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North American Specification for the Design of Cold-Formed Steel Structural Members, 2012 Edition

American Iron and Steel Institute

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AISI STANDARD

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

2012 EDITION

Approved in Canada by the CSA Group

Endorsed in Mexico by CANACERO

CSA Group

CANACERO

The material contained herein has been developed by a joint effort of the American Iron and Steel Institute (AISI) Committee on Specifications, CSA Group 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 2007 and previous editions 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 S136, *Cold Formed Steel Structural Members,* published by CSA Group.

The *Specification* was developed by a joint effort of the American Iron and Steel Institute (AISI) Committee on Specifications, CSA 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 the CSA 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 Appendices 1 and 2, that is intended for use in all three countries, and two country-specific appendices (A and B). 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 *ASD* and *LRFD* 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." In the *Specification*, the terms that are specifically applicable to *LSD* are included in square brackets.

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 Group.

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 Group, and CANACERO acknowledge the continuing dedication of the members of the specifications committees and their subcommittees. The membership of these committees follows this Preface.

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

Materials

- Material standard ASTM A1063 is added.
- All referenced ASTM material standards are reorganized in accordance with the ranges of the minimum specified elongation.

Elements

- Section B1.3, Corner Radius-to-Thickness Ratios, is added, which limits the applicability of the design provisions in Chapter B to members with corner radius-to-*thickness* ratio not exceeding 10.
- Section B2.5, Uniformly Compressed Elements Restrained by Intermittent Connections, is added, which determines the *effective widths* of multiple flute built-up members.

Members

- Country-specific provisions on tension member design (Section C2) are unified and moved from Appendices A and B to the main body of the *Specification*.
- Revisions are made in Section C3.1.1, such that the *resistance factor* for bending is the same for stiffened, partially stiffened, or unstiffened compression *flanges*.
- The simplified provisions for determining *distortional buckling strength* of C- or Z-section beams (Section C3.1.4) and columns (Section C4.2) are moved to the *Commentary*.
- The reduction factor, as given in Section C3.6, for combined bending and torsional loading is revised.

Built-Up Section Members

- Clarifications are made to Section D1.1, Flexural Members Composed of Two Back-to-Back C-Sections.

Member Bracing

- Sections D3 and D3.1 are revised for clarifications.
- Section D3.3 is revised to be consistent with the AISC bracing design provisions. The *second-order analysis* is now permitted to determine the *required bracing strength*.

Wall Stud and Wall Stud Assemblies

- Reference to nonstructural members is removed from Section D4.
- Reference to AISI S213, *North American Cold-Formed Steel Framing Standard–Lateral*, is moved from Section D4 in Appendix A to the main body of the *Specification*.

Metal Roof and Wall System

- The following applicability requirements in Section D6.1.1 are revised or added: member depth, depth to *flange* width ratio, *flange* width, and ratio of *tensile strength* to design *yield stress*.
- Clarification is made to Section D6.2.1a regarding the application of the 0.67 factor

specifically to clips, fasteners and standing seam roof panels.

Connections

- The whole chapter is reorganized with the rupture check consolidated to Section E6. In addition, the following provisions are added or revised:
    - New provisions (Section E2.2.4) on combined shear and tension on arc spot welds are added.
    - New provisions (Section E2.4) on top arc seam sidelap welds are added.
    - Section E2.6, Flare Groove Welds, is revised to be consistent with the provisions in AWS D1.1-2006.
    - Section E3, Bolted Connections, is revised with added provisions for alternative short-slotted holes, applicable to *connections* where the deformation of the hole is not a consideration and the bolt diameter equals 1/2 in.
    - Table E3.4-1, Nominal Tensile and Shear Strengths for Bolts, in Appendix A is revised to be consistent with the values provided in ANSI/AISC 360.
    - New provisions (Section E4.5) are added for screw combined shear and pull-over, combined shear and pull out, and combined shear and tension in screws.
    - New provisions (Section E5) on power-actuated fasteners are added.
    - The reduction factor due to staggered hole patterns is eliminated in Section E6.

Tests

- Determination of *available strength* [*factored resistance*] by evaluation of a *rational engineering analysis* model via verification tests is added.

Appendix 1

- The geometric and material limitations of pre-qualified columns and beams for using the *safety* and *resistance factors* defined in Sections 1.2.1 and 1.2.2 are expanded.
- Provisions for determining the flexural and compressive strength of perforated members are added in Sections 1.2.1 and 1.2.2.1.
- Provisions for determining the *web* shear strength using the *Direct Strength Method* approach are added as Section 1.2.2.2.
- Provisions for considering beam or column reserve capacity are added in Section 1.2.2.1.

Appendix 2

- For braced members, the requirement to meet the specified maximum-out-of-straightness is added.

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

American Iron and Steel Institute
CSA Group
Camara Nacional de la Industria del Hierro y del Acero
November 2012

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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_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*	E6.2
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 cross-section	A1.3, C2.1, C4.2, E6.2, 1.2.1.1.1
A_{gv}	*Gross area* subject to shear	E6.3
A_{nt}	*Net area* subject to tension	E6.2, E6.3
A_{nv}	*Net area* subject to shear	E6.1, E6.3
A_n	*Net area* of cross-section	A1.3, C2.2
A_{net}	*Net area* of cross-section at the location of a hole	1.2.1.2.2
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, 1.2.2.2
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
a	Major diameter of the tapered *PAF* head	E5, E5.2.3
B_c	Term for determining tensile *yield stress* of corners	A7.2

SYMBOLS AND DEFINITIONS

Symbol	Definition	Section
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
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_t	Hat or trapezoid shape stiffener over all width	1.1.1.2
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, 1.1.1.1
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
C_w	Torsional warping constant of cross-section	C3.1.2.1

SYMBOLS AND DEFINITIONS

Symbol	Definition	Section
C_{wf}	Torsional warping constant of *flange*	C3.1.4, C4.2
C_y	Compression strain factor	C3.1.1
C_{yd}	Compression strain factor	1.2.2.1.3.2
$C_{y\ell}$	Compression strain factor	1.2.2.1.2.2
C_{yt}	Ratio of maximum tension strain to yield strain	1.2.2.1.2.2
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	C3.1.3, C4.1.5
D	Overall depth of lip	B1.1, B2.5, B4, 1.1.1.1, 1.1.1.2
D	Shear stiffener coefficient	C3.7.3
D	Dead load	A3.1, A6.1.2
DS	Width of *web* stiffener	1.1.1.2
D_2, D_3	Lip dimension	1.1.1.1, 1.1.1.2
d	Depth of section	B1.1, B2.5, C3.1.2.1, C3.4.1, C3.4.2, C3.7.2, D3.2.1, D6.1.1, D6.1.3, D6.1.4, D6.3.1, D6.3.2
d	Nominal screw diameter	E4, E4.1, E4.2, E4.3.1, E4.4.1, E4.5.1.1, E4.5.1.2, E4.5.2.1, E4.5.2.2
d	Flat depth of lip defined in Figure B4-1	B4
d	Visible width of arc seam weld	E2.3.1, E2.3.2.1, E2.3.2.2
d	Visible diameter of outer surface of arc spot weld	E2.2.1, E2.2.2.1, E2.2.2.2, E2.2.4
d	Nominal bolt diameter	E3, E3.1, E3.2, E3.3.1, E6.2
d	Fastener diameter measured at near side of embedment or d_s for *PAF* installed such that entire point is located behind far side of the embedment material	E5, E5.2.1, E5.3.1
d_a	Average diameter of arc spot weld at mid-thickness of t	E2.2.2.1, E2.2.2.2, E2.2.3, E2.2.4
d_a	Average width of seam weld	E2.3.2.1, E2.3.2.2

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Symbol	Definition	Section
d_{ae}	Average embedded diameter, computed as average of installed fastener diameters measured at near side and far side of embedment material or d_s for *PAF* installed such that entire point is located behind far side of embedment material	E5, E5.3.3
d_b	Nominal diameter (body or shank diameter)	G4
d_e	Effective diameter of fused area	E2.2, E2.2.2.1, E2.2.2.2, E2.2.3
d_e	*Effective width* of arc seam weld at fused surfaces	E2.3.2.1
d_h	Diameter of hole	B2.2, E6.1, E6.2
d_h	Depth of hole	B2.2, B2.4, C3.2.2, C3.4.2
d_h	Screw head diameter or hex washer head integral washer diameter	E4, E4.4, E4.4.2
$d_{p_{i,j}}$	Distance along roof slope between the ith *purlin* line and the jth anchorage device	D6.3.1
d_s	Reduced *effective width* of stiffener	B2.5, B4
d_s	Depth of stiffener	1.1.1.2
d_s	Nominal shank diameter	E5, E5.1, E5.2.3, E5.3.2, E5.3.3, E5.3.4, E5.3.4
d'_s	*Effective width* of stiffener calculated according to B3.1	B4
d_w	Steel washer diameter	E4, E4.4, E4.4.2
d_w	Larger value of screw head or washer diameter	E4.5.1.1, E4.5.1.2
d'_w	Effective pull-over resistance diameter	E4, E4.4.2
d'_w	Actual diameter of washer or fastener head in contact with retained substrate	E5, E5.2.3
d_1,d_2	Weld offset from flush condition	E2.6
E	Modulus of elasticity of steel, 29,500 ksi (203,000 MPa, or 2,070,000 kg/cm^2)	A2.3.2, A2.3.3, B1.1, B2.1, B2.5, B4, B5.1, C3.1.1, C3.1.2.1, C3.1.2.2, C3.1.3, C3.1.4, C3.2.1, C3.5.1, C3.5.2,C3.7.1, C3.7.3, C4.1.1, C4.1.5, C4.2, C5.2.1, C5.2.2, D1.3, D6.1.3, D6.3.1, E2.2.2.1, E5.3.3, 1.1.1.1, 1.1.1.2, 2.2.3
E	Live load due to earthquake	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	*Flat width* between first line of connector and edge stiffener	B2.5

SYMBOLS AND DEFINITIONS

Symbol	Definition	Section
e_{net}	Clear distance between end of material and edge of fastener hole or weld	E6.1
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_{bs}	Base *stress* parameter (66,00 psi (455 MPa))	E5, E5.2.1
F_{SR}	Design *stress* range	G3
F_{TH}	Threshold *fatigue stress* range	G1, G3, G4
F_c	Critical *buckling stress*	B2.1, B2.5, C3.1.2.1, C3.1.3
F_{cr}	Plate elastic *buckling stress*	A2.3.2, B2.1, B2.5, 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* of bolts	E3.4
F_{nt}	*Nominal tensile strength* of bolts	E3.4
F_{nv}	*Nominal shear strength* of bolts	E3.4
F'_{nt}	*Nominal tensile strength* for bolts subject to combination of shear and tension	E3.4
F_{sy}	*Yield stress* as specified in Section A2.1 or A2.2	A2.3.2, A2.3.3, A2.3.4, E2.4.1
F_u	*Tensile strength* as specified in Section A2.1 or A2.2	A2.3.2, A2.3.3, C2.2, 2.2.2.1, E2.2.2.2, E2.2.3, E2.2.4, E2.3.2.1, E2.3.2.2,E2.4.1, E2.6, E3.3.1, E3.3.2, E4.5.2.1, E4.5.2.2, E6.1, E6.2, E6.3
F_{uh}	*Tensile strength* of hardened *PAF* steel	E5, E5.2.1, E5.3.1
F_{uv}	*Tensile strength* of *virgin steel* specified by Section A2 or established in accordance with Section F3.3	A7.2
F_{ut}	*Tensile strength* of non-hardened *PAF* steel	E5
F_{u1}, F_{u2}	*Tensile strengths* of connected parts corresponding to *thicknesses* t_1 and t_2	E2.5
F_{u1}	*Tensile strength* of member in contact with screw head	E4, E4.3.1, E4.4.2, E4.5.1.1, E4.5.1.2
F_{u1}	*Tensile strength* of member in contact with *PAF* head or washer	E5, E5.2.3, E5.3.2

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Symbol	Definition	Section
F_{u2}	*Tensile strength* of member not in contact with screw head	E4, E4.3.1, E4.4.1, E4.5.2.1, E4.5.2.2
F_{u2}	*Tensile strength* of member not in contact with *PAF* head or washer	E5
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, E2.2.2.1, E2.2.2.2, E2.2.3, E2.2.4, E2.3.2.1, E2.3.2.2, E2.4.1, E2.5, E2.6
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	A2.3.3, A2.3.4, A7.1, A7.2, B2.1, B2.5, C2.1, C3.1.1, C3.1.2.1, C3.1.2.2, C3.1.3, C3.2.1, C3.4.1, C3.5.1, C3.5.2, C3.7.1, C3.7.2, C3.7.3, C4.1, C4.1.2, C4.1.5, C C4.2, 5.1.2, C5.1.1, C5.2.1, C5.2.2, D1.3, D6.1.1,D6.1.2, D6.1.4, E2.1, E2.2.4, E4.5.2.1, E4.5.2.2, E6.3, G1, 1.1.1.1, 1.1.1.2, 1.2.1.1.1, 1.2.1.2.2, 1.2.2.1.1.1.1, 1.2.2.1.1.2, 1.2.2.1.2.1.2, 1.2.2.2, 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
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_{y2}	*Yield stress* of t_2 sheet steel	E5, E5.3.3
f	*Stress* in compression element computed on basis of *effective design width*	B2.1, B2.2, B2.4, B2.5, B3.1, B3.2, B4, B5.1, B5.1.1, B5.1.2, B5.2
f′	*Stress* used in Section B4(a) for determining *effective width* of edge stiffener	B2.5
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
$f_{bending}$	Bending stress at location in cross section where combined bending and torsion *stress* is maximum	C3.6
$f_{bending\_max}$	Bending stress at extreme fiber, taken on same side of neutral axis as $f_{bending}$	C3.6

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Symbol	Definition	Section
$f_{torsion}$	Torsional warping stress at location in cross section where combined bending and torsion stress effect is maximum	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, B2.5, 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
GS	Center-to-center spacing of flat widths plus two interior stiffeners	1.1.1.2
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	E6.2
H	A *permanent load* due to lateral earth pressure, including groundwater	A3.1, A3.2
HRC_p	Rockwell C hardness of *PAF* steel	E5, E5.2.1
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, 1.2.2.2
h	Width of elements adjoining stiffened element	B5.1
h	Height of lip	E2.6
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

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Symbol	Definition	Section
h_s	Depth of soil supported by the structure	A6.1.2
h_{st}	Nominal seam height	E2.4.1
h_{wc}	Coped flat *web* depth	E6.1
h_{xf}	x distance from centroid of *flange* to *flange*/*web* junction	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, B2.5, 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*, using full unreduced section	C3.1.2.1
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

SYMBOLS AND DEFINITIONS

Symbol	Definition	Section
K	Effective length factor	C4.1.1, D1.2
K′	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
KL	Effective length	A2.3.2
k	Plate *buckling* coefficient	B2.1, B2.2, B2.3, B2.5, 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
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 *flange* to *flange*/*web* juncture	C3.1.4, C4.2
$\tilde{k}_{\phi fg}$	Geometric rotational stiffness demanded by *flange* from *flange*/*web* juncture	C3.1.4, C4.2
$k_{\phi we}$	Elastic rotational stiffness provided by *web* to *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	D1.1, D6.3.1, D6.3.2
L	Length of weld	E2.1, E2.6
L	Length of longitudinal weld or length of *connection*	E6.2
L	Length of seam weld not including circular ends	E2.3.2.1

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Symbol	Definition	Section
L	Length of fillet weld	E2.5
L	Unbraced length of member	C4.1.1, C5.2.1, C5.2.2, D1.2
L	Overall length	2.2.1
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 individual concentrically loaded compression member to be braced	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_{cr}	Critical unbraced length of *distortional buckling*	C3.1.4, C4.2
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_o	Overhang length measured from the edge of bearing to the end of member	C3.4.1
L_{st}	Length of bearing stiffener	C3.7.1
L_t	Unbraced length of compression member for torsion	C3.1.2.1
L_u	Limit of unbraced length below which lateral-torsional buckling is not considered	C3.1.2.2
L_w	Length of *top arc seam sidelap weld*	E2.4.1
L_x	Unbraced length of compression member for bending about x-axis	C3.1.2.1, C5.2.1, C5.2.2
L_y	Unbraced length of compression member for bending about y-axis	C3.1.2.1, C3.1.2.2, C5.2.1, C5.2.2
L_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
ℓ_{dp}	*PAF* point length	E5, E5.2.1, E5.2.2, E5.3.1, E5.3.2
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, 1.2.2.1.3.1.1, 1.2.2.1.3.1.2
M_{cre}	Overall buckling moment	1.1.2, 1.2.2.1, 1.2.2.1.1.1.1, 1.2.2.1.1.1.2, 1.2.2.1.1.2
$M_{cr\ell}$	*Local buckling* moment	1.1.2, 1.2.2.1.2.1.1, 1.2.2.1.2.2
M_d	Nominal moment with consideration of deflection	1.1.3

SYMBOLS AND DEFINITIONS

Symbol	Definition	Section
M_{d2}	*Nominal flexural strength* [*resistance*] of *distortional buckling* at λ_2	1.2.2.1.3.2
M_f	Moment due to *factored loads*	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, 1.2.2.1
M_{nd}	*Nominal flexural strength* [*resistance*] for *distortional buckling*	1.2.2.1, 1.2.2.3, 1.2.2.1.3.1.1, 1.2.2.1.3.1.2, 1.2.2.1.3.2
M_{ne}	*Nominal flexural strength* [*resistance*] for overall *buckling*	1.2.2.1, 1.2.2.1, 1.2.2.1.1, 1.2.2.1.1.1.1, 1.2.2.1.1.1.2, 1.2.2.1.1.2, 1.2.2.1.2.1.1, 1.2.2.1.2.2, 1.2.2.3
$M_{n\ell}$	*Nominal flexural strength* [*resistance*] for *local buckling*	1.2.2.1, 1.2.2.1.2, 1.2.2.1.2.1.1, 1.2.2.1.2.1.2, 1.2.2.1.2.2
$M_{n\ell o}$	*Nominal flexural strength* [*resistance*] for *local buckling* with M_{ne}=M_y	1.2.2.3
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, 1.2.2.3
M_{nxt}, M_{nyt}	*Nominal flexural strengths* [*resistances*] about centroidal axes determined using gross, unreduced cross-section properties	C5.1.1, C5.1.2
M_p	Plastic moment	1.2.2.1.1.2, 1.2.2.1.2.2, 1.2.2.1.3.2
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

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Symbol	Definition	Section
M_y	Moment causing maximum strain e_y	B2.1
M_y	Yield moment ($=S_fF_y$)	C3.1.4, 1.1.3, 1.2.2.1.1.1.1, 1.2.2.1.2.1.2, 1.2.2.1.2.2, 1.2.2.1.3.1.1, 1.2.2.1.3.1.2, 1.2.2.1.3.2, 1.2.2.3
M_{yc}	Moment at which yielding initiates in compression (after yielding in tension).	1.2.2.1.2.2, 1.2.2.1.3.2
M_{ynet}	Yield moment of net cross-section	1.2.2.1.2.1.2, 1.2.2.1.3.1.2
M_{yt3}	Yield moment at maximum tensile strain	1.2.2.1.2.1.2, 1.2.2.1.3.1.2
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* [moment due to *factored loads*]	C3.3.2, C3.5.2
$\overline{M}_x$, $\overline{M}_y$	*Required flexural strengths* [moments due to *factored loads*]	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 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 fasteners on critical cross-section	E6.1
n	Number of threads per inch	G4
n_b	Number of fasteners along failure path being analyzed	E6.1, E6.2
n_c	Number of compression *flange* stiffeners	1.1.1.2
n_w	Number of *web* stiffeners and/or folds	1.1.1.2

SYMBOLS AND DEFINITIONS

Symbol	Definition	Section
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 compressive axial strength* for *ASD*	A2.3.5, C5.2.1
P	Professional factor	F1.1, 1.1.1.1
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_{d2}	*Nominal axial strength* [*resistance*] of *distortional buckling* at λ_2	1.2.1.3.2
P_{Ex}, P_{Ey}	Elastic *buckling* strengths	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, 1.2.1.3.1
P_{cre}	Overall *buckling* load	1.1.2, 1.2.1.1.1, 1.2.1.1.2
$P_{cr\ell}$	*Local buckling* load	1.1.2, 1.2.1.2.1
P_f	Axial force due to *factored loads*	A2.3.5, C5.2.2
P_f	Concentrated load or reaction due to *factored loads*	C3.5.2
P_i	Lateral force introduced into system at ith *purlin*	D6.3.1
P_m	Mean value of tested-to-predicted load ratios	F1.1, 1.1.1.1
P_n	*Nominal web crippling strength* [*resistance*]	C3.4.1, C3.5.1, C3.5.2,
P_n	*Nominal axial strength* [*resistance*] of member	A2.3.5, C4.1, C4.2, C5.2.1, C5.2.2, 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, E2.2.2.1, E2.2.2.2, E2.2.3 E2.3.2.1, E2.3.2.2, E2.4.1, E2.5, E2.6, E2.7
P_n	*Nominal bearing strength* [*resistance*]	E3.3.1, E3.3.2
P_n	*Nominal bolt strength* [*resistance*]	E3.4
P_{nbp}	*Nominal bearing* and *tilting strength* [*resistance*] per *PAF*	E5, E5.3.2
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, 1.2.1.3.1, 1.2.1.3.2
P_{ne}	*Nominal axial strength* [*resistance*] for overall *buckling*	1.2.1, 1.2.1.1.1, 1.2.1.1.2, 1.2.1.2.1
$P_{n\ell}$	*Nominal axial strength* [*resistance*] for *local buckling*	1.2.1, 1.2.1.2.1, 1.2.1.2.2

SYMBOLS AND DEFINITIONS

Symbol	Definition	Section
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_{nos}	*Nominal pull-out strength* [*resistance*] in shear per *PAF*	E5, E5.3.3
P_{not}	*Nominal pull-out strength* [*resistance*] of sheet per screw	E4, E4.4.1, E4.5.2.1, E4.5.2.2
P_{not}	*Nominal pull-out strength* [*resistance*] in tension per *PAF*	E5
P_{nov}	*Nominal pull-over strength* [*resistance*] of sheet per screw	E4, E4.4.2, E4.5.1.1, E4.5.1.2
P_{nov}	*Nominal pull-over strength* [*resistance*] per *PAF*	E5
P_{ns}	*Nominal shear strength* [*resistance*] of sheet per screw	E4, E4.3.1, E4.5.1.1, E4.5.1.2, E4.5.2.1, E4.5.2.2
P_{ns}	*Nominal shear strength* [*resistance*] given by Section E2.2.2	E2.2.4.1, E2.2.4.2
P_{nsp}	*Nominal shear strength* [*resistance*] per *PAF*	E5, E5.3.1
P_{nt}	*Nominal tension strength*[*resistance*] given by Section E2.2.3	E2.2.4.1, E2.2.4.2
P_{ntp}	*Nominal tensile strength* [*resistance*] per *PAF*	E5, E5.2.1
P_{n1}, P_{n2}	*Nominal shear strength* [*resistance*] corresponding to connected *thicknesses* t_1 and t_2	E2.5
P_{ra}	*Required axial compressive strength* [compressive axial force due to *factored loads*] using *ASD*, *LRFD* or *LSD load combinations*	2.2.3
P_{ra}	*Required compressive axial strength* [compressive axial force due to *factored loads*] of individual concentrically loaded compression member to be braced, which is calculated in accordance with *ASD*, *LRFD*, or *LSD load combinations* depending on the design method used	D3.3
P_{rb}	*Required brace strength* [brace force due to *factored loads*] to brace a single compression member with an axial *load* P_{ra}	D3.3
P_s	Concentrated load or reaction	D1.1
P_{sp}	*Nominal shear strength* [*resistance*] of *PAF*	E5
P_{ss}	*Nominal shear strength* [*resistance*] of screw as reported by manufacturer or determined by independent laboratory testing	E4, E4.3.2, E4.5.3.1, E4.5.3.2
P_{tp}	*Nominal tensile strength* [*resistance*] of *PAF*	E5
P_{ts}	*Nominal tension strength* [*resistance*] of screws as reported by manufacturer or determined by independent laboratory testing	E4, E4.4.3, E4.5.3.1, E4.5.3.2
P_u	*Required axial strength* for *LRFD*	A2.3.5, C5.2.2
P_u	*Required strength* for concentrated load or reaction in presence of bending moment for *LRFD*	C3.5.2

SYMBOLS AND DEFINITIONS

Symbol	Definition	Section
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, 1.2.1.3.1, 1.2.1.3.2, 2.2.3
P_{ynet}	Member yield strength on net cross-section	1.2.1.2.2, 1.2.1.3.2
$\overline{P}$	*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* [compressive axial force due to *factored loads*]	C5.2.2
PAF	Power-actuated fasteners	E5, E5.1, E5.2, E5.2.1, E5.2.2, E5.2.3, E5.3, E5.3.1, E5.3.2, E5.3.3, E5.3.4, E5.4, E6
p	Pitch (mm per thread for SI units and cm per thread for MKS units)	G4
Q	*Required allowable shear strength* per *connection* fastener	E2.2.4.1, E4.5.1.1, E4.5.3.1
$\overline{Q}$	*Required shear strength* [shear force due to *factored loads*] per *connection* fastener	E2.2.4.2, E4.5.1.2, E4.5.3.2
Q_f	Shear force due to *factored loads* per *connection* fastener for *LSD*	E2.2.4.2
Q_i	*Load effect*	F1.1
Q_u	*Required shear strength* per *connection* fastener for *LRFD*	E2.2.4.2
q	*Design load* [factored load] on beam for determining longitudinal spacing of connections	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

SYMBOLS AND DEFINITIONS

Symbol	Definition	Section
R	Inside bend radius	A7.2, C3.4.1, C3.5.1, C3.5.2
R	Radius of outside bend surface	E2.6
R_I	I_s/I_a	B4
R_a	*Allowable design strength*	F1.2
R_b	Reduction factor	A2.3.3
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*]	E6.3
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_1, R_2	Radius of outside bend surface	E2.6
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
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	Stiffener spacing	1.1.1.2
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, 1.2.2.1.1.1.1
S_{fnet}	Net section modulus referenced to the extreme fiber in first yield	1.2.2.1.2.1.2
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 cross-section relative to extreme fiber in first yielding	C3.1.4

SYMBOLS AND DEFINITIONS

Symbol	Definition	Section
S_n	In-plane *diaphragm nominal shear strength* [*resistance*]	D5
s	Center-to-center hole spacing	B2.2
s	Center-to-center spacing of connectors in line of compression *stress*	B2.5
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	E6.2
s	Weld spacing	D1.1
s′	Longitudinal center-to-center spacing of any consecutive holes	E6.2
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* per *connection* fastener	E2.2.4.1, E4.5.1.1, E4.5.2.1, E4.5.3.1
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 per *connection* fastener for *LSD*	E2.2.4.2, E4.5.1.2, E4.5.3.2
T_n	*Nominal tensile strength* [*resistance*]	C2.1, C2.2, C5.1.1, C5.1.2
T_n	*Nominal tensile rupture strength* [*resistance*]	E6.2
T_r	*Required strength* [force due to *factored loads*] for *connection* in tension	D1.1
T_u	*Required tensile axial strength* for *LRFD*	C5.1.2
T_u	*Required tension strength* per *connection* fastener for *LRFD*	E2.2.4.2, E4.5.1.2
$\overline{T}$	*Required tensile axial strength* [tensile force due to *factored loads*]	C5.1.2
$\overline{T}$	*Required tension strength* [tensile force due to *factored loads*] per *connection* fastener	E2.2.4.2, E4.5.1.2, E4.5.2.2, E4.5.3.2
t	Base steel *thickness* of any element or section	A1.3, A2.3.3, A2.4, A7.2, B1.1, B1.2, B2.1, B2.2, B2.4, B2.5, B3.2, B4, B5.1, B5.1.1, B5.1.2, B5.2, 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,

SYMBOLS AND DEFINITIONS

Symbol	Definition	Section
		C4.1.5, C4.2, D1.3, D6.1.3, D6.1.4, D6.3.1, E3.3.1, E3.3.2, E6.1, E6.2,1.1.1.1, 1.1.1.2
t	*Thickness* of coped *web*	E6.1
t	Total *thickness* of two welded sheets	E2.2.2.1, E2.2.2.2, E2.2.3, E2.2.4, E2.3.2.1, E2.3.2.2
t	*Thickness* of thinner connected sheet	E2.4.1
t	*Thickness* of thinnest connected part	E2.5, E2.7
t	*Thickness* of flare-bevel groove welded member	E2.6
t_c	Lesser of depth of penetration and t_2	E4, E4.4.1, E4.5.2.1, E4.5.2.2
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	E2.5, E2.6
t_w	Steel washer thickness	E4.4.2, E5, E5.1
t_{wf}	Effective throat of groove weld that is filled flush to surface, determined in accordance with Table E2.6-1	E2.6
t_1	*Thickness* of member in contact with screw head	E4, E4.3.1, E4.4, E4.4.2, E4.5.1.1, E4.5.1.2
t_1	*Thickness* of member in contact with *PAF* head or washer	E5, E5.2.3, E5.3.2
t_2	*Thickness* of member not in contact with screw head	E4, E4.3.1, E4.5.1.1, E4.5.1.2, E4.5.2.1, E4.5.2.2
t_2	*Thickness* of member not in contact with *PAF* head or washer	E5, E5.3.2, E5.3.3
t_1, t_2	Based *thicknesses* connected with fillet weld	E2.5
U_{bs}	Non-uniform block shear factor	E6.3
U_{s1}	Shear lag factor determined in Table E6.2-1	E6.2
V	*Required allowable shear strength* for *ASD*	C3.3.1
V_{cr}	Shear buckling load	1.1.2, 1.2.2.2
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	Shear force due to *factored loads* per *connection* fastener for *LSD*	E4.5.1.2, E4.5.3.2
V_M	Coefficient of variation of material factor	D6.2.1, F1.1
V_n	*Nominal shear strength* [*resistance*]	C3.2.1, C3.3.1, C3.3.2, E6.1, 1.1.1, 1.2.2.2, 1.2.2.3

SYMBOLS AND DEFINITIONS

Symbol	Definition	Section
V_P	Coefficient of variation of tested-to-predicted load ratios	D6.2.1, F1.1, 1.1.1.1
V_Q	Coefficient of variation of *load effect*	D6.2.1, F1.1
V_y	Yield shear force of section	1.2.2.2
V_u	*Required shear strength* for *LRFD*	C3.3.2
V_u	*Required shear strength* per *connection* fastener for *LRFD*	E4.5.1.2, E4.5.3.2
$\overline{V}$	*Required shear strength* [shear force due to *factored loads*]	C3.3.2
W	Wind load, a *variable load* due to wind	A3.1, A6.1.2, A6.1.2.1
W	*Required strength* [*factored load*] 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* [*factored load*] W	D3.2.1
WS	Depth of stiffeners	1.1.1.2
w	*Flat width* of element exclusive of radii	A2.3.3, B1.1, B2.1, B2.2, B3.1, B3.2, B4, C3.1.1, C3.7.1
w	*Flat width* of element measured between longitudinal connection lines and exclusive of radii at stiffeners	B2.5
w′	Equivalent *flat width* for determining *effective width* of edge stiffener	B2.5
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_f	Face width of weld	E2.6
w_i	Required distributed gravity load supported by ith *purlin* per unit length	D6.3.1
w_o	Out-to-out width	B2.2
w_1	Leg of weld	E2.5
w_1	Transverse spacing between first and second line of fasteners in compression element	B2.5
w_2	Leg of weld	E2.5
x	Non-dimensional fastener location	D6.1.3
x	Nearest distance between *web* hole and edge of bearing	C3.4.2

SYMBOLS AND DEFINITIONS

Symbol	Definition	Section
x_o	Distance from shear center to centroid along principal x-axis	C3.1.2.1, C4.1.2
x_{of}	x distance from centroid of *flange* to shear center of *flange*	C3.1.4, C4.2
$\bar{x}$	Distance from shear plane to centroid of cross-section	E6.2
Y	Yield stress of *web* steel divided by yield stress 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_{of}	y distance from centroid of *flange* to shear center of *flange*	C3.1.4
Z_f	Plastic section modulus	1.2.2.1.1.1.2
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 increase due to overhang	C3.4.1
α	Second-order amplification coefficient	2.2.3
α	Coefficient	E5.2.3
α_b	Coefficient	E5.3.2
l/α_x, l/α_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
β_o	Target reliability index	D6.2.1, F1.1
β_{rb}	Minimum required brace stiffness to brace a single compression member	D3.3
γ, γ_i,	Coefficients	B5.1.1, B5.1.2
γ_i	*Load factor*	F1.1
δ, δ_i,	Coefficients	B5.1.1, B5.1.2
η	Variable	E2.6
θ	Angle between vertical and plane of *web* of Z-section, degrees	D6.3.1
θ	Angle between an element and its edge stiffener	B4, 1.1.1.1, 1.1.1.2

SYMBOLS AND DEFINITIONS

Symbol	Definition	Section
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, B2.5, B3.2, B5.1, C3.1.1, C4.1, 1.2.1.1.1
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, 1.2.2.1.2.1.1, 1.2.2.1.2.2
λ_d	Slenderness factor of column or beam	C3.1.4, C4.2, 1.2.1.3.1, 1.2.1.3.2, 1.2.2.1.3
λ_{d1}, λ_{d2}	Slenderness factors of column or beam	1.2.1.3.2, 1.2.2.1.3
λ_t	Slenderness factor	B2.5
λ_v	Slenderness factor	1.2.2.2
μ	Poisson's ratio of steel = 0.30	B2.1, C3.2.1, C3.1.4, C4.2
ξ_{web}	*Stress* gradient in *web*	C3.1.4
ρ	Reduction factor	A7.2, B2.1, B2.5, B3.2, B5.1, F3.1
ρ_m	Reduction factor	B2.5
ρ_t	Reduction factor	B2.5
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
τ_b	Parameter for reduced stiffness using second-order analysis	2.2.3
ϕ	*Resistance factor*	A1.2, A1.3, A5.1.1, A6.1.1, D6.2.1, C3.5.2, C3.7.2, D3.3, D6.1.3, D6.3.1, D6.3.2, E2.1, E2.2.2.1, E2.2.2.2, E2.2.3, E2.3.2.1, E2.3.2.2, E2.4.1, E2.5, E2.6, E2.7, E3.3.1, E3.3.2, E3.4, E4, E4.3.2, E4.4.3,

SYMBOLS AND DEFINITIONS

Symbol	Definition	Section
		E4.5.1.2, E4.5.2.2, E4.5.3.2, E5.2.1, E5.2.2, E5.2.3, E5.3.1, E5.3.2, E5.3.3, E6, F1.1, F1.2, 1.1.1, 1.1.1.1, 1.1.1.2, 1.2.1, 1.2.2.1
ϕ_b	*Resistance factor* for bending strength	C3.1, C3.1.1, C3.1.2, C3.1.3, C3.1.4, C3.3.2, C3.5.2, C5.1.2, C5.2.2, D6.1.1, D6.1.2, 1.2.2.1
ϕ_c	*Resistance factor* for concentrically loaded compression strength	A2.3.5, C3.7.1, C4.1, C4.2, C5.2.2, 1.2.1
ϕ_d	*Resistance factor* for *diaphragms*	D5
ϕ_s	*Resistance factor* for shear strength	E2.2.4.2
ϕ_t	*Resistance factor* for tension strength	C2.1, C2.2, C5.1.2, E2.2.4.2
ϕ_u	*Resistance factor* for *rupture*	E6
ϕ_v	*Resistance factor* for shear strength	C3.2.1, C3.3.2, 1.2.2.2
ϕ_w	*Resistance factor* for *web crippling* strength	C3.4.1, C3.5.2
ω_i	Coefficient	B5.1.2
ψ	$\lvert f_2/f_1 \rvert$	B2.3, B3.2, C3.1.1
Δ_{tf}	Lateral displacement of *purlin* top *flange* at the line of restraint	D6.3.1
Ω	*Safety factor*	A1.2, A1.3, A4.1.1, D6.2.1, C3.5.1, C3.7.2, D3.3, D6.1.3, D6.3.1, D6.3.2, E2.1, E2.2.2.1, E2.2.2.2, E2.3.2.1, E2.3.2.2, E2.4.1, E2.5, E2.6, E2.7, E3.3.1, E3.3.2, E3.4, E4, E4.3.2, E4.4.3, E4.5.1.1, E4.5.2.1, E4.5.3.1, E5.2.1, E5.2.2, E5.2.3, E5.3.1, E5.3.2, E5.3.3, E6, F1.2, 1.1.1, 1.1.1.1, 1.1.1.2, 1.2.1, 1.2.2.1
Ω_b	*Safety factor* for bending strength	C3.1, C3.1.1, C3.1.2, C3.1.3, C3.1.4, C3.3.1, C3.5.1, C5.1.1, C5.2.1, D6.1.1, D6.1.2, 1.2.2.1
Ω_c	*Safety factor* for concentrically loaded compression	A2.3.5, C4.1, C4.2, C5.2.1, 1.2.1

SYMBOLS AND DEFINITIONS

Symbol	Definition	Section
	strength	
Ω_c	*Safety factor* for *bearing* strength	C3.7.1
Ω_d	*Safety factor* for *diaphragms*	D5
Ω_s	*Safety factor* for shear strength	E2.2.4.1
Ω_t	*Safety factor* for tension strength	C2.1, C2.2, C5.1.1, E2.2.4.1
Ω_v	*Safety factor* for shear strength	C3.2.1, C3.3.1, 1.2.2.2
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

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.

The symbol ☞x is used to point out that additional provisions that are specific to a certain country are provided in the corresponding appendices indicated by the letter(s) "x."

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

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

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

The *nominal strength* [*resistance*] and stiffness of cold-formed steel components such as elements, members, assemblies, *connections*, and details shall be determined in accordance with the provisions in Chapters B through G, Appendices A and B, and Appendices 1 and 2 of the *Specification*.

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

(a) *Available strength* [*factored resistance*] or stiffness by tests only. Specifically, the *available strength* [*factored resistance*] is determined from tested *nominal strength* [*resistance*] by applying the *safety factors* or the *resistance factors* evaluated in accordance with Section F1.1(a);
(b) *Available strength* [*factored resistance*] by *rational engineering analysis* with verification tests. Specifically, the *available strength* [*factored resistance*] is determined from the calculated *nominal strength* [*resistance*] by applying the *safety factors* or *resistance factors* evaluated in

accordance with Section F1.1(b);

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

For members

Ω = 2.00 (*ASD*)
ϕ = 0.80 (*LRFD*)
= 0.75 (*LSD*)

For *connections*

Ω = 2.50 (*ASD*)
ϕ = 0.65 (*LRFD*)
= 0.60 (*LSD*)

When *rational engineering analysis* is used in accordance with A1.2(b) or A1.2(c) to determine the *nominal strength* [*resistance*] for a *limit state* already provided in this *Specification*, the *safety factor* shall not be less than the applicable *safety factor* (Ω), nor shall the *resistance factor* exceed the applicable *resistance factor* (ϕ) for the prescribed *limit state*.

A1.3 Definitions

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

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

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

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

General Terms

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

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

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

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

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

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

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

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

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 width*s 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 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*.

Power-Actuated Fasteners (PAFs). Hardened steel fasteners driven through steel members into embedment material using either powder cartridges or compressed gas as the energy-driving source.

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

Published Specification. Requirements for a steel listed by a manufacturer, processor, producer, purchaser, or other body, which (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* [*resistance*] from the actual strength and for the manner and consequences of failure.

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

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

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

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 Steel Structural Member.

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.

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

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

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

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

A2 Material

A2.1 Applicable Steels

This *Specification* requires the use of steels intended for structural applications as defined in general by the specifications of ASTM International listed in this section. The term *SS* designates structural steels and the terms HSLAS and HSLAS-F designate high-strength low-alloy steels. Applicable steels have been grouped by their minimum elongation requirements over a two-inch (50-mm) gage length.

A2.1.1 Steels With a Specified Minimum Elongation of Ten Percent or Greater (Elongation ≥ 10%)

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, 55 (380) and 60 (410); 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*

Exception: SS Grade 60 (410) with *thicknesses* less than or equal to 0.028 in. (0.71 mm) is excluded from this elongation group.

ASTM A792/A792M (Grades 33 (230), 37 (255), 40 (275), 50 (340) Class 1 and Class 4, and 60 (410)), *Standard Specification for Steel Sheet, 55% Aluminum-Zinc Alloy-Coated by the Hot-Dip Process*

Exception: Grade 60 (410) with *thicknesses* less than or equal to 0.028 in. (0.71 mm) is excluded from this elongation group.

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); HSLAS Classes 1 and 2, Grades 45 (310), 50 (340), 55 (380), 60 (410), and 65 (450)), *Standard Specification for Steel, Sheet, Hot-Rolled, Carbon, Commercial and Structural, Produced by the Twin-Roll Casting Process*

Exception: SS Grades 55 (380), 60 (410), 70 (480), and 80 (550) with *thicknesses* outside the range of 0.064 in. (1.6 mm) to 0.078 in. (2.0 mm) are excluded from this elongation group.

ASTM A1063/A1063M (SS Grades 40 (275), 50 (340); HSLAS Classes 1 and 2, Grades 45 (310), 50 (340), 55 (380), 60 (410), and 65 (450)), *Standard Specification for Steel Sheet, Twin-Roll Cast, Zinc-Coated (Galvanized) by the Hot-Dip Process*

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A2.1.2 Steels With a Specified Minimum Elongation From Three Percent to Less Than Ten Percent (3% ≤ Elongation < 10%)

ASTM A653/A653M (SS Grades 60 (410), 70 (480) and 80 (550) Class 3), *Standard Specification for Steel Sheet, Zinc-Coated (Galvanized) or Zinc-Iron Alloy-Coated (Galvannealed) by the Hot-Dip Process*

Exception: SS Grade 60 (410) with *thicknesses* greater than 0.028 in. (0.71 mm) is excluded from this elongation group.

ASTM A792/A792M (Grades 60 (410), 70 (480), and 80 (550) Class 3), *Standard Specification for Steel Sheet, 55% Aluminum-Zinc Alloy-Coated by the Hot-Dip Process*

Exception: Grade 60 (410) with *thicknesses* greater than 0.028 in. (0.71 mm) is excluded from this elongation group.

ASTM A1039/A1039M (SS Grades 55 (380), 60 (410), 70 (480), and 80 (550); HSLAS Classes 1 and 2, Grades 70 (480) and 80 (550)), *Standard Specification for Steel, Sheet, Hot Rolled, Carbon, Commercial and Structural, Produced by the Twin-Roll Casting Process*

Exception: SS grades with *thicknesses* greater than or equal to 0.064 in. (1.6 mm) are excluded from this elongation group.

ASTM A1063/A1063M (SS Grades 55 (380), 60 (410), 70 (480), Grade 80 (550) Class 1); (HSLAS Grade 70 (480) Classes 1 and 2, Grade 80 (550) Classes 1 and 2), *Standard Specification for Steel Sheet, Twin-Roll Cast, Zinc-Coated (Galvanized) by the Hot-Dip Process*

A2.1.3 Steels With a Specified Minimum Elongation of Less Than Three Percent (Elongation < 3%)

ASTM A653/A653M (SS Grade 80 (550) Classes 1 and 2), *Standard Specification for Steel Sheet, Zinc-Coated (Galvanized) or Zinc-Iron Alloy-Coated (Galvannealed) by the Hot-Dip Process*

ASTM A792/A792M (Grade 80 (550) Classes 1 and 2), *Standard Specification for Steel Sheet, 55% Aluminum-Zinc Alloy-Coated by the Hot-Dip Process*

ASTM A875/A875M (SS Grade 80 (550)), *Standard Specification for Steel Sheet, Zinc-5% Aluminum Alloy-Coated by the Hot-Dip Process*

ASTM A1008/A1008M (SS Grade 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 A1063/A1063M (SS Grade 80 (550) Class 2), *Standard Specification for Steel Sheet,* *Twin-Roll Cast, Zinc-Coated (Galvanized) by the Hot-Dip Process*

A2.2 Other Steels

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

A2.3 Permitted Uses and Restrictions of Applicable Steels

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

Exception: For steels used in composite slabs for the condition where the steel deck acts as the tensile reinforcement of the slabs, Section A2.3.4 shall be followed exclusively.

A2.3.1 Steels With a Specified Minimum Elongation of Ten Percent or Greater (Elongation ≥ 10%)

Steel grades listed in Section A2.1.1, as well as any other *SS* steel, are permitted to be used without restriction under the provisions of this *Specification* provided:

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

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

A2.3.2 Steels With a Specified Minimum Elongation From Three Percent to Less Than Ten Percent (3% ≤ Elongation < 10%)

Steel grades listed in Section A2.1.2, as well as any other *SS* steel that has a minimum elongation of 3 percent in a two-inch (50-mm) gage length standard specimen tested in accordance with ASTM A370 or ASTM A1058, are permitted to be used provided that the *available strengths* [*factored resistances*] of *structural members* and *connections* are calculated in accordance with Chapters B through G (excluding welded *connections* in Chapter E), Appendices A and B, and Appendices 1 and 2. For the purposes of these calculations, a reduced *yield stress* 0.9 F_{sy} shall be used in place of F_{sy}, and a reduced *tensile strength* of 0.9 F_u shall be used in place of F_u. ☜A,B

For concentrically loaded compression members with a closed box section, 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 1.1 L_0, where L_0 is given by Eq. A2.3.2-1, and R_r is given by Eq. A2.3.2-2.

$$L_0 = \pi r\sqrt{\frac{E}{F_{cr}}} \qquad (Eq.\ A2.3.2\text{-}1)$$

$$R_r = 0.65 + \frac{0.35(KL)}{1.1L_0} \qquad (Eq.\ A2.3.2\text{-}2)$$

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

A2.3.3 Steels With a Specified Minimum Elongation Less than Three Percent (Elongation < 3%)

Steel grades listed in Section A2.1.3, as well as other steel grades that do not meet the requirements of A2.3.1 or A2.3.2, are permitted to be used only for multiple *web* configurations such as roofing, siding, and floor decking provided the following adjustments are made to the design parameters:

(a) A reduced *specified minimum yield stress*, R_bF_{sy}, is used for determining the *nominal flexural strength* [*resistance*] in Section C3.1.1(a), for which the reduction factor, R_b, is determined in accordance with (1) or (2):

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

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

$R_b = 1.0$

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

$$R_b = 1 - 0.26[wF_{sy}/(tE) - 0.067]^{0.4} \qquad (Eq.\ A2.3.3\text{-}1)$$

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

$R_b = 0.75$

(2) For unstiffened compression *flanges*

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

$R_b = 1.0$

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

$$R_b = 1.079 - 0.6\sqrt{wF_{sy}/(tE)} \qquad (Eq.\ A2.3.3\text{-}2)$$

where

w = *Flat width* of compression *flange*

t = *Thickness* of section

E = Modulus of elasticity of steel

F_{sy} = *Specified minimum yield stress* determined in accordance with Section A7.1 ≤ 80 ksi (550 MPa, or 5620 kg/cm^2)

(b) The *yield stress*, F_y, used for determining *nominal strength* [*resistance*] in Chapters B, C, D, and E exclusive of Section C3.1.1(a) is taken as 75 percent of the *specified minimum yield stress* or 60 ksi (414 MPa or 4220 kg/cm^2), whichever is less, and

(c) The *tensile strength*, F_u, used for determining *nominal strength* [*resistance*] in Chapter 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.

A2.3.4 Steel Deck as Tensile Reinforcement for Composite Deck-Slabs

For steels used in composite slabs for the condition where the steel deck acts as the tensile reinforcement of the slab, the following requirements shall be followed:

(a) If the ductility of the steel measured over a two-inch (50-mm) gage length is 10 percent or greater:
33 ksi (228 MPa or 2320 kg/cm^2) ≤ F_y ≤ 50 ksi (345 MPa or 3520 kg/cm^2) or F_{sy}, whichever is smaller.

(b) If the ductility of the steel measured over a two-inch (50-mm) gage length is less than 10 percent:
33 ksi (228 MPa or 2320 kg/cm^2) ≤ F_y ≤ 50 ksi (345 MPa or 3520 kg/cm^2) or 0.75 F_{sy}, whichever is smaller.

In addition, the ability of the steel to be formed without cracking or splitting shall be demonstrated.

A2.3.5 Ductility Requirements of Other Steels

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

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

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

When material ductility is determined on the basis of these criteria, the use of such material shall be restricted to the design of *purlins, girts*, and *curtain wall studs* in accordance with Sections C3.1.1(a), C3.1.2, D6.1.1, D6.1.2, D6.2.1, and country-specific requirements given in A2.3.5a of 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

A2.4 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 *thickness* is 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 *ASD load combinations*.

The design shall be performed in accordance with Eq. 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 *LRFD load combinations*.

The design shall be performed in accordance with Eq. 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 Eq. 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; as modified by Sections A2.3.2, A2.3.3, or A2.3.4, as appropriate; 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 is permitted by substituting F_{ya} for F_y, where F_{ya} is the average *yield stress* of the full section. Such increase shall be limited to 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} = $B_cF_{yv}/(R/t)^m$, 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 $\leq 120^o$

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), 25 Massachusetts Avenue, NW, Suite 800, Washington, DC 20001:

   AISI S200-12, *North American Standard for Cold-Formed Steel Framing – General Provisions*

   AISI S210-07(2012), *North American Standard for Cold-Formed Steel Framing – Floor and Roof System Design* (Reaffirmed 2012)

   AISI S211-07/wS1-12, *North American Standard for Cold-Formed Steel Framing – Wall Stud Design* with Supplement 1 (Reaffirmed 2012)

   AISI S212-07(2012), *North American Standard for Cold-Formed Steel Framing – Header Design* (Reaffirmed 2012)

   AISI S213-07/wS1-09(2012), *North American Standard for Cold-Formed Steel Framing – Lateral Design* with Supplement No. 1 (Reaffirmed 2012)

AISI S214-12, *North American Standard for Cold-Formed Steel Framing – Truss Design*

AISI S901-08, *Rotational-Lateral Stiffness Test Method for Beam-to-Panel Assemblies*

AISI S902-08, *Stub-Column Test Method for Effective Area of Cold-Formed Steel Columns*

AISI S906-08, *Standard Procedures for Panel and Anchor Structural Tests*

2. American Society of Mechanical Engineers (ASME), Three Park Avenue, New York, NY 10016:

ASME B46.1-2009, *Surface Texture, Surface Roughness, Waviness, and Lay*

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

ASTM A36/A36M-08, *Standard Specification for Carbon Structural Steel*

ASTM A194/A194M-11, *Standard Specification for Carbon and Alloy Steel Nuts for Bolts for High-Pressure and High-Temperature Service, or Both*

ASTM A242/A242M-04(2009), *Standard Specification for High-Strength Low-Alloy Structural Steel*

ASTM A283/A283M-03(2007), *Standard Specification for Low and Intermediate Tensile Strength Carbon Steel Plates*

ASTM A307-10, *Standard Specification for Carbon Steel Bolts and Studs, 60,000 PSI Tensile Strength*

ASTM A325-10, *Standard Specification for Structural Bolts, Steel, Heat Treated, 120/105 ksi Minimum Tensile Strength*

ASTM A325M-09, *Standard Specification for Structural Bolts, Steel, Heat Treated, 830 MPa Minimum Tensile Strength [Metric]*

ASTM A354-11, *Standard Specification for Quenched and Tempered Alloy Steel Bolts, Studs, and Other Externally Threaded Fasteners*

ASTM A370-12, *Standard Specification for Standard Test Methods and Definitions for Mechanical Testing of Steel Products*

ASTM A449-10, *Standard Specification for Hex Cap Screws, Bolts, and Studs, Steel, Heat Treated, 120/105/90 ksi Minimum Tensile Strength, General Use*

ASTM A490-12, *Standard Specification for Structural Bolts, Alloy Steel, Heat Treated, 150 ksi Minimum Tensile Strength*

ASTM A490M-12, *Standard Specification for High-Strength Steel Bolts, Classes 10.9 and 10.9.3, for Structural Steel Joints [Metric]*

ASTM A500/A500M-10a, *Standard Specification for Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes*

ASTM A529/A529M-05(2009), *Standard Specification for High-Strength Carbon-Manganese Steel of Structural Quality*

ASTM A563-07a, *Standard Specification for Carbon and Alloy Steel Nuts*

ASTM A563M-07, *Standard Specification for Carbon and Alloy Steel Nuts [Metric]*

ASTM A572/A572M12, *Standard Specification for High-Strength Low-Alloy Columbium-Vanadium Structural Steel*

ASTM A588/A588M-10, *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/A606M-09a, *Standard Specification for Steel, Sheet and Strip, High-Strength, Low-Alloy, Hot-Rolled and Cold-Rolled, With Improved Atmospheric Corrosion Resistance*

ASTM A653/A653M-11, *Standard Specification for Steel Sheet, Zinc-Coated (Galvanized) or Zinc-Iron Alloy-Coated (Galvannealed) by the Hot-Dip Process*

ASTM A792/A792M-10, *Standard Specification for Steel Sheet, 55% Aluminum-Zinc Alloy-Coated by the Hot-Dip Process*

ASTM A847/A847M-11, *Standard Specification for Cold-Formed Welded and Seamless High-Strength, Low-Alloy Structural Tubing With Improved Atmospheric Corrosion Resistance*

ASTM A875/A875M-10, *Standard Specification for Steel Sheet, Zinc-5% Aluminum Alloy-Coated by the Hot-Dip Process*

ASTM A1003/A1003M-12, *Standard Specification for Steel Sheet, Carbon, Metallic- and Nonmetallic-Coated for Cold-Formed Framing Members*

ASTM A1008/A1008M-12, *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-12, *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-12, *Standard Specification for Steel, Sheet, Hot-Rolled, Carbon, Commercial and Structural, Produced by the Twin-Roll Casting Process*

ASTM A1058-12, *Standard Test Methods for Mechanical Testing of Steel Products – Metric*

ASTM A1063/A1063M-11a, *Standard Specification for Steel Sheet, Twin-Roll Cast, Zinc-Coated (Galvanized) by the Hot-Dip Process*

ASTM E1592-12, *Standard Test Method for Structural Performance of Sheet Metal Roof and Siding Systems by Uniform Static Air Pressure Difference*

ASTM F436-11, *Standard Specification for Hardened Steel Washers*

ASTM F436M-11, *Standard Specification for Hardened Steel Washers [Metric]*

ASTM F844-07a, *Standard Specification for Washers, Steel, Plain (Flat), Unhardened for General Use*

ASTM F959-09, *Standard Specification for Compressible Washer-Type Direct Tension Indicators for Use With Structural Fasteners*

ASTM F959M-07, *Standard Specification for Compressible Washer-Type Direct Tension Indicators for Use With Structural Fasteners [Metric]*

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

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

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

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

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, and 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.

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 is permitted to be applied to compression and tension *flanges*, either stiffened or unstiffened, as follows:

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

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 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

B1.3 Corner Radius-to-Thickness Ratios

The *effective width* provisions of Chapter B shall apply to sections with inside bend radius-to-*thickness* ratios no larger than 10. For inside bend radius-to-*thickness* ratios larger than 10, rational analysis is permitted.

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:

$b = w \quad \text{when } \lambda \le 0.673$ (*Eq*. B2.1-1)

$b = \rho w \quad \text{when } \lambda > 0.673$ (*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 which is 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 yielding).

