, sheathing) to the flange/web juncture of a member (zero if the compression flange is unrestrained)

$\tilde{k}_{\phi fg}$ = Geometric rotational stiffness (divided by the stress F_d) demanded by the flange from the flange/web juncture

$$= \left(\frac{\pi}{L}\right)^2\left[A_f\left((x_o-h_x)^2\left(\frac{I_{xyf}}{I_{yf}}\right)^2-2y_o(x_o-h_x)\left(\frac{I_{xyf}}{I_{yf}}\right)+h_x^2+y_o^2\right)+I_{xf}+I_{yf}\right]$$

(*Eq.* C3.1.4-15)

where

A_f = *Cross-sectional area* of the compression flange plus edge stiffener about an x-y axis located at the centroid of the flange, with the x-axis measured positive to the right from the centroid, and the y-axis positive down from the centroid

y_o = y distance from the flange/web junction to the centroid of the flange

$\tilde{k}_{\phi wg}$ = Geometric rotational stiffness (divided by the stress F_d) demanded by the web from the flange/web juncture

$$= \frac{h_o t\pi^2}{13440}\left(\frac{[45360(1-\xi_{web})+62160]\left(\frac{L}{h_o}\right)^2+448\pi^2+\left(\frac{h_o}{L}\right)^2[53+3(1-\xi_{web})]\pi^4}{\pi^4+28\pi^2\left(\frac{L}{h_o}\right)^2+420\left(\frac{L}{h_o}\right)^4}\right)$$

(*Eq.* C3.1.4-16)

where

ξ_{web} = $(f_1 - f_2)/f_1$, stress gradient in the web, where f_1 and f_2 are the stresses at the opposite ends of the web, $f_1 > f_2$, compression is positive, tension is negative, and the stresses are calculated on the basis of the gross section, (e.g., pure symmetrical bending, $f_1 = -f_2$, $\xi_{web} = 2$)

(c) Rational Elastic Buckling Analysis

A rational elastic *buckling* analysis that considers distortional buckling shall be permitted to be used in lieu of the expressions given in Section C3.1.4 (a) or (b). The safety and resistance factors in Section C3.1.4 shall apply.

C3.2 Shear

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

The nominal shear strength [resistance], V_n, shall be calculated in accordance with Eq. C3.2.1-1. The *safety factor* and *resistance factors* given in this section shall be used to determine the allowable shear strength or design shear strength [factored shear resistance] in accordance with the applicable design method in Section A4, A5, or A6.

$V_n = A_w F_v$ (*Eq*. C3.2.1-1)

$\Omega_v = 1.60$ (*ASD*)

$\phi_v = 0.95$ (*LRFD*)

$= 0.80$ (*LSD*)

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

$F_v = 0.60F_y$ (*Eq*. C3.2.1-2)

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

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

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

$$F_v = \frac{\pi^2 E k_v}{12(1-\mu^2)(h/t)^2} \quad (Eq.\ C3.2.1\text{-}4a)$$

$= 0.904\ Ek_v/(h/t)^2$ (*Eq*. C3.2.1-4b)

where

V_n = Nominal shear strength [resistance]

A_w = Area of *web* element

$= ht$ (*Eq*. C3.2.1-5)

where

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

t = Web *thickness*

F_v = Nominal shear *stress*

E = Modulus of elasticity of steel

k_v = *Shear buckling* coefficient calculated in accordance with (1) or (2) as follows:

(1) For unreinforced webs, $k_v = 5.34$

(2) For webs with transverse stiffeners satisfying the requirements of Section C3.7

when $a/h \le 1.0$

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

when $a/h > 1.0$

$$k_v = 5.34 + \frac{4.00}{(a/h)^2} \quad (Eq.\ C3.2.1\text{-}7)$$

where
a = Shear panel length of unreinforced web element
= Clear distance between transverse stiffeners of reinforced web elements
F_y = Design yield stress as determined in accordance with Section A7.1
μ = Poisson's ratio
= 0.3

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

The provisions of this section shall apply within the following limits:

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

where
d_h = Depth of web hole
h = Depth of flat portion of web measured along plane of web
t = Web thickness
L_h = Length of web hole

For C-Section webs with holes, the shear strength shall be calculated in accordance with Section C3.2.1, multiplied by the reduction factor, q_s, as defined in this section.

When $c/t \ge 54$
$q_s = 1.0$

When $5 \le c/t < 54$
$$q_s = c/(54t) \qquad (Eq.\ C3.2.2\text{-}1)$$

where
$$c = h/2 - d_h/2.83 \quad \text{for circular holes} \qquad (Eq.\ C3.2.2\text{-}2)$$
$$= h/2 - d_h/2 \quad \text{for non-circular holes} \qquad (Eq.\ C3.2.2\text{-}3)$$

C3.3 Combined Bending and Shear

C3.3.1 ASD Method

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

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

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

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:

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

where:

M_n = Nominal flexural strength when bending alone is considered

Ω_b = *Safety factor* for bending (See Section C3.1.1)

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

Ω_v = Safety factor for shear (See Section C3.2)

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], $\overline{M}$, and the required shear strength [factored shear], $\overline{V}$, shall not exceed $\phi_b M_n$ and $\phi_v V_n$, respectively.

For beams with unreinforced *webs*, the required flexural strength [factored moment], $\overline{M}$, and the required shear strength [factored shear], $\overline{V}$, shall also satisfy the following interaction equation:

$$\sqrt{\left(\frac{\overline{M}}{\phi_b M_{nxo}}\right)^2+\left(\frac{\overline{V}}{\phi_v V_n}\right)^2}\le 1.0 \qquad (Eq.\ C3.3.2\text{-}1)$$

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

$$0.6\left(\frac{\overline{M}}{\phi_b M_{nxo}}\right)+\left(\frac{\overline{V}}{\phi_v V_n}\right)\le 1.3 \qquad (Eq.\ C3.3.2\text{-}2)$$

where:

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

$\overline{M}$ = Required flexural strength [factored moment]
= M_u (*LRFD*)
= M_f (*LSD*)

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

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

$\overline{V}$ = Required shear strength [factored shear]
= V_u (LRFD)
= V_f (LSD)

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

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

C3.4 Web Crippling

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

The nominal *web crippling* strength [*resistance*], P_n, shall be determined in accordance with Eq. C3.4.1-1 or Eq. C3.4.1-2, as applicable. The *safety factors* and *resistance factors* in Tables C3.4.1-1 to C3.4.1-5 shall be used to determine the *allowable strength* or *design strength* [factored resistance] in accordance with the applicable design method in Section A4, A5, or A6.

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

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
t = *Web thickness*
F_y = Design *yield stress* as determined in accordance with Section A7.1
θ = Angle between plane of web and plane of bearing surface, $45° \le \theta \le 90°$
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
R = Inside bend radius
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
N = Bearing length [3/4 in. (19 mm) minimum]
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
h = Flat dimension of web measured in plane of web

Alternatively, for an end-one-flange loading condition on a C- or Z-section, the nominal web crippling strength [resistance], P_{nc}, with an overhang on one side, shall be permitted to be calculated as follows, except that P_{nc} shall not be larger than the interior-one-flange loading condition:

$$P_{nc} = \alpha P_n \qquad (Eq.\ C3.4.1\text{-}2)$$

where

P_{nc} = Nominal web crippling strength [resistance] of C and Z-sections with overhang(s)

$$\alpha = \frac{1.34(L_o / h)^{0.26}}{0.009(h/t) + 0.3} \ge 1.0 \qquad (Eq.\ C3.4.1\text{-}3)$$

where

L_o = Overhang length measured from edge of bearing to the end of the member
P_n = Nominal web crippling strength [resistance] with end one-flange loading as calculated by Eq. C3.4.1-1 and Tables C3.4.1-2 and C3.4.1-3

Eq. C3.4.1-2 shall be limited to $0.5 \le L_o/h \le 1.5$ and $h/t \le 154$. For L_o/h or h/t outside these limits, α=1.

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

P_n and P_{nc} shall represent the *nominal strengths* [resistances] for *load* or reaction for one solid web connecting top and bottom flanges. For webs consisting of two or more such sheets, P_n, and P_{nc} 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 shall be defined as the condition where the clear distance between the bearing edges of adjacent opposite concentrated loads or reactions is equal to or greater than 1.5h.

Two-flange loading or reaction shall be defined as the condition where the clear distance between the bearing edges of adjacent opposite concentrated loads or reactions is less than 1.5h.

End loading or reaction shall be defined as the condition where the distance from the edge of the bearing to the end of the member is equal to or less than 1.5h.

Interior loading or reaction shall be defined as the condition where the distance from the edge of the bearing to the end of the member is greater than 1.5h, except as otherwise noted herein.

Table C3.4.1-1 shall apply to I-beams made from two channels connected back-to-back where $h/t \le 200$, $N/t \le 210$, $N/h \le 1.0$ and $\theta = 90°$. See Section C3.4.1 of *Commentary* for further explanation.

TABLE C3.4.1-1
Safety Factors, Resistance Factors, and Coefficients for 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	$R/t \le 5$
			Interior	20.5	0.17	0.11	0.001	1.75	0.85	0.75	$R/t \le 5$
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	$R/t \le 5$
			Interior	20.5	0.17	0.11	0.001	1.75	0.85	0.75	$R/t \le 3$
		Two-Flange Loading or Reaction	End	15.5	0.09	0.08	0.04	2.00	0.75	0.65	$R/t \le 3$
			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	$R/t \le 5$
			Interior	20.5	0.17	0.11	0.001	1.75	0.85	0.75	$R/t \le 3$

Table C3.4.1-2 shall apply to single web channel and C-Sections members where $h/t \leq 200$, $N/t \leq 210$, $N/h \leq 2.0$, and $\theta = 90°$. In Table C3.4.1-2, 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. For unfastened cases, the distance from the edge of bearing to the end of the member shall be extended at least 1.5h.

TABLE C3.4.1-2
Safety Factors, Resistance Factors, and Coefficients for Single Web Channel and C-Sections

Support and Flange Conditions		Load Cases		C	C_R	C_N	C_h	USA and Mexico		Canada LSD ϕ_w	Limits
								ASD Ω_w	LRFD ϕ_w		
Fastened to Support	Stiffened or Partially Stiffened Flanges	One-Flange Loading or Reaction	End	4	0.14	0.35	0.02	1.75	0.85	0.75	$R/t \leq 9$
			Interior	13	0.23	0.14	0.01	1.65	0.90	0.80	$R/t \leq 5$
		Two-Flange Loading or Reaction	End	7.5	0.08	0.12	0.048	1.75	0.85	0.75	$R/t \leq 12$
			Interior	20	0.10	0.08	0.031	1.75	0.85	0.75	$R/t \leq 12$
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	$R/t \leq 5$
			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	

Table C3.4.1-3 shall apply to single web Z-section members where $h/t \le 200$, $N/t \le 210$, $N/h \le 2.0$, and $\theta = 90°$. In Table C3.4.1-3, 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; for unfastened cases, the distance from the edge of bearing to the end of the member shall be extended at least 1.5h.

TABLE C3.4.1-3
Safety Factors, Resistance Factors, and Coefficients for Single Web Z-Sections

Support and Flange Conditions		Load Cases		C	C_R	C_N	C_h	USA and Mexico ASD Ω_w	USA and Mexico LRFD ϕ_w	Canada LSD ϕ_w	Limits
Fastened to Support	Stiffened or Partially Stiffened Flanges	One-Flange Loading or Reaction	End	4	0.14	0.35	0.02	1.75	0.85	0.75	$R/t \le 9$
			Interior	13	0.23	0.14	0.01	1.65	0.90	0.80	$R/t \le 5.5$
		Two-Flange Loading or Reaction	End	9	0.05	0.16	0.052	1.75	0.85	0.75	$R/t \le 12$
			Interior	24	0.07	0.07	0.04	1.85	0.80	0.70	$R/t \le 12$
Unfastened	Stiffened or Partially Stiffened Flanges	One-Flange Loading or Reaction	End	5	0.09	0.02	0.001	1.80	0.85	0.75	$R/t \le 5$
			Interior	13	0.23	0.14	0.01	1.65	0.90	0.80	
		Two-Flange Loading or Reaction	End	13	0.32	0.05	0.04	1.65	0.90	0.80	$R/t \le 3$
			Interior	24	0.52	0.15	0.001	1.90	0.80	0.65	
	Unstiffened Flanges	One-Flange Loading or Reaction	End	4	0.40	0.60	0.03	1.80	0.85	0.70	$R/t \le 2$
			Interior	13	0.32	0.10	0.01	1.80	0.85	0.70	$R/t \le 1$
		Two-Flange Loading or Reaction	End	2	0.11	0.37	0.01	2.00	0.75	0.65	$R/t \le 1$
			Interior	13	0.47	0.25	0.04	1.90	0.80	0.65	

Table C3.4.1-4 shall apply to single hat section members where $h/t \le 200$, $N/t \le 200$, $N/h \le 2$, and $\theta = 90°$.

TABLE C3.4.1-4
Safety Factors, Resistance Factors, and Coefficients for 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	1.80	0.85	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	1.80	0.85	0.70	$R/t \le 4$

Table C3.4.1-5 shall apply to multi-web section members where $h/t \le 200$, $N/t \le 210$, $N/h \le 3$, and $45° \le \theta \le 90°$.

TABLE C3.4.1-5
Safety Factors, Resistance Factors, and Coefficients for 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	4	0.04	0.25	0.025	1.70	0.90	0.80	$R/t \le 20$
		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	0.04	0.29	0.028	2.45	0.60	0.50	$R/t \le 20$
		Interior	8	0.10	0.17	0.004	1.75	0.85	0.75	
	Two-Flange Loading or Reaction	End	6	0.16	0.15	0.050	1.65	0.90	0.80	$R/t \le 5$
		Interior	17	0.10	0.10	0.046	1.65	0.90	0.80	

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

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

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

The provisions of this section shall apply within the following limits:

(1) $d_h/h \le 0.7$,
(2) $h/t \le 200$,
(3) Hole centered at mid-depth of web,
(4) Clear distance between holes ≥ 18 in. (457 mm),
(5) Distance between end of member and edge of hole ≥ d,
(6) Non-circular holes, corner radii ≥ 2t,
(7) Non-circular holes, $d_h \le 2.5$ in. (64 mm) and $L_h \le 4.5$ in. (114 mm),
(8) Circular holes, diameters ≤ 6 in. (152 mm), and
(9) $d_0 > 9/16$ in. (14 mm).

where

d_h = Depth of web hole
h = Depth of flat portion of web measured along plane of web
t = Web *thickness*
d = Depth of cross-section
L_h = Length of web hole

For end-one flange reaction (Equation C3.4.1-1 with Table C3.4.1-2) where a web hole is not within the bearing length, the reduction factor, R_c, shall be calculated as follows:

$$R_c = 1.01 - 0.325 d_h/h + 0.083 x/h \le 1.0 \quad (Eq.\ C3.4.2\text{-}1)$$

N ≥ 1 in. (25 mm)

For interior-one flange reaction (Equation C3.4.1-1 with Table C3.4.1-2) where any portion of a web hole is not within the bearing length, the reduction factor, R_c, shall be calculated as follows:

$$R_c = 0.90 - 0.047 d_h/h + 0.053 x/h \le 1.0 \quad (Eq.\ C3.4.2\text{-}2)$$

N ≥ 3 in. (76 mm)

where

x = Nearest distance between web hole and edge of bearing
N = Bearing length

C3.5 Combined Bending and Web Crippling

C3.5.1 ASD Method

Unreinforced flat *webs* of shapes subjected to a combination of bending and concentrated *load* or reaction shall be designed such that the moment, M, and the concentrated load or reaction, P, satisfy $M \le M_{nxo}/\Omega_b$, and $P \le P_n/\Omega_w$. In addition, the following requirements in (a), (b), and (c), as applicable, shall be satisfied.

(a) For shapes having single unreinforced webs, Eq. C3.5.1-1 shall be satisfied as follows:

$$0.91\left(\frac{P}{P_n}\right) + \left(\frac{M}{M_{nxo}}\right) \le \frac{1.33}{\Omega} \quad (Eq.\ C3.5.1\text{-}1)$$

Exception: At the interior supports of continuous spans, Eq. C3.5.1-1 shall not apply to deck or beams with two or more single webs, provided the compression edges of adjacent webs are laterally supported in the negative moment region by continuous or intermittently connected flange elements, rigid cladding, or lateral bracing, and the spacing between adjacent webs does not exceed 10 in. (254 mm).

(b) For shapes having multiple unreinforced webs such as I-sections made of two C-sections connected back-to-back, or similar sections that provide a high degree of restraint against rotation of the web (such as I-sections made by welding two angles to a C-section), Eq. C3.5.1-2 shall be satisfied as follows:

$$0.88\left(\frac{P}{P_n}\right)+\left(\frac{M}{M_{nxo}}\right)\le\frac{1.46}{\Omega} \qquad (Eq.\ C3.5.1\text{-}2)$$

(c) For the support point of two nested Z-shapes, Eq. C3.5.1-3 shall be satisfied as follows:

$$0.86\left(\frac{P}{P_n}\right)+\left(\frac{M}{M_{nxo}}\right)\le\frac{1.65}{\Omega} \qquad (Eq.\ C3.5.1\text{-}3)$$

Eq. C3.5.1-3 shall apply to shapes that meet the following limits:

$h/t \le 150$,
$N/t \le 140$,
$F_y \le 70$ ksi (483 MPa or 4920 kg/cm^2), and
$R/t \le 5.5$.

The following conditions shall also be satisfied:

(1) The ends of each section are connected to the other section by a minimum of two 1/2 in. (12.7 mm) diameter A307 bolts through the web.
(2) The combined section is connected to the support by a minimum of two 1/2 in. (12.7 mm) diameter A307 bolts through the flanges.
(3) The webs of the two sections are in contact.
(4) The ratio of the thicker to the thinner part does not exceed 1.3.

The following notation shall apply to this section:

M = Required flexural strength at, or immediately adjacent to, the point of application of the concentrated load or reaction, P
P = Required strength for concentrated load or reaction in the presence of bending moment
M_{nxo} = Nominal flexural strength about the centroidal x-axis determined in accordance with Section C3.1.1
Ω_b = *Safety factor* for bending (See Section C3.1.1)
P_n = *Nominal strength* for concentrated load or reaction in absence of bending moment determined in accordance with Section C3.4
Ω_w = Safety factor for *web crippling* (See Section C3.4)
Ω = Safety factor for combined bending and web crippling
= 1.70

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 such that the moment, $\overline{M}$, and the concentrated load or reaction, $\overline{P}$, satisfy $\overline{M} \leq \phi_b M_{nxo}$ and $\overline{P} \leq \phi_w P_n$. In addition, the following requirements in (a), (b), and (c), as applicable, shall be satisfied.

(a) For shapes having single unreinforced webs, Eq. C3.5.2-1 shall be satisfied as follows:

$$0.91\left(\frac{\overline{P}}{P_n}\right)+\left(\frac{\overline{M}}{M_{nxo}}\right)\leq 1.33\phi \quad (Eq.\ C3.5.2\text{-}1)$$

where

ϕ = 0.90 (*LRFD*)

= 0.75 (*LSD*)

Exception: At the interior supports of continuous spans, Eq. C3.5.2-1 shall not apply to deck or beams with two or more single webs, provided the compression edges of adjacent webs are laterally supported in the negative moment region by continuous or intermittently connected flange elements, rigid cladding, or lateral bracing, and the spacing between adjacent webs does not exceed 10 in. (254 mm).

(b) For shapes having multiple unreinforced webs such as I-sections made of two C-sections connected back-to-back, or similar sections that provide a high degree of restraint against rotation of the web (such as I-sections made by welding two angles to a C-section), Eq. C3.5.2-2 shall be satisfied as follows:

$$0.88\left(\frac{\overline{P}}{P_n}\right)+\left(\frac{\overline{M}}{M_{nxo}}\right)\leq 1.46\phi \quad (Eq.\ C3.5.2\text{-}2)$$

where

ϕ = 0.90 (LRFD)

= 0.75 (LSD)

(c) For two nested Z-shapes, Eq. C3.5.2-3 shall be satisfied as follows:

$$0.86\left(\frac{\overline{P}}{P_n}\right)+\left(\frac{\overline{M}}{M_{nxo}}\right)\leq 1.65\phi \quad (Eq.\ C3.5.2\text{-}3)$$

where

ϕ = 0.90 (LRFD)

= 0.80 (LSD)

Eq. C3.5.2-3 shall apply to shapes that meet the following limits:

$h/t \leq 150$,

$N/t \leq 140$,

$F_y \leq 70$ ksi (483 MPa or 4920 kg/cm²), and

$R/t \leq 5.5$.

The following conditions shall also be satisfied:

(1) The ends of each section are connected to the other section by a minimum of two 1/2 in. (12.7 mm) diameter A307 bolts through the web.
(2) The combined section is connected to the support by a minimum of two 1/2 in. (12.7 mm) diameter A307 bolts through the flanges.
(3) The webs of the two sections are in contact.
(4) The ratio of the thicker to the thinner part does not exceed 1.3.

The following notation shall apply in this section:

$\overline{M}$ = Required flexural strength [factored moment] at, or immediately adjacent to, the point of application of the concentrated load or reaction $\overline{P}$
= M_u (LRFD)
= M_f (LSD)

$\overline{P}$ = *Required strength* for concentrated load or reaction [factored concentrated load or reaction] in presence of bending moment
= P_u (LRFD)
= P_f (LSD)

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

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

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

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

C3.6 Combined Bending and Torsional Loading

For laterally unrestrained flexural members subjected to both bending and torsional loading, the available flexural strength [factored moment resistance] calculated in accordance with Section C3.1.1(a) shall be reduced by multiplying it by a reduction factor, R.

As specified in Equation C3.6-1, the reduction factor, R, shall be equal to the ratio of the normal *stresses* due to bending alone divided by the combined stresses due to both bending and torsional warping at the point of maximum combined stress on the cross-section.

$$R = \frac{f_{bending}}{f_{bending} + f_{torsion}} \leq 1 \qquad (Eq.\ C3.6\text{-}1)$$

Stresses shall be calculated using full section properties for the torsional stresses and effective section properties for the bending stresses. For C-sections with edge stiffened flanges, if the maximum combined compressive stresses occur at the junction of the *web* and flange, the R factor shall be permitted to be increased by 15 percent, but the R factor shall not be greater than 1.0.

The provisions of this section shall not be applied when the provisions of Sections D6.1.1 and D6.1.2 are used.

C3.7 Stiffeners

C3.7.1 Bearing Stiffeners

Bearing stiffeners attached to beam *webs* at points of concentrated *loads* or reactions shall be designed as compression members. Concentrated loads or reactions shall be applied directly into the stiffeners, or each stiffener shall be fitted accurately to the flat portion of the flange to provide direct load bearing into the end of the stiffener. Means for shear transfer between the stiffener and the web shall be provided in accordance with Chapter E. For concentrated loads or reactions, the *nominal strength* [resistance], P_n, shall be the smaller value calculated by (a) and (b) of this section. The *safety factor* and *resistance factors* provided in this section shall be used to determine the *allowable strength*, or *design* *strength* [*factored resistance*] in accordance with the applicable design method in Section A4, A5, or A6.

Ω_c = 2.00 (*ASD*)

ϕ_c = 0.85 (*LRFD*)

= 0.80 (*LSD*)

(a) $P_n = F_{wy}A_c$ (*Eq*. C3.7.1-1)

(b) P_n = Nominal axial strength [resistance] evaluated in accordance with Section C4.1(a), with A_e replaced by A_b

where

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

A_c = $18t^2 + A_s$, for bearing stiffener at interior support or under concentrated load (*Eq*. C3.7.1-2)

= $10t^2 + A_s$, for bearing stiffener at end support (*Eq*. C3.7.1-3)

where

t = Base steel *thickness* of beam web

A_s = *Cross-sectional area* of bearing stiffener

A_b = $b_1t + A_s$, for bearing stiffener at interior support or under concentrated load (*Eq*. C3.7.1-4)

= $b_2t + A_s$, for bearing stiffener at end support (*Eq*. C3.7.1-5)

where

$b_1 = 25t\,[0.0024(L_{st}/t) + 0.72] \le 25t$ (*Eq*. C3.7.1-6)

$b_2 = 12t\,[0.0044(L_{st}/t) + 0.83] \le 12t$ (*Eq*. C3.7.1-7)

where

L_{st} = Length of bearing stiffener

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

C3.7.2 Bearing Stiffeners in C-Section Flexural Members

For two-flange loading of C-section flexural members with bearing stiffeners that do not meet the requirements of Section C3.7.1, the *nominal strength* [resistance], P_n, shall be calculated in accordance with Eq. C3.7.2-1. The *safety factor* and *resistance factors* in this section shall be used to determine the *allowable strength* or *design strength* [*factored resistance*] in accordance with the applicable design method in Section A4, A5, or A6.

$P_n = 0.7(P_{wc} + A_eF_y) \ge P_{wc}$ (*Eq*. C3.7.2-1)

Ω = 1.70 (*ASD*)

ϕ = 0.90 (*LRFD*)

= 0.80 (*LSD*)

where

P_{wc} = Nominal *web crippling* strength [resistance] for C-section flexural member calculated in accordance with Eq. C3.4.1-1 for single *web* members, at end or interior locations

A_e = *Effective area* of bearing stiffener subjected to uniform compressive *stress*, calculated at *yield stress*

F_y = Yield stress of bearing stiffener steel

Eq. C3.7.2-1 shall apply within the following limits:

(1) Full bearing of the stiffener is required. If the bearing width is narrower than the stiffener such that one of the stiffener flanges is unsupported, P_n is reduced by 50 percent.

(2) Stiffeners are C-section stud or track members with a minimum web depth of 3-1/2 in. (89 mm) and a minimum base steel *thickness* of 0.0329 in. (0.84 mm).

(3) The stiffener is attached to the flexural member web with at least three fasteners (screws or bolts).

(4) The distance from the flexural member flanges to the first fastener(s) is not less than d/8, where d is the overall depth of the flexural member.

(5) The length of the stiffener is not less than the depth of the flexural member minus 3/8 in. (9 mm).

(6) The bearing width is not less than $1\text{-}^1/_2$ in. (38 mm).