(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:

$b_d = w \quad \text{when } \lambda \le 0.673$ (*Eq*. B2.1-6)

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

where

w = *Flat width*

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

(1) Procedure I:

A conservative estimate of the *effective width* is obtained from 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 $\lambda \le 0.673$

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

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

$\rho \le 1$ for all cases.

where

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

$$\lambda_c = 0.256 + 0.328\,(w/t)\sqrt{F_y/E} \quad (Eq.\ B2.1\text{-}10)$$

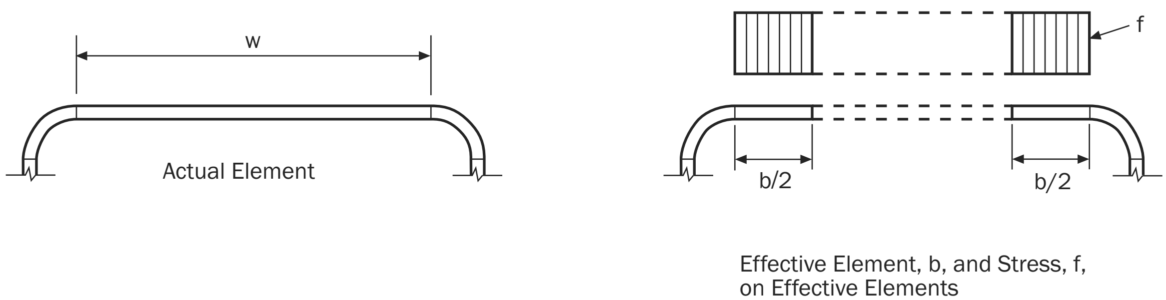


Figure B2.1-1 Stiffened Elements

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 $0.50 \ge \dfrac{d_h}{w} \ge 0$, and $\dfrac{w}{t} \le 70$, and

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

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

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

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

where

w = *Flat width*

t = *Thickness* of element

d_h = Diameter of holes

λ = Slenderness factor as defined in Section B2.1 with k = 4.0

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

Alternatively, the *effective width*, b, is 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.

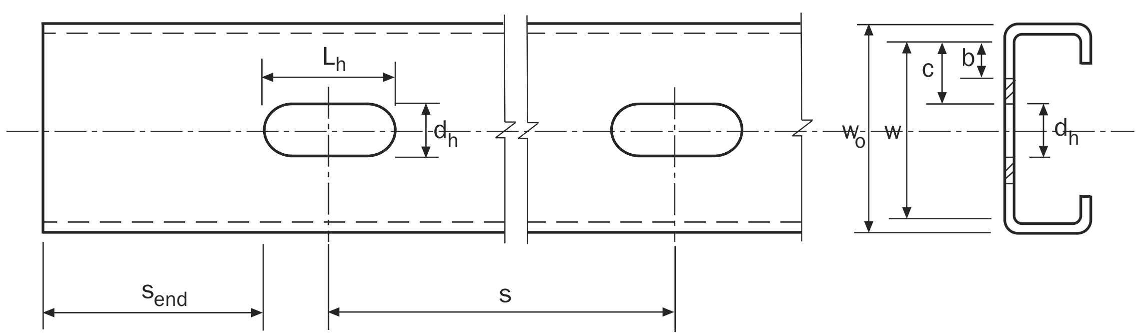


Figure B2.2-1 Uniformly Compressed Stiffened Elements With Non-Circular Holes

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

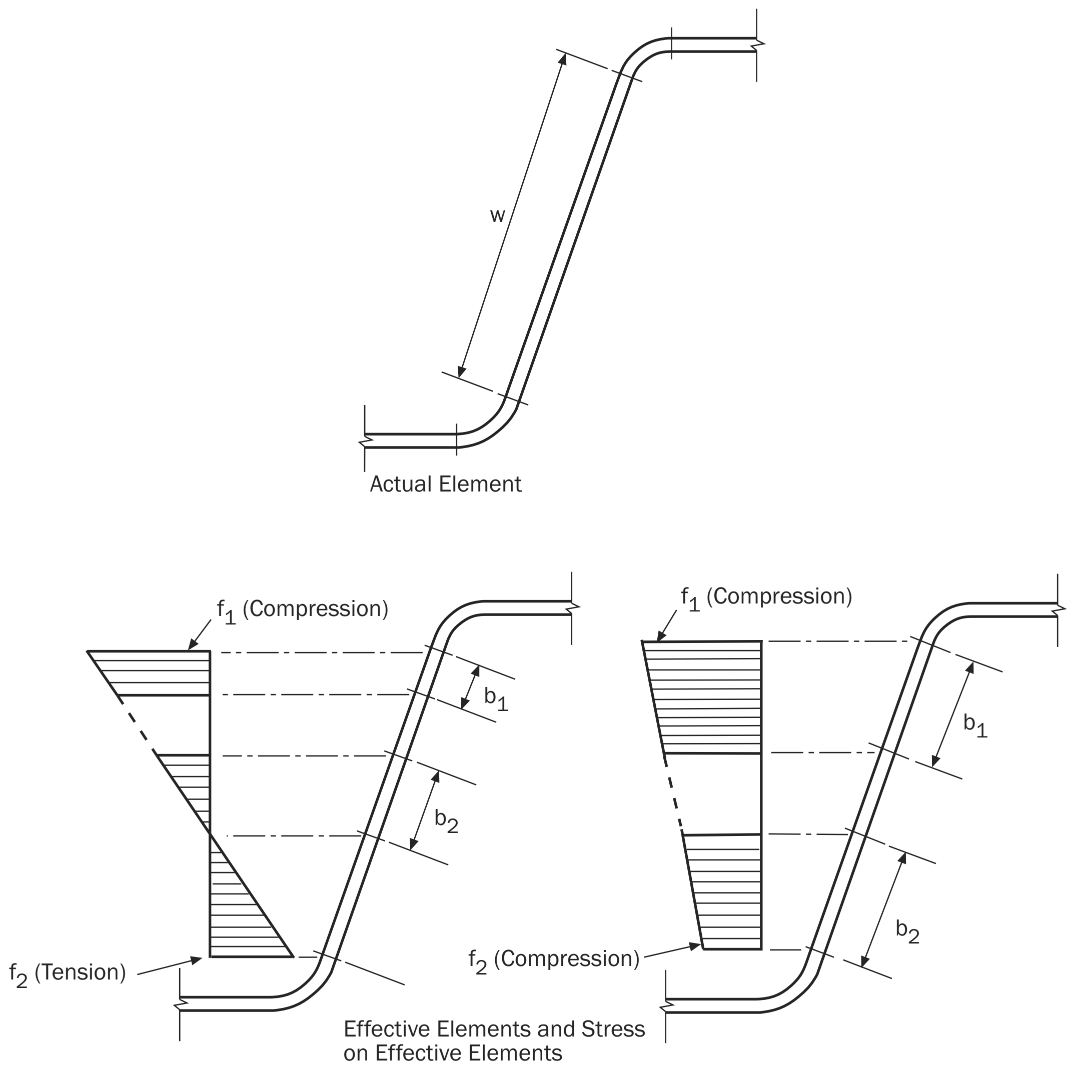
(1) For *webs* under *stress* gradient (f_1 in compression and f_2 in tension as shown in Figure B2.3-1(a)), the *effective widths* and plate *buckling* coefficient shall be calculated as follows:

$k = 4 + 2(1 + \psi)^3 + 2(1 + \psi)$ (*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 $\psi \le 0.236$ (Eq. B2.3-5)

In addition, $b_1 + b_2$ shall not exceed the compression portion of the *web* calculated on the basis of effective section.

For $h_o/b_o > 4$

$b_1 = b_e/(3 + \psi)$ (*Eq.* B2.3-6)

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

(2) 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.

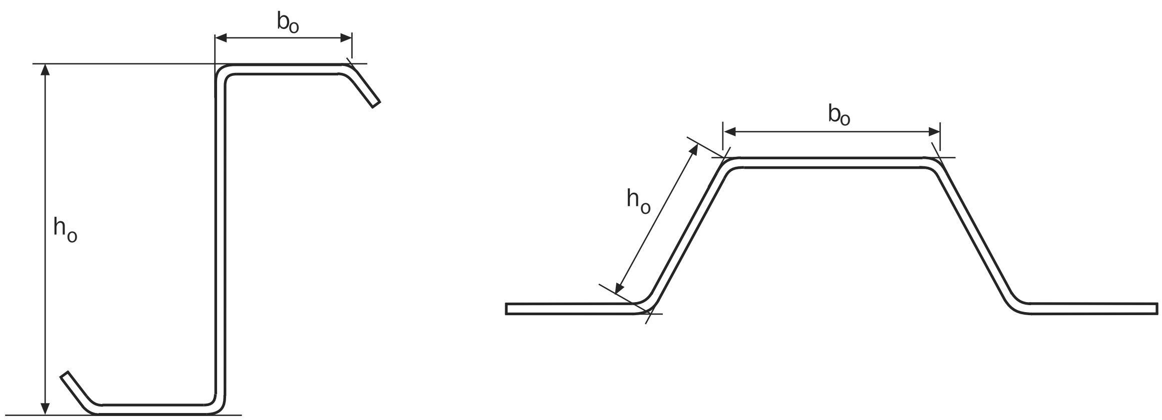


Figure B2.3-2 Out-to-Out Dimensions of Webs and Stiffened Elements Under Stress Gradient

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. (63.5 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. (14.3 mm).

where

d_h = Depth of *web* hole

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

B2.5 Uniformly Compressed Elements Restrained by Intermittent Connections

The provisions of this section shall apply to compressed elements of flexural members only. The provisions shall be limited to multiple flute built-up members having edge-stiffened cover plates. When the spacing of fasteners, s, of a uniformly compressed element restrained by intermittent *connections* is not greater than the limits specified in Section D1.3, the *effective width* shall be calculated in accordance with Section B2.1. When the spacing of fasteners is greater than the limits specified in Section D1.3, the *effective width* shall be determined in accordance with (a) and (b) below.

(a) *Strength Determination*

The *effective width* of the uniformly compressed element restrained by intermittent *connections* shall be determined as follows:

(1) When $f < F_c$, the *effective width* of the compression element between *connection* lines shall be calculated in accordance with Section B2.1(a).

(2) When $f \geq F_c$, the *effective width* of the compression element between *connection* lines shall be calculated in accordance with Section B2.1(a), except that the reduction factor ρ shall be determined as follows:

$$\rho = \rho_t \rho_m \leq (1 - 0.22/\lambda)/\lambda \qquad (Eq.\ B2.5\text{-}1)$$

where

$$\rho_t = (1.0 - 0.22/\lambda_t)/\lambda_t \leq 1.0 \qquad (Eq.\ B2.5\text{-}2)$$

where

$$\lambda_t = \sqrt{\frac{F_c}{F_{cr}}} \qquad (Eq.\ B2.5\text{-}3)$$

F_c = Critical column *buckling stress* of compression element

$$= 3.29\,E/(s/t)^2 \qquad (Eq.\ B2.5\text{-}4)$$

where

s = Center-to-center spacing of connectors in line of compression *stress*
E = Modulus of elasticity of steel
t = *Thickness* of cover plate in compression

F_{cr} = Critical *buckling stress* defined in Eq. B2.1-5 where w is the transverse spacing of connectors

$$\rho_m = 8\left(\frac{F_y}{f}\right)\sqrt{\frac{tF_c}{df}} \le 1.0 \qquad (Eq.\ B2.5\text{-}5)$$

where

F_y = Design *yield stress* of the compression element restrained by intermittent *connections*

d = Overall depth of the built-up member

f = *Stress* in compression element restrained by intermittent *connections* when the controlling extreme fiber *stress* is F_y

The provisions of this section shall apply to shapes that meet the following limits:

1.5 in. (38.1 mm) ≤ d ≤ 7.5 in. (191 mm)

0.035 in. (0.889 mm) ≤ t ≤ 0.060 in. (1.52 mm)

2.0 in. (50.8 mm) ≤ s ≤ 8.0 in. (203 mm)

33 ksi (228 MPa or 2320 kg/cm^2) ≤ F_y ≤ 60 ksi (414 MPa or 4220 kg/cm^2)

100 ≤ w/t ≤ 350

The *effective width* of the edge stiffener and the flat portion, e, shall be determined in accordance with Section B4(a) with modifications as follows:

For $f < F_c$

$$w = e \qquad (Eq.\ B2.5\text{-}6)$$

For $f \ge F_c$

For the flat portion, e, the *effective width*, b, in Eqs. B4-4 and B4-5 shall be calculated in accordance with Section B2.1(a) with

(i) w taken as e;

(ii) if D/e ≤ 0.8

k is determined in accordance with Table B4-1

if D/e > 0.8

k=1.25; and

(iii) ρ calculated using Eq. B2.5-1 in lieu of Eq. B2.1-3.

where

w = *Flat width* of element measured between longitudinal *connection* lines and exclusive of radii at stiffeners

e = *Flat width* between the first line of connector and the edge stiffener. See Figure B2.5-1

D = Overall length of stiffener as defined in Section B4

For the edge stiffener, d_s and I_a shall be determined using w′ and f′ in lieu of w and f, respectively.

$$w' = 2e + \text{minimum of } (0.75s \text{ and } w_1) \qquad (Eq.\ B2.5\text{-}7)$$

$$f' = \text{Maximum of } (\rho_m f \text{ and } F_c) \qquad (Eq.\ B2.5\text{-}8)$$

where

f′ = *Stress* used in Section B4(a) for determining *effective width* of edge stiffener

F_c = *Buckling stress* of cover plate determined in accordance with Eq. B2.5-4

w′ = Equivalent *flat width* for determining the *effective width* of edge stiffener

w_1 = Transverse spacing between the first and the second line of connectors in the compression element. See Figure B2.5-1

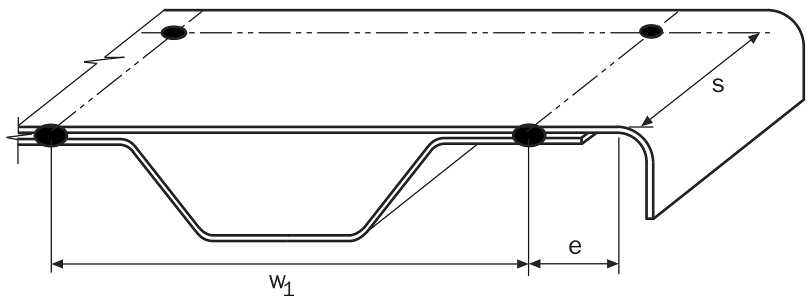


Figure B2.5-1 Dimension Illustration of Cellular Deck

The provisions of this section shall not apply to single flute members having compression plates with edge stiffeners.

(b) Serviceability Determination

The *effective width* of the uniformly compressed element restrained by intermittent *connections* used for computing deflection shall be determined in accordance with Section B2.5(a) except that: 1) f_d shall be substituted for f, where f_d is the computed compression *stress* in the element being considered at *service load*, and 2) the maximum extreme fiber *stress* in the built-up member shall be substituted for F_y.

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.

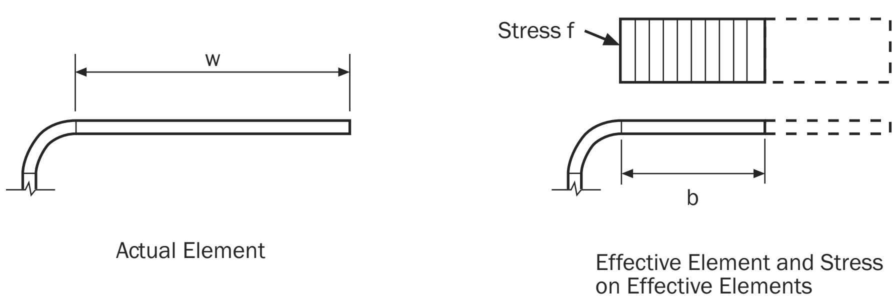


Figure B3.1-1 Unstiffened Element With Uniform Compression

(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.

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. See Figure 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 $w/t \leq 60$

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 (Figure 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)):

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

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

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

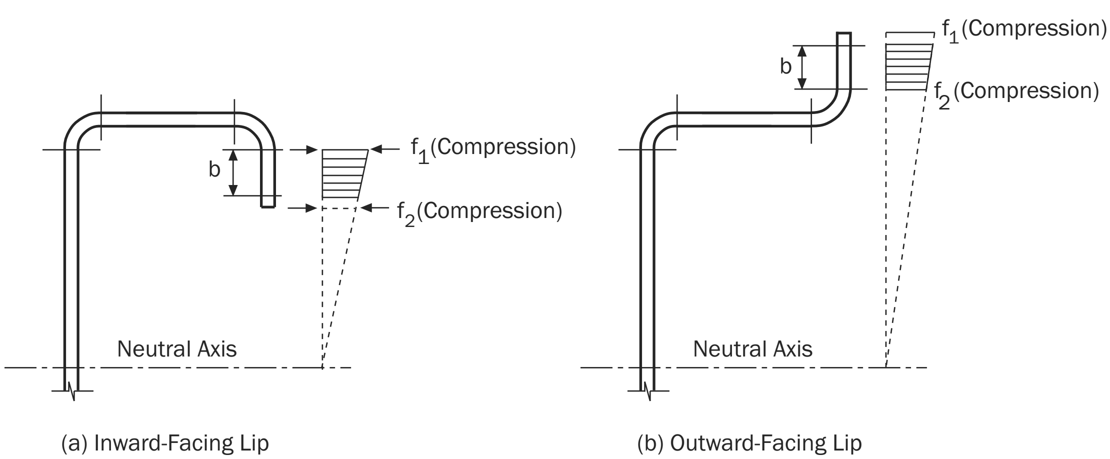


Figure B3.2-1 Unstiffened Elements Under Stress Gradient, Both Longitudinal Edges in Compression

(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 $\lambda \le 0.673(1 + \psi)$

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

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

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

For $\psi < 1$

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

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

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

For $\psi \ge 1$,

$\rho = 1$

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

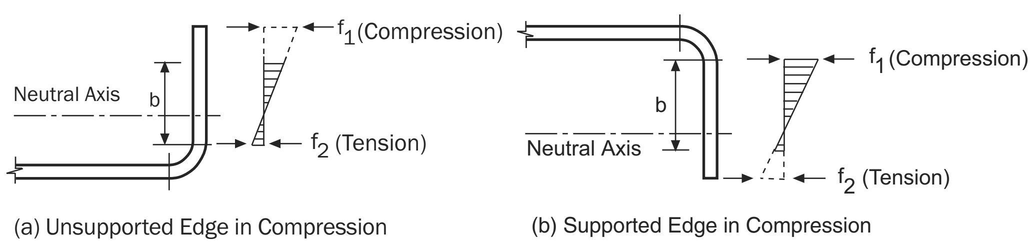


Figure B3.2-2 Unstiffened Elements Under Stress Gradient, One Longitudinal Edge in Compression and the Other Longitudinal Edge in Tension

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)):

$b = w$ when $\lambda \le 0.856$ (*Eq.* B3.2-8)

$b = \rho w$ when $\lambda > 0.856$ (*Eq.* B3.2-9)

where

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

$k = 0.145(b_o/h_o) + 1.256$ (*Eq.* B3.2-11)

$0.1 \le b_o/h_o \le 1.0$

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

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.

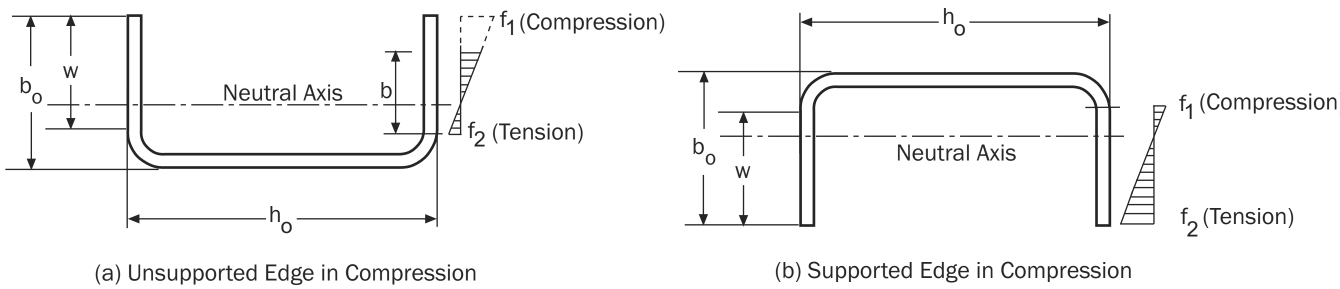


Figure B3.2-3 Unstiffened Elements of C-Section Under Stress Gradient for Alternative Methods

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

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

(a) Strength Determination

For $w/t \leq 0.328S$:

$I_a = 0$ (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)

E = Modulus of elasticity of steel

f = Stress in compression flange

w = Flat dimension of *flange* (see 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

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

b = *Effective design width*

b_1, b_2 = Portions of *effective design width* (see Figure B4-1)

d_s = Reduced *effective width* of stiffener (see Figure B4-1), and which is used in computing overall effective section properties

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

$$(R_I) = I_s/I_a \leq 1 \qquad (Eq.\ B4\text{-}9)$$

where

I_s = Unreduced moment of inertia of stiffener about its own centroidal axis parallel to element to be stiffened. For edge stiffeners, the round corner between stiffener and element to be stiffened is not considered 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

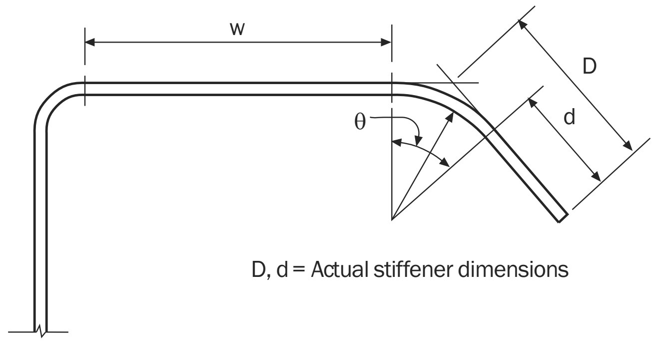
Simple Lip Edge Stiffener (140° ≥ θ ≥ 40°)	
$D/w \leq 0.25$	$0.25 < D/w \leq 0.8$
$3.57(R_I)^n + 0.43 \leq 4$	$(4.82 - \frac{5D}{w})(R_I)^n + 0.43 \leq 4$

where

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

(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.



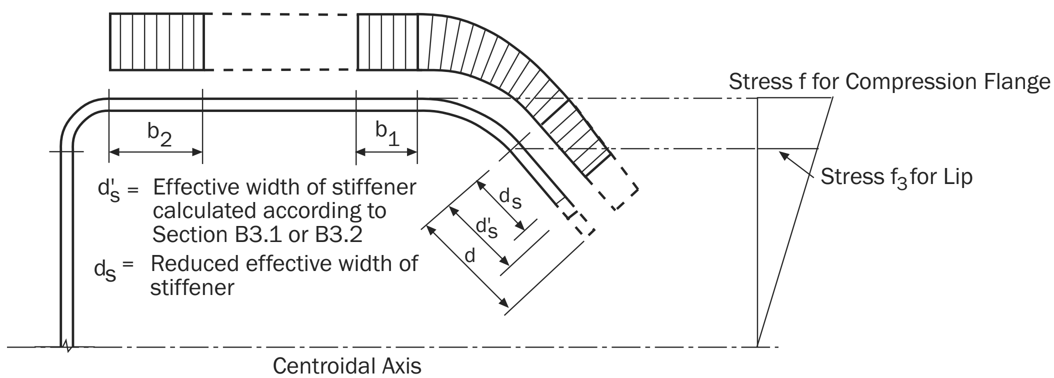


Figure B4-1 Element With Simple Lip Edge Stiffener

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 notations 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

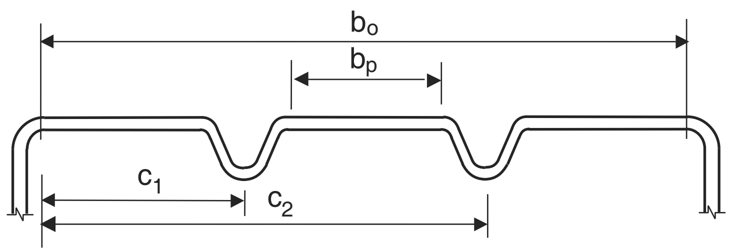


Figure B5.1-1 Plate Widths and Stiffener Locations

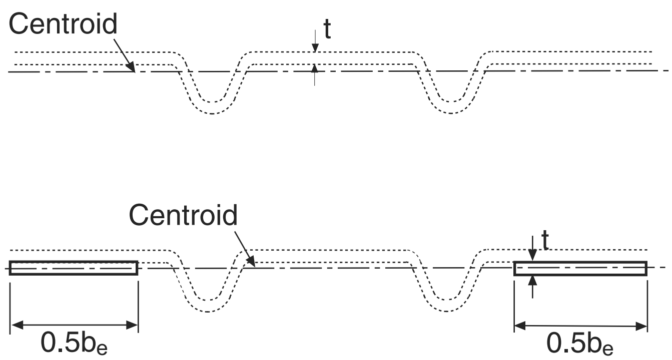


Figure B5.1-2 Effective Width Locations

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

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

where

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

$\rho = (1 - 0.22/\lambda)/\lambda$ when $\lambda > 0.673$ (*Eq*. B5.1-2)

where

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

where

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

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 $b_o/h < 1$

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

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

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

(a) *Strength Determination*

$$k_{loc} = 4(b_o/b_p)^2 \qquad (Eq.\ B5.1.1\text{-}1)$$

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

where

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

where

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

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

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

(b) *Serviceability Determination*

The *effective width*, b_d, used in determining serviceability shall be calculated as in Section 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 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 is permitted to be substituted for β to account for increased capacity due to bracing.

(b) Serviceability Determination

The *effective width*, b_d, used in determining serviceability shall be calculated as in Section 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 $b_o/t \le 0.328S$, the element is fully effective and no *local buckling* reduction is required.

If $b_o/t > 0.328S$, the plate *buckling* coefficient, k, is determined in accordance with Section 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

For axially loaded tension members, the *available tensile strength* [*factored resistance*] shall be the lesser of the values obtained in accordance with Sections C2.1 and C2.2, where the *nominal strengths* [*resistance*] and the corresponding *safety* and *resistance factors* are provided. The *available strengths* [*factored resistance*] shall be determined in accordance with the applicable design method in Section A4, A5, or A6.

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

C2.1 Yielding of Gross Section

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

$$T_n = A_g F_y \qquad (Eq.\ C2.1\text{-}1)$$

Ω_t = 1.67 (*ASD*)

ϕ_t = 0.90 (*LRFD*)

= 0.90 (*LSD*)

where

A_g = *Gross area* of cross-section

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

C2.2 Rupture of Net Section

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

$$T_n = A_n F_u \qquad (Eq.\ C2.2\text{-}2)$$

Ω_t = 2.00 (*ASD*)

ϕ_t = 0.75 (*LRFD*)

= 0.75 (*LSD*)

where

A_n = *Net area* of cross-section

F_u = *Tensile strength* as specified in either Section A2.1 or A2.3.2

C3 Flexural Members

C3.1 Bending

The *design flexural strength* [*factored resistance*], $\phi_b M_n$, and the *allowable flexural strength*, M_n/Ω_b, 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* [*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.

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

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

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 is 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 defined in Eq. C3.1.1-3.

(4) The shear force does not exceed $0.35F_y$ for *ASD*, and $0.6F_y$ for *LRFD* and *LSD* times the *web* area (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* [*resistance*], M_n, shall not exceed either $1.25 S_e F_y$, as determined in accordance with Procedure I of Section C3.1.1 (a), or that causing a maximum compression strain of $C_y e_y$ (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 of steel

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

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

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

(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:

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

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

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

where

λ = Slenderness factor defined in Section B3.2

λ_3 = 0.43

λ_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* [*resistances*] 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 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* [*resistance*], M_n, shall be calculated in accordance with Eq. C3.1.2.1-1.

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

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* [*factored 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:

(1) For bending about the symmetry axis:

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

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

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

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

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

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 cross-section relative to extreme compression fiber

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

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

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

where
G = Shear modulus of steel
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 is permitted to be calculated using the equation given in (b) for *doubly-symmetric* I-*sections*, *singly-symmetric* C-*sections*, or *point-symmetric* Z-*sections*.

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

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

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

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

where
K_x = Effective length factors for bending about x-axis
L_x = Unbraced length of member for bending about x-axis

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

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

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

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

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

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

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* [*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* [*resistance*] shall be determined in accordance with Section C3.1.1. L_u shall be calculated as follows:

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

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* [*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:

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

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* [*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 resistance*] in accordance with the applicable design method in Section A4, A5, or A6.

$M_n = F_c S_f$ (*Eq*. C3.1.3-1)

$\Omega_b = 1.67$ (*ASD*)

$\phi_b = 0.95$ (*LRFD*)

$= 0.90$ (*LSD*)

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

$F_c = 1.25\ F_y$ (*Eq*. C3.1.3-2)

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

$F_c = 0.328E/(D/t)$ (*Eq*. C3.1.3-4)

where

D = Outside diameter of cylindrical tube

t = Wall *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* [*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 resistance*] in accordance with the applicable design method in Section A4, A5, or A6.

$\Omega_b = 1.67$ (*ASD*)

$\phi_b = 0.90$ (*LRFD*)

$= 0.85$ (*LSD*)

For $\lambda_d \le 0.673$

$M_n = M_y$ (*Eq*. C3.1.4-1)

For $\lambda_d > 0.673$

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

where

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

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

where

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

$M_{crd} = S_fF_d$ (*Eq*. C3.1.4-5)

where

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

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

(a) For C- and Z-Sections or any Open Cross-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 are permitted to apply to any open cross-section with a single *web* and single edge-stiffened compression *flange*. The *distortional buckling stress*, F_d, shall be calculated in accordance with Eq. C3.1.4-6 as follows:

$$F_d = \beta \frac{k_{\phi fe} + k_{\phi we} + k_{\phi}}{\tilde{k}_{\phi fg} + \tilde{k}_{\phi wg}} \quad (Eq.\ C3.1.4\text{-}6)$$

where

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

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

where

L = Minimum of L_{cr} and L_m

where

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

(*Eq*. C3.1.4-8)

where

h_o = Out-to-out *web* depth (see Figure B2.3-2)

μ = Poisson's ratio of steel

t = Base steel *thickness*

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

x_{of} = x distance from the centroid of the *flange* to the shear center of the *flange*

h_{xf} = x distance from the centroid of the *flange* to the *flange*/*web* junction

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_{of}, and h_{xf} 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 = Smaller and larger end moments, respectively, in the unbraced segment (L_m) of the beam; M_1/M_2 is positive when the moments cause reverse curvature and negative when bent in single curvature

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

$$= \left(\frac{\pi}{L}\right)^4 \left(EI_{xf}(x_{of} - h_{xf})^2 + EC_{wf} - E\frac{I_{xyf}^2}{I_{yf}}(x_{of} - h_{xf})^2 \right) + \left(\frac{\pi}{L}\right)^2 GJ_f \qquad (Eq.\ C3.1.4\text{-}9)$$

where

E = Modulus of elasticity of steel

G = Shear modulus of steel

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

$$= \frac{Et^3}{12(1-\mu^2)}\left(\frac{3}{h_o} + \left(\frac{\pi}{L}\right)^2 \frac{19h_o}{60} + \left(\frac{\pi}{L}\right)^4 \frac{h_o{}^3}{240}\right) \qquad (Eq.\ C3.1.4\text{-}10)$$

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

$$= \left(\frac{\pi}{L}\right)^2 \left[A_f\left((x_{of} - h_{xf})^2 \left(\frac{I_{xyf}}{I_{yf}}\right)^2 - 2y_{of}(x_{of} - h_{xf})\left(\frac{I_{xyf}}{I_{yf}}\right) + h_{xf}^2 + y_{of}^2 \right) + I_{xf} + I_{yf} \right]$$

(*Eq*. C3.1.4-11)

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_{of} = y distance from the centroid of the *flange* to the shear center 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.* C3.1.4-12)

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, $f_1=-f_2$, $\xi_{web} = 2$)

(b) Rational Elastic Buckling Analysis

A rational elastic *buckling* analysis that considers *distortional buckling* is permitted to be used in lieu of the expressions given in Section C3.1.4 (a). 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 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)

Ω_v = 1.60 (*ASD*)

ϕ_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 $\sqrt{Ek_v/F_y} < h/t \le 1.51\sqrt{Ek_v/F_y}$

$$F_v = \frac{0.60\sqrt{Ek_vF_y}}{(h/t)}$$ (*Eq.* C3.2.1-3)

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

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

$= 0.904\ Ek_v/(h/t)^2$ (*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 $a/h \leq 1.0$

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

when $a/h > 1.0$

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

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 of steel

= 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:

(a) $d_h/h \leq 0.7$,

(b) $h/t \leq 200$,

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

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

(e) Non-circular holes, corner radii ≥ 2t,

(f) Non-circular holes, $d_h \leq 2.5$ in. (63.5 mm) and $L_h \leq 4.5$ in. (114 mm) ,

(g) Circular holes, diameter ≤ 6 in. (152 mm), and

(h) $d_h > 9/16$ in. (14.3 mm).

where

d_h = Depth of *web* hole

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

t = *Web thickness*

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 $c/t \geq 54$

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 without shear stiffeners as defined in Section C3.7.3, 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 shear stiffeners as defined in Section C3.7.3, 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* [moment due to *factored loads*], $\overline{M}$, and the *required shear strength* [shear force due to *factored loads*], $\overline{V}$, shall not exceed $\phi_b M_n$ and $\phi_v V_n$, respectively.

For beams without shear stiffeners as defined in Section C3.7.3, the *required flexural strength* [moment due to *factored loads*], $\overline{M}$, and the *required shear strength* [shear force due to *factored loads*], $\overline{V}$, shall also satisfy the following interaction equation:

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

For beams with shear stiffeners as defined in Section C3.7.3, 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* [*resistance*] when bending alone is considered

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

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

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

$\overline{V}$ = *Required shear strength* [shear force due to *factored loads*]
= 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, is permitted to be calculated as follows, except that P_{nc} shall not be larger than the interior one-*flange* loading condition:

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

where

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

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

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 $0.5 \leq L_o/h \leq 1.5$ and $h/t \leq 154$. For L_o/h or h/t outside these limits, α=1.

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

P_n and P_{nc} shall represent the *nominal strengths* [*resistances*] for *load* or reaction for one solid *web* connecting top and bottom *flanges*. For hat, multi-*web* sections and C- or Z-sections, P_n or P_{nc} shall be the *nominal strength* [*resistance*] for a single *web*, and the total *nominal strength* [*resistance*] shall be computed by multiplying P_n or P_{nc} by the number of *webs* at the considered cross-section.

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

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

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

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

Table 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 per Web

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	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-section members where h/t ≤ 200, N/t ≤ 210, N/h ≤ 2.0, and θ = 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 the bearing to the end of the member shall be extended at least 2.5h. For unfastened cases, the distance from the edge of the bearing to the end of the member shall be extended at least 1.5h.

TABLE 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 ASD Ω_w	USA and Mexico LRFD ϕ_w	Canada LSD ϕ_w	Limits
Fastened to Support	Stiffened or Partially Stiffened Flanges	One-Flange Loading or Reaction	End	4	0.14	0.35	0.02	1.75	0.85	0.75	R/t ≤ 9
			Interior	13	0.23	0.14	0.01	1.65	0.90	0.80	R/t ≤ 5
		Two-Flange Loading or Reaction	End	7.5	0.08	0.12	0.048	1.75	0.85	0.75	R/t ≤ 12
			Interior	20	0.10	0.08	0.031	1.75	0.85	0.75	R/t ≤ 12 d[1]≥4.5 in. (110 mm)
Unfastened	Stiffened or Partially Stiffened Flanges	One-Flange Loading or Reaction	End	4	0.14	0.35	0.02	1.85	0.80	0.70	R/t ≤ 5
			Interior	13	0.23	0.14	0.01	1.65	0.90	0.80	
		Two-Flange Loading or Reaction	End	13	0.32	0.05	0.04	1.65	0.90	0.80	R/t ≤ 3
			Interior	24	0.52	0.15	0.001	1.90	0.80	0.65	
	Unstiffened Flanges	One-Flange Loading or Reaction	End	4	0.40	0.60	0.03	1.80	0.85	0.70	R/t ≤ 2
			Interior	13	0.32	0.10	0.01	1.80	0.85	0.70	R/t ≤ 1
		Two-Flange Loading or Reaction	End	2	0.11	0.37	0.01	2.00	0.75	0.65	R/t ≤ 1
			Interior	13	0.47	0.25	0.04	1.90	0.80	0.65	

Note: [1] d = Out-to-out depth of section in the plane of the *web*

Table C3.4.1-3 shall apply to single *web* Z-section members where h/t ≤ 200, N/t ≤ 210, N/h ≤ 2.0, and θ = 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 the bearing to the end of the member shall be extended at least 2.5h; for unfastened cases, the distance from the edge of the bearing to the end of the member shall be extended at least 1.5h.

TABLE 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		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 ≤ 9
			Interior	13	0.23	0.14	0.01	1.65	0.90	0.80	R/t ≤ 5.5
		Two-Flange Loading or Reaction	End	9	0.05	0.16	0.052	1.75	0.85	0.75	R/t ≤ 12
			Interior	24	0.07	0.07	0.04	1.85	0.80	0.70	R/t ≤ 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 ≤ 5
			Interior	13	0.23	0.14	0.01	1.65	0.90	0.80	
		Two-Flange Loading or Reaction	End	13	0.32	0.05	0.04	1.65	0.90	0.80	R/t ≤ 3
			Interior	24	0.52	0.15	0.001	1.90	0.80	0.65	
	Unstiffened Flanges	One-Flange Loading or Reaction	End	4	0.40	0.60	0.03	1.80	0.85	0.70	R/t ≤ 2
			Interior	13	0.32	0.10	0.01	1.80	0.85	0.70	R/t ≤ 1
		Two-Flange Loading or Reaction	End	2	0.11	0.37	0.01	2.00	0.75	0.65	R/t ≤ 1
			Interior	13	0.47	0.25	0.04	1.90	0.80	0.65	

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 per Web

Support Conditions	Load Cases		C	C_R	C_N	C_h	USA and Mexico ASD Ω_w	USA and Mexico LRFD ϕ_w	Canada LSD ϕ_w	Limits
Fastened to Support	One-Flange Loading or Reaction	End	4	0.25	0.68	0.04	2.00	0.75	0.65	$R/t \le 5$
		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	$R/t \le 5$
		Interior	17	0.13	0.13	0.04	1.80	0.85	0.70	$R/t \le 10$

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 per Web

Support Conditions	Load Cases		C	C_R	C_N	C_h	USA and Mexico ASD Ω_w	USA and Mexico LRFD ϕ_w	Canada LSD ϕ_w	Limits
Fastened to Support	One-Flange Loading or Reaction	End	4	0.04	0.25	0.025	1.70	0.90	0.80	$R/t \le 20$
		Interior	8	0.10	0.17	0.004	1.75	0.85	0.75	
	Two-Flange Loading or Reaction	End	9	0.12	0.14	0.040	1.80	0.85	0.70	$R/t \le 10$
		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	

Note: Multi-*web* deck sections are considered unfastened for any support fastener spacing greater than 18 in. (460 mm).

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:

(a) $d_h/h \le 0.7$,
(b) $h/t \le 200$,
(c) Hole centered at mid-depth of *web*,
(d) Clear distance between holes ≥ 18 in. (457 mm),
(e) Distance between end of member and edge of hole ≥ d,
(f) Non-circular holes, corner radii ≥ 2t,
(g) Non-circular holes, $d_h \le 2.5$ in. (63.5 mm) and $L_h \le 4.5$ in. (114 mm),
(h) Circular holes, diameters ≤ 6 in. (152 mm), and
(i) $d_h > 9/16$ in. (14.3 mm).

where

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

For end one-*flange* reaction (Equation 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:

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

N ≥ 1 in. (25.4 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:

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

N ≥ 3 in. (76.2 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, satisfies $M \le M_{nxo}/\Omega_b$, 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:

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

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:

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

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

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

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

$h/t \leq 150$,
$N/t \leq 140$,
$F_y \leq 70$ ksi (483 MPa or 4920 kg/cm^2), 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 notations 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:

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

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:

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

where

ϕ = 0.90 (*LRFD*)

= 0.75 (*LSD*)

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

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

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/cm^2), 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 notations shall apply in this section:

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

$\overline{P}$ = *Required strength* [force due to *factored loads*] for 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* [*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 torsionally unrestrained flexural members subjected to both bending and torsional loading, the *available flexural strength* [*factored resistance*] calculated in accordance with Section C3.1.1(a) shall be multiplied by a reduction factor, R.

As specified in Eq. C3.6-1, the reduction factor, R, shall be equal to the ratio of the maximum 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. Eq. C3.6-1 is limited to *singly*- or *doubly-symmetric sections* subject to bending about an axis of symmetry and not subject to bi-axial bending. The torsional effect for other sections shall be considered using r*ational engineering analysis*.

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

where

$f_{bending\_max}$ = Bending *stress* at extreme fiber, taken on the same side of the neutral axis as $f_{bending}$

$f_{bending}$ = Bending *stress* at location in cross-section where combined bending and torsion stress is maximum

$f_{torsion}$ = Torsional warping *stress* at location in cross-section where combined bending and torsion *stress* is maximum

Stresses shall be calculated using full unreduced section properties. For C-sections with edge-stiffened *flanges*, if the maximum combined *stresses* occur at the junction of the *web* and *flange*, the R factor is 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 apply if 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.

$$P_n = 0.7(P_{wc} + A_eF_y) \geq P_{wc} \qquad (Eq.\ C3.7.2\text{-}1)$$

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:

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

(b) Stiffeners are C-section stud or track members with a minimum *web* depth of 3-1/2 in. (88.9 mm) and a minimum base steel *thickness* of 0.0329 in. (0.836 mm).

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

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

(e) The length of the stiffener is not less than the depth of the flexural member minus 3/8 in. (9.53 mm).

(f) The bearing width is not less than 1-1/2 in. (38.1 mm).

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 Eq. 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

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

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

where

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

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

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

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(c).

C4 Concentrically Loaded Compression Members

The *available axial strength* [*factored 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* [*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 resistance*] in accordance with the applicable design method in Section A4, A5, or A6.

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

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 $\lambda_c \leq 1.5$

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

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 is 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 are 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

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

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 Sections D6.1.3 and D6.1.4. The *nominal axial strength* [*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* [*factored resistance*] in accordance with the applicable design method in Section A4, A5, or A6.

Ω_c = 1.80 (*ASD*)

ϕ_c = 0.85 (*LRFD*)

= 0.80 (*LSD*)

For $\lambda_d \le 0.561$

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

For $\lambda_d > 0.561$

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

where

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

P_n = *Nominal axial strength* [*resistance*]

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

where

A_g = *Gross area* of the cross-section

F_y = *Yield stress*

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

where

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

(a) For C- and Z-Sections or Hat Sections or any Open Cross-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 cross-section with stiffened *flanges* of equal dimension.

$$F_d = \frac{k_{\phi fe} + k_{\phi we} + k_{\phi}}{\tilde{k}_{\phi fg} + \tilde{k}_{\phi wg}} \qquad (Eq.\ C4.2\text{-}6)$$

where

$k_{\phi fe}$ = Elastic rotational stiffness provided by the *flange* to the *flange*/*web* juncture, in accordance with Eq. C3.1.4-9

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

$$= \frac{Et^3}{6h_o(1-\mu^2)} \qquad (Eq.\ C4.2\text{-}7)$$

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. C3.1.4-11

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

$$= \left(\frac{\pi}{L}\right)^2 \frac{th_o^3}{60} \qquad (Eq.\ C4.2\text{-}8)$$

where

L = Minimum of L_{cr} and L_m

where

$$L_{cr} = \left(\frac{6\pi^4 h_o\left(1-\mu^2\right)}{t^3}\left(I_{xf}\left(x_{of}-h_{xf}\right)^2 + C_{wf} - \frac{I_{xyf}^2}{I_{yf}}\left(x_{of}-h_{xf}\right)^2\right)\right)^{1/4} \qquad (Eq.\ C4.2\text{-}9)$$

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

See Section C3.1.4 (a) for definition of variables in Eq. C4.2-9.

(b) Rational Elastic Buckling Analysis

A rational elastic *buckling* analysis that considers *distortional buckling* is permitted to be used in lieu of the expressions given in Section C4.2(a). 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:

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

and

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

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* [effects of *factored loads*] $\overline{T}$, $\overline{M}_x$, and $\overline{M}_y$ shall satisfy the following interaction equations:

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

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

where

$\overline{M}_x$, $\overline{M}_y$ = *Required flexural strengths* [moment due to *factored loads*] 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 (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* [tensile axial force due to *factored loads*]

	=	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* [*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 r*equired 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 is 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 is 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

$$\alpha_x = 1 - \frac{\Omega_c P}{P_{Ex}} > 0 \quad (Eq.\ C5.2.1\text{-}4)$$

$$\alpha_y = 1 - \frac{\Omega_c P}{P_{Ey}} > 0 \quad (Eq.\ C5.2.1\text{-}5)$$

where

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

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

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

$$C_m = 0.6 - 0.4\,(M_1/M_2) \quad (Eq.\ C5.2.1\text{-}8)$$

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* [effects due to *factored loads*] $\overline{P}$, $\overline{M}_x$, and $\overline{M}_y$ shall be determined using first order elastic analysis and shall satisfy the following interaction equations. Alternatively, the *required strengths* [effects of *factored loads*] $\overline{P}$, $\overline{M}_x$, 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*, $\overline{M}_y$ is permitted to be taken as the *required flexural strength* [moment due to *factored loads*] only. For other angle sections or singly-symmetric unstiffened angles 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* [moment due to *factored loads*] or the *required flexural strength* [moment due to *factored loads*] plus ($\overline{P}$)L/1000, whichever results in a lower permissible value of $\overline{P}$.

$$\frac{\overline{P}}{\phi_c P_n} + \frac{C_{mx}\overline{M}_x}{\phi_b M_{nx}\alpha_x} + \frac{C_{my}\overline{M}_y}{\phi_b M_{ny}\alpha_y} \le 1.0 \qquad (Eq.\ C5.2.2\text{-}1)$$

$$\frac{\overline{P}}{\phi_c P_{no}} + \frac{\overline{M}_x}{\phi_b M_{nx}} + \frac{\overline{M}_y}{\phi_b M_{ny}} \le 1.0 \qquad (Eq.\ C5.2.2\text{-}2)$$

When $\overline{P}/\phi_c P_n \le 0.15$, the following equation is permitted to be used in lieu of the above two equations:

$$\frac{\overline{P}}{\phi_c P_n} + \frac{\overline{M}_x}{\phi_b M_{nx}} + \frac{\overline{M}_y}{\phi_b M_{ny}} \le 1.0 \qquad (Eq.\ C5.2.2\text{-}3)$$

where

$\overline{P}$ = *Required compressive axial strength* [compressive axial force due to *factored loads*]
= 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* [moment due to *factored loads*] with respect to centroidal axes of effective section determined for *required compressive axial strength* [compressive axial force due to *factored loads*] 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 (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* [*resistances*] about centroidal axes determined in accordance with Section C3.1

$$\alpha_x = 1 - \frac{\overline{P}}{P_{Ex}} > 0 \quad \text{(Eq. C5.2.2-4)}$$

$$\alpha_y = 1 - \frac{\overline{P}}{P_{Ey}} > 0 \quad \text{(Eq. C5.2.2-5)}$$

where

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

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

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

$$C_m = 0.6 - 0.4\,(M_1/M_2) \quad \text{(Eq. C5.2.2-8)}$$

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 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 *connections* (one or more welds or other connectors), s_{max}, joining two C-sections to form an I-section shall be:

$$s_{max} = L / 6 \text{ or } \frac{2gT_s}{mq}, \text{ whichever is smaller} \qquad (Eq.\ D1.1\text{-}1)$$

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 determining longitudinal spacing of *connections* (See below for methods of determination.)

The *load*, q, shall be obtained by dividing the concentrated *loads* or reactions by the length of bearing. For beams designed for a uniformly distributed *load*, q shall be taken as equal to three times the uniformly distributed *load*, based on the critical *load combinations* for *ASD*, *LRFD*, and *LSD*. If the length of bearing of a concentrated *load* or reaction is smaller than the longitudinal *connection* spacing, s, the *required strength* [force due to *factored loads*] of the *connections* closest to the *load* or reaction shall be calculated as follows:

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

where
P_s = Concentrated *load* [*factored load*] or reaction based on critical *load combinations* for *ASD*, *LRFD*, and *LSD*
T_r = *Required strength* [force due to *factored loads*] of *connection* in tension

The allowable maximum spacing of *connections*, s_{max}, shall depend upon the intensity of the *load* directly at the *connection*. Therefore, if uniform spacing of *connections* is used over the whole length of the beam, it shall be determined at the point of maximum local *load* intensity. In cases where this procedure would result in uneconomically close spacing, either one of the following methods is permitted to be adopted:

(a) The *connection* spacing varies along the beam according to the variation of the *load* intensity, or
(b) Reinforcing cover plates are welded to the *flanges* at points where concentrated *loads* occur. The *available shear strength* [*factored resistance*] of the *connections* joining these plates to the *flanges* is then used for T_s, and g is taken as the depth of the beam.

D1.2 Compression Members Composed of Two Sections in Contact

For compression members composed of two sections in contact, the *available axial strength* [*factored 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 and spacing shall satisfy the following:

(a) 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.

(b) 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.

(c) The intermediate fastener(s) or weld(s) at any longitudinal member tie location are capable of transmitting the *required strength* [force due to *factored loads*] in any direction of 2.5 percent of the *available axial strength* [*factored resistance*] of the built-up member.

D1.3 Spacing of Connections in Cover-Plated Sections

To develop the strength required of the compression element, the spacing, s, in the line of *stress*, of welds, rivets, or bolts connecting a cover plate, sheet, or a non-integral stiffener in compression to another element shall not exceed (a), (b), and (c) as follows:

(a) That which is required to transmit the shear between the connected parts on the basis of the *available strength* [*factored resistance*] per *connection* specified elsewhere herein;

(b) $1.16t\sqrt{E/f_c}$

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.

When any of the limits (a), (b), or (c) in this section are exceeded, the *effective width* shall be determined in accordance with Section B2.5.

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 and bracing systems, including *connections*, shall be designed with adequate strength and stiffness to restrain lateral bending or twisting of a loaded beam or column, and to avoid local crippling at the points of attachment. Braces and bracing systems, including *connections*, shall also be designed considering strength and stiffness requirements, as applicable.

C-section and Z-section beam bracing shall meet the requirements specified in Section D3.2.

Bracing of axially loaded compression members shall meet the requirements as specified in Section D3.3.

See Appendix B for additional requirements applicable to Canada. ☜B

D3.1 Symmetrical Beams and Columns

The provision of this section shall only apply to Canada. See Section D3.1 of Appendix B.

☜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 applicable to Canada. ☜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

$$P_{L1} = 1.5[W_y K' - (W_x / 2) + (M_z / d)] \qquad (\textit{Eq. } D3.2.1\text{-}1)$$

$$P_{L2} = 1.5[W_y K' - (W_x / 2) - (M_z / d)] \qquad (\textit{Eq. } D3.2.1\text{-}2)$$

When the uniform *load*, W, acts through the plane of the *web*, i.e., W_y = W:

$$P_{L1} = -P_{L2} = 1.5(m / d)W \quad \text{for C-sections} \qquad (\textit{Eq. } D3.2.1\text{-}3)$$

$$P_{L1} = P_{L2} = 1.5\left(\frac{I_{xy}}{2I_x}\right)W \qquad \text{for Z-sections} \qquad (Eq.\ D3.2.1\text{-}4)$$

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 *ASD*, *LRFD* or *LSD load combinations*, depending on the design method used) within a distance of 0.5a on each side of the brace

where

a = Longitudinal distance between centerline of braces

K' = 0 for C-sections

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

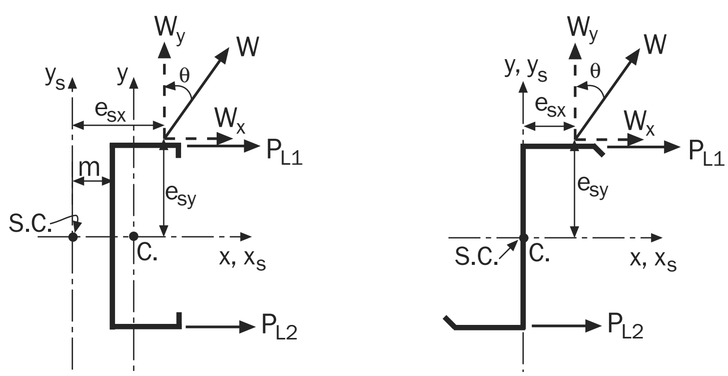


Figure D3.2.1-1 Coordinate Systems and Positive Force Directions

(b) For concentrated loads,

$$P_{L1} = P_y K' - (P_x/2) + (M_z/d) \qquad (Eq.\ D3.2.1\text{-}6)$$

$$P_{L2} = P_y K' - (P_x/2) - (M_z/d) \qquad (Eq.\ D3.2.1\text{-}7)$$

When a *design load* [*factored load*] acts through the plane of the *web*, i.e., P_y = P:

$$P_{L1} = -P_{L2} = (m/d)P \qquad \text{for C-sections} \qquad (Eq.\ D3.2.1\text{-}8)$$

$$P_{L1} = P_{L2} = \left(\frac{I_{xy}}{2I_x}\right)P \qquad \text{for Z-sections} \qquad (Eq.\ D3.2.1\text{-}9)$$

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 = $-P_xe_{sy} + P_ye_{sx}$, 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 *ASD*, *LRFD*, or *LSD load combinations*, depending on the design method used

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 as to effectively restrain the section against lateral deflection of both *flanges* at the ends and at any intermediate brace points.

When all *loads* and reactions on a beam are transmitted through members that frame into the section in such a manner as to effectively restrain the section against torsional rotation and lateral displacement, no additional braces shall be required except those required for strength in accordance with Section C3.1.2.1.

D3.3 Bracing of Axially Loaded Compression Members

To provide an adequate intermediate brace (or braces) that will allow an individual concentrically loaded compression member to develop its *required axial strength* [compressive axial force due to *factored loads*], the *required strength* [brace force due to *factored loads*] acting on the brace (or braces) shall be calculated in accordance with Eq. D3.3-1.

$$P_{rb} = 0.01\ P_{ra} \qquad (Eq.\ D3.3\text{-}1)$$

where

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

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

The stiffness of each brace shall equal or exceed β_{rb}, as calculated in Eq. D3.3-2:

For *ASD*

$$\beta_{rb} = \frac{2[4-(2/n)]}{L_b}(\Omega P_{ra}) \qquad (Eq.\ D3.3\text{-}2a)$$

Ω = 2.00

For *LRFD* and *LSD*

$$\beta_{rb} = \frac{2[4-(2/n)]}{L_b}\left(\frac{P_{ra}}{\phi}\right) \quad (Eq.\ D3.3\text{-}2b)$$

ϕ = 0.75 for *LRFD*
= 0.70 for *LSD*

where

β_{rb} = Minimum required brace stiffness to brace a single compression member
n = Number of equally spaced intermediate brace locations
L_b = Distance between braces on individual concentrically loaded compression member to be braced

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

Alternatively, the *required brace strength* [brace force due to *factored loads*] and stiffness are permitted to be determined by a *second-order analysis* in accordance with the requirements of Appendix 2.

D4 Cold-Formed Steel Light-Frame Construction

The design and installation of *structural members* utilized in cold-formed steel repetitive framing applications where the specified minimum base steel *thickness* is not greater than 0.1180 in. (2.997 mm) shall be in accordance with the AISI S200 and the following, as applicable:

(a) Framing for floor and roof systems in buildings shall be designed in accordance with AISI S210, or solely in accordance with this *Specification*.
(b) 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* are 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.
(c) Headers shall be designed in accordance with AISI S212, or solely in accordance with this *Specification*.
(d) 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.
(e) Trusses shall be designed in accordance with AISI S214.

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 Ω_d (ASD)	ϕ_d (LRFD)	ϕ_d (LSD)	Panel Buckling* 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, unless sufficient data exist to establish a *diaphragm* system effect in accordance with Section F1.1. Fastener shear strength calibration shall include the *diaphragm* material type. Calibration of individual fastener shear strengths 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* [*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 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

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 ≤ 12 (305)	Z	0.50
8.5 (216) < d ≤ 12 (305)	C	0.40

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

(a) Member depth ≤ 12 in. (305 mm),
(b) Member *flanges* with edge stiffeners,
(c) 60 ≤ depth/*thickness* ≤ 170,
(d) 2.8 ≤ depth/*flange* width ≤ 5.5,
(e) *Flange* width ≥ 2.125 in. (54.0 mm),
(f) 16 ≤ *flat width*/*thickness* of *flange* ≤ 43,
(g) For continuous span systems, the lap length at each interior support in each direction (distance from center of support to end of lap) is not less than 1.5d,
(h) Member span length is not greater than 33 feet (10 m),
(i) Both *flanges* are prevented from moving laterally at the supports,
(j) Roof or wall panels are steel sheets with 50 ksi (340 MPa or 3520 kg/cm^2) minimum *yield stress*, and a minimum of 0.018 in. (0.46 mm) base metal *thickness*, having a minimum rib depth of 1-1/8 in. (29 mm), spaced at a maximum of 12 in. (305 mm) on centers and attached in a manner to effectively inhibit relative movement between the panel and *purlin flange*,
(k) Insulation is glass fiber blanket 0 to 6 in. (152 mm) thick compressed between the member and panel in a manner consistent with the fastener being used,
(l) Fastener type is, at minimum, No. 12 self-drilling or self-tapping sheet metal screws or 3/16 in. (4.76 mm) rivets, having washers 1/2 in. (12.7 mm) diameter,

(m) Fasteners are not standoff type screws,
(n) Fasteners are spaced not greater than 12 in. (305 mm) on centers and placed near the center of the beam flange, and adjacent to the panel high rib, and
(o) The ratio of tensile strength to design yield stress shall not be less than 1.08.

If variables fall outside any of the above stated limits, the user shall perform full-scale tests in accordance with Section 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 is permitted to perform tests in accordance with Section F1 as an alternative to the procedure described in this section.

For simple span members, R shall be reduced for the effects of compressed insulation between the sheeting and the member. The reduction shall be calculated by multiplying R from Table 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 resistance*] in accordance with the applicable design method in Section A4, A5, or A6.

$$P_n = C_1 C_2 C_3 AE/29500 \quad (Eq.\ D6.1.3\text{-}1)$$

$\Omega = 1.80$ (*ASD*)

$\phi = 0.85$ (*LRFD*)

$= 0.80$ (*LSD*)

where

$$C_1 = (0.79x + 0.54) \quad (Eq.\ D6.1.3\text{-}2)$$

$$C_2 = (1.17\alpha t + 0.93) \quad (Eq.\ D6.1.3\text{-}3)$$

$$C_3 = \alpha(2.5b - 1.63d) + 22.8 \quad (Eq.\ D6.1.3\text{-}4)$$

where

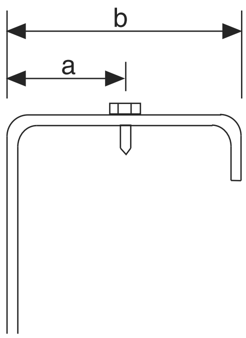
x = For Z-sections, fastener distance from outside *web* edge divided by *flange* width, as shown in Figure D6.1.3

= For C-sections, *flange* width minus fastener distance from outside *web* edge divided by *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



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

Eq. D6.1.3-1 shall be limited to roof and wall systems meeting the following conditions:

(1) $t \le 0.125$ in. (3.22 mm),
(2) 6 in. (152mm) $\le d \le 12$ in. (305 mm),
(3) *Flanges* are edge-stiffened compression elements,
(4) $70 \le d/t \le 170$,
(5) $2.8 \le d/b \le 5$,
(6) $16 \le$ *flange flat width* / $t \le 50$,
(7) Both *flanges* are prevented from moving laterally at the supports,
(8) Steel roof or steel wall panels with fasteners spaced 12 in. (305 mm) on center or less and having a minimum rotational lateral stiffness of 0.0015 k/in./in. (10,300 N/m/m or 0.105 kg/cm/cm) (fastener at mid-flange width for stiffness determination) determined in accordance with AISI S901,
(9) C- and Z-sections having a minimum *yield stress* of 33 ksi (228 MPa or 2320 kg/cm^2), and
(10) Span length not exceeding 33 feet (10.1 m).

(b) The strong axis *available strength* [*factored resistance*] shall be determined in accordance with Sections C4.1 and C4.1.1.

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.

☜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:

(a) The Uplift Pressure Test Procedure for Class 1 Panel Roofs in FM 4471 is permitted.

(b) Existing tests conducted in accordance with CEGS 07416 uplift test procedure prior to the adoption of these provisions are permitted.

The open-open end configuration, although not prescribed by the ASTM E1592 test procedure, is permitted provided the tested end conditions represent the installed condition, and the test follows the requirements given in 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. ☜A

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

β_o = Target reliability index
= 2.0 for USA and Mexico and 2.5 for Canada for panel flexural limits
= 2.5 for USA and Mexico and 3.0 for Canada for anchor limits

F_m = Mean value of the fabrication factor
= 1.0

M_m = Mean value of the material factor
= 1.1

V_M = Coefficient of variation of the material factor
= 0.08 for anchor failure mode
= 0.10 for other failure modes

V_F = Coefficient of variation of the fabrication factor
= 0.05

V_Q = Coefficient of variation of the *load effect*
= 0.21

V_P = Actual calculated coefficient of variation of the test results, without limit

n = Number of anchors in the test assembly with same tributary area (for anchor failure) or number of panels with identical spans and loading to the failed span (for non-anchor failures)

The *safety factor*, Ω, shall not be less than 1.67, and the *resistance factor*, ϕ, shall not be greater than 0.9 (*LRFD* and *LSD*).

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

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*.

$$P_{L_j} = \sum_{i=1}^{N_p}\left(P_i \frac{K_{eff_{i,j}}}{K_{total_i}}\right) \qquad (Eq.\ D6.3.1\text{-}1)$$

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*

$$= (C1)W_{p_i}\left\{\left[\left(\frac{C2}{1000}\right)\frac{I_{xy}L}{I_x d} + (C3)\frac{(m+0.25b)t}{d^2}\right]\alpha\cos\theta - (C4)\sin\theta\right\} \qquad (Eq.\ D6.3.1\text{-}2)$$

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

$$= w_i L \qquad (Eq.\ D6.3.1\text{-}3)$$

where

w_i = Required distributed gravity *load* supported by the i^{th} *purlin* per unit length (determined from the critical *ASD*, *LRFD* or *LSD load combination* depending on the design method used)

I_{xy} = Product of inertia of full unreduced section about centroidal axes parallel and perpendicular to the *purlin web* (I_{xy} = 0 for C-sections)

L = *Purlin* span length

m = Distance from shear center to mid-plane of *web* (m = 0 for Z-sections)

b = Top *flange* width of *purlin*

t = *Purlin thickness*

I_x = Moment of inertia of full unreduced section about centroidal axis perpendicular to the *purlin web*

d = Depth of *purlin*

α = +1 for top *flange* facing in the up-slope direction
-1 for top *flange* facing in the down-slope direction

θ = Angle between vertical and plane of *purlin web*

$K_{eff_{i,j}}$ = Effective lateral stiffness of the j^{th} anchorage device with respect to the i^{th} *purlin*

$$= \left[\frac{1}{K_a} + \frac{d_{p_{i,j}}}{(C6)LA_pE}\right]^{-1} \quad (Eq.\ D6.3.1\text{-}4)$$

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

$$= \sum_{j=1}^{N_a}\left(K_{eff_{i,j}}\right) + K_{sys} \quad (Eq.\ D6.3.1\text{-}5)$$

where

K_{sys} = Lateral stiffness of the roof system, neglecting anchorage devices

$$= \left(\frac{C5}{1000}\right)(N_p)\frac{ELt^2}{d^2} \quad (Eq.\ D6.3.1\text{-}6)$$

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 of the values of the two bays.

For systems with multiple spans and anchorage devices at either 1/3 points or mid-points, where the adjacent bays have different section properties or span lengths than the bay under consideration, the following procedures 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

$$K_{req} = \Omega \frac{20\left|\sum_{i=1}^{N_p} P_i\right|}{d} \ (ASD) \qquad (Eq.\ D6.3.1\text{-}8a)$$

$$K_{req} = \frac{1}{\phi}\frac{20\left|\sum_{i=1}^{N_p} P_i\right|}{d} \ (LRFD, LSD) \qquad (Eq.\ D6.3.1\text{-}8b)$$

Ω = 2.00 (*ASD*)
ϕ = 0.75 (*LRFD*)
= 0.70 (*LSD*)

In lieu of Eqs. D6.3.1-1 through D6.3.1-6, lateral restraint forces are permitted to be determined from alternative analysis. Alternative analysis shall include the first- or second-order effect and account for the effects of roof slope, torsion resulting from applied *loads* eccentric to shear center, torsion resulting from the lateral resistance provided by the sheathing, and *load* applied oblique to the principal axes. Alternative analysis shall also include the effects of the lateral and rotational restraint provided by sheathing attached to the top *flange*. Stiffness of the anchorage device shall be considered and shall account for flexibility of the *purlin web* above the attachment of the anchorage device *connection*.

When lateral restraint forces are determined from rational 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:

$$\Delta_{tf} \leq \frac{1}{\Omega}\frac{d}{20} \ (ASD) \qquad (Eq.\ D6.3.1\text{-}9a)$$

$$\Delta_{tf} \leq \phi\frac{d}{20} \ (LRFD, LSD) \qquad (Eq.\ D6.3.1\text{-}9b)$$

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	13	11	0.35	2.4	0.25
		First Interior Frame Line	1.0	1.7	69	0.77	1.6	0.13
		All Other Locations	1.0	4.3	55	0.71	1.4	0.17

Table 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
	Standing Seam (SS)		0.5	7.3	21	0.73	0.19	0.18
Multiple Spans	TF	End Bay Exterior Anchor	0.5	15	17	0.98	0.72	0.043
		End Bay Int. Anchor and 1st Int. Bay Ext. Anchor	0.5	2.4	50	0.96	0.82	0.20
		All Other Locations	0.5	6.1	41	0.96	0.69	0.12
	SS	End Bay Exterior Anchor	0.5	13	13	0.72	0.59	0.035
		End Bay Int. Anchor and 1st Int. Bay Ext. Anchor	0.5	0.84	56	0.64	0.20	0.14
		All Other Locations	0.5	3.8	45	0.65	0.10	0.014

D6.3.2 Alternative Lateral and Stability Bracing for Purlin Roof Systems

Torsional bracing that prevents twist about the longitudinal axis of a member in combination with lateral restraints that resist lateral displacement of the top *flange* at the frame line is permitted in lieu of the requirements of Section 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* [brace force due to *factored loads*] of lateral restraints at the frame line shall be determined by *rational engineering analysis* or testing. The lateral displacement of the top *flange* of the C- or Z-section at the frame line shall be limited to $d/(20\Omega)$ for *ASD* calculated at *nominal 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* [force due to *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), the specifications or standards stipulated in the corresponding Section E2a of Appendix A or B shall be followed. ☜A,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, the *nominal strength* [*resistance*], P_n, shall be calculated in accordance with Eq. E2.1-1:

$$P_n = Lt_eF_y \qquad (Eq.\ E2.1\text{-}1)$$

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:

$$P_n = Lt_e\,0.6F_{xx} \qquad (Eq.\ E2.1\text{-}2)$$

Ω = 1.90 (*ASD*)
ϕ = 0.80 (*LRFD*)
= 0.70 (*LSD*)

$$P_n = Lt_eF_y/\sqrt{3} \qquad (Eq.\ E2.1\text{-}3)$$

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 sheet 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 in. (1.27 mm) and 0.08 in. (2.03 mm) with a minimum pre-punched hole of 3/8 in. (9.53 mm) in diameter. Sheet-to-sheet welds shall not require weld washers.

Arc spot welds shall be specified by a minimum effective diameter of fused area, d_e. The minimum allowable effective diameter shall be 3/8 in. (9.53 mm).

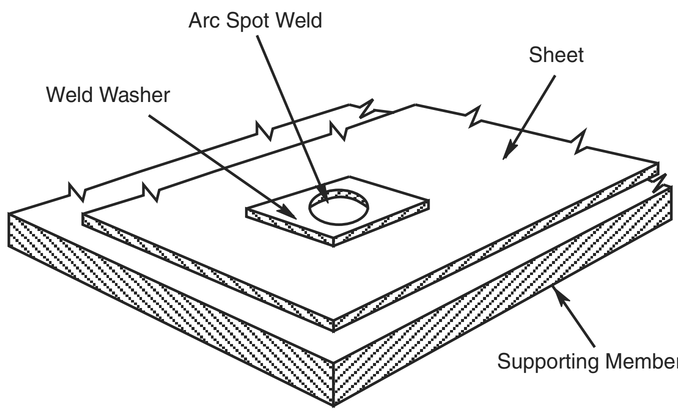


Figure E2.2-1 Typical Weld Washer

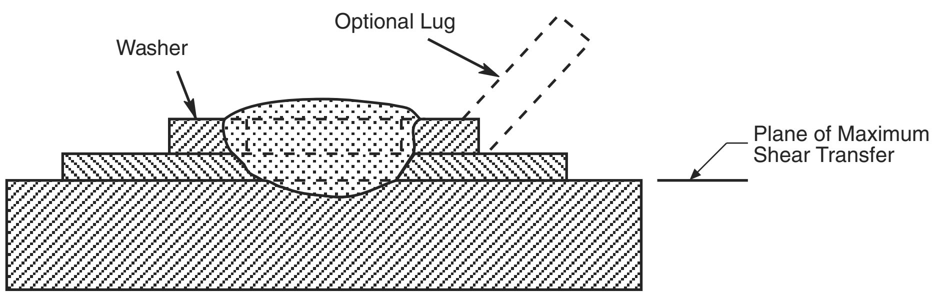


Figure E2.2-2 Arc Spot Weld Using Washer

E2.2.1 Minimum Edge and End Distance

The distance from the center line of an arc spot weld to the end or edge of the connected member shall not be less than 1.5d. In no case shall the clear distance between welds and the end or edge of the member be less than 1.0d, where d is the visible diameter of the outer surface of the arc spot weld. See Figures E2.2.1-1 and E2.2.1-2 for details.

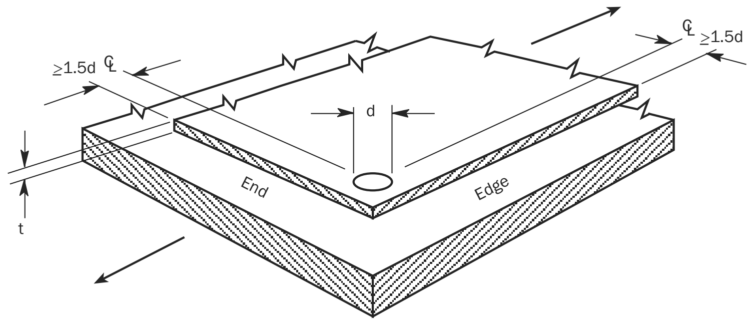


Figure E2.2.1-1 End and Edge Distance for Arc Spot Welds – Single Sheet

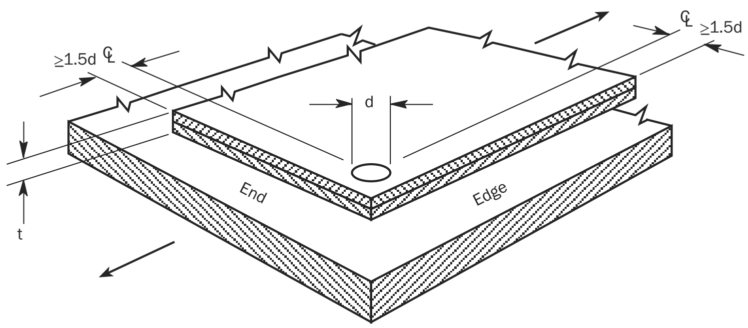


Figure E2.2.1-2 End and Edge Distance for Arc Spot Welds – Double Sheet

E2.2.2 Shear

E2.2.2.1 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) $P_n = \dfrac{\pi d_e^2}{4} 0.75F_{xx}$ (*Eq*. E2.2.2.1-1)

$\Omega = 2.55$ (*ASD*)
$\phi = 0.60$ (*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*. E2.2.2.1-2)

$\Omega = 2.20$ (*ASD*)
$\phi = 0.70$ (*LRFD*)
$= 0.60$ (*LSD*)

For $0.815\sqrt{E/F_u} < (d_a/t) < 1.397\sqrt{E/F_u}$

$$P_n = 0.280\left[1 + 5.59\frac{\sqrt{E/F_u}}{d_a/t}\right]td_aF_u \qquad (Eq.\ E2.2.2.1\text{-}3)$$

Ω = 2.80 (*ASD*)

ϕ = 0.55 (*LRFD*)

= 0.45 (*LSD*)

For $(d_a/t) \geq 1.397\sqrt{E/F_u}$

$$P_n = 1.40\ td_aF_u \qquad (Eq.\ E2.2.2.1\text{-}4)$$

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

$$= 0.7d - 1.5t \leq 0.55d \qquad (Eq.\ E2.2.2.1\text{-}5)$$

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

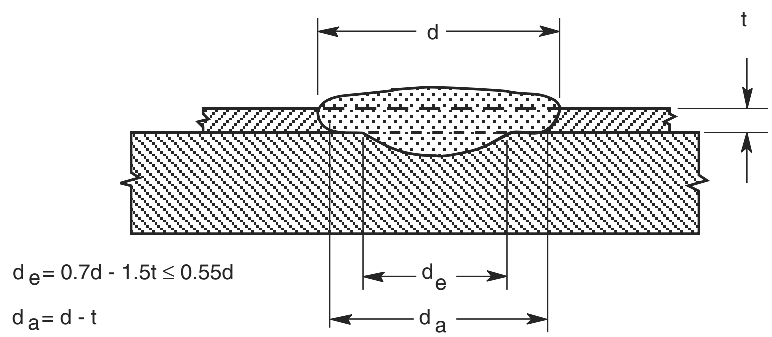


Figure E2.2.2.1-1 Arc Spot Weld – Single Thickness of Sheet

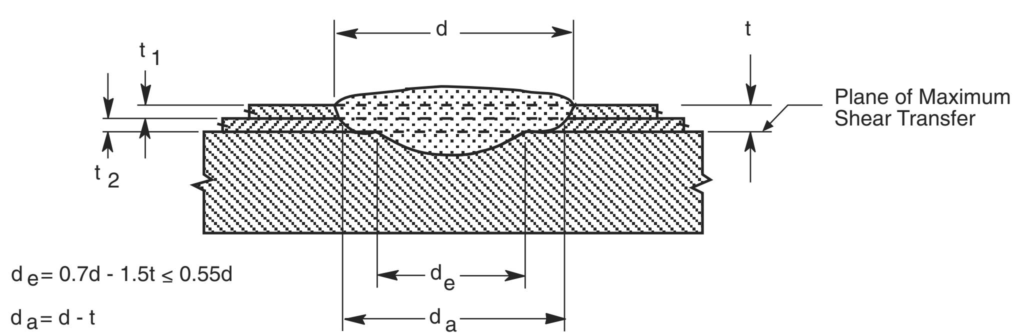


Figure E2.2.2.1-2 Arc Spot Weld – Double Thickness of Sheet

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 E2.2.2.1-1 and E2.2.2.1-2 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

E2.2.2.2 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. E2.2.2.2-1. 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.

$$P_n = 1.65 t d_a F_u \qquad (Eq.\ E2.2.2.2\text{-}1)$$

Ω = 2.20 (*ASD*)

ϕ = 0.70 (*LRFD*)

= 0.60 (*LSD*)

where

P_n = *Nominal shear strength* [*resistance*] of sheet-to-sheet *connection*

t = Base steel *thickness* (exclusive of coatings) of single welded sheet

d_a = Average diameter of arc spot weld at mid-thickness of t. See Figure E2.2.2.2-1 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

$$= 0.7d - 1.5t \le 0.55d \qquad (Eq.\ E2.2.2.2\text{-}2)$$

F_u = *Tensile strength* of sheet as determined in accordance with Section A2.1 or A2.2

In addition, the following limits shall apply:

(a) $F_u \le$ 59 ksi (407 MPa or 4150 kg/cm^2),

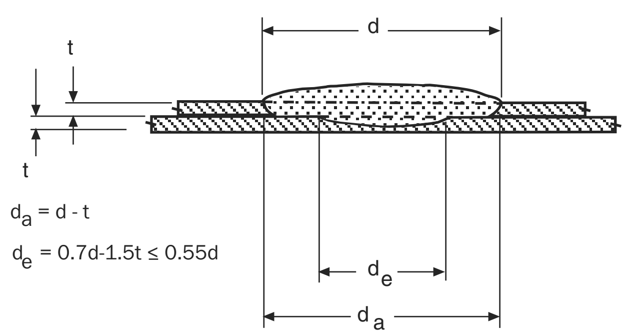


Figure E2.2.2.2-1 Arc Spot Weld – Sheet-to-Sheet

(b) $F_{xx} > F_u$, and

(c) 0.028 in. (0.71 mm) $\le t \le$ 0.0635 in. (1.61 mm).

See Section E2.2.2.1 for definition of F_{xx}.

E2.2.3 Tension

The uplift *nominal tensile strength* [*resistance*], P_n, of each concentrically loaded arc spot weld connecting sheet(s) and supporting member shall be computed as the smaller of either Eq. E2.2.3-1 or Eq. E2.2.3-2, 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.

$$P_n = \frac{\pi d_e^2}{4} F_{xx} \qquad (Eq.\ E2.2.3\text{-}1)$$

$$P_n = 0.8(F_u/F_y)^2\, t\, d_a F_u \qquad (Eq.\ E2.2.3\text{-}2)$$

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 (13.3 kN or 1360 kg),
$F_{xx} \ge$ 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 E2.2.2.1 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 shall be determined by using the sum of the sheet *thicknesses* as given by Eq. E2.2.3-2.

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 is permitted to be used provided the particular welding procedure used for making those welds is followed.

E2.2.4 Combined Shear and Tension on an Arc Spot Weld

For arc spot weld *connections* subjected to a combination of shear and tension, Section E2.2.4.1 or Section E2.2.4.2 shall be applied. In addition, the following limitations shall be satisfied:

$F_u \le$ 105 ksi (724 MPa or 7380 kg/cm^2)
$F_{xx} \ge$ 60 ksi (414 MPa or 4220 kg/cm^2)

$td_aF_u \le 3$ kips (13.3 kN or 1360 kg)
$F_u/F_y \ge 1.02$
0.47 in. (11.9 mm) $\le d \le$ 1.02 in. (25.9 mm)
See Section E2.2.2.1 for definition of variables.

E2.2.4.1 ASD Method

For arc spot weld *connections* subjected to a combination of shear and tension forces, the following requirements shall be met for *ASD*:

If $\left(\dfrac{\Omega_t T}{P_{nt}}\right)^{1.5} \le 0.15$, no interaction check is required.

If $\left(\dfrac{\Omega_t T}{P_{nt}}\right)^{1.5} > 0.15$,

$$\left(\frac{\Omega_s Q}{P_{ns}}\right)^{1.5} + \left(\frac{\Omega_t T}{P_{nt}}\right)^{1.5} \le 1 \qquad (Eq.\ \text{E2.2.4.1-1})$$

where
Ω_t = Corresponding *safety factor* for P_{nt} given by Section E2.2.3
T = *Required allowable tensile strength* per *connection* fastener
P_{nt} = *Nominal tension strength* as given by Section E2.2.3
Ω_s = Corresponding *safety factor* for P_{ns} given by Section E2.2.2
Q = *Required allowable shear strength* per *connection* fastener
P_{ns} = *Nominal shear strength* as given by Section E2.2.2

E2.2.4.2 LRFD and LSD Methods

For arc spot weld *connections* subjected to a combination of shear and tension forces, the following requirements shall be met for *LRFD* or *LSD*:

If $\left(\dfrac{\overline{T}}{\phi_t P_{nt}}\right)^{1.5} \le 0.15$, no interaction check is required.

If $\left(\dfrac{\overline{T}}{\phi_t P_{nt}}\right)^{1.5} > 0.15$,

$$\left(\frac{\overline{Q}}{\phi_s P_{ns}}\right)^{1.5} + \left(\frac{\overline{T}}{\phi_t P_{nt}}\right)^{1.5} \le 1 \qquad (Eq.\ \text{E2.2.4.2-1})$$

where
$\overline{T}$ = *Required tensile strength* [tensile force due to *factored loads*] per *connection* fastener
= T_u for *LRFD*
= T_f for *LSD*
ϕ_t = *Resistance factor* corresponding to P_{nt} as given by Section E2.2.3
P_{nt} = *Nominal tension strength* [*resistance*] as given by Section E2.2.3

P_{ns} = *Nominal shear strength* [*resistance*] as given by Section E2.2.2

$\overline{Q}$ = *Required shear strength* [shear force due to *factored loads*] per *connection* fastener

= Q_u for *LRFD*

= Q_f for *LSD*

ϕ_s = *Resistance factor* corresponding to P_{ns} as given by Section E2.2.2

E2.3 Arc Seam Welds

Arc seam welds covered by this *Specification* shall apply only to the following *joints*:

(a) Sheet to thicker supporting member in the flat position (See Figure E2.3-1), and
(b) Sheet to sheet in the horizontal or flat position.

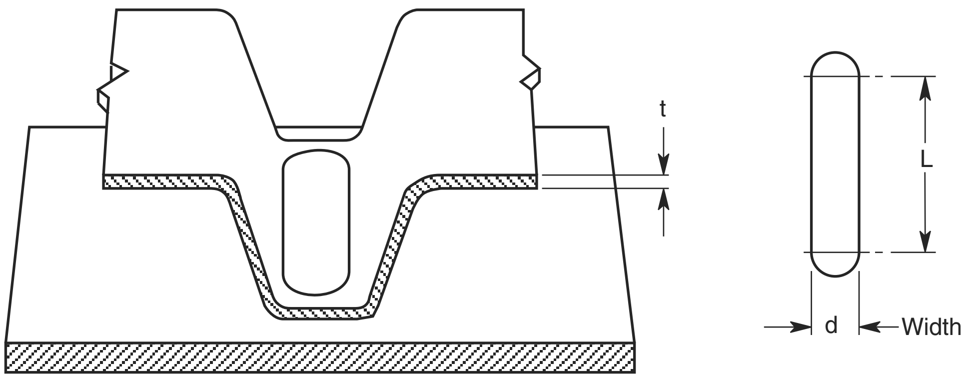


Figure E2.3-1 Arc Seam Welds – Sheet to Supporting Member in Flat Position

E2.3.1 Minimum Edge and End Distance

The distance from the center line of an arc seam weld to the end or edge of the connected member shall not be less than 1.5d. In no case shall the clear distance between welds and the end or edge of the member be less than 1.0d, where d is the visible width of the arc seam weld. See Figure E2.3.1-1 for details.

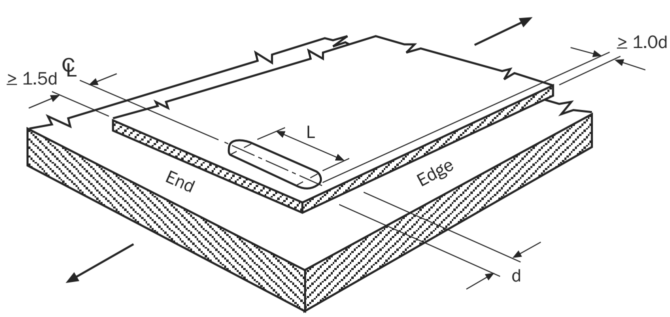


Figure E2.3.1-1 End and Edge Distances for Arc Seam Welds

E2.3.2 Shear

E2.3.2.1 Shear Strength [Resistance] for Sheet(s) Welded to a Thicker Supporting Member

The *nominal shear strength* [*resistance*], P_n, of arc seam welds shall be determined by using the smaller of either Eq. E2.3.2.1-1 or Eq. E2.3.2.1-2. 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.

$$P_n = \left[\frac{\pi d_e^2}{4} + L d_e\right] 0.75 F_{xx} \qquad (Eq.\ E2.3.2.1\text{-}1)$$

$$P_n = 2.5 t F_u (0.25L + 0.96 d_a) \qquad (Eq.\ E2.3.2.1\text{-}2)$$

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 arc seam weld at fused surfaces

$$= 0.7d - 1.5t \qquad (Eq.\ E2.3.2.1\text{-}3)$$

where

d = Visible width of arc seam weld

L = Length of seam weld not including circular ends (For computation purposes, L shall not exceed 3d)

d_a = Average width of arc seam weld

$$= (d - t) \text{ for single or double sheets} \qquad (Eq.\ E2.3.2.1\text{-}4)$$

F_u, F_{xx}, and t = Values as defined in Section E2.2.2.1

E2.3.2.2 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. E2.3.2.2-1. 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.

$$P_n = 1.65 t d_a F_u \qquad (Eq.\ E2.3.2.2\text{-}1)$$

Ω = 2.20 (*ASD*)

ϕ = 0.70 (*LRFD*)

= 0.60 (*LSD*)

where

P_n = *Nominal shear strength* [*resistance*] of sheet-to-sheet *connection*

d_a = Average width of arc seam weld at mid-*thickness* of t. See Figure E2.3.2.2-1 for width definitions.

= (d – t)

where
d = Visible width of the outer surface of arc seam weld
t = Base steel *thickness* (exclusive of coatings) of single welded sheet
F_u = *Tensile strength* of sheet as determined in accordance with Section A2.1 or A2.2

In addition, the following limits shall apply:
(a) $F_u \leq$ 59 ksi (407 MPa or 4150 kg/cm^2)
(b) $F_{xx} > F_u$
(c) 0.028 in. (0.711 mm) ≤ t ≤ 0.0635 in. (1.61 mm)

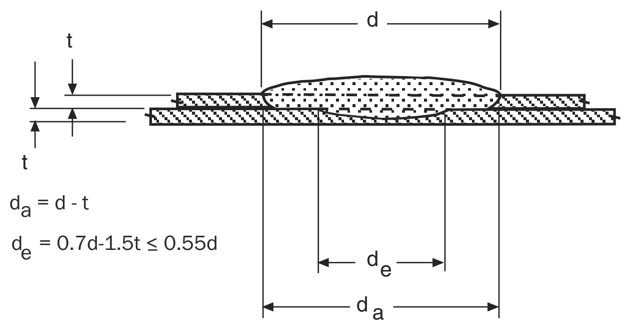


Figure E2.3.2.2-1 Arc Seam Weld – Sheet-to-Sheet

E2.4 Top Arc Seam Sidelap Welds

E2.4.1 Shear Strength [Resistance] of Top Arc Seam Sidelap Welds

The *nominal shear strength* [*resistance*] for longitudinal loading of *top arc seam sidelap* welds shall be determined in accordance with Eq. E2.4.1-1. The following limits shall apply:
(a) $h_{st} \leq$ 1.25 in. (31.8 mm),
(b) $F_{xx} \geq$ 60 ksi (414 MPa),
(c) 0.028 in. (0.711 mm) ≤ t ≤ 0.064 in. (1.63 mm),
(d) 1.0 in. (25.4 mm) ≤ L_w ≤ 2.5 in. (63.5 mm).

where
h_{st} = Nominal seam height. See Figure E2.4.1-1
F_{xx} = *Tensile strength* of electrode classification
L_w = Length of *top arc seam sidelap weld*
t = Base steel *thickness* (exclusive of coatings) of thinner connected sheet

$$P_n = [4.0(F_u/F_{sy})-1.52](t/L_w)^{0.33}L_w t F_u \qquad (Eq.\ E2.4.1\text{-}1)$$

Ω = 2.60 (*ASD*)
ϕ = 0.60 (*LRFD*)
= 0.55 (*LSD*)

where
P_n = *Nominal shear strength* [*resistance*] of *top arc seam sidelap weld*
F_u = Specified minimum *tensile strength* of connected sheets as determined in accordance with Section A2.3.1, A2.3.2 or A2.3.3

F_{sy} = *Specified minimum yield stress* of connected sheets as determined in accordance with Section A2.3.1, A2.3.2 or A2.3.3

It is permitted to exclude the *connection* design reduction specified in Sections A2.3.2, A2.3.3(b) and A2.3.3(c) for *top arc seam welds* provided the arc seam welds meet minimum spacing requirements along steel deck *diaphragm* side laps.

The minimum end distance and the weld spacing shall satisfy the shear rupture requirements in Section E6.

The *top arc seam sidelap weld connection* shall be made as follows:

(1) Vertical legs in either vertical leg and overlapping hem *joints* or vertical leg *joints* fit snugly, and

(2) In hem *joints*, the overlapping hem is crimped onto the vertical leg and the crimp length shall be longer than the specified weld length, L_w.

Holes or openings in the hem at either one or both ends of the weld are permitted.

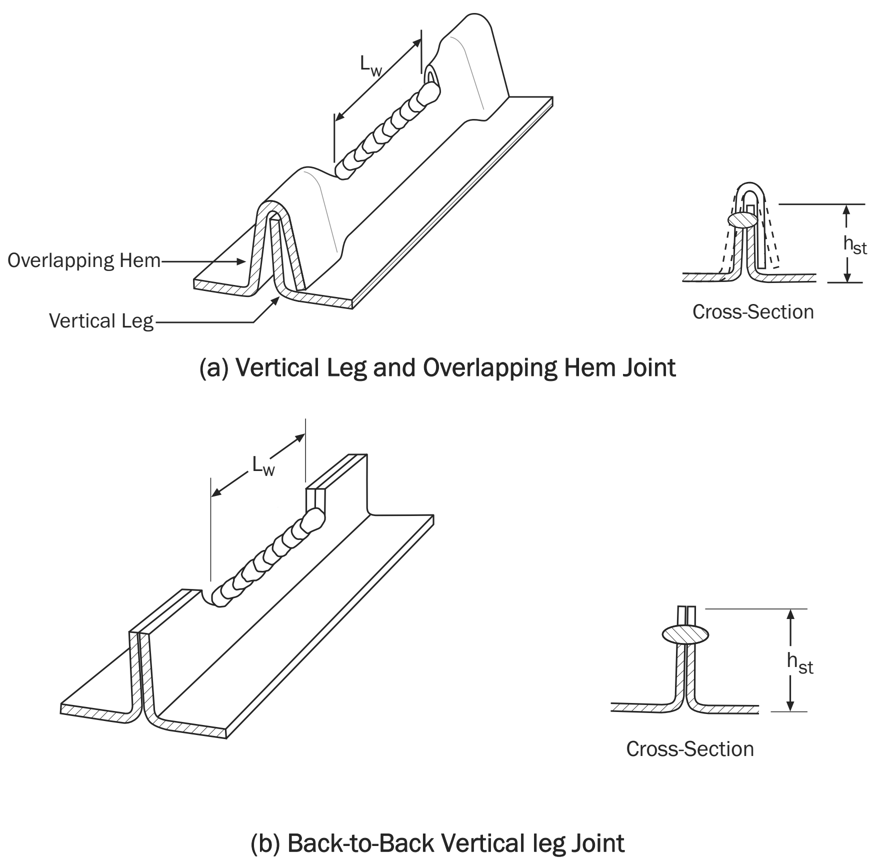


Figure E2.4.1-1 Top Arc Seam Sidelap Weld

E2.5 Fillet Welds

Fillet welds covered by this *Specification* shall apply to the welding of *joints* in any position, either:

(a) Sheet to sheet, or

(b) Sheet to thicker steel member.

The *nominal shear strength* [*resistance*], P_n, of a fillet weld shall be the lesser of P_{n1} and P_{n2} as 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 $L/t < 25$

$$P_{n1} = \left(1 - \frac{0.01L}{t_1}\right) L t_1 F_{u1} \qquad (Eq.\ E2.5\text{-}1)$$

$$P_{n2} = \left(1 - \frac{0.01L}{t_2}\right) L t_2 F_{u2} \qquad (Eq.\ E2.5\text{-}2)$$

Ω = 2.55 (*ASD*)
ϕ = 0.60 (*LRFD*)
= 0.50 (*LSD*)

For $L/t \geq 25$

$$P_{n1} = 0.75\, t_1 L F_{u1} \qquad (Eq.\ E2.5\text{-}3)$$

$$P_{n2} = 0.75\, t_2 L F_{u2} \qquad (Eq.\ E2.5\text{-}4)$$

Ω = 3.05 (*ASD*)
ϕ = 0.50 (*LRFD*)
= 0.40 (*LSD*)

(2) For transverse loading:

$$P_{n1} = t_1 L F_{u1} \qquad (Eq.\ E2.5\text{-}5)$$

$$P_{n2} = t_2 L F_{u2} \qquad (Eq.\ E2.5\text{-}6)$$

Ω = 2.35 (*ASD*)
ϕ = 0.65 (*LRFD*)
= 0.60 (*LSD*)

where

t_1, t_2 = *Thickness* of connected parts, as shown in Figures E2.5-1 and E2.5-2
t = Lesser value of t_1 and t_2
F_{u1}, F_{u2} = *Tensile strength* of connected parts corresponding to *thicknesses* t_1 and t_2
P_{n1}, P_{n2} = *Nominal shear strength* [*resistance*] corresponding to connected *thicknesses* t_1 and t_2

In addition, for $t > 0.10$ in. (2.54 mm), the *nominal strength* [*resistance*] determined in accordance with (1) and (2) shall not exceed the following value of P_n:

$$P_n = 0.75\, t_w L F_{xx} \qquad (Eq.\ E2.5\text{-}7)$$

Ω = 2.55 (*ASD*)
ϕ = 0.60 (*LRFD*)
= 0.50 (*LSD*)

where

P_n = *Nominal fillet weld strength* [*resistance*]
L = Length of fillet weld
F_{xx} = Tensile strength of electrode classification

t_w = Effective throat
= $0.707\ w_1$ or $0.707\ w_2$, whichever is smaller. A larger effective throat is permitted if measurement shows that the welding procedure to be used consistently yields a larger value of t_w.

where

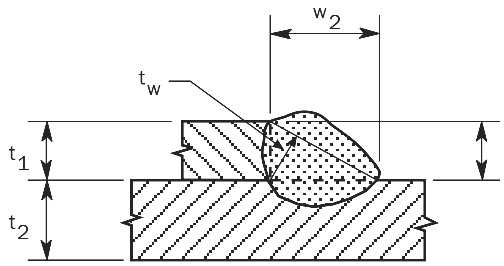


Figure E2.5-1 Fillet Welds – Lap Joint

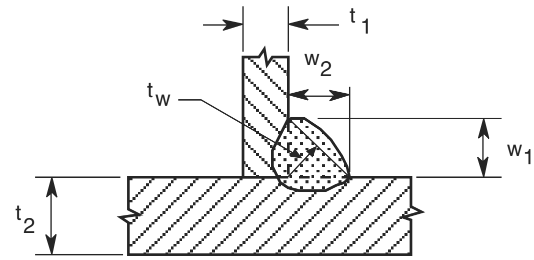


Figure E2.5-2 Fillet Welds – T-Joint

w_1, w_2 = leg of weld (see Figures E2.5-1 and E2.5-2) and $w_1 \le t_1$ in lap *joints*

E2.6 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.

Larger effective throat thicknesses, t_w, than those determined by Eq. E2.6-5 or Eq. E2.6-7, as appropriate, are permitted, provided the fabricator can establish by qualification the consistent production of such larger effective throat thicknesses. Qualification shall consist of sectioning the weld normal to its axis, at mid-length and terminal ends. Such sectioning shall be made on a number of combinations of material sizes representative of the range to be used in the fabrication.

(a) For flare-bevel groove welds, transverse loading (see Figure E2.6-1):

$P_n = 0.833tLF_u$ (*Eq.* E2.6-1)

Ω = 2.55 (*ASD*)

ϕ = 0.60 (*LRFD*)

= 0.50 (*LSD*)

(b) For flare groove welds, longitudinal loading (see Figures E2.6-2 and E2.6-3):

(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.* E2.6-2)

Ω = 2.80 (*ASD*)

ϕ = 0.55 (*LRFD*)

= 0.45 (*LSD*)

(2) For $t_w \ge 2t$ with the lip height, h, equal to or greater than weld length, L:

$P_n = 1.50tLF_u$ (*Eq.* E2.6-3)

Ω = 2.80 (*ASD*)
ϕ = 0.55 (*LRFD*)
= 0.45 (*LSD*)

(c) For t > 0.10 in. (2.54 mm), the *nominal strength* [*resistance*] determined in accordance with (a) or (b) shall not exceed the value of P_n calculated in accordance with Eq. E2.6-4.

$$P_n = 0.75 t_w L F_{xx} \quad (Eq.\ E2.6\text{-}4)$$

Ω = 2.55 (*ASD*)
ϕ = 0.60 (*LRFD*)
= 0.50 (*LSD*)

where
P_n = *Nominal flare groove weld strength* [*resistance*]
t = *Thickness* of welded member as illustrated in Figures E2.6-1 to E2.6-3
L = Length of weld
F_u and F_{xx} = Values as defined in Section E2.2.2.1
h = Height of lip
t_w = Effective throat of flare groove weld determined using Eqs. E2.6-5 or E2.6-7

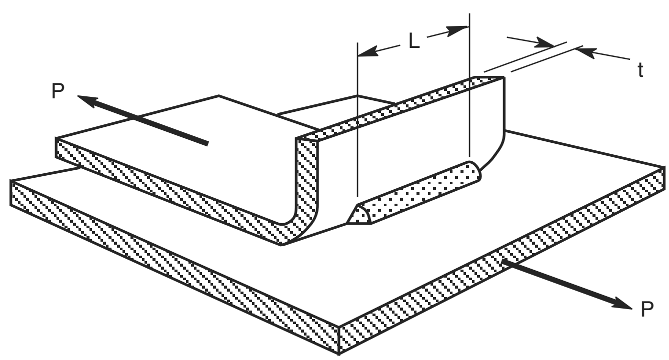
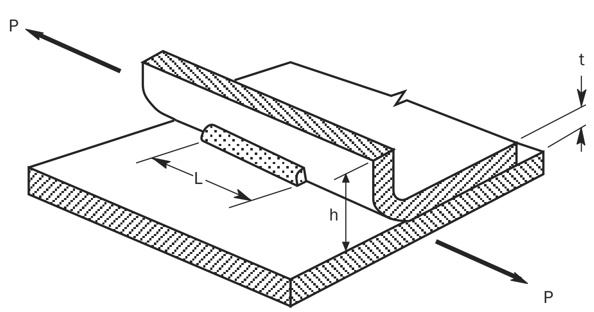


Figure E2.6-1 Flare-Bevel Groove Weld



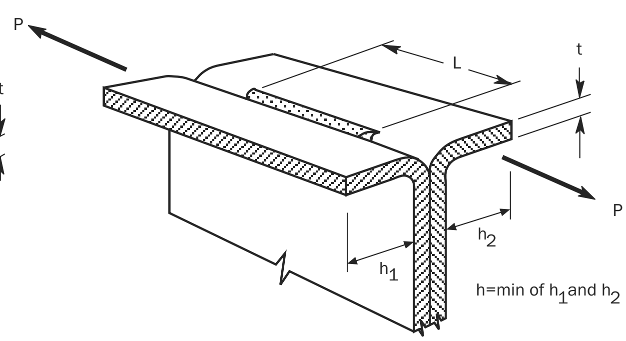


Figure E2.6-2 Shear in Flare-Bevel Groove Weld

Figure E2.6-3 Shear in Flare V-Groove Weld

(i) For a flare-bevel groove weld

$$t_w = \left[w_2 + t_{wf} - R + \sqrt{2Rw_1 - w_1^2}\right]\left(\frac{w_1}{w_f}\right) - R\,\eta\left(\frac{w_2}{w_f}\right) \qquad (Eq.\ E2.6\text{-}5)$$

where

w_1, w_2 = Leg of weld (see Figure E2.6-4)

t_{wf} = Effective throat of groove weld that is filled flush to the surface, $w_1 = R$, determined in accordance with Table E2.6-1

R = Radius of outside bend surface

η = [1 – cos(equivalent angle)] determined in accordance with Table E2.6-1

w_f = Face width of weld

$$= \sqrt{w_1^2 + w_2^2} \qquad (Eq.\ E2.6\text{-}6)$$

Table E2.6-1
Flare-Bevel Groove Welds

Welding Process	Throat Depth (t_{wf})	η
SMAW, FCAW-S[1]	5/16 R	0.274
GMAW, FCAW-G[2]	5/8 R	0.073
SAW	5/16 R	0.274

Note:
[1] In Canada, FCAW-S is known as FCAW (self shielded).
[2] In Canada, FCAW-G is known as FCAW (gas shielded).

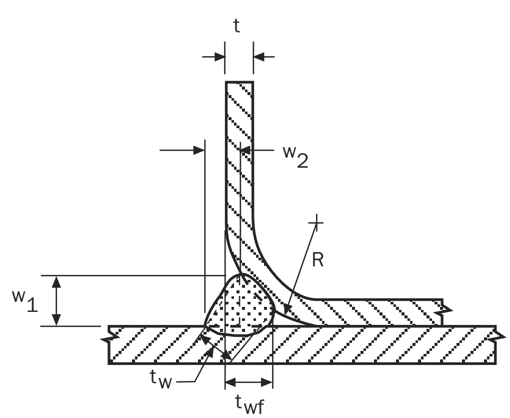


Figure E2.6-4 Flare-Bevel Groove Weld

(ii) For a flare V-groove weld

t_w = smaller of $(t_{wf} - d_1)$ and $(t_{wf} - d_2)$ (*Eq*. E2.6-7)

where

d_1 and d_2 = Weld offset from flush condition (see Figure E2.6-5)

t_{wf} = Effective throat of groove weld that is filled flush to the surface (i.e. $d_1 = d_2 = 0$), determined in accordance with Table E2.6-2

R_1 and R_2 = Radius of outside bend surface as illustrated in Figure E2.6-5

Table E2.6-2
Flare V-Groove Welds

Welding Process	Throat Depth (t_{wf})
SMAW, FCAW-S[1]	5/8 R
GMAW, FCAW-G[2]	3/4 R
SAW	1/2 R
Note: R is the lesser of R_1 and R_2	

Note:
[1] In Canada, FCAW-S is known as FCAW (self shielded).
[2] In Canada, FCAW-G is known as FCAW (gas shielded).

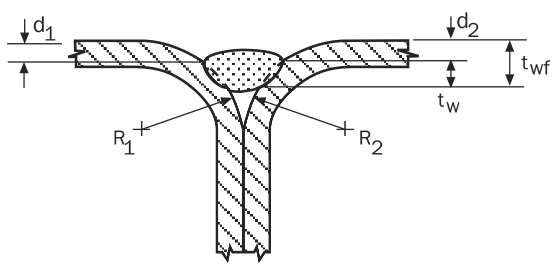


Figure E2.6-5 Flare V-Groove Weld

E2.7 Resistance Welds

The *nominal shear strength* [*resistance*], P_n, of resistance (spot) welds shall be determined in accordance with this section. The *safety factor* and *resistance factors* given in this section shall be used to determine the *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. $\leq$ t < 0.14 in.

$$P_n = 144t^{1.47} \quad (Eq.\ E2.7\text{-}1)$$

For 0.14 in. $\leq$ t $\leq$ 0.18 in.

$$P_n = 43.4t + 1.93 \quad (Eq.\ E2.7\text{-}2)$$

When t is in millimeters and P_n is in kN:

For 0.25 mm $\leq$ t < 3.56 mm

$$P_n = 5.51t^{1.47} \quad (Eq.\ E2.7\text{-}3)$$

For 3.56 mm $\leq$ t $\leq$ 4.57 mm

$$P_n = 7.6t + 8.57 \quad (Eq.\ E2.7\text{-}4)$$

When t is in centimeters and P_n is in kg:

For 0.025 cm $\leq$ t < 0.356 cm

$$P_n = 16600t^{1.47} \quad (Eq.\ E2.7\text{-}5)$$

For 0.356 cm ≤ t ≤ 0.457 cm

$$P_n = 7750t + 875 \quad (Eq.\ E2.7\text{-}6)$$

where

P_n = *Nominal resistance weld strength* [*resistance*]

t = *Thickness* of thinnest outside sheet

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 3/16 in. (4.76 mm) or less. For bolted *connections* in which the *thickness* of the thinnest connected part is greater than 3/16 in. (4.76 mm), the 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, *Standard Specification for Carbon and Alloy Steel Nuts for Bolts for High-Pressure and High-Temperature Service, or Both*

ASTM A307 (Type A), *Standard Specification for Carbon Steel Bolts and Studs, 60,000 PSI Tensile Strength*

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

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

ASTM A354 (Grade BD), *Standard Specification for Quenched and Tempered Alloy Steel Bolts, Studs, and Other Externally Threaded Fasteners* (for diameter of bolt smaller than 1/2 in.)

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

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

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

ASTM A563, *Standard Specification for Carbon and Alloy Steel Nuts*

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

ASTM F436, *Standard Specification for Hardened Steel Washers*

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

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

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

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

When bolts, nuts, and washers other than the above are used, drawings shall clearly indicate 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*.

The holes for bolts shall not exceed the sizes specified in Table E3, except that larger holes are permitted to be used in column base details or structural systems connected to concrete walls.

TABLE E3
Maximum Size of Bolt Holes, in Inches

Nominal Bolt Diameter, d in.	Standard Hole Diameter, d_h in.	Oversized Hole Diameter, d_h in.	Short-Slotted Hole Dimensions in.	Long-Slotted Hole Dimensions in.	Alternative Short-Slotted Hole* 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)	
≥ 1/2	d + 1/16	d + 1/8	(d + 1/16) by (d + 1/4)	(d + 1/16) by (2 1/2 d)	9/16 by 7/8

Note: * The alternative short-slotted hole is only applicable for d=1/2 in.

TABLE E3
Maximum Size of Bolt Holes, in Millimeters

Nominal Bolt Diameter, d mm	Standard Hole Diameter, d_h mm	Oversized Hole Diameter, d_h mm	Short-Slotted Hole Dimensions mm	Long-Slotted Hole Dimensions mm	Alternative Short-Slotted Hole* 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)	14.3 by 22.2

Note: * The alternative short-slotted hole is only applicable for d=12.7 mm.

The distance between the centers of fasteners shall not be less than 3d. In addition, the minimum distance between centers of bolt holes shall provide clearance for bolt heads, nuts, washers and the wrench. For oversized and slotted holes, the clear distance between the edges of two adjacent holes shall not be less than 2d.

E3.2 Minimum Edge and End Distances

The distance from the center of a fastener to the edge or end of any part shall not be less than 1.5d. For oversized and slotted holes, the distance between the edge of the hole and the edge or end of the member shall not be less than d.

E3.3 Bearing

The *available bearing strength* [*factored 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 Bearing 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 is 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

	Connections With Standard Holes		Connections With Oversized or Short-Slotted Holes	
Thickness of Connected Part, t, in. (mm)	Ratio of Fastener Diameter to Member Thickness, d/t	C	Ratio of Fastener Diameter to Member Thickness, d/t	C
$0.024 \le t < 0.1875$ ($0.61 \le t < 4.76$)	$d/t < 10$	3.0	$d/t < 7$	3.0
	$10 \le d/t \le 22$	$4 - 0.1(d/t)$	$7 \le d/t \le 18$	$1+14/(d/t)$
	$d/t > 22$	1.8	$d/t > 18$	1.8

Note: Oversized or short-slotted holes within the lap of lapped or nested Z-members as defined in Section E3 are permitted to be considered as standard holes.

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 Using Standard Holes With Washers Under both Bolt Head and Nut	1.00
Single Shear and Outside Sheets of Double Shear Connection Using Standard Holes Without Washers Under Both Bolt Head and Nut, or With Only One Washer	0.75
Single Shear and Outside Sheets of Double Shear Connection Using Oversized or Short-Slotted Holes Parallel to the Applied Load Without Washers Under Both Bolt Head and Nut, or With Only One Washer	0.70
Single Shear and Outside Sheets of Double Shear Connection Using Short-Slotted Holes Perpendicular to the Applied Load Without Washers Under Both Bolt Head and Nut, or With Only One Washer	0.55
Inside Sheet of Double Shear Connection Using Standard Holes With or Without Washers	1.33
Inside Sheet of Double Shear Connection Using Oversized or Short-Slotted Holes Parallel to the Applied Load With or Without Washers	1.10
Inside Sheet of Double Shear Connection Using Short-Slotted Holes Perpendicular to the Applied Load With or Without Washers	0.90

Note: Oversized or short-slotted holes within the lap of lapped or nested Z-members as defined in Section E3 are permitted to be considered as standard holes.

E3.3.2 Bearing 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* [*factored resistance*] shall not exceed the *available strength* [*factored resistance*] obtained in accordance with Section E3.3.1.

$$P_n = (4.64\alpha t + 1.53)dtF_u \qquad (Eq.\ E3.3.2\text{-}1)$$

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 Appendix A or B for provisions provided in this section. ☜A,B

E4 Screw Connections

All provisions in Section E4 shall apply to screws with 0.08 in. (2.03 mm) $\leq d \leq$ 0.25 in. (6.35 mm). The screws shall be thread-forming or thread-cutting, with or without a self-drilling point. Screws shall be installed and tightened in accordance with the manufacturer's recommendations.

The *nominal screw connection strengths* [*resistances*] shall also be limited by Section C2.

For *diaphragm* applications, Section D5 shall be used.

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 are permitted to be based on tests, with the *safety factor*, Ω, and the *resistance factor*, ϕ, determined in accordance with 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*] of sheet 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*] of sheet per screw
P_{nov} = *Nominal pull-over strength* [*resistance*] of sheet 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 or end of any part shall not be less than 1.5d.

E4.3 Shear

E4.3.1 Shear Strength [Resistance] Limited by Tilting and Bearing

The *nominal shear strength* [*resistance*] of sheet per screw, P_{ns}, shall be determined in accordance with this section.

For $t_2/t_1 \le 1.0$, P_{ns} shall be taken as the smallest of

$$P_{ns} = 4.2\,(t_2^3 d)^{1/2} F_{u2} \quad (Eq.\ E4.3.1\text{-}1)$$

$$P_{ns} = 2.7\, t_1\, d\, F_{u1} \quad (Eq.\ E4.3.1\text{-}2)$$

$$P_{ns} = 2.7\, t_2\, d\, F_{u2} \quad (Eq.\ E4.3.1\text{-}3)$$

For $t_2/t_1 \ge 2.5$, P_{ns} shall be taken as the smaller of

$$P_{ns} = 2.7\, t_1\, d\, F_{u1} \quad (Eq.\ E4.3.1\text{-}4)$$

$$P_{ns} = 2.7\, t_2\, d\, F_{u2} \quad (Eq.\ E4.3.1\text{-}5)$$

For $1.0 < t_2/t_1 < 2.5$, P_{ns} shall be calculated by linear interpolation between the above two cases.