C3.7.3 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 calculated in accordance with Equation C3.7.3-1 as follows:

$$I_{smin} = 5ht^3[h/a - 0.7(a/h)] \geq (h/50)^4 \qquad (Eq.\ C3.7.3\text{-}1)$$

where

h and t = Values as defined in Section B1.2

a = Distance between shear stiffeners

The *gross area* of shear stiffeners shall not be less than:

$$A_{st} = \frac{1 - C_v}{2}\left[\frac{a}{h} - \frac{(a/h)^2}{(a/h) + \sqrt{1 + (a/h)^2}}\right]YDht \qquad (Eq.\ C3.7.3\text{-}2)$$

where

$$C_v = \frac{1.53Ek_v}{F_y(h/t)^2} \text{ when } C_v \leq 0.8 \qquad (Eq.\ C3.7.3\text{-}3)$$

$$= \frac{1.11}{h/t}\sqrt{\frac{Ek_v}{F_y}} \text{ when } C_v > 0.8 \qquad (Eq.\ C3.7.3\text{-}4)$$

where

$$k_v = 4.00 + \frac{5.34}{(a/h)^2} \quad \text{when } a/h \leq 1.0 \qquad (Eq.\ C3.7.3\text{-}5)$$

$$= 5.34 + \frac{4.00}{(a/h)^2} \quad \text{when } a/h > 1.0 \qquad (Eq.\ C3.7.3\text{-}6)$$

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

D = 1.0 for stiffeners furnished in pairs
= 1.8 for single-angle stiffeners
= 2.4 for single-plate stiffeners

C3.7.4 Non-Conforming Stiffeners

The *available strength* [*factored resistance*] of members with stiffeners that do not meet the requirements of Section C3.7.1, C3.7.2, or C3.7.3, such as stamped or rolled-in stiffeners, shall be determined by tests in accordance with Chapter F or *rational engineering analysis* in accordance with Section A1.2(b).

C4 Concentrically Loaded Compression Members

The available axial strength [factored compressive resistance] shall be the smaller of the values calculated in accordance with Sections C4.1, C4.2, D1.2, D6.1.3, and D6.1.4, where applicable.

C4.1 Nominal Strength for Yielding, Flexural, Flexural-Torsional and Torsional Buckling

This section shall apply 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 in accordance with Eq. C4.1-1. The *safety factor* and *resistance factors* in this section shall be used to determine the allowable axial strength or design axial strength [factored compressive resistance] in accordance with the applicable design method in Section A4, A5, or A6.

$$P_n = A_e F_n \qquad (Eq.\ C4.1\text{-}1)$$

Ω_c = 1.80 (*ASD*)

ϕ_c = 0.85 (*LRFD*)

= 0.80 (*LSD*)

where

A_e = *Effective area* calculated at stress F_n. For sections with circular holes, A_e is determined from the *effective width* in accordance with 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, it is permitted to determine A_e by ignoring the holes. For closed cylindrical tubular members, A_e is provided in Section C4.1.5.

F_n shall be calculated as follows:

For $\lambda_c \le 1.5$

$$F_n = \left(0.658^{\lambda_c^2}\right) F_y \qquad (Eq.\ C4.1\text{-}2)$$

For $\lambda_c > 1.5$

$$F_n = \left[\frac{0.877}{\lambda_c^2}\right] F_y \qquad (Eq.\ C4.1\text{-}3)$$

where

$$\lambda_c = \sqrt{\frac{F_y}{F_e}} \qquad (Eq.\ C4.1\text{-}4)$$

F_e = The least of the applicable elastic *flexural*, *torsional* and *flexural-torsional buckling* stress determined in accordance with Sections C4.1.1 through C4.1.5

(b) Concentrically loaded angle sections shall be designed for an additional bending moment as specified in the definitions of M_x and M_y (ASD) or $\overline{M}_x$ and $\overline{M}_y$ (LRFD or LSD) in Section C5.2.

C4.1.1 Sections Not Subject to Torsional or Flexural-Torsional Buckling

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

$$F_e = \frac{\pi^2 E}{(KL/r)^2} \qquad (Eq.\ C4.1.1\text{-}1)$$

where

E = Modulus of elasticity of steel
K = Effective length factor
L = Laterally unbraced length of member
r = Radius of gyration of full unreduced cross section about 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 that 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 is suitable. In a frame that 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.

C4.1.2 Doubly- or Singly-Symmetric Sections Subject to Torsional or Flexural-Torsional Buckling

For *singly-symmetric sections* subject to *flexural-torsional buckling*, F_e shall be taken as the smaller of F_e calculated in accordance with Section C4.1.1 and F_e calculated as follows:

$$F_e = \frac{1}{2\beta}\left[(\sigma_{ex} + \sigma_t) - \sqrt{(\sigma_{ex} + \sigma_t)^2 - 4\beta\sigma_{ex}\sigma_t}\right] \qquad (Eq.\ C4.1.2\text{-}1)$$

Alternatively, a conservative estimate of F_e shall be permitted to be calculated as follows:

$$F_e = \frac{\sigma_t \sigma_{ex}}{\sigma_t + \sigma_{ex}} \qquad (Eq.\ C4.1.2\text{-}2)$$

where

$\beta = 1 - (x_o/r_o)^2$ (*Eq*. C4.1.2-3)

σ_t and σ_{ex} = Values as defined in Section C3.1.2.1

For singly-symmetric sections, the x-axis shall be selected as the axis of symmetry.

For *doubly-symmetric sections* subject to *torsional buckling*, F_e shall be taken as the smaller of F_e calculated in accordance with Section C4.1.1 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), F_e shall be computed using Eq. C4.1.1-1 where r is the least radius of gyration.

C4.1.3 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 C4.1.1 using the minor principal axis of the section.

C4.1.4 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 shall be permitted to be tested in accordance with Chapter F.

C4.1.5 Closed Cylindrical Tubular Sections

For closed cylindrical tubular members having a ratio of outside diameter to wall thickness, D/t, not greater than 0.441 E/F_y and 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 elastic *flexural buckling* stress, F_e, shall be calculated in accordance with Section C4.1.1, and the *effective area*, A_e, shall be calculated as follows:

$$A_e = A_o + R(A - A_o) \qquad (Eq.\ C4.1.5\text{-}1)$$

where

$$A_o = \left[\frac{0.037}{(DF_y)/(tE)} + 0.667\right] A \le A \quad \text{for} \frac{D}{t} \le 0.441\frac{E}{F_y} \qquad (Eq.\ C4.1.5\text{-}2)$$

where

D = Outside diameter of cylindrical tube

F_y = *Yield stress*

t = *Thickness*

E = Modulus of elasticity of steel

A = Area of full unreduced cross-section

$$R = F_y/(2F_e) \le 1.0 \qquad (Eq.\ C4.1.5\text{-}3)$$

C4.2 Distortional Buckling Strength [Resistance]

The provisions of this section shall apply to I-, Z-, C-, Hat, and other open cross-section members that employ flanges with edge stiffeners, with the exception of members that are designed in accordance with Section D6.1.2. The *nominal axial strength* [compressive *resistance*] shall be calculated in accordance with Eqs. C4.2-1 and C4.2-2. The *safety factor* and *resistance factors* in this section shall be used to determine the allowable compressive strength or design compressive strength [resistance] in accordance with the applicable design method in Section A4, A5, or A6.

Ω_b = 1.80 (*ASD*)
ϕ_b = 0.85 (*LRFD*)
= 0.80 (*LSD*)

For $\lambda_d \le 0.561$

$$P_n = P_y \qquad (Eq.\ C4.2\text{-}1)$$

For $\lambda_d > 0.561$

$$P_n = \left(1 - 0.25\left(\frac{P_{crd}}{P_y}\right)^{0.6}\right)\left(\frac{P_{crd}}{P_y}\right)^{0.6} P_y \qquad (Eq.\ C4.2\text{-}2)$$

where

$$\lambda_d = \sqrt{P_y/P_{crd}} \qquad (Eq.\ C4.2\text{-}3)$$

P_n = Nominal axial strength

$$P_y = A_g F_y \qquad (Eq.\ C4.2\text{-}4)$$

where

A_g = *Gross area* of the cross-section

F_y = *Yield stress*

$$P_{crd} = A_g F_d \qquad (Eq.\ C4.2\text{-}5)$$

where

F_d = Elastic *distortional buckling stress* calculated in accordance with either Section C4.2(a), (b), or (c)

(a) Simplified Provision for Unrestrained C- and Z-Sections with Simple Lip Stiffeners

For C- and Z-sections that have no rotational restraint of the flange and that are within the dimensional limits provided in this section, Eq. C4.2-6 shall be permitted to be used to calculate a conservative prediction of distortional buckling stress, F_d. See Section C4.2(b) or C4.2(c) for alternative options for members outside the dimensional limits.

The following dimensional limits shall apply:

(1) $50 \le h_o/t \le 200$,
(2) $25 \le b_o/t \le 100$,
(3) $6.25 < D/t \le 50$,
(4) $45° \le \theta \le 90°$,
(5) $2 \le h_o/b_o \le 8$, and
(6) $0.04 \le D \sin\theta/b_o \le 0.5$.

where
h_o = Out-to-out *web* depth as defined in Figure B2.3-2
b_o = Out-to-out flange width as defined in Figure B2.3-2
D = Out-to-out lip dimension as defined in Figure B4-1
t = Base steel *thickness*
θ = Lip angle as defined in Figure B4-1

The distortional buckling stress, F_d, shall be calculated in accordance with Eq. C4.2-6:

$$F_d = \alpha k_d \frac{\pi^2 E}{12(1-\mu^2)}\left(\frac{t}{b_o}\right)^2 \qquad (Eq.\ C4.2\text{-}6)$$

where
α = A value that accounts for the benefit of an unbraced length, L_m, shorter than L_{cr}, but can be conservatively taken as 1.0
= 1.0 for $L_m \geq L_{cr}$
= $(L_m/L_{cr})^{\ln(L_m/L_{cr})}$ for $L_m < L_{cr}$ (*Eq.* C4.2-7)

where
L_m = Distance between discrete restraints that restrict distortional buckling (for continuously restrained members $L_m = L_{cr}$, but the restraint can be included as a rotational spring, k_ϕ, in accordance with the provisions in C4.2 (b) or (c))

$$L_{cr} = 1.2 h_o \left(\frac{b_o D \sin\theta}{h_o t}\right)^{0.6} \leq 10 h_o \qquad (Eq.\ C4.2\text{-}8)$$

$$k_d = 0.05 \leq 0.1\left(\frac{b_o D \sin\theta}{h_o t}\right)^{1.4} \leq 8.0 \qquad (Eq.\ C4.2\text{-}9)$$

E = Modulus of elasticity of steel
μ = Poisson's ratio

(b) For C- and Z-Sections or Hat Sections or any Open Section with Stiffened Flanges of Equal Dimension where the Stiffener is either a Simple Lip or a Complex Edge Stiffener

The provisions of this section shall apply to any open section with stiffened flanges of equal dimension, including those meeting the geometric limits of C4.2(a).

$$F_d = \frac{k_{\phi fe} + k_{\phi we} + k_\phi}{\tilde{k}_{\phi fg} + \tilde{k}_{\phi wg}} \qquad (Eq.\ C4.2\text{-}10)$$

where
$k_{\phi fe}$ = Elastic rotational stiffness provided by the flange to the flange/web juncture, in accordance with Eq. C3.1.4-13
$k_{\phi we}$ = Elastic rotational stiffness provided by the web to the flange/web juncture

$$= \frac{Et^3}{6h_o(1-\mu^2)} \qquad (Eq.\ C4.2\text{-}11)$$

k_ϕ = Rotational stiffness provided by restraining elements (brace, panel, sheathing) to the flange/web juncture of a member (zero if the flange is unrestrained). If rotational stiffness provided to the two flanges is dissimilar, the smaller rotational stiffness is used.

$\tilde{k}_{\phi fg}$ = Geometric rotational stiffness (divided by the stress F_d) demanded by the flange from the flange/web juncture, in accordance with Eq. C3.1.4-15

$\tilde{k}_{\phi wg}$ = Geometric rotational stiffness (divided by the stress F_d) demanded by the web from the flange/web juncture

$$= \left(\frac{\pi}{L}\right)^2 \frac{t h_o^3}{60} \qquad \text{(Eq. C4.2-12)}$$

where

L = Minimum of L_{cr} and L_m

where

$$L_{cr} = \left(\frac{6\pi^4 h_o \left(1-\mu^2\right)}{t^3} \left(I_{xf}\left(x_o - h_x\right)^2 + C_{wf} - \frac{I_{xyf}^2}{I_{yf}}\left(x_o - h_x\right)^2 \right) \right)^{1/4} \qquad \text{(Eq. C4.2-13)}$$

L_m = Distance between discrete restraints that restrict distortional buckling (for continuously restrained members $L_m = L_{cr}$)

See Section C3.1.4 (b) for definition of variables in Eq. C4.2-13.

(c) Rational elastic buckling analysis

A rational elastic buckling analysis that considers distortional buckling shall be permitted to be used in lieu of the expressions given in Section C4.2(a) or (b). The safety and resistance factors in Section C4.2 shall apply.

C5 Combined Axial Load and Bending

C5.1 Combined Tensile Axial Load and Bending

C5.1.1 ASD Method

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

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

and

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

where

Ω_b = 1.67

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

M_{nxt}, M_{nyt} = $S_{ft}F_y$ (Eq. C5.1.1-3)

where

S_{ft} = Section modulus of full unreduced section relative to extreme tension fiber about appropriate axis

F_y = Design *yield stress* determined in accordance with Section A7.1
Ω_t = 1.67
T = Required tensile axial strength
T_n = Nominal tensile axial strength determined in accordance with Section C2
M_{nx}, M_{ny} = Nominal flexural strengths about centroidal axes determined in accordance with Section C3.1

C5.1.2 LRFD and LSD Methods

The *required strengths* [factored tension and moments] $\overline{T}$, $\overline{M}_x$, and $\overline{M}_y$ shall satisfy the following interaction equations:

$$\frac{\overline{M}_x}{\phi_b M_{nxt}} + \frac{\overline{M}_y}{\phi_b M_{nyt}} + \frac{\overline{T}}{\phi_t T_n} \leq 1.0 \qquad (Eq.\ C5.1.2\text{-}1)$$

$$\frac{\overline{M}_x}{\phi_b M_{nx}} + \frac{\overline{M}_y}{\phi_b M_{ny}} - \frac{\overline{T}}{\phi_t T_n} \leq 1.0 \qquad (Eq.\ C5.1.2\text{-}2)$$

where
$\overline{M}_x$, $\overline{M}_y$ = Required flexural strengths [factored moments] with respect to centroidal axes
$\overline{M}_x = M_{ux}$, $\overline{M}_y = M_{uy}$ (*LRFD*)
$\overline{M}_x = M_{fx}$, $\overline{M}_y = M_{fy}$ (*LSD*)
ϕ_b = For flexural strength [moment 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)
For closed cylindrical tubular members (Section C3.1.3), ϕ_b = 0.95 (LRFD) and 0.90 (LSD)
M_{nxt}, M_{nyt} = $S_{ft}F_y$ (*Eq.* C5.1.2-3)
where
S_{ft} = Section modulus of full unreduced section relative to extreme tension fiber about appropriate axis
F_y = Design *yield stress* determined in accordance with Section A7.1
$\overline{T}$ = Required tensile axial strength [factored tension]
= T_u (LRFD)
= T_f (LSD)
ϕ_t = 0.95 (LRFD)
= 0.90 (LSD)
T_n = Nominal tensile axial strength [resistance] determined in accordance with Section C2
M_{nx}, M_{ny} = Nominal flexural strengths [moment resistances] about centroidal axes determined in accordance with Section C3.1

C5.2 Combined Compressive Axial Load and Bending

C5.2.1 ASD Method

The *required strengths* P, M_x, and M_y shall be determined using first order elastic analysis and shall satisfy the following interaction equations. Alternatively, the required strengths P, M_x, and M_y shall be determined in accordance with Appendix 2 and shall satisfy the following interaction equations using the values for K_x, K_y, α_x, α_y, C_{mx}, and C_{my} specified in Appendix 2. In addition, each individual ratio in Eqs. C5.2.1-1 to C5.2.1-3 shall not exceed unity.

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.

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

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

When $\Omega_c P/P_n \le 0.15$, the following equation shall be permitted to be used in lieu of the above two equations:

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

where
Ω_c = 1.80
P = Required compressive axial strength
P_n = Nominal axial strength determined in accordance with Section C4
Ω_b = 1.67
M_x, M_y = Required flexural strengths with respect to centroidal axes of effective section determined for required compressive axial strength alone.
M_{nx}, M_{ny} = Nominal flexural strengths about centroidal axes determined in accordance with Section C3.1

$$\alpha_x = 1 - \frac{\Omega_c P}{P_{Ex}} > 0 \qquad (Eq.\ C5.2.1\text{-}4)$$

$$\alpha_y = 1 - \frac{\Omega_c P}{P_{Ey}} > 0 \qquad (Eq.\ C5.2.1\text{-}5)$$

where

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

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

where

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

K_x = Effective length factor for buckling about x-axis

L_x = Unbraced length for bending about x-axis

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

K_y = Effective length factor for buckling about y-axis

L_y = Unbraced length for bending about y-axis

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

C_{mx}, C_{my} = Coefficients whose values are determined in accordance with (a), (b), or (c) as follows:

(a) For compression members in frames subject to *joint* translation (sidesway)

$C_m = 0.85$

(b) For restrained compression members in frames braced against joint translation and not subject to transverse loading between their supports in the plane of bending

$$C_m = 0.6 - 0.4\ (M_1/M_2) \qquad (Eq.\ C5.2.1\text{-}8)$$

where

M_1/M_2 = 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

(c) 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 is to be determined by rational analysis. However, in lieu of such analysis, the following values are permitted to be used:

(1) For members whose ends are restrained, $C_m = 0.85$, and

(2) For members whose ends are unrestrained, $C_m = 1.0$.

C5.2.2 LRFD and LSD Methods

The *required strengths* [factored compression and moments] $\overline{P}$, $\overline{M}_x$, and $\overline{M}_y$ shall be determined using first order elastic analysis and shall satisfy the following interaction equations. Alternatively, the required strengths [factored axial force and moment] $\overline{P}$, $\overline{M}_x$, and $\overline{M}_y$ shall be determined in accordance with Appendix 2 and shall satisfy the following interaction equations using the values for K_x, K_y, α_x, α_y, C_{mx}, and C_{my} specified in Appendix 2. In addition, each individual ratio in Eqs. C5.2.2-1 to C5.2.2-3 shall not exceed unity.

For *singly-symmetric* unstiffened angle sections with unreduced *effective area*, $\overline{M}_y$ 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), $\overline{M}_y$ shall be taken either as the required flexural strength [factored moment] or the required flexural strength [factored moment] plus $(\overline{P})L/1000$, whichever results in a lower permissible value of $\overline{P}$.

$$\frac{\overline{P}}{\phi_c P_n} + \frac{C_{mx}\overline{M}_x}{\phi_b M_{nx}\alpha_x} + \frac{C_{my}\overline{M}_y}{\phi_b M_{ny}\alpha_y} \leq 1.0 \qquad (Eq.\ C5.2.2\text{-}1)$$

$$\frac{\overline{P}}{\phi_c P_{no}} + \frac{\overline{M}_x}{\phi_b M_{nx}} + \frac{\overline{M}_y}{\phi_b M_{ny}} \leq 1.0 \qquad (Eq.\ C5.2.2\text{-}2)$$

When $\overline{P}/\phi_c P_n \leq 0.15$, the following equation shall be permitted to be used in lieu of the above two equations:

$$\frac{\overline{P}}{\phi_c P_n} + \frac{\overline{M}_x}{\phi_b M_{nx}} + \frac{\overline{M}_y}{\phi_b M_{ny}} \leq 1.0 \qquad (Eq.\ C5.2.2\text{-}3)$$

where

$\overline{P}$ = Required compressive axial strength [factored compressive force]
= P_u (*LRFD*)
= P_f (*LSD*)

ϕ_c = 0.85 (LRFD)
= 0.80 (LSD)

P_n = Nominal axial strength [resistance] determined in accordance with Section C4

$\overline{M}_x$, $\overline{M}_y$ = Required flexural strengths [factored moments] with respect to centroidal axes of effective section determined for required compressive axial strength [factored axial force] alone.
$\overline{M}_x = M_{ux}$, $\overline{M}_y = M_{uy}$ (LRFD)
$\overline{M}_x = M_{fx}$, $\overline{M}_y = M_{fy}$ (LSD)

ϕ_b = For flexural strength [resistance] (Section C3.1.1), ϕ_b = 0.90 or 0.95 (LRFD) and 0.90 (LSD)
For laterally unbraced flexural members (Section C3.1.2), ϕ_b = 0.90 (LRFD and LSD)
For closed cylindrical tubular members (Section C3.1.3), ϕ_b = 0.95 (LRFD) and 0.90 (LSD)

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

$$\alpha_x = 1 - \frac{\overline{P}}{P_{Ex}} > 0 \qquad (Eq.\ C5.2.2\text{-}4)$$

$$\alpha_y = 1 - \frac{\overline{P}}{P_{Ey}} > 0 \qquad (Eq.\ C5.2.2\text{-}5)$$

where

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

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

where

I_x = Moment of inertia of full unreduced cross-section about x-axis
K_x = Effective length factor for buckling about x-axis
L_x = Unbraced length for bending about x-axis
I_y = Moment of inertia of full unreduced cross-section about y-axis
K_y = Effective length factor for buckling about y-axis
L_y = Unbraced length for bending about y-axis

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

C_{mx}, C_{my} = Coefficients whose values are determined in accordance with (a), (b), or (c) as follows:

(a) For compression members in frames subject to *joint* translation (sidesway)

$C_m = 0.85$

(b) For restrained compression members in frames braced against joint translation and not subject to transverse loading between their supports in the plane of bending

$$C_m = 0.6 - 0.4\ (M_1/M_2) \quad (Eq.\ C5.2.2\text{-}8)$$

where

M_1/M_2 = 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

(c) 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 are permitted to be determined by rational analysis. However, in lieu of such analysis, the following values are permitted to be used:

(1) For members whose ends are restrained, $C_m = 0.85$, and

(2) For members whose ends are unrestrained, $C_m = 1.0$.

D. STRUCTURAL ASSEMBLIES AND SYSTEMS

D1 Built-Up Sections

D1.1 Flexural Members Composed of Two Back-to-Back C-Sections

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

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

where

L = Span of beam

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

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

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

q = *Design load* [factored load] on beam for spacing of connectors (See below for methods of determination.)

The *load*, q, shall be obtained by dividing the concentrated loads or reactions by the length of bearing. For beams designed for a uniformly distributed load, q shall be taken as equal to three times the uniformly distributed load, based on the critical *load combinations* for *ASD*, *LRFD*, and *LSD*. If the length of bearing of a concentrated load or reaction is smaller than the weld spacing, s, the available strength [factored resistance] of the welds or connections closest to the load or reaction shall be calculated as follows:

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

where

P_s = Concentrated load [factored load] or reaction based on critical load combinations for ASD, LRFD, and LSD.

The allowable maximum spacing of connections, s_{max}, shall depend upon the intensity of the load directly at the connection. Therefore, if uniform spacing of connections is used over the whole length of the beam, it shall be determined at the point of maximum local load intensity. In cases where this procedure would result in uneconomically close spacing, either one of the following methods shall be permitted to be adopted:

(a) the connection spacing varies along the beam according to the variation of the load intensity, or

(b) reinforcing cover plates are welded to the flanges at points where concentrated loads occur. The available shear strength [factored resistance] of the connections joining these plates to the flanges is then used for T_s, and g is taken as the depth of the beam.

D1.2 Compression Members Composed of Two Sections in Contact

For compression members composed of two sections in contact, the available axial strength [factored axial resistance] shall be determined in accordance with Section C4.1(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 $(KL/r)_m$ calculated as follows:

$$\left(\frac{KL}{r}\right)_m = \sqrt{\left(\frac{KL}{r}\right)_o^2 + \left(\frac{a}{r_i}\right)^2} \qquad (Eq.\ D1.2\text{-}1)$$

where

$(KL/r)_o$ = Overall slenderness ratio of entire section about 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

See Section C4.1.1 for definition of other symbols.

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

(1) The intermediate fastener or spot weld spacing, a, is 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 are 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) The intermediate fastener(s) or weld(s) at any longitudinal member tie location are capable of transmitting a force in any direction of 2.5 percent of the nominal axial strength [compressive resistance] of the built-up member.

D1.3 Spacing of Connections in Cover Plated Sections

The spacing, s, in the line of *stress*, of welds, rivets, or bolts connecting a cover plate, sheet, or a non-integral stiffener in compression to another element shall not exceed (a), (b), and (c) as follows:

(a) that which is required to transmit the shear between the connected parts on the basis of the *available strength* [factored resistance] per *connection* specified elsewhere herein;

(b) $1.16t\sqrt{E/f_c}$

where

t = *Thickness* of the cover plate or sheet

f_c = Compressive stress at *nominal load* [*specified load*] in the cover plate or sheet

(c) three times the flat width, w, of the narrowest unstiffened compression element tributary to the *connections*, but need not be less than $1.11t\sqrt{E/F_y}$ if $w/t < 0.50\sqrt{E/F_y}$, or $1.33t\sqrt{E/F_y}$ if $w/t \geq 0.50\sqrt{E/F_y}$, unless closer spacing is required by (a) or (b) above.

In the case of intermittent fillet welds parallel to the direction of stress, the spacing shall be taken as the clear distance between welds, plus 1/2 in. (12.7 mm). In all other cases, the spacing shall be taken as the center-to-center distance between connections.

Exception: The requirements of this section do not apply to cover sheets that act only as sheathing material and are not considered load-carrying elements.

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 and Stability 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. See Appendix B for additional requirements. ☜B

D3.1 Symmetrical Beams and Columns

Braces and bracing systems, including *connections*, shall be designed considering strength and stiffness requirements. See Appendix B for additional requirements. ☜B

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* shall apply only when neither flange is connected to deck or sheathing material in such a manner as to effectively restrain lateral deflection of the connected flange. When only the top flange is so connected, see Section D6.3.1. Also, see Appendix B for additional requirements. ☜B

Where both flanges are so connected, no further bracing is required.

D3.2.1 Neither Flange Connected to Sheathing that Contributes to the Strength and Stability of the C- or Z- section

Each intermediate brace at the top and bottom flanges of C- or Z-section members shall be designed with resistance of P_{L1} and P_{L2}, where P_{L1} is the brace force required on the flange in the quadrant with both x and y axes positive, and P_{L2} is the brace force on the other flange. The x-axis shall be designated as the centroidal axis perpendicular to the *web*, and the y-axis shall be designated as the centroidal axis parallel to the web. The x and y coordinates shall be oriented such that one of the flanges is located in the quadrant with both positive x and y axes. See Figure D3.2.1-1 for illustrations of coordinate systems and positive force directions.

(a) For uniform loads

$$P_{L1} = 1.5[W_y K' - (W_x/2) + (M_z/d)] \qquad (Eq.\ D3.2.1\text{-}1)$$

$$P_{L2} = 1.5[W_y K' - (W_x/2) - (M_z/d)] \qquad (Eq.\ D3.2.1\text{-}2)$$

When the uniform load, W, acts through the plane of the web, i.e., W_y = W:

$$P_{L1} = -P_{L2} = 1.5(m/d)W \quad \text{for C-sections} \qquad (Eq.\ D3.2.1\text{-}3)$$

$$P_{L1} = P_{L2} = 1.5\left(\frac{I_{xy}}{2I_x}\right)W \quad \text{for Z-sections} \qquad (Eq.\ D3.2.1\text{-}4)$$

where

W_x, W_y = Components of *design load* [factored load] W parallel to the x- and y-axis, respectively. W_x and W_y are positive if pointing to the positive x- and y-direction, respectively

where

W = Design load [factored load] (applied load determined in accordance with the most critical *load combinations* for *ASD*, *LRFD* or *LSD*, whichever is applicable) within a distance of 0.5a each side of the brace

where

a = Longitudinal distance between centerline of braces

K' = 0 for C-sections

= $I_{xy}/(2I_x)$ for Z-sections (*Eq.* D3.2.1-5)

where

I_{xy} = Product of inertia of full unreduced section

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

M_z = $-W_x e_{sy} + W_y e_{sx}$, torsional moment of W about shear center

where

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

d = Depth of section

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

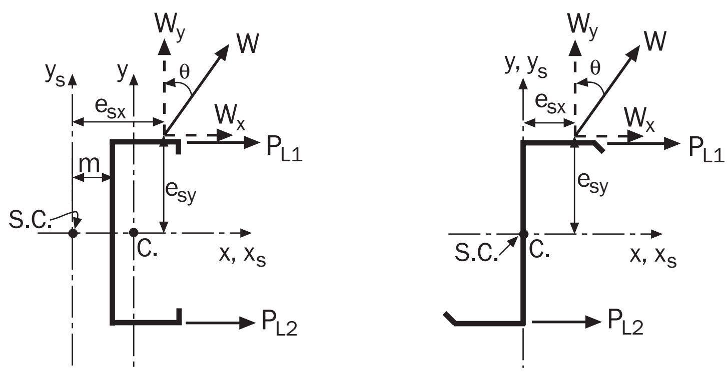


Figure D3.2.1-1 Coordinate Systems and Positive Force Directions

(b) For concentrated loads,

$$P_{L1} = P_y K' - (P_x/2) + (M_z/d) \quad (Eq.\ D3.2.1\text{-}6)$$

$$P_{L2} = P_y K' - (P_x/2) - (M_z/d) \quad (Eq.\ D3.2.1\text{-}7)$$

When a design load [factored load] acts through the plane of the web, i.e., P_y = P:

$$P_{L1} = -P_{L2} = (m/d)P \quad \text{for C-sections} \quad (Eq.\ D3.2.1\text{-}8)$$

$$P_{L1} = P_{L2} = \left(\frac{I_{xy}}{2I_x}\right)P \quad \text{for Z-sections} \quad (Eq.\ D3.2.1\text{-}9)$$

where

P_x, P_y = Components of design load [factored load] P parallel to the x- and y-axis, respectively. P_x and P_y are positive if pointing to the positive x- and y-direction, respectively.

M_z = $-P_x e_{sy} + P_y e_{sx}$, torsional moment of P about shear center

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

where

l = Distance from concentrated load to the brace

See Section D3.2.1(a) for definitions of other variables.

The bracing force, P_{L1} or P_{L2}, is positive where restraint is required to prevent the movement of the corresponding flange in the negative x-direction.

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

When all loads and reactions on a beam are transmitted through members that frame into the section in such a manner as to effectively restrain the section against torsional rotation and lateral displacement, no additional braces shall be required except those required for strength [resistance] in accordance with Section C3.1.2.1.

D3.3 Bracing of Axially Loaded Compression Members

The required brace strength [resistance] to restrain lateral translation at a brace point for an individual compression member shall be calculated as follows:

$$P_{br,1} = 0.01P_n \qquad (Eq.\ D3.3\text{-}1)$$

The required brace stiffness to restrain lateral translation at a brace point for an individual compression member shall be calculated as follows:

$$\beta_{br,1} = \frac{2[4-(2/n)]P_n}{L_b} \qquad (Eq.\ D3.3\text{-}2)$$

where

$P_{br,1}$ = Required nominal brace strength [resistance] for a single compression member

P_n = Nominal axial compression strength [resistance] of a single compression member

$\beta_{br,1}$ = Required brace stiffness for a single compression member

n = Number of equally spaced intermediate brace locations

L_b = Distance between braces on one compression member

D4 Cold-Formed Steel Light-Frame Construction

The design and installation of *structural members* and non-structural members utilized in cold-formed steel repetitive framing applications where the specified minimum base steel *thickness* is between 0.0179 in. (0.455 mm) and 0.1180 in. (2.997 mm) shall be in accordance with the AISI S200 and the following, as applicable:

(a) Headers, including box and back-to-back headers, and double and single L-headers, shall be designed in accordance with AISI S212 or solely in accordance with this *Specification*.

(b) Trusses shall be designed in accordance with AISI S214.

(c) Wall studs shall be designed in accordance with AISI S211, or solely in accordance with this *Specification* either on the basis of an all-steel system in accordance with Section D4.1 or on the basis of sheathing braced design in accordance with an appropriate theory, tests, or *rational engineering analysis*. 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.

(d) Framing for floor and roof systems in buildings shall be designed in accordance with AISI

S210 or solely in accordance with this *Specification.*
See Appendix A for additional country requirements. A

D4.1 All-Steel Design of Wall Stud Assemblies

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 or non-circular web perforations, the effective section properties shall be determined in accordance with Section B2.2.

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. The *safety factors* and *resistance factors* for diaphragms given in Table D5 shall apply to both methods. If the nominal shear strength [resistance] is only established by test without defining all *limit state* thresholds, the *safety factors* and *resistance factors* shall be limited by the values given in Table D5 for *connection* types and connection-related failure modes. The more severe factored limit state shall control the design. Where fastener combinations are used within a diaphragm system, the more severe factor shall be used.

Ω_d = As specified in Table D5 (*ASD*)

ϕ_d = As specified in Table D5 (*LRFD* and *LSD*)

TABLE D5
Safety Factors and Resistance Factors for Diaphragms

Load Type or Combinations Including	Connection Type	Limit State					
		Connection Related			Panel Buckling*		
		Ω_d (ASD)	ϕ_d (LRFD)	ϕ_d (LSD)	Ω_d (ASD)	ϕ_d (LRFD)	ϕ_d (LSD)
Earthquake	Welds	3.00	0.55	0.50	2.00	0.80	0.75
	Screws	2.50	0.65	0.60			
Wind	Welds	2.35	0.70	0.65			
	Screws						
All Others	Welds	2.65	0.60	0.55			
	Screws	2.50	0.65	0.60			

Note:
*Panel *buckling* is out-of-plane buckling and not *local buckling* at fasteners.

For mechanical fasteners other than screws:

(a) Ω_d shall not be less than the Table D5 values for screws, and

(b) ϕ_d shall not be greater than the Table D5 values for screws.

In addition, the value of Ω_d and ϕ_d using mechanical fasteners other than screws shall be limited by the Ω and ϕ values established through calibration of the individual fastener shear strength [resistance], unless sufficient data exist to establish a diaphragm system effect in accordance with Section F1.1. Fastener shear strength [resistance] calibration shall include the diaphragm material type. Calibration of individual fastener shear strengths [resistance] shall be in accordance with Section F1.1. The test assembly shall be such that the tested failure mode is representative of the design. The impact of the thickness of the supporting material on the failure mode shall be considered.

D6 Metal Roof and Wall Systems

The provisions of Section D6.1 through D6.3 shall apply to metal roof and wall systems that include cold-formed steel *purlins*, *girts*, through-fastened wall/roof and wall panels, or standing seam roof panels, as applicable.

D6.1 Purlins, Girts and Other Members

D6.1.1 Flexural Members Having One Flange Through-Fastened to Deck or Sheathing

This section shall not apply to a continuous beam for the region between inflection points adjacent to a support or to a cantilever beam.

The nominal flexural strength [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 in accordance with Eq. D6.1.1-1. The *safety factor* and *resistance factors* given in this section shall be used to determine the allowable flexural strength or design flexural strength [factored moment resistance] in accordance with the applicable design method in Section A4, A5, or A6.

$$M_n = RS_eF_y \qquad (Eq.\ D6.1.1\text{-}1)$$

Ω_b = 1.67 (*ASD*)

ϕ_b = 0.90 (*LRFD*)

= 0.90 (*LSD*)

where R is obtained from Table D6.1.1-1 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 = Values as 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 $\le$ 11.5 in. (292 mm),
(2) Member flanges with edge stiffeners,
(3) 60 $\le$ depth/*thickness* $\le$ 170,
(4) 2.8 $\le$ depth/flange width $\le$ 4.5,
(5) 16 $\le$ *flat width*/thickness of flange $\le$ 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) is not less than 1.5d,
(7) Member span length is not greater than 33 feet (10 m),
(8) Both flanges are prevented from moving laterally at the supports,
(9) Roof or wall panels are steel sheets with 50 ksi (340 MPa or 3520 kg/cm^2) minimum *yield stress*, and a minimum of 0.018 in. (0.46 mm) base metal thickness, having a minimum rib depth of 1-1/8 in. (29 mm), spaced 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,
(10) Insulation is glass fiber blanket 0 to 6 in. (152 mm) thick compressed between the member and panel in a manner consistent with the fastener being used,

(11) Fastener type is, at minimum, No. 12 self-drilling or self-tapping sheet metal screws or 3/16 in. (4.76 mm) rivets, having washers 1/2 in. (12.7 mm) diameter,
(12) Fasteners is not standoff type screws,
(13) Fasteners are 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, and
(14) The design yield stress of the member does exceed 60 ksi (410 MPa or 4220 kg/cm^2).

If variables fall outside any of the above stated limits, the user shall perform full-scale tests in accordance with Section F1 of this *Specification* or apply a *rational engineering analysis* procedure. For continuous purlin systems in which adjacent bay span lengths vary by more than 20 percent, the R values for the adjacent bays shall be taken from Table D6.1.1-1. The user shall be permitted to perform tests in accordance with Section F1 as an alternate to the procedure described in this section.

TABLE D6.1.1-1
Simple Span C- or Z-Section R Values

Depth Range, in. (mm)	Profile	R
d ≤ 6.5 (165)	C or Z	0.70
6.5 (165) < d ≤ 8.5 (216)	C or Z	0.65
8.5 (216) < d ≤ 11.5 (292)	Z	0.50
8.5 (216) < d ≤ 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 D6.1.1-1 by the following correction factor, r:

$r = 1.00 - 0.01\, t_i$ when t_i is in inches (*Eq*. D6.1.1-2)

$r = 1.00 - 0.0004\, t_i$ when t_i is in millimeters (*Eq*. D6.1.1-3)

where

t_i = Thickness of uncompressed glass fiber blanket insulation

D6.1.2 Flexural Members Having One Flange Fastened to a Standing Seam Roof System

See Section D6.1.2 of Appendix A or B for the provisions of this section. ☜A,B

D6.1.3 Compression Members Having One Flange Through-Fastened to Deck or Sheathing

These provisions shall apply to C- or Z-sections concentrically loaded along their longitudinal axis, with only one flange attached to deck or sheathing with through fasteners.

The nominal axial strength [resistance] of simple span or continuous C- or Z-sections shall be calculated in accordance with (a) and (b).

(a) The weak axis *nominal strength* [resistance] shall be calculated in accordance with Eq. D6.1.3-1. The *safety factor* and *resistance factors* given in this section shall be used to determine the allowable axial strength or design axial strength [factored compressive resistance] in accordance with the applicable design method in Section A4, A5, or A6.

$P_n = C_1C_2C_3AE/29500$ (*Eq*. D6.1.3-1)

Ω = 1.80 (*ASD*)

ϕ = 0.85 (*LRFD*)

= 0.80 (*LSD*)

where

C_1 = (0.79x + 0.54) (*Eq*. D6.1.3-2)

C_2 = (1.17αt + 0.93) (*Eq*. D6.1.3-3)

C_3 = α(2.5b - 1.63d) + 22.8 (*Eq*. D6.1.3-4)

where

x = For Z-sections, the fastener distance from the outside *web* edge divided by the flange width, as shown in Figure D6.1.3

= For C-sections, the flange width minus the fastener distance from the outside web edge divided by the flange width, as shown in Figure D6.1.3.

α = Coefficient for conversion of units

= 1 when t, b, and d are in inches

= 0.0394 when t, b, and d are in mm

= 0.394 when t, b, and d are in cm

t = C- or Z-section *thickness*

b = C- or Z-section flange width

d = C- or Z-section depth

A = *Full unreduced cross-sectional area* of C- or Z-section

E = Modulus of elasticity of steel

= 29,500 ksi for U.S. customary units

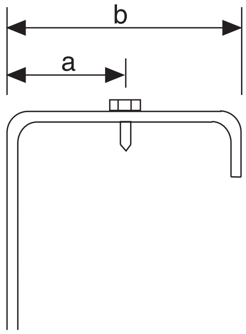
= 203,000 MPa for SI units

= 2,070,000 kg/cm^2 for MKS units

Eq. D6.1.3-1 shall be limited to roof and wall systems meeting the following conditions:

(1) t ≤ 0.125 in. (3.22 mm),
(2) 6 in. (152mm) ≤ d ≤ 12 in. (305 mm),
(3) Flanges are edge stiffened compression elements,
(4) 70 ≤ d/t ≤ 170,
(5) 2.8 ≤ d/b ≤ 5,
(6) 16 ≤ 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. (10,300 N/m/m or 0.105 kg/cm/cm) (fastener at mid-flange width for stiffness determination) determined in accordance with AISI S901,
(9) C- and Z-sections having a minimum *yield stress* of 33 ksi (230 MPa or 2320 kg/cm^2), and
(10) Span length not exceeding 33 feet (10 m).

(b) The strong axis *available strength* [factored resistance] shall be determined in accordance with Sections C4.1 and C4.1.1.



For Z-section, $x = \frac{a}{b}$ (*Eq*. D6.1.3-5)

For C-section, $x = \frac{b-a}{b}$ (*Eq*. D6.1.3-6)

Figure D6.1.3 Definition of x

D6.1.4 Compression of Z-Section Members Having One Flange Fastened to a Standing Seam Roof

The provisions of this section shall apply only to the United States and Mexico. See Section D6.1.4 of Appendix A.

D6.2 Standing Seam Roof Panel Systems

D6.2.1 Strength [Resistance] of Standing Seam Roof Panel Systems

Under gravity loading, the *nominal strength* [*resistance*] of standing seam roof panels shall be determined in accordance with Chapters B and C of this *Specification* or shall be tested in accordance with AISI S906. Under uplift loading, the nominal strength [resistance] of standing seam roof panel systems shall be determined in accordance with AISI S906. Tests shall be performed in accordance with AISI S906 with the following exceptions:

(1) The Uplift Pressure Test Procedure for Class 1 Panel Roofs in FM 4471 shall be permitted.

(2) Existing tests conducted in accordance with CEGS 07416 uplift test procedure prior to the adoption of these provisions shall be permitted.

The open-open end configuration, although not prescribed by the ASTM E1592 test procedure, shall be permitted provided the tested end conditions represent the installed condition, and the test follows the requirements given in AISI S906. All test results shall be evaluated in accordance with this section.

For *load combinations* that include wind uplift, additional provisions are provided in Section D6.2.1a of Appendix A.

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

β_o = Target reliability index
= 2.0 for USA and Mexico and 2.5 for Canada for panel flexural limits
= 2.5 for USA and Mexico and 3.0 for Canada for anchor limits

F_m = Mean value of the fabrication factor
= 1.0

M_m = Mean value of the material factor
= 1.1

V_M = Coefficient of variation of the material factor
= 0.08 for anchor failure mode
= 0.10 for other failure modes
V_F = Coefficient of variation of the fabrication factor
= 0.05
V_Q = Coefficient of variation of the load effect
= 0.21
V_P = Actual calculated coefficient of variation of the test results, without limit
n = Number of anchors in the test assembly with same tributary area (for anchor failure) or number of panels with identical spans and loading to the failed span (for non-anchor failures)

The safety factor, Ω, shall not be less than 1.67, and the resistance factor, ϕ, shall not be greater than 0.9 (*LRFD* and *LSD*).

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

D6.3 Roof System Bracing and Anchorage

D6.3.1 Anchorage of Bracing for Purlin Roof Systems Under Gravity Load with Top Flange Connected to Metal Sheathing

Anchorage, in the form of a device capable of transferring force from the roof *diaphragm* to a support, shall be provided for roof systems with C-sections or Z-sections, designed in accordance with Sections C3.1 and D6.1, having through-fastened or standing seam sheathing attached to the top flanges. Each anchorage device shall be designed to resist the force, P_L, determined by Eq. D6.3.1-1 and shall satisfy the minimum stiffness requirement of Eq. D6.3.1-7. In addition, *purlins* shall be restrained laterally by the sheathing so that the maximum top flange lateral displacements between lines of lateral anchorage at *nominal loads* [*specified loads*] do not exceed the span length divided by 360.

Anchorage devices shall be located in each purlin bay and shall connect to the purlin at or near the purlin top flange. If anchorage devices are not directly connected to all purlin lines of each purlin bay, provision shall be made to transmit the forces from other purlin lines to the anchorage devices. It shall be demonstrated that the required force, P_L, can be transferred to the anchorage device through the roof sheathing and its fastening system. The lateral stiffness of the anchorage device shall be determined by analysis or testing. This analysis or testing shall account for the flexibility of the purlin *web* above the attachment of the anchorage device *connection*.

$$P_{L_j} = \sum_{i=1}^{N_p}\left(P_i \frac{K_{\text{eff}_{i,j}}}{K_{\text{total}_i}}\right) \qquad (Eq.\ D6.3.1\text{-}1)$$

where

P_{L_j} = Lateral force to be resisted by the j^{th} anchorage device (positive when restraint is required to prevent purlins from translating in the upward roof slope direction)
N_p = Number of purlin lines on roof slope

i = Index for each purlin line (i=1, 2, …, N_p)

j = Index for each anchorage device (j=1,2, …, N_a)

where

N_a = Number of anchorage devices along a line of anchorage

P_i = Lateral force introduced into the system at the i^{th} purlin

$$= (C1)W_{p_i}\left\{\left[\left(\frac{C2}{1000}\right)\frac{I_{xy}L}{I_x d}+(C3)\frac{(m+0.25b)t}{d^2}\right]\alpha\cos\theta-(C4)\sin\theta\right\} \quad (Eq.\ D6.3.1\text{-}2)$$

where

C1, C2, C3, and C4 = Coefficients tabulated in Tables D6.3.1-1 to D6.3.1-3

W_{p_i} = Total required vertical *load* supported by the i^{th} purlin in a single bay

$$= w_i L \quad (Eq.\ D6.3.1\text{-}3)$$

where

w_i = Required distributed gravity load supported by the i^{th} purlin per unit length (determined from the *critical load combination* for *ASD*, *LRFD* or *LSD*)

I_{xy} = Product of inertia of full unreduced section about centroidal axes parallel and perpendicular to the purlin web (I_{xy} = 0 for C-sections)

L = Purlin span length

m = Distance from shear center to mid-plane of web (m = 0 for Z-sections)

b = Top flange width of purlin

t = Purlin *thickness*

I_x = Moment of inertia of full unreduced section about centroidal axis perpendicular to the purlin web

d = Depth of purlin

α = +1 for top flange facing in the up-slope direction
-1 for top flange facing in the down-slope direction

θ = Angle between vertical and plane of purlin web

$K_{eff_{i,j}}$ = Effective lateral stiffness of the j^{th} anchorage device with respect to the i^{th} purlin

$$= \left[\frac{1}{K_a}+\frac{d_{p_{i,j}}}{(C6)LA_pE}\right]^{-1} \quad (Eq.\ D6.3.1\text{-}4)$$

where

$d_{p_{i,j}}$ = Distance along roof slope between the i^{th} purlin line and the j^{th} anchorage device

K_a = Lateral stiffness of the anchorage device

C6 = Coefficient tabulated in Tables D6.3.1-1 to D6.3.1-3

A_p = *Gross cross-sectional area* of roof panel per unit width

E = Modulus of elasticity of steel

K_{total_i} = Effective lateral stiffness of all elements resisting force P_i

$$= \sum_{j=1}^{N_a} \left(K_{eff\,i,j} \right) + K_{sys} \qquad (Eq.\ D6.3.1\text{-}5)$$

where

K_{sys} = Lateral stiffness of the roof system, neglecting anchorage devices

$$= \left(\frac{C5}{1000} \right) (N_p) \frac{ELt^2}{d^2} \qquad (Eq.\ D6.3.1\text{-}6)$$

For multi-span systems, force P_i, calculated in accordance with Eq. D6.3.1-2 and coefficients C1 to C4 from Tables D6.3.1-1 to D6.3.1-3 for the "Exterior Frame Line", "End Bay", or "End Bay Exterior Anchor" cases, shall not be taken as less than 80 percent of the force determined using the coefficients C2 to C4 for the corresponding "All Other Locations" case.

For systems with multiple spans and anchorage devices at supports (support restraints), where the two adjacent bays have different section properties or span lengths, the following procedures shall be used. The values for P_i in Eq. D6.3.1-1 and Eq. D6.3.1-8 shall be taken as the average of the values found from Eq. D6.3.1-2 evaluated separately for each of the two bays. The values of K_{sys} and $K_{eff_{i,j}}$ in Eq. D6.3.1-1 and Eq. D6.3.1-5 shall be calculated using Eq. D6.3.1-4 and Eq. D6.3.1-6, with L, t, and d taken as the average values of the two bays.

For systems with multiple spans and anchorage devices at either 1/3 points or mid-points, where the adjacent bays have different section properties or span lengths than the bay under consideration, the following procedures shall be used to account for the influence of the adjacent bays. The values for P_i in Eq. D6.3.1-1 and Eq. D6.3.1-8 shall be taken as the average of the values found from Eq. D6.3.1-2 evaluated separately for each of the three bays. The value of K_{sys} in Eq. D6.3.1-5 shall be calculated using Eq. D6.3.1-6, with L, t, and d taken as the average of the values from the three bays. The values of $K_{eff_{i,j}}$ shall be calculated using Eq. D6.3.1-4, with L taken as the span length of the bay under consideration. At an end bay, when computing the average values for P_i or averaging the properties for computing K_{sys}, the averages shall be found by adding the value from the first interior bay and two times the value from the end bay and then dividing the sum by three.

The total effective stiffness at each purlin shall satisfy the following equation:

$$K_{total\,i} \geq K_{req} \qquad (Eq.\ D6.3.1\text{-}7)$$

where

$$K_{req} = \Omega \frac{20 \left| \sum_{i=1}^{N_p} P_i \right|}{d} \text{ (ASD)} \qquad (Eq.\ D6.3.1\text{-}8a)$$

$$K_{req} = \frac{1}{\phi} \frac{20 \left| \sum_{i=1}^{N_p} P_i \right|}{d} \text{ (LRFD, LSD)} \qquad (Eq.\ D6.3.1\text{-}8b)$$

Ω = 2.00 (ASD)

ϕ = 0.75 (LRFD)

= 0.70 (LSD)

In lieu of the Eqs. D6.3.1-1 through D6.3.1-6, lateral restraint forces shall be permitted to be determined from alternate analysis. Alternate analysis shall include the first or second order effect and account for the effects of roof slope, torsion resulting from applied loads eccentric to shear center, torsion resulting from the lateral resistance provided by the sheathing, and load applied oblique to the principal axes. Alternate analysis shall also include the effects of the lateral and rotational restraint provided by sheathing attached to the top flange. Stiffness of the anchorage device shall be considered and shall account for flexibility of the purlin web above the attachment of the anchorage device connection.