E4.3.2 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* is 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*).

E4.4 Tension

For screws that carry tension, the head of the screw or washer, if a washer is provided, shall have a diameter d_h or d_w not less than 5/16 in. (7.94 mm). The nominal washer thickness shall be at least 0.050 in. (1.27 mm) for t_1 greater than 0.027 in. (0.686 mm) and at least 0.024 in. (0.610 mm) for t_1 equal to or less than 0.027 in. (0.686 mm). The washer shall be at least 0.063 in. (1.60 mm) thick when 5/8 in. (15.9 mm) $< d_w \le$ 3/4 in. (19.1 mm).

E4.4.1 Pull-Out Strength [Resistance]

The *nominal pull-out strength* [*resistance*] of sheet per screw, P_{not}, shall be calculated as follows:

$$P_{not} = 0.85\, t_c\, d\, F_{u2} \quad (Eq.\ E4.4.1\text{-}1)$$

E4.4.2 Pull-Over Strength [Resistance]

The *nominal pull-over strength* [*resistance*] of sheet per screw, P_{nov}, shall be calculated as follows:

$$P_{nov} = 1.5 t_1 d'_w F_{u1} \quad (Eq.\ E4.4.2\text{-}1)$$

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)), pancake screw washer head (Figure E4.4.2(2)), or hex washer head (Figure E4.4.2(3)) screw with an independent and solid steel washer beneath the screw head:

$$d'_w = d_h + 2t_w + t_1 \le d_w \qquad (Eq.\ E4.4.2\text{-}2)$$

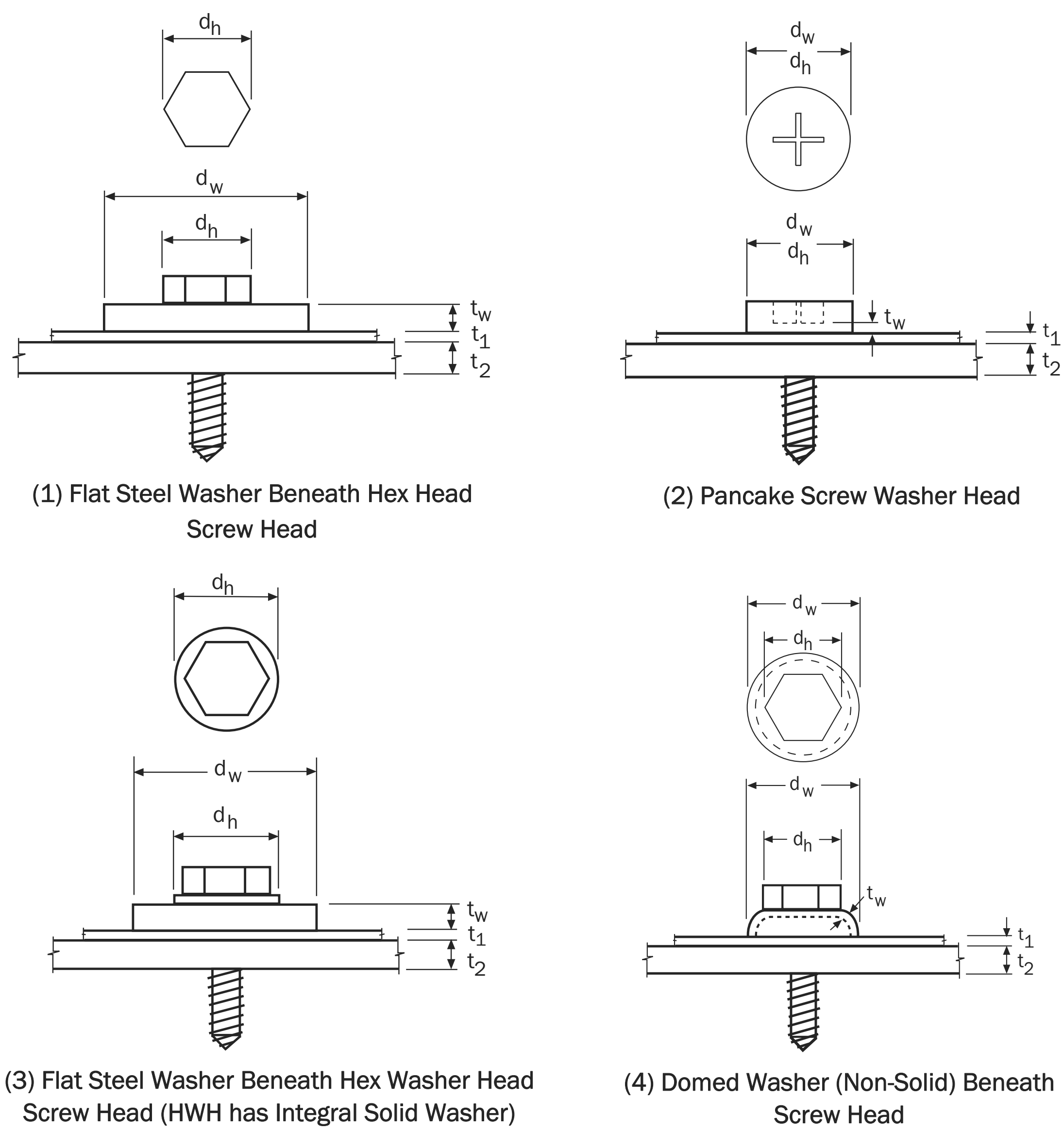
where

t_w = Steel washer thickness

(b) For a round head, a hex head, or a hex washer head screw without an independent washer beneath the screw head:

$d'_w = d_h$ but not larger than 3/4 in. (19.1 mm)

(c) For a domed (non-solid and either independent or integral) washer beneath the screw head (Figure E4.4.2(4)), it is permitted to use d'_w as calculated in Eq. E4.4.2-2, where t_w is the thickness of the domed washer. In the equation, d'_w shall not exceed 3/4 in. (19.1 mm).



(1) Flat Steel Washer Beneath Hex Head Screw Head

(2) Pancake Screw Washer Head

(3) Flat Steel Washer Beneath Hex Washer Head Screw Head (HWH has Integral Solid Washer)

(4) Domed Washer (Non-Solid) Beneath Screw Head

Figure E4.4.2 Screw Pull-Over With Washer

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* is 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*).

E4.5 Combined Shear and Tension

E4.5.1 Combined Shear and Pull-Over

E4.5.1.1 ASD Method

The following requirement shall be met:

$$\frac{Q}{P_{ns}} + 0.71\frac{T}{P_{nov}} \leq \frac{1.10}{\Omega} \qquad (Eq. \text{ E4.5.1.1-1})$$

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* per *connection* screw

T = *Required allowable tension strength* per *connection* screw

P_{ns} = *Nominal shear strength* of sheet per screw

= $2.7t_1dF_{u1}$ (*Eq*. E4.5.1.1-2)

P_{nov} = *Nominal pull-over strength of* sheet per screw

= $1.5t_1d_wF_{u1}$ (*Eq*. E4.5.1.1-3)

where

d_w = Larger of screw head diameter or washer diameter

Ω = 2.35

Eq. E4.5.1.1-1 shall be valid for *connections* that meet the following limits:

(a) 0.0285 in. (0.724 mm) $\leq t_1 \leq$ 0.0445 in. (1.13 mm),

(b) No. 12 and No. 14 self-drilling screws with or without washers,

(c) $d_w \leq$ 0.75 in. (19.1 mm),

(d) Washer dimension limitations of Section E4.4 apply,

(e) $F_{u1} \leq$ 70 ksi (483 MPa or 4920 kg/cm^2), and

(f) $t_2/t_1 \geq 2.5$.

For eccentrically loaded *connections* that produce a non-uniform pull-over force on the screw, the *nominal pull-over strength* shall be taken as 50 percent of P_{nov}.

E4.5.1.2 LRFD and LSD Methods

The following requirements shall be met:

$$\frac{\overline{Q}}{P_{ns}} + 0.71\frac{\overline{T}}{P_{nov}} \leq 1.10\phi \qquad (Eq. \text{ E4.5.1.2-1})$$

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* [shear force due to *factored loads*] per *connection* screw
= V_u for *LRFD*
= V_f for *LSD*

$\overline{T}$ = *Required tension strength* [tensile force due to *factored loads*] per *connection* screw
= T_u for *LRFD*
= T_f for *LSD*

P_{ns} = *Nominal shear strength* [*resistance*] of sheet per screw

$$= 2.7t_1dF_{u1} \quad (Eq.\ E4.5.1.2\text{-}2)$$

P_{nov} = *Nominal pull-over strength* [*resistance*] of sheet per screw

$$= 1.5t_1d_wF_{u1} \quad (Eq.\ E4.5.1.2\text{-}3)$$

where
d_w = Larger of screw head diameter or washer diameter

ϕ = 0.65 (*LRFD*)
= 0.55 (*LSD*)

Eq. E4.5.1.2-1 shall be valid for *connections* that meet the following limits:

(a) 0.0285 in. (0.724 mm) $\le t_1 \le$ 0.0445 in. (1.13 mm),
(b) No. 12 and No. 14 self-drilling screws with or without washers,
(c) $d_w \le$ 0.75 in. (19.1 mm),
(d) Washer dimension limitations of E4.4 apply,
(e) $F_{u1} \le$ 70 ksi (483 MPa or 4920 kg/cm^2), and
(f) $t_2/t_1 \ge 2.5$.

For eccentrically loaded *connections* that produce a non-uniform pull-over force on the screw, the *nominal pull-over strength* [*resistance*] shall be taken as 50 percent of P_{nov}.

E4.5.2 Combined Shear and Pull-Out

E4.5.2.1 ASD Method

The following requirement shall be met:

$$\frac{Q}{P_{ns}} + \frac{T}{P_{not}} \le \frac{1.15}{\Omega} \quad (Eq.\ E4.5.2.1\text{-}1)$$

In addition, Q and T shall not exceed the corresponding *allowable strength* determined by Sections E4.3 and E4.4, respectively.

where

P_{ns} = *Nominal shear strength* of sheet per screw

$$= 4.2(t_2^3d)^{1/2}F_{u2} \quad (Eq.\ E4.5.2.1\text{-}2)$$

P_{not} = *Nominal pull-out strength* of sheet per screw

$$= 0.85t_cdF_{u2} \quad (Eq.\ E4.5.2.1\text{-}3)$$

Ω = 2.55

Other variables are as defined in Section E4.5.1.1.

Eq. E4.5.2.1-1 shall be valid for *connections* that meet the following limits:

(a) 0.0297 in. (0.754 mm) $\le t_2 \le$ 0.0724 in. (1.84 mm),

(b) No. 8, 10, 12, or 14 self-drilling screws with or without washers,

(c) $F_{u2} \le$ 121 ksi (834MPa or 8510 kg/cm^2), and

(d) $1.0 \le F_u/F_y \le 1.62$.

E4.5.2.2 LRFD and LSD Methods

The following requirement shall be met:

$$\frac{\overline{Q}}{P_{ns}} + \frac{\overline{T}}{P_{not}} \le 1.15\phi \qquad (Eq.\ E4.5.2.2\text{-}1)$$

In addition, $\overline{Q}$ and $\overline{T}$ shall not exceed the corresponding *design strength* [*factored resistance*] determined by Sections E4.3 and E4.4, respectively.

where

P_{ns} = *Nominal shear strength* [*resistance*] of sheet per screw

$$= 4.2(t_2^3 d)^{1/2} F_{u2} \qquad (Eq.\ E4.5.2.2\text{-}2)$$

P_{not} = *Nominal pull-out strength* [*resistance*] of sheet per screw

$$= 0.85 t_c d F_{u2} \qquad (Eq.\ E4.5.2.2\text{-}3)$$

ϕ = 0.60 for *LRFD*

= 0.50 for *LSD*

Other variables are as defined in Section E4.5.1.2.

Eq. E4.5.2.2-1 shall be valid for *connections* that meet the following limits:

(a) 0.0297 in. (0.754 mm) $\le t_2 \le$ 0.0724 in. (1.84 mm),

(b) No. 8, 10, 12, or 14 self-drilling screws with or without washers,

(c) $F_{u2} \le$ 121 ksi (834MPa or 8510 kg/cm^2), and

(d) $1.0 \le F_u/F_y \le 1.62$.

E4.5.3 Combined Shear and Tension in Screws

E4.5.3.1 ASD Method

For screws subject to a combination of shear and tension forces, the following requirement shall be met:

$$\frac{Q}{P_{ss}} + \frac{T}{P_{ts}} \le \frac{1.3}{\Omega} \qquad (Eq.\ E4.5.3.1\text{-}1)$$

In addition, Q and T shall not exceed the corresponding *allowable strength* determined by Sections E4.3.2 and E4.4.2, respectively.

where

Q = *Required shear strength* determined in accordance with *ASD load combinations*

P_{ss} = *Nominal shear strength* of screw as reported by manufacturer or determined by independent laboratory testing

T = *Required tension strength* determined in accordance with *ASD load combinations*

P_{ts} = *Nominal tension strength* of screw as reported by manufacturer or determined by independent laboratory testing

Ω = *Safety factor* in accordance with Section E4

E4.5.3.2 LRFD and LSD Methods

For screws subject to a combination of shear and tension forces, the following requirement shall be met:

$$\frac{\overline{Q}}{P_{ss}} + \frac{\overline{T}}{P_{ts}} \le 1.3\phi \qquad \text{(Eq. E4.5.3.2-1)}$$

In addition, $\overline{Q}$ and $\overline{T}$ shall not exceed the corresponding *design strength* [*factored resistance*] determined by Sections E4.3.2 and E4.4.2, respectively.

where

$\overline{Q}$ = *Required shear strength* [shear force due to *factored loads*] determined in accordance with *LRFD* or *LSD load combinations*
= V_u for *LRFD*
= V_f for *LSD*

P_{ss} = *Nominal shear strength* [*resistance*] of screw as reported by manufacturer or determined by independent laboratory testing

$\overline{T}$ = *Required tension strength* [tensile force due to *factored loads*] determined in accordance with *LRFD* or *LSD load combinations*
= T_u for *LRFD*
= T_f for *LSD*

P_{ts} = *Nominal tension strength* [*resistance*] of screw as reported by manufacturer or determined by independent laboratory testing

ϕ = *Resistance factor* in accordance with Section E4

E5 Power-Actuated Fasteners

The provisions of this section shall apply to *power-actuated fasteners* (*PAFs*) that are driven into steel substrates. The *thickness* of the substrate not in contact with *PAF* head shall be limited to a maximum of 0.75 in. (19.1 mm). The *thickness* of the substrate in contact with *PAF* head shall be limited to a maximum of 0.06 in. (1.52 mm). The washer diameter shall not exceed 0.6 in. (15.2 mm) in computations, although the actual diameter may be larger. *Power-actuated fastener* diameter shall be limited to a range of 0.106 in. (2.69 mm) to 0.206 in. (5.23 mm).

For *diaphragm* applications, the provisions of Section D5 shall be used.

Alternatively, the *available strengths* [*factored resistances*] for any particular application are permitted to be determined through independent laboratory testing, with the *resistance factors*, ϕ, and *safety factors*, Ω, determined in accordance with Chapter F. The values of P_{ntp} and P_{nsp} are permitted to be reported by the manufacturer.

The following notation shall apply to Section E5:

a = Major diameter of tapered *PAF* head

d = Fastener diameter measured at near side of embedment
= d_s for *PAF* installed such that entire point is located behind far side of embedment material

d_{ae} = Average embedded diameter, computed as average of installed fastener diameters measured at near side and far side of embedment material
= d_s for *PAF* installed such that entire point is located behind far side of embedment material

d_s = Nominal shank diameter

d'_w = Actual diameter of washer or fastener head in contact with retained substrate $\leq$ 0.60 in. (15.2 mm) in computation

F_{bs} = Base *stress* parameter
= 66,000 psi (455 MPa or 4640 kg/cm^2)

F_{u1} = *Tensile strength* of member in contact with *PAF* head or washer

F_{u2} = *Tensile strength* of member not in contact with *PAF* head or washer

F_{uh} = *Tensile strength* of hardened *PAF* steel

F_{ut} = *Tensile strength* of non-hardened *PAF* steel

F_{y2} = *Yield stress* of member not in contact with *PAF* head or washer

HRC_p = Rockwell C hardness of *PAF* steel

ℓ_{dp} = *PAF* point length. See Figure E5

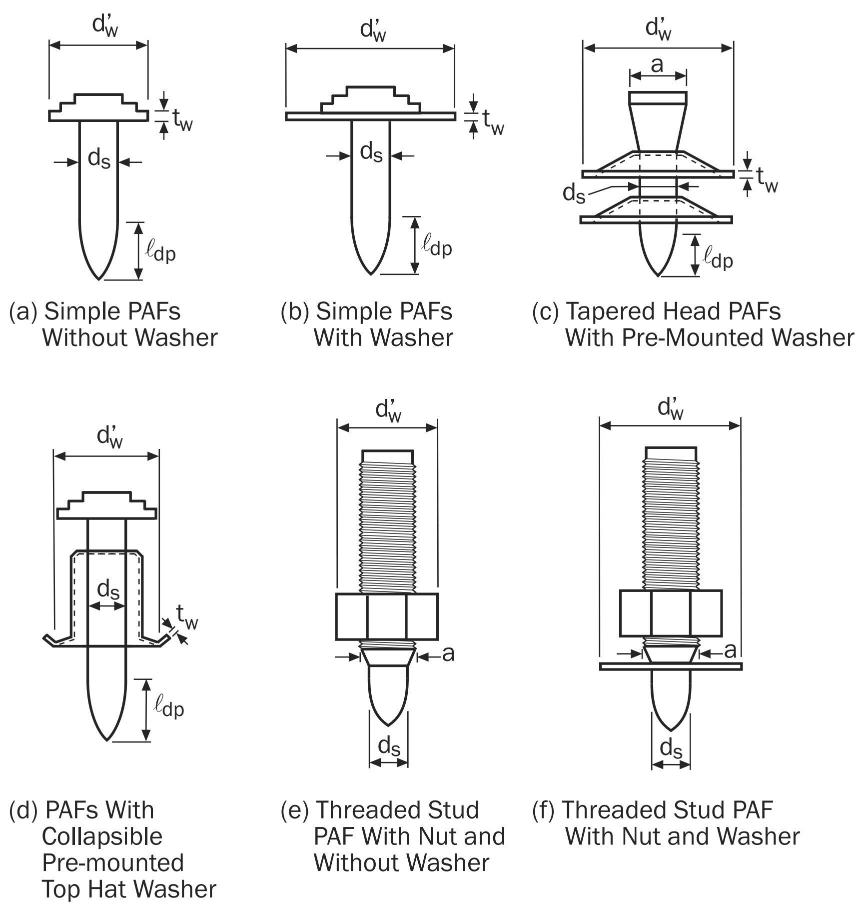
P_{nbp} = *Nominal bearing* and *tilting strength* [*resistance*] per *PAF*

P_{nsp} = *Nominal shear strength* [*resistance*] per *PAF*

P_{ntp} = *Nominal tensile strength* [*resistance*] per *PAF*

P_{not} = *Nominal pull-out strength* [*resistance*] in tension per *PAF*

P_{nos} = *Nominal pull-out strength* [*resistance*] in shear per *PAF*



(a) Simple PAFs Without Washer

(b) Simple PAFs With Washer

(c) Tapered Head PAFs With Pre-Mounted Washer

(d) PAFs With Collapsible Pre-mounted Top Hat Washer

(e) Threaded Stud PAF With Nut and Without Washer

(f) Threaded Stud PAF With Nut and Washer

Figure E5 Geometric Variables in Power-Actuated Fasteners

P_{nov} = *Nominal pull-over strength* [*resistance*] per *PAF*
P_{sp} = *Nominal shear strength* [*resistance*] of *PAF*
P_{tp} = *Nominal tensile strength* [*resistance*] of *PAF*
t_1 = *Thickness* of member in contact with *PAF* head or washer
t_2 = *Thickness* of member not in contact with *PAF* head or washer
t_w = Steel washer thickness

Various fastener dimensions used throughout Section E5 are shown in Figure E5.

E5.1 Minimum Spacing, Edge and End Distances

The minimum center-to-center spacing of the *PAFs* and the minimum distance from center of the fastener to any edge of the connected part, regardless of the direction of the force, shall be as provided by Table E5.1-1.

Table E5.1-1
Minimum Required Edge and Spacing Distances

PAF Shank Diameter, d_s, in. (mm)	Minimum PAF Spacing in. (mm)	Minimum Edge Distance in. (mm)
0.106 (2.69) $\le d_s <$ 0.200 (5.08)	1.00 (25.4)	0.50 (12.7)
0.200 (5.08) $\le d_s <$ 0.206 (5.23)	1.60 (40.6)	1.00 (25.4)

E5.2 Power-Actuated Fasteners in Tension

The *available tensile strength* [*factored resistance*] per *PAF* shall be the minimum of the *available strengths* [*factored resistance*] determined by the applicable Sections E5.2.1 through E5.2.3. Washer thickness, t_w, limitations of Section E4 shall apply, except that for tapered head fasteners, the minimum thickness, t_w, shall not be less than 0.039 in. (0.991 mm). The thickness of collapsible pre-mounted top-hat washers shall not exceed 0.020 in. (0.508 mm).

E5.2.1 Tension Strength [Resistance]

The *nominal tension strength* [*resistance*], P_{ntp}, is permitted to be calculated in accordance with Eq. E5.2.1-1, and the following *safety factor* or *resistance factors* shall be applied to determine the *available strength* [*factored resistance*] in accordance with Section A4, A5 or A6:

$$P_{ntp} = (d/2)^2 \pi F_{uh} \qquad (Eq.\ E5.2.1\text{-}1)$$

Ω = 2.65 (*ASD*)
ϕ = 0.60 (*LRFD*)
= 0.50 (*LSD*)

F_{uh} in Eq. E5.2.1-1 shall be calculated with Eq. E5.2.1-2. Alternatively, for fasteners with HRC_p of 52 or more, F_{uh} is permitted to be taken as 260,000 psi (1790 MPa).

$$F_{uh} = F_{bs} e^{(HRC_p/40)} \qquad (Eq.\ E5.2.1\text{-}2)$$

where
e = 2.718

E5.2.2 Pull-Out Strength [Resistance]

The *nominal pull-out strength* [*resistance*], P_{not}, shall be determined through independent laboratory testing with the *safety factor* or the *resistance factor* determined in accordance with Chapter F. Alternatively, for *connections* with the entire *PAF point length*, ℓ_{dp}, below t_2, the following *safety factor* or *resistance factors* are permitted to determine the *available strength* [*factored resistance*] in accordance with Section A4, A5 or A6:

Ω = 4.00 (*ASD*)
ϕ = 0.40 (*LRFD*)
= 0.30 (*LSD*)

E5.2.3 Pull-Over Strength [Resistance]

The *nominal pull-over strength* [*resistance*], P_{nov}, is permitted to be computed in accordance with Eq. E5.2.3-1, and the following *safety factor* or *resistance factors* shall be applied to determine the *available strength* [*factored resistance*] in accordance with Section A4, A5 or A6:

$$P_{nov} = \alpha_w t_1 d'_w F_{u1} \quad (Eq.\ E5.2.3\text{-}1)$$

Ω = 3.00 (*ASD*)
ϕ = 0.50 (*LRFD*)
= 0.40 (*LSD*)

where

α_w = 1.5 for screw-, bolt-, nail-like flat heads or simple *PAF*, with or without head washers (see Figures E5(a) and E5(b))
= 1.5 for threaded stud *PAFs* and for *PAFs* with tapered standoff heads that achieve pull-over by friction and locking of the pre-mounted washer (see Figure E5(c)), with a/d_s ratio of no less than 1.6 and $(a - d_s)$ of no less than 0.12 in. (3.1 mm)
= 1.25 for threaded stud *PAFs* and for *PAFs* with tapered standoff heads that achieve pull-over by friction and locking of pre-mounted washer (see Figure E5 (c)), with a/d_s ratio of no less than 1.4 and $(a - d_s)$ of no less than 0.08 in. (2.0 mm)
= 2.0 for *PAFs* with collapsible spring washer (see Figure E5(d))

E5.3 Power-Actuated Fasteners in Shear

The *available shear strength* [*factored resistance*] shall be the minimum of the *available strengths* [*factored resistances*] determined by the applicable Sections E5.3.1 through E5.3.5.

E5.3.1 Shear Strength [Resistance]

The *nominal shear strength* [*resistance*], P_{nsp}, is permitted to be computed in accordance with Eq. E5.3.1-1, and the *safety factor* and *resistance factors* shall be applied to determine the *available strength* [*factored resistance*] in accordance with Section A4, A5 or A6:

$$P_{nsp} = 0.6(d/2)^2 \pi F_{uh} \quad (Eq.\ E5.3.1\text{-}1)$$

Ω = 2.65 (*ASD*)

ϕ = 0.60 (*LRFD*)
= 0.55 (*LSD*)

where

F_{uh} is determined in accordance with Section E5.2.1

E5.3.2 Bearing and Tilting Strength [Resistance]

For *PAFs* embedded such that the entire length of *PAF point length*, ℓ_{dp}, is below t_2, the *nominal bearing and tilting strength* [*resistance*], P_{nbp}, is permitted to be computed in accordance with Eq. E5.3.2-1, and the following *safety factor* or *resistance factors* shall be applied to determine the *available strength* [*factored resistance*] in accordance with Section A4, A5 or A6:

$$P_{nbp} = \alpha_b d_s t_1 F_{u1} \qquad (Eq.\ E5.3.2\text{-}1)$$

Ω = 2.05 (*ASD*)
ϕ = 0.80 (*LRFD*)
= 0.65 (*LSD*)

where

α_b = 3.7 for *connections* with *PAF* types as shown in Figures E5(c) and E5(d)
= 3.2 for other types of *PAFs*

Eq. E5.3.2-1 shall apply for *connections* within the following limits:

$t_2/t_1 \geq 2$

$t_2 \geq 1/8$ in. (3.18 mm)

0.146 in. (3.71 mm) $\leq d_s \leq$ 0.177 in. (4.50 mm).

E5.3.3 Pull-Out Strength [Resistance] in Shear

For *PAFs* driven through a depth of at least $0.6t_2$, the *nominal pull-out strength* [*resistance*], P_{nos}, in shear is permitted to be computed in accordance with Eq. E5.3.3-1, and the following *safety factor* and the *resistance factors* shall be applied to determine the *available strength* [*factored resistance*] in accordance with Section A4, A5 or A6:

$$P_{nos} = \frac{d_{ae}^{1.8} t_2^{0.2} \left(F_{y2} E^2\right)^{1/3}}{30} \qquad (Eq.\ E5.3.3\text{-}1)$$

Ω = 2.55 (*ASD*)
ϕ = 0.60 (*LRFD*)
= 0.50 (*LSD*)

Eq. E5.3.3-1 shall apply for *connections* within the following limits:

0.113 in. (2.87 mm) $\leq t_2 \leq 3/4$ in. (19.1 mm)

0.106 in. (2.69 mm) $\leq d_s \leq$ 0.206 in. (5.23 mm).

E5.3.4 Net Section Rupture Strength [Resistance]

The *available strength* [*factored resistance*] due to net cross-section rupture and block shear shall be determined in accordance with Section E6. In computations of net section rupture and block shear limit states, the hole size shall be taken as 1.10 times the nominal *PAF* shank diameter, d_s.

E5.3.5 Shear Strength [Resistance] Limited by Edge Distance

The *available shear strength* [*factored resistance*] limited by edge distance shall be computed in accordance with Section E6.1 and the applicable *safety factor* or the *resistance factors* provided in Table E6-1 shall be applied to determine the *available strength* [*factored resistance*] in accordance with Section A4, A5 or A6. The consideration of edge distance shall be based upon nominal shank diameter, d_s.

E5.4 Combined Shear and Tension

Effects of combined shear and tension on the *PAF connection*, including the interaction due to combined shear and pull-out, combined shear and pull-over, and combined shear and tension on the *PAF*, shall be considered in design.

E6 Rupture

The design criteria of this section shall apply where the *thickness* of the thinnest connected part is 3/16 in. (4.76 mm) or less. For *connections* where the *thickness* of the thinnest connected part is greater than 3/16 in. (4.76 mm), the specifications and standards stipulated in Section E6a of Appendix A or B shall apply. ☜A,B

For *connection* types utilizing welds or bolts, the *nominal rupture strength* [*resistance*], R_n, shall be the smallest of the values obtained in accordance with Sections E6.1, E6.2, and E6.3, as applicable. For *connection* types utilizing screws and *PAFs*, the *nominal rupture strength* [*resistance*], R_n, shall be the lesser of the values obtained in accordance with Sections E6.1 and E6.2, as applicable. The corresponding *safety factor* and *resistance factors* given in Table E6-1 shall be applied to determine the *allowable strength* or *design strength* [*factored resistance*] in accordance with the applicable design method in Section A4, A5 or A6.

Table E6-1
Safety Factors and Resistance Factors for Rupture

Connection Type	Ω (ASD)	φ (LRFD)	ϕ_u (LSD)
Welds	2.50	0.60	0.75
Bolts	2.22	0.65	0.75
Screws and Power-Actuated Fasteners	3.00	0.50	0.75

E6.1 Shear Rupture

The *nominal shear strength* [*resistance*], V_n, shall be calculated in accordance with Eq. E6.1-1.

$$V_n = 0.6\, F_u\, A_{nv} \qquad (Eq.\ E6.1\text{-}1)$$

where

F_u = *Tensile strength* of connected part as specified in Section A2.1 or A2.2

A_{nv} = *Net area* subject to shear (parallel to force):

For a *connection* where each individual fastener pulls through the material towards the limiting edge individually:

$A_{nv} = 2n\, t\, e_{net}$ (*Eq*. E6.1-2)

where

n = Number of fasteners on critical cross-section

t = Base steel *thickness* of section

e_{net} = Clear distance between end of material and edge of fastener hole or weld

For a beam-end connection where one or more of the *flanges* are coped:

$$A_{nv} = (h_{wc} - n_b d_h)t \quad (Eq.\ E6.1\text{-}3)$$

where

h_{wc} = Coped flat *web* depth

n_b = Number of fasteners along failure path being analyzed

d_h = Diameter of hole

t = *Thickness* of coped *web*

E6.2 Tension Rupture

The *nominal tensile rupture strength* [*resistance*], T_n, shall be calculated in accordance with Eq. E6.2-1.

$$T_n = F_u A_e \quad (Eq.\ E6.2\text{-}1)$$

where

A_e = Effective *net area* subject to tension

$$= U_{sl} A_{nt} \quad (Eq.\ E6.2\text{-}2)$$

where

U_{sl} = Shear lag factor determined in Table E6.2-1

A_{nt} = *Net area* subject to tension (perpendicular to force), except as noted in Table E6.2-1

$$= A_g - n_b d_h t + (\Sigma s'^2/4g)t \quad (Eq.\ E6.2\text{-}3)$$

where

A_g = *Gross area* of member

s' = Longitudinal center-to-center spacing of any two consecutive holes

g = Transverse center-to-center spacing between fastener gage lines

n_b = Number of fasteners along failure path being analyzed

d_h = Diameter of a standard hole

t = Base steel *thickness* of section

F_u = *Tensile strength* of connected part as specified in Section A2.1 or A2.2

Table E6.2-1
Shear Lag Factors for Connections to Tension Members

Description of Element	Shear Lag Factor, U_{sl}
(a) For flat sheet *connections* not having staggered hole patterns	
(1) For multiple connectors in the line parallel to the force	$U_{sl} = 1.0$
(2) For a single connector, or a single row of connectors perpendicular to the force	
(i) For single shear and outside sheets of double shear connections with washers provided under the bolt head and the nut	$U_{sl} = 3.33\ d/s \le 1.0$ (*Eq*. E6.2-4)
(ii) For single shear and outside sheets of double shear connections when washers are not provided or only one washer is provided under either the bolt head or the nut	$U_{sl} = 2.5\ d/s \le 1.0$ (*Eq*. E6.2-5)
(iii) For inside sheets of double shear connections with or without washers	$U_{sl} = 4.15\ d/s \le 1.0$ (*Eq*. E6.2-6)
(b) For flat sheet *connections* having staggered hole patterns	$U_{sl} = 1.0$
(c) For other than flat sheet connections	
(1) When load is transmitted only by transverse welds	$U_{sl} = 1.0$ and A_{nt} = Area of the directly connected elements
(2) When load is transmitted directly to all the cross-sectional elements	$U_{sl} = 1.0$
(3) For connections of angle members not meeting (c)(1) or (c)(2) above	$U_{sl} = 1.0 - 1.20\ \bar{x}/L \le 0.9$ (*Eq*. E6.2-7) but U_{sl} shall not be less than 0.4.
(4) For connections of channel members not meeting (c)(1) or (c)(2) above	$U_{sl} = 1.0 - 0.36\ \bar{x}/L \le 0.9$ (*Eq*. E6.2-8) but U_{sl} shall not be less than 0.5.

The variables in Table E6.2-1 shall be defined as follows:

$\bar{x}$ = Distance from shear plane to centroid of cross-section
L = Length of longitudinal weld or length of connection
s = Sheet width divided by number of bolt holes in cross-section being analyzed
d = Nominal bolt diameter

E6.3 Block Shear Rupture

The *nominal block shear rupture strength* [*resistance*], R_n, shall be determined as the lesser of the following:

$$R_n = 0.6F_y A_{gv} + U_{bs} F_u A_{nt} \qquad (Eq.\ E6.3\text{-}1)$$

$$R_n = 0.6F_u A_{nv} + U_{bs} F_u A_{nt} \qquad (Eq.\ E6.3\text{-}2)$$

where

A_{gv} = *Gross area* subject to shear (parallel to force)
A_{nv} = *Net area* subject to shear (parallel to force)
A_{nt} = *Net area* subject to tension(perpendicular to force), except as noted in Table E6.2-1

U_{bs} = Non-uniform block shear factor
= 0.5 for coped beam shear conditions with more than one vertical row of connectors
= 1.0 for all other cases
F_y = *Yield stress* of connected part as specified in Section A2.1 or A2.2
F_u = *Tensile strength* of connected part as specified in Section A2.1 or A2.2

E7 Connections to Other Materials

E7.1 Bearing

Provisions shall be made to transfer bearing forces from steel components covered by this *Specification* to adjacent *structural components* made of other materials.

E7.2 Tension

The pull-over shear or tension forces in the steel sheet around the head of the fastener shall be considered, as well as the pull-out force resulting from axial *loads* and bending moments transmitted onto the fastener from various adjacent *structural components* in the assembly.

The *nominal tensile strength* [*resistance*] of the fastener and the *nominal embedment strength* [*resistance*] of the adjacent structural component shall be determined by applicable product code approvals, product specifications, product literature, or combination thereof.

E7.3 Shear

Provisions shall be made to transfer shearing forces from steel components covered by this *Specification* to adjacent structural components made of other materials. The *required shear* and/or *bearing strength* [shear or bearing force due to *factored loads*] on the steel components shall not exceed that allowed by this *Specification*. The *available shear strength* [*factored resistance*] on the fasteners and other material shall not be exceeded. Embedment requirements shall be met. Provisions shall also be made for shearing forces in combination with other forces.

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 in accordance with A1.2(a) or by *rational engineering analysis* with verification tests in accordance with A1.2(b) shall be evaluated with the following performance procedure:

(a) Evaluation of the test results for use with A1.2(a) shall be made on the basis of the average value of test data resulting from tests of not fewer than three identical specimens, provided the deviation of any individual test result from the average value obtained from all tests does not exceed ±15 percent. If such deviation from the average value exceeds 15 percent, more tests of the same kind shall be made until the deviation of any individual test result from the average value obtained from all tests does not exceed ±15 percent or until at least three additional tests have been made. No test result shall be eliminated unless a rationale for its exclusion is given. The average value of all tests made shall then be regarded as the *nominal strength* [*resistance*], R_n, for the series of the tests. R_n and the coefficient of variation V_P of the test results shall be determined by statistical analysis.

(b) Evaluation of a *rational engineering analysis* model by verification tests for use with A1.2(b): The correlation coefficient, C_c, between the tested *strength* [*resistance*] (R_t) and the *nominal strength* [*resistance*] (R_n) predicted from the *rational engineering analysis* model shall be greater than or equal to 0.80. Only one limit state is permitted for evaluation of the *rational engineering analysis* model being verified, and the test result shall reflect the limit state under consideration.

The *rational engineering analysis* model is only verified within parameters varied in the testing. Extrapolation outside of the tested parameters is not permitted. For each parameter being evaluated: (i) all other parameters shall be held constant; (ii) the nominally selected values of the parameter to be tested shall not bias the study to a specific region of the parameter; and (iii) a minimum of three tests shall be performed. No test results shall be eliminated unless a rationale for their exclusion is given.

Dimensions and material properties shall be measured for all test specimens. The as-measured dimensions and properties shall be used in determination of the calculated *nominal strength* [*resistance*] ($R_{n,i}$) as employed in determining the *resistance factor* or *safety factor* in accordance with (c). The specified dimensions and properties shall be used in the determination of the calculated *nominal strength* [*resistance*] for design. The bias and variance between the as-measured dimensions and properties and the nominally specified dimensions and properties shall be reflected in the selected material (M_m, V_M) and fabrication (F_m, V_F) factors per Table F1. Otherwise, the selected values of M_m and F_m shall not be greater than in Table F1, and the values of V_M and V_F shall not be less than the values given in Table F1.

Section F1.1(b) is not applicable to floor, roof, or wall steel *diaphragms* in accordance with Section D5.

(c) The strength [*resistance*] of the tested elements, assemblies, *connections*, or members shall satisfy Eq. F1.1-1a or Eq. F1.1-1b as applicable.

$$\Sigma\gamma_i Q_i \leq \phi R_n \quad \text{for } LRFD \qquad (Eq.\ F1.1\text{-}1a)$$

$$\phi R_n \geq \Sigma\gamma_i Q_i \quad \text{for } LSD \qquad (Eq.\ F1.1\text{-}1b)$$

where

$\Sigma\gamma_i Q_i$ = *Required strength* [effect of *factored loads*] based on the most critical load combination determined in accordance with Section 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*

$$= C_\phi (M_m F_m P_m)\, e^{-\beta_o \sqrt{V_M^2 + V_F^2 + C_P V_P^2 + V_Q^2}} \qquad (Eq.\ F1.1\text{-}2)$$

where

C_ϕ = Calibration coefficient
= 1.52 for *LRFD*
= 1.42 for *LSD*
= 1.6 for *LRFD* for beams having tension *flange* through-fastened to deck or sheathing and with compression *flange* laterally unbraced
= 1.42 for *LSD* for beams having tension *flange* through-fastened to deck or sheathing and with compression *flange* laterally unbraced

M_m = Mean value of material factor, M, determined by statistical analysis or where applicable, as limited by Table F1 for type of component involved

F_m = Mean value of fabrication factor, F, determined by statistical analysis or where applicable, as limited by Table F1 for type of component involved

P_m = Mean value of professional factor, P, for tested component
= 1.0, if the *available strength* [*factored resistance*] is determined in accordance with Section F1.1(a); or

$$= \frac{\sum_{i=1}^{n} \frac{R_{t,i}}{R_{n,i}}}{n}$$, when the *available strength* [*factored resistance*] is determined in accordance with Section F1.1(b) (*Eq*. F1.1-3)

where

i = Index of tests
= 1 to n

n = Total number of tests

$R_{t,i}$ = Tested strength [resistance] of test i

$R_{n,i}$ = Calculated nominal strength [resistance] of test i per rational engineering analysis model

e = Natural logarithmic base
= 2.718

β_o = Target reliability index
= 2.5 for *structural members* and 3.5 for *connections* for *LRFD*
= 3.0 for *structural members* and 4.0 for *connections* for *LSD*
= 1.5 for *LRFD* for beams having tension *flange* through-fastened to deck or sheathing and with compression *flange* laterally unbraced
= 3.0 for *LSD* for beams having tension *flange* through-fastened to deck or

sheathing and with compression *flange* laterally unbraced

V_M = Coefficient of variation of material factor listed in Table 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*. F1.1-4)

= 5.7 for n = 3

where

n = Number of tests

m = Degrees of freedom

= n-1

V_P = Coefficient of variation of test results, but not less than 0.065

= $\frac{s_t}{R_n}$, if the *available strength* [*factored resistance*] is determined in accordance with Section F1.1(a) or (*Eq*. F1.1-5)

= $\frac{s_c}{P_m}$, if the *available strength* [*factored resistance*] is determined in accordance with Section F1.1 (b) (*Eq*. F1.1-6)

where

s_t = Standard deviation of all of the test results

s_c = Standard deviation of $R_{t,i}$ divided by $R_{n,i}$ for all of the test results

V_Q = Coefficient of variation of *load effect*

= 0.21 for *LRFD* and *LSD*

= 0.43 for *LRFD* for beams having tension *flange* through-fastened to deck or sheathing and with compression *flange* laterally unbraced

= 0.21 for the *LSD* for beams having tension *flange* through-fastened to deck or sheathing and with compression *flange* laterally unbraced

C_c = Correlation coefficient

$$= \frac{n\sum R_{t,i}R_{n,i} - \left(\sum R_{t,i}\right)\left(\sum R_{n,i}\right)}{\sqrt{n\left(\sum R_{t,i}^2\right) - \left(\sum R_{t,i}\right)^2}\sqrt{n\left(\sum R_{n,i}^2\right) - \left(\sum R_{n,i}\right)^2}} \qquad (Eq.\ F1.1\text{-}7)$$

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, the values of M_m and V_M shall be determined by the statistical analysis for the materials used.

When distortions interfere with the proper functioning of the specimen in actual use, the *load effects* based on the critical *load* combination at the occurrence of the acceptable distortion shall also satisfy Eq. F1.1-1a 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.

(d) For strength [*resistance*] determined in accordance with F1.1(a) or F1.1(b), the mechanical properties of the steel sheet shall be determined based on representative samples of the material taken from the test specimen or the flat sheet used to form the test specimen. Alternatively, for connectors or devices that are too small to obtain standard size or sub-size tensile specimens per ASTM A370, and are produced from steel sheet coils that have not undergone a secondary process to alter the mechanical or chemical properties, mechanical properties are permitted to be determined based on mill certificates, and the mean value of the material factor, M_m, shall be equal to 0.85. If the *yield stress* of the steel is larger than the specified value, the test results shall be adjusted down to the *specified minimum yield stress* of the steel that the manufacturer intends to use. The test results shall not be adjusted upward if the *yield stress* of the test specimen is less than the *specified minimum yield stress*. Similar adjustments shall be made on the basis of *tensile strength* instead of *yield stress* where *tensile strength* is the critical factor.

Consideration shall also be given to any variation or differences between the design *thickness* and the *thickness* of the specimens used in the tests.

TABLE 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 and *Power-Actuated Fastener* Connections				
Shear Strength	1.10	0.10	1.00	0.10
Tensile Strength	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 and Pull-Out in Shear	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 of Screws	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 *ASD 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* are permitted to be made to demonstrate the strength is not less than the *nominal strength* [*resistance*], R_n, specified in this *Specification* or its specific references for the type of behavior involved.

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 five (5) percent with the *yield stress* that would be determined by the 0.2 percent offset method.

(c) Where the principal effect of the loading to which the member will be subjected in service will be to produce bending *stresses*, the *yield stress* shall be determined for the *flanges* only. In determining such *yield stress*, each specimen shall consist of one complete *flange* plus a portion of the *web* of such *flat width* ratio that the value of ρ for the specimen is unity.

(d) For acceptance and control purposes, one full section test shall be made from each *master coil*.

(e) At the option of the manufacturer, either tension or compression tests are permitted to be used for routine acceptance and control purposes, provided the manufacturer demonstrates that such tests reliably indicate the *yield stress* of the section when subjected to the kind of *stress* under which the member is to be used.

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.

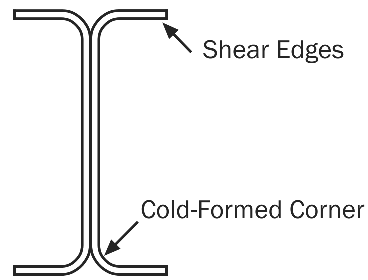
Stress range shall be defined as the magnitude of the change in *stress* due to the application or removal of the unfactored live *load*. In the case of a *stress* reversal, the *stress* range shall be computed as the sum of the absolute values of maximum repeated tensile and compressive *stresses* or 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.

Evaluation of *fatigue* strength [*resistance*] shall not be required if the number of cycles of application of live *load* is less than 20,000.

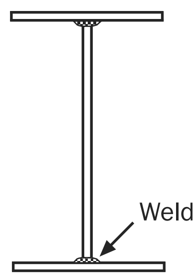
Table G1
Fatigue Design Parameters for Cold-Formed Steel Structures

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



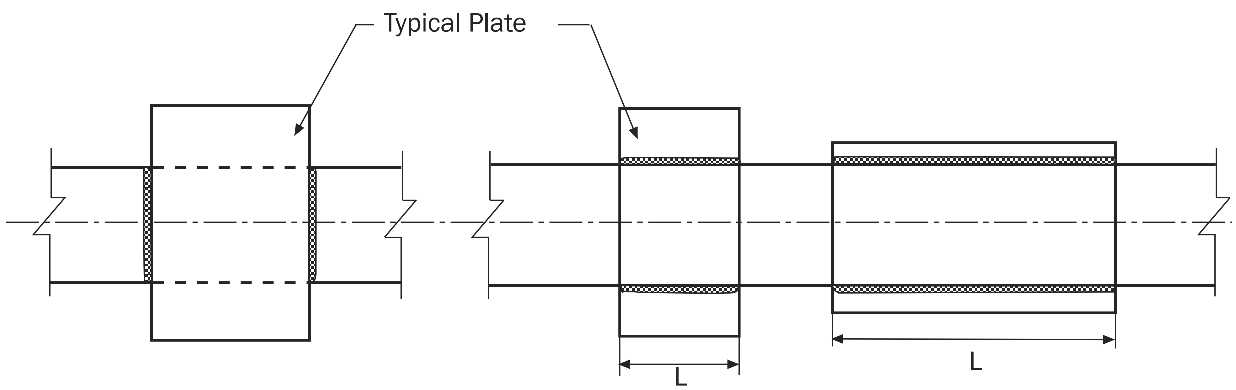
Cold-Formed Steel Channels, Stress Category I

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
Stress Category III

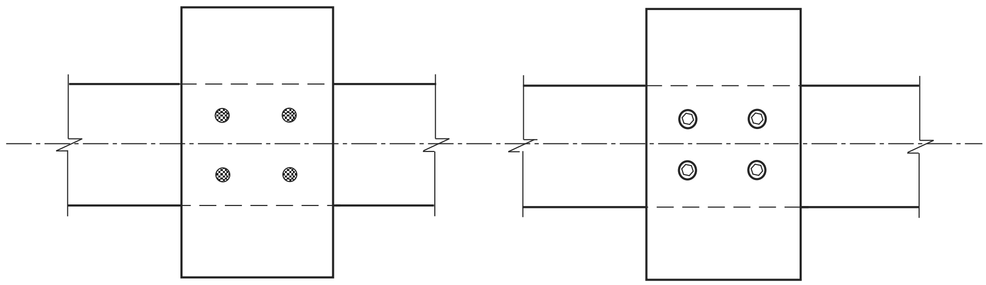
(b) Longitudinal Welds
For Stress Category III , L≤ 2 in. (50.8 mm)
For Stress Category IV, 2 in. (50.8 mm)< L ≤ 4 in. (101.6 mm)

Figure G1-3 Typical Attachments for Stress Categories III and IV

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.



(a) Arc Spot or Plug Weld (b) Screws

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 Eq. G3-1 for all *stress* categories as follows:

$$F_{SR} = (\alpha C_f/N)^{0.333} \geq F_{TH} \quad (Eq.\ G3\text{-}1)$$

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 × 365 × 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* [*specified*] *loads* 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 Eq. 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 are permitted to remain in place, and if used, shall be continuous.

Backing bars that are perpendicular to the *stress* field, if used, shall be removed and the *joint* back gouged and welded.

Flame cut edges subject to cyclic *stress* ranges shall have a surface roughness not to exceed 1,000 μin. (25 μm) in accordance with ASME B46.1.

Re-entrant corners at cuts, copes, and weld access holes shall form a radius of not less than 3/8 in. (9.53 mm) by pre-drilling or sub-punching and reaming a hole, or by thermal cutting to form the radius of the cut. If the radius portion is formed by thermal cutting, the cut surface shall be ground to a bright metal contour to provide a radiused transition, free of notches, with a surface roughness not to exceed 1,000 μin. (25 μm) in accordance with ASME B46.1 or other equivalent approved standards.

For transverse butt *joints* in regions of high tensile *stress*, weld tabs shall be used to provide for cascading the weld termination outside the finished *joint*. End dams shall not be used. Weld tabs shall be removed and the end of the weld finished flush with the edge of the member. Exception: Weld tabs shall not be required for sheet material if the welding procedures used result in smooth, flush edges.

Appendix 1

Design of Cold-Formed Steel Structural Members Using the Direct Strength Method

2012 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 are permitted to be used to determine the *nominal axial* (P_n), *flexural* (M_n) and *shear* (V_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 criteria of Section 1.1.1.1 for columns and Section 1.1.1.2 for beams have been prequalified for use, and the calibrated *safety factor*, Ω, and *resistance factor*, ϕ, given in 1.2.1 and 1.2.2 are permitted to apply. The use of the provisions of Sections 1.2.1 and 1.2.2 for other columns and beams is permitted, but the standard Ω and ϕ factors for *rational engineering analysis* (Section A1.2(c) 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, *web crippling*, combined bending and *web crippling*, or combined axial *load* and bending (beam-columns). Also, no provisions are given for structural assemblies or *connections* and *joints*. As detailed in Section A1.2, the provisions of the main *Specification*, when applicable, shall be used for all cases listed above.

It is permitted to substitute the *nominal strengths* [*resistances*], (P_n, M_n, and V_n), *resistance factors* and *safety factors* (ϕ and Ω) from this Appendix for the corresponding values in Sections C3.1, C3.2, C3.3, C3.5, C4.1, C4.2, C5, 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 is 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 A1.2(c) of the main *Specification*:

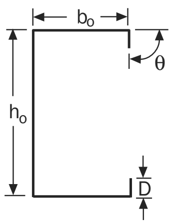
(1) Applicable provisions of the main *Specification* shall be followed when they exist, and
(2) Increased *safety factors*, Ω, and reduced *resistance factors*, ϕ, shall be employed for strength when *rational engineering analysis* is conducted.

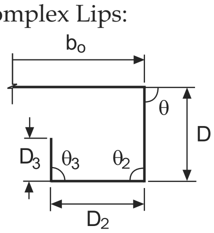
1.1.1.1 Prequalified Columns

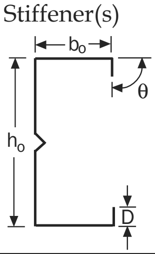
Columns that fall within the geometric and material limitations given in Table 1.1.1-1 are permitted to be designed using the *safety factor*, Ω, and *resistance factor*, ϕ, defined in Section 1.2.1 and are permitted to be designed either with or without holes. There shall be no limitations on the size, shape and spacing of the holes.

Columns which fall outside of the geometric and material limitations of Table 1.1.1-1 are permitted to still use the Ω or ϕ of Section 1.2.1 if, through the use of Chapter F of the main *Specification*, the predicted ϕ from Chapter F provides an equal or higher ϕ (equal or higher level of reliability) to that of Section 1.2.1. In the use of Chapter F, the professional factor, P, shall be the test-to-predicted ratio where the prediction is that of the *Direct Strength Method* expressions of Section 1.2.1, P_m is the mean of P, and V_P is the coefficient of variation of P. At least three tests shall be conducted. If V_P is less than or equal to 15 percent, C_p is permitted to be set to 1.0.

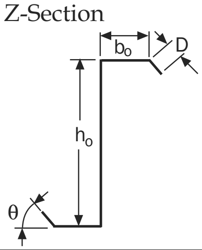
Table 1.1.1-1
Limits for Prequalified Columns*

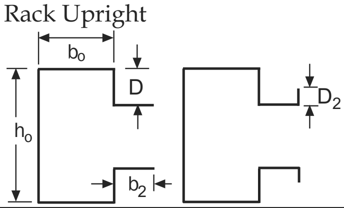
<table>
<tr>
<td>Lipped C-Sections
Simple Lips:


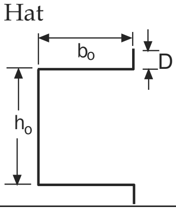
Complex Lips:


</td>
<td>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/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, outward, etc.)
b) θ_3 is permitted to vary (D_3 lip is permitted to angle up, down, etc.)</td>
</tr>
<tr>
<td>Lipped C-Section With Web Stiffener(s)


</td>
<td>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/cm²)]</td>
</tr>
<tr>
<td>Z-Section


</td>
<td>$h_o/t < 137$
$b_o/t < 56$
$0 < D/t < 36$
$1.5 < h_o/b_o < 2.7$
$0.00 < D/b_o < 0.73$
$\theta = 50°$
$E/F_y > 590$ [$F_y < 50$ ksi (345 MPa or 3520 kg/cm²)]</td>
</tr>
<tr>
<td>Rack Upright


</td>
<td>See C-Section With Complex Lips</td>
</tr>
<tr>
<td>Hat


</td>
<td>$h_o/t < 50$
$b_o/t < 43$
$4 < D/t < 6$
$1.0 < h_o/b_o < 1.2$
$D/b_o = 0.13$
$E/F_y > 428$ [$F_y < 69$ ksi (476 MPa or 4850 kg/cm²)]</td>
</tr>
</table>

Note: * $r/t < 20$, where r is the centerline bend radius
b_o = Overall width; D = Overall lip depth; t = Base metal *thickness*; h_o= Overall depth

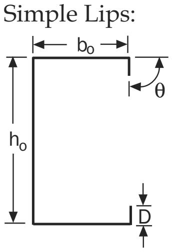
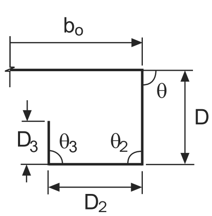
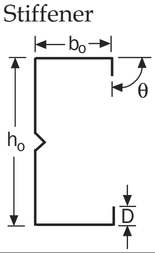
1.1.1.2 Prequalified Beams

Beams in bending that fall within the geometric and material limitations given in Table 1.1.1-2 are permitted to be designed using the *safety factor*, Ω, and *resistance factor*, ϕ, defined in Section 1.2.2.1 and are permitted to be designed either with or without holes. There shall be no limitations on the size, shape and spacing of the holes for beams designed in bending.

Beams in shear without holes that fall within the geometric and material limitations given in Table 1.1.1-3 are permitted to be designed using the *safety factor*, Ω, and *resistance factor*, ϕ, defined in Sections 1.2.2.2.

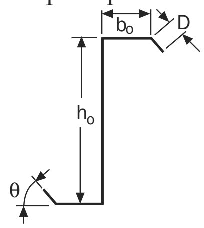
Beams which fall outside of the geometric and material limitations of Tables 1.1.1-2 and 1.1.1-3 are permitted to still use the Ω or ϕ of Section 1.2.2 if, through the use of Chapter F of the main *Specification*, the predicted ϕ from Chapter F provides an equal or higher ϕ (equal or higher level of reliability) to that of Section 1.2.2. In the use of Chapter F, the professional factor, P, shall be the test-to-predicted ratio where the prediction is that of the *Direct Strength Method* expressions of Section 1.2.2, P_m is the mean of P, and V_P is the coefficient of variation of P. At least three tests shall be conducted. If V_P is less than or equal to 15 percent, C_P is permitted to be set to 1.0.

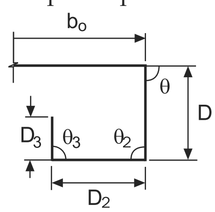
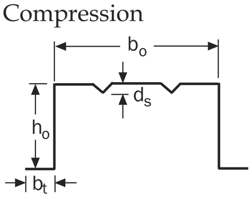
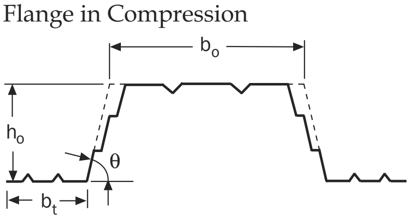
Table 1.1.1-2
Limitations for Prequalified Beams for Bending*

C-Sections Simple Lips:  Complex Lips: 	For all C-sections $h_o/t < 321$ $b_o/t < 75$ $0 < D/t < 34$ $1.5 < h_o/b_o < 17.0$ $0 < D/b_o < 0.70$ $44° < \theta < 90°$ $E/F_y > 421$ [$F_y < 70$ ksi (483 MPa or 4920 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 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 	$h_o/t < 358$ $b_o/t < 58$ $14 < D/t < 17$ $5.5 < h_o/b_o < 11.7$ $0.27 < D/b_o < 0.56$ $\theta = 90°$ $E/F_y > 578$ [$F_y < 51$ ksi (352 MPa or 3590 kg/cm^2)]

(Continued)

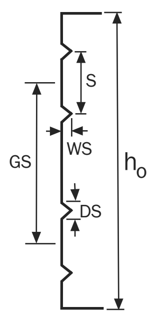
Table 1.1.1-2
Limitations for Prequalified Beams for Bending* (Continued)

<table>
<tr>
<td>Z-Sections
Simple Lips:


Complex Lips:

</td>
<td>For all Z-sections:
$h_o/t < 183$
$b_o/t < 71$
$10 < D/t < 16$
$2.5 < h_o/b_o < 4.1$
$0.15 < D/b_o < 0.34$
$36° < \theta < 90°$
$E/F_y > 440$ [$F_y < 67$ ksi (462 MPa or 4710 kg/cm^2)]
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 angle inward, outward, etc.)
b) θ_3 is permitted to vary (D_3 lip is permitted to angle up, down, etc.)</td>
</tr>
<tr>
<td>Hats (Decks) With Stiffened Flange in Compression

</td>
<td>$h_o/t < 97$
$b_o/t < 467$
$0 < d_s/t < 26$ (d_s =Depth of stiffener)
$0.14 < h_o/b_o < 0.87$
$0.88 < b_o/b_t < 5.4$
$0 < n \leq 4$ (n = Number of compression flange stiffeners)
$E/F_y > 492$ [$F_y < 60$ ksi (414 MPa or 4220 kg/cm^2)]</td>
</tr>
<tr>
<td>Trapezoids (Decks) With Stiffened Flange in Compression

</td>
<td>$h_o/t < 203$
$b_o/t < 231$
$0.42 < (h_o/\sin\theta)/b_o < 1.91$
$1.10 < b_o/b_t < 3.38$
$0 < n_c \leq 2$ (n_c = Number of compression flange stiffeners)
$0 < n_w \leq 2$ (n_w = Number of web stiffeners and/or folds)
$0 < n_t \leq 2$ (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/cm^2)]</td>
</tr>
</table>

Note: * $r/t < 20$, 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.

Table 1.1.1-3
Limitations for Prequalified Webs for Shear

Flat Webs:	$h_o/t < 256$
Multiple Web Stiffener(s): 	For two or four stiffeners: $h_o/t < 167$ WS < 0.165 in (4.2mm) DS/WS < 1.38 $0.0 < S/h_o < 0.28$ $0.4 < GS/h_o < 0.55$ n = 2 or 4 (n = Number of web stiffeners)

1.1.2 Elastic Buckling

Analysis shall be used for the determination of the elastic compressive and *shear buckling loads*, and 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 1.2.2.1), and the elastic *shear buckling loads* (V_{cr} of Section 1.2.2.2). In some cases, for a given column or beam, not all *buckling* modes exist. In such cases, the nonexistent 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, including the calculation of elastic *buckling* properties for columns and beams with hole(s).

1.1.3 Serviceability Determination

The bending deflection at any moment, M, due to *nominal loads* [*specified loads*] is permitted to be determined by reducing the gross moment of inertia, I_g, to an effective moment of inertia for deflection, as given in Eq. 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 1.2.2.1, 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 A1.2(c), shall apply. The *available strength* [*factored resistance*] shall be determined in accordance with the applicable design method in Section A4, A5, or A6 of the main *Specification*.

1.2.1.1 Flexural, Torsional, or Flexural-Torsional Buckling

1.2.1.1.1 Columns Without Holes

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 = Member yield strength

$= A_g F_y$ (*Eq.* 1.2.1-4)

A_g = *Gross area* of cross-section

F_y = *Yield stress*

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

1.2.1.1.2 Columns With Hole(s)

The *nominal axial strength* [*resistance*], P_{ne}, for *flexural*, *torsional*, or *flexural-torsional buckling* of columns with hole(s) shall be calculated in accordance with Section 1.2.1.1.1, except P_{cre} shall be determined including the influence of hole(s).

1.2.1.2 Local Buckling

1.2.1.2.1 Columns Without Holes

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

$\lambda_\ell = \sqrt{P_{ne}/P_{cr\ell}}$ (*Eq.* 1.2.1-7)

P_{ne} = *Nominal axial strength* [*resistance*] as defined in Section 1.2.1.1.1

$P_{cr\ell}$ = Critical elastic column *local buckling load* determined by analysis in accordance with Section 1.1.2

1.2.1.2.2 Columns With Hole(s)

The *nominal axial strength* [*resistance*], $P_{n\ell}$, for *local buckling* of columns with hole(s) shall be calculated in accordance with Section 1.2.1.2.1, except $P_{cr\ell}$ shall be determined including the influence of hole(s), and:

$P_{n\ell} \le P_{ynet}$ (*Eq.* 1.2.1-8)

where

P_{ynet} = Member yield strength on net cross-section

= $A_{net}F_y$ (*Eq.* 1.2.1-9)

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

1.2.1.3 Distortional Buckling

1.2.1.3.1 Columns Without Holes

The *nominal axial strength* [*resistance*], P_{nd}, for *distortional buckling* shall be calculated in accordance with the following:

(a) For $\lambda_d \le 0.561$

$P_{nd} = P_y$ (*Eq.* 1.2.1-10)

(b) For $\lambda_d > 0.561$

$$P_{nd} = \left(1 - 0.25\left(\frac{P_{crd}}{P_y}\right)^{0.6}\right)\left(\frac{P_{crd}}{P_y}\right)^{0.6} P_y \quad (Eq.\ 1.2.1\text{-}11)$$

where

$\lambda_d = \sqrt{P_y/P_{crd}}$ (*Eq.* 1.2.1-12)

where

P_y = Member yield strength as given in Eq. 1.2.1-4

P_{crd} = Critical elastic column *distortional buckling load* determined by analysis in accordance with Section 1.1.2

1.2.1.3.2 Columns With Hole(s)

The *nominal axial strength* [*resistance*], P_{nd}, for *distortional buckling* of columns with hole(s) shall be calculated in accordance with Section 1.2.1.3.1, except P_{crd} shall be determined including the influence of hole(s), and if $\lambda_d \le \lambda_{d2}$, then:

(a) For $\lambda_d \le \lambda_{d1}$

$P_{nd} = P_{ynet}$ (*Eq.* 1.2.1-13)

(b) For $\lambda_{d1} < \lambda_d \le \lambda_{d2}$

$$P_{nd} = P_{ynet} - \left(\frac{P_{ynet} - P_{d2}}{\lambda_{d2} - \lambda_{d1}}\right)(\lambda_d - \lambda_{d1}) \qquad (Eq.\ 1.2.1\text{-}14)$$

where

$$\lambda_d = \sqrt{P_y / P_{crd}} \qquad (Eq.\ 1.2.1\text{-}15)$$

$$\lambda_{d1} = 0.561(P_{ynet}/P_y) \qquad (Eq.\ 1.2.1\text{-}16)$$

$$\lambda_{d2} = 0.561\left(14\left(P_y/P_{ynet}\right)^{0.4} - 13\right) \qquad (Eq.\ 1.2.1\text{-}17)$$

$$P_{d2} = \left(1 - 0.25\left(1/\lambda_{d2}\right)^{1.2}\right)\left(1/\lambda_{d2}\right)^{1.2} P_y \qquad (Eq.\ 1.2.1\text{-}18)$$

P_y = Member yield strength as given in Eq. 1.2.1-4

P_{ynet} = Yield strength of net section as given in Eq. 1.2.1-9

1.2.2 Beam Design

1.2.2.1 Bending

The *nominal flexural strength* [*resistance*], M_n, shall be the minimum of M_{ne}, $M_{n\ell}$, and M_{nd} as given in Sections 1.2.2.1.1 to 1.2.2.1.3. 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 A1.2(c), shall apply. The *available strength* [*factored resistance*] shall be determined in accordance with the applicable design method in Section A4, A5, or A6 of the main *Specification*.

1.2.2.1.1 Lateral-Torsional Buckling

The *nominal flexural strength* [*resistance*], M_{ne}, for *lateral-torsional buckling* shall be calculated in accordance with this section. The *nominal strength* [*resistance*] increase for inelastic reserve in *lateral-torsional buckling* is permitted in accordance with Section 1.2.2.1.1.1.2, as applicable.

1.2.2.1.1.1 Beams Without Holes

1.2.2.1.1.1.1 Lateral-Torsional Buckling Strength [Resistance]

The *nominal flexural strength* [*resistance*], M_{ne}, for *lateral-torsional buckling* shall be calculated in accordance with the following:

(a) For $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \qquad (Eq.\ 1.2.2\text{-}1)$$

(b) For $2.78M_y \ge M_{cre} \ge 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \qquad (Eq.\ 1.2.2\text{-}2)$$

(c) For $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \qquad (Eq.\ 1.2.2\text{-}3)$$

where

M_{cre} = Critical elastic *lateral-torsional buckling* moment; see Section 1.1.2

M_y = Member yield moment

$$= S_f F_y \qquad (Eq.\ 1.2.2\text{-}4)$$

where

S_f = Gross section modulus referenced to the extreme fiber at first yield

1.2.2.1.1.1.2 Inelastic Reserve Lateral-Torsional Buckling Strength [Resistance]

For $M_{cre} > 2.78\ M_y$

$$M_{ne} = M_p - (M_p - M_y)\frac{\sqrt{M_y / M_{cre}} - 0.23}{0.37} \le M_p \qquad (Eq.\ 1.2.2\text{-}5)$$

where

M_{cre} = Critical elastic *lateral-torsional buckling* moment, determined in accordance with Section 1.1.2

M_y = Member yield moment as defined in Eq. 1.2.2-4

M_p = Member plastic moment

$$= Z_f F_y \qquad (Eq.\ 1.2.2\text{-}6)$$

where

Z_f = Plastic section modulus

1.2.2.1.1.2 Beams With Hole(s)

The *nominal flexural strength* [*resistance*], M_{ne}, for *lateral-torsional buckling* of beams with hole(s) shall be calculated in accordance with Section 1.2.2.1.1.1.1, except M_{cre} shall be determined including the influence of hole(s).

1.2.2.1.2 Local Buckling

The *nominal flexural strength* [*resistance*], $M_{n\ell}$, for *local buckling* shall be calculated in accordance with this section. The *nominal strength* [*resistance*] increase for inelastic reserve in *local buckling* is permitted in accordance with Section 1.2.2.1.2.1.2, as applicable.

1.2.2.1.2.1 Beams Without Holes

1.2.2.1.2.1.1 Local Buckling Strength [Resistance]

(a) For $\lambda_\ell \le 0.776$

$$M_{n\ell} = M_{ne} \qquad (Eq.\ 1.2.2\text{-}7)$$

(b) For $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \qquad (Eq.\ 1.2.2\text{-}8)$$

where

$$\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}} \qquad (Eq.\ 1.2.2\text{-}9)$$

M_{ne} = *Nominal flexural strength* [*resistance*] for *lateral-torsional buckling* as defined in Section 1.2.2.1.1.1

$M_{cr\ell}$ = Critical elastic *local buckling* moment, determined in accordance with Section 1.1.2

1.2.2.1.2.1.2 Inelastic Reserve Local Buckling Strength [Resistance]

For $\lambda_\ell \leq 0.776$ and $M_{ne} \geq M_y$

Sections symmetric about the axis of bending or sections with first yield in compression:

$$M_{n\ell} = M_y + (1 - 1/C_{y\ell}^2)(M_p - M_y) \qquad (Eq.\ 1.2.2\text{-}10)$$

Sections with first yield in tension:

$$M_{n\ell} = M_{yc} + (1 - 1/C_{y\ell}^2)(M_p - M_{yc}) \leq M_{yt3} \qquad (Eq.\ 1.2.2\text{-}11)$$

where

$$\lambda_\ell = \sqrt{M_y / M_{cr\ell}} \qquad (Eq.\ 1.2.2\text{-}12)$$

M_{ne} = *Nominal flexural strength* [*resistance*] as defined in Section 1.2.2.1.1.1

$$C_{y\ell} = \sqrt{0.776/\lambda_\ell} \leq 3 \qquad (Eq.\ 1.2.2\text{-}13)$$

$M_{cr\ell}$ = Critical elastic *local buckling* moment, determined in accordance with Section 1.1.2

M_p = Member plastic moment as given in Eq. 1.2.2-6

M_y = Member yield moment as given in Eq. 1.2.2-4

M_{yc} = Moment at which yielding initiates in compression (after yielding in tension). $M_{yc} = M_y$ may be used as a conservative approximation.

$$M_{yt3} = M_y + (1 - 1/C_{yt}^2)(M_p - M_y) \qquad (Eq.\ 1.2.2\text{-}14)$$

C_{yt} = Ratio of maximum tension strain to yield strain

= 3

1.2.2.1.2.2 Beams With Hole(s)

The *nominal flexural strength* [*resistance*], $M_{n\ell}$, for *local buckling* of beams with hole(s) shall be calculated in accordance with Section 1.2.2.1.2.1.1, except $M_{cr\ell}$ shall be determined including the influence of hole(s) and when $\lambda_d \leq \lambda_{d2}$, then:

$$M_{n\ell} \leq M_{ynet} \qquad (Eq.\ 1.2.2\text{-}15)$$

where

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

$$= S_{fnet}F_y \qquad (Eq.\ 1.2.2\text{-}16)$$

where

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

1.2.2.1.3 Distortional Buckling

The *nominal flexural strength* [*resistance*], M_{nd}, for *distortional buckling* shall be calculated in accordance with this section. The *nominal strength* [*resistance*] increase for inelastic reserve in *distortional buckling* is permitted in accordance with Section 1.2.2.1.3.1.2, as applicable.

1.2.2.1.3.1 Beams Without Holes

1.2.2.1.3.1.1 Distortional Buckling Strength [Resistance]

(a) For $\lambda_d \leq 0.673$

$$M_{nd} = M_y \qquad (Eq.\ 1.2.2\text{-}17)$$

(b) For $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \qquad (Eq.\ 1.2.2\text{-}18)$$

where

$$\lambda_d = \sqrt{M_y / M_{crd}} \qquad (Eq.\ 1.2.2\text{-}19)$$

M_y = Member yield moment as given in Eq. 1.2.2-4

M_{crd} = Critical elastic *distortional buckling* moment, determined in accordance with Section 1.1.2

1.2.2.1.3.1.2 Inelastic Reserve Distortional Buckling Strength [Resistance]

For $\lambda_d \leq 0.673$

Sections symmetric about the axis of bending or sections with first yield in compression:

$$M_{nd} = M_y + (1 - 1/C_{yd}^2)(M_p - M_y) \qquad (Eq.\ 1.2.2\text{-}20)$$

Sections with first yield in tension:

$$M_{nd} = M_{yc} + (1 - 1/C_{yd}^2)(M_p - M_{yc}) \leq M_{yt3} \qquad (Eq.\ 1.2.2\text{-}21)$$

where

$$\lambda_d = \sqrt{M_y / M_{crd}} \qquad (Eq.\ 1.2.2\text{-}22)$$

$$C_{yd} = \sqrt{0.673 / \lambda_d} \leq 3 \qquad (Eq.\ 1.2.2\text{-}23)$$

M_{crd} = Critical elastic *distortional buckling* moment, determined in accordance with Section 1.1.2

M_p = Member plastic moment as given in Eq. 1.2.2-6

M_y = Member yield moment as given in Eq. 1.2.2-4

M_{yc} = Moment for yield in compression as defined in Section 1.2.2.1.2.1.2

M_{yt3} = Maximum moment for yielding in tension as given in Eq. 1.2.2-14

1.2.2.1.3.2 Beams With Hole(s)

The *nominal flexural strength* [*resistance*], M_{nd}, for *distortional buckling* shall be calculated in accordance with Section 1.2.2.1.3.1.1, except M_{crd} shall be determined including the influence of hole(s), and when $\lambda_d \le \lambda_{d2}$ then:

(a) For $\lambda_d \le \lambda_{d1}$

$$M_{nd} = M_{ynet} \qquad (Eq.\ 1.2.2\text{-}24)$$

(b) For $\lambda_{d1} < \lambda_d \le \lambda_{d2}$

$$M_{nd} = M_{ynet} - \left(\frac{M_{ynet} - M_{d2}}{\lambda_{d2} - \lambda_{d1}}\right)(\lambda_d - \lambda_{d1}) \le \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y$$

(*Eq.* 1.2.2-25)

where

$$\lambda_d = \sqrt{M_y / M_{crd}} \qquad (Eq.\ 1.2.2\text{-}26)$$

$$\lambda_{d1} = 0.673(M_{ynet}/M_y)^3 \qquad (Eq.\ 1.2.2\text{-}27)$$

λ_{d2} = Limit of distortional slenderness transition

$$= 0.673\left(1.7(M_y/M_{ynet})^{2.7} - 0.7\right) \qquad (Eq.\ 1.2.2\text{-}28)$$

$$M_{d2} = (1 - 0.22(1/\lambda_{d2}))(1/\lambda_{d2})M_y \qquad (Eq.\ 1.2.2\text{-}29)$$

M_y = Member yield moment as given in Eq. 1.2.2-4

M_{ynet} = Member yield moment of net section as given in Eq. 1.2.2-16

1.2.2.2 Shear

The *nominal shear strength* [*resistance*], V_n, of beams without hole(s) in the *web*(*s*) shall be calculated in accordance with this section, as applicable. For beams meeting the geometric and material criteria of Table 1.1.1-3, Ω_v and ϕ_v shall be as follows:

Ω_v = 1.60 (*ASD*)

ϕ_v = 0.95 (*LRFD*)

= 0.80 (*LSD*)

For all other beams, Ω and ϕ of the main *Specification*, Section A1.2(c), shall apply. The *available strength* [*factored resistance*] shall be determined in accordance with the applicable design method in Section A4, A5, or A6 of the main *Specification*.

1.2.2.2.1 Beams Without Web Stiffeners

For $\lambda_v \le 0.815$,

$$V_n = V_y \qquad (Eq.\ 1.2.2\text{-}30)$$

For $0.815 < \lambda_v \le 1.227$

$$V_n = 0.815\sqrt{V_{cr} V_y} \qquad (Eq.\ 1.2.2\text{-}31)$$

For $\lambda_v > 1.227$

$$V_n = V_{cr} \qquad (Eq.\ 1.2.2\text{-}32)$$

where

$$\lambda_v = \sqrt{\frac{V_y}{V_{cr}}} \qquad (Eq.\ 1.2.2\text{-}33)$$

V_y = Yield shear force of section

$$= 0.6\, A_w F_y \qquad (Eq.\ 1.2.2\text{-}34)$$

where

A_w = Area of *web* element

$$= ht \qquad (Eq.\ 1.2.2\text{-}35)$$

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

V_{cr} = Elastic *shear buckling* force of section determined in accordance with Section 1.1.2

1.2.2.2.2 Beams With Web Stiffeners

For a reinforced *web* with *web* stiffener spacing not exceeding twice the *web* depth, this section is permitted to be used to determine the *nominal shear strength* [*resistance*] in lieu of Section 1.2.2.2.1.

For $\lambda_v \leq 0.776$,

$$V_n = V_y \qquad (Eq.\ 1.2.2\text{-}36)$$

For $\lambda_v > 0.776$,

$$V_n = \left[1 - 0.15\left(\frac{V_{cr}}{V_y}\right)^{0.4}\right]\left(\frac{V_{cr}}{V_y}\right)^{0.4} V_y \qquad (Eq.\ 1.2.2\text{-}37)$$

1.2.2.3 Combined Bending and Shear

For beams subjected to combined bending and shear, Section C3.3 of the main *Specification* shall be used with M_{nxo} replaced by $M_{n\ell o}$ and V_n as follows:

$M_{n\ell o}$ = *Nominal flexural strength* [*resistance*] for *local buckling* (See Section 1.2.2.1.2) with $M_{ne} = M_y$

V_n = *Nominal shear strength* [*resistance*] with consideration of shear alone (See Section 1.2.2.2)

If Section 1.2.2.2.2 is used to compute V_n, M_{nxo} shall be calculated as follows:

M_{nxo} = Lesser of *nominal flexural strength* [*resistance*] for *local buckling* (See Section 1.2.2.1.2) with $M_{ne} = M_y$ and *nominal flexural strength* [*resistance*] for *distortional buckling* (See Section 1.2.2.1.3)

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*, braced compression members, or combinations thereof.

2.1 General Requirements

Members shall satisfy the provisions of Section C5 with the *nominal column strengths* [*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* [effects of *factored loads*] 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 is 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.

For braced compression members, the *second-order analysis* shall be carried out by assuming the maximum–out-of-straightness for the members to be braced but not less than L/960. All component and connection deformations that contribute to the lateral displacement of the braced compression members shall be considered.

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 $\alpha P_{ra}/P_y \le 0.5$

= $4[\alpha P_{ra}/P_y\,(1 - \alpha P_{ra}/P_y)]$ for $\alpha P_{ra}/P_y > 0.5$

P_{ra} = *Required compressive axial strength* [compressive axial force due to *factored loads*] using *ASD*, *LRFD* or *LSD load combinations*, kips (N)

P_y = Member yield strength (=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 the 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 $\alpha P_{ra}/P_y > 0.5$, $\tau_b = 1.0$ is 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 is permitted to be adjusted proportionally to a value not less than 1/500.

Appendix A:

Provisions Applicable to

the United States and Mexico

2012 EDITION

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:

(1) Beams and compression members (C- and Z-sections) for standing seam roofs, and

(2) Bolted and welded connections

Efforts are being made to minimize these differences in future editions of the *Specification*.

APPENDIX A: PROVISIONS APPLICABLE TO THE UNITED STATES AND MEXICO

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:

(a) The steel shall conform to the chemical and mechanical requirements of one of the listed specifications or other *published specification*.

(b) 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 ASTM A500 (for carbon steel) or ASTM A847 (for HSLA steel).

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

(d) The steel shall meet the requirements of Section A2.3.1, A2.3.2, A2.3.3, or A2.3.4, as appropriate.

(e) 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 AWS D1.3, as applicable.

If the identification and documentation of the production of the steel have not been established, then in addition to requirements (a) through (e), 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*.

A2.3.5a Ductility Requirements of Other Steels

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 A2.3.5, *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, IL 60601-1802:

   ANSI/AISC 360-10, *Specification for Structural Steel Buildings*

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

   AISI S908-08, *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 7-10, *Minimum Design Loads in Buildings and Other Structures*

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

   ASTM A6/A6M-12a, *Standard Specification for General Requirements for Rolled Structural Steel Bars, Plates, Shapes and Sheet Piling*

   ASTM A568/A568M-11b, *Standard Specification for Steel, Sheet, Carbon, Structural, and High-Strength, Low-Alloy, Hot-Rolled and Cold-Rolled, General Requirements for*

   ASTM A924/A924M-10a, *Standard Specification for General Requirements for Steel Sheet, Metallic Coated by the Hot Dip Process*

5. American Welding Society (AWS), 8669 Doral Boulevard, Suite 130, Doral, FL 33166:

   AWS D1.1/D1.1M-2010, *Structural Welding Code–Steel*

   AWS D1.3-2008, *Structural Welding Code–Sheet Steel*

   AWS C1.1/C1.1M-2012, *Recommended Practices for Resistance Welding*

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 design method in Section A4 or A5.

$$M_n = RS_eF_y \qquad (Eq.\ D6.1.2\text{-}1)$$

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 are 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 design method in Section A4 or A5.

(a) For weak axis *available strength*

$$P_n = k_{af}RF_yA \qquad (Eq.\ D6.1.4\text{-}1)$$

Ω = 1.80 (*ASD*)

ϕ = 0.85 (*LRFD*)

where

For $d/t \le 90$

k_{af} = 0.36

For $90 < d/t \le 130$

$$k_{af} = 0.72 - \frac{d}{250t} \qquad (Eq.\ D6.1.4\text{-}2)$$

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) $\le t \le$ 0.125 in. (3.22 mm)

(2) 6 in. (152 mm) $\le d \le 12$ in. (305 mm)
(3) *Flanges* are edge-*stiffened compression elements*
(4) $70 \le d/t \le 170$
(5) $2.8 \le d/b < 5$, where b = Z section *flange* width
(6) $16 \le \dfrac{\text{flange flat width}}{t} < 50$
(7) Both *flanges* are prevented from moving laterally at the supports
(8) *Yield stress*, $F_y \le 70$ ksi (483 MPa or 4920 kg/cm^2)

(b) The *available strength* about the strong axis shall be determined in accordance with Sections 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*, to be applied to the standing seam roof panel, clips and fasteners, is permitted to be multiplied by 0.67 provided the tested system and wind load evaluation satisfy the following conditions:
(a) The roof system is tested in accordance with AISI S906.
(b) The wind load is calculated using ASCE/SEI 7 for components and cladding.
(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). The *nominal wind load* applied to Zone 2 or Zone 3, after the 0.67 multiplier is applied, shall not be less than the *nominal wind load* applied 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:
  (1) The standing seam roof clip mechanically fails by separating from the panel sidelap.
  (2) The standing seam roof clip mechanically fails by the sliding tab separating from the stationary base.

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 shall apply to 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 3/16 in. (4.76 mm) or less. Bolted *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.

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 *safety factor* and the *resistance factor* given in this section shall be used to determine the *available strengths* in accordance with the applicable design method in Section A4 or A5.

$$P_n = A_b F_n \tag{Eq. E3.4-1}$$

Ω = 2.00 (*ASD*)

ϕ = 0.75 (*LRFD*)

where

A_b = Gross *cross-sectional area* of bolt

F_n = *Nominal strength* ksi (MPa), determined in accordance with (a) or (b) as follows:

(a) When bolts are subjected to shear only or tension only, F_n shall be given by F_{nv} or F_{nt} in Table E3.4-1.

The pull-over strength of the connected sheet at the bolt head, nut or washer shall be considered where bolt tension is involved. See Section 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*

$$F'_{nt} = 1.3\,F_{nt} - \frac{\Omega F_{nt}}{F_{nv}} f_v \le F_{nt} \tag{Eq. E3.4-2}$$

For *LRFD*

$$F'_{nt} = 1.3\,F_{nt} - \frac{F_{nt}}{\phi F_{nv}} f_v \leq F_{nt} \qquad (\textit{Eq}.\ \text{E3.4-3})$$

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)

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.

In Table E3.4-1, the *nominal 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.

TABLE E3.4-1
Nominal Tensile and Shear Strengths for Bolts

	Nominal Tensile Strength F_{nt}, ksi (MPa)	Nominal Shear Strength F_{nv}, ksi (MPa) [a]
ASTM A307 Bolts, Grade A 1/4 in. (6.4 mm) ≤ d <1/2 in. (12.7 mm)	40 (280)	27 (188) [b]
ASTM A307 Bolts, Grade A d ≥ 1/2 in (12.7 mm)	45 (310)	27 (188) [b]
ASTM A325 Bolts, when threads are not excluded from shear planes	90 (620)	54 (372)
ASTM A325 Bolts, when threads are excluded from shear planes	90 (620)	68 (457)
ASTM 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 (700)	68 (457)
ASTM 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 (700)	84 (579)
ASTM A449 Bolts 1/4 in. (6.4 mm) ≤ d < 1/2 in. (12.7 mm), when threads are not excluded from shear planes	81 (560)	54 (372)
ASTM A449 Bolts 1/4 in. (6.4 mm) ≤ d < 1/2 in. (12.7 mm), when threads are excluded from shear planes	81 (560)	68 (457)
ASTM A490 Bolts, when threads are not excluded from shear planes	113 (780)	68 (457)
ASTM A490 Bolts, when threads are excluded from shear planes	113 (780)	84 (579)
Threaded parts, when threads are not excluded from shear planes	0.75 F_u	0.450 F_u
Threaded parts, when threads are excluded from shear planes	0.75 F_u	0.563 F_u

Notes:

a. For end-loaded *connections* with a fastener pattern length greater than 38 in. (965 mm), F_{nv} should be reduced to 83.3% of the tabulated values. Fastener pattern length is the maximum distance parallel to the line of force between the centerline of the bolts connecting two parts with one faying surface.

b. Threads permitted in shear planes.

E6a Rupture

Connections in which the *thickness* of the thinnest part is greater than 3/16 in. (4.76 mm) shall be in accordance with ANSI/AISC 360.

Appendix B:

Provisions Applicable to Canada

2012 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* (*NBCC*).

While this document is referred to as a "*Specification,*" in Canada it is considered a "*Standard*."

Also included in Appendix B are technical items where full agreement between the three countries was not reached. The most noteworthy of these items are the following:

(1) Beams (C- and Z-sections) for standing seam roofs,
(2) Bolted and welded connections, and
(3) Lateral and stability bracing.

Efforts will be made to minimize these differences in future editions of the *Specification.*

In Canada, SI units are the units of record for the purpose of this *Specification*.

APPENDIX B: PROVISIONS APPLICABLE TO CANADA

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 in this Appendix presents the entire design provision.

A1.3a Definitions

The following additional definitions apply 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.

A2.1.1a Applicable Steels

These steels are in addition to those listed in Section A2.1.1 of the *Specification*:

CSA G40.20-13/G40.21-13, *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 of the *Specification*, 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 of the *Specification*.

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 of the *Specification*. 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 of the *Specification*.

A2.3.5a Ductility Requirements of Other Steels

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 A2.3.5 of the *Specification*, 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 2, Division B, Part 4, Earthquake Load and Effects, of the *NBCC*.

A3 Loads

The *resistance factors* adopted in this *Specification* are correlated with the *loads* and *load factors* for buildings specified in the *NBCC*. 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 of this Appendix, with 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:

(a) When the dead *load* acts to resist overturning, uplift, sliding, and failure due to *stress* reversal; and

(b) 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, 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 of this Appendix.

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, or 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 defence 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 in Table A6.1.2.2-1 of this Appendix.

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. CSA Group, 5060 Spectrum Way, Suite 100, Mississauga, ON, Canada, L4W 5N6:

   G40.20-13/G40.21-13, *General requirements for rolled or welded structural quality steel/Structural quality steel*

   S16-09, *Design of steel structures*

   W47.1-09, *Certification of companies for fusion welding of steel*

   W55.3-08, *Certification of companies for resistance welding of steel and aluminum*

   W59 (*under development*), *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 (*NBCC*), 2010

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

Discrete bracing of axially loaded compression members shall meet the requirements as specified in Section D3.3 of the *Specification*. In addition, the provisions of Sections D3.1.1a and D3.1.2a of this Appendix shall apply to symmetric sections in compression or bending in which the applied *load* does not induce twist.

D3.1.1a 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.

D3.1.2a 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 D3.1.2a 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 D3.1.2a 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 fabricators and erectors certified by the Canadian Welding Bureau (CWB) to the requirements of CSA W47.1 (Division 1 or Division 2). The work may be sublet to a Division 3 fabricator or erector; however, the Division 1 or Division 2 fabricator or erector shall retain responsibility for the sublet work. Resistance welding shall be performed by fabricators or erectors certified by the CWB to the requirements of 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 mm and 4.76 mm in base steel *thickness*, welding shall conform to the requirements contained herein and shall be performed in accordance with the applicable requirements of CSA W59. Except as provided for in Section E2.2 of the *Specification*, where at least one of the connected parts is less than 0.70 mm in base steel *thickness*, welds shall be considered to have no structural value unless a value is substantiated by appropriate tests. For arc spot welds connecting sheets to a thicker supporting member, the applicable base steel *thickness* limits shall be 0.70 mm to 5.84 mm.

The resistance in tension or compression of butt welds shall be the same as that prescribed for the lower strength of base metal being joined. The butt weld shall fully penetrate the *joint*.

E3a Bolted Connections

In addition to the design criteria given in Section E3 of the *Specification*, the following design requirements shall be followed for bolted *connections* used for *cold-formed steel structural members* in which 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. Bolted *connections* in which the *thickness* of the thinnest connected part is greater than 4.76 mm shall comply with CSA S16.

Unless otherwise specified, the standard hole diameter 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.

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 A307 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:

$$P_n = A_b F_n \quad (Eq.\ E3.4\text{-}1)$$

where

A_b = Gross *cross-sectional area* of bolt

F_n = A value determined in accordance with Items (a) and (b) below, as applicable:

(a) When bolts are subjected to shear or tension,
F_n is given by F_{nt} or F_{nv} in Table E3.4-1, as well as the ϕ values.

(b) 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	$324 - 2.4f_v \leq 279$	0.65

Note: The actual shear *stress*, f_v, shall also satisfy Table E3.4-1 of this Appendix.

E6a Rupture

When the *thickness* of connected steels is larger than 4.76 mm, the requirements of CSA S16 shall be met for *connection* design.

For *connection* types utilizing screws, the *nominal rupture resistance*, R_n, shall be the lesser of the values obtained in accordance with Sections E6.1, E6.2, and E6.3 of the *Specification*, as applicable.

F1.1a Load and Resistance Factor Design and Limit States Design

To calculate the *resistance factor* of an interior partition wall stud in a composite steel-framed wall system with gypsum sheathing attached to both *flanges* and that is limited to a transverse (out-of-plane) *specified load* of not more than 0.5 kPa, a superimposed *specified* axial *load*, exclusive of sheathing materials, of not more than 1.46 kN/m, or a superimposed *specified* axial *load* not more than 0.89 kN, the following shall apply:

C_ϕ = 1.42
M_m = 1.10
F_m = 1.00
V_M = 0.10
V_F = 0.05
β_o = 1.82

These provisions shall not apply to members in walls acting as guards, as defined in the *NBCC*.

American Iron and Steel Institute

AISI STANDARD

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

2012 EDITION

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

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

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

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PREFACE

This document provides a commentary on the 2012 edition of the *North American Specification for the Design of Cold-Formed Steel Structural Members.* This *Commentary* should be used in combination with the 2013 edition of the *AISI Cold-Formed Steel Design Manual*.

The purpose of the *Commentary* is: (a) to provide a record of the reasoning behind, and justification for, the various provisions of the *North American Specification* by cross-referencing the published supporting research data, and to discuss the changes made in the current *Specification;* (b) to offer a brief but coherent presentation of the characteristics and performance of cold-formed steel structures to structural engineers and other interested individuals; (c) to furnish the background material for a study of cold-formed steel design methods to educators and students; and (d) to provide the needed information to those who will be responsible for future revisions of the *Specification.* The readers who wish to have more complete information, or who may have questions which are not answered by the abbreviated presentation of this *Commentary*, should refer to the original research publications.

Consistent with the *Specification*, the *Commentary* contains a main document, Chapters A through G, Appendices 1 and 2, and Appendices A and B. A symbol ☞**A,B** is used in the main document to point out that additional discussions are provided in the corresponding country-specific provisions in Appendices A or B.

The assistance and close cooperation of the North American Specification Committee under the Chairmanship of Professor Reinhold M. Schuster and the AISI Committee on Specifications under the Chairmanship of Mr. Roger L. Brockenbrough and the Vice Chairmanship of Mr. Richard Haws are gratefully acknowledged. The Institute is very grateful to members of the Editorial Task Group and all members of the AISI Committee on Specifications for their careful review of the document and their valuable comments and suggestions. The background materials provided by various subcommittees are appreciated.

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November 2012

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

INTRODUCTION

Cold-formed steel members have been used economically for building construction and other applications (Winter, 1959a, 1959b; Yu and LaBoube, 2010). These types of sections are cold-formed from steel sheet, strip, plate or flat bar in roll-forming machines or by press brake or bending operations. The *thicknesses* of steel sheets or strips generally used for *cold-formed steel structural members* range from 0.0147 in. (0.373 mm) to about 1/4 in. (6.35 mm). Steel plates and bars as thick as 1 in. (25.4 mm) can be cold-formed successfully into structural shapes.

In general, *cold-formed steel structural members* can offer several advantages for building construction (Winter, 1970; Yu and LaBoube, 2010): (1) Light members can be manufactured for relatively light loads and/or short spans, (2) Unusual sectional configurations can be produced economically by cold-forming operations and consequently favorable strength-to-weight ratios can be obtained, (3) Load-carrying panels and decks can provide useful surfaces for floor, roof and wall construction, and in some cases they can also provide enclosed cells for electrical and other conduits, and (4) Panels and decks not only withstand loads normal to their surfaces, but they can also act as shear *diaphragms* to resist forces in their own planes if they are adequately interconnected to each other and to supporting members.

The use of cold-formed steel members in building construction began around the 1850s. However, in North America, such steel members were not widely used in buildings until the publication of the first edition of the American Iron and Steel Institute (AISI) *Specification* in 1946 (AISI, 1946). This first design standard was primarily based on the research work sponsored by AISI at Cornell University since 1939. It was revised subsequently by the AISI Committee in 1956, 1960, 1962, 1968, 1980, and 1986 to reflect the technical developments and the results of continuing research. In 1991, AISI published the first edition of the *Load and Resistance Factor Design Specification for Cold-Formed Steel Structural Members* (AISI, 1991). Both *Allowable Stress Design* (*ASD*) and *Load and Resistance Factor Design* (*LRFD*) *Specifications* were combined into a single document in 1996. In Canada, the Canadian Standards Association (CSA) published its first edition of *Design of Light Gauge Steel Structural Members* in 1963 based on the 1962 edition of the AISI *Specification*. Subsequent editions were published in 1974, 1984, 1989 and 1994. The Canadian *Standard for Cold Formed Steel Structural Members* (CSA, 1994) was based on the *Limit States Design* (*LSD*) method.

In Mexico, *cold-formed steel structural members* have also been designed on the basis of AISI *Specifications*. The 1962 edition of the AISI *Design Manual* (AISI, 1962) was translated to Spanish in 1965 (Camara, 1965).

The first edition of the unified *North American Specification* (AISI, 2001) was prepared and issued in 2001. It was applicable to the United States, Canada, and Mexico for the design of *cold-formed steel structural members*. The 2001 edition of the *Specification* was developed on the basis of the 1996 AISI *Specification* with the 1999 *Supplement* (AISI, 1996, 1999), the 1994 CSA *Standard* (CSA, 1994), and subsequent developments. In 2001, the term "Allowable Stress Design" was renamed to "Allowable Strength Design" to clarify the nature of this design method. In the *North American Specification*, the *ASD* and *LRFD* methods are used in the United States and Mexico, while the LSD method is used in Canada. The *Supplement* to the 2001 edition of the *North American Specification* was published in 2004 (AISI, 2004b), in which the new *Direct*

Strength Method was added in the *Specification* as Appendix 1. Following the successful use of the first *North American Specification*, it was revised and expanded in 2007 on the basis of the results of continued research and new developments (AISI, 2007a). The 2007 edition of the *Specification* includes Appendix 2 for the Second-Order Analysis of structural systems. Additionally, Appendix A has been expanded to be applicable to Mexico and, consequently, Appendix C has been deleted. In 2009 and 2010, Supplements 1 and 2 to the *North American Specification* (AISI, 2009; AISI 2010) were published, and in 2012, a new edition of the *North American Specification* (AISI, 2012) was published. In the 2012 edition of the *Specification*, the design of *power-actuated fasteners* is included, and the design using the Direct Strength Method has been broadened to include compression and flexural strength for perforated members, shear strength for non-perforated members, and member reserve capacities.

In addition to the issuance of the design specification, AISI also published the first edition of the *Design Manual* in 1949 (AISI, 1949). This *Allowable Stress Design* manual was revised in 1956, 1961, 1962, 1968, 1977, 1983, and 1986. In 1991, the *LRFD Design Manual* was published for using the *Load and Resistance Factor Design* criteria. The AISI 1996 *Cold-Formed Steel Design Manual* was prepared for the combined AISI *ASD* and *LRFD Specifications.* To assist the users to better understand the *North American Specification*, AISI updates and publishes a new edition of the *Cold-Formed Steel Design Manual* (AISI, 2002; AISI 2008; AISI, 2013). In 2013, the new *Cold-Formed Steel Design Manual* (AISI, 2013) is published by AISI based on the 2012 edition of the *North American Specification*.

During the period from 1958 through 1983, AISI published *Commentaries* on several editions of the AISI design *Specifications*, which were prepared by Professor George Winter of Cornell University in 1958, 1961, 1962, and 1970. Since 1983, the format used for the AISI *Commentary* has been changed so that the same section numbers are used for the *Commentary* as for the *Specification.* The *Commentary* on the 1996 AISI *Specification* was prepared by Professor Wei-Wen Yu of the University of Missouri-Rolla (Yu, 1996). The 2001 edition of the *Commentary* (AISI, 2001) was based on the *Commentary* for the 1996 AISI *Specification*. The current edition of the *Commentary* (AISI, 2012b) is updated for the 2012 edition of the *North American Specification*, and it contains Chapters A through G, Appendices 1 and 2, and Appendices A and B, where commentary on provisions that are only applicable to a specific country is included in the corresponding Appendix.

As in previous editions of the *Commentary*, this document contains a brief presentation of the characteristics and performance of cold-formed steel structural members, *connections* and assemblies. In addition, it provides a record of the reasoning behind, and the justification for, various provisions of the *Specification*. A cross-reference is provided between various design provisions and the published research data.

In this *Commentary,* the individual sections, equations, figures, and tables are identified by the same notation as in the *Specification* and the material is presented in the same sequence. Bracketed terms used in the *Commentary* are equivalent terms that apply particularly to the *LSD* method in Canada.

The *Specification* and *Commentary* are intended for use by design professionals with demonstrated engineering competence in their fields.

A. GENERAL PROVISIONS

A1 Scope, Applicability, and Definitions

A1.1 Scope

The cross-sectional configurations, manufacturing processes and fabrication practices of *cold-formed steel structural members* differ in several respects from those of hot-rolled steel shapes. For cold-formed steel sections, the forming process is performed at, or near, room temperature by the use of bending brakes, press brakes, or roll-forming machines. Some of the significant differences between cold-formed sections and hot-rolled shapes are: (1) absence of the residual *stresses* caused by uneven cooling due to hot-rolling, (2) lack of corner fillets, (3) presence of increased *yield stress* with decreased proportional limit and ductility resulting from cold-forming, (4) presence of cold-reducing *stresses* when cold-rolled steel stock has not been finally annealed, (5) prevalence of elements having large width-to-*thickness* ratios, (6) rounded corners, and (7) different characteristics of *stress*-strain curves, that can be either the sharp-yielding type or gradual-yielding type.

The *Specification* is applicable only to cold-formed sections not more than 1 inch (25.4 mm) in *thickness*. Research conducted at the University of Missouri-Rolla (Yu, Liu, and McKinney, 1973b and 1974) has verified the applicability of the *Specification*'s provisions for such cases.

In view of the fact that most of the design provisions have been developed on the basis of the experimental work subject to static loading, the *Specification* is intended for the design of *cold-formed steel structural members* to be used for load-carrying purposes in buildings. For structures other than buildings, appropriate allowances should be made for dynamic effects.

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A1.2 Applicability

The *Specification* (AISI, 2012a) is limited to the design of steel structural members cold-formed from carbon or low-alloy sheet, strip, plate or bar. The design can be made by using either the *Allowable Strength Design* (*ASD*) method or the *Load and Resistance Factor Design* (*LRFD*) method for the United States and Mexico. Only the *Limit States Design* (*LSD*) method is permitted in Canada.

In this *Commentary*, the bracketed terms are equivalent terms that apply particularly to *LSD*. A symbol ☞x is used to point out that additional provisions are provided in the country-specific appendices as indicated by the letter, x.

Because of the diverse forms of *cold-formed steel structural members* and *connections*, it is not possible to cover all design configurations by the design rules presented in the *Specification*. For those special cases where the *available strength* [*factored resistance*] and/or stiffness cannot be so determined, it can be established by: (a) testing in accordance with the provisions of Section F1.1(a), (b) *rational engineering analysis* and verification testing evaluated in accordance with the provisions of Section F1.1(b), or (c) *rational engineering analysis* only in accordance with the provisions of Section A1.2(c). Prior to 2001, the only option in such cases was testing. Since 2001, in recognition of the fact that this was not always practical or necessary, the *rational engineering analysis* options were added. It is essential that such analysis be based on theory that is appropriate for the situation and sound engineering judgment. *Specification* Section A1.2(b) was added for components that have significant geometric variations such that it becomes impractical to test each variation in accordance with *Specification* Section A1.2(a). This is particularly useful when the following applies:

(1) a form of cold-formed steel component is being evaluated that is outside the scope of the *Specification,*

(2) the member or assembly being evaluated has a degree of variation, such as variations in cross-sectional dimensions, that makes it impractical to test each individual variation,

(3) more accurate *safety* and *resistance factors* than those prescribed by Section A1.2(c) are desired, and

(4) a test program can be conducted in accordance with Chapter F.

In any case, *safety* and *resistance factors* should not be used if applicable *safety factors* or *resistance factors* in the main *Specification* are more conservative, where the main *Specification* refers to Chapters A through G, Appendices A and B, and Appendix 2. These provisions must not be used to circumvent the intent of the *Specification*. Where the provisions of Chapters B through G of the *Specification* and Appendices A and B apply, those provisions must be used and cannot be avoided by testing or rational analysis.

In 2004, Appendix 1, Design of Cold-Formed Steel Structural Members Using the Direct Strength Method, was introduced (AISI, 2004b). The Appendix provides an alternative design procedure for several sections of Chapter C. The *Direct Strength Method* detailed in Appendix 1 requires: (1) determination of the elastic *buckling* behavior of the member, and then provides (2) a series of *nominal strength* [*resistance*] curves for predicting the member strength [resistance] based on the elastic *buckling* behavior. The procedure does not require *effective width* calculations nor iteration, and instead uses gross properties and the elastic *buckling* behavior of the cross-section to predict the *strength* [*resistance*]. The applicability of the provided provisions is detailed in the General Provisions of Appendix 1.

In 2007, Appendix 2, Second-Order Analysis, was added in the *Specification* (AISI, 2007a). The provisions of this Appendix are based on the studies conducted by Sarawit and Peköz (2006) at Cornell University with due considerations given to *flexural-torsional buckling*, semi-rigid *joints*, and local instabilities. The *second-order analysis* was found to be more accurate than the effective length approach.

A1.3 Definitions

Many of the definitions in *Specification* Section A1.3 for *ASD*, *LRFD* and *LSD* are self-explanatory. Only those which are not self-explanatory are briefly discussed below.

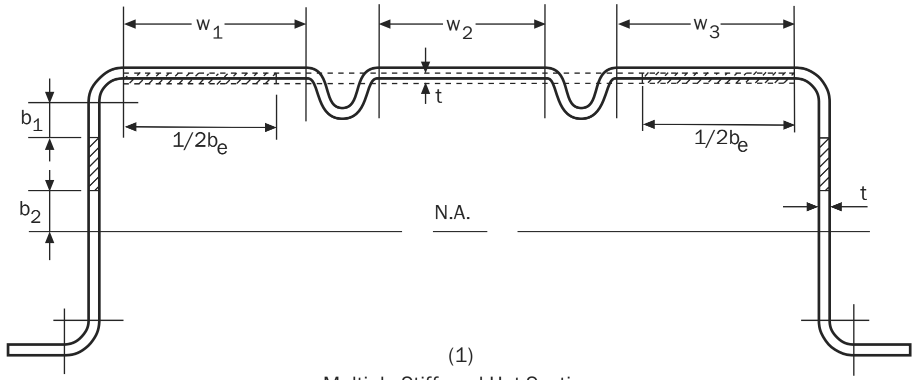
General Terms

Effective Design Width

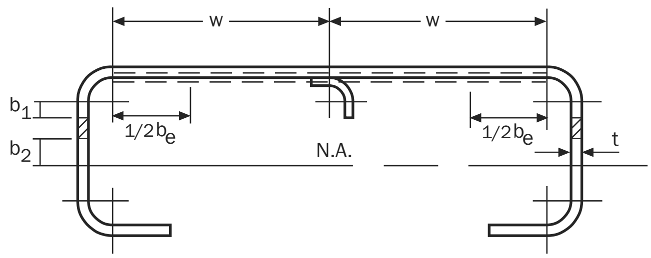
The *effective design width* is a concept which facilitates taking account of *local buckling* and post-*buckling* strength for compression elements. The effect of shear lag on short, wide *flanges* is also handled by using an *effective design width*. These matters are treated in *Specification* Chapter B, and the corresponding *effective widths* are discussed in the *Commentary* on that chapter.

Multiple-Stiffened Elements

Multiple-stiffened elements of two sections are shown in Figure C-A1.3-1. Each of the two



(1)
Multiple Stiffened Hat-Section



(2)
Multiple Stiffened Inverted "U"-Type Section

Flexural Members, Such as Beams

Figure C-A1.3-1 Multiple-Stiffened Compression Elements

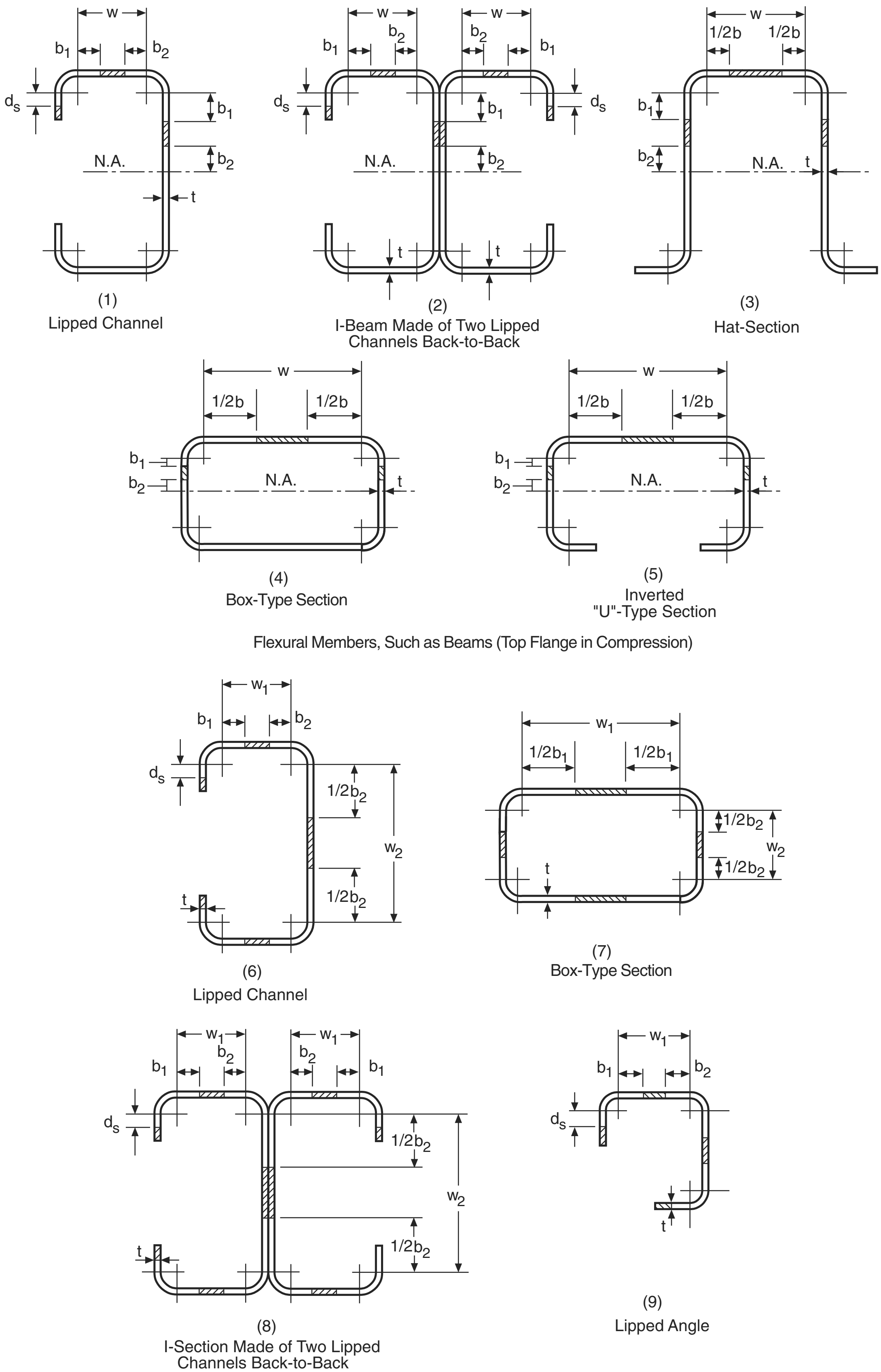
outer sub-elements of section (1) is stiffened by a *web* and an intermediate stiffener while the middle sub-element is stiffened by two intermediate stiffeners. The two sub-elements of section (2) are stiffened by a *web* and the attached intermediate middle stiffener.

Stiffened or Partially Stiffened Compression Elements

Stiffened compression elements of various sections are shown in Figure C-A1.3-2, in which sections (1) through (5) are for flexural members, and sections (6) through (9) are for compression members. Sections (1) and (2) each have a *web* and a lip to stiffen the compression element (i.e., the compression *flange*), the ineffective portion of which is shown shaded. For the explanation of these ineffective portions, see the discussion of *Effective Design Width* and Chapter B. Sections (3), (4), and (5) show compression elements stiffened by two *webs*. Sections (6) and (8) show edge-stiffened *flange* elements that have a vertical element (*web*) and an edge stiffener (lip) to stiffen the elements while the *web* itself is stiffened by the *flanges*. Section (7) has four compression elements stiffening each other, and section (9) has each stiffened element stiffened by a lip and the other stiffened element.

Thickness

In calculating section properties, the reduction in *thickness* that occurs at corner bends is ignored, and the base metal *thickness* of the flat steel stock, exclusive of coatings, is used in all calculations for load-carrying purposes.