When lateral restraint forces are determined from rational analysis, the maximum top flange lateral displacement of the purlin between lines of lateral bracing at nominal loads shall not exceed the span length divided by 360. The lateral displacement of the purlin top flange at the line of restraint, Δ_{tf}, shall be calculated at factored load levels for LRFD or LSD and nominal load levels for ASD and shall be limited to:

$$\Delta_{tf} \le \frac{1}{\Omega}\frac{d}{20} \quad \text{(ASD)} \qquad (\textit{Eq. } \text{D6.3.1-9a})$$

$$\Delta_{tf} \le \phi\frac{d}{20} \quad \text{(LRFD, LSD)} \qquad (\textit{Eq. } \text{D6.3.1-9b})$$

Table D6.3.1-1
Coefficients for Support Restraints

			C1	C2	C3	C4	C5	C6
Simple Span	Through Fastened (TF)		0.5	8.2	33	0.99	0.43	0.17
	Standing Seam (SS)		0.5	8.3	28	0.61	0.29	0.051
Multiple Spans	TF	Exterior Frame Line	0.5	14	6.9	0.94	0.073	0.085
		First Interior Frame Line	1.0	4.2	18	0.99	2.5	0.43
		All Other Locations	1.0	6.8	23	0.99	1.8	0.36
	SS	Exterior Frame Line	0.5	1.3	11	0.35	2.4	0.25
		First Interior Frame Line	1.0	1.7	69	0.77	1.6	0.13
		All Other Locations	1.0	4.3	55	0.71	1.4	0.17

Table D6.3.1-2
Coefficients for Mid-Point Restraints

			C1	C2	C3	C4	C5	C6
Simple Span	Through Fastened (TF)		1.0	7.6	44	0.96	0.75	0.42
	Standing Seam (SS)		1.0	7.5	15	0.62	0.35	0.18
Multiple Spans	TF	End Bay	1.0	8.3	47	0.95	3.1	0.33
		First Interior Bay	1.0	3.6	53	0.92	3.9	0.36
		All Other Locations	1.0	5.4	46	0.93	3.1	0.31
	SS	End Bay	1.0	7.9	19	0.54	2.0	0.080
		First Interior Bay	1.0	2.5	41	0.47	2.6	0.13
		All Other Locations	1.0	4.1	31	0.46	2.7	0.15

Table D6.3.1-3
Coefficients for One-Third Point Restraints

			C1	C2	C3	C4	C5	C6
Simple Span	Through Fastened (TF)		0.5	7.8	42	0.98	0.39	0.40
Simple Span	Standing Seam (SS)		0.5	7.3	21	0.73	0.19	0.18
Multiple Spans	TF	End Bay Exterior Anchor	0.5	15	17	0.98	0.72	0.043
Multiple Spans	TF	End Bay Int. Anchor and 1st Int. Bay Ext. Anchor	0.5	2.4	50	0.96	0.82	0.20
Multiple Spans	TF	All Other Locations	0.5	6.1	41	0.96	0.69	0.12
Multiple Spans	SS	End Bay Exterior Anchor	0.5	13	13	0.72	0.59	0.035
Multiple Spans	SS	End Bay Int. Anchor and 1st Int. Bay Ext. Anchor	0.5	0.84	56	0.64	0.20	0.14
Multiple Spans	SS	All Other Locations	0.5	3.8	45	0.65	0.10	0.014

D6.3.2 Alternate Lateral and Stability Bracing for Purlin Roof Systems

Torsional bracing that prevents twist about the longitudinal axis of a member in combination with lateral restraints that resist lateral displacement of the top flange at the frame line shall be permitted in lieu of the requirements of Section D6.3.1. A torsional brace shall prevent torsional rotation of the cross-section at a discrete location along the span of the member. *Connection* of braces shall be made at or near both flanges of ordinary open sections, including C- and Z-sections. The effectiveness of torsional braces in preventing torsional rotation of the cross-section and the *required strength* of lateral restraints at the frame line shall be determined by *rational engineering analysis* or testing. The lateral displacement of the top flange of the C- or Z-section at the frame line shall be limited to $d/(20\Omega)$ for *ASD* calculated at *nominal load* [specified load] levels or $\phi d/20$ for *LRFD* and *LSD* calculated at *factored load* levels, where d is the depth of the C- or Z-section member, Ω is the *safety factor* for ASD, and ϕ is the *resistance factor* for LRFD and LSD. Lateral displacement between frame lines, calculated at nominal load levels, shall be limited to L/180, where L is the span length of the member. For pairs of adjacent *purlins* that provide bracing against twist to each other, external anchorage of torsional brace forces shall not be required.

where

Ω = 2.0 (ASD)

ϕ = 0.75 (LRFD)

= 0.70 (LSD)

E. CONNECTIONS AND JOINTS

E1 General Provisions

Connections shall be designed to transmit the *required strength* [factored loads] acting on the connected members with consideration of eccentricity where applicable.

E2 Welded Connections

The following design criteria shall apply to welded *connections* used for *cold-formed steel structural members* in which the *thickness* of the thinnest connected part is 3/16 in. (4.76 mm) or less. For the design of welded connections in which the thickness of the thinnest connected part is greater than 3/16 in. (4.76 mm), refer to the specifications or standards stipulated in the corresponding Section E2a of Appendix A or B.

Welds shall follow the requirements of the weld standards also stipulated in Section E2a of Appendix A or B. For *diaphragm* applications, Section D5 shall apply. ☜A,B

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 in accordance with (a) or (b), as applicable. The corresponding *safety factor* and *resistance factors* shall be used to determine the *allowable strength* or *design strength* [*factored resistance*] in accordance with the applicable design method in Section A4, A5, or A6.

(a) For tension or compression normal to the effective area or parallel to the axis of the weld, the nominal strength [resistance], P_n, shall be calculated in accordance with Eq. E2.1-1:

$$P_n = Lt_eF_y \quad (Eq.\ E2.1\text{-}1)$$

Ω = 1.70 (*ASD*)
ϕ = 0.90 (*LRFD*)
= 0.80 (*LSD*)

(b) For shear on the effective area, the nominal strength [resistance], P_n, shall be the smaller value calculated in accordance with Eqs. E2.1-2 and E2.1-3:

$$P_n = Lt_e\,0.6F_{xx} \quad (Eq.\ E2.1\text{-}2)$$

Ω = 1.90 (ASD)
ϕ = 0.80 (LRFD)
= 0.70 (LSD)

$$P_n = Lt_eF_y/\sqrt{3} \quad (Eq.\ E2.1\text{-}3)$$

Ω = 1.70 (ASD)
ϕ = 0.90 (LRFD)
= 0.80 (LSD)

where
P_n = Nominal strength [resistance] of groove weld
L = Length of weld
t_e = Effective throat dimension of groove weld
F_y = *Yield stress* of lowest strength base steel

F_{xx} = *Tensile strength* of electrode classification

E2.2 Arc Spot Welds

Arc spot welds, where permitted by this *Specification*, shall be for welding sheet steel to thicker supporting members or sheet-to-sheet in the flat position. Arc spot welds (puddle welds) shall not be made on steel where the thinnest connected part exceeds 0.15 in. (3.81 mm) in *thickness*, nor through a combination of steel sheets having a total thickness over 0.15 in. (3.81 mm).

Weld washers, as shown in Figures E2.2-1 and E2.2-2, shall be used where the thickness of the sheet is less than 0.028 in. (0.711 mm). Weld washers shall have a thickness between 0.05 (1.27 mm) and 0.08 in. (2.03 mm) with a minimum prepunched hole of 3/8 in. (9.53 mm) diameter. Sheet-to-sheet welds shall not require weld washers.

Arc spot welds shall be specified by minimum effective diameter of fused area, d_e. The minimum allowable effective diameter shall be 3/8 in. (9.5 mm). ☜B

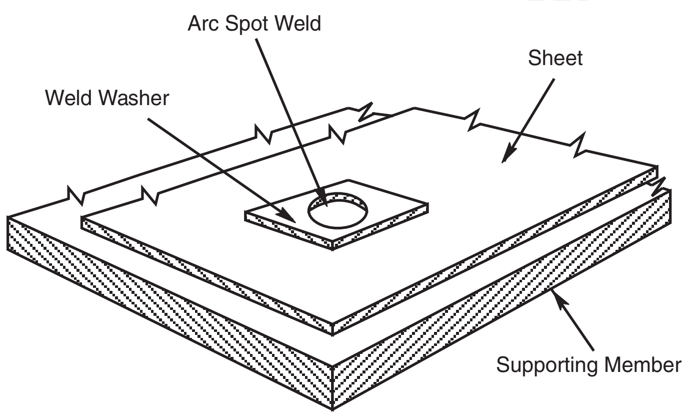


Figure E2.2-1 Typical Weld Washer

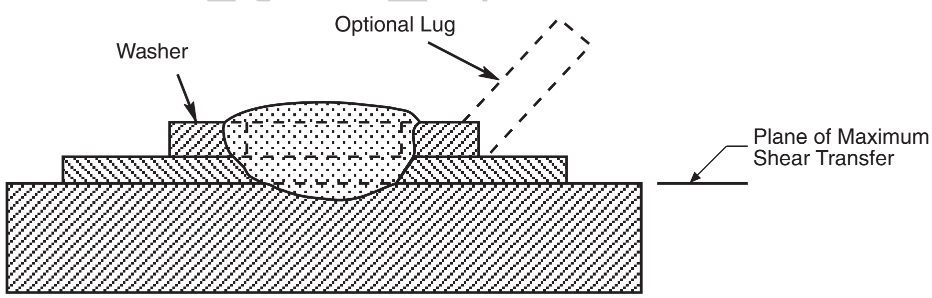


Figure E2.2-2 Arc Spot Weld Using Washer

E2.2.1 Shear

E2.2.1.1 Minimum Edge Distance

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} determined in accordance with Eq. E2.2.1.1-1 or Eq. E2.2.1.1-2, as applicable. See Figures E2.2.1.1-1 and E2.2.1.1-2 for edge distance of arc welds. The corresponding *safety factors* and *resistance factors* shall be used to determine the *allowable strength* or *design strength* [*factored resistance*] in accordance with the applicable design method in Section A4, A5, or A6.

$$e_{min} = \frac{P\Omega}{F_u t} \quad \text{for } ASD \qquad (Eq.\ E2.2.1.1\text{-}1)$$

$$e_{min} = \frac{\overline{P}}{\phi F_u t} \quad \text{for } LRFD \text{ and } LSD \qquad (Eq.\ E2.2.1.1\text{-}2)$$

When $F_u/F_{sy} \geq 1.08$

Ω = 2.20 (ASD)
ϕ = 0.70 (LRFD)
= 0.60 (LSD)

When $F_u/F_{sy} < 1.08$

Ω = 2.55 (ASD)
ϕ = 0.60 (LRFD)
= 0.50 (LSD)

where

P = Required shear strength (nominal force) transmitted by weld (ASD)
F_u = *Tensile strength* as determined in accordance with A2.1, A2.2, or A2.3.2
t = Total combined base steel *thickness* (exclusive of coatings) of sheet(s) involved in shear transfer above plane of maximum shear transfer
$\overline{P}$ = Required shear strength [factored shear load] transmitted by weld
= P_u (LRFD)
= P_f (LSD)
F_{sy} = *Yield stress* as determined in accordance with Section A2.1, A2.2, or A2.3.2

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.

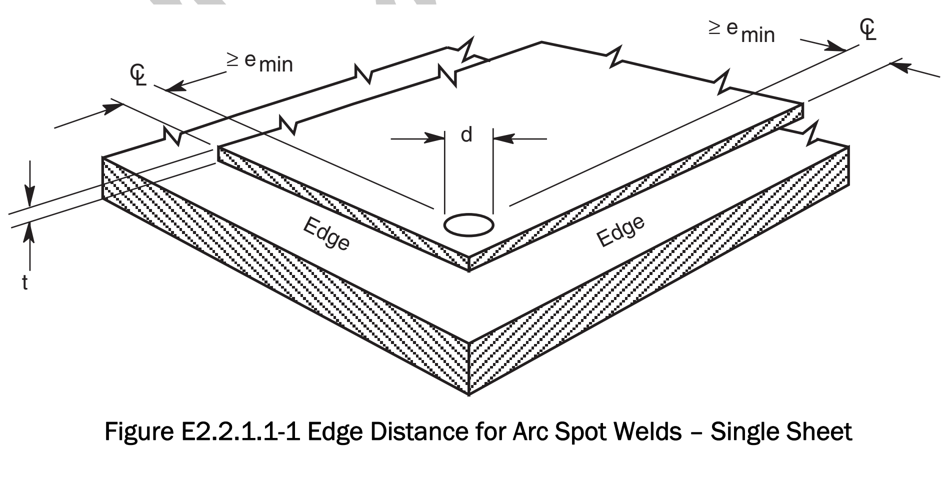


Figure E2.2.1.1-1 Edge Distance for Arc Spot Welds – Single Sheet

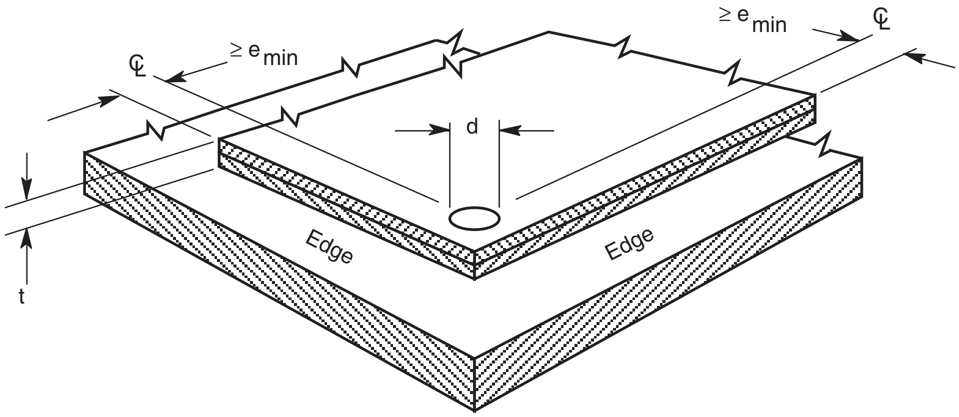


Figure E2.2.1.1-2 Edge Distance for Arc Spot Welds – Double Sheet

E2.2.1.2 Shear Strength [Resistance] for Sheet(s) Welded to a Thicker Supporting Member

The nominal shear strength [resistance], P_n, of each arc spot weld between the sheet or sheets and a thicker supporting member shall be determined by using the smaller of either (a) or (b). The corresponding *safety factor* and *resistance factors* shall be used to determine the *allowable strength* or *design strength* [*factored resistance*] in accordance with the applicable design method in Section A4, A5, or A6.

(a) $P_n = \dfrac{\pi d_e^2}{4} 0.75 F_{xx}$ (*Eq*. E2.2.1.2-1)

Ω = 2.55 (*ASD*)
ϕ = 0.60 (*LRFD*)
= 0.50 (*LSD*)

(b) For $(d_a/t) \le 0.815\sqrt{E/F_u}$

$P_n = 2.20\, t\, d_a\, F_u$ (*Eq*. E2.2.1.2-2)

Ω = 2.20 (ASD)
ϕ = 0.70 (LRFD)
= 0.60 (LSD)

For $0.815\sqrt{E/F_u} < (d_a/t) < 1.397\sqrt{E/F_u}$

$$P_n = 0.280\left[1 + 5.59\frac{\sqrt{E/F_u}}{d_a/t}\right] t d_a F_u \qquad (Eq.\ E2.2.1.2\text{-}3)$$

Ω = 2.80 (ASD)
ϕ = 0.55 (LRFD)
= 0.45 (LSD)

For $(d_a/t) \ge 1.397\sqrt{E/F_u}$

$P_n = 1.40\, t\, d_a\, F_u$ (*Eq*. E2.2.1.2-4)

Ω = 3.05 (ASD)
ϕ = 0.50 (LRFD)
= 0.40 (LSD)

where

P_n = Nominal shear strength [resistance] of arc spot weld

d_e = Effective diameter of fused area at plane of maximum shear transfer

$= 0.7d - 1.5t \le 0.55d$ (*Eq*. E2.2.1.2-5)

where

d = Visible diameter of outer surface of arc spot weld

t = Total combined base steel *thickness* (exclusive of coatings) of sheets involved in shear transfer above plane of maximum shear transfer

F_{xx} = *Tensile strength* of electrode classification

d_a = Average diameter of arc spot weld at mid-thickness of t where $d_a = (d - t)$ for single sheet or multiple sheets not more than four lapped sheets over a supporting member. See Figures E2.2.1.2-1 and E2.2.1.2-2 for diameter definitions.

E = Modulus of elasticity of steel

F_u = Tensile strength as determined in accordance with Section A2.1, A2.2, or A2.3.2

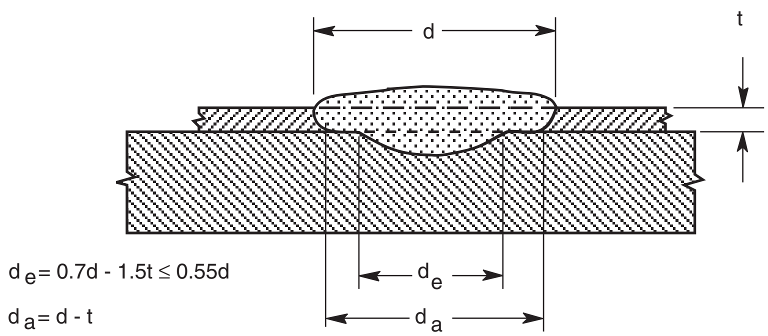


Figure E2.2.1.2-1 Arc Spot Weld – Single Thickness of Sheet

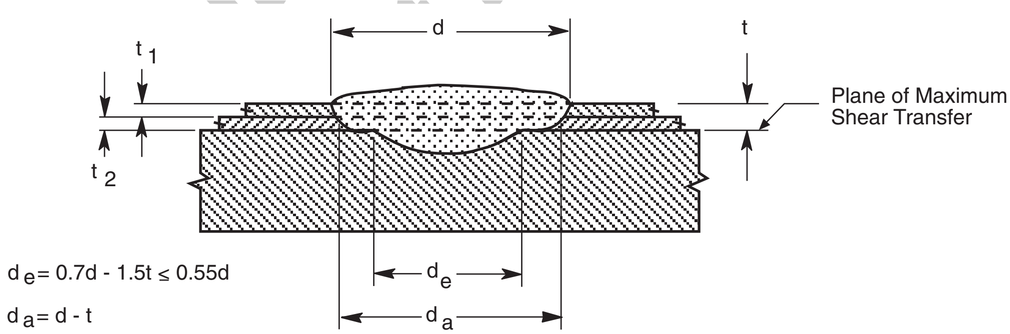


Figure E2.2.1.2-2 Arc Spot Weld – Double Thickness of Sheet

E2.2.1.3 Shear Strength [Resistance] for Sheet-to-Sheet Connections

The nominal shear strength [resistance] for each weld between two sheets of equal *thickness* shall be determined in accordance with Eq. E2.2.1.3-1. The *safety factor* and *resistance factors* in this section shall be used to determine the *allowable strength* or *design strength* [*factored resistance*] in accordance with the applicable design method in Section A4, A5, or A6.

$P_n = 1.65 t d_a F_u$ (*Eq*. E2.2.1.3-1)

Ω = 2.20 (*ASD*)

ϕ = 0.70 (*LRFD*)

= 0.60 (*LSD*)

where

P_n = Nominal shear strength [resistance] of sheet-to-sheet *connection*

t = Total combined base steel *thickness* (exclusive of coatings) of sheets involved in shear transfer above plane of maximum shear transfer

d_a = Average diameter of arc spot weld at mid-thickness of t. See Figure E2.2.1.3-1 for diameter definitions.

= (d - t)

where

d = Visible diameter of the outer surface of arc spot weld

d_e = Effective diameter of fused area at plane of maximum shear transfer

= $0.7d - 1.5t \le 0.55d$ (*Eq*. E2.2.1.3-2)

F_u = *Tensile strength* of sheet as determined in accordance with Section A2.1 or A2.2

In addition, the following limits shall apply:

(1) $F_u \le$ 59 ksi (407 MPa or 4150 kg/cm^2),

(2) $F_{xx} > F_u$, and

(3) 0.028 in. (0.71 mm) $\le t \le$ 0.0635 in. (1.61 mm).

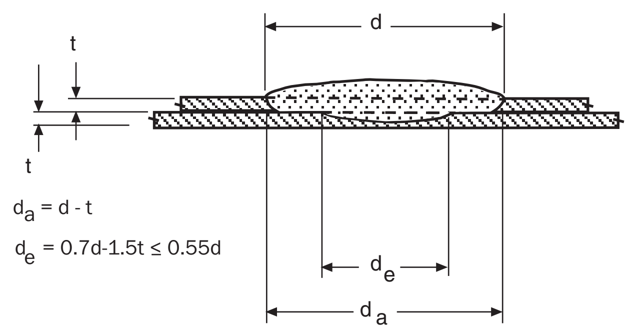


Figure E2.2.1.3-1 Arc Spot Weld – Sheet-to-Sheet

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 Eq. E2.2.2-1 or Eq. E2.2.2-2 as follows. The *safety factor* and *resistance factors* shall be used to determine the *allowable strength* or *design strength* [*factored resistance*] in accordance with the applicable design method in Section A4, A5, or A6.

$$P_n = \frac{\pi d_e^2}{4} F_{xx} \quad (Eq.\ E2.2.2\text{-}1)$$

$$P_n = 0.8(F_u/F_y)^2 t d_a F_u \quad (Eq.\ E2.2.2\text{-}2)$$

For panel and deck applications:

Ω = 2.50 (*ASD*)
ϕ = 0.60 (*LRFD*)
= 0.50 (*LSD*)

For all other applications:

Ω = 3.00 (ASD)
ϕ = 0.50 (LRFD)
= 0.40 (LSD)

The following limits shall apply:

$t\, d_a\, F_u \le 3$ kips (13.34 kN),
$e_{min} \ge d$,
$F_{xx} \ge$ 60 ksi (410 MPa or 4220 kg/cm^2),
$F_u \le$ 82 ksi (565 MPa or 5770 kg/cm^2) (of connecting sheets), and
$F_{xx} > F_u$.

See Section E2.2.1 for definitions of variables.

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 Eq. 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.

Where it is shown by measurement that a given weld procedure consistently gives a larger effective diameter, d_e, or average diameter, d_a, as applicable, this larger diameter shall be permitted to be used provided the particular welding procedure used for making those welds is followed.

E2.3 Arc Seam Welds

Arc seam welds (See Figure E2.3-1) covered by this *Specification* shall apply only to the following *joints*: ☜B

(a) Sheet to thicker supporting member in the flat position, and
(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 Eq. E2.3-1 or Eq. E2.3-2. The *safety factor* and *resistance factors* in this section shall be used to determine the *allowable strength* or *design strength* [*factored resistance*] in accordance with the applicable design method in Section A4, A5, or A6.

$$P_n = \left[\frac{\pi d_e^2}{4} + L d_e\right] 0.75 F_{xx} \quad (Eq.\ E2.3\text{-}1)$$

$$P_n = 2.5 t F_u (0.25L + 0.96 d_a) \quad (Eq.\ E2.3\text{-}2)$$

Ω = 2.55 (*ASD*)
ϕ = 0.60 (*LRFD*)
= 0.50 (*LSD*)

where

P_n = Nominal shear strength [resistance] of arc seam weld

d_e = Effective width of seam weld at fused surfaces

= $0.7d - 1.5t$ (*Eq*. E2.3-3)

where

d = Width of arc seam weld

L = Length of seam weld not including circular ends

(For computation purposes, L shall not exceed 3d)

d_a = Average width of seam weld

= $(d - t)$ for single or double sheets (*Eq*. E2.3-4)

F_u, F_{xx}, and t = Values as defined in Section E2.2.1

The minimum edge distance shall be as determined for the arc spot weld in accordance with Section E2.2.1. See Figure E2.3-2 for details.

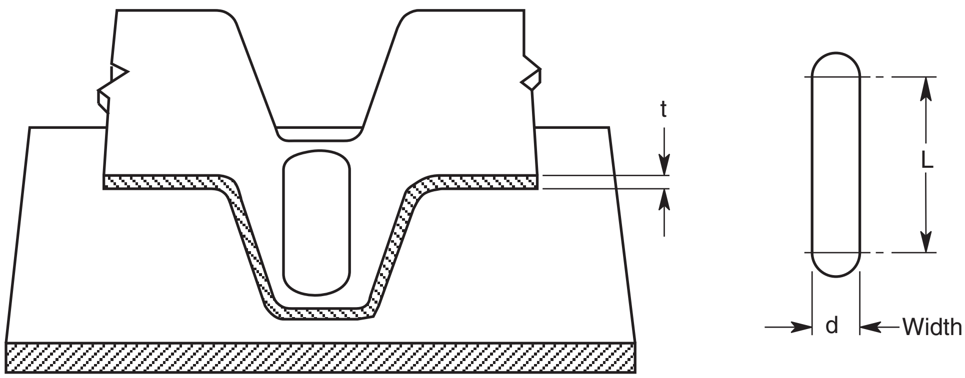


Figure E2.3-1 Arc Seam Welds - Sheet to Supporting Member in Flat Position

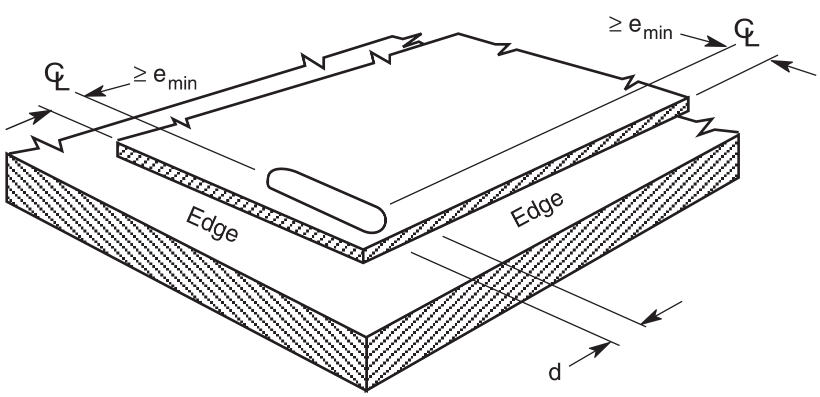


Figure E2.3-2 Edge Distances for Arc Seam Welds

E2.4 Fillet Welds

Fillet welds covered by this *Specification* shall apply to the welding of *joints* in any position, either sheet to sheet, or sheet to thicker steel member.

The nominal shear strength [resistance], P_n, of a fillet weld shall be determined in accordance with this section. The corresponding *safety factors* and *resistance factors* given in this section shall be used to determine the *allowable strength* or *design strength* [*factored resistance*] in accordance with the applicable design method in Section A4, A5, or A6.

(1) For longitudinal loading:

For $L/t < 25$

$$P_n = \left(1 - \frac{0.01L}{t}\right)LtF_u \qquad (Eq.\ E2.4\text{-}1)$$

$\Omega = 2.55$ (ASD)
$\phi = 0.60$ (LRFD)
$= 0.50$ (LSD)

For $L/t \geq 25$

$$P_n = 0.75\, tLF_u \qquad (Eq.\ E2.4\text{-}2)$$

$\Omega = 3.05$ (ASD)
$\phi = 0.50$ (LRFD)
$= 0.40$ (LSD)

(2) For transverse loading:

$$P_n = tLF_u \qquad (Eq.\ E2.4\text{-}3)$$

$\Omega = 2.35$ (ASD)
$\phi = 0.65$ (LRFD)
$= 0.60$ (LSD)

where

t = Least value of t_1 or t_2, as shown in Figures E2.4-1 and E2.4-2

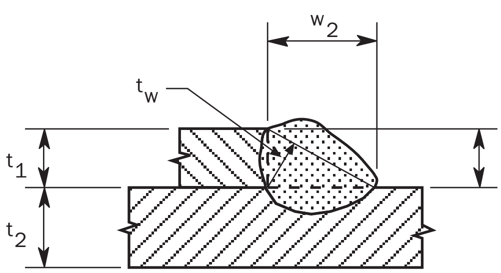


Figure E2.4-1 Fillet Welds – Lap Joint

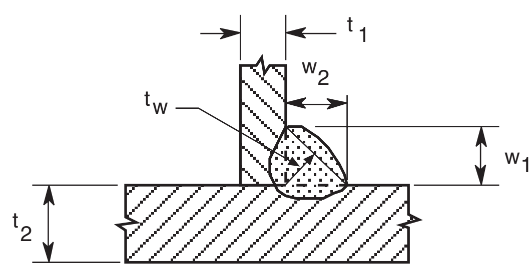


Figure E2.4-2 Fillet Welds – T Joint

In addition, for $t > 0.10$ in. (2.54 mm), the *nominal strength* [*resistance*] determined in accordance with (1) and (2) shall not exceed the following value of P_n:

$$P_n = 0.75\, t_w LF_{xx} \qquad (Eq.\ E2.4\text{-}4)$$

$\Omega = 2.55$ (ASD)
$\phi = 0.60$ (LRFD)
$= 0.50$ (LSD)

where

P_n = Nominal strength [resistance] of fillet weld

L = Length of fillet weld

F_u and F_{xx} = Values as defined in Section E2.2.1

t_w = Effective throat
= $0.707\, w_1$ or $0.707\, w_2$, whichever is smaller. A larger effective throat is permitted if measurement shows that the welding procedure to be used consistently yields a larger value of t_w.

where
w_1 and w_2 = leg of weld (see Figures E2.4-1 and E2.4-2) and $w_1 \le t_1$ in lap joints

E2.5 Flare Groove Welds

Flare groove welds covered by this *Specification* shall apply to welding of *joints* in any position, either sheet to sheet for flare-V groove welds, sheet to sheet for flare-bevel groove welds, or sheet to thicker steel member for flare-bevel groove welds.