(1) Lipped Channel

(2) I-Beam Made of Two Lipped Channels Back-to-Back

(3) Hat-Section

(4) Box-Type Section

(5) Inverted "U"-Type Section

Flexural Members, Such as Beams (Top Flange in Compression)

(6) Lipped Channel

(7) Box-Type Section

(8) I-Section Made of Two Lipped Channels Back-to-Back

(9) Lipped Angle

Compression Members, Such as Columns

Figure C-A1.3-2 Stiffened Compression Elements

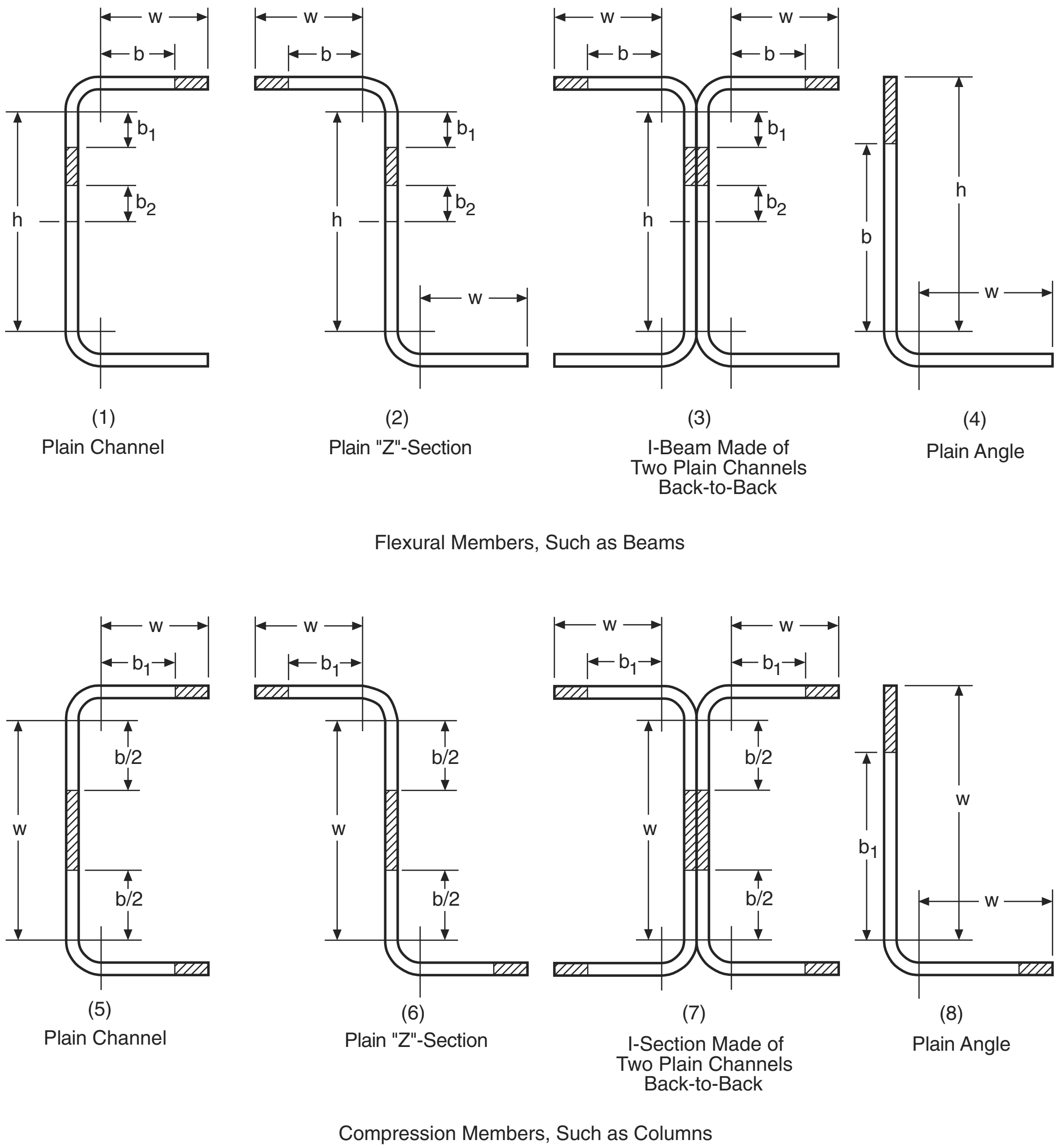


Figure C-A1.3-3 Unstiffened Compression Elements

Flexural-Torsional Buckling

The 1968 edition of the *Specification* pioneered methods for computing column loads of cold-formed steel sections prone to buckling by simultaneous twisting and bending. This complex behavior may result in lower column loads than would result from primary *buckling* by flexure alone.

Unstiffened Compression Elements

Unstiffened elements of various sections are shown in Figure C-A1.3-3, in which sections (1) through (4) are for flexural members and sections (5) through (8) are for compression members. Sections (1), (2), and (3) have only a *web* to stiffen the compression *flange* element. The legs of section (4) provide mutual stiffening action to each other along their common edges. Sections (5), (6), and (7), acting as columns, have vertical stiffened elements (*webs*) which provide support for one edge of the unstiffened *flange* elements. The legs of section (8) provide mutual stiffening action to each other.

ASD and LRFD Terms (USA and Mexico)

ASD (Allowable Strength Design, formerly *referred to as Allowable Stress Design)*

Allowable Strength Design (*ASD*) is a method of designing *structural components* such that the *allowable strength* (force or moment) permitted by various sections of the *Specification* is not exceeded when the structure is subjected to all appropriate combinations of *nominal loads* as given in Section A4.1.2 of Appendix A of the *Specification*.

LRFD (Load and Resistance Factor Design)

Load and Resistance Factor Design (*LRFD*) is a method of designing *structural components* such that the applicable limit state is not exceeded when the structure is subjected to all appropriate load combinations as given in Section A5.1.2 of Appendix A of the *Specification*. See also *Specification* Section A5.1.1 for *LRFD* strength requirements.

LSD Terms (Canada)

LSD (Limit States Design)

Limit States Design (*LSD*) is a method of designing *structural components* such that the applicable limit state is not exceeded when the structure is subjected to all appropriate load combinations as given in Section A6.1.2 of Appendix B of the *Specification*. See also *Specification* Section A6.1.1 for *LSD* requirements.

In the *North American Specification*, the terminologies for *Limit States Design* (*LSD*) are given in brackets parallel to those for *load and resistance factor design* (*LRFD*). The inclusion of *LSD* terminology is intended to help engineers who are familiar with *LSD* better understand the *Specification*.

It should be noted that the design concept used for the *LRFD* and the *LSD* methods is the same, except that the *load factors*, load combinations, assumed dead-to-live ratios, and target reliability indexes are slightly different. In most cases, same *nominal strength* [*resistance*] equations are used for *ASD*, *LRFD*, and *LSD* approaches.

A1.4 Units of Symbols and Terms

The non-dimensional character of the majority of the *Specification* provisions is intended to facilitate design in any compatible systems of units (U.S. customary, SI or metric, and MKS systems).

The conversion of U.S. customary into SI metric units and MKS systems are given in parentheses throughout the entire text of the *Specification* and *Commentary*. Table C-A1.4-1 is a conversion table for these three different units.

Table C-A1.4-1
Conversion Table

	To Convert	To	Multiply by
Length	in.	mm	25.4
	mm	in.	0.03937
	ft	m	0.30480
	m	ft	3.28084
Area	in^2	mm^2	645.160
	mm^2	in^2	0.00155
	ft^2	m^2	0.09290
	m^2	ft^2	10.7639
Force	kip	kN	4.448
	kip	kg	453.5
	lb	N	4.448
	lb	kg	0.4535
	kN	kip	0.2248
	kN	kg	101.96
	kg	kip	0.0022
	kg	N	9.808
Stress	ksi	MPa	6.895
	ksi	kg/cm^2	70.30
	MPa	ksi	0.145
	MPa	kg/cm^2	10.196
	kg/cm^2	ksi	0.0142
	kg/cm^2	MPa	0.0981

A2 Material

A2.1 Applicable Steels

ASTM International is the basic source of steel designations for use with the *Specification.* Section A2.1 contains the complete list of ASTM Standards for steels that are accepted by the *Specification.* Dates of issue are included in Section A9. Other standards that are applicable to a specific country are listed in the corresponding appendix.

In 2012, the list of applicable steels was enhanced by categorizing them into three groups based on the specified minimum elongation in a 2-inch (50-mm) gage length: ten (10) percent or greater elongation, three (3) percent to ten (10) percent elongation, and less than three (3) percent elongation. This eliminated the need to identify specific steel grades in subsequent sections.

In the AISI 1996 *Specification*, the ASTM A446 Standard was replaced by the ASTM A653/A653M Standard. At the same time, the ASTM A283/A283M Standard, High-Strength, Low-Alloy Steel (HSLAS) Grades 70 (480) and 80 (550) of ASTM A653/A653M and ASTM A715 were added.

In 2001, the ASTM A1008/A1008M and ASTM A1011/A1011M Standards replaced the ASTM A570/A570M, ASTM A607, ASTM A611, and ASTM A715 Standards. ASTM A1003/A1003M was added to the list of *Specification* Section A2.1.

In 2007, the ASTM A1039 Standard was added to the list of *Specification* Section A2.1. For all grades of steel, ASTM A1039 complies with the minimum required F_u/F_y ratio of 1.08. *Thicknesses* equal to or greater than 0.064 in. (1.6 mm) and less than or equal to 0.078 in. (2.0 mm) also meet the minimum elongation requirements of *Specification* Section A2.3.1 and no reduction in the *specified minimum yield stress* is required. However, steel *thicknesses* less than 0.064 in. (1.6 mm) with *yield stresses* greater than 55 ksi (380 MPa) do not meet the requirements of *Specification* Section A2.3.1 and are subject to the limitations of *Specification* Section A2.3.2.

In 2012, the ASTM A1063/A1063M Standard was added to the list of *Specification* Section A2.1. The ASTM A1063/A1063M Standard is intended to be a match to ASTM A653/A653M, but the materials are produced using a "twin-roll casting process," which is also used to produce materials conforming to the ASTM A1039/A1039M Standard.

The important material properties for the design of cold-formed steel members are *yield stress*, *tensile strength*, and ductility. Ductility is the ability of steel to undergo sizable plastic or permanent strains before fracturing and is important both for structural safety and for cold-forming. It is usually measured by the elongation in a 2-inch (50-mm) gage length. The ratio of the *tensile strength* to the *yield stress* is also an important material property; this is an indication of strain hardening and the ability of the material to redistribute *stress*.

A2.1.1 Steels With a Specified Minimum Elongation of Ten Percent or Greater (Elongation ≥ 10%)

For the listed ASTM Standards, the *yield stresses* of steels range from 24 to 80 ksi (165 to 550 MPa or 1690 to 5620 kg/cm^2) and the *tensile strengths* vary from 42 to 100 ksi (290 to 690 MPa or 2950 to 7030 kg/cm^2). The tensile-to-yield ratios are no less than 1.13, and the elongations are no less than 10 percent. Exceptions are ASTM A653/A653M *SS* Grade 80 (550); specific *thicknesses* of ASTM A1039/A1039M 55 (380), 60 (410), 70 (480), and 80 (550), ASTM A1008/A1008M *SS* Grade 80 (550); and ASTM A792/A792M *SS* Grade 80 (550) steels with a *specified minimum yield stress* of 80 ksi (550 MPa or 5620 kg/cm^2), a specified minimum *tensile strength* of 82 ksi (565 MPa or 5770 kg/cm^2), and with no stipulated minimum elongation in two (2) inches (51 mm). These low-ductility steels permit only limited amounts of cold forming, require fairly large corner radii, and have other limits on their applicability for structural framing members. Nevertheless, they have been used successfully for specific applications, such as decks and panels with large corner radii and little, if any, *stress* concentrations. The conditions for use of these *SS* Grade 80 (550) steels are outlined in *Specification* Section A2.3.2.

For ASTM A1003/A1003M steel, even though the minimum *tensile strength* is not specified in the ASTM Standard for each of Types H and L steels, the footnote of Table 2 of the Standard states that for Type H steels, the ratio of *tensile strength* to *yield stress* shall not be less than 1.08. Thus, a conservative value of $F_u = 1.08\ F_y$ can be used for the design of cold-formed steel members using Type H steels. Based on the same Standard, a conservative value of $F_u = F_y$ can be used for the design of *purlins* and *girts* using Type L steels. In 2004, the *Specification* listing of ASTM A1003/A1003M steel was revised to list only the grades designated Type H, because this is the only grade that satisfies the criterion for unrestricted usage. Grades designated Type L can still be used but are subject to the restrictions of *Specification* Section A2.3.5.

Certain grades of ASTM A653, A792, and A1039 have elongations that vary based upon the *thickness* of the material. Exceptions are provided for those steels that do not belong to the designated group. ☜B

A2.1.2 Steels With a Specified Minimum Elongation From Three Percent to Less Than Ten Percent (3% ≤ Elongation < 10%)

Steels listed in this section have specified minimum elongations less than the 10 percent limitation for unlimited utilization within the *Specification*. However, they do have some defined ductility. Their use is limited based on the restrictions specified in *Specification* Section A2.3.2.

A2.1.3 Steels With a Specified Minimum Elongation of Less Than Three Percent (Elongation < 3%)

ASTM A653/A653M *SS* Grade 80 (550) Class 1 and 2; ASTM A792/A792M Grade 80 (550) Class 1 and 2; ASTM A875 *SS* Grade 80 (550); and ASTM A1008/A1008M *SS* Grade 80 (550) steels have a *specified minimum yield stress* of 80 ksi (550 MPa or 5620 kg/cm^2), a specified minimum *tensile strength* of 82 ksi (565 MPa or 5770 kg/cm^2), and no stipulated minimum elongation in a 2-inch (50-mm) gage length. These low-ductility steels permit only limited amounts of cold forming, require fairly large corner radii, and have other limits on their applicability for structural framing members. Nevertheless, they have been used successfully for specific applications, such as decks and panels with large corner radii and little, if any, *stress* concentrations. The conditions for use of these *SS* Grade 80 (550) steels are outlined in *Specification* Section A2.3.3.

A2.2 Other Steels

Comments on other steels are provided in the corresponding appendices of this *Commentary*. ☜A,B

A2.3 Permitted Uses and Restrictions of Applicable Steels

The nature and importance of ductility and the ways in which this property is measured were briefly discussed in *Commentary* Section A2.1.

A2.3.1 Steels With a Specified Minimum Elongation of Ten Percent or Greater (Elongation ≥ 10%)

Low-carbon sheet and strip steels with *specified minimum yield stress* from 24 to 50 ksi (165 to 345 MPa or 1690 to 3520 kg/cm^2) need to meet ASTM A370 (A1058) specified minimum elongations in a 2-inch (50-mm) gage length of 11 to 30 percent. In order to meet the ductility requirements, steels with *yield stresses* higher than 50 ksi (345 MPa or 3520 kg/cm^2) are often low-alloy steels.

A2.3.2 Steels With a Specified Minimum Elongation From Three Percent to Less Than Ten Percent (3% ≤ Elongation < 10%)

For the determination of the tension strength of members and *connections* in Grade 80 (550) Class 3 steels produced to ASTM A653/A653M and A792/A792M, tension tests on sheet steels and shear tests on *connections* using steel produced to Australian Standard AS1397 G550 (Standards Australia, 2001), which is similar in minimum ductility (2%) to ASTM A792 Grade 80 (550) Class 3 (minimum ductility 3%), were performed at the University of Sydney by Rogers and Hancock. These included sheet steels in tension with and without perforations (Rogers and Hancock, 1997), bolted *connections* in shear (Rogers and Hancock, 1998, Rogers and Hancock, 1999b), screw *connections* in shear (Rogers and Hancock, 1999a), and sheet steel fracture toughness tests (Rogers and Hancock, 2001).

For the determination of the compression strength of members of Grade 80 (550) Class 3 steels produced to ASTM A653/A653M and A792/792M, compression tests of steel produced to Australian Standard AS1397 G550 (which is similar to ASTM A792 Grade 80 (550) Class 3) were performed at the University of Sydney by Yang and Hancock (2004a, 2004b), and Yang, Hancock and Rasmussen (2004). For short-box sections where $F_n = F_y$, the study (Yang and Hancock, 2004a) shows that the limit of the *yield stress* used in design can be 90 percent of the *specified minimum yield stress* F_{sy} for low-ductility steels. For edge-stiffened elements with intermediate stiffener(s), stub compression testing on channel sections (Yang and Hancock 2004b) confirms *Specification* Section B5.2. For long column tests of channel sections (Yang and Hancock, 2004b), *distortional buckling* as well as the interaction of *local* and *distortional buckling* controls the design. The use of $0.9\,F_{sy}$ in the *distortional buckling* equations produces reliable results.

Further, for calculating the *nominal strength* [*resistance*] of concentrically loaded compression members with a closed-box section, Equations A2.3.2-1 and A2.3.2-2, based on the University of Sydney research findings (Yang, Hancock and Rasmussen, 2002), were added in the *Specification* Section A2.3.2 when determining the *nominal axial strength* [*resistance*] according to Section C4.1. The reduction factor R_r specified in Equation A2.3.2-2 is to be applied to the radius of gyration r and allows for the interaction of *local* and *flexural* (Euler) *buckling* of thin high-strength low-ductility steel sections. The reduction factor is a function of the length varying from 0.65 at KL = 0 to 1.0 at $KL = 1.1L_0$, where L_0 is the length at which the *local buckling stress* equals the *flexural buckling stress*.

A2.3.3 Steels With a Specified Minimum Elongation Less Than Three Percent (Elongation < 3%)

SS Grade 80 (550) of ASTM A653/A653M, *SS* Grade 80 (550) of A1008/A1008M, *SS*

Grade 80 (550) of A792/A792M, and *SS* Grade 80 (550) of A875/A875M steels are carbon steels, for which *specified minimum yield stress* is 80 ksi (552 MPa or 5620 kg/cm^2) and no elongation requirement is specified. These steels do not have adequate ductility as defined by *Specification* Section A2.3.1. Their use has been limited in *Specification* Section A2.3.3 to particular multiple-*web* configurations such as roofing, siding, and floor decking.

In the past, the *yield stress* used in design was limited to 75 percent of the *specified minimum yield stress*, or 60 ksi (414 MPa or 4220 kg/cm^2), and the *tensile strength* used in design was limited to 75 percent of the specified minimum *tensile strength*, or 62 ksi (427 MPa or 4360 kg/cm^2), whichever was lower. This introduced a higher *safety factor*, but still made low-ductility steels, such as *SS* Grade 80 and Grade E, useful for the named applications.

Based on the UMR research findings (Wu, Yu, and LaBoube, 1996), Equation A2.3.3-1 was added in *Specification* Section A2.3.3 to determine the reduced *yield stress*, R_bF_{sy}, for the calculation of the *nominal flexural strength* [*resistance*] of multiple-*web* sections such as roofing, siding and floor decking (AISI, 1999). For the unstiffened compression *flange*, Equation A2.3.3-2 was added on the basis of a 1988 UMR study (Pan and Yu, 1988). This revision allows the use of a higher *nominal bending strength* [*resistance*] than previous editions of the AISI *Specification.* When the multiple-*web* section is composed of both stiffened and unstiffened compression *flange* elements, the smallest R_b should be used to determine the reduced *yield stress* for use on the entire section. Different values of the reduced *yield stress* could be used for positive and negative moments.

The equations provided in *Specification* Section A2.3.3 can also be used for calculating the *nominal flexural strength* [*resistance*] when the *available strengths* [*factored resistances*] are determined on the basis of tests as permitted by the alternative method.

It should be noted that Section A2.3.4 should be followed for steel deck used for composite slabs when the deck is used as the tensile reinforcement.

For the calculation of *web crippling* strength of deck panels, although the UMR study (Wu, Yu, and LaBoube, 1997) shows that the *specified minimum yield stress* can be used to calculate the *web crippling* strength of deck panels, the *Specification* is adopting a conservative approach in Section C3.4. The lesser of 0.75 F_{sy} and 60 ksi (414 MPa or 4220 kg/cm^2) is used to determine both the *web crippling* strength and the shear strength for the low-ductility steels. This is consistent with the previous edition of the AISI *Specification*.

Another UMR study (Koka, Yu, and LaBoube, 1997) confirmed that for the *connection* design using *SS* Grade 80 (550) of A653/A653M steel, the *tensile strength* used in design should be taken as 75 percent of the specified minimum *tensile strength* or 62 ksi (427 MPa or 4360 kg/cm^2), whichever is less. It should be noted that the current design provisions are limited only to the design of members and *connections* subjected to static loading without the considerations of *fatigue* strength.

Load tests are permitted, but not for the purpose of using higher loads than can be calculated under *Specification* Chapters B through G.

A2.3.4 Steel Deck as Tensile Reinforcement for Composite Deck-Slabs

Section A2.3.4 needs to be followed for the condition where the steel deck, used as tensile reinforcement, is acting compositely with the concrete in a composite steel deck-slab. During construction, where the deck is acting as a form, this section is not applicable and Sections A2.3.1, A2.3.2 and A2.3.3 are to be followed. The ability of low-ductility steels to adequately perform as tensile reinforcement for composite deck-slabs has been demonstrated by full-scale testing. If the steel can be roll-formed into final shape using the tooling intended for that application without cracking or splitting at the corners or embossments, the steel is acceptable for use. Verification of this acceptability can be accomplished by simple visual observation of the deck after forming.

The steel ultimate strength, F_u, is not used as a design parameter for composite steel deck-slab design, and is therefore not addressed in *Specification* Section A2.3.4.

The design provisions in this section follow the requirements of SDI-C-2011 (SDI, 2011).

A2.3.5 Ductility Requirements of Other Steels

In 1968, because new steels of higher strengths were being developed, sometimes with lower elongations, the question of how much elongation is really needed in a structure was the focus of a study initiated at Cornell University. Steels were studied that had *yield strengths* ranging from 45 to 100 ksi (310 to 690 MPa or 3160 to 7030 kg/cm^2), elongations in 2 inches (50-mm) ranging from 50 to 1.3 percent, and *tensile strength*-to-*yield strength* ratios ranging from 1.51 to 1.00 (Dhalla, Errera and Winter, 1971; Dhalla and Winter, 1974a; Dhalla and Winter, 1974b). The investigators developed elongation requirements for ductile steels. These measurements are more accurate but cumbersome to make; therefore, the investigators recommended the following determination for adequately ductile steels: (1) The *tensile strength*-to-*yield strength* ratio shall not be less than 1.08, and (2) The total elongation in a 2-inch (50-mm) gage length shall not be less than 10 percent, or not less than 7 percent in an 8-inch (200-mm) gage length. Also, the *Specification* limits the use of Chapters B through E to adequately ductile steels. In lieu of the *tensile strength*-to-*yield strength* limit of 1.08, the *Specification* permits the use of elongation requirements using the measurement technique as given by Dhalla and Winter (1974a) (Yu and LaBoube, 2010). Further information on the test procedure should be obtained from AISI S903, *Standard Methods for Determination of Uniform and Local Ductility* (AISI, 2013b). Because of limited experimental verification of the structural performance of members using materials having a *tensile strength*-to-*yield strength* ratio less than 1.08 (Macadam et al., 1988), the *Specification* limits the use of this material to *purlins*, *girts*, and *curtain wall studs* meeting the elastic design requirements of Sections C3.1.1(a), C3.1.2, D6.1.1, D6.1.2, D6.2.1, and additional country-specific requirements given in the appendices. Thus, the use of such steels in other applications is prohibited. However, in *purlins*, *girts*, and *curtain wall studs* (with special country-specific requirements given in the appendices), concurrent axial loads of relatively small magnitude are acceptable providing the requirements of *Specification* Section C5.2 are met and $\Omega_c P/P_n$ does not exceed 0.15 for *allowable strength design*, $P_u/\phi_c P_n$ does not exceed 0.15 for the *Load and Resistance Factor Design*, and $P_f/\phi_c P_n$ does not exceed 0.15 for the *Limit States Design*.

In 2007, *curtain wall studs* were added to the applications for materials having a *tensile strength*-to-*yield strength* ratio less than 1.08. *Curtain wall studs* are repetitive framing members that are typically spaced more closely than *purlins* and *girts*. *Curtain wall studs* are analogous to vertical girts; as such, they are not subjected to snow or other significant sustained gravity *loads*. Pending future research regarding the cyclic performance of *connections*, an exception is noted on use of these lower ductility steels for *curtain wall studs* supporting heavyweight exterior walls in high seismic areas.

With the addition of the provisions of *Specification* Section A2.3.2 in 2012, the use of this alternative approach for the limited range of structural usage is largely superseded by the provisions of *Specification* Section A2.3.2.

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A2.4 Delivered Minimum Thickness

Sheet and strip steels, both coated and uncoated, may be ordered to nominal or minimum *thickness*. If the steel is ordered to minimum *thickness*, all *thickness* tolerances are over (+) and nothing under (-). If the steel is ordered to nominal *thickness*, the *thickness* tolerances are divided equally between over and under. Therefore, in order to provide the similar material *thickness* between the two methods of ordering sheet and strip steel, it was decided to require that the delivered *thickness* of a cold-formed product be at least 95 percent of the design *thickness*. Thus, it is apparent that a portion of the *safety factor* or *resistance factor* may be considered to cover minor negative *thickness* tolerances.

Generally, *thickness* measurements should be made in the center of *flanges*. For decking and siding, measurements should be made as close as practical to the center of the first full flat of the section. *Thickness* measurements should not be made closer to edges than the minimum distances specified in ASTM A568 Standard.

The responsibility of meeting this requirement for a cold-formed product is clearly that of the manufacturer of the product, not the steel producer.

In 2004, the country-specific section, *Specification* Section A2.4a, was deleted from Appendix B.

A3 Loads

Comments on loads and load combinations for different countries are provided in the corresponding appendices of this *Commentary*.

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A4 Allowable Strength Design

A4.1 Design Basis

The *Allowable Strength Design* method has been featured in AISI *Specifications* beginning with the 1946 edition. It is included in the *Specification* along with the *LRFD* and the *LSD* methods for use in the United States, Mexico, and Canada since the 2001 edition.

A4.1.1 ASD Requirements

In the *Allowable Strength Design* approach, the *required strengths* (bending moments, axial forces, and shear forces) in structural members are computed by accepted methods of *structural analysis* for the specified nominal or working *loads* for all applicable load combinations determined according to *Specification* Section A4.1.2. These *required strengths* are not to exceed the *allowable strengths* permitted by the *Specification*. According to *Specification* Section A4.1.1, the *allowable strength* is determined by dividing the *nominal strength* by a *safety factor* as follows:

$$R \leq R_n/\Omega \tag{C-A4.1.1-1}$$

where

R = *required strength*

R_n = *nominal strength*

Ω = *safety factor*

The fundamental nature of the *safety factor* is to compensate for uncertainties inherent in the design, fabrication, or erection of building components, as well as uncertainties in the estimation of applied *loads*. Appropriate *safety factors* are explicitly specified in various sections of the *Specification*. Through experience it has been established that the present *safety factors* provide satisfactory design. It should be noted that the *ASD* method employs only one *safety factor* for a given condition regardless of the type of *load*.

A4.1.2 Load Combinations for ASD

Comments for *load* combinations are provided in Appendix A of this *Commentary*.

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A5 Load and Resistance Factor Design

A5.1 Design Basis

A limit state is the condition at which the structural usefulness of a *load*-carrying element or member is impaired to such an extent that it becomes unsafe for the occupants of the structure, or the element no longer performs its intended function. Typical limit states for cold-formed steel members are excessive deflection, *yielding*, *buckling* and attainment of maximum strength after *local buckling* (i.e., post-*buckling* strength). These limit states have been established through experience in practice or in the laboratory, and they have been thoroughly investigated through analytical and experimental research. The background for the establishment of the limit states is extensively documented (Winter, 1970; Peköz, 1986b; and Yu and LaBoube, 2010), and a continuing research effort provides further improvement in understanding them.

Two types of limit states are considered in the *Load and Resistance Factor Design* method. They are: (1) the limit state of the strength required to resist the extreme *loads* during the intended life of the structure, and (2) the limit state of the ability of the structure to perform its intended function during its life. These two limit states are usually referred to as the limit state of strength and limit state of serviceability. Like the *ASD* method, the *LRFD* method focuses on the limit state of strength in *Specification* Section A5.1.1 and the limit state of serviceability in *Specification* Section A8.

A5.1.1 LRFD Requirements

For the limit state of strength, the general format of the *LRFD* method is expressed by the following equation:

$$\Sigma\gamma_i Q_i \le \phi R_n \qquad \text{(C-A5.1.1-1)}$$

or

$$R_u \le \phi R_n$$

where

R_u = $\Sigma\gamma_i Q_i$ = *required strength*

R_n = nominal *resistance*

ϕ = *resistance factor*

γ_i = *load factors*

Q_i = *load effects*

ϕR_n = *design strength*

The nominal *resistance* is the strength of the element or member for a given limit state, computed for nominal section properties and for minimum specified material properties according to the appropriate analytical model which defines the strength. The *resistance factor* ϕ accounts for the uncertainties and variabilities inherent in the R_n, and it is usually less than unity. The *load effects* Q_i are the forces on the cross-section (i.e., bending moment, axial force, or shear force) determined from the specified *nominal loads* by *structural analysis* and γ_i are the corresponding *load factors*, which account for the uncertainties and variabilities of the *loads*. The *load factors* for *LRFD* are discussed in the *Commentary* on Appendix A for the United States and Mexico.

The advantages of *LRFD* are: (1) the uncertainties and the variabilities of different types of *loads* and resistances are different (e.g., dead *load* is less variable than wind *load*), and so these differences can be accounted for by use of multiple factors, and (2) by using probability theory, designs can ideally achieve a more consistent reliability. Thus *LRFD* provides the basis for a more rational and refined design method than is possible with the *ASD* method.

(a) Probabilistic Concepts

Safety Factors or *load factors* are provided against the uncertainties and variabilities which are inherent in the design process. Structural design consists of comparing nominal *load effects* Q to nominal *resistances* R, but both Q and R are random parameters (see Figure C-A5.1.1-1). A limit state is violated if $R<Q$. While the possibility of this event ever occurring is never zero, a successful design should, nevertheless, have only an acceptably small probability of exceeding the limit state. If the exact probability distributions of Q and R were known, then the probability of $(R - Q) < 0$ could be exactly determined for any design. In general, the distributions of Q and R are not known, and only the means, Q_m and R_m, and the standard deviations, σ_Q and σ_R, are available. Nevertheless it is possible to determine relative reliabilities of several designs by the scheme illustrated in Figure C-A5.1.1-2. The distribution curve shown is for $ln(R/Q)$, and a limit state is exceeded when $ln(R/Q) \leq 0$. The area under $ln(R/Q) \leq 0$ is the probability of violating the limit state. The size of this area is dependent on the distance between the origin and the mean of $ln(R/Q)$. For given statistical data R_m, Q_m, σ_R and σ_Q, the area under $ln(R/Q) \leq 0$ can be varied by changing the value of β (Figure C-A5.1.1-2), since $\beta\sigma_{ln(R/Q)} = ln(R/Q)_m$, from which approximately

$$\beta = \frac{\ln(R_m / Q_m)}{\sqrt{V_R^2 + V_Q^2}} \qquad \text{(C-A5.1.1-2)}$$

where $V_R = \sigma_R/R_m$ and $V_Q = \sigma_Q/Q_m$, the coefficients of variation of R and Q, respectively. The index β is called the "reliability index", and it is a relative measure of the safety of the design. When two designs are compared, the one with the larger β is more reliable.

The concept of the reliability index can be used for determining the relative reliability inherent in current design, and it can be used in testing out the reliability of new design formats, as illustrated by the following example of simply supported, braced beams subjected to dead and live loading.

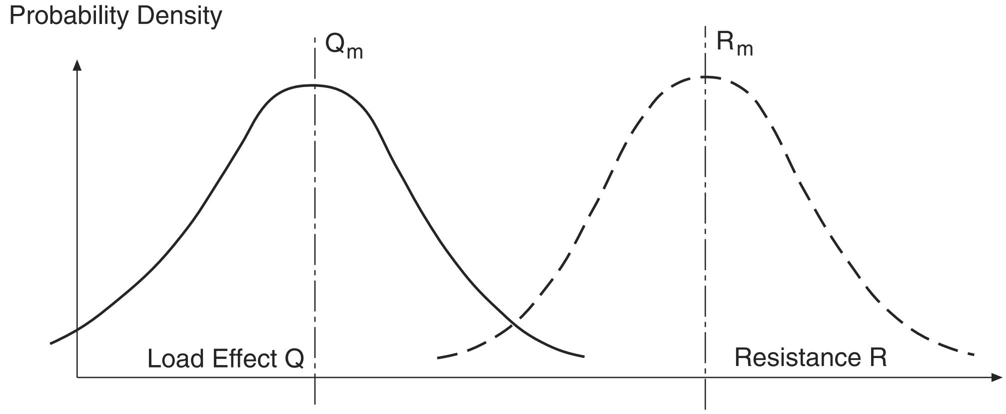

Figure C-A5.1.1-1 Definition of the Randomness Q and R

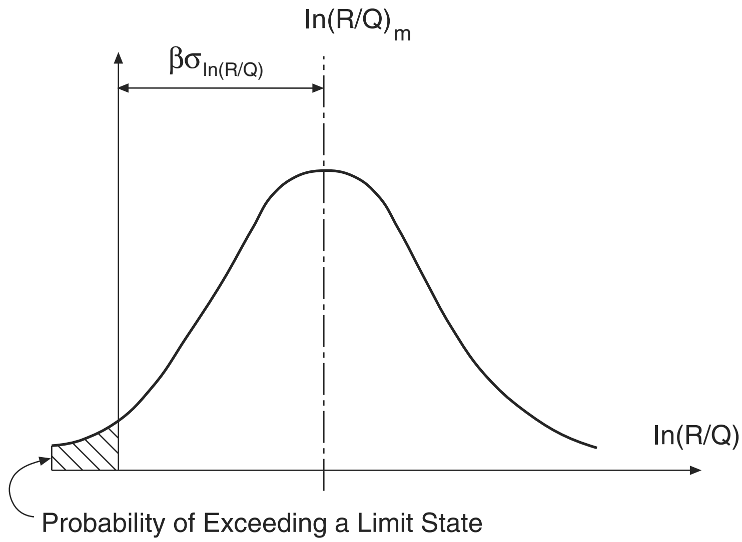

Figure C-A5.1.1-2 Definition of the Reliability Index β

The *ASD* design requirement of the *Specification* for such a beam is

$$S_eF_y/\Omega = (L_s^2s/8)(D+L) \quad \text{(C-A5.1.1-3)}$$

where

S_e = elastic section modulus based on the effective section

Ω = 5/3 = the *safety factor* for bending

F_y = specified *yield stress*

L_s = span length, and s = beam spacing

D and L are, respectively, the code-specified dead and live *load* intensities.

The mean *resistance* is defined as (Ravindra and Galambos, 1978)

$$R_m = R_n(P_mM_mF_m) \quad \text{(C-A5.1.1-4)}$$

In the above equation, R_n is the nominal *resistance*, which in this case is

$$R_n = S_eF_y \quad \text{(C-A5.1.1-5)}$$

that is, the nominal moment predicted on the basis of the post-*buckling* strength of the compression *flange* and the *web*. The mean values P_m, M_m, and F_m, and the corresponding coefficients of variation V_P, V_M, and V_F, are the statistical parameters, which define the variability of the *resistance*:

P_m = mean ratio of the experimentally determined moment to the predicted moment for the actual material and cross-sectional properties of the test specimens

M_m = mean ratio of the actual *yield stress* to the minimum specified value

F_m = mean ratio of the actual section modulus to the specified (nominal) value

The coefficient of variation of R equals

$$V_R = \sqrt{V_P{}^2 + V_M{}^2 + V_F{}^2} \quad \text{(C-A5.1.1-6)}$$

The values of these data were obtained from examining the available tests on beams having different compression *flanges* with partially and fully effective *flanges* and *webs*, and from analyzing data on *yield stress* values from tests and cross-sectional dimensions from many measurements. This information was developed from research (Hsiao, Yu, and Galambos, 1988a and 1990; Hsiao, 1989) and is given below:

P_m = 1.11, V_P = 0.09; M_m = 1.10, V_M = 0.10; F_m = 1.0, V_F = 0.05 and thus $R_m = 1.22R_n$ and V_R = 0.14.

The mean *load effect* is equal to

$$Q_m = (L_s{}^2 s/8)(D_m + L_m) \quad \text{(C-A5.1.1-7)}$$

and

$$V_Q = \frac{\sqrt{(D_m V_D)^2 + (L_m V_L)^2}}{D_m + L_m} \quad \text{(C-A5.1.1-8)}$$

where D_m and L_m are the mean dead and live *load* intensities, respectively, and V_D and V_L are the corresponding coefficients of variation.

Load statistics have been analyzed in a study of the National Bureau of Standards (NBS) (Ellingwood et al., 1980), where it was shown that D_m = 1.05D, V_D = 0.1; L_m = L, V_L = 0.25.

The mean live *load* intensity equals the code live *load* intensity if the tributary area is small enough so that no live *load* reduction is included. Substitution of the *load* statistics into Equations C-A5.1.1-7 and C-A5.1.1-8 gives

$$Q_m = \frac{L_s{}^2 s}{8}\left(\frac{1.05D}{L} + 1\right)L \quad \text{(C-A5.1.1-9)}$$

$$V_Q = \frac{\sqrt{(1.05D/L)^2 V_D{}^2 + V_L{}^2}}{(1.05D/L + 1)} \quad \text{(C-A5.1.1-10)}$$

Q_m and V_Q thus depend on the dead-to-live *load* ratio. Cold-formed steel beams typically have small D/L ratio, which may vary for different applications. Different D/L ratio may be assumed by different countries for developing design criteria. For the purposes of checking the reliability of these *LRFD* criteria, it has been assumed that D/L = 1/5, and so $Q_m = 1.21L(L_s{}^2 s/8)$ and V_Q = 0.21.

From Equations C-A5.1.1-3 and C-A5.1.1-5, the nominal *resistance*, R_n, can be obtained for D/L = 1/5 and Ω = 5/3 as follows:

$R_n = 2L(L_s{}^2 s/8)$

In order to determine the reliability index, β, from Equation C-A5.1.1-2, the R_m/Q_m ratio is required by considering $R_m = 1.22R_n$:

$$\frac{R_m}{Q_m} = \frac{1.22x2.0xL(L_s{}^2 s/8)}{1.21L(L_s{}^2 s/8)} = 2.02$$

Therefore, from Equation C-A5.1.1-2,

$$\beta = \frac{\ln(2.02)}{\sqrt{0.14^2 + 0.21^2}} = 2.79$$

Of itself, β= 2.79 for beams having different compression *flanges* with partially and fully effective *flanges* and *webs* designed by the *Specification* means nothing. However, when this is compared to β for other types of cold-formed steel members, and to β for designs of various types from hot-rolled steel shapes or even for other materials, then it is possible to say that this particular cold-formed steel beam has about an average reliability (Galambos et al., 1982).

(b) Basis for LRFD of Cold-Formed Steel Structures

A great deal of work has been performed for determining the values of the reliability index β inherent in traditional design as exemplified by the current structural design specifications such as the ANSI/AISC S360 for hot-rolled steel, the AISI *Specification* for cold-formed steel, the ACI Code for reinforced concrete members, etc. The studies for hot-rolled steel are summarized by Ravindra and Galambos (1978), where also many other papers are referenced which contain additional data. The determination of β for cold-formed steel elements or members is presented in several research reports of the University of Missouri-Rolla (Hsiao, Yu, and Galambos, 1988a; Rang, Galambos, and Yu, 1979a, 1979b, 1979c, and 1979d; Supornsilaphachai, Galambos, and Yu, 1979), where both the basic research data as well as the β's inherent in the AISI *Specification* are presented in great detail. The β's computed in the above referenced publications were developed with slightly different *load* statistics than those of this *Commentary,* but the essential conclusions remain the same.

The entire set of data for hot-rolled steel and cold-formed steel designs, as well as data for reinforced concrete, aluminum, laminated timber, and masonry walls, was re-analyzed by Ellingwood, Galambos, MacGregor, and Cornell (Ellingwood et al., 1980; Galambos et al., 1982; Ellingwood et al., 1982) using (a) updated *load* statistics and (b) a more advanced level of probability analysis which was able to incorporate probability distributions and to describe the true distributions more realistically. The details of this extensive reanalysis are presented by the investigators. Only the final conclusions from the analysis are summarized below.

The values of the reliability index β vary considerably for the different kinds of loading, the different types of construction, and the different types of members within a given material design specification. In order to achieve more consistent reliability, it was suggested by Ellingwood et al. (1982) that the following values of β would provide this improved consistency while at the same time give, on the average, essentially the same design by the *LRFD* method as is obtained by current design for all materials of construction. These target reliabilities β_o for use in *LRFD* are:

Basic case: Gravity loading, $\beta_o = 3.0$

For *connections*: $\beta_o = 4.5$

For wind loading: $\beta_o = 2.5$

These target reliability indices are the ones inherent in the *load factors* recommended in the ASCE 7-98 Load Standard (ASCE, 1998).

For simply supported, braced cold-formed steel beams with stiffened *flanges*, which were designed according to the *allowable strength design* method in the current *Specification* or to any previous version of the AISI *Specification*, it was shown that for the representative dead-to-live *load* ratio of 1/5, the reliability index $\beta = 2.79$. Considering the fact that for other such *load* ratios, or for other types of members, the reliability index inherent in current cold-formed steel construction could be more or less than this value of 2.79, a somewhat lower target reliability index of $\beta_o = 2.5$ is recommended as a lower limit in the United States. The *resistance factors* ϕ were selected such that $\beta_o = 2.5$ is essentially the lower bound of the actual β's for members. In order to ensure that failure of a structure is not initiated in the *connections*, a higher target reliability of $\beta_o = 3.5$ is recommended for *joints* and fasteners in the United States. These two targets of 2.5 and 3.5 for members and *connections*, respectively, are somewhat lower than those recommended by the ASCE 7-98 (i.e., 3.0 and 4.5, respectively), but they are essentially the same targets as are the basis for the AISC *LRFD Specification* (AISC, 1999). For wind loading, the same ASCE target value of $\beta_o = 2.5$ is used for *connections* in the U.S. *LRFD* method. For flexural members such as individual *purlins*, *girts*, panels, and roof decks subjected to the combination of dead and wind *loads*, the target β_o value used in the United States is reduced to 1.5. With this reduced target reliability index, the design based on the U.S. *LRFD* method is comparable to the U.S. *allowable strength design* method.

(c) Resistance Factors

The following portions of this *Commentary* present the background for the *resistance factors* ϕ which are recommended for various members and *connections* in Chapters B through E (AISI, 1996). These ϕ factors are determined in conformance with the ASCE 7 *load factors* to provide approximately a target β_o of 2.5 for members and 3.5 for *connections*, respectively, for a typical *load* combination 1.2D+1.6L. For practical reasons, it is desirable to have relatively few different *resistance factors*, and so the actual values of β will differ from the derived targets. This means that

$$\phi R_n = c(1.2D+1.6L) = (1.2D/L+1.6)cL \qquad \text{(C-A5.1.1-11)}$$

where c is the deterministic influence coefficient translating *load* intensities to *load effects*.

By assuming D/L = 1/5, Equations C-A5.1.1-11 and C-A5.1.1-9 can be rewritten as follows:

$$R_n = 1.84(cL/\phi) \qquad \text{(C-A5.1.1-12)}$$

$$Q_m = (1.05D/L+1)cL = 1.21cL \qquad \text{(C-A5.1.1-13)}$$

Therefore,

$$R_m/Q_m = (1.521/\phi)(R_m/R_n) \qquad \text{(C-A5.1.1-14)}$$

The ϕ factor can be computed from Equation C-A5.1.1-15 on the basis of Equations C-A5.1.1-2, C-A5.1.1-4 and C-A5.1.1-14 (Hsiao, Yu and Galambos, 1988b, AISI 1996):

$$\phi = 1.521\,(P_m M_m F_m)\exp\left(-\beta_o\sqrt{V_R^2 + V_Q^2}\right) \qquad \text{(C-A5.1.1-15)}$$

in which, β_o is the target reliability index. Other symbols were defined previously.

By knowing the ϕ factor, the corresponding *safety factor*, Ω, for *allowable strength design* can be computed for the *load* combination 1.2D+1.6L as follows:

$$\Omega = (1.2D/L + 1.6)/[\phi(D/L + 1)] \tag{C-A5.1.1-16}$$

where D/L is the dead-to-live *load* ratio for the given condition.

A5.1.2 Load Factors and Load Combinations for LRFD

Comments for *load factors* and *load* combinations are provided in Appendix A of this *Commentary*.

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A6 Limit States Design

A6.1 Design Basis

As with the *LRFD* method, a limit state is the condition at which the structural usefulness of a *load*-carrying element or member is impaired to such an extent that it becomes unsafe for the occupants of the structure, or the element no longer performs its intended function. Typical limit states for cold-formed steel members are excessive deflection, *yielding*, *buckling* and attainment of maximum strength after *local buckling* (i.e., post-*buckling* strength). These limit states have been established through experience in practice or in the laboratory, and they have been thoroughly investigated through analytical and experimental research.

Two types of limit states are considered in the *Limit States Design* method. They are: (1) the limit state of the strength required to resist the extreme *loads* during the intended life of the structure, and (2) the limit state of the ability of the structure to perform its intended function during its life. These two limit states are usually referred to as the limit state of strength and limit state of serviceability. The *LSD* method focuses on the limit state of strength in *Specification* Section A6.1.1 and the limit state of serviceability in *Specification* Section A8.

A6.1.1 LSD Requirements

For the limit state of strength, the general format of the *LSD* method is expressed by the following equation:

$$\phi R_n \geq \Sigma\gamma_i Q_i \tag{C-A6.1.1-1}$$

or

$$\phi R_n \geq R_f$$

where

R_f = $\Sigma\gamma_i Q_i$ = effect of *factored loads*

R_n = nominal *resistance*

ϕ = *resistance factor*

γ_i = *load factors*

Q_i = *load effects*

ϕR_n = *factored resistance*

The nominal *resistance* is the strength of the element or member for a given limit state, computed for nominal section properties and for minimum specified material properties according to the appropriate analytical model which defines the *resistance*. The *resistance factor* ϕ accounts for the uncertainties and variabilities inherent in the R_n, and it is usually less than unity. The *load effects* Q_i are the forces on the cross-section (i.e., bending moment, axial force, or shear force) determined from the specified *nominal loads* by *structural analysis* and γ_i are the corresponding *load factors*, which account for the uncertainties and variabilities of the *loads*. The *load factors* for *LSD* are discussed in the *Commentary* on Appendix B.

Since the design basis for the *LSD* and the *LRFD* is the same, further discussions on how to obtain *resistance factor* using probability analysis can be obtained from Section A5.1.1 (c) of the *Commentary*. However, attention should be paid that target values for members and *connections* as well as the dead-to-live *load* ratio may vary from country to country. These variations lead to the differences in *resistance factors*. The dead-to-live ratio used in Canada is assumed to be 1/3, and the target of the reliability index for *cold-formed steel structural members* is 3.0 for members and 4.0 for *connections*. These target values are consistent with those used in other CSA design standards.

A6.1.2 Load Factors and Load Combinations for LSD

Comments for *load factors* and *load* combinations are provided in *Commentary* Section A6 of Appendix B.

☜B

A7 Yield Stress and Strength Increase From Cold Work of Forming

A7.1 Yield Stress

The strength of *cold-formed steel structural members* depends on the *yield stress*, except in those cases where elastic *local buckling* or overall *buckling* is critical. Because the *stress*-strain curve of steel sheet or strip can be either the sharp-yielding type (Figure C-A7.1-1(a)) or gradual-yielding type (Figure C-A7.1-1(b)), the method for determining the *yield point* for sharp-yielding steel and the *yield strength* for gradual-yielding steel are based on the ASTM Standard A370 (ASTM, 2012). As shown in Figure C-A7.1-2(a), the *yield point* for sharp-yielding steel is defined by the *stress* level of the plateau. For gradual-yielding steel, the *stress*-strain curve is rounded out at the "knee" and the *yield strength* is determined by either the offset method (Figure C-A7.1-2(b)) or the extension under the *load* method (Figure C-A7.1-2(c)). The term *yield stress* used in the *Specification* applies to either *yield point* or *yield strength*. Section 1.2 of the AISI *Design Manual* (AISI, 2013) lists the minimum mechanical properties specified by the ASTM specifications for various steels.

The strength of members that are governed by *buckling* depends not only on the *yield stress* but also on the modulus of elasticity of steel, E, and the tangent modulus of steel, E_t. The modulus of elasticity is defined by the slope of the initial straight portion of the *stress*-strain curve (Figure C-A7.1-1). The measured values of E on the basis of the standard methods usually range from 29,000 to 30,000 ksi (200 to 207 GPa or $2.0\text{x}10^6$ to $2.1\text{x}10^6$ kg/cm^2). A value of 29,500 ksi (203 GPa or $2.07\text{x}10^6$ kg/cm^2) is used in the *Specification* for design purposes. The tangent modulus is defined by the slope of the *stress*-strain curve at any *stress* level, as shown in Figure C-A7.1-1(b).

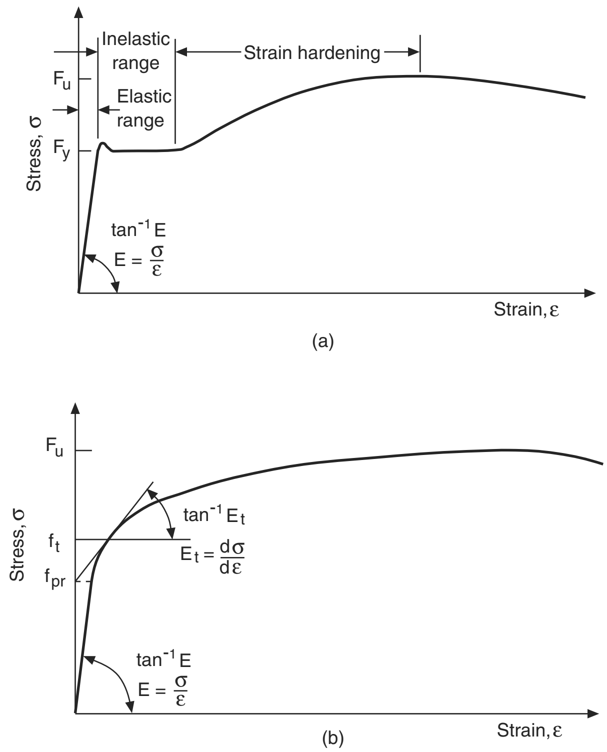
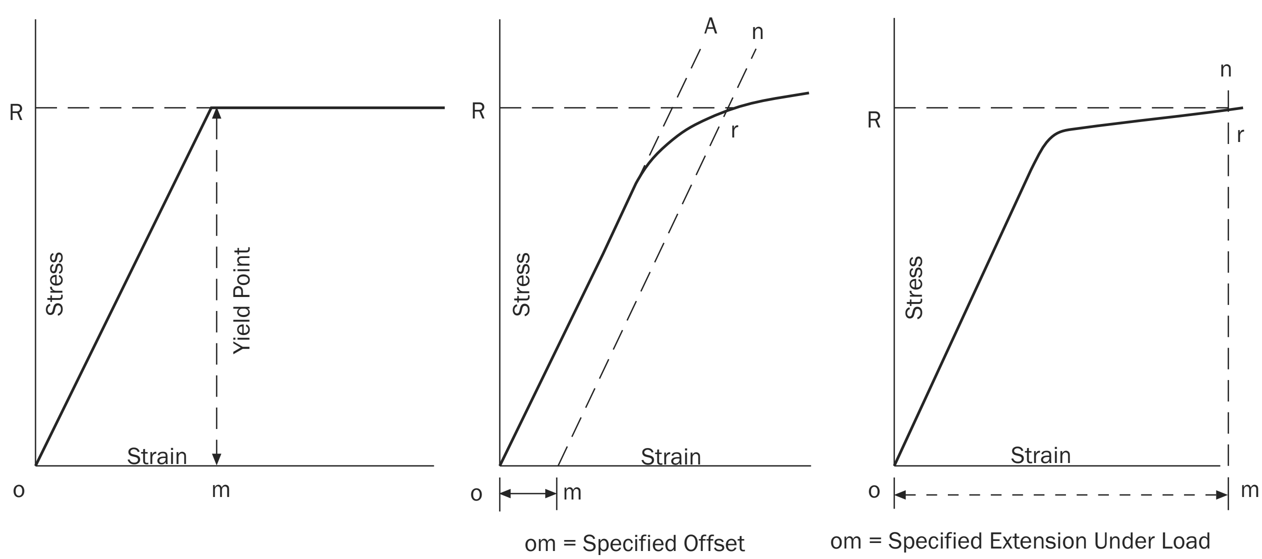


Figure C-A7.1-1 Stress-Strain Curves of Carbon Steel Sheet or Strip

(a) Sharp Yielding, (b) Gradual Yielding



(a) Showing Yield Point Corresponding With Top of Knee.

(b) Showing Yield Point or Yield Strength by the Offset Method (Also Used for Proportional Limit).

(c) Determination of Yield Strength by Extension Under Load Method.

Figure C-A7.1-2 Stress-Strain Diagrams Showing Methods of Yield Point and Yield Strength Determination

For sharp-yielding steels, $E_t = E$ up to the *yield point*, but with gradual-yielding steels, $E_t = E$ only up to the proportional limit, f_{pr}. Once the *stress* exceeds the proportional limit, the tangent modulus E_t becomes progressively smaller than the initial modulus of elasticity.

Various *buckling* provisions of the *Specification* have been written for gradual-yielding steels whose proportional limit is not lower than about 70 percent of the *specified minimum yield stress*.

Determination of proportional limits for information purposes can be done simply by using the offset method shown in Figure C-A7.1-2(b) with the distance "om" equal to 0.0001 length/length (0.01 percent offset) and calling the *stress* R where "mn" intersects the *stress*-strain curve at "r", the proportional limit.

A7.2 Strength Increase From Cold Work of Forming

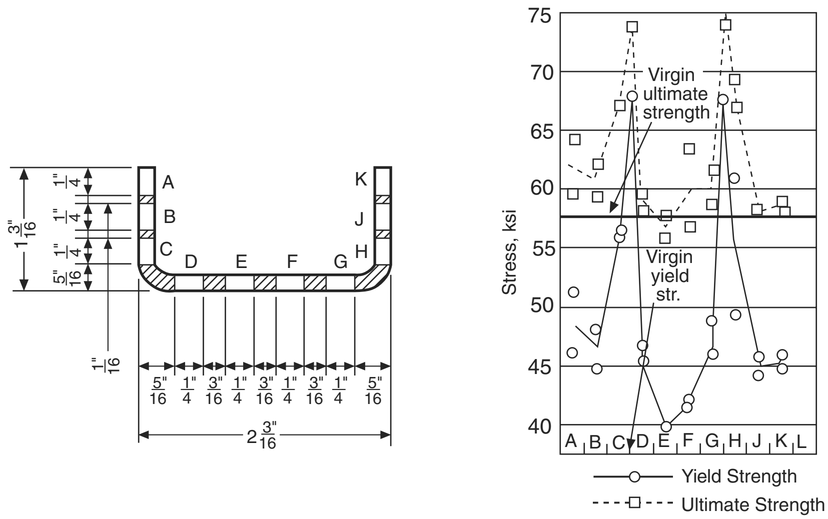
The mechanical properties of the flat steel sheet, strip, plate or bar, such as *yield stress*, *tensile strength*, and elongation may be substantially different from the properties exhibited by the cold-formed steel sections. Figure C-A7.2-1 illustrates the increase of *yield stress* and *tensile strength* from those of the virgin material at the section locations in a cold-formed steel channel section and a joist chord (Karren and Winter, 1967). This difference can be attributed to cold working of the material during the cold-forming process.