The nominal shear strength [resistance], P_n, of a flare groove weld shall be determined in accordance with this section. The corresponding *safety factors* and *resistance factors* given in this section shall be used to determine the *allowable strength* or *design strength* [*factored resistance*] in accordance with the applicable design method in Section A4, A5, or A6.

(a) For flare-bevel groove welds, transverse loading (see Figure E2.5-1):

$P_n = 0.833tLF_u$ (*Eq*. E2.5-1)

$\Omega = 2.55$ (*ASD*)
$\phi = 0.60$ (*LRFD*)
$= 0.50$ (*LSD*)

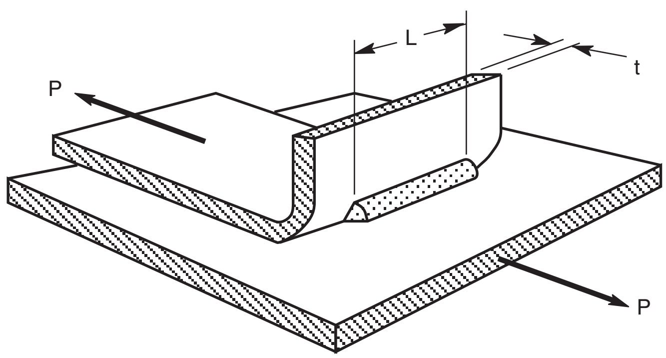


Figure E2.5-1 Flare-Bevel Groove Weld

(b) For flare groove welds, longitudinal loading (see Figures E2.5-2 through E2.5-7):

(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)

$\Omega = 2.80$ (ASD)
$\phi = 0.55$ (LRFD)
$= 0.45$ (LSD)

(2) For $t_w \ge 2t$ with the lip height, h, equal to or greater than weld length, L:

$P_n = 1.50tLF_u$ (*Eq*. E2.5-3)

$\Omega = 2.80$ (ASD)
$\phi = 0.55$ (LRFD)
$= 0.45$ (LSD)

In addition, for t > 0.10 in. (2.54 mm), the *nominal strength* [*resistance*] determined in accordance with (a) and (b) shall not exceed the value of P_n calculated in accordance with Eq. E2.5-4.

$$P_n = 0.75 t_w L F_{xx} \qquad (Eq.\ E2.5\text{-}4)$$

Ω = 2.55 (ASD)

ϕ = 0.60 (LRFD)

= 0.50 (LSD)

where

P_n = Nominal strength [resistance] of flare groove weld

t = *Thickness* of welded member as defined in Figures E2.5-1 to E2.5-7

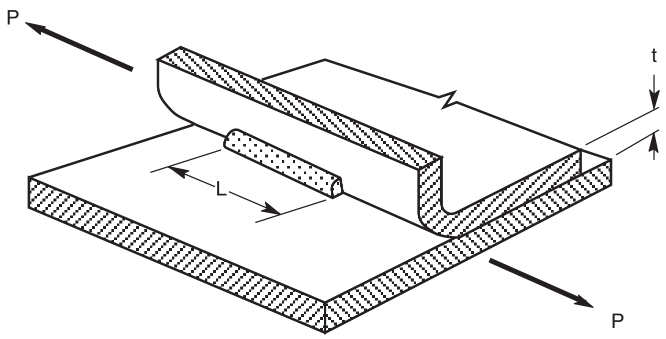


Figure E2.5-2 Shear in Flare Bevel Groove Weld

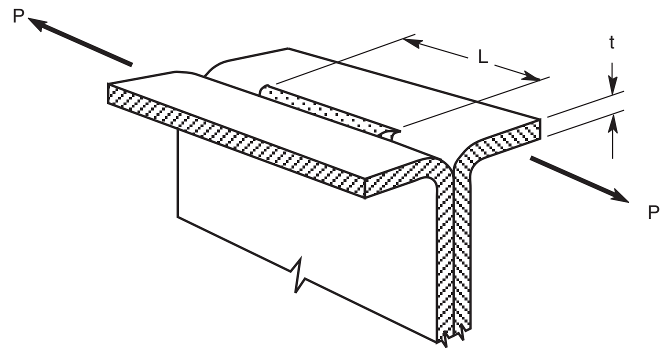


Figure E2.5-3 Shear in Flare V-Groove Weld

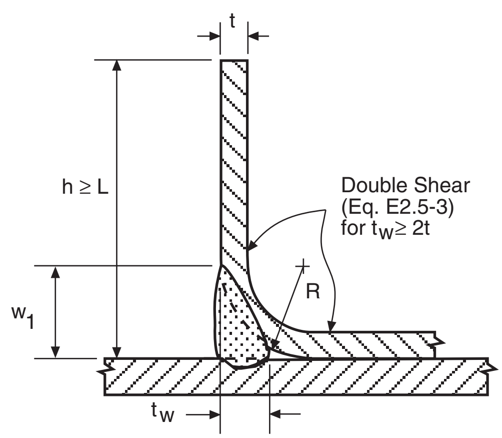


Figure E2.5-4 Flare Bevel Groove Weld (Filled flush to surface, w_1 = R)

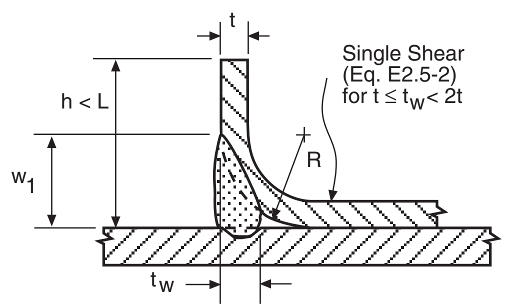


Figure E2.5-5 Flare Bevel Groove Weld (Filled flush to surface, w_1 = R)

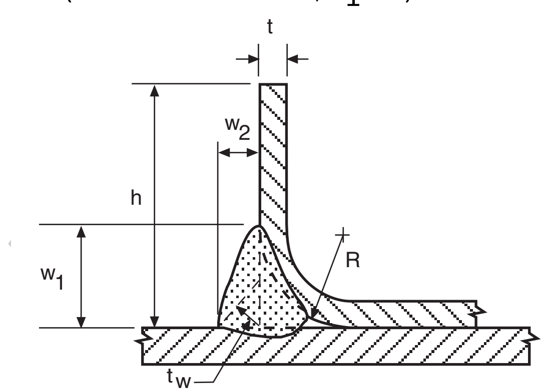


Figure E2.5-6 Flare Bevel Groove Weld (Not filled flush to surface, w_1 > R)

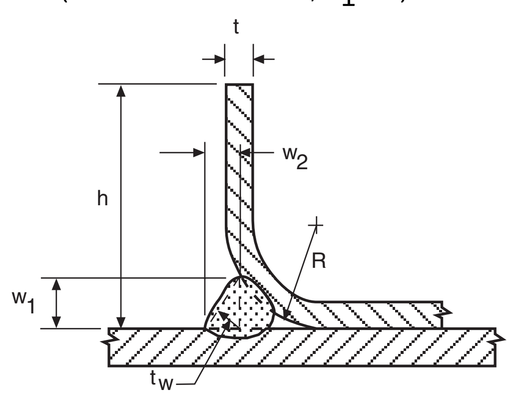


Figure E2.5-7 Flare Bevel Groove Weld (Not filled flush to surface, w_1 < R)

L = Length of weld
F_u and F_{xx} = Values as defined in Section E2.2.1
h = Height of lip
t_w = Effective throat of flare groove weld filled flush to surface (See Figures E2.5-4 and E2.5-5):
= (5/16)R for flare bevel groove weld
= (1/2)R when $R \le 1/2$ in. (12.7mm) for flare V-groove weld
= (3/8)R when $R > 1/2$ in. (12.7mm) for flare V-groove weld
= Effective throat of flare groove weld not filled flush to surface:
= $0.707w_1$ or $0.707w_2$, whichever is smaller (see Figures E2.5-6 and E2.5-7)
= A larger effective throat than those above is permitted if measurement shows that the welding procedure to be used consistently yields a larger value of t_w

where
R = Radius of outside bend surface
w_1 and w_2 = Leg of weld (see Figures E2.5-6 and E2.5-7)

E2.6 Resistance Welds

The nominal shear strength [resistance], P_n, of spot welds shall be determined in accordance with this section. The *safety factor* and *resistance factors* given in this section shall be used to determine the *allowable strength* or *design strength* [*factored resistance*] in accordance with the applicable design method in Section A4, A5, or A6.

Ω = 2.35 (*ASD*)
ϕ = 0.65 (*LRFD*)
= 0.55 (*LSD*)

When t is in inches and P_n is in kips:

For 0.01 in. $\le t <$ 0.14 in.

$$P_n = 144t^{1.47} \quad (Eq.\ E2.6\text{-}1)$$

For 0.14 in. $\le t \le$ 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 $\le t <$ 3.56 mm

$$P_n = 5.51t^{1.47} \quad (Eq.\ E2.6\text{-}3)$$

For 3.56 mm $\le t \le$ 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 $\le t <$ 0.356 cm

$$P_n = 16600t^{1.47} \quad (Eq.\ E2.6\text{-}5)$$

For 0.356 cm $\le t \le$ 0.457 cm

$$P_n = 7750t + 875 \quad (Eq.\ E2.6\text{-}6)$$

where
P_n = *Nominal strength* [resistance] of resistance weld
t = *Thickness* of thinnest outside sheet

E2.7 Rupture in Net Section of Members other than Flat Sheets (Shear Lag)

The nominal tensile strength [resistance] of a welded member shall be determined in accordance with Section C2. For rupture and/or *yielding* in the effective net section of the connected part, the nominal tensile strength [resistance], P_n, shall be determined in accordance with Eq. E2.7-1. The *safety factor* and *resistance factors* given in this section shall be used to determine the *allowable strength* or *design strength* [*factored resistance*] in accordance with the applicable design method in Section A4, A5, or A6.

$$P_n = A_e F_u \qquad (Eq.\ E2.7\text{-}1)$$

Ω = 2.50 (*ASD*)

ϕ = 0.60 (*LRFD*)

= 0.50 (*LSD*)

where

F_u = *Tensile strength* of the connected part as determined in accordance with 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 load is transmitted directly to all of the cross-sectional elements.

Otherwise the reduction coefficient U shall be determined in accordance with (a) or (b):

(a) For angle members

$$U = 1.0 - 1.20\,\bar{x}/L < 0.9 \qquad (Eq.\ E2.7\text{-}2)$$

but $U \geq 0.4$.

(b) For channel members

$$U = 1.0 - 0.36\,\bar{x}/L < 0.9 \qquad (Eq.\ E2.7\text{-}3)$$

but $U \geq 0.5$.

where

$\bar{x}$ = Distance from shear plane to centroid of cross-section

L = Length of longitudinal weld

E3 Bolted Connections

The following design criteria and the requirements stipulated in Section E3a of Appendices A and B shall apply to 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), the specifications and standards stipulated in Section E3a of Appendix A or

B shall apply.

A,B

Bolts, nuts, and washers conforming to one of the following ASTM specifications shall be approved for use under this *Specification*:

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

See Section E3.1 of the Appendix A or B for the provisions of this section. A,B

E3.2 Rupture in Net Section (Shear Lag)

See Section E3.2 of the Appendix A or B for the provisions of this section. A,B

E3.3 Bearing

The nominal *bearing* strength [*resistance*] of bolted *connections* shall be determined in accordance with Sections E3.3.1 and E3.3.2. For conditions not shown, the available bearing strength [factored resistance] of bolted connections shall be determined by tests. B

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 in accordance with Eq. E3.3.1-1. The *safety factor* and *resistance factors* given in this section shall be used to determine the *allowable strength* or *design strength* [*factored resistance*] in accordance with the applicable design method in Section A4, A5, or A6.

$$P_n = C m_f d t F_u \qquad (Eq.\ E3.3.1\text{-}1)$$

Ω = 2.50 (*ASD*)
ϕ = 0.60 (*LRFD*)
= 0.50 (*LSD*)

where

C = Bearing factor, determined in accordance with Table E3.3.1-1
m_f = Modification factor for type of bearing *connection*, which shall be determined according to Table E3.3.1-2
d = Nominal bolt diameter
t = Uncoated sheet *thickness*
F_u = *Tensile strength* of sheet as defined in Section A2.1 or A2.2

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, m_f, for Type of Bearing Connection

Type of Bearing Connection	m_f
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 be calculated in accordance with Eq. E3.3.2-1. The *safety factor* and *resistance factors* given in this section shall be used to determine the *available* *strength* [*factored resistance*] in accordance with the applicable design method in Section A4, A5, or A6. In addition, the *available strength* shall not exceed the available strength obtained in accordance with Section E3.3.1.

$$P_n = (4.64\alpha t + 1.53)dtF_u \quad (Eq.\ E3.3.2\text{-}1)$$

Ω = 2.22 (*ASD*)
ϕ = 0.65 (*LRFD*)
= 0.55 (*LSD*)

where
α = Coefficient for conversion of units
= 1 for US customary units (with t in inches)
= 0.0394 for SI units (with t in mm)
= 0.394 for MKS units (with t in cm)
See Section E3.3.1 for definitions of other variables.

E3.4 Shear and Tension in Bolts

See Section E3.4 of the Appendix A or B for provisions provided in this section. A,B

E4 Screw Connections

All E4 requirements shall apply to screws with 0.08 in. (2.03 mm) $\le$ d $\le$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.

Except where otherwise indicated, the following *safety factor* or *resistance factor* shall be used to determine the *allowable strength* or *design strength* [*factored resistance*] in accordance with the applicable design method in Section A4, A5, or A6.

Ω = 3.00 (*ASD*)
ϕ = 0.50 (*LRFD*)
= 0.40 (*LSD*)

Alternatively, design values for a particular application shall be permitted to be based on tests, with the safety factor, Ω, and the resistance factor, ϕ, determined according to Chapter F.

The following notation shall apply to Section E4:

d = Nominal screw diameter
d_h = Screw head diameter or hex washer head integral washer diameter
d_w = Steel washer diameter
d'_w = Effective pull-over resistance diameter
P_{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_{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 screw head or washer

t_2 = Thickness of member not in contact with screw head or washer

t_c = Lesser of depth of penetration and thickness t_2

F_{u1} = *Tensile strength* of member in contact with screw head or washer

F_{u2} = Tensile strength of member not in contact with screw head or washer

E4.1 Minimum Spacing

The distance between the centers of fasteners shall not be less than 3d.

E4.2 Minimum Edge and End Distances

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 [resistance] per screw, P_{ns}, shall be limited by Section E4.3.2.

E4.3 Shear

E4.3.1 Connection Shear Limited by Tilting and Bearing

The nominal shear strength [resistance] per screw, P_{ns}, shall be determined in accordance with this section.

For $t_2/t_1 \leq 1.0$, P_{ns} shall be taken as the smallest of

$$P_{ns} = 4.2\,(t_2^3 d)^{1/2} F_{u2} \qquad (Eq.\ E4.3.1\text{-}1)$$

$$P_{ns} = 2.7\,t_1\,d\,F_{u1} \qquad (Eq.\ E4.3.1\text{-}2)$$

$$P_{ns} = 2.7\,t_2\,d\,F_{u2} \qquad (Eq.\ E4.3.1\text{-}3)$$

For $t_2/t_1 \geq 2.5$, P_{ns} shall be taken as the smaller of

$$P_{ns} = 2.7\,t_1\,d\,F_{u1} \qquad (Eq.\ E4.3.1\text{-}4)$$

$$P_{ns} = 2.7\,t_2\,d\,F_{u2} \qquad (Eq.\ E4.3.1\text{-}5)$$

For $1.0 < t_2/t_1 < 2.5$, P_{ns} shall be calculated by linear interpolation between the above two cases.

E4.3.2 Connection Shear Limited by End Distance

See Section E4.3.2 of the Appendix A or B for provisions of this section. ☜A,B

E4.3.3 Shear in Screws

The nominal shear strength [*resistance*] of the screw shall be taken as P_{ss}.

In lieu of the value provided in Section E4, the *safety factor* or the *resistance factor* shall be permitted to be determined in accordance with Section F1 and shall be taken as $1.25\Omega \leq 3.0$ (*ASD*), $\phi/1.25 \geq 0.5$ (*LRFD*), or $\phi/1.25 \geq 0.4$ (*LSD*).

E4.4 Tension

For screws that carry tension, the head of the screw or washer, if a washer is provided, shall have a diameter d_h or d_w not less than 5/16 in. (7.94 mm). 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:

$$P_{not} = 0.85\, t_c\, d\, F_{u2} \qquad (Eq.\ E4.4.1\text{-}1)$$

E4.4.2 Pull-Over

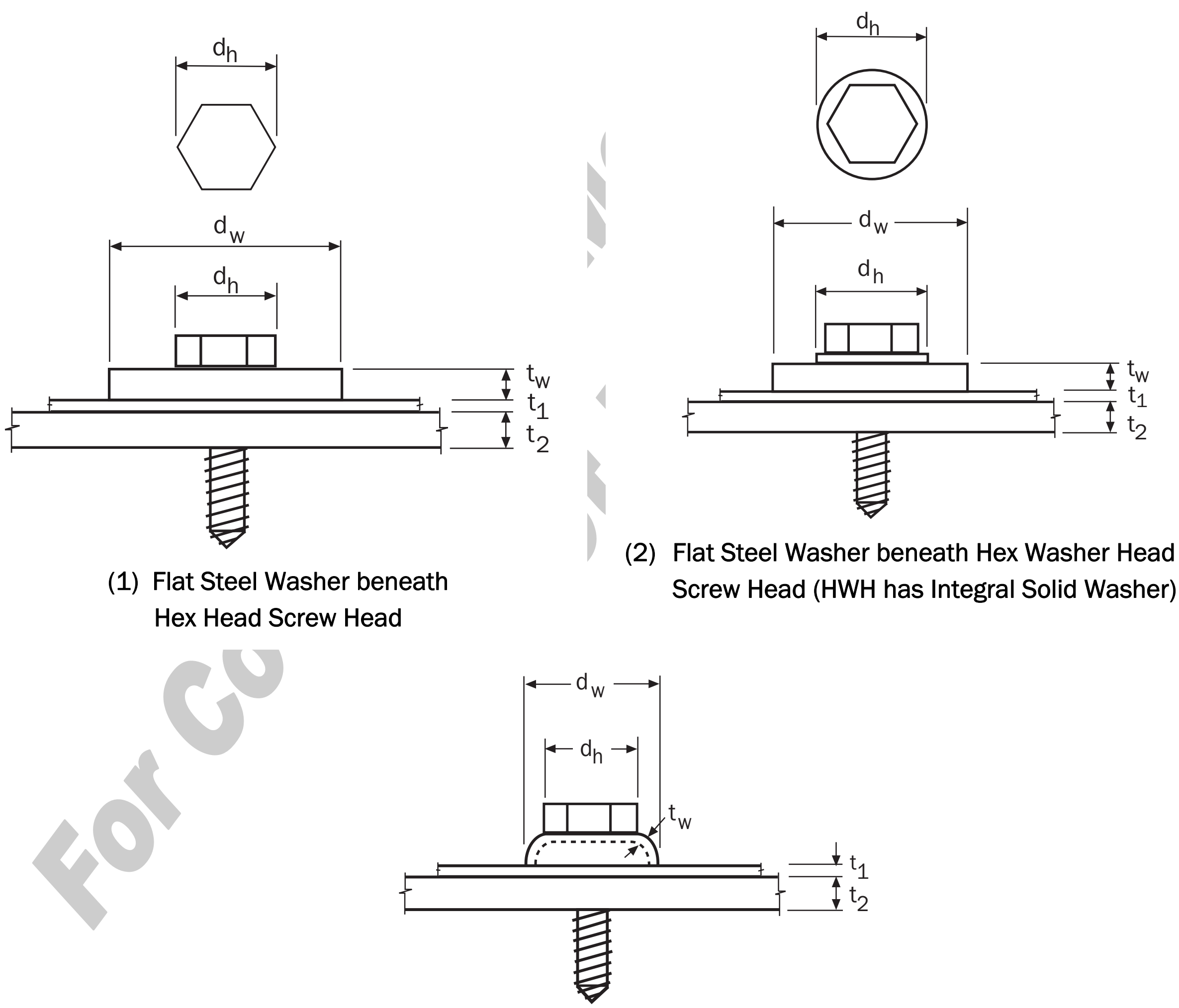
The nominal pull-over strength [*resistance*], P_{nov}, shall be calculated as follows:

$$P_{nov} = 1.5 t_1 d'_w F_{u1} \qquad (Eq.\ E4.4.2\text{-}1)$$

where

d'_w = Effective pull-over diameter determined in accordance with (a), (b), or (c) as follows:

(a) For a round head, a hex head (Figure E4.4.2(1)), or hex washer head (Figure E4.4.2(2)) screw with an independent and solid steel washer beneath the



(1) Flat Steel Washer beneath Hex Head Screw Head

(2) Flat Steel Washer beneath Hex Washer Head Screw Head (HWH has Integral Solid Washer)

(3) Domed Washer (Non-Solid) beneath Screw Head

Figure E4.4.2 Screw Pull-Over with Washer screw head

$d'_w = d_h + 2t_w + t_1 \le d_w$ (*Eq.* E4.4.2-2)

where

d_h = Screw head diameter or hex washer head integral washer diameter

t_w = Steel washer *thickness*

d_w = Steel washer diameter

(b) For a round head, a hex head, or hex washer head screw without an independent washer beneath the screw head:

$d'_w = d_h$ but not larger than 1/2 in. (12.7 mm)

(c) For a domed (non-solid and independent) washer beneath the screw head (Figure E4.4.2(3)), it is permissible to use d'_w as calculated in Eq. E4.4.2-2, with d_h, t_w, and t_1 as defined in Figure E4.4.2(3). In the equation, d'_w can not exceed 5/8 in. (16 mm). Alternatively, pull-over design values for domed washers, including the *safety factor*, Ω, and the *resistance factor*, ϕ, shall be permitted to be determined by test in accordance with Chapter F.

E4.4.3 Tension in Screws

The nominal tension strength [resistance] of the screw shall be taken as P_{ts}.

In lieu of the value provided in Section E4, the *safety factor* or the *resistance factor* shall be permitted to be determined in accordance with Section F1 and shall be taken as $1.25\Omega \le 3.0$ (*ASD*), $\phi/1.25 \ge 0.5$ (*LRFD*), or $\phi/1.25 \ge 0.4$ (*LSD*).

E4.5 Combined Shear and Pull-Over

E4.5.1 ASD Method

For screw *connections* subjected to a combination of shear and tension forces, the following requirement shall be met:

$$\frac{Q}{P_{ns}} + 0.71\frac{T}{P_{nov}} \le \frac{1.10}{\Omega} \quad (Eq.\ E4.5.1\text{-}1)$$

In addition, Q and T shall not exceed the corresponding *allowable strength* determined by Sections E4.3 and E4.4, respectively.

where

Q = Required allowable shear strength of connection

T = Required allowable tension strength of connection

P_{ns} = Nominal shear strength of connection

$= 2.7t_1dF_{u1}$ (*Eq.* E4.5.1-2)

P_{nov} = Nominal pull-over strength of connection

$= 1.5t_1d_wF_{u1}$ (*Eq.* E4.5.1-3)

where

d_w = Larger of screw head diameter or washer diameter

Ω = 2.35

Eq. E4.5.1-1 shall be valid for connections that meet the following limits:

(1) 0.0285 in. (0.724 mm) ≤ t_1 ≤ 0.0445 in. (1.130 mm),

(2) No. 12 and No. 14 self-drilling screws with or without washers,
(3) $d_w \le 0.75$ in. (19.1 mm),
(4) $F_{u1} \le 70$ ksi (483 MPa or 4920 kg/cm^2), and
(5) $t_2/t_1 \ge 2.5$.

For eccentrically loaded connections that produce a non-uniform pull-over force on the fastener, the nominal pull-over strength shall be taken as 50 percent of P_{nov}.

E4.5.2 LRFD and LSD Methods

For screw *connections* subjected to a combination of shear and tension forces, the following requirements shall be met:

$$\frac{\overline{Q}}{P_{ns}} + 0.71\frac{\overline{T}}{P_{nov}} \le 1.10\phi \qquad (Eq.\ E4.5.2\text{-}1)$$

In addition, $\overline{Q}$ and $\overline{T}$ shall not exceed the corresponding *design strength* [*factored resistance*] determined in accordance with Sections E4.3 and E4.4, respectively.

where

$\overline{Q}$ = Required shear strength [factored shear force] of connection
= V_u for *LRFD*
= V_f for *LSD*

$\overline{T}$ = Required tension strength [factored tensile force] of connection
= T_u for LRFD
= T_f for LSD

P_{ns} = Nominal shear strength [resistance] of connection
= $2.7t_1dF_{u1}$ (*Eq*. E4.5.2-2)

P_{nov} = Nominal pull-over strength [resistance] of *connection*
= $1.5t_1d_wF_{u1}$ (*Eq*. E4.5.2-3)

where
d_w = Larger of screw head diameter or washer diameter

ϕ = 0.65 (LRFD)
= 0.55 (LSD)

Eq. E4.5.2-1 shall be valid for connections that meet the following limits:
(1) 0.0285 in. (0.724 mm) $\le t_1 \le$ 0.0445 in. (1.13 mm),
(2) No. 12 and No. 14 self-drilling screws with or without washers,
(3) $d_w \le 0.75$ in. (19.1 mm),
(4) $F_{u1} \le 70$ ksi (483 MPa or 4920 kg/cm^2), and
(5) $t_2/t_1 \ge 2.5$.

For eccentrically loaded connections that produce a non-uniform pull-over force on the fastener, the nominal pull-over strength [resistance] shall be taken as 50 percent of P_{nov}.

E5 Rupture

See Section E5 of Appendix A or B for the provisions of this section. ☜A,B

E6 Connections to Other Materials

E6.1 Bearing

Provisions shall be made to transfer *bearing* forces from steel components covered by this *Specification* to adjacent *structural components* made of other materials.

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, product specifications, product literature, or combination thereof.

E6.3 Shear

Provisions shall be made to transfer shearing forces from steel components covered by this *Specification* to adjacent structural components made of other materials. The required shear and/or *bearing* strength [resistance] on the steel components shall not exceed that allowed by this *Specification*. The available shear strength [resistance] on the fasteners and other material shall not be exceeded. Embedment requirements shall be met. Provisions shall also be made for shearing forces in combination with other forces.

F. TESTS FOR SPECIAL CASES

Tests shall be made by an independent testing laboratory or by a testing laboratory of a manufacturer.

The provisions of Chapter F shall 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 that 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 is given. The average value of all tests made shall then be regarded as the *nominal strength* [*nominal 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. F1.1-1a or Eq. F1.1-1b as applicable.

$\Sigma\gamma_i Q_i \leq \phi R_n$ for *LRFD* (*Eq*. F1.1-1a)

$\phi R_n \geq \Sigma\gamma_i Q_i$ for *LSD* (*Eq*. F1.1-1b)

where

$\Sigma\gamma_i Q_i$ = *Required strength* [*factored loads*] based on the most critical *load combination* determined in accordance with Section A5.1.2 for LRFD or A6.1.2 for LSD. γ_i and Q_i are *load factors* and *load effects*, respectively.

ϕ = *Resistance factor*

$$= C_\phi (M_m F_m P_m) e^{-\beta_o \sqrt{V_M^2 + V_F^2 + C_P V_P^2 + V_Q^2}} \quad (Eq.\ F1.1\text{-}2)$$

where

C_ϕ = Calibration coefficient
= 1.52 for LRFD
= 1.6 for LRFD for beams having tension flange through-fastened to deck or sheathing and with compression flange laterally unbraced
= 1.42 for LSD
= 1.42 for LSD for beams having tension flange through-fastened to deck or sheathing and with compression flange laterally unbraced

M_m = Mean value of material factor, M, listed in Table F1 for type of component involved

F_m = Mean value of fabrication factor, F, listed in Table F1 for type of component involved

P_m = Mean value of professional factor, P, for tested component

= 1.0

e = Natural logarithmic base

= 2.718

β_o = Target reliability index

= 2.5 for *structural members* and 3.5 for connections for LRFD

= 1.5 for LRFD for beams having tension flange through-fastened to deck or sheathing and with compression flange laterally unbraced

= 3.0 for structural members and 4.0 for connections for LSD

= 3.0 for LSD for beams having tension flange through-fastened to deck or sheathing and with compression flange laterally unbraced

V_M = Coefficient of variation of material factor listed in Table F1 for type of component involved

V_F = Coefficient of variation of fabrication factor listed in Table F1 for type of component involved

C_P = Correction factor

= $(1+1/n)m/(m-2)$ for $n \geq 4$ (*Eq*. F1.1-3)

= 5.7 for n = 3

where

n = Number of tests

m = Degrees of freedom

= n-1

V_P = Coefficient of variation of test results, but not less than 6.5 percent

V_Q = Coefficient of variation of load effect

= 0.21 for LRFD and LSD

= 0.43 for LRFD for beams having tension flange through-fastened to deck or sheathing and with compression flange laterally unbraced

= 0.21 for the LSD for beams having tension flange through-fastened to deck or sheathing and with compression flange laterally unbraced

R_n = Average value of all test results

The listing in Table F1 shall not exclude the use of other documented statistical data if they are established from sufficient results on material properties and fabrication.

For steels not listed in Section A2.1, 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. F1.1-1a or Eq. F1.1-1b, as applicable, except that the resistance factor ϕ shall be taken as unity and the load factor for dead load shall be taken as 1.0.

(c) The mechanical properties of the steel sheet shall be determined based on representative samples of the material taken from the test specimen or the flat sheet used to form the test specimen. Mechanical properties reported by the steel supplier shall not be used in the evaluation of the test results. If the *yield stress* 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 stress* of the steel that the manufacturer intends to use. The test results shall not be adjusted upward if the yield stress of the test specimen is less than the minimum specified yield stress. Similar adjustments shall be made on the basis of *tensile* *strength* instead of yield stress where tensile strength is the critical factor.

Consideration shall also be given to any variation or differences between the design *thickness* and the thickness of the specimens used in the tests.

TABLE 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

Continued

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
Tensile 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				
Shear Strength of Bolt	1.10	0.08	1.00	0.05
Tensile Strength of Bolt	1.10	0.08	1.00	0.05
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

Continued

TABLE F1 (Continued)
Statistical Data for the Determination of Resistance Factor

Type of Component	M_m	V_M	F_m	V_F
Screw Connections				
Shear Strength of Screw	1.10	0.10	1.00	0.10
Tensile Strength of Screw	1.10	0.10	1.00	0.10
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
Tilting and Bearing Strength	1.10	0.08	1.00	0.05
Pull-Out	1.10	0.10	1.00	0.10
Pull-Over	1.10	0.10	1.00	0.10
Combined Shear and Pull-Over	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 strength* shall be calculated as follows:

$$R = R_n/\Omega \qquad (Eq.\ F1.2\text{-}1)$$

where

R_n = Average value of all test results

Ω = *Safety factor*

$$= \frac{1.6}{\phi} \qquad (Eq.\ F1.2\text{-}2)$$

where

ϕ = A value evaluated in accordance with Section F1.1

The *required strength* shall be determined from *nominal loads* and *load combinations* as described in Section A4.

F2 Tests for Confirming Structural Performance

For structural members, *connections*, and assemblies for which the *nominal strength* [*resistance*] is computed in accordance with this *Specification* or its specific references, *confirmatory tests* shall be permitted to be made to demonstrate the strength is not less than the nominal strength [resistance], R_n, specified in this *Specification* or its specific references for the type of behavior involved.

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 conducted in accordance with this section.

(a) Tensile testing procedures shall agree with ASTM A370.

(b) Compressive *yield stress* determinations shall be made by means of compression tests of short specimens of the section. See AISI S902.
The compressive *yield stress* shall be taken as the smaller value of either the maximum compressive strength of the sections divided by the *cross-sectional area* or the *stress* defined by one of the following methods:
   (1) For sharp yielding steel, the yield stress is determined by the autographic diagram method or by the total strain under load method.
   (2) For gradual yielding steel, the yield stress is determined by the strain under load method or by the 0.2 percent offset method.
When the total strain under load method is used, there shall be evidence that the yield stress so determined agrees within 5 percent with the yield stress that would be determined by the 0.2 percent offset method.

(c) Where the principal effect of the loading to which the member will be subjected in service will be to produce bending stresses, the yield stress shall be determined for the flanges only. In determining such yield stress, each specimen shall consist of one complete flange plus a portion of the web of such *flat width* ratio that the value of ρ for the specimen is unity.

(d) For acceptance and control purposes, one full section test shall be made from each *master coil*.

(e) At the option of the manufacturer, either tension or compression tests shall be permitted to be used for routine acceptance and control purposes, provided the manufacturer demonstrates that such tests reliably indicate the yield stress of the section when subjected to the kind of stress under which the member is to be used.

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 this section.

The *yield stress* of flats, F_{yf}, shall be established by means of a weighted average of the yield stresses of standard tensile coupons taken longitudinally from the flat portions of a representative cold-formed member. The weighted average shall be the sum of the products of the average yield stress for each flat portion times its *cross-sectional area*, divided by the total area of flats in the cross-section. Although the exact number of such coupons will depend on the shape of the member, i.e., on the number of flats in the cross-section, at least one tensile coupon shall be taken from the middle of each flat. If the actual virgin yield stress exceeds the specified minimum yield stress, the yield stress of the flats, F_{yf}, shall be adjusted by multiplying the test values by the ratio of the specified minimum yield stress to the actual virgin yield stress.

F3.3 Virgin Steel

The following provisions shall apply to steel produced to other than the ASTM Specifications listed in Section A2.1 when used in sections for which the increased *yield stress* of the steel after cold forming is computed from the *virgin steel properties* in accordance with Section 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 stress 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 structural members* and *connections* subject to cyclic loading within the elastic range of *stresses* of frequency and magnitude sufficient to initiate cracking and progressive failure (fatigue).

G1 General

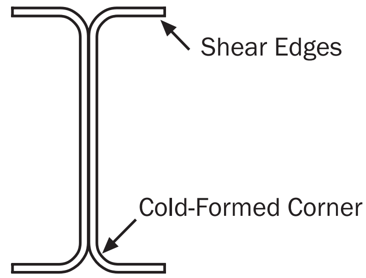
When cyclic loading is a design consideration, the provisions of this chapter shall apply to *stresses* calculated on the basis of unfactored *loads*. The maximum permitted tensile stress due to unfactored loads shall be 0.6 F_y.

Stress range shall be defined as the magnitude of the change in stress due to the application or removal of the 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.

Since the occurrence of full design wind or earthquake loads is too infrequent to warrant consideration in *fatigue* design, the evaluation of fatigue resistance shall not be 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 strength [*resistance*] shall also not be required.

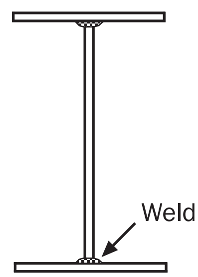
Table G1
Fatigue Design Parameters for Cold-Formed Steel Structures

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]	G1-1
As-received base metal and weld metal in members connected by continuous longitudinal welds	II	1.0×10^{10}	15 (103) [1050]	G1-2
Welded attachments to a plate or a beam, transverse fillet welds, and continuous longitudinal fillet welds less than or equal to 2 in. (50.8 mm), bolt and screw *connections*, and spot welds	III	3.2×10^9	16 (110) [1120]	G1-3, G1-4
Longitudinal fillet welded attachments greater than 2 in. (50.8 mm) parallel to the direction of the applied stress, and intermittent welds parallel to the direction of the applied force	IV	1.0×10^9	9 (62) [633]	G1-4



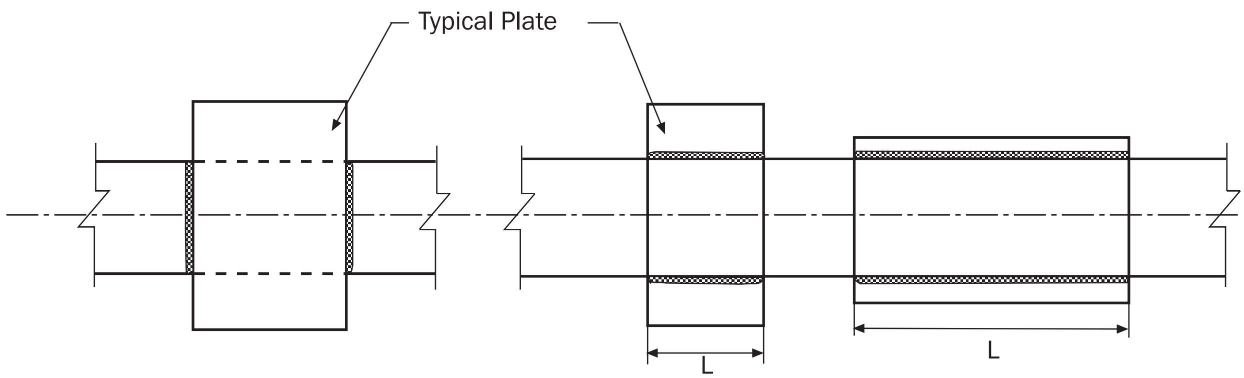
Cold-Formed Steel Channels, Stress Category I

Figure G1-1 Typical Detail for Stress Category I



Welded I Beam, Stress Category II

Figure G1-2 Typical Detail for Stress Category II



(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. (102 mm)

Figure G1-3 Typical Attachments for Stress Categories III and IV

Evaluation of fatigue strength [resistance] shall not be required if the number of cycles of application of live load is less than 20,000.

The fatigue strength [resistance] determined by the provisions of this chapter shall be applicable to structures with corrosion protection or subject only to non-aggressive atmospheres.

The fatigue strength [resistance] determined by the provisions of this chapter shall be applicable only to structures subject to temperatures not exceeding 300°F (149°C).

The contract documents shall either provide complete details including weld sizes, or specify the planned cycle life and the maximum range of moments, shears, and reactions for the connections.

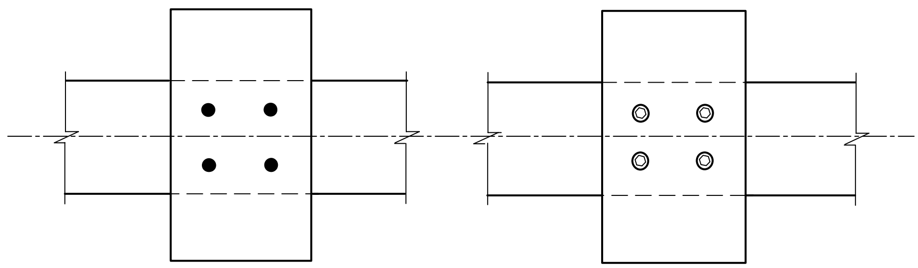


Figure G1-4 Typical Attachments for Stress Category III

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* [specified] shall not exceed the design stress range computed using Equation G3-1 for all stress categories as follows:

$$F_{SR} = (\alpha C_f/N)^{0.333} \geq F_{TH} \qquad (Eq.\ G3\text{-}1)$$

where

F_{SR} = Design stress range

α = Coefficient for conversion of units
= 1 for US customary units
= 327 for SI units
= 352,000 for MKS units

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

G4 Bolts and Threaded Parts

For mechanically fastened *connections* loaded in shear, the maximum range of *stress* in the connected material at *service loads* [specified] shall not exceed the design stress range computed using Equation G3-1. The factor C_f shall be taken as 22 x 10^8. The threshold stress, F_{TH}, shall be taken as 7 ksi (48 MPa or 492 kg/cm^2).

For not-fully-tightened high-strength bolts, common bolts, and threaded anchor rods with cut, ground, or rolled threads, the maximum range of tensile stress on the net tensile area from applied axial load and moment plus load due to prying action shall not exceed the design stress range computed using Equation G3-1. The factor C_f shall be taken as 3.9 x 10^8. The threshold stress, F_{TH}, shall be taken as 7 ksi (48 MPa or 492 kg/cm^2). The net tensile area shall be calculated by Eq. G4-1a or G4-1b as applicable.

$A_t = (\pi/4)\,[d_b - (0.9743/n)]^2$ for US Customary units (*Eq*. G4-1a)

$A_t = (\pi/4)\,[d_b - (0.9382p)]^2$ for SI or MKS units (*Eq*. G4-1b)

where:

A_t = Net tensile area
d_b = Nominal diameter (body or shank diameter)
n = Number of threads per inch
p = Pitch (mm per thread for SI units and cm per thread for MKS units)

G5 Special Fabrication Requirements

Backing bars in welded *connections* that are parallel to the *stress* field shall be permitted to remain in place, and if used, shall be continuous.

Backing bars that are perpendicular to the stress field, if used, shall be removed and the *joint* back gouged and welded.

Flame cut edges subject to cyclic stress ranges shall have a surface roughness not to exceed 1,000 μin. (25 μm) in accordance with ASME B46.1.

Re-entrant corners at cuts, copes, and weld access holes shall form a radius of not less than 3/8 in. (9.53 mm) by pre-drilling or sub-punching and reaming a hole, or by thermal cutting to form the radius of the cut. If the radius portion is formed by thermal cutting, the cut surface shall be ground to a bright metal contour to provide a radiused transition, free of notches, with a surface roughness not to exceed 1,000 μin. (25 μm) in accordance with ASME B46.1 or other equivalent approved standards.

For transverse butt joints in regions of high tensile stress, weld tabs shall be used to provide for cascading the weld termination outside the finished joint. End dams shall not be used. Weld tabs shall be removed and the end of the weld finished flush with the edge of the member. Exception: Weld tabs shall not be required for sheet material if the welding procedures used result in smooth, flush edges.

Appendix 1

Design of Cold-Formed Steel Structural Members Using the Direct Strength Method

2007 EDITION

PREFACE

This Appendix provides alternative design procedures to portions of the *North American Specification for the Design of Cold-Formed Steel Structural Members*, Chapters A through G, and Appendices A and B (herein referred to as the main *Specification*). The Direct Strength Method detailed in this Appendix requires determination of the elastic buckling behavior of the member, and then provides a series of nominal strength [resistance] curves for predicting the member strength based on the elastic buckling behavior. The procedure does not require effective width calculations or iteration; instead, it uses gross properties and the elastic buckling behavior of the cross-section to predict the strength. The applicability of these provisions is detailed in the General Provisions of this Appendix.

APPENDIX 1: Design of Cold-Formed Steel Structural Members Using the Direct Strength Method

1.1 General Provisions

1.1.1 Applicability

The provisions of this Appendix shall be permitted to be used to determine the nominal axial (P_n) and flexural (M_n) strengths [resistances] of *cold-formed steel members*. Sections 1.2.1 and 1.2.2 present a method applicable to all cold-formed steel columns and beams. Those members meeting the geometric and material limitations of Section 1.1.1.1 for columns and Section 1.1.1.2 for beams have been pre-qualified for use, and the calibrated *safety factor*, Ω, and *resistance factor*, ϕ, given in 1.2.1 and 1.2.2 shall be permitted to apply. The use of the provisions of Sections 1.2.1 and 1.2.2 for other columns and beams shall be permitted, but the standard Ω and ϕ factors for *rational engineering analysis* (Section A1.1(b) of the main *Specification*) shall apply. The main *Specification* refers to Chapters A through G, Appendices A and B, and Appendix 2 of the *North American Specification for the Design of Cold-Formed Steel Structural Members*.

Currently, the Direct Strength Method provides no explicit provisions for members in tension, shear, combined bending and shear, *web crippling*, combined bending and web crippling, or combined axial load and bending (beam-columns). Further, no provisions are given for structural assemblies or *connections* and *joints*. As detailed in main *Specification*, Section A1.1, the provisions of the main *Specification*, when applicable, shall be used for all cases listed above.

It shall be permitted to substitute the *nominal strengths* [*resistances*], resistance factors, and safety factors from this Appendix for the corresponding values in Sections C3.1, C4.1.1, C4.1.2, C4.1.3, C4.1.4, D6.1.1, and D6.1.2 of the main *Specification*.

For members or situations to which the main *Specification* is not applicable, the Direct Strength Method of this Appendix shall be permitted to be used, as applicable. The usage of the Direct Strength Method shall be subjected to the same provisions as any other rational engineering analysis procedure, as detailed in Section A1.1(b) of the main *Specification*:

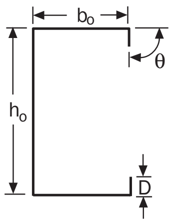
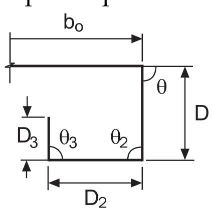
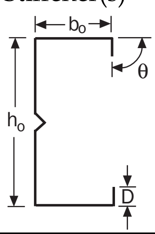
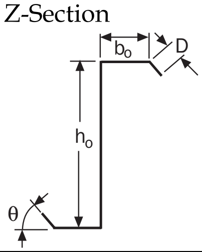
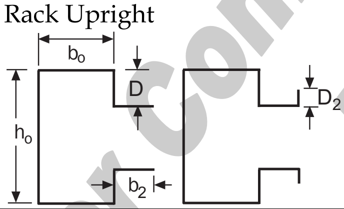
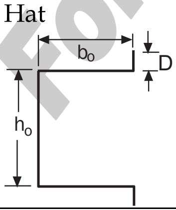
(1) applicable provisions of the main *Specification* shall be followed when they exist, and

(2) increased safety factors, Ω, and reduced resistance factors, ϕ, shall be employed for strength when rational engineering analysis is conducted.

1.1.1.1 Pre-qualified Columns

Unperforated columns that fall within the geometric and material limitations given in Table 1.1.1-1 shall be permitted to be designed using the *safety factor*, Ω, and *resistance factor*, ϕ, defined in Section 1.2.1.

Table 1.1.1-1
Limits for Pre-qualified Columns*

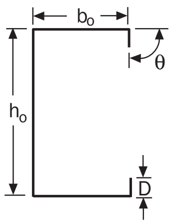
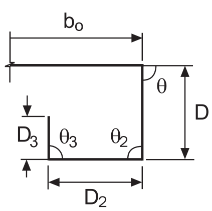
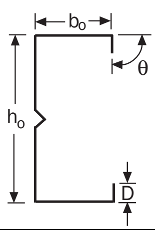
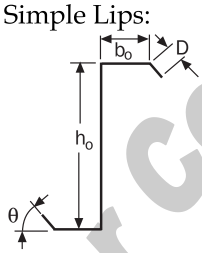
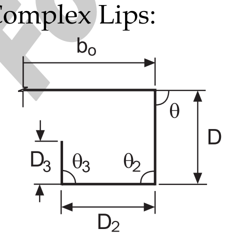
Lipped C-Sections Simple Lips:  Complex Lips: 	For all C-sections: $h_o/t < 472$ $b_o/t < 159$ $4 < D/t < 33$ $0.7 < h_o/b_o < 5.0$ $0.05 < D/b_o < 0.41$ $\theta = 90°$ $E/F_y > 340$ [$F_y < 86$ ksi (593 MPa or 6050 kg/cm^2)] For C-sections with complex lips: $D_2/t < 34$ $D_2/D < 2$ $D_3/t < 34$ $D_3/D_2 < 1$ Note: a) θ_2 is permitted to vary (D_2 lip is permitted to angle inward, outward, etc.) b) θ_3 is permitted to vary (D_3 lip is permitted to angle up, down, etc.)
Lipped C-Section with Web Stiffener(s) 	For one or two intermediate stiffeners: $h_o/t < 489$ $b_o/t < 160$ $6 < D/t < 33$ $1.3 < h_o/b_o < 2.7$ $0.05 < D/b_o < 0.41$ $E/F_y > 340$ [$F_y < 86$ ksi (593 MPa or 6050 kg/cm^2)]
Z-Section 	$h_o/t < 137$ $b_o/t < 56$ $0 < D/t < 36$ $1.5 < h_o/b_o < 2.7$ $0.00 < D/b_o < 0.73$ $\theta = 50°$ $E/F_y > 590$ [$F_y < 50$ ksi (345 MPa or 3520 kg/cm^2)]
Rack Upright 	See C-Section with Complex Lips
Hat 	$h_o/t < 50$ $b_o/t < 20$ $4 < D/t < 6$ $1.0 < h_o/b_o < 1.2$ $D/b_o = 0.13$ $E/F_y > 428$ [$F_y < 69$ ksi (476 MPa or 4850 kg/cm^2)]

Note: * $r/t < 10$, where r is the centerline bend radius
b_o = overall width; D =overall lip depth; t = base metal *thickness*; h_o = overall depth

1.1.1.2 Pre-qualified Beams

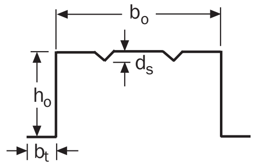
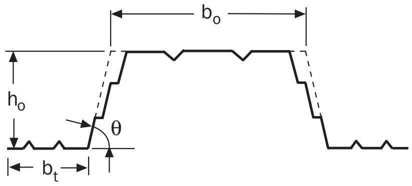
Unperforated beams that fall within the geometric and material limitations given in Table 1.1.1-2 shall be permitted to be designed using the *safety factor*, Ω, and *resistance factor*, ϕ, defined in Section 1.2.2.

Table 1.1.1-2
Limitations for Pre-qualified Beams*

C-Sections Simple Lips:  Complex Lips:  	For all C-sections $h_o/t < 321$ $b_o/t < 75$ $0 < D/t < 34$ $1.5 < h_o/b_o < 17.0$ $0 < D/b_o < 0.70$ $44° < θ < 90°$ $E/F_y > 421$ [$F_y < 70$ ksi (483 MPa or 4920 kg/cm²)] For C-sections with complex lips: $D_2/t < 34$ $D_2/D < 2$ $D_3/t < 34$ $D_3/D_2 < 1$ Note: a) $θ_2$ is permitted to vary (D_2 lip is permitted to angle inward or outward) b) $θ_3$ is permitted to vary (D_3 lip is permitted to angle up or down).
Lipped C-Sections with Web Stiffener  	$h_o/t < 358$ $b_o/t < 58$ $14 < D/t < 17$ $5.5 < h_o/b_o < 11.7$ $0.27 < D/b_o < 0.56$ $θ = 90°$ $E/F_y > 578$ [$F_y < 51$ ksi (352 MPa or 3590 kg/cm²)]
Z-Sections Simple Lips:  Complex Lips:  	For all Z-sections: $h_o/t < 183$ $b_o/t < 71$ $10 < D/t < 16$ $2.5 < h_o/b_o < 4.1$ $0.15 < D/b_o < 0.34$ $36° < θ < 90°$ $E/F_y > 440$ [$F_y < 67$ ksi (462 MPa or 4710 kg/cm²)] For Z-sections with complex lips: $D_2/t < 34$ $D_2/D < 2$ $D_3/t < 34$ $D_3/D_2 < 1$ Note: a) $θ_2$ is permitted to vary (D_2 lip is permitted to is permitted to angle inward, outward, etc.) b) $θ_3$ is permitted to vary (D_3 lip is permitted to angle up, down, etc.)