The influence of cold work on mechanical properties was investigated by Chajes, Britvec, Winter, Karren, and Uribe at Cornell University in the 1960s (Chajes, Britvec, and Winter, 1963; Karren, 1967; Karren and Winter, 1967; Winter and Uribe, 1968). It was found that the changes of mechanical properties due to cold-stretching are caused mainly by strain-hardening and strain-aging, as illustrated in Figure C-A7.2-2 (Chajes, Britvec, and Winter, 1963). In this figure, Curve A represents the *stress*-strain curve of the virgin material. Curve B is due to unloading in the strain-hardening range, Curve C represents immediate reloading, and Curve D is the *stress*-strain curve of reloading after strain-aging. It is interesting to note that the *yield stresses* of both Curves C and D are higher than the *yield point* of the virgin material and that the ductilities decrease after strain hardening and strain aging.

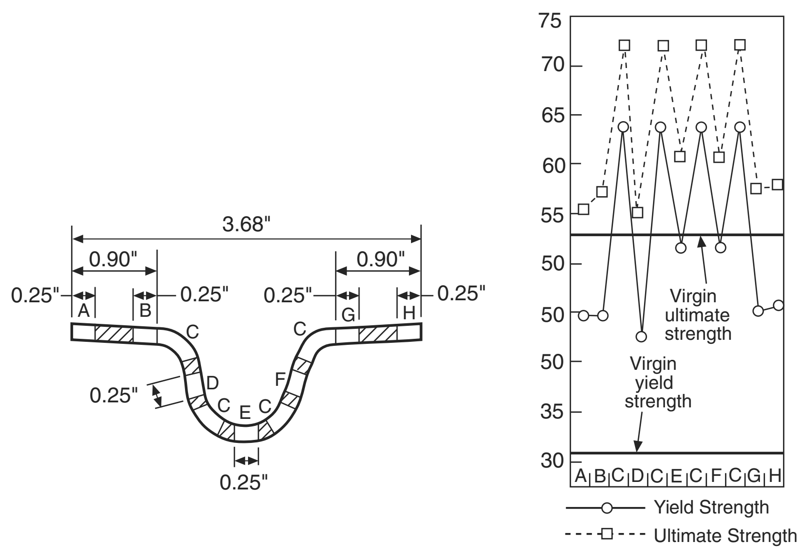
Cornell research also revealed that the effects of cold work on the mechanical properties of corners usually depend on: (1) the type of steel, (2) the type of *stress* (compression or tension), (3) the direction of *stress* with respect to the direction of cold work (transverse or longitudinal), (4) the F_u/F_y ratio, (5) the inside radius-to-*thickness* ratio (R/t), and (6) the amount of cold work. Among the above items, the F_u/F_y and R/t ratios are the most important factors to affect the change in mechanical properties of formed sections. Virgin material with a large F_u/F_y ratio possesses a large potential for strain hardening. Consequently, as the F_u/F_y ratio increases, the effect of cold work on the increase in the *yield stress* of steel increases. Small inside radius-to-*thickness* ratios, R/t, correspond to a large degree of cold work in a corner, and therefore, for a given material, the smaller the R/t ratio, the larger the increase in *yield stress*.

Investigating the influence of cold work, Karren derived the following equations for the ratio of corner *yield stress*-to-virgin *yield stress* (Karren, 1967):

$$\frac{F_{y_c}}{F_{yv}} = \frac{B_c}{(R/t)^m} \tag{C-A7.2-1}$$



(a)



(b)

Figure C-A7.2-1 Effect of Cold Work on Mechanical Properties in Cold-Formed Steel Sections. (a) Channel Section, (b) Joist Chord

where

$$B_c = 3.69\frac{F_{uv}}{F_{yv}} - 0.819\left(\frac{F_{uv}}{F_{yv}}\right)^2 - 1.79$$

and

$$m = 0.192\frac{F_{uv}}{F_{yv}} - 0.068$$

F_{yc} = corner *yield stress*
F_{yv} = virgin *yield stress*
F_{uv} = virgin ultimate *tensile strength*
R = inside bend radius

t = sheet *thickness*

With regard to the full-section properties, the tensile *yield stress* of the full section may be approximated by using a weighted average as follows:

$$F_{ya} = CF_{yc} + (1 - C)F_{yf} \tag{C-A7.2-2}$$

where

F_{ya} = full-section tensile *yield stress*

F_{yc} = average tensile *yield stress* of corners = $B_cF_{yv}/(R/t)^m$

F_{yf} = average tensile *yield stress* of flats

C = ratio of corner area to total cross-sectional area. For flexural members having unequal *flanges*, the one giving a smaller C value is considered to be the controlling *flange*

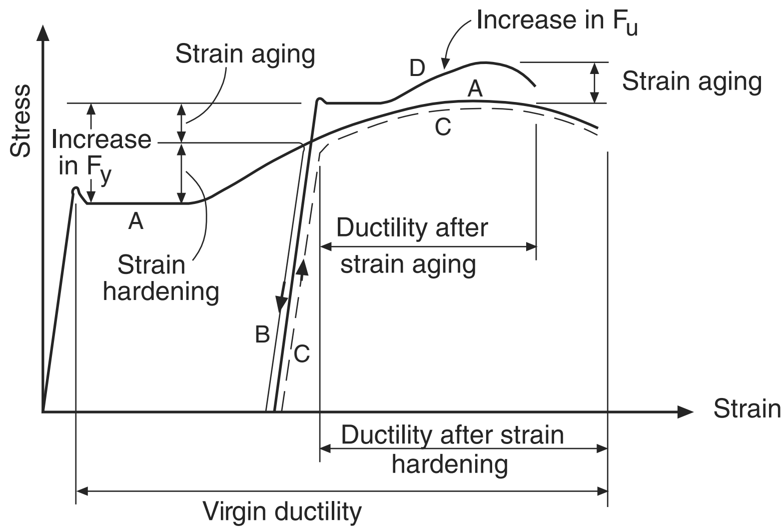


Figure C-A7.2-2 Effect of Strain Hardening and Strain Aging on Stress-Strain Characteristics

Good agreements between the computed and the tested *stress*-strain characteristics for a channel section and a joist chord section were demonstrated by Karren and Winter (Karren and Winter, 1967).

The limitation $F_{ya} \leq F_{uv}$ places an upper bound on the average *yield stress*. The intent of the upper bound is to limit *stresses* in flat elements that may not see significant increases in *yield stress* and *tensile strength* as compared to the *virgin steel properties*.

In the last three decades, additional studies have been made by numerous investigators. These investigations dealt with the cold-formed sections having large R/t ratios and with thick materials. They also considered residual *stress* distribution, simplification of design methods, and other related subjects. For details, see Yu and LaBoube (2010).

In 1962, the AISI *Specification* permitted the utilization of cold work of forming on the basis of full section tests. Since 1968, the AISI *Specification* has allowed the use of the increased average *yield stress* of the section, F_{ya}, to be determined by: (1) full section tensile tests, (2) stub column tests, or (3) computed in accordance with Equation C-A7.2-2. However, such a strength increase is limited only to relatively compact sections designed according to *Specification* Section C2 (tension members), Section C3.1 (bending strength excluding the use of inelastic reserve capacity), Section C4 (concentrically loaded compression members), Section C5 (combined axial *load* and bending), Section D4 (cold-formed steel light-frame construction), and Section D6.1 (*purlins*, *girts* and other members). A design example in the *Cold-Formed Steel Design Manual* (AISI, 2013) demonstrates the use of strength increase from cold work of forming for a channel section to be used as a beam.

In some cases, when evaluating the *effective width* of the *web*, the reduction factor ρ according to *Specification* Section B2.3 may be less than unity but the sum of b_1 and b_2 of Figure B2.3-1 of the *Specification* may be such that the *web* is fully effective, and cold work of forming may be used. This situation only arises when the *web* width to *flange* width ratio, h_o/b_o, is less than or equal to 4.

In the development of the AISI *LRFD Specification,* the following statistical data on material and cross-sectional properties were developed by Rang, Galambos and Yu (1979a and 1979b) for use in the derivation of *resistance factors* ϕ:

$(F_y)_m = 1.10F_y$; $M_m = 1.10$; $V_{fy} = V_M = 0.10$

$(F_{ya})_m = 1.10F_{ya}$; $M_m = 1.10$; $V_{Fya} = V_M = 0.11$

$(F_u)_m = 1.10F_u$; $M_m = 1.10$; $V_{Fu} = V_M = 0.08$

$F_m = 1.00$; $V_F = 0.05$

In the above expressions, m refers to mean value; V represents coefficient of variation; M and F are, respectively, the ratios of the actual-to-the-nominal material property and cross-sectional property; and F_y, F_{ya}, and F_u are, respectively, the *specified minimum yield stress*, the average *yield stress* including the effect of cold forming, and the specified minimum *tensile strength*.

These statistical data are based on the analysis of many samples (Rang et al., 1978), and they are representative properties of materials and cross-sections used in the industrial application of cold-formed steel structures.

A8 Serviceability

Serviceability limit states are conditions under which a structure can no longer perform its intended functions. Safety and strength considerations are generally not affected by *serviceability limit states*. However, serviceability criteria are essential to ensure functional performance and economy of design.

Common conditions which may require serviceability limits are:

a. Excessive deflections or rotations which may affect the appearance or functional use of the structure. Deflections which may cause damage to non-structural elements should be considered.
b. Excessive vibrations which may cause occupant discomfort or equipment malfunctions.
c. Deterioration over time, which may include corrosion or appearance considerations.

When checking serviceability, the designer should consider appropriate *service loads*, the response of the structure, and the reaction of building occupants.

Service loads that may require consideration include static *loads*, snow or rain *loads*, temperature fluctuations, and dynamic *loads* from human activities, wind-induced effects, or the operation of equipment. The *service loads* are actual *loads* that act on the structure at an arbitrary point in time. Appropriate *service loads* for checking *serviceability limit states* may only be a fraction of the *nominal loads*.

The response of the structure to *service loads* can normally be analyzed assuming linear elastic behavior. However, members that accumulate residual deformations under *service loads* may require consideration of this long-term behavior.

Serviceability limits depend on the function of the structure and on the perceptions of the observer. In contrast to the strength limit states, it is not possible to specify general serviceability limits that are applicable to all structures. The *Specification* does not contain explicit requirements; however, guidance is generally provided by the *applicable building code*. In the absence of specific criteria, guidelines may be found in Fisher and West (1990), Ellingwood (1989), Murray (1991), AISC (2010) and ATC (1999).

A9 Referenced Documents

Other specifications and standards to which the *Specification* makes references have been listed and updated in *Specification* Section A9 to provide the effective dates of these standards at the time of approval of this *Specification*. References for country-specific provisions are provided in the corresponding appendices.

Additional references which the designer may use for related information are listed at the end of the *Commentary.*

B. ELEMENTS

In cold-formed steel construction, individual elements of steel structural members are thin and the width-to-*thickness* ratios are large as compared with hot-rolled steel shapes. These thin elements may buckle locally at a *stress* level lower than the *yield stress* of steel when they are subjected to compression in flexural bending, axial compression, shear, or *bearing*. Figure C-B-1 illustrates some *local buckling* patterns of certain beams and columns (Yu and LaBoube 2010).

Because *local buckling* of individual elements of cold-formed steel sections is a major design criterion, the design of such members should provide sufficient safety against the failure by local *instability* with due consideration given to the post-*buckling* strength of *structural components*. Chapter B of the *Specification* contains the design requirements for width-to-thickness ratios and the design equations for determining the *effective widths* of stiffened compression elements, *unstiffened compression elements*, elements with edge stiffeners or intermediate stiffeners, and beam *webs*. The design provisions are provided for the use of stiffeners in *Specification* Section C3.7 for flexural members.

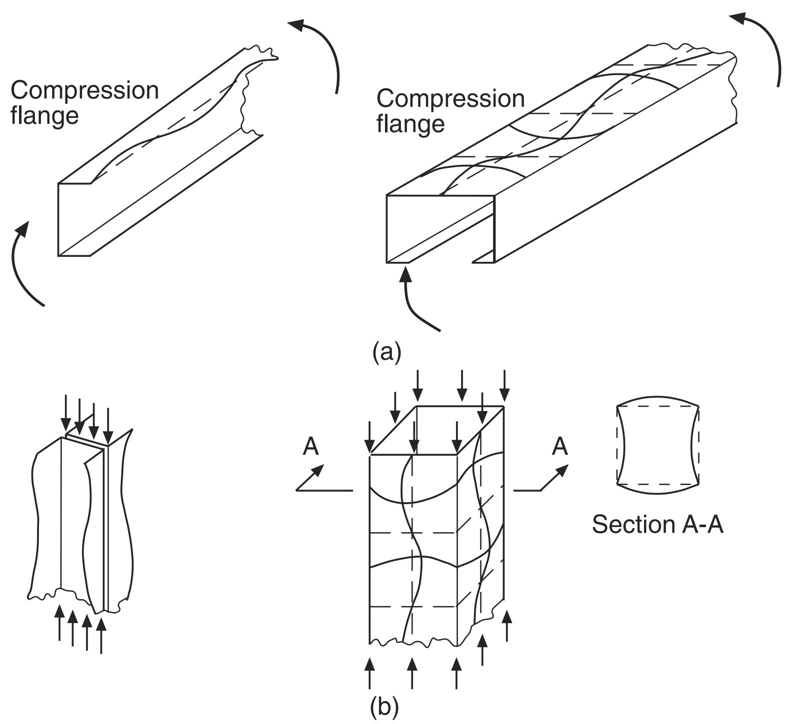


Figure C-B-1 Local Buckling of Compression Elements
(a) Beams, (b) Columns

B1 Dimensional Limits and Considerations

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

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

Section B1.1 (a) of the *Specification* contains limitations on permissible *flat-width-to-thickness ratios* of compression elements. To some extent, these limitations are arbitrary. They do, however, reflect a long-term experience and are intended to delimit practical ranges (Winter, 1970).

The limitation to a maximum w/t of 60 for the compression *flanges* having one longitudinal edge connected to a *web* and the other edge stiffened by a simple lip is based on the fact that if the w/t ratio of such a *flange* exceeds 60, a simple lip with a relatively large depth would be required to stiffen the *flange* (Winter, 1970). The local *instability* of the lip would necessitate a reduction of the bending capacity to prevent premature *buckling* of the stiffening lip. This is the reason why the w/t ratio is limited to 60 for stiffened compression elements having one longitudinal edge connected to a *web* or *flange* element and the other stiffened by a simple lip.

The limitation to w/t = 90 for compression *flanges* with any other kind of stiffeners indicates that thinner *flanges* with large w/t ratios are quite flexible and liable to be damaged in transport, handling and erection. The same is true for the limitation to w/t = 500 for stiffened compression elements with both longitudinal edges connected to other stiffened elements and for the limitation to w/t = 60 for *unstiffened compression elements*. The provision specifically states that wider *flanges* are not unsafe, but that when the w/t ratio of unstiffened *flanges* exceeds 30 and the w/t ratio of stiffened *flanges* exceeds 250, it is likely to develop noticeable deformation at the full *available strength* [*factored resistance*], without affecting the ability of the member to develop strength. In both cases, the maximum w/t is set at twice that ratio at which first noticeable deformations are likely to appear, based on observations of such members under tests. These upper limits will generally keep such deformations to reasonable limits. In such cases where the limits are exceeded, tests in accordance with *Specification* Chapter F are required.

(b) Flange Curling

In beams which have unusually wide and thin, but stable *flanges* (i.e., primarily tension *flanges* with large w/t ratios), there is a tendency for these *flanges* to curl under bending. That is, the portions of these *flanges* most remote from the *web* (edges of I-beams, center portions of *flanges* of box or hat beams) tend to deflect toward the neutral axis. An approximate, analytical treatment of this problem was given by Winter (1948b). Equation B1.1-1 of the *Specification* permits one to compute the maximum permissible *flange* width, w_f, for a given amount of *flange* curling, c_f.

It should be noted that Section B1.1(b) does not stipulate the amount of curling which can be regarded as tolerable, but an amount of curling in the order of 5 percent of the depth of the section is not excessive under usual conditions. In general, *flange* curling is not a critical factor to govern the *flange* width. However, when the appearance of the section is important, the out-of-plane distortion should be closely controlled in practice. Example of the AISI *Cold-Formed Steel Design Manual* (AISI, 2013) illustrates the design consideration for *flange* curling.

(c) Shear Lag Effects - Short Spans Supporting Concentrated Loads

For the beams of usual shapes, the normal *stresses* are induced in the *flanges* through shear *stresses* transferred from the *web* to the *flange*. These shear *stresses* produce shear strains in the *flange* which, for ordinary dimensions, have negligible effects. However, if *flanges* are unusually wide (relative to their length), these shear strains have the effect that the normal bending *stresses* in the *flanges* decrease with increasing distance from the *web*. This phenomenon is known as shear lag. It results in a non-uniform *stress* distribution across the width of the flange, similar to that in stiffened compression elements (see Section B2 of the *Commentary*), though for entirely different reasons. The simplest way of accounting for this *stress* variation in design is to replace the non-uniformly stressed flange of actual width, w_f, by one of reduced, *effective width* subject to uniform *stress* (Winter, 1970).

Theoretical analyses by various investigators have arrived at results which differ numerically (Roark, 1965). The provisions of Section B1.1(c) are based on the analysis and supporting experimental evidence obtained by detailed *stress* measurements on eleven beams (Winter, 1940). In fact, the values of *effective widths* in *Specification* Table B1.1(c) are taken directly from Curve A of Figure 4 of Winter (1940).

It will be noted that according to *Specification* Section B1.1(c), the use of a reduced width for stable, wide *flanges* is required only for concentrated *load* as shown in Figure C-B1.1-1. For uniform *load*, it is seen from Curve B of the figure that the width reduction due to shear lag for any unrealistically large span-width ratios is so small as to be practically negligible.

The phenomenon of shear lag is of considerable consequence in naval architecture and aircraft design. However, in cold-formed steel construction it is infrequent that beams are so wide as to require significant reductions according to *Specification* Section B1.1(c). For design purpose, see Example of the AISI *Design Manual* (AISI, 2013).

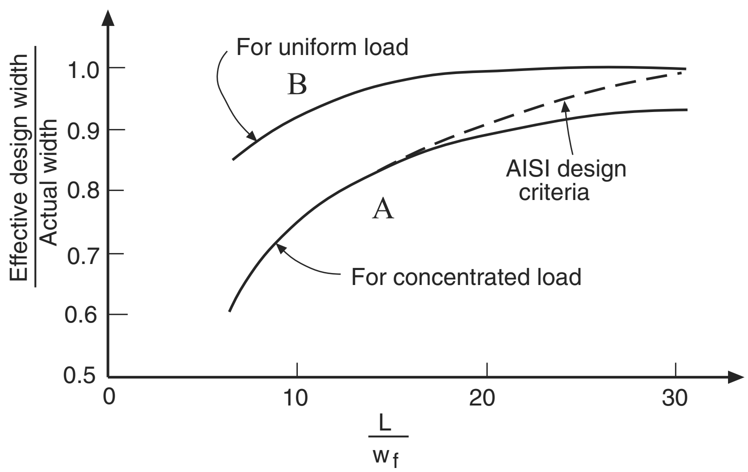


Figure C-B1.1-1 Analytical Curves for Determining Effective Width of Flange of Short Span Beams

B1.2 Maximum Web Depth-to-Thickness Ratios

Prior to 1980, the maximum *web* depth-to-*thickness* ratio, h/t, was limited to: (a) 150 for cold-formed steel members with unreinforced *webs*, and (b) 200 for members which are provided with the adequate means of transmitting concentrated *loads* and/or reactions into the *web*. Based on the studies conducted at the University of Missouri-Rolla in the 1970s (LaBoube and Yu, 1978a, 1978b, and 1982b; Hetrakul and Yu, 1978 and 1980; Nguyen and Yu, 1978a and 1978b), the maximum h/t ratios were increased to: (a) 200 for unreinforced *webs*, (b) 260 for using bearing stiffeners, and (c) 300 for using bearing and intermediate stiffeners in the 1980 edition of the AISI *Specification*. These h/t limitations are the same as that used in the AISC Specification (AISC, 1989) for plate girders and are retained in the current edition of the *Specification*. Because the definition for "h" was changed in the 1986 edition of the AISI

Specification from the "clear distance between *flanges*" to the "depth of flat portion," measured along the plane of *web*, the prescribed maximum h/t ratio may appear to be more liberal. An unpublished study by LaBoube concluded that the present definition for h had negligible influence on the *web* strength.

B1.3 Corner Radius-to-Thickness Ratios

The *effective width* provisions of Chapter B provide no reductions for corners. For inside bend radius-to-*thickness* ratios (R/t) in excess of 10, this is shown to be unconservative based on the studies of Sarawit (2003), and Zeinoddini and Schafer (2010). For members with large radius-to-*thickness*, the *Direct Strength Method* of Appendix 1 may be employed. Alternatively, the *Specification* specifically allows for *rational engineering analysis*. Using an equivalent centerline model to determine the *effective width* of the flats or appropriately reducing the plate *buckling* coefficient are examples of such rational analyses.

In Zeinoddini and Schafer (2010), the following method is shown to provide a rational reduction for $10 < R/t \leq 20$. A reduced plate *buckling* coefficient, k_R, is determined by applying reduction factors based on the R/t value at each edge of the element. For unstiffened elements, only one reduction factor is applied. The plate *buckling* coefficient, k_R, which replaces k in Chapter B, is determined as follows:

$$k_R = k\,R_{R1}\,R_{R2} \qquad \text{(C-B1.3-1)}$$

where

k = Plate *buckling* coefficient determined in accordance with *Specification* Sections B2, B3, B4 and B5, as applicable

$$R_{R1} = 1.08 - (R_1/t)/50 \leq 1.0 \qquad \text{(C-B1.3-2)}$$

$$R_{R2} = 1.08 - (R_2/t)/50 \leq 1.0 \qquad \text{(C-B1.3-3)}$$

where

R_1, R_2 = inside bend radius. See Figure C-B1.3-1

t = *thickness* of element. See Figure C-B1.3-1

Engineers are reminded that when *rational engineering analysis* methods are employed, such as presented here for r/t>10, the *safety* and *resistance factors* of Section A1.2 apply.

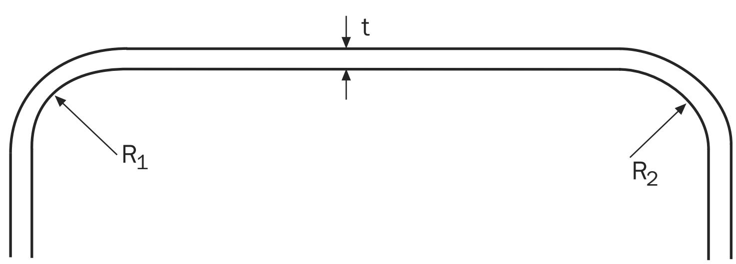


Figure C-B1.3-1 Corner Radius

B2 Effective Widths of Stiffened Elements

It is well known that the structural behavior and the *load*-carrying capacity of the stiffened compression element such as the compression *flange* of the hat section depend on the w/t ratio and the supporting condition along both longitudinal edges. If the w/t ratio is small, the *stress* in the compression *flange* can reach the *yield stress* of steel and the strength of the compression element is governed by yielding. For the compression *flange* with large w/t ratios, *local buckling* (Figure C-B2-1) will occur at the following elastic critical *buckling stress*:

$$f_{cr} = \frac{k\pi^2 E}{12(1-\mu^2)(w/t)^2} \tag{C-B2-1}$$

where

k = plate *buckling* coefficient (Table C-B2-1)

= 4 for stiffened compression elements supported by a *web* on each longitudinal edge

E = modulus of elasticity of steel

μ = Poisson's ratio = 0.3 for steel in the elastic range

w = *flat width* of the compression element

t = *thickness* of the compression element

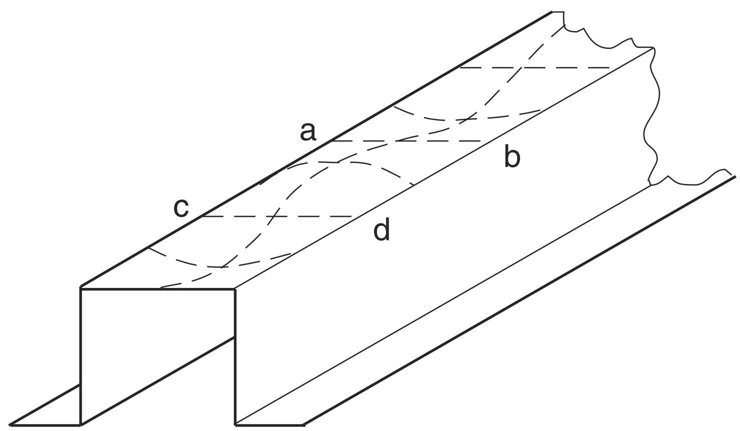


Figure C-B2-1 Local Buckling of Stiffened Compression Flange of Hat-Shaped Beam

When the elastic critical *buckling stress* computed according to Equation C-B2-1 exceeds the proportional limit of the steel, the compression element will buckle in the inelastic range (Yu and LaBoube, 2010).

Unlike one-dimensional structural members such as columns, stiffened compression elements will not collapse when the *buckling stress* is reached. An additional *load* can be carried by the element after *buckling* by means of a redistribution of *stress*. This phenomenon is known as post-*buckling* strength of the compression elements and is most pronounced for stiffened compression elements with large w/t ratios. The mechanism of the post-*buckling* action of compression elements was discussed by Winter in previous editions of the AISI *Commentary* (Winter, 1970).

Imagine for simplicity a square plate uniformly compressed in one direction, with the unloaded edges simply supported. Since it is difficult to visualize the performance of such two-dimensional elements, the plate will be replaced by a model which is shown in Figure C-B2-2. It consists of a grid of longitudinal and transverse bars in which the material of the actual plate is thought to be concentrated. Since the plate is uniformly compressed, each of the longitudinal struts represents a column loaded by P/5, if P is the total *load* on the plate. As the *load* is gradually increased, the compression *stress* in each of these struts will reach the critical column *buckling* value and all five struts will tend to buckle simultaneously. If these struts were simple columns, unsupported except at the ends, they would simultaneously collapse through unrestrained increasing lateral deflection. It is evident that this cannot occur in the grid model

Table C-B2-1
Values of Plate Buckling Coefficients

Case	Boundary Condition	Type of Stress	Value of k for Long Plate
(a)	s.s. s.s. s.s. s.s.	Compression	4.0
(b)	Fixed s.s. s.s. Fixed	Compression	6.97
(c)	s.s. s.s. s.s. Free	Compression	0.425
(d)	Fixed s.s. s.s. Free	Compression	1.277
(e)	Fixed s.s. s.s. s.s.	Compression	5.42
(f)	s.s. s.s. s.s. s.s.	Shear	5.34
(g)	Fixed Fixed Fixed Fixed	Shear	8.98
(h)	s.s. s.s. s.s. s.s.	Bending	23.9
(i)	Fixed Fixed Fixed Fixed	Bending	41.8

of the plate. Indeed, as soon as the longitudinal struts start deflecting at their *buckling stress*, the transverse bars, which are connected to them, must stretch like ties in order to accommodate the imposed deflection. Like any structural material, they resist stretch and, thereby, have a restraining effect on the deflections of the longitudinal struts.

The tension forces in the horizontal bars of the grid model correspond to the so-called membrane *stresses* in a real plate. These *stresses*, just as in the grid model, come into play as soon as the compression *stresses* begin to cause *buckling* waves. They consist mostly of transverse tension, but also of some shear *stresses*, and they counteract increasing wave deflections, i.e. they tend to stabilize the plate against further *buckling* under the applied increasing longitudinal compression. Hence, the resulting behavior of the model is as follows: (a) there is no collapse by unrestrained deflections, as in unsupported columns, and (b) the various struts will deflect unequal amounts–those nearest the supported edges being held almost straight by the ties, and those nearest the center being able to deflect most.

In consequence of (a), the model will not collapse and fail when its *buckling stress* (Equation C-B2-1) is reached; in contrast to columns it will merely develop slight deflections but will continue to carry increasing *load*. In consequence of (b), the struts (strips of the plate) closest to the center, which deflect most, "get away from the *load*," and hardly participate in carrying any further *load* increases. These center strips may, in fact, even transfer part of their pre-*buckling* load to their neighbors. The struts (or strips) closest to the edges, held straight by the ties, continue to resist increasing *load* with hardly any increasing deflection. For the plate, this means that the hitherto uniformly distributed compression *stress* redistributes itself in a manner shown in Figure C-B2-3, the *stresses* being largest at the edges and smallest in the center. With further increase in *load* this non-uniformity increases further, as also shown in Figure C-B2-3. The plate fails, i.e., refuses to carry any further *load* increases, only when the most highly stressed strips near the supported edges begin to yield, i.e., when the compression *stress* f_{max} reaches the *yield stress* F_y.

This post-*buckling* strength of plates was discovered experimentally in 1928, and an approximate theory of it was first given by Th. v. Karman in 1932 (Bleich, 1952). It has been used in aircraft design ever since. A graphic illustration of the phenomenon of *post-buckling* strength can be found in the series of photographs in Figure 7 of Winter (1959b).

The model of Figure C-B2-2 is representative of the behavior of a compression element supported along both longitudinal edges, as the *flange* in Figure C-B2-1. In fact, such elements buckle into approximately square waves.

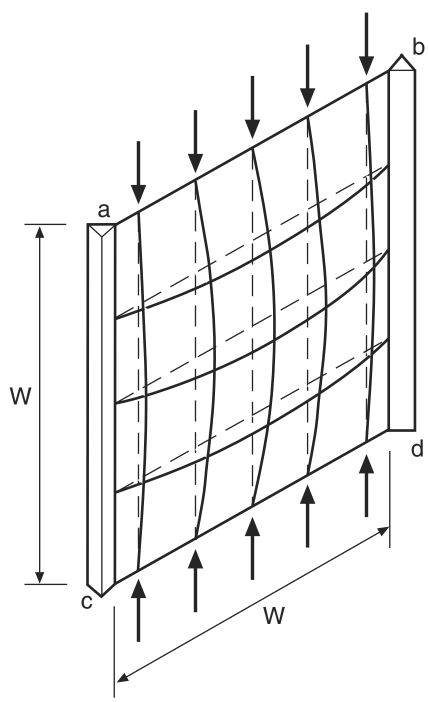


Figure C-B2-2 Post-Buckling Strength [Resistance] Model

In order to utilize the post-*buckling* strength of the stiffened compression element for design purposes, the AISI *Specification* has used the *effective design width* approach to determine the sectional properties since 1946. In Section B2 of the present *Specification*, design equations for computing the *effective widths* are provided for the following four cases: (1) uniformly compressed stiffened elements, (2) uniformly compressed stiffened elements with circular or non-circular holes, (3) *webs* and other stiffened elements with *stress* gradient, (4) unstiffened elements with uniform or gradient *stress*, and (5) C-section *webs* with holes under *stress* gradient. The background information on various design requirements is discussed in subsequent sections.

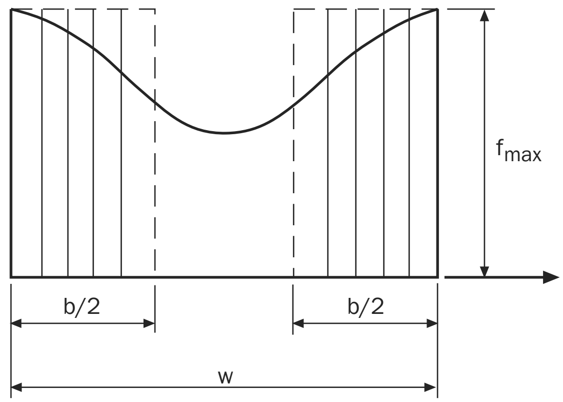


Figure C-B2-3 Stress Distribution in Stiffened Compression Elements

B2.1 Uniformly Compressed Stiffened Elements

(a) Effective Width for Strength [Resistance] Determination

In the *effective design width* approach, instead of considering the nonuniform distribution of *stress* over the entire width of the plate *w*, it is assumed that the total *load* is carried by a fictitious *effective width b*, subject to a uniformly distributed *stress* equal to the edge *stress* f_{max}, as shown in Figure C-B2-3. The width b is selected so that the area under the curve of the actual nonuniform *stress* distribution is equal to the sum of the two parts of the equivalent rectangular shaded area with a total width b and an intensity of *stress* equal to the edge *stress* f_{max}.

Based on the concept of *effective width* introduced by von Karman et al. (von Karman, Sechler and Donnell, 1932) and the extensive investigation on light-gage, cold-formed steel sections at Cornell University, the following equation was developed by Winter in 1946 for determining the *effective width* b for stiffened compression elements simply supported along both longitudinal edges:

$$b = 1.9t\sqrt{\frac{E}{f_{max}}}\left[1 - 0.475\left(\frac{t}{w}\right)\sqrt{\frac{E}{f_{max}}}\right] \tag{C-B2.1-1}$$

The above equation can be written in terms of the ratio of F_{cr}/f_{max} as follows:

$$\frac{b}{w} = \sqrt{\frac{F_{cr}}{f_{max}}}\left(1 - 0.25\sqrt{\frac{F_{cr}}{f_{max}}}\right) \tag{C-B2.1-2}$$

where F_{cr} is the critical elastic *buckling stress* of a plate, and is expressed in Equation C-B2-1.

Thus, the *effective width* expression (e.g., Equation C-B2.1-1) provides a prediction of the *nominal strength* [*resistance*] based only on the critical elastic *buckling stress* and the applied *stress* of the plate. During the period from 1946 to 1968, the AISI design provision for the determination of the *effective design width* was based on Equation C-B2.1-1. Accumulated experience has demonstrated that a more realistic equation as shown below may be used for the determination of the *effective width* b (Winter, 1970):

$$b = 1.9t\sqrt{\frac{E}{f_{max}}}\left[1 - 0.415\left(\frac{t}{w}\right)\sqrt{\frac{E}{f_{max}}}\right] \qquad \text{(C-B2.1-3)}$$

The correlation between the test data on stiffened compression elements and Equation C-B2.1-3 is illustrated by Yu and LaBoube (2010).

It should be noted that Equation C-B2.1-3 may also be rewritten in terms of the F_{cr}/f_{max} ratio as follows:

$$\frac{b}{w} = \sqrt{\frac{F_{cr}}{f_{max}}}\left(1 - 0.22\sqrt{\frac{F_{cr}}{f_{max}}}\right) \qquad \text{(C-B2.1-4)}$$

Therefore, the *effective width*, b, can be determined as

$$b = \rho w \qquad \text{(C-B2.1-5)}$$

where ρ = reduction factor

$$= (1 - 0.22/\sqrt{f_{max}/F_{cr}})/\sqrt{f_{max}/F_{cr}} = (1 - 0.22/\lambda)/\lambda \le 1 \qquad \text{(C-B2.1-6)}$$

In Equation C-B2.1-6, λ is a slenderness factor determined below.

$$\lambda = \sqrt{f_{max}/F_{cr}} \qquad \text{(C-B2.1-7)}$$

Figure C-B2.1-1 shows the relationship between ρ and λ. It can be seen that when $\lambda \le 0.673$, $\rho = 1.0$.

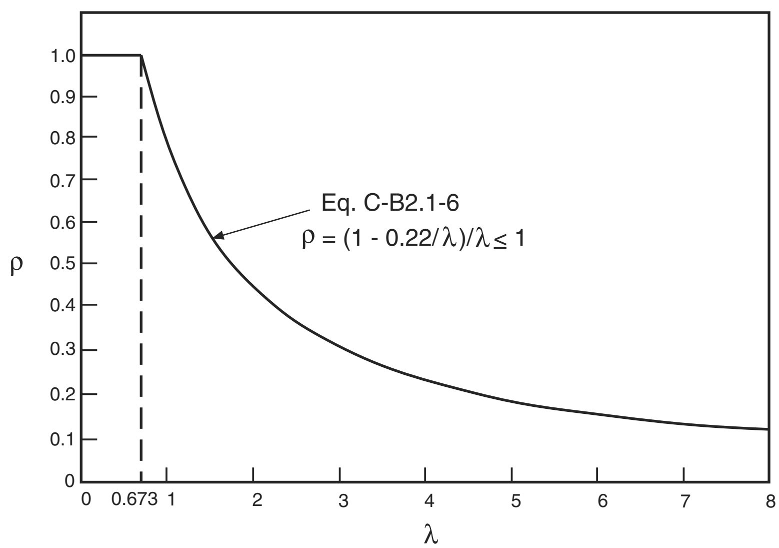


Figure C-B2.1-1 Reduction Factor, ρ, vs. Slenderness Factor, λ

Based on Equations C-B2.1-5 through C-B2.1-7 and the unified approach proposed by Peköz (1986b and 1986c), the 1986 edition of the AISI *Specification* adopted the non-dimensional format in Section B2.1 for determining the *effective design width*, b, for uniformly compressed stiffened elements. The same design equations were used in the 1996 edition of the AISI *Specification* and were retained in this edition of the *North American Specification*. For design examples, see Part I of the AISI *Design Manual* (AISI, 2013).

(b) *Effective Width for Serviceability Determination*

The *effective design width* equations discussed above for strength determination can also be used to obtain a conservative *effective width*, b_d, for serviceability determination. It is included in Section B2.1(b) of the *Specification* as Procedure I.

For stiffened compression elements supported by a *web* on each longitudinal edge, a study conducted by Weng and Peköz (1986) indicated that Equations B2.1-8 through B2.1-10 of the *Specification* can yield a *more accurate estimate* of the *effective width*, b_d, for serviceability. These equations are given in Procedure II for additional design information. The design engineer has the option of using one of the two procedures for determining the *effective width* to be used for serviceability determination.

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

In *cold-formed steel structural members*, holes are sometimes provided in *webs* and/or *flanges* of beams and columns for duct work, piping, and other construction purposes. The presence of such holes may result in a reduction of the strength of individual component elements and the overall strength and stiffness of the members depending on the size, shape, and arrangement of holes, the geometric configuration of the cross-section, and the mechanical properties of the material.

The exact analysis and the design of steel sections having perforations are complex, particularly when the shapes and the arrangement of holes are unusual. The limited design provisions included in Section B2.2 of the *Specification* for uniformly compressed stiffened elements with circular holes are based on a study conducted by Ortiz-Colberg and Peköz at Cornell University (Ortiz-Colberg and Peköz, 1981). For additional information on the structural behavior of perforated elements, see Yu and Davis (1973a) and Yu and LaBoube (2010).

In 2004, the *Specification* Equation B2.2-2 was revised to provide continuity at $\lambda = 0.673$.

In 2007, the provisions for non-circular holes were moved from *Specification* Section D4 to Section B2.2. Within the limitations stated for the size and spacing of perforations and section depth, the provisions were deemed appropriate for members with uniformly compressed stiffened elements, not just wall studs. The validity of this approach for C-section wall studs was verified in a Cornell University project on wall studs reported by Miller and Peköz (1989 and 1994). The limitations included in *Specification* Section B2.2 for the size and spacing of perforations and the depth of studs are based on the parameters used in the test program. Although Figure B2.2-1 in the *Specification* shows a hole centered within the *flat width*, w, holes not centered within w are allowed. For such a case, the unstiffened strip, c, and resulting *effective width*, b, must be calculated separately for the strips on each side of the hole. For sections with perforations which do not meet these limits, the *effective area*, A_e, can be determined by stub column tests.

The geometric limitations (w/t, etc.) and hole size for the circular and non-circular hole provisions in *Specification* Section B2.2 are not consistent with one another. This anomaly in the limitations is due to the differing scopes of the test programs that serve as the basis for these *effective width* equations. The provisions for non-circular holes generally give a more conservative prediction of the *effective width* than the provisions for circular holes, as long as $d_h/w < 0.4$. Provisions for designing perforated members using the *Direct Strength Method* (DSM) can be found in *Specification* Appendix 1.

B2.3 Webs and Other Stiffened Elements Under Stress Gradient

When a beam is subjected to bending moment, the compression portion of the *web* may buckle due to the compressive *stress* caused by bending. The theoretical critical *buckling stress* for a flat rectangular plate under pure bending can be determined by Equation C-B2-1, except that the depth-to-*thickness* ratio, h/t, is substituted for the width-to-*thickness* ratio, w/t, and the plate *buckling* coefficient, k, is equal to 23.9 for simple supports as listed in Table C-B2-1.

Prior to 1986, the design of cold-formed steel beam *webs* was based on the full *web* depth with the allowable bending *stress* specified in the AISI *Specification*. In order to unify the design methods for *web* elements and compression *flanges*, the effective design depth approach was adopted in the 1986 edition of the AISI *Specification* on the basis of the studies made by Peköz (1986b), Cohen and Peköz (1987). This is a different approach as compared with the past practice of using a full area of the *web* element in conjunction with a reduced *stress* to account for *local buckling* and post-*buckling* strength (LaBoube and Yu, 1982b; Yu, 1985).

Prior to 2001, the b_1 and b_2 expressions used in the AISI *Specification* for the *effective width* of *webs* (Equations B2.3-3 through B2.3-5) implicitly assumed that the *flange* provided beneficial restraint to the *web*. Collected data (Cohen and Peköz (1987), Elhouar and Murray (1985), Ellifritt et al (1997), Hancock et al (1996), LaBoube and Yu (1978), Moreyra and Peköz (1993), Rogers and Schuster (1995), Schardt and Schrade (1982), Schuster (1992), Shan et al (1994), and Willis and Wallace (1990) as summarized in Schafer and Peköz (1999)) on flexural tests of C's and Z's indicate that *Specification* Equations B2.3-3 through B2.3-5 can be unconservative if the overall *web* width (h_o) to overall *flange* width (b_o) ratio exceeds 4. Consequently, in 2001, in the absence of a comprehensive method for handling local *web* and *flange* interaction, the *North American Specification* adopted a two-part approach for the *effective width* of *webs*: an additional set of alternative expressions (Equations B2.3-6 and B2.3-7), originally developed by Cohen and Peköz (1987), were adopted for $h_o/b_o > 4$; while the expressions adopted in the 1986 edition of the AISI *Specification* (Equations B2.3-3 through B2.3-5) remain for $h_o/b_o \le 4$. For flexural members with *local buckling* in the *web*, the effect of these changes is that the strengths will be somewhat lower when $h_o/b_o > 4$ compared with the 1996 AISI *Specification* (AISI, 1996). When compared with the CSA S136 (CSA, 1994) *Standard*, there are only minor changes for members with $h_o/b_o > 4$, but an increase in strength will be experienced when $h_o/b_o \le 4$.

It should be noted that in the *North American Specification*, the *stress* ratio ψ is defined as an absolute value. As a result, some signs for ψ have been changed in *Specification* Equations B2.3-2, B2.3-3, B2.3-6 and B2.3-7 as compared with the 1996 edition of the AISI *Specification* (AISI, 1996).

B2.4 C-Section Webs With Holes Under Stress Gradient

Studies of the behavior of *web* elements with holes conducted at the University of Missouri-Rolla (UMR) serve as the basis for the design recommendations for bending alone, shear, *web crippling*, combinations of bending and shear, and bending and *web crippling* (Shan et al., 1994; Langan et al., 1994; Uphoff, 1996; Deshmukh, 1996). The *Specification* considers a hole to be any flat punched opening in the *web* without any edge-stiffened openings.

The UMR design recommendations for a perforated *web* with *stress* gradient are based on the tests of full-scale C-section beams having h/t ratios as large as 200 and d_h/h ratios as large as 0.74. The test program considered only stud and joist industry standard *web* holes. These holes were rectangular with fillet corners, punched during the rolling process. For non-circular holes, the corner radii recommendation was adopted to avoid the potential of high *stress* concentration at the corners of a hole. *Webs* with circular holes and a *stress* gradient were not tested; however, the provisions are conservatively extended to cover this case. Other shaped holes must be evaluated by the virtual hole method described below, by test, or by other provisions of the *Specification*. The *Specification* is not intended to cover cross-sections having repetitive 1/2 in. diameter holes.

Based on the study by Shan et al. (1994), it was determined that the *nominal bending strength* [*resistance*] of a C-section with a *web* hole is unaffected when $d_h/h < 0.38$. For situations where the $d_h/h \geq 0.38$, the effective depth of the *web* can be determined by treating the flat portion of the remaining *web* that is in compression as an unstiffened compression element.

Although these provisions are based on tests of singly-symmetric C-sections having the *web* hole centered at mid-depth of the section, the provisions may be conservatively applied to sections for which the full unreduced compression region of the *web* is less than the tension region. However, for cross-sections having a compression region greater than the tension region, the *web* strength must be determined by test in accordance with Section F1.

The provisions for circular and non-circular holes also apply to any hole pattern that fits within an equivalent virtual hole. For example, Figure C-B2.4-1 illustrates the L_h and d_h that may be used for a multiple-hole pattern that fits within a non-circular virtual hole. Figure C-B2.4-2 illustrates the d_h that may be used for a rectangular hole that exceeds the 2.5 in. (64 mm) by 4.5 in. (114 mm) limit but still fits within an allowed circular virtual hole. For each case, the design provisions apply to the geometry of the virtual hole, not the actual hole or holes.

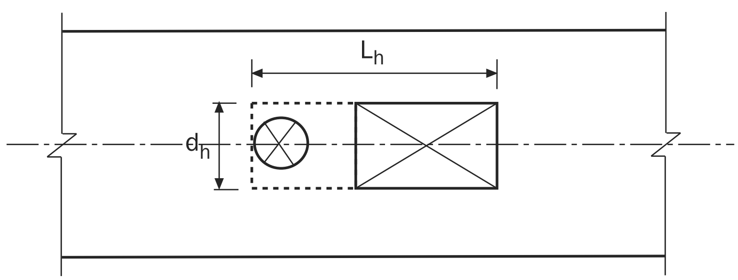


Figure C-B2.4-1 Virtual Hole Method for Multiple Openings

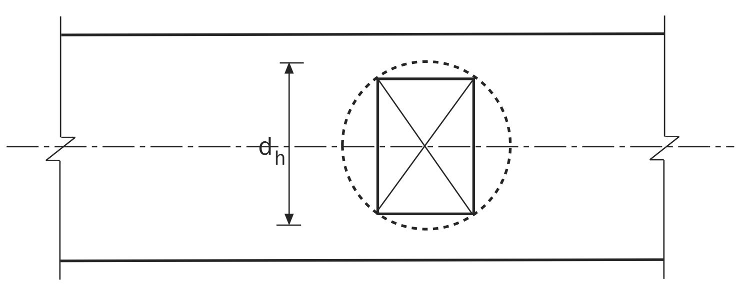


Figure C-B2.4-2 Virtual Hole Method for Opening Exceeding Limit

The effects of holes on shear strength and *web crippling* strength of C-section *webs* are discussed in Sections C3.2.2 and C3.4.2 of the *Commentary*, respectively.

B2.5 Uniformly Compressed Elements Restrained by Intermittent Connections

Section D1.3 limits the spacing of *connections* in compression elements so that the strength of the section is fully developed before *buckling* occurs between *connections*. In practice this limitation is often exceeded. Luttrell and Balaji (1992) and Snow and Easterling (2008) developed a method to determine the *effective width* of compression elements when greater connection spacing exists. The design provisions in *Specification* Section B2.5 were based on the research work by Snow and Easterling (2008) with 82 standard roof deck tests. All test specimens had multiple flutes and the depth range was between 1-½ in. (38.1 mm) and 7-½ in. (191 mm). As shown in Figures C-B2.5-1 and C-B2.5-2, all test compression plates had edge stiffeners.

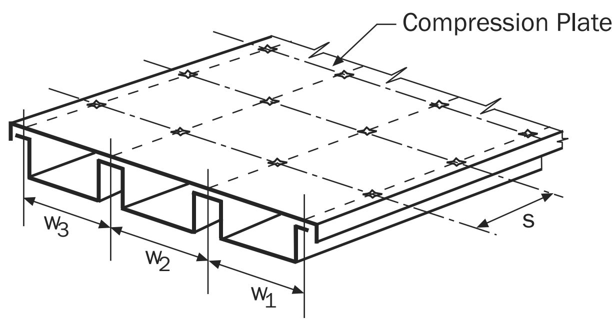


Figure C-B2.5-1 Built-Up Deck

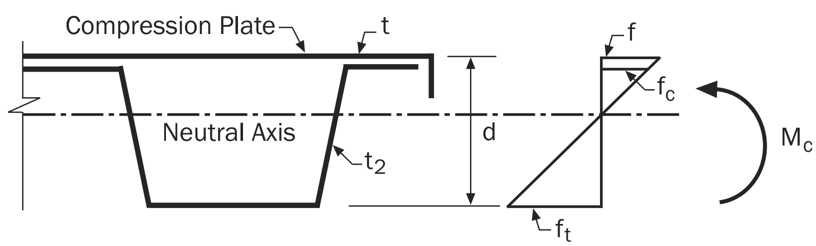


Figure C-B2.5-2 Built-Up Deck in Bending

The full *stress* potential of the "built-up" section is determined by recognizing the post-*buckling* strength of the compression plate after local waves form between *connections*. The method models an equivalent composite transformed section and maintains the classical assumption of linear strain distribution. The critical compression *stress*, F_c, is based on "column-like" *buckling* in the plate. The *connections* provide fixed-end column restraint and K = 0.5. Before such *buckling* occurs ($f < F_c$), the *effective width* of the section is calculated using Section B2.1 with the *connection* lines treated as *webs*. When the critical *stress* is reached and exceeded ($f \geq F_c$), the compression plate might not resist the same *stress*, f_c, as the adjacent element that theoretically has slightly less strain. An equivalent width is determined to provide the approximate true force contribution of the buckled plate in resisting bending.

This equivalent width is assumed to have an artificially high *stress*, f, which is compatible with both a constant "E" and linear strain distribution across the "built-up" section; however, the actual *stress* might be between F_c and f. ρ_t provides the *effective width* at F_c, and ρ_m allows further *effective width* reduction to provide the equivalent force. The equivalent transformed section properties cannot be greater than the section calculated using *Specification* Section B2.1 at the *stress* level, f. The moment of inertia for deflection is determined by substituting the maximum *stress* at *service load* for F_y and the compression *stress* at *service load*, f_d, for f in *Specification* Section B2.5.

Figure C-B2.5-2 shows the built-up deck section in bending. In Figures C-B2.5-1 and C-B2.5-2, s is the center-to-center connection spacing along the plate, w is the center-to-center connection spacing across the plate, t is the *thickness* of cover plate, t_2 is the *thickness* of the member connected to the cover plate, f is the compression *stress* in the cover plate, f_c is the compression *stress* in the element connected to the cover plate, f_t is the maximum tension *stress* in the member connected to the cover plate, and d is the overall depth of the built-up member.

In 2012, provisions for determining the *effective width* between the first line of fasteners and the edge stiffener and the effective length of the stiffener were added. The post-*buckling stress* at the first interior line of *connections* is applied across the first interior width, w_1 or w_3, as illustrated in Figure C-B2.5-1, and at the edge stiffener. *Specification* Equation B2.5-7 is based on the approximate shape of the half sine wave restrained by the connectors in the compression element and by the edge stiffener. w′ given in *Specification* Equation B2.5-7 is twice the distance from the stiffener to the apex of the wave and models w in *Specification* Section B4 for the same wave length. Equation B2.5-6 sets w as e before "column-like" *buckling* occurs. *Specification* Equations B4-7 to B4-10 are then applied based on w and f. When f reaches or exceeds F_c, *Specification* Equations B4-7 to B4-10 are applied based on w′ and f′ to evaluate the stiffener. $\rho_m f$ approximates the post-*buckling stress* that cannot be less than F_c since the stiffener must resist F_c as *buckling* begins.

Jones (Jones, et al, 1997) validated Luttrell's method (1992), but the researchers cautioned its use for single-fluted members having compression plates with edge stiffeners. Luttrell and Balaji (1992) tested built-up deck with compression plate *thickness* between 0.045 in. (1.14 mm) and 0.06 in. (1.52 mm). Jones (1997) investigated unstiffened cover plates to 0.017 in. (0.432 mm) in *thickness*. The research work at the University of Missouri-Rolla (UMR) indicated that the method worked reasonably well for single-fluted members having unstiffened compression plates when the plate *thickness* exceeded 0.045 in. (1.14 mm). See the illustrative example in the AISI *Cold-Formed Steel Design Manual* (AISI, 2013).

B3 Effective Widths of Unstiffened Elements

Similar to stiffened compression elements, the *stress* in the *unstiffened compression elements* can reach to the *yield stress* of steel if the w/t ratio is small. Because the unstiffened element has one longitudinal edge supported by the *web* and the other edge is free, the limiting width-to-*thickness* ratio of unstiffened elements is much less than that for stiffened elements.

When the w/t ratio of the unstiffened element is large, *local buckling* (Figure C-B3-1) will occur at the elastic critical *stress* determined by Equation C-B2-1 with a value of k = 0.43. This *buckling* coefficient is listed in Table C-B2-1 for case (c). For the intermediate range of w/t ratios, the unstiffened element will buckle in the inelastic range. Figure C-B3-2 shows the relationship between the maximum *stress* for *unstiffened compression elements* and the w/t ratio, in which Line A is the *yield stress* of steel, Line B represents the inelastic *buckling stress*, and Curves C and D illustrate the elastic *buckling stress*. The equations for Curves A, B, C, and D have been developed from previous experimental and analytical investigations and used for determining the allowable *stresses* in the AISI *Specification* up to 1986 (Winter, 1970; Yu and LaBoube, 2010). Also shown in Figure C-B3-2 is Curve E, which represents the maximum *stress* on the basis of the post-*buckling* strength of the unstiffened element. The correlation between some test data on unstiffened elements and the predicted maximum *stresses* is shown in Figure C-B3-3 (Yu and LaBoube, 2010).

Prior to 1986, it had been a general practice to design cold-formed steel members with unstiffened *flanges* by using the A*llowable Stress Design* approach. The *effective width* equation was not used in earlier editions of the AISI *Specification* due to lack of extensive experimental verification and the concern for excessive out-of-plane distortions under *service loads*.