(Continued)

Table 1.1.1-2
Limitations for Pre-qualified Beams (Continued)

Hats (Decks) with Stiffened Flange in Compression 	$h_o/t < 97$ $b_o/t < 467$ $0 < d_s/t < 26$ (d_s =Depth of stiffener) $0.14 < h_o/b_o < 0.87$ $0.88 < b_o/b_t < 5.4$ $0 < n \le 4$ (n = Number of compression flange stiffeners) $E/F_y > 492$ [$F_y < 60$ ksi (414 MPa or 4220 kg/cm²)]
Trapezoids (Decks) with Stiffened Flange in Compression 	$h_o/t < 203$ $b_o/t < 231$ $0.42 < (h_o/\sin\theta)/b_o < 1.91$ $1.10 < b_o/b_t < 3.38$ $0 < n_c \le 2$ (n_c = Number of compression flange stiffeners) $0 < n_w \le 2$ (n_w = Number of web stiffeners and/or folds) $0 < n_t \le 2$ (n_t = Number of tension flange stiffeners) $52° < \theta < 84°$ (θ = Angle between web and horizontal plane) $E/F_y > 310$ [$F_y < 95$ ksi (655 MPa or 6680 kg/cm²)]

Note:
* $r/t < 10$, where r is the centerline bend radius.
See Section 1.1.1.1 for definitions of other variables given in Table 1.1.1-2.

1.1.2 Elastic Buckling

Analysis shall be used for the determination of the elastic *buckling loads* and/or moments used in this Appendix. For columns, this includes the *local*, *distortional*, and overall *buckling* loads ($P_{cr\ell}$, P_{crd}, and P_{cre} of Section 1.2.1). For beams, this includes the local, distortional, and overall buckling moments ($M_{cr\ell}$, M_{crd}, and M_{cre} of Section 1.2.2). In some cases, for a given column or beam, all three modes do not exist. In such cases, the non-existent mode shall be ignored in the calculations of Sections 1.2.1 and 1.2.2. The commentary to this Appendix provides guidance on appropriate analysis procedures for elastic buckling determination.

1.1.3 Serviceability Determination

The bending deflection at any moment, M, due to *nominal loads* shall be permitted to be determined by reducing the gross moment of inertia, I_g, to an effective moment of inertia for deflection, as given in Eq. 1.1.3-1:

$$I_{eff} = I_g(M_d/M) \le I_g \qquad (Eq.\ 1.1.3\text{-}1)$$

where

M_d = Nominal flexural strength [resistance], M_n, defined in Section 1.2.2, but with M_y replaced by M in all equations of Section 1.2.2

M = Moment due to nominal loads [specified loads] on member to be considered ($M \le M_y$)

1.2 Members

1.2.1 Column Design

The nominal axial strength [resistance], P_n, shall be the minimum of P_{ne}, $P_{n\ell}$, and P_{nd} as given in Sections 1.2.1.1 to 1.2.1.3. For columns meeting the geometric and material criteria of

Section 1.1.1.1, Ω_c and ϕ_c shall be as follows:

Ω_c = 1.80 (*ASD*)
ϕ_c = 0.85 (*LRFD*)
= 0.80 (*LSD*)

For all other columns, Ω and ϕ of the main *Specification*, Section A1.1(b), shall apply. The *available strength* [*factored resistance*] shall be determined in accordance with applicable method in Section A4, A5, or A6 of the main *Specification*.

1.2.1.1 Flexural, Torsional, or Flexural-Torsional Buckling

The nominal axial strength [resistance], P_{ne}, for flexural, torsional, or flexural-torsional buckling shall be calculated in accordance with the following:

(a) For $\lambda_c \leq 1.5$

$$P_{ne} = \left(0.658^{\lambda_c^2}\right)P_y \qquad (Eq.\ 1.2.1\text{-}1)$$

(b) For $\lambda_c > 1.5$

$$P_{ne} = \left(\frac{0.877}{\lambda_c^2}\right)P_y \qquad (Eq.\ 1.2.1\text{-}2)$$

where

$$\lambda_c = \sqrt{P_y/P_{cre}} \qquad (Eq.\ 1.2.1\text{-}3)$$

where

$$P_y = A_g F_y \qquad (Eq.\ 1.2.1\text{-}4)$$

P_{cre} = Minimum of the critical elastic column buckling load in *flexural*, *torsional*, or *flexural-torsional buckling* determined by analysis in accordance with Section 1.1.2

1.2.1.2 Local Buckling

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

(a) For $\lambda_\ell \leq 0.776$

$$P_{n\ell} = P_{ne} \qquad (Eq.\ 1.2.1\text{-}5)$$

(b) For $\lambda_\ell > 0.776$

$$P_{n\ell} = \left[1 - 0.15\left(\frac{P_{cr\ell}}{P_{ne}}\right)^{0.4}\right]\left(\frac{P_{cr\ell}}{P_{ne}}\right)^{0.4} P_{ne} \qquad (Eq.\ 1.2.1\text{-}6)$$

where

$$\lambda_\ell = \sqrt{P_{ne}/P_{cr\ell}} \qquad (Eq.\ 1.2.1\text{-}7)$$

P_{ne} = A value as defined in Section 1.2.1.1

$P_{cr\ell}$ = Critical elastic local column buckling *load* determined by analysis in accordance with Section 1.1.2

1.2.1.3 Distortional Buckling

The nominal axial strength [resistance], P_{nd}, for *distortional buckling* shall be calculated in accordance with the following:

(a) For $\lambda_d \le 0.561$

$$P_{nd} = P_y \quad (Eq.\ 1.2.1\text{-}8)$$

(b) For $\lambda_d > 0.561$

$$P_{nd} = \left(1 - 0.25\left(\frac{P_{crd}}{P_y}\right)^{0.6}\right)\left(\frac{P_{crd}}{P_y}\right)^{0.6} P_y \quad (Eq.\ 1.2.1\text{-}9)$$

where

$$\lambda_d = \sqrt{P_y / P_{crd}} \quad (Eq.\ 1.2.1\text{-}10)$$

where

P_y = A value as given in Eq. 1.2.1-4

P_{crd} = Critical elastic distortional column buckling *load* determined by analysis in accordance with Section 1.1.2

1.2.2 Beam Design

The nominal flexural strength [resistance], M_n, shall be the minimum of M_{ne}, $M_{n\ell}$, and M_{nd} as given in Sections 1.2.2.1 to 1.2.2.3. For beams meeting the geometric and material criteria of Section 1.1.1.2, Ω_b and ϕ_b shall be as follows:

Ω_b = 1.67 (*ASD*)

ϕ_b = 0.90 (*LRFD*)

= 0.85 (*LSD*)

For all other beams, Ω and ϕ of the main *Specification*, Section A1.1(b), shall apply. The *available strength* [*factored resistance*] shall be determined in accordance with applicable method in Section A4, A5, or A6 of the main *Specification*.

1.2.2.1 Lateral-Torsional Buckling

The nominal flexural strength [resistance], M_{ne}, for lateral-torsional buckling shall be calculated in accordance with the following:

(a) For $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

(b) For $2.78M_y \ge M_{cre} \ge 0.56M_y$

$$M_{ne} = \frac{10}{9} M_y \left(1 - \frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

(c) For $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

where

M_{cre} = Critical elastic *lateral-torsional buckling* moment determined by analysis in accordance with Section 1.1.2

$$M_y = S_f F_y \quad (Eq.\ 1.2.2\text{-}4)$$

where
S_f = Gross section modulus referenced to the extreme fiber in first yield

1.2.2.2 Local Buckling

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

(a) For $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

(b) For $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where

$$\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}} \quad (Eq.\ 1.2.2\text{-}7)$$

M_{ne} = A value as defined in Section 1.2.2.1
$M_{cr\ell}$ = Critical elastic local buckling moment determined by analysis in accordance with Section 1.1.2

1.2.2.3 Distortional Buckling

The nominal flexural strength [resistance], M_{nd}, for *distortional buckling* shall be calculated in accordance with the following:

(a) For $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

(b) For $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where

$$\lambda_d = \sqrt{M_y/M_{crd}} \quad (Eq.\ 1.2.2\text{-}10)$$

M_y = A value as given in Eq. 1.2.2-4
M_{crd} = Critical elastic distortional buckling moment determined by analysis in accordance with Section 1.1.2

Appendix 2

Second-Order Analysis

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APPENDIX 2: Second-Order Analysis

This Appendix addresses *second-order analysis* for structural systems comprised of *moment frames*, *braced frames*, *shear walls*, or combinations thereof.

2.1 General Requirements

Members shall satisfy the provisions of Section C5 with the nominal column strengths [nominal axial resistance], P_n, determined using K_x and K_y = 1.0, as well as α_x= 1.0, α_y= 1.0, C_{mx} = 1.0, and C_{my} = 1.0. The *required strengths* [factored forces and moments] for members, *connections*, and other structural elements shall be determined using a *second-order analysis* as specified in this Appendix. All component and connection deformations that contribute to the lateral displacement of the structure shall be considered in the analysis.

2.2 Design and Analysis Constraints

2.2.1 General

The *second-order analysis* shall consider both the effect of loads acting on the deflected shape of a member between *joints* or nodes (*P-δ effects*) and the effect of loads acting on the displaced location of joints or nodes in a structure (*P-Δ effects*). It shall be permitted to perform the analysis using any general second-order analysis method. Analyses shall be conducted according to the design and loading requirements specified in Chapter A. For the *ASD*, the second-order analysis shall be carried out under 1.6 times the ASD *load combinations* and the results shall be divided by 1.6 to obtain the *required strengths* at allowable load levels.

2.2.2 Types of Analysis

It shall be permissible to carry out the *second-order analysis* either on the out-of-plumb geometry without *notional loads* or on the plumb geometry by applying notional loads or minimum lateral loads as defined in Section 2.2.4.

For second-order elastic analysis, axial and flexural stiffnesses shall be reduced as specified in Section 2.2.3.

2.2.3 Reduced Axial and Flexural Stiffnesses

Flexural and axial stiffnesses shall be reduced by using E* in place of E as follows for all members whose flexural and axial stiffnesses are considered to contribute to the lateral stability of the structure:

$$E^* = 0.8\,\tau_b E \qquad (Eq.\ 2\text{-}1)$$

where

τ_b = 1.0 for $\alpha P_r/P_y \le 0.5$

= $4[\alpha P_r/P_y (1 - \alpha P_r/P_y)]$ for $\alpha P_r/P_y > 0.5$

P_r = Required axial compressive strength [factored axial compressive force], kips (N)

P_y = Member yield strength [resistance] (=AF_y, where A is the *full unreduced cross-sectional area*), kips (N)

α = 1.0 (*LRFD* and *LSD*)

= 1.6 (*ASD*)

In cases where flexibility of other structural components such as *connections*, flexible column base details, or horizontal trusses acting as *diaphragms* is modeled explicitly in the analysis, the stiffnesses of the other structural components shall be reduced by a factor of 0.8.

If *notional loads* are used, in lieu of using $\tau_b < 1.0$ where $\alpha P_r/P_y > 0.5$, $\tau_b = 1.0$ shall be permitted to be used for all members, provided that an additional notional load of $0.001Y_i$ is added to the notional load required in Section 2.2.4.

2.2.4 Notional loads

Notional loads shall be applied to the lateral framing system to account for the effects of geometric imperfections. Notional loads are lateral *loads* that are applied at each framing level and specified in terms of the gravity loads applied at that level. The gravity load used to determine the notional load shall be equal to or greater than the gravity load associated with the *load combination* being evaluated. Notional loads shall be applied in the direction that adds to the destabilizing effects under the specified load combination.

A notional load, $N_i = (1/240)\ Y_i$, shall be applied independently in two orthogonal directions as a lateral load in all load combinations. This load shall be in addition to other lateral loads, if any.

N_i = Notional lateral load applied at level i, kips (N)

Y_i = Gravity load from the *LRFD* or *LSD* load combination or 1.6 times the *ASD* load combination applied at level i, kips (N)

The notional load coefficient of 1/240 is based on an assumed initial story out-of-plumbness ratio of 1/240. Where a different assumed out-of-plumbness is justified, the notional load coefficient shall be permitted to be adjusted proportionally to a value not less than 1/500.

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Appendix A:

Provisions Applicable to the United States and Mexico

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PREFACE TO APPENDIX A

Appendix A provides specification provisions that apply to the United States and Mexico. Included are provisions of a broad nature relating to the design method used, ASD or LRFD, and use of ASCE/SEI 7 for loads and load combinations where there is not an applicable building code. Reference documents that are used by both countries are listed here as well.

Also included in Appendix A are technical items where full agreement between countries was not reached. Such items included certain provisions pertaining to the design of

- Beams and compression members (C and Z sections) for standing seam roofs,
- Bolted connections, and
- Tension members

Efforts are being made to minimize these differences in future editions of the *Specification*.

APPENDIX A: PROVISIONS APPLICABLE TO THE UNITED STATES AND MEXICO

This Appendix provides design provisions or supplements to Chapters A through G that specifically applies to the United States and Mexico. This appendix is considered mandatory for applications in the United States and Mexico.

A section number ending with a letter indicates that the provisions herein supplement the corresponding section in Chapters A through G of the *Specification*. A section number not ending with a letter indicates that the section gives the entire design provision.

A1.1a Scope

Designs shall be made in accordance with the provisions for *Load and Resistance Factor Design*, or with the provisions for *Allowable Strength Design*.

A2.2 Other Steels

The listing in Section A2.1 shall not exclude the use of steel up to and including 1 in. (25.4 mm) in *thickness*, ordered or produced to other than the listed specifications, provided the following requirements are met:

(1) The steel shall conform to the chemical and mechanical requirements of one of the listed specifications or other *published specification*.

(2) The chemical and mechanical properties shall be determined by the producer, the supplier, or the purchaser, in accordance with the following specifications. For coated sheets, ASTM A924/A924M; for hot-rolled or cold-rolled sheet and strip, ASTM A568/A568M; for plate and bar, ASTM A6/A6M; for hollow structural sections, such tests shall be made in accordance with the requirements of A500 (for carbon steel) or A847 (for HSLA steel).

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

(4) The steel shall meet the requirements of Section A2.3.

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

If the identification and documentation of the production of the steel have not been established, then in addition to requirements (1) through (5), the manufacturer of the cold-formed steel product shall establish that the *yield stress* and *tensile strength* of the *master coil* are at least 10 percent greater than specified in the referenced published specification.

A2.3.1a Ductility

In seismic design category D, E or F (as defined by ASCE/SEI 7), when material ductility is determined on the basis of the local and uniform elongation criteria of Section A2.3.1, *curtain wall studs* shall be limited to the dead *load* of the curtain wall assembly divided by its surface area, but no greater than 15 psf (0.72 kN/m^2 or 7.32 g/cm^2).

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 ASCE/SEI 7.

A4.1.2 Load Combinations for ASD

The structure and its components shall be designed so that *allowable 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 ASCE/SEI 7.

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 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 ASCE/SEI 7.

A9a Referenced Documents

The following documents are referenced in Appendix A:

1. American Institute of Steel Construction (AISC), One East Wacker Drive, Suite 700, Chicago, Illinois 60601-1802:

   ANSI/AISC 360-05, Specification for Structural Steel Buildings

2. American Iron and Steel Institute (AISI), 1140 Connecticut Avenue, NW, Washington, DC 20036:

   AISI S213-07, North American Standard for Cold-Formed Steel Framing – Lateral Design
   AISI S908-04, Base Test Method for Purlins Supporting a Standing Seam Roof System

3. American Society of Civil Engineers (ASCE), 1801 Alexander Bell Drive, Reston VA, 20191:
   ASCE/SEI 7-05, Minimum Design Loads in Buildings and Other Structures

4. American Welding Society (AWS), 550 N.W. LeJeune Road, Miami, Florida 33135:
   AWS D1.3-98, Structural Welding Code - Sheet Steel
   AWS C1.1/C1.1M-2000, Recommended Practices for Resistance Welding

C2 Tension Members

For axially loaded tension members, the nominal tensile strength, T_n, shall be the smallest value obtained in accordance with the *limit states* of (a), (b) and (c). Unless otherwise specified, the corresponding *safety factor* and the *resistance factor* provided in this section shall be used to determine the *available strengths* in accordance with the applicable method in Section A4 or A5.

(a) For *yielding* in gross section

$$T_n = A_g F_y \quad (Eq.\ C2\text{-}1)$$

$\Omega_t = 1.67$ (*ASD*)

ϕ_t = 0.90 (*LRFD*)

where

T_n = *Nominal strength* of member when loaded in tension

A_g = *Gross area* of cross section

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

(b) For rupture in net section away from *connection*

$$T_n = A_n F_u \quad (Eq.\ C2\text{-}2)$$

Ω_t = 2.00 (ASD)

ϕ_t = 0.75 (LRFD)

where

A_n = *Net area* of cross section

F_u = *Tensile strength* as specified in either Section A2.1 or A2.3.2

(c) For rupture in net section at connection

The available tensile strength shall also be limited by Sections E2.7, E3, and E5 for tension members using welded connections, bolted connections, and screw connections.

D4a Light-Frame Steel Construction

In addition to the cold-formed steel framing standards listed in Section D4, the following standard shall be followed, as applicable:

(e) Light-framed shear walls, diagonal strap bracing (that is part of a structural wall) and diaphragms to resist wind, seismic and other in-plane lateral loads shall be designed in accordance with AISI S213.

D6.1.2 Flexural Members Having One Flange Fastened to a Standing Seam Roof System

The available flexural strength of a C- or Z-section, loaded in a plane parallel to the web with the top flange supporting a standing seam roof system shall be determined using discrete point bracing and the provisions of Section C3.1.2.1, or shall be calculated in accordance with this section. The *safety factor* and the *resistance factor* provided in this section shall be applied to the nominal strength, M_n, calculated by Eq. D6.1.2-1 to determine the *available strengths* in accordance with the applicable method in Section A4 or A5.

$$M_n = RS_eF_y \quad (Eq.\ D6.1.2\text{-}1)$$

Ω_b = 1.67 (ASD)

ϕ_b = 0.90 (LRFD)

where

R = Reduction factor determined in accordance with AISI S908

See Section C3.1.1 for definitions of S_e and F_y.

D6.1.4 Compression of Z-Section Members Having One Flange Fastened to a Standing Seam Roof

These provisions shall apply to Z-sections concentrically loaded along their longitudinal axis, with only one flange attached to standing seam roof panels.

Alternatively, design values for a particular system shall be permitted to be based on discrete point bracing locations, or on tests in accordance with Chapter F.

The nominal axial strength of simple span or continuous Z-sections shall be calculated in accordance with (a) and (b). Unless otherwise specified, the *safety factor* and the *resistance factor* provided in this section shall be used to determine the *available strengths* in accordance with the applicable method in Section A4 or A5.

(a) For weak axis available strength

$$P_n = k_{af} R F_y A \qquad (Eq.\ D6.1.4\text{-}1)$$

Ω = 1.80 (*ASD*)

ϕ = 0.85 (*LRFD*)

where

For $d/t \leq 90$

$k_{af} = 0.36$

For $90 < d/t \leq 130$

$$k_{af} = 0.72 - \frac{d}{250t} \qquad (Eq.\ D6.1.4\text{-}2)$$

For $d/t > 130$

$k_{af} = 0.20$

R = Reduction factor determined from uplift tests performed using AISI S908

A = *Full unreduced cross-sectional area* of Z-section.

d = Z-section depth

t = Z-section *thickness*

See Section C3.1.1 for definition of F_y.

Eq. D6.1.4-1 shall be limited to roof systems meeting the following conditions:

(1) *Purlin* thickness, 0.054 in. (1.37 mm) $\leq t \leq$ 0.125 in. (3.22 mm)
(2) 6 in. (152 mm) $\leq d \leq$ 12 in. (305 mm)
(3) Flanges are edge *stiffened compression elements*
(4) $70 \leq d/t \leq 170$
(5) $2.8 \leq d/b < 5$, where b = Z section flange width.
(6) $16 \leq \dfrac{\text{flange flat width}}{t} < 50$
(7) Both flanges are prevented from moving laterally at the supports
(8) *Yield stress*, $F_y \leq 70$ ksi (483 MPa or 4920 kg/cm^2)

(b) The available strength about the strong axis shall be determined in accordance with Section C4.1 and C4.1.1.

D6.2.1a Strength [Resistance] of Standing Seam Roof Panel Systems

In addition to the provisions provided in Section D6.2.1, for *load combinations* that include wind uplift, the nominal wind *load* shall be permitted to be multiplied by 0.67 provided the tested system and wind load evaluation satisfies the following conditions:

(a) The roof system is tested in accordance with AISI S906.
(b) The wind load is calculated using ASCE/SEI 7 for components and cladding, Method 1 (Simplified Procedure) or Method 2 (Analytical Procedure).
(c) The area of the roof being evaluated is in Zone 2 (edge zone) or Zone 3 (corner zone),

as defined in ASCE/SEI 7, i.e. the 0.67 factor does not apply to the field of the roof (Zone 1).

(d) The base metal *thickness* of the standing seam roof panel is greater than or equal to 0.023 in. (0.59 mm) and less than or equal to 0.030 in. (0.77 mm).

(e) For trapezoidal profile standing seam roof panels, the distance between sidelaps is no greater than 24 in. (610 mm).

(f) For vertical rib profile standing seam roof panels, the distance between sidelaps is no greater than 18 in. (460 mm).

(g) The observed failure mode of the tested system is one of the following:

(i) The standing seam roof clip mechanically fails by separating from the panel sidelap.

(ii) The standing seam roof clip mechanically fails by the sliding tab separating from the stationary base.

E2a Welded Connections

Welded connections in which the *thickness* of the thinnest connected part is greater than 3/16 in. (4.76 mm) shall be in accordance with ANSI/AISC-360.

Except as modified herein, arc welds on steel where at least one of the connected parts is 3/16 in. (4.76 mm) or less in thickness shall be made in accordance with AWS D1.3. Welders and welding procedures shall be qualified as specified in AWS D1.3. These provisions are intended to cover the welding positions as listed 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). Bolted *connections* in which the thickness of the thinnest connected part is equal to or greater than 3/16 in. (4.76 mm) shall be in accordance with ANSI/AISC-360.

The holes for bolts shall not exceed the sizes specified in Table E3a, except that larger holes are permitted to 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 shall be permitted to be used as approved by the designer. The length of 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. In the situation where the hole occurs within the lap of lapped and nested zee members, the above requirements regarding the direction of the slot and the use of washers shall be permitted not to apply, subject to the following limits:

1) 1/2 in. (12.7 mm) diameter bolts only,
2) Maximum slot size is 9/16 in. x 7/8 in. (14.3 mm x 22.2 mm) slotted vertically,
3) Maximum oversize hole is 5/8 in. (15.9 mm) diameter,
4) Minimum member thickness is 0.060 in. (1.52 mm) nominal,
5) Maximum member *yield stress* is 60 ksi (410 MPa, and 4220 kg/cm^2),
6) Minimum lap length measured from center of frame to end of lap is 1.5 times the member depth.

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 $(2\,1/2\ d)$

E3.1 Shear, Spacing and Edge Distance

The nominal shear strength, P_n, of the connected part as affected by spacing and edge distance in the direction of applied force shall be calculated in accordance with Eq. E3.1-1. The corresponding *safety factor* and the *resistance factor* provided in this section shall be used to determine the *available strengths* in accordance with the applicable method in Section A4 or A5.

$P_n = teF_u$ (*Eq*. E3.1-1)

(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* per bolt

e = Distance measured in line of force from center of a standard hole to nearest edge of adjacent hole or to end of connected part

t = *Thickness* of thinnest connected part

F_u = *Tensile strength* of connected part as specified in Section A2.1, A2.2 or A2.3.2

F_{sy} = *Yield stress* of connected part as specified in Section A2.1, A2.2 or A2.3.2

In addition, 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 used in Eq. E3.1-1, 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 Rupture in Net Section (Shear Lag)

The nominal tensile strength of a bolted member shall be determined in accordance with Section C2. For rupture in the effective net section of the connected part, the nominal tensile strength [resistance], P_n, shall be determined in accordance with this section. Unless otherwise specified, the corresponding *safety factor* and the *resistance factor* provided in this section shall be used to determine the *available strengths* in accordance with the applicable method in Section A4 or A5.

(a) For flat sheet *connections* not having staggered hole patterns

$P_n = A_nF_t$ (*Eq*. E3.2-1)

(1) 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

$F_t = (0.1 + 3d/s)\ F_u \leq F_u$ (*Eq*. E3.2-2)

For multiple bolts in the line parallel to the force

$F_t = F_u$ (*Eq*. E3.2-3)

For double shear:

$\Omega = 2.00$ (*ASD*)

$\phi = 0.65$ (*LRFD*)

For single shear:

Ω = 2.22 (ASD)

ϕ = 0.55 (LRFD)

(2) 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

For a single bolt, or a single row of bolts perpendicular to the force

$$F_t = (2.5d/s)\, F_u \le F_u \qquad (Eq.\ E3.2\text{-}4)$$

For multiple bolts in the line parallel to the force

$$F_t = F_u \qquad (Eq.\ E3.2\text{-}5)$$

Ω = 2.22 (ASD)

ϕ = 0.65 (LRFD)

where

A_n = *Net area* of connected part

F_t = Nominal tensile stress in flat sheet

d = Nominal bolt diameter

s = Sheet width divided by number of bolt holes in cross section being analyzed (when evaluating F_t)

F_u = *Tensile strength* of connected part as specified in Section A2.1, A2.2 or A2.3.2

(b) For flat sheet connections having staggered hole patterns

$$P_n = A_n F_t \qquad (Eq.\ E3.2\text{-}6)$$

Ω = 2.22 (ASD)

ϕ = 0.65 (LRFD)

where

F_t is determined in accordance with Eqs. E3.2-2 to E3.2-5.

$$A_n = 0.90\,[A_g - n_b d_h t + (\sum s'^2/4g)t] \qquad (Eq.\ E3.2\text{-}7)$$

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

See Section E3.1 for the definition of t.

(c) For other than flat sheet

$$P_n = A_e F_u \qquad (Eq.\ E3.2\text{-}8)$$

Ω = 2.22 (ASD)

ϕ = 0.65 (LRFD)

where

A_e = $A_n U$, 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:

(1) For angle members having two or more bolts in the line of force

$$U = 1.0 - 1.20\,\bar{x}/L < 0.9 \qquad (Eq.\ E3.2\text{-}9)$$

but $U \ge 0.4$.

(2) For channel members having two or more bolts in the line of force

$$U = 1.0 - 0.36\ \bar{x}/L < 0.9 \qquad (Eq.\ E3.2\text{-}10)$$

but $U \geq 0.5$.

where

$\bar{x}$ = 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, P_n, resulting from shear, tension or a combination of shear and tension shall be calculated in accordance with this section. The corresponding *safety factor* and the *resistance factor* provided in Table E3.4-1 shall be used to determine the *available strengths* in accordance with the applicable method in Section A4 or A5.

$$P_n = A_b F_n \qquad (Eq.\ E3.4\text{-}1)$$

where

A_b = *Gross cross-sectional area* of bolt

F_n = *Nominal strength* ksi (MPa), is determined in accordance with (a) or (b) as follows:

(a) When bolts are subjected to shear only or tension only

F_n shall be given by F_{nv} or F_{nt} in Table E3.4-1.

Corresponding *safety* and *resistance factors*, Ω and ϕ, shall be in accordance with Table E3.4-1.

The pullover strength of the connected sheet at the bolt head, nut or washer shall be considered where bolt tension is involved. See Section E6.2.

(b) When bolts are subjected to a combination of shear and tension, F_n, is given by F'_{nt} in Eq. E3.4-2 or E3.4-3 as follows

For *ASD*

$$F'_{nt} = 1.3\,F_{nt} - \frac{\Omega F_{nt}}{F_{nv}} f_v \leq F_{nt} \qquad (Eq.\ E3.4\text{-}2)$$

For *LRFD*

$$F'_{nt} = 1.3\,F_{nt} - \frac{F_{nt}}{\phi F_{nv}} f_v \leq F_{nt} \qquad (Eq.\ E3.4\text{-}3)$$

where

F'_{nt} = Nominal tensile stress modified to include the effects of required shear stress, ksi (MPa)

F_{nt} = Nominal tensile stress from Table E3.4-1

F_{nv} = Nominal shear stress from Table E3.4-1

f_v = Required shear stress, ksi (MPa)

Ω = Safety factor for shear from Table E3.4-1

ϕ = Resistance factor for shear from Table E3.4-1

In addition, the required shear stress, f_v, shall not exceed the allowable shear stress, F_{nv} / Ω (ASD) or the design shear stress, ϕF_{nv} (LRFD), of the fastener.

TABLE E3.4-1
Nominal Tensile and Shear Strengths for Bolts

	Tensile Strength			Shear Strength		
	Safety Factor Ω (ASD)	Resistance Factor ϕ (LRFD)	Nominal Stress F_{nt}, ksi (MPa)	Safety Factor Ω (ASD)	Resistance Factor ϕ (LRFD)	Nominal Stress F_{nv}, ksi (MPa)
A307 Bolts, Grade A 1/4 in. (6.4 mm) ≤ d <1/2 in. (12.7 mm)	2.25	0.75	40.5 (279)	2.4	0.65	24.0 (165)
A307 Bolts, Grade A d ≥ 1/2 in (12.7 mm).	2.25		45.0 (310)			27.0 (186)
A325 bolts, when threads are not excluded from shear planes	2.0		90.0 (621)			54.0 (372)
A325 bolts, when threads are excluded from shear planes			90.0 (621)			72.0 (496)
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)			59.0 (407)
A354 Grade BD Bolts 1/4 in. (6.4 mm) ≤ d < 1/2 in. (12.7 mm), when threads are excluded from shear planes			101.0 (696)			90.0 (621)
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)			47.0 (324)
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)			72.0 (496)
A490 Bolts, when threads are not excluded from shear planes			112.5 (776)			67.5 (465)
A490 Bolts, when threads are excluded from shear planes			112.5 (776)			90.0 (621)

In Table E3.4-1, the shear strength shall apply 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.

E4.3.2 Connection Shear Limited by End Distance

The nominal shear strength per screw, P_{ns} shall not exceed that calculated in accordance with Eq. E4.3.2-1 where the distance to an end of the connected part is parallel to the line of the applied force. The *safety factor* and the *resistance factor* provided in this section shall be used to determine the *available strengths* in accordance with the applicable method in Section A4 or A5.

$$P_{ns} = teF_u \qquad (Eq.\ E4.3.2\text{-}1)$$

Ω = 3.00 (*ASD*)

ϕ = 0.50 (*LRFD*)

where

t = *Thickness* of part in which end distance is measured

e = Distance measured in line of force from center of a standard hole to nearest end of connected part.

F_u = *Tensile strength* of part in which 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, V_n, shall be calculated in accordance with Eq. E5.1-1. The *safety factor* and the *resistance factor* provided in this section shall be used to determine the *available strengths* in accordance with the applicable method in Section A4 or A5.

$$V_n = 0.6\, F_u A_{wn} \qquad (Eq.\ E5.1\text{-}1)$$

Ω = 2.00 (*ASD*)

ϕ = 0.75 (*LRFD*)

where

$$A_{wn} = (h_{wc} - nd_h)t \qquad (Eq.\ E5.1\text{-}2)$$

h_{wc} = Coped flat web depth

n = Number of holes in critical plane

d_h = Hole diameter

F_u = *Tensile strength* of connected part as specified in Section A2.1 or A2.2

t = *Thickness* of coped *web*

E5.2 Tension Rupture

The available tensile rupture strength 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

When the *thickness* of the thinnest connected part is less than 3/16 in. (4.76 mm), the block shear rupture *nominal strength*, R_n, shall be determined in accordance with this section. *Connections* in which the thickness of the thinnest connected part is equal to or greater than 3/16 in. (4.76 mm) shall be in accordance with ANSI/AISC-360.

The nominal block shear rupture strength, R_n, shall be determined as the lesser of Eqs. E5.3-1 and E5.3-2. The corresponding *safety factor* and the *resistance factor* provided in this section shall be used to determine the *available strengths* in accordance with the applicable method in Section A4 or A5.

$$R_n = 0.6F_yA_{gv} + F_uA_{nt} \quad (Eq.\ E5.3\text{-}1)$$

$$R_n = 0.6F_uA_{nv} + F_uA_{nt} \quad (Eq.\ E5.3\text{-}2)$$

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_{nv} = *Net area* subject to shear

A_{nt} = Net area subject to tension

CANADIAN STANDARDS ASSOCIATION

Appendix B:

Provisions Applicable to

Canada

2007 EDITION

PREFACE TO APPENDIX B:

Appendix B provides specification provisions that are applicable only to Canada. Included are items of a general nature such as specific reference documents and provisions on loads and load combinations in accordance with the *National Building Code of Canada*.

While this document is referred to as a "*Specification*", in Canada it is considered a "*Standard*".

Also included in Appendix B are technical items where full agreement between the three countries was not reached. The most noteworthy of these items are

- Beams (C- and Z- sections) for standing seam roofs,
- Bolted connections, and
- Tension members

Efforts will be made to minimize these differences in future editions of the *Specification*.

APPENDIX B: PROVISIONS APPLICABLE TO CANADA

The material contained in this Appendix provides design provisions and supplements that, in addition to those in Chapters A through G, are mandatory for use in Canada. A section number ending with the letter "a" indicates that the provisions herein supplement the corresponding section in Chapters A through G of the *Specification*. A section number not ending with the letter "a" indicates that the section presents the entire design provision.

A1.3a Definitions

The following additional definition applies in Appendix B:

Importance Factor. A factor applied to the *specified loads*, other than dead *load*, to take into account the consequences of failure as related to the *limit state* and the use and occupancy of the building.

Load factor. A factor applied to a specified load that, for the limit states under consideration, takes into account the variability in magnitude of the load, the loading patterns, and the analysis of their effects.

A2.1a Applicable Steels

These steels are in addition to those listed in Section A2.1:

CSA Standards G40.20/G40.21-03, *General requirements for rolled or welded structural quality steel/Structural quality steel*.

A2.2 Other Steels

A2.2.1 Other Structural Quality Steels

For structural quality steels not listed in Section A2.1, F_y and F_u shall be the specified minimum values as given in the material standard or *published specification*. These steels shall also meet the requirements of Section A2.3.

A2.2.2 Other Steels

For steels not covered by Section A2.1 of the *Specification* and A2.2.1 of this Appendix, tensile tests shall be conducted in accordance with Section F3. F_y and F_u shall be 0.8 times the *yield strength* and 0.8 times the *tensile strength* determined from the tests. These steels shall also meet the requirements of Section A2.3.

A2.3.1a Ductility

In buildings with specified short-period spectral acceleration ratios greater than 0.35, and when material ductility is determined on the basis of the local and uniform elongation criteria of Section A2.3.1, the use of *curtain wall studs* shall be limited to wall assemblies whose dead *load* divided by its surface area is not greater than 0.72 kN/m^2.

The specified short-period acceleration ratio is given by the expression $I_E F_a S_a(0.2)$. The terms I_E, F_a, and $S_a(0.2)$ are defined in Volume 1, Division B, Part 4 earthquake load and effects of the National Building Code of Canada.

A3 Loads

The *resistance factors* adopted in this *Specification* are correlated with the *loads* and *load factors* for buildings specified in the *National Building Code of Canada*. For other cases, load factors shall be established in such a way that, in conjunction with the resistance factors used in this *Specification*, the required level of reliability is maintained.

A3.1 Loads and Effects

The following *loads*, forces, and effects shall be considered in the design of *cold-formed steel structural members* and their *connections*:

D = Dead load (a *permanent load* due to the weight of building components, including the mass of the member and all permanent materials of construction, partitions, permanent equipment, and supported earth, plants and trees, multiplied by the acceleration due to gravity to convert mass (kg) to force (N)),

E = Earthquake load and effects (a rare load due to earthquake),

H = A permanent load due to lateral earth pressure, including groundwater,

L = Live load (a variable load depending on intended use and occupancy, including loads due to movable equipment, cranes, and pressure of liquids in containers),

S = Variable load due to snow, including ice and associated rain, or rain,

T = Effects due to contraction, expansion, or deflection caused by temperature changes, shrinkage, moisture changes, creep, ground settlement, or any combination thereof,

W = Wind load (a variable load due to wind).

A3.2 Temperature, Earth, and Hydrostatic Pressure Effects

Where the effects due to lateral earth pressure, H, and imposed deformation, T, affect structural safety, they shall be taken into account in the calculations. H shall have a *load factor* of 1.5, and T shall have a load factor of 1.25.

A6.1.2 Load Factors and Load Combinations for LSD

The effect of *factored loads* for a building or *structural component* shall be determined in accordance with the *load combination cases* listed in Table A6.1.2-1, and the applicable combination being that which results in the most critical effect.

Table A6.1.2-1
Load Combinations for Ultimate Limit States

CASE	Load Combination	
	Principal Loads	Companion Loads
1	1.4D	—
2	(1.25D(4) or 0.9D(1)) + 1.5L(2)	0.5S or 0.4W
3	(1.25D(4) or 0.9D(1)) + 1.5S	0.5L(3) or 0.4W
4	(1.25D(4) or 0.9D(1)) + 1.4W	0.5L(3) or 0.5S
5	1.0D(1) + 1.0E(5)	0.5L(3) + 0.25S

Notes to Table A6.1.2-1:

(1) Except for rocking footings, the counteracting factored dead load, 0.9D in load combination cases (2), (3), and (4), and 1.0D in load combination case (5), shall be used when the dead load acts to resist overturning, uplift, sliding, failure due to stress reversal, and to determine anchorage requirements and the factored resistance of members.

(2) The principal-load factor 1.5 for live load, L, may be reduced to 1.25 for liquids in tanks.

(3) The companion-load factor 0.5 for live load, L, shall be increased to 1.0 for storage areas, and equipment areas, and service rooms.

(4) The load factor 1.25 for dead load, D, for soil, superimposed earth, plants, and trees shall be increased to 1.5, except that when the soil depth exceeds 1.2 m, the factor may be reduced to $1+0.6/h_S$ but not less than 1.25, where h_S is the depth of soil in metres supported by the structure.

(5) Earthquake load, E, in load combination case (5) includes horizontal earth pressure due to earthquake.

A6.1.2.1 Importance Categories

For the purpose of determining *specified loads* S, W, or E, buildings shall be assigned an importance category, based on intended use and occupancy, in accordance with Table A6.1.2.1-1.

Table A6.1.2.1-1
Importance Categories for Buildings

Use and Occupancy	**Importance Category**
Buildings that represent a low direct or indirect hazard to human life in the event of failure, including: • low human-occupancy buildings, where it can be shown that collapse is not likely to cause injury or other serious consequences • minor storage buildings	Low
All buildings except those listed in Categories Low, High, and Post-disaster	Normal
Buildings that are likely to be used as post-disaster shelters, including buildings whose primary use is: • as an elementary, middle, and secondary school • as a community centre Manufacturing and storage facilities containing toxic, explosive, or other hazardous substances in sufficient quantities to be dangerous to the public if released	High

Post-disaster buildings are buildings that are essential to the provision of services in the event of a disaster, and include:
• hospitals, emergency treatment facilities, and blood banks
• telephone exchanges
• power generating stations and electrical substations
• control centres for air, land, and marine transportation
• public water treatment and storage facilities and pumping stations
• sewage treatment facilities and buildings having critical national defense functions
• buildings of the following types, unless exempted from this designation by the authority having jurisdiction:
    • emergency response facilities
    • fire, rescue, and police stations, and housing for vehicles, aircraft, or boats used for such purposes
    • communications facilities, including radio and television stations
	Post-disaster

For buildings in the Low Importance Category, a factor of 0.8 may be applied to the live load.

A6.1.2.2 Importance Factor (I)

The *importance factor* for snow, wind, and earthquake shall be as provided for in Table A6.1.2.2-1.

Table A6.1.2.2-1
Importance Factors for Snow, Wind, and Earthquake

Importance Category	Importance Factor for Ultimate Limit States		
	Snow, I_S	Wind, I_W	Earthquake, I_E
Low	0.8	0.8	0.8
Normal	1.0	1.0	1.0
High	1.15	1.15	1.3
Post-disaster	1.25	1.25	1.5

A9a Reference Documents

This Appendix refers to the following publications, and where such reference is made, it shall be to the edition listed below including all amendments published thereto:

1. Canadian Standards Association (CSA), 5060 Spectrum Way, Suite 100, Mississauga, ON, Canada, L4W 5N6:

   G40.20-04/G40.21-04, *General requirements for rolled or welded structural quality steel/Structural quality steel*

   CAN/CSA-S16-01 (including 2005 Supplement), *Limit states design of steel structures*

   W47.1-03, *Certification of companies for fusion welding of steel*

   W55.3-1965 (R2003), *Resistance Welding Qualification Code for Fabricators of Structural Members Used in Buildings*

   W59-03, *Welded steel construction (metal arc welding)*

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

   National Building Code of Canada, 2005

C2 Tension Members

The nominal tensile *resistance*, T_n, shall be the lesser of the values determined in Sections C2.1 and C2.2 of this Appendix. The nominal tensile resistance shall also be limited by Sections E2.7 of the *Specification*, E3.2 of this Appendix, and E3.3 of the *Specification* for tension members using welded, bolted, and screw connections.

C2.1 Yielding of Gross Section

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

$$T_n = A_g F_y \qquad (Eq.\ C2.1\text{-}1)$$

$$\phi_t = 0.90$$

where

A_g = *Gross area* of cross-section

F_y = *Yield stress* defined in Section A7.1

C2.2 Rupture of Net Section

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

$T_n = A_nF_u$ (*Eq.* C2.2-1)

$\phi_u = 0.75$

where

A_n = Critical *net area* of connected part

$= L_ct$ (*Eq.* C2.2-2)

where

L_c = Summation of critical path lengths of each segment along a potential failure path of minimum strength. L_c shall be determined as follows:

(a) For failure normal to force due to direct tension:

$L_c = L_t$ not involving stagger (*Eq.* C2.2-3)

$L_c = 0.9L_s$ involving stagger (*Eq.* C2.2-4)

(b) For failure parallel to force due to shear:

$L_c = 0.6L_{nv}$ (*Eq.* C2.2-5)

(c) For failure due to block tear-out at end of member:

$L_c = L_t + 0.6L_v$ not involving stagger (*Eq.* C2.2-6)

$L_c = 0.9(L_t + L_s) + 0.6L_v$ involving stagger (*Eq.* C2.2-7)

(d) For failure of coped beams:

$L_c = 0.5L_t + 0.6L_v$ not involving stagger (*Eq.* C2.2-8)

$L_c = 0.45(L_t + L_s) + 0.6L_v$ involving stagger (*Eq.* C2.2-9)

where

L_v = the lesser of CL_{gv} and L_{nv} in (c) and (d)

$C = F_y/F_u$ (*Eq.* C2.2-10)

L_t = Net failure path length normal to force due to direct tension

L_s = Net failure path length inclined to force (including ($s^2/4g$) allowance for staggered holes)

L_{gv} = Gross failure path length parallel to force (i.e., in shear)

L_{nv} = Net failure path length parallel to force (i.e., in shear)

s = Pitch, spacing of fastener parallel to force

g = Gauge, spacing of fastener perpendicular to force

t = Base steel *thickness*

F_u = *Tensile strength* as specified in Section A2

D3a Lateral and Stability Bracing

Structural members and assemblies shall be adequately braced to prevent collapse and to maintain their integrity during the anticipated service life of the structure. Care shall be taken to ensure that the bracing of the entire structural system is complete, particularly when there is interdependence between walls, floors, or roofs acting as diaphragms.

Erection diagrams shall show the details of the essential bracing requirements, including any details necessary to assure the effectiveness of the bracing or bracing system.

The spacing of braces shall not be greater than the unbraced length assumed in the design of the member or component being braced.

D3.1a Symmetrical Beams and Columns

The provisions of Sections D3.1.1 and D3.1.2 of this Appendix apply to symmetric sections in compression or bending in which the applied load does not induce twist.

D3.1.1 Discrete Bracing for Beams

The *factored resistance* of braces shall be at least 2% of the factored compressive force in the compressive flange of a member in bending at the braced location. When more than one brace acts at a common location and the nature of the braces is such that combined action is possible, the bracing force may be shared proportionately. The slenderness ratio of compressive braces shall not exceed 200.

D3.1.2 Bracing by Deck, Slab, or Sheathing for Beams and Columns

The factored resistance of the attachments along the entire length of the braced member shall be at least 5% of either the maximum factored compressive force in a compressive member or the maximum factored compressive force in the compressive flange of a member in bending.

D3.2a C-Section and Z-Section Beams

The provisions of Sections D3.2.2, D3.2.3, and D3.2.4 of this Appendix apply to members in bending in which the applied load in the plane of the web induces twist. Braces shall be designed to avoid local crippling at the points of attachment to the member.

D3.2.2 Discrete Bracing

Braces shall be connected so as to effectively restrain both flanges of the section at the ends and at intervals not greater than one-quarter of the span length in such a manner as to prevent tipping at the ends and lateral deflection of either flange in either direction at the intermediate braces. Fewer braces may be used if this approach can be shown to be acceptable by rational analysis, testing, or Section D6.1.1 of the *Specification*, taking into account the effects of both lateral and torsional displacements.

If fewer braces are used (when shown to be acceptable by rational analysis or testing), those sections used as purlins with "floating"-type roof sheathings that allow for expansion and contraction independent of the purlins shall have a minimum of one brace per bay for spans $\leq$ 7 m and two braces per bay for spans > 7 m.

If one-third or more of the total load on the member is concentrated over a length of one-twelfth or less of the span of the beam, an additional brace shall be placed at or near the centre of this loaded length.

D3.2.3 One Flange Braced by Deck, Slab, or Sheathing

The *factored resistance* of the attachment of the continuous deck, slab, or sheathing shall be in accordance with Section D3.1.2 of this Appendix. Discrete bracing shall be provided to restrain the flange that is not braced by the deck, slab, or sheathing. The spacing of discrete bracing shall be in accordance with Section D3.2.2 of this Appendix.

D3.2.4 Both Flanges Braced by Deck, Slab, or Sheathing

The *factored resistance* of the attachment shall be as given by Section D3.1.2 of this Appendix.

D6.1.2 Flexural Members Having One Flange Fastened to a Standing Seam Roof System

This type of member shall have discrete bracing in accordance with Section D3.2.2 of this Appendix.

E2a Welded Connections

Arc welding shall be performed by a fabricator or erector certified in accordance with CSA W47.1. Resistance welding shall be performed by a fabricator or erector certified in accordance with CSA W55.3.

Where each connected part is over 4.76 mm in base steel thickness, welding shall conform to CSA W59. Where at least one of the connected parts is between 0.70 and 4.76 mm in base steel thickness, welding shall conform to the requirements contained herein and shall be performed in accordance with the applicable requirements of CSA W59. Except as provided for in Section E2.2, where at least one of the connected parts is less than 0.70 mm in base steel thickness, welds shall be considered to have no structural value unless a value is substantiated by appropriate tests.

The resistance in tension or compression of butt welds shall be the same as that prescribed for the lower strength of base metal being joined. The butt weld shall fully penetrate the joint.

E2.2a Arc Spot Welds

This section replaces the first paragraph of Section E2.2 but does not pertain to Section E2.2.1.3.

Arc spot welds (circular in shape) covered by this *Specification* are for welding sheet steel to thicker supporting members in the flat position. The weld is formed by melting through the steel sheet to fuse with the underlying supporting member, whose thickness at the weld location shall be at least 2.5 times the steel sheet thickness (aggregate sheet thickness in the case of multiple plies). The materials to be joined shall be of weldable quality, and the electrodes to be used shall be suited to the materials, the welding method, and the ambient conditions during welding.

The following maximum and minimum sheet thicknesses shall apply:

(a) maximum single sheet thickness shall be 2.0 mm;
(b) minimum sheet thickness shall be 0.70 mm; and
(c) maximum aggregate sheet thickness of double sheets shall be 2.5 mm.

E2.3a Arc Seam Welds

The information in Section E2.2a also applies to arc seam welds that are oval in shape.

E3a Bolted Connections

In addition to the design criteria given in Section E3 of the *Specification*, the design requirements given in Sections E3.1 and E3.2 of this Appendix shall be followed for bolted connections where the thickness of the thinnest connected part is 4.76 mm or less, there are no gaps between connected parts, and fasteners are installed with sufficient tightness to achieve satisfactory performance of the connection under anticipated service conditions. Refer to CSA S16 for the design of mechanically fastened connections in which the thickness of all connected parts exceeds 4.76 mm.

Unless otherwise specified, circular holes for bolts shall not be greater than the nominal bolt diameter, d, plus 1 mm for bolt sizes up to 13 mm and plus 2 mm for bolt sizes over 13 mm.

Slotted or oversized holes may be used when the hole occurs within the lap of lapped or nested Z-members, subject to the following restrictions:

(1) 12.7 mm diameter bolts only, with or without washers,
(2) Maximum slot size is 14.3 x 22.2 mm slotted vertically,
(3) Maximum oversize hole is 15.9 mm diameter,
(4) Minimum member thicknesses is 1.52 mm nominal,
(5) Maximum member yield stress is 410 MPa, and
(6) Minimum lap length measured from centre of frame to end of lap is 1.5 times the member depth.

E3.1 Shear, Spacing, and Edge Distance

The nominal shear resistance per bolt as affected by spacing and edge distance in the direction of the applied force shall be calculated in accordance with the requirements of Section C2.2 of this Appendix.

The center-to-center distance between fasteners shall not be less than 2.5d, and the distance from the center of a fastener to an edge or end shall not be less than 1.5d, where d = nominal diameter of fastener.

E3.2 Rupture of Net Section (Shear Lag)

The nominal tensile *resistance*, P_n, of a tension member other than a flat sheet shall be determined as follows:

$$P_n = A_e F_u \qquad (Eq.\ E3.2\text{-}1)$$

$$\phi = 0.55$$

where

F_u = *Tensile strength* of connected part as specified in Section A2

A_e = A_nU, effective *net area* with reduction coefficient, U

where

U = 1.0 for members when the load is transmitted directly to all of the cross-sectional elements. Otherwise, U shall be determined as follows:

a) For angle members having two or more bolts in the line of force

$$U = 1.0 - 1.2\bar{x}/L < 0.9 \qquad (Eq.\ E3.2\text{-}2)$$

$$U \geq 0.4$$

b) For channel members having two or more bolts in the line of force

$$U = 1.0 - 0.36\bar{x}/L < 0.9 \qquad (Eq.\ E3.2\text{-}3)$$

$$U \geq 0.5.$$

$\bar{x}$ = Distance from shear plane to centroid of cross-section

L = Length of connection

A_n = Net area of connected part

E3.3a Bearing

When the thickness of connected steels is equal to or larger than 4.76 mm, the requirements of CSA S16 shall be met for connection design.

E3.4 Shear and Tension in Bolts

For ASTM A 307 bolts less than 12.7 mm in diameter, refer to Tables E3.4-1 and E3.4-2 of this Appendix. For all other bolts, refer to CSA S16.

The nominal bolt resistance, P_n, resulting from shear, tension, or a combination of shear and tension shall be calculated as follows:

$$P_n = A_b F_n \qquad (Eq.\ E3.4\text{-}1)$$

where

A_b = Gross cross-sectional area of bolt

F_n = A value determined in accordance with i) and ii) below, as applicable:

i) When bolts are subjected to shear or tension

F_n is given by F_{nt} or F_{nv} in Table E3.4-1, as well as the ϕ values

ii) When bolts are subjected to a combination of shear and tension

F_n is given by F'_{nt} in Table E3.4-2, as well as the ϕ value

The pull-over resistance of the connected sheet at the bolt head, nut, or washer shall be considered where bolt tension is involved. See Section E6.2 of the *Specification*.

TABLE E3.4-1
Nominal Tensile and Shear Stresses for Bolts

Description of Bolts	Nominal Tensile Stress, F_{nt} (MPa)	Resistance Factor, ϕ	Nominal Shear Stress, F_{nv} (MPa)	Resistance Factor, ϕ
A307 Bolts, Grade A 6.4 mm ≤ d < 12.7 mm	279	0.65	165	0.55

TABLE E3.4-2
Nominal Tensile Stress for Bolts
Subjected to the Combination of Shear and Tension

Description of Bolts	Nominal Tensile Stress, F'_{nt} (MPa)	Resistance Factor, ϕ
A307 Bolts, Grade A When 6.4 mm ≤ d < 12.7 mm	$324 - 2.4f_v \le 279$	0.65

The actual shear stress, f_v, shall also satisfy Table E3.4-1 of this Appendix.

E4.3.2 Connection Shear Limited by End Distance

The nominal shear *resistance* per screw as affected by end distance in the direction of the applied force shall be calculated in accordance with the requirements of Section C2.2 of this Appendix. For spacing requirements, see Section E3.1 of this Appendix.

E5 Rupture

Shear rupture, tension rupture, and block shear rupture shall be determined in accordance with the requirements of Section C2.2 of this Appendix.