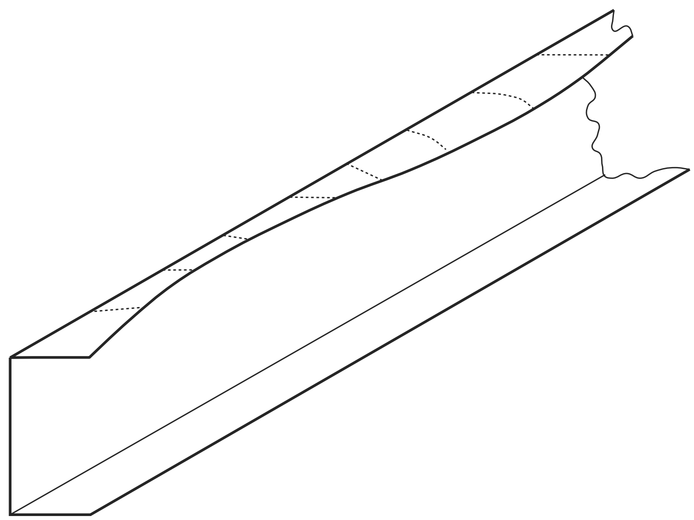


Figure C-B3-1 Local Buckling of Unstiffened Compression Flange

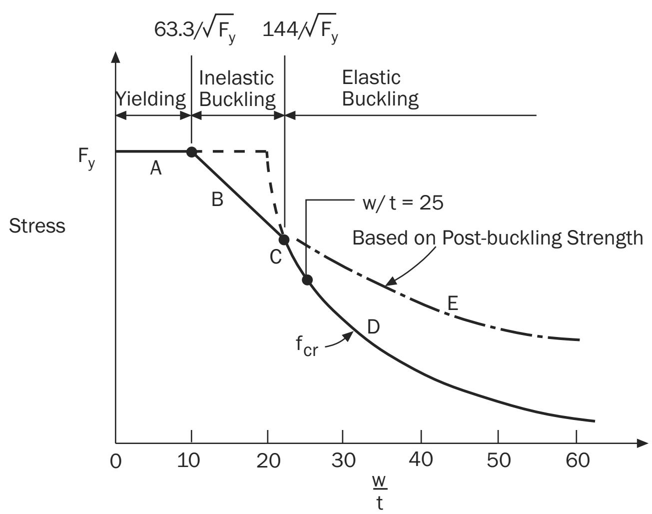


Figure C-B3-2 Maximum Stress for Unstiffened Compression Elements

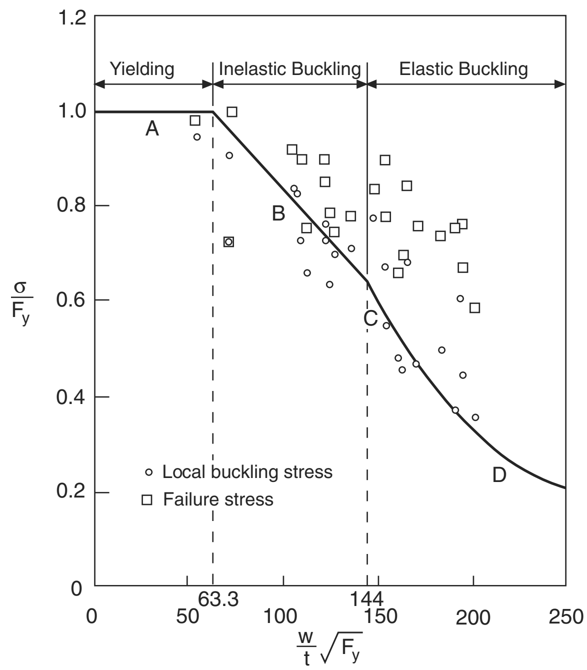


Figure C-B3-3 Correlation Between Test Data and Predicted Maximum Stress

In the 1970s, the applicability of the *effective width* concept to unstiffened elements under uniform compression was studied in detail by Kalyanaraman, Peköz, and Winter at Cornell University (Kalyanaraman, Peköz, and Winter, 1977; Kalyanaraman and Peköz, 1978). The evaluation of the test data using k = 0.43 was presented and summarized by Peköz in the AISI report (Peköz, 1986b), which indicates that Equation C-B2.1-6 developed for stiffened compression elements gives a conservative lower bound to the test results of *unstiffened compression elements*. In addition to the strength determination, the same study also investigated the out-of-plane deformations in unstiffened elements. The results of theoretical calculations and the test results on the sections having unstiffened elements with w/t = 60 were presented by Peköz in the same report. It was found that the maximum amplitude of the out-of-plane deformation at failure can be twice the *thickness* as the w/t ratio approaches 60. However, the deformations are significantly less under the *service loads*. Based on the above reasons and justifications, the *effective design width* approach was adopted for the first time in Section B3 of the 1986 AISI *Specification* for the design of cold-formed steel members having *unstiffened compression elements*.

B3.1 Uniformly Compressed Unstiffened Elements

In the present *Specification*, it is specified that the *effective widths*, *b*, of uniformly compressed unstiffened elements can be determined in accordance with Section B2.1(a) of the *Specification* with the exception that the *buckling* coefficient, k, is taken as 0.43. This is a theoretical value for long plates. See case (c) in Table C-B2-1. For serviceability determination, the *effective widths* of uniformly compressed unstiffened elements can only be determined according to Procedure I of Section B2.1(b) of the *Specification*, because Procedure II was developed only for stiffened compression elements. See Part I of the AISI *Design Manual* for design examples (AISI, 2013).

B3.2 Unstiffened Elements and Edge Stiffeners With Stress Gradient

In concentrically loaded compression members and in flexural members where the unstiffened compression element is parallel to the neutral axis, the *stress* distribution is uniform prior to *local buckling*. However, when edge stiffeners of the compression element are present, the compressive *stress* in the edge stiffener is not uniform but varies in proportion to the distance from the neutral axis. The unstiffened element (the edge stiffener) in this case has compressive *stress* applied at both longitudinal edges. The unstiffened element of a section may also be subjected to *stress* gradients causing tension at one longitudinal edge and compression at the other longitudinal edge. This can occur in I-sections, plain channel sections and angle sections in minor axis bending.

Previous to the 2001 edition of the *Specification*, unstiffened elements with *stress* gradient were designed using the Winter *effective width* equation (Equation C-B2.1-4) and k = 0.43. In 2004, Section B3.2 of the *Specification* adopted the *effective width* method for unstiffened elements with *stress* gradient proposed by Bambach and Rasmussen (2002a, 2002b and 2002c), based on an extensive experimental investigation of unstiffened plates tested as isolated elements in combined compression and bending. The *effective width*, b, (measured from the supported edge) of unstiffened elements with *stress* gradient causing compression at both longitudinal edges, is calculated using the Winter equation. For unstiffened elements with *stress* gradients causing tension at one longitudinal edge and compression at the other longitudinal edge, modified Winter equations are specified when tension exists at either the supported or the unsupported edges. The *effective width* equations apply to any unstiffened element under *stress* gradient, and are not restricted to particular cross-sections. Figure C-B3.2-1 demonstrates how the *effective width* of an unstiffened element increases as the *stress* at the supported edge changes from compression to tension. As shown in the figure, the *effective width* curve is independent of the *stress* ratio, ψ, when both edges are in compression. In this case, the effect of *stress* ratio is accounted for by the plate *buckling* coefficient, k, which varies with *stress* ratio and affects the slenderness, λ. When the supported edge is in tension and the

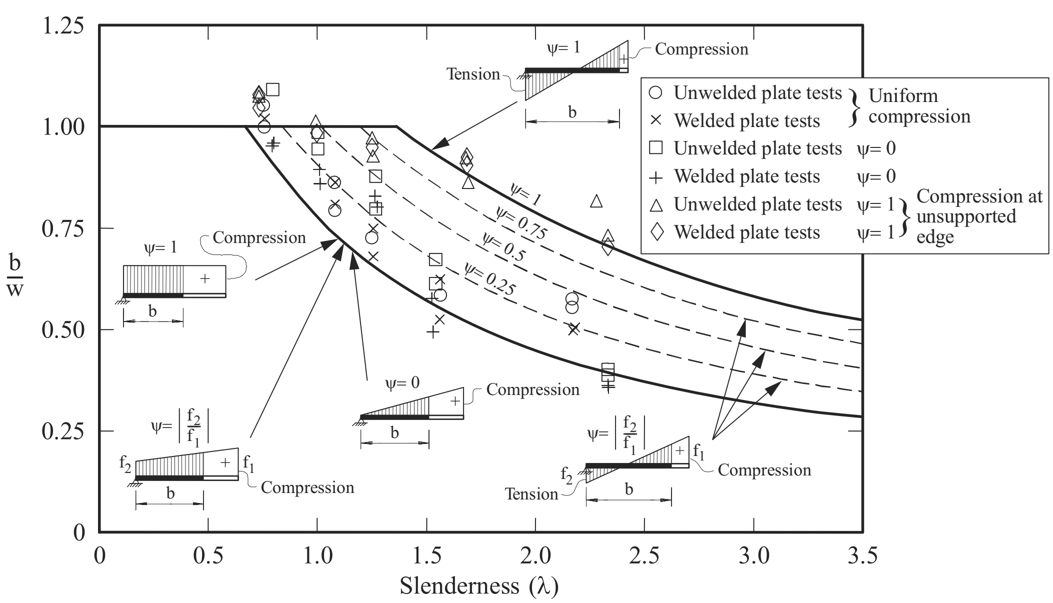


Figure C-B3.2-1 Effective Width vs. Plate Slenderness

unsupported edge is in compression, both the *effective width* curve and the plate *buckling* coefficient depend on the *stress* ratio, as per Equations B3.2-4 and B3.2-5 of the *Specification*.

Equations are provided for k, determined from the *stress* ratio, ψ, applied to the full element width such that iteration is not required, and k will usually be higher than 0.43. The equations for k are theoretical solutions for long plates assuming simple support along the longitudinal edge. A more accurate determination of k by accounting for interaction between adjoining elements is permitted for plain channels in minor axis bending (causing compression at the unsupported edge of the unstiffened element), based on research of plain channels in compression and bending by Yiu and Peköz (2001).

The *effective width* is located adjacent to the supported edge for all *stress* ratios, including those producing tension at the unsupported edge. Research has found (Bambach and Rasmussen 2002a) that for the unsupported edge to be effective, tension must be applied over at least half of the width of the element starting at the unsupported edge. For less tension, the unsupported edge will buckle and the effective part of the element is located adjacent to the supported edge. Further, when tension is applied over half of the element or more starting at the unsupported edge, the compressed part of the element will remain effective for elements with w/t ratios less than the limits set out in Section B1.1 of the *Specification*.

The method for serviceability determination is based on the method used for stiffened elements with *stress* gradient in Section B2.3(b) of the *Specification*.

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

An edge stiffener is used to provide continuous support along a longitudinal edge of the compression *flange* to improve the *buckling stress*. In most cases, the edge stiffener takes the form of a simple lip. Other types of edge stiffeners can be beneficial and are also used for cold-formed steel members, but are not covered in *Specification* Section B4.

In order to provide necessary support for the compression element, the edge stiffener must possess sufficient rigidity. Otherwise it may buckle perpendicular to the plane of the element to be stiffened. As far as the design provisions are concerned, the 1980 and earlier editions of the AISI *Specification* included the requirements for the minimum moment of inertia of stiffeners to provide sufficient rigidity. When the size of the actual stiffener does not satisfy the required moment of inertia, the *load*-carrying capacity of the beam has to be determined either on the basis of a flat element disregarding the stiffener or through tests.

Both theoretical and experimental studies on the local stability of compression *flanges* stiffened by edge stiffeners have been carried out in the past. The design requirements included in Section B4 of the 1986 AISI *Specification* were based on the investigations of adequately stiffened and partially stiffened elements conducted by Desmond, Peköz and Winter (1981a), with additional research work by Peköz and Cohen (Peköz, 1986b). These design provisions were developed on the basis of the critical *buckling* criterion and the post-*buckling* strength criterion.

Specification Section B4 recognizes that the necessary stiffener rigidity depends upon the slenderness (w/t) of the plate element being stiffened. The interaction of the plate elements, as well as the degree of edge support, full or partial, is compensated for in the expressions for k, d_s, and A_s (Peköz, 1986b).

In the 1996 edition of the AISI *Specification* (AISI, 1996), the design equations for *buckling* coefficient were changed for further clarity. The requirement of $140° \geq \theta \geq 40°$ for the applicability of these provisions was decided on an intuitive basis. For design examples, see Part I of the *Cold-Formed Steel Manual* (AISI, 2013).

Test data to verify the accuracy of the simple lip stiffener design was collected from a number of sources, both university and industry. These tests showed good correlation with the equations in *Specification* Section B4.

The 1996 *Commentary* provided a warning to the user that lip lengths with a d/t ratio greater than 14 may give unconservative results. Examination of available experimental data on both flexural members (Rogers and Schuster, 1996; Schafer and Peköz, 1999) and compression members (Schafer, 2000) with edge stiffeners indicates that the *Specification* does not have an inherent problem for members with large d/t ratios. Existing experimental data covers d/t ratios as high as 35 for both flexural and compression members.

In 2001, Dinovitzer's expressions (Dinovitzer, et al., 1992) for n (Equation B4-11) were adopted, which eliminated a discontinuity that existed in the previous design expressions. The revised equation gives n =1/2 for w/t = 0.328S and n = 1/3 for w/t = S, in which S is also the maximum w/t ratio for a stiffened element to be fully effective.

In 2007, the expressions were limited to cover only simple lip edge stiffeners, as the previously employed expressions for complex lip stiffeners were found to be unconservative in comparison with rigorous nonlinear finite element analysis (Schafer, et al., 2006). Design of members with complex lips may be handled via the methods of *Specification* Appendix 1. In addition, the design provisions for the uniformly compressed elements with one intermediate stiffener were deleted in the 2007 edition of the *Specification* due to the fact that the *effective width* of such members can be determined in accordance with *Specification* Section B5.1.

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

B5.1 Effective Width of Uniformly Compressed Stiffened Elements With Single or Multiple Intermediate Stiffeners

The structural efficiency of a stiffened element always exceeds that of an unstiffened element with the same w/t ratio by a sizeable margin, except for low w/t ratios, for which the compression element is fully effective. When stiffened elements with large w/t ratios are used, the material is not employed economically inasmuch as an increasing proportion of the width of the compression element becomes ineffective. On the other hand, in many applications of cold-formed steel construction, such as panels and decks, maximum coverage is desired and, therefore, large w/t ratios are called for. In such cases, structural economy can be improved by providing intermediate stiffeners between *webs*.

The *buckling* behavior of rectangular plates with central stiffeners is discussed by Bulson (1969). For the design of cold-formed steel beams using intermediate stiffeners, the 1980 AISI *Specification* contained provisions for the minimum required moment of inertia, which was based on the assumption that an intermediate stiffener needed to be twice as rigid as an edge stiffener. In view of the fact that for some cases the design requirements for intermediate stiffeners included in the 1980 *Specification* could be unduly conservative (Peköz, 1986b), the AISI design provisions were revised in 1986 according to Peköz's research findings (Peköz, 1986b and 1986c) and prior to 2007 could be found in Section B4.1 of the *Specification*. In 2007, the design of uniformly compressed elements with multiple or single intermediate stiffeners was merged. The multiple intermediate stiffener provisions were developed based on

Peköz's continuing research on intermediate stiffeners (Schafer and Peköz 1998) and the finding that the method developed in B5.1 of the *Specification* for multiple intermediate stiffeners could provide the same reliability as the *Specification* Section B4.1 (AISI, 2001) method for single intermediate stiffeners (Yang and Schafer, 2006).

Prior to 2001, the AISI *Specification* and the Canadian *Standard* provided different design provisions for determination of the *effective widths* of uniformly compressed stiffened elements with multiple intermediate stiffeners or edge-stiffened elements with intermediate stiffeners. In the AISI *Specification*, the design requirements of Section B5 dealt with: (1) the minimum moment of inertia of the intermediate stiffener, (2) the number of intermediate stiffeners considered to be effective, (3) the "equivalent element" of *multiple-stiffened element* having closely spaced intermediate stiffeners, (4) the *effective width* of sub-element with $w/t >$ 60, and (5) the reduced area of stiffeners. In the Canadian *Standard*, a different design equation was used to determine the equivalent *thickness*.

In 2010, *Specification* Equation B5.1.1-1 was replaced by

$$k_{loc} = 4(b_o/b_p)^2 \tag{C-B5.1-1}$$

where

k_{loc} = plate *buckling* coefficient of element

b_o = total *flat width* of stiffened element

b_p = sub-element *flat width* for *flange* with equally spaced stiffeners

This replacement ensures that *Specification* Sections B5.1.1 and B5.1.2 provide the same answer for sub-element *local buckling*, and replaces the overly conservative estimate of the 2007 edition of the *Specification* Equation B5.1.1-1, which ignored the stiffener width (Schafer, 2009).

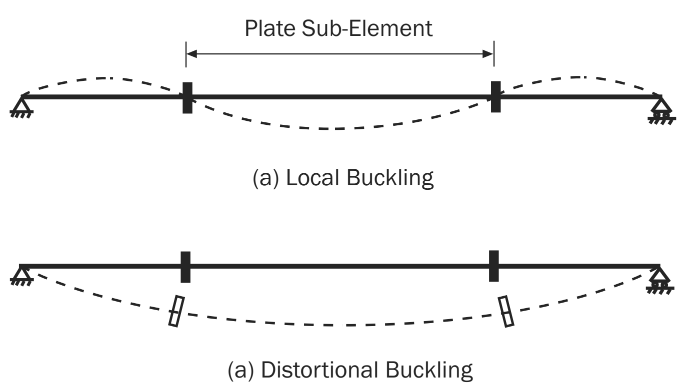


Figure C-B5.1-1 Local and Distortional Buckling of a Uniformly Compressed Element With Multiple Intermediate Stiffeners

In 2001, *Specification* Section B5.1 was revised to reflect recent research findings for flexural members with multiple intermediate stiffeners in the compression *flange* (Papazian et. al., 1994; Schafer and Peköz, 1998; Acharya and Schuster, 1998). The method is based on determining the plate *buckling* coefficient for the two competing modes of *buckling*: *local buckling*, in which the stiffener does not move; and *distortional buckling*, in which the stiffener buckles with the entire plate. See Figure C-B5.1-1. Experimental research shows that the distortional mode is prevalent for members with multiple intermediate stiffeners.

The reduction factor, ρ, is applied to the entire element (*gross area* of the element/*thickness*) instead of only the flat portions. Reducing the entire element to an *effective width*, which ignores the geometry of the stiffeners, for effective section property calculation allows *distortional buckling* to be treated consistently with the rest of the *Specification*, rather than as an "*effective area*" or other method. The resulting *effective width* must act at the centroid of the original element including the stiffeners. This ensures that the neutral axis location for the member is unaffected by the use of the simple *effective width*, which replaces the more complicated geometry of the element with multiple intermediate stiffeners. One possible result of this approach is that the calculated *effective width* (b_e) may be greater than b_o. This may occur when ρ is near 1, and is due to the fact that b_e includes contributions from the stiffener area and b_o does not. As long as the calculated b_e is placed at the centroid of the entire element, the use of $b_e > b_o$ is correct.

B5.2 Edge-Stiffened Elements With Intermediate Stiffener(s)

The *buckling* modes for edge-stiffened elements with one or more intermediate stiffeners include *local* sub-element *buckling*, *distortional buckling* of the intermediate stiffener, and *distortional buckling* of the edge stiffener, as shown in Figure C-B5.2-1. If the edge-stiffened element is stocky ($b_o/t < 0.328S$) or the stiffener is large enough ($I_s > I_a$ and thus k = 4, per the rules of *Specification* Section B4), then the edge-stiffened element performs as a stiffened element. In this case, *effective width* for *local* sub-element *buckling* and *distortional buckling* of the intermediate stiffener may be predicted by the rules of *Specification* Section B5.1. However, an edge-stiffened element does not have the same *web* rotational restraint as a stiffened element; therefore, the constant R of *Specification* Section B5.1 is conservatively limited to be less than or equal to 1.0.

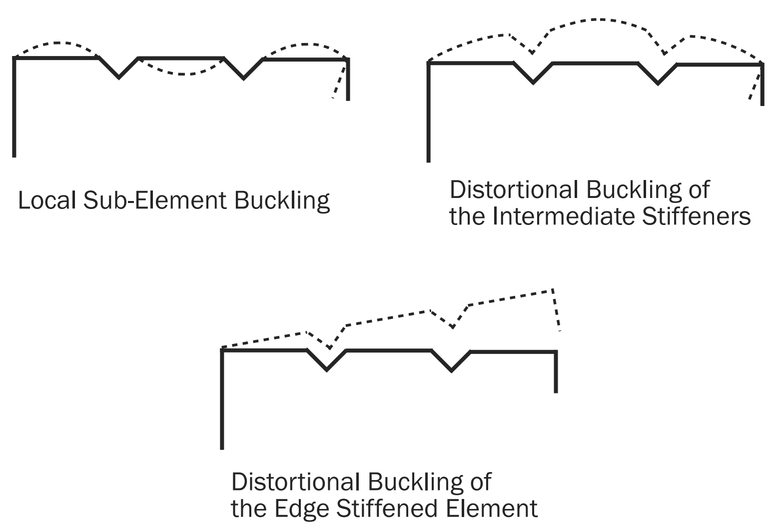


Figure C-B5.2-1 Buckling Modes in an Edge-stiffened Element With Intermediate Stiffeners

If the edge-stiffened element is partially effective ($b_o/t > 0.328S$ and $I_s < I_a$ and thus k < 4, per the rules of *Specification* Section B4), then the intermediate stiffener(s) should be ignored and the provisions of *Specification* Section B4 followed. Elastic *buckling* analysis of the distortional mode for an edge-stiffened element with intermediate stiffener(s) indicates that the effect of intermediate stiffener(s) on the *distortional buckling stress* is ±10 percent for practical intermediate and edge stiffener sizes.

When applying *Specification* Section B5.2 for *effective width* determination of edge-stiffened elements with intermediate stiffeners, the *effective width* of the intermediately stiffened *flange*, b_e, is replaced by an equivalent flat section (as shown in *Specification* Figure B5.1-2). The edge stiffener should not be used in determining the centroid location of the equivalent flat *effective width*, b_e, for the intermediately stiffened *flange*.

Stub compression testing performed in 2003 demonstrates the adequacy of this approach (Yang and Hancock, 2003).

C. MEMBERS

This chapter provides the design requirements for: (a) tension members, (b) flexural members, (c) concentrically loaded compression members, and (d) members subjected to combined axial *load* and bending.

In 2007, the following design provisions were moved from *Specification* Chapter C, Members, to Section D6, Metal Roof and Wall Systems: (1) Flexural Members Having One *Flange* Through-Fastened to Deck or Sheathing, (2) Flexural Members Having One *Flange* Fastened to a Standing Seam Roof System, (3) Compression Members Having One *Flange* Through-Fastened to Deck or Sheathing, and (4) Strength [Resistance] of Standing Seam Panel Systems. For closed cylindrical tubular members, the design provisions have been moved to the new Section C3.1.3 for flexural members and the new Section C4.1.5 for compression members.

In general, a common *nominal strength* [*resistance*] equation is provided in the *Specification* for a given limit state with a required *safety factor* (Ω) for *Allowable Strength Design* (*ASD*) and a *resistance factor* (ϕ) for *Load and Resistance Factor Design* (*LRFD*) or *Limit States Design* (*LSD*). Design provisions that are applicable to a specific country are provided in the corresponding appendix.

C1 Properties of Sections

The geometric properties of a member (i.e., area, moment of inertia, section modulus, radius of gyration, etc.) are evaluated using conventional methods of structural design. These properties are based upon full cross-section dimensions, *effective widths*, or net section, as applicable.

For the design of tension members, both gross and net sections are employed when computing the *nominal tensile strength* [*resistance*] of the axially loaded tension members.

For flexural members and axially loaded compression members, both full and effective dimensions are used to compute sectional properties. The full dimensions are used when calculating the critical *load* or moment, while the effective dimensions, evaluated at the *stress* corresponding to the critical *load* or moment, are used to calculate the *nominal strength* [*resistance*]. For serviceability consideration, the effective dimension should be determined for the compressive *stress* in the element corresponding to the *service load*. Peköz (1986a and 1986b) discussed this concept in more detail.

Section 3 of Part I of the AISI *Design Manual* (AISI, 2013) deals with the calculation of sectional properties for C-sections, Z-sections, angles, hat sections, and decks.

C2 Tension Members

In 2010, the provisions for tension members were consolidated and moved from the country-specific appendices to the main *Specification*. The *available tensile strength* [*factored resistance*] of axially loaded cold-formed steel tension members is determined either by yielding of the *gross area* of the cross-section or by rupture of the *net area* of the cross-section. At locations of *connections*, the *nominal tensile strength* [*resistance*] is also limited by the *available strengths* [*factored resistances*] specified in *Specification* Chapter E for tension in connected parts.

C2.1 Yielding of Gross Section

Yielding in the gross section indirectly provides a limit on the deformation that a tension member can achieve. The definition of yielding in the gross section to determine the tensile strength is well established in hot-rolled steel construction.

The *resistance factor* ϕ_t = 0.90 and *safety factor* Ω_t = 1.67 used for yielding of the gross section are consistent with the factors used in ANSI/AISC 360 Specification (AISC, 2010) and CSA S16 Specification (CSA, 2009).

C2.2 Rupture of Net Section

The *resistance factor* of ϕ_t = 0.75 and *safety factor* of Ω_t = 2.00 used for rupture of the net section are consistent with the factors used in the ANSI/AISC 360 Specification (AISC, 2010) and CSA S16 Specification (CSA, 2009).

C3 Flexural Members

For the design of cold-formed steel flexural members, consideration should be given to several design features: (a) bending strength and serviceability, (b) shear strength of *webs* and combined bending and shear, (c) *web crippling* strength and combined bending and *web crippling*, and (d) bracing requirements. For some cases, special consideration should also be given to shear lag and *flange* curling due to the use of thin material. The design provisions for items (a), (b) and (c) are provided in *Specification* Sections C3, D6.1 and D6.2; while the requirements for lateral and stability bracing are given in *Specification* Sections D3 and D6.3. The treatments for *flange* curling and shear lag were discussed in Section B1.1(b) and (c) of the *Commentary*, respectively.

Example problems are given in Part II of the AISI *Cold-Formed Steel Design Manual* (AISI, 2013) for the design of flexural members.

C3.1 Bending

Bending strengths of flexural members are differentiated according to whether or not the member is laterally braced. If such members are laterally supported, then they are proportioned according to the nominal section strength *(Specification* Section C3.1.1). Since the *distortional buckling* has an intermediate *buckling* half wavelength, the *distortional buckling* still needs to be considered even for braced members. See the *Direct Strength Method Design Guide* (AISI, 2006) for detailed discussion and design examples. If they are laterally unbraced, then the limit state is *lateral-torsional buckling* (*Specification* Section C3.1.2). For C- or Z-sections with the tension *flange* attached to deck or sheathing and with compression *flange* laterally unbraced, the bending capacity is less than that of a fully braced member but greater than that of an unbraced member (*Specification* Section D6.1.1). For C- or Z-sections supporting a standing seam roof system under gravity or uplift *loads*, the bending capacity is greater than that of an unbraced member and may be equal to that of a fully braced member (*Specification* Section D6.1.2). Similarly, for standing seam roof systems, design provisions are provided in *Specification* Section D6.2.1 for evaluating the bending strength of the system based on tests. The governing *available bending strength* [*factored resistance*] is the smallest of the values determined from the applicable conditions.

C3.1.1 Nominal Section Strength [Resistance]

Specification Section C3.1.1 includes two design procedures for calculating the *nominal section strength* [*resistance*] of flexural members. Procedure I is based on initiation of yielding and Procedure II is based on inelastic reserve capacity.

(a) *Procedure I - Based on Initiation of Yielding*

In Procedure I, the nominal moment, M_n, of the cross-section is the effective *yield moment*, M_y, determined on the basis of the *effective areas* of *flanges* and the beam *web*. The *effective width* of the compression *flange* and the effective depth of the *web* can be computed from the design equations given in Chapter B of the *Specification*.

Similar to the design of hot-rolled steel shapes, the *yield moment*, M_y, of a cold-formed steel beam is defined as the moment at which an outer fiber (tension, compression, or both) first attains the *yield stress* of the steel. This is the maximum bending capacity to be used in elastic design. Figure C-C3.1.1-1 shows several types of *stress* distributions for *yield moment* based on different locations of the neutral axis. For balanced sections (Figure C-C3.1.1-1(a)) the outer fibers in the compression and tension *flanges* reach the *yield stress* at the same time. However, if the neutral axis is eccentrically located, as shown in Figures C-C3.1.1-1(b) and (c), the initial yielding takes place in the tension *flange* for case (b) and in the compression *flange* for case (c).

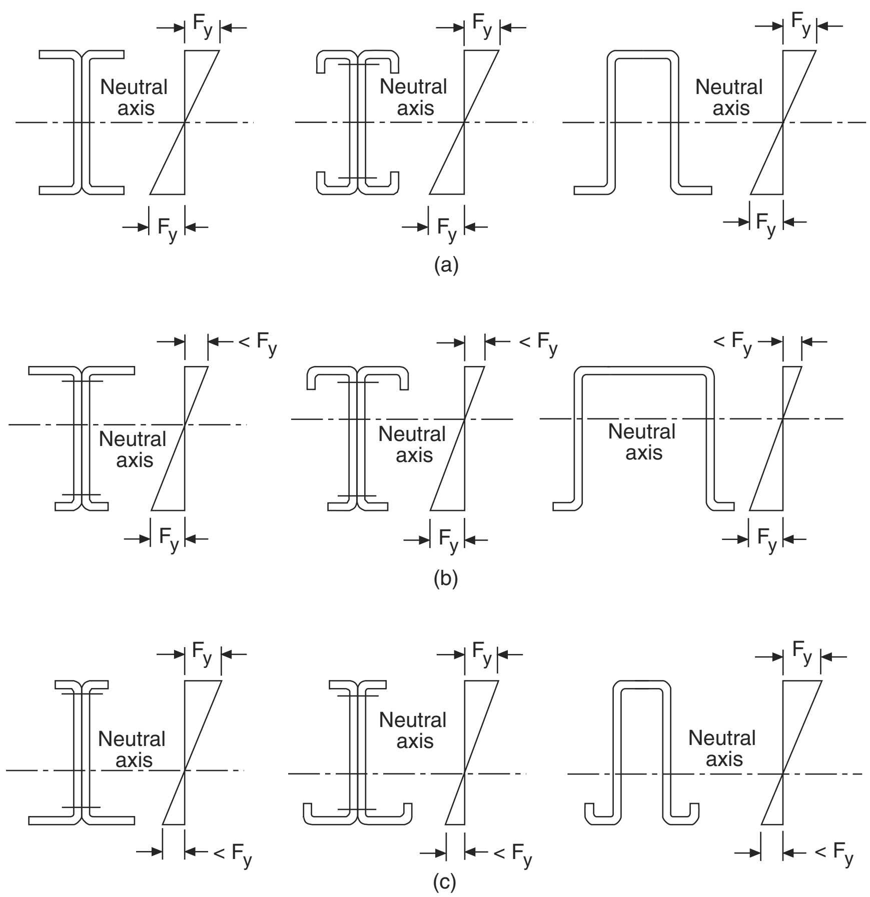


Figure C-C3.1.1-1 Stress Distribution for Yield Moment
(a) Balanced Sections, (b) Neutral Axis Close to Compression Flange,
(c) Neutral Axis Close to Tensions Flange

Accordingly, the *nominal section strength* [*resistance*] for *initiation of yielding* is calculated by using Equation C-C3.1.1-1:

$$M_n = S_e F_y \tag{C-C3.1.1-1}$$

where

F_y = design *yield stress*

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

For cold-formed steel design, S_e is usually computed by using one of the following two cases:

1. If the neutral axis is closer to the tension than to the compression *flange*, the maximum *stress* occurs in the compression *flange*, and therefore the plate slenderness ratio λ and the *effective width* of the compression *flange* are determined by the w/t ratio and $f = F_y$. Of course, this procedure is also applicable to those beams for which the neutral axis is located at the mid-depth of the section.
2. If the neutral axis is closer to the compression than to the tension *flange*, the maximum *stress* of F_y occurs in the tension *flange*. The *stress* in the compression *flange* depends on the location of the neutral axis, which is determined by the *effective area* of the section. The latter cannot be determined unless the compressive *stress* is known. The closed-form solution of this type of design is possible but would be a very tedious and complex procedure. It is therefore customary to determine the sectional properties of the section by successive approximation.

Prior to the 2008 edition of the AISI *Specification*, the *design flexural strength* [*factored resistance*], $\phi_b M_n$, employed different ϕ_b factors depending on the compression *flange*. Based on the 1991 edition of the AISI *Specification*, and the work of Hsiao, Yu and Galambos (1988a), unstiffened *flanges* were specified at ϕ_b= 0.90 and edge stiffened or stiffened *flanges* at ϕ_b = 0.95 (*ASD* used one Ω factor for all cases). Examination of more recently available test data (Schafer and Trestain, 2002; Yu and Schafer, 2003) and consideration of the fact that the higher ϕ_b existed in part due to inelastic reserve strength, which is already addressed in *Specification* Section C3.1.1(b), a uniform ϕ_b= 0.90 was adopted for all members. This change also removed a conflict with the ϕ_b factors adopted in 2007 for *Specification* Section C3.1.4, when the member is fully effective.

(b) Procedure II - Based on Inelastic Reserve Capacity

Prior to 1980, the inelastic reserve capacity of beams was not included in the AISI *Specification* because most cold-formed steel shapes have large width-to-*thickness* ratios that are considerably in excess of the limits required by plastic design.

In the 1970s and early 1980s, research work on the inelastic strength of cold-formed steel beams was carried out by Reck, Peköz, Winter, and Yener at Cornell University (Reck, Peköz and Winter, 1975; Yener and Peköz, 1985a, 1985b). These studies showed that the *inelastic reserve* strength of cold-formed steel beams due to partial plastification of the cross-section and the moment redistribution of statically indeterminate beams can be significant for certain practical shapes. With proper care, this reserve strength can be utilized to achieve more economical design of such members.

In order to utilize the *available inelastic reserve strength* [*factored resistance*] of certain cold-formed steel beams, design provisions based on the partial plastification of the cross-section were added in the 1980 edition of the AISI *Specification*. The same provisions are retained in the 2001 and the 2007 editions of the *Specification*. According to Procedure II of Section C3.1.1(b) of the *Specification*, the *nominal section strength* [*resistance*], M_n, of those beams satisfying certain specific limitations can be determined on the basis of the inelastic reserve capacity with a limit of $1.25M_y$, where M_y is the effective *yield moment*. The ratio of M_n/M_y represents the inelastic reserve strength of a beam cross-section.

The *nominal moment* [*resistance*], M_n, is the maximum bending capacity of the beam by considering the inelastic reserve strength through partial plastification of the cross-section. The inelastic *stress* distribution in the cross-section depends on the maximum strain in the compression *flange*, ε_{cu}. Based on the Cornell research work on hat sections having stiffened compression *flanges* (Reck, Peköz and Winter, 1975), the AISI design provision limits the maximum compression strain to be $C_y\varepsilon_y$, where C_y is a compression strain factor determined by using the equations provided in *Specification* Section C3.1.1(b) (i) as shown in Figure C-C3.1.1-2.

On the basis of the maximum compression strain ε_{cu} allowed in the *Specification*, the neutral axis can be located by using Equation C-C3.1.1-2 and the nominal moment [resistance] M_n can be determined by using Equation C-C3.1.1-3:

$$\int \sigma dA = 0 \tag{C-C3.1.1-2}$$

$$\int \sigma y dA = M_n \tag{C-C3.1.1-3}$$

where σ is the *stress* in the cross-section.

The calculation of M_n based on inelastic reserve capacity is illustrated in Part I of the AISI *Cold-Formed Steel Design Manual* (AISI, 2013) and the textbook by Yu and LaBoube (2010).

In 2001, the shear force upper limit was clarified. The *stress* upper limit is $0.35F_y$ for *ASD* and $0.6F_y$ for *LRFD* and *LSD* in the *North American Specification*.

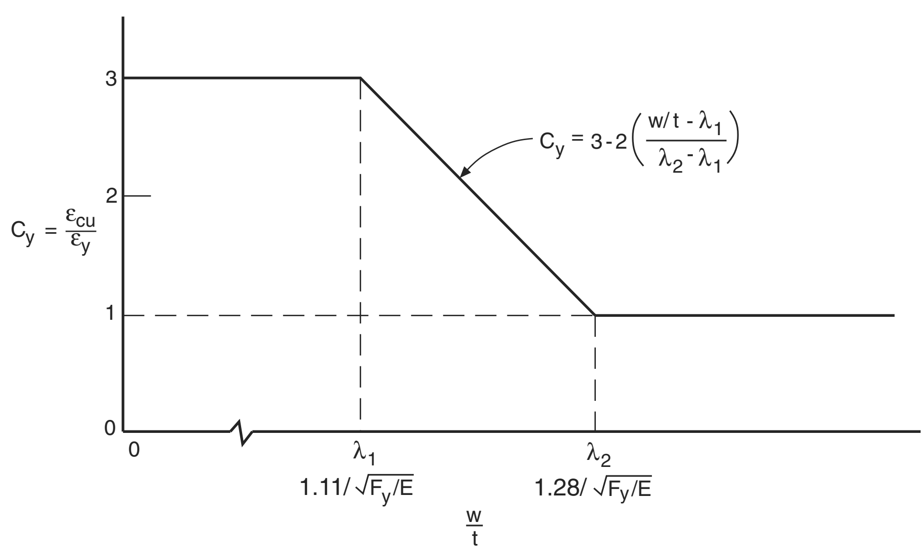


Figure C-C3.1.1-2 Factor C_y for Stiffened Compression Elements Without Intermediate Stiffeners

In 2004, additional *Specification* equations were provided in Section C3.1.1(b) for determining the *nominal moment strength* [*resistance*], M_n, based on inelastic reserve capacity, for sections containing *unstiffened compression elements* under *stress* gradient. Based on research by Bambach and Rasmussen (2002b, 2002c) on I- and plain channel sections in minor axis bending, a compression strain factor, C_y, determines the maximum compressive strain on the unstiffened element of the section. The C_y values are dependent on the *stress* ratio ψ and slenderness ratio λ of the unstiffened element, determined in accordance with Section B3.2(a) of the *Specification*.

C3.1.2 Lateral-Torsional Buckling Strength [Resistance]

The bending capacity of flexural members is not only governed by the strength of the cross-section, but can also be limited by the *lateral-torsional buckling* strength of the member if braces are not adequately provided. The design provisions for determining the *nominal lateral-torsional buckling strength* [*resistance*] are given in *Specification* Section C3.1.2.1 for open cross-section members and C3.1.2.2 for closed tubular members.

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

If a doubly-symmetric or singly-symmetric member in bending is laterally unbraced, it can fail in *lateral-torsional buckling*. For a beam having simply supported end conditions both laterally and torsionally, the elastic critical *lateral-torsional buckling stress* can be determined by Equation C-C3.1.2.1-1.

$$\sigma_{cr} = \frac{\pi}{LS_f}\sqrt{EI_yGJ\left(1+\frac{\pi^2 EC_w}{GJL^2}\right)} \qquad \text{(C-C3.1.2.1-1)}$$

For other than simply supported end conditions, Equation C-C3.1.2.1-1 can be generalized as given in Equation C-C3.1.2.1-1a (Galambos, 1998):

$$\sigma_{cr} = \frac{\pi}{(K_yL_y)S_f}\sqrt{EI_yGJ\left[1+\frac{\pi^2 EC_w}{GJ(K_tL_t)^2}\right]} \qquad \text{(C-C3.1.2.1-1a)}$$

In the above equation, K_y and K_t are effective length factors and L_y and L_t are unbraced lengths for bending about the y-axis and for twisting, respectively, E is the modulus of elasticity, G is the shear modulus, S_f is the elastic section modulus of the full unreduced section relative to the extreme compression fiber, I_y is the moment of inertia about the y-axis, C_w is the torsional warping constant, J is the Saint-Venant torsion constant, and L is the unbraced length.

For equal-*flange* I-members with simply supported end conditions both laterally and torsionally, Equation C-C3.1.2.1-2 can be used to calculate the elastic critical *buckling stress* (Winter, 1947a; Yu and LaBoube, 2010):

$$\sigma_{cr} = \frac{\pi^2 E}{2(L/d)^2}\sqrt{\left(\frac{I_y}{2I_x}\right)^2+\left(\frac{JI_y}{2(1+\mu)I_x{}^2}\right)\left(\frac{L}{\pi d}\right)^2} \qquad \text{(C-C3.1.2.1-2)}$$

In Equation C-C3.1.2.1-2, the first term under the square root represents the lateral bending rigidity of the member, and the second term represents the Saint-Venant torsional rigidity. For thin-walled cold-formed steel sections, the first term usually exceeds the second term by a considerable margin.

For simply supported I-members with unequal *flanges*, the following equation has been derived by Winter for the *lateral-torsional buckling stress* (Winter, 1943):

$$\sigma_{cr} = \frac{\pi^2 Ed}{2L^2 S_f}\left(I_{yc} - I_{yt} + I_y\sqrt{1+\frac{4GJL^2}{\pi^2 I_y Ed^2}}\right) \qquad \text{(C-C3.1.2.1-3)}$$

where I_{yc} and I_{yt} are the moments of inertia of the compression and tension portions of the full section, respectively, about the centroidal axis parallel to the *web*. Other symbols were defined previously. For equal-*flange* sections, $I_{yc} = I_{yt} = I_y/2$, Equations C-C3.1.2.1-2 and C-C3.1.2.1-3 are identical.

For other than simply supported end conditions, Equation C-C3.1.2.1-3 can be generalized as given in Equation C-C3.1.2.1-3a:

$$\sigma_{cr} = \frac{\pi^2 Ed}{2(K_y L_y)^2 S_f}\left(I_{yc} - I_{yt} + I_y\sqrt{1+\frac{4GJ(K_t L_t)^2}{\pi^2 I_y Ed^2}}\right) \qquad \text{(C-C3.1.2.1-3a)}$$

In Equation C-C3.1.2.1-3a, the second term under the square root represents the Saint-Venant torsional rigidity, which can be neglected without any loss in economy. Therefore, Equation C-C3.1.2.1-3a can be simplified as shown in Equation C-C3.1.2.1-4 by considering $I_y = I_{yc} + I_{yt}$ and neglecting the term $4GJ(K_tL_t)^2/(\pi^2 I_y Ed^2)$:

$$\sigma_{cr} = \frac{\pi^2 EdI_{yc}}{(K_y L_y)^2 S_f} \qquad \text{(C-C3.1.2.1-4)}$$

Equation C-C3.1.2.1-4 was derived on the basis of a uniform bending moment and is conservative for other cases. For this reason, σ_{cr} is modified by multiplying the right-hand side by a bending coefficient, C_b, to account for non-uniform bending and the symbol F_e is used for σ_{cr}, i.e.,

$$F_e = \frac{C_b \pi^2 EdI_{yc}}{(K_y L_y)^2 S_f} \qquad \text{(C-C3.1.2.1-5)}$$

where C_b is the bending coefficient, which can conservatively be taken as unity, or calculated from

$$C_b = 1.75 + 1.05\,(M_1/M_2) + 0.3\,(M_1/M_2)^2 \le 2.3 \qquad \text{(C-C3.1.2.1-6)}$$

in which M_1 is the smaller and M_2 the larger bending moment at the ends of the unbraced length.

The above equation was used in the 1968, 1980, 1986, and 1991 editions of the AISI *Specification*. Because it is valid only for straight-line moment diagrams, Equation C-C3.1.2.1-6 was replaced by the following equation for C_b in the 1996 edition of the AISI *Specification* and is retained in this edition of the *Specification*:

$$C_b = \frac{12.5M_{max}}{2.5M_{max} + 3M_A + 4M_B + 3M_C} \qquad \text{(C-C3.1.2.1-7)}$$

where

M_{max} = absolute value of maximum moment in the unbraced segment

M_A = absolute value of moment at quarter point of unbraced segment

M_B = absolute value of moment at centerline of unbraced segment

M_C = absolute value of moment at three-quarter point of unbraced segment

Equation C-C3.1.2.1-7, derived from Kirby and Nethercot (1979), can be used for various shapes of moment diagrams within the unbraced segment. It gives more accurate solutions for fixed-end members in bending and moment diagrams which are not straight lines. This equation is the same as that being used in the ANSI/AISC S360 (AISC, 2010).

Figure C-C3.1.2.1-1 shows the differences between Equations C-C3.1.2.1-6 and C-C3.1.2.1-7 for a straight line moment diagram.

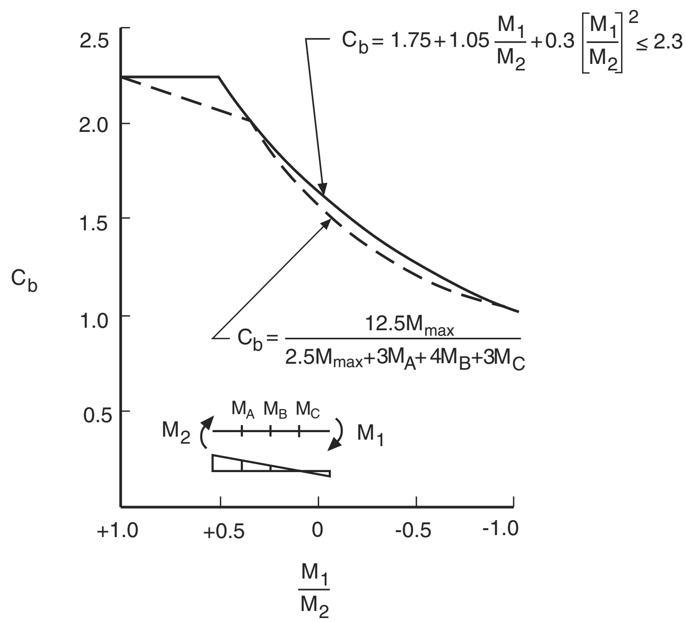


Figure C-C3.1.2.1-1 C_b for Straight Line Moment Diagram

In 2001, effective length factor about the y-axis, K_y, was added to *Specification* Equations C3.1.2.1-14 and C3.1.2.1-15 on the basis of Equation C-C3.1.2.1-5. The K_y factor provides for other than simply supported end conditions. In addition, *Specification* Equation C3.1.2.1-14 has been permitted to be used for the design of singly-symmetric C-sections and I sections since the 1968 edition of the AISI *Specification*, and C3.1.2.1-15 has been permitted to be used for Z-sections since the 1996 edition of the AISI *Specification*.

Also in 2001, the requirement of taking C_b equal to unity when considering axial *load* and bending moment in *Specification* Section C5 was removed. This requirement was in place since both C_b and C_m in *Specification* Section C5 are adjustments for the moment gradient in the member and it was conservative to take C_b equal to unity. C_b is an adjustment to the critical moment for *lateral-torsional buckling* when the bending moment is not constant, and C_m adjusts the magnitude of the second order p-delta moment in the member. Since these are two separate quantities, it is appropriate to use both C_b and C_m

in evaluating the member under combined *loads*. However, it is still conservative to take C_b equal to unity.

It should be noted that Equations C-C3.1.2.1-1a and C-C3.1.2.1-5 apply only to elastic *buckling* of cold-formed steel members in bending when the computed theoretical *buckling stress* is less than or equal to the proportional limit. When the computed *stress* exceeds the proportional limit, the beam behavior will be governed by inelastic *buckling*. The inelastic *buckling stress*, F_c, can be computed from Equation C-C3.1.2.1-8 (Yu and LaBoube, 2010):

$$F_c = \frac{10}{9} F_y \left(1 - \frac{10F_y}{36F_e}\right) \tag{C-C3.1.2.1-8}$$

where F_e is the elastic critical *lateral-torsional buckling stress*.

Equations C-C3.1.2.1-5 and C-C3.1.2.1-8 with K_y = 1.0 and L_y = L were used in the 1968, 1980 and 1986 editions of the AISI *Specification* to develop the *allowable stress design* equations for *lateral-torsional buckling* of I-members. In the 1986 edition of the AISI *Specification*, in addition to the use of Equations C-C3.1.2.1-5 and C-C3.1.2.1-8 for determining the critical *stresses*, more design equations (*Specification* Equations C3.1.2.1-4, C3.1.2.1-5, and C3.1.2.1-10) for elastic critical *stress* were added as alternative methods. These additional equations were developed from the previous studies conducted by Peköz, Winter and Celebi on *flexural-torsional buckling* of thin-walled sections under eccentric *loads* (Peköz and Winter, 1969a; Peköz and Celebi, 1969b) and are retained in this edition of the *Specification*. These general design equations can be used for *singly-*, *doubly-* and *point-symmetric sections*. Consequently, the elastic critical *lateral-torsional buckling stress* can be determined by the following equation:

$$F_e = \frac{C_b A r_o}{S_f} \sqrt{\sigma_{ey} \sigma_t} \tag{C-C3.1.2.1-9}$$

where σ_{ey} and σ_t are the elastic *buckling stresses* as defined in *Specification* Equations C3.1.2.1-8 and C3.1.2.1-9, respectively.

It should be noted that *point-symmetric sections* such as Z-sections with equal *flanges* will buckle laterally at lower strengths than *doubly-* and *singly-symmetric sections*. A conservative design approach has been and is being used in the *Specification*, in which the elastic critical *buckling stress* is taken to be one-half of that for I-members.

Regarding the inelastic critical *buckling stress*, the following equation was used for calculating the critical moment in Section C3.1.2(a) of the 1986 edition of the AISI *Specification* instead of using Equation C-C3.1.2.1-8 for inelastic critical *buckling stress*:

$$(M_{cr})_I = M_y \left[1 - \frac{M_y}{4(M_{cr})_e}\right] \tag{C-C3.1.2.1-10}$$

in which $(M_{cr})_e$ is the elastic critical *buckling* moment. In 1996, the basic inelastic *lateral-torsional buckling* curve for *singly-*, *doubly-*, and *point-symmetric sections* in AISI *Specification* Section C3.1.2.1(a) was redefined to be consistent with the inelastic *lateral-torsional buckling* curve for I- or Z-sections in *Specification* Section C3.1.2.1(b). The general shape of the curve as represented by Equation C-C3.1.2.1-8 is also consistent with the preceding edition of the *Specification* (AISI, 1986).

As specified in *Specification* Section C3.1.2.1, *lateral-torsional buckling* is considered to be elastic up to a *stress* equal to $0.56F_y$. The inelastic region is defined by a Johnson parabola from $0.56F_y$ to $(10/9)F_y$ at an unsupported length of zero. The (10/9) factor is based on the partial plastification of the section in bending (Galambos, 1963). A flat plateau is created by limiting the maximum *stress* to F_y, which enables the calculation of the maximum unsupported length for which there is no *stress* reduction due to lateral-torsional *instability*. This maximum unsupported length can be calculated by setting F_y equal to F_c in Equation C-C3.1.2.1-8.

This liberalization of the inelastic *lateral-torsional buckling* curve for *singly-*, *doubly-*, and *point-symmetric* sections has been confirmed by research in beam-columns (Peköz and Sumer, 1992) and wall studs (Niu and Peköz, 1994).

The elastic and inelastic critical *stresses* for the *lateral-torsional buckling* strength are shown in Figure C-C3.1.2.1-2. For any unbraced length, L, less than L_u, *lateral-torsional buckling* does not need to be considered. L_u is determined by setting $F_e = 2.78F_y$ and $L_u = L_y = L_t$. L_u may then be calculated using the expression given below (AISI, 1996):

(a) for Singly-, Doubly- and Point-Symmetric Sections:

$$L_u = \left\{ \frac{GJ}{2C_1} + \left[\frac{C_2}{C_1} + \left(\frac{GJ}{2C_1} \right)^2 \right]^{0.5} \right\}^{0.5} \quad \text{(C-C3.1.2.1-11)}$$

where

$$C_1 = \frac{7.72}{AE}\left[\frac{K_y F_y S_f}{C_b \pi r_y}\right]^2 \text{ for } \textit{singly- } \text{and } \textit{doubly-symmetric sections} \quad \text{(C-C3.1.2.1-12)}$$

$$C_1 = \frac{30.9}{AE}\left[\frac{K_y F_y S_f}{C_b \pi r_y}\right]^2 \text{ for } \textit{point-symmetric sections} \quad \text{(C-C3.1.2.1-13)}$$

$$C_2 = \frac{\pi^2 E C_w}{(K_t)^2} \quad \text{(C-C3.1.2.1-14)}$$

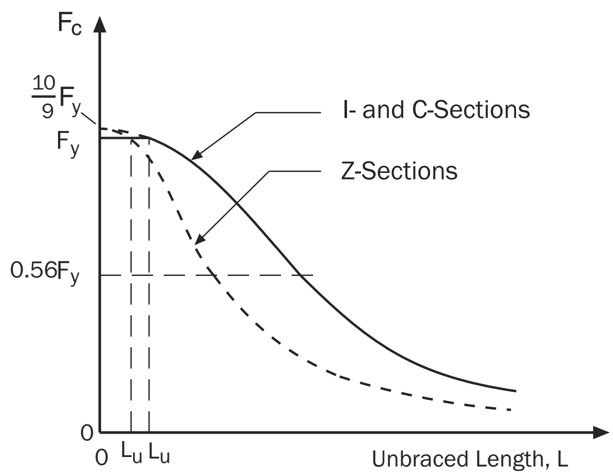


Figure C-C3.1.2.1-2 Lateral-Torsional Buckling Stress

(b) *For I-Sections, Singly-Symmetric C-Sections, or Z-Sections Bent About the Centroidal Axis Perpendicular to the Web*

The following equations may be used in lieu of (a) (AISI, 1996):

For doubly-symmetric I-sections and singly-symmetric C-sections:

$$L_u = \frac{1}{K_y}\left[\frac{0.36C_b\pi^2 EdI_{yc}}{F_y S_f}\right]^{0.5} \quad \text{(C-C3.1.2.1-15)}$$

For point-symmetric Z-sections:

$$L_u = \frac{1}{K_y}\left[\frac{0.18C_b\pi^2 EdI_{yc}}{F_y S_f}\right]^{0.5} \quad \text{(C-C3.1.2.1-16)}$$

For members with unbraced length, $L \le L_u$, or elastic *lateral-torsional buckling stress*, $F_e \ge 2.78F_y$, the flexural strength is determined in accordance with C3.1.1(a).

The above discussion dealt only with the *lateral-torsional buckling* strength of locally stable beams. For locally unstable beams, the interaction of the *local buckling* of the compression elements and overall *lateral-torsional buckling* of members may result in a reduction of the *lateral-torsional buckling* strength of the member. The effect of *local buckling* on the critical moment is considered in Section C3.1.2.1 of the *Specification* by using the elastic section modulus S_c based on an effective section, i.e.,

$$M_n = F_c S_c \quad \text{(C-C3.1.2.1-17)}$$

where

F_c = elastic or inelastic critical *lateral-torsional buckling stress*

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

Using the above *nominal lateral-torsional buckling strength* [*resistance*] with a *resistance factor* of ϕ_b = 0.90, the values of β vary from 2.4 to 3.8 for the *LRFD* method.

The research conducted by Ellifritt, Sputo and Haynes (1992) has indicated that when the unbraced length is defined as the spacing between intermediate braces, the equations used in *Specification* Section C3.1.2.1 may be conservative for cases where one mid-span brace is used, but may be unconservative where more than one intermediate brace is used.

The above mentioned research (Ellifritt, Sputo, and Haynes, 1992) and the study of Kavanagh and Ellifritt (1993 and 1994) have shown that a discretely braced beam, not attached to deck and sheathing, may fail either by *lateral-torsional buckling* between braces, or by *distortional buckling* at or near the braced point. See Section C3.1.4 for commentary on *distortional buckling* strength.

The problems discussed above dealt with the type of *lateral-torsional buckling* of I-members, C-sections, and Z-shaped sections for which the entire cross-section rotates and deflects in the lateral direction as a unit. But this is not the case for U-shaped beams and the combined sheet-stiffener sections as shown in Figure C-C3.1.2.1-3. For this case, when the section is loaded in such a manner that the brims and the *flanges* of stiffeners are in compression, the tension *flange* of the beam remains straight and does not displace laterally; only the compression *flange* tends to buckle separately in the lateral direction, accompanied by out-of-plane bending of the *web*, as shown in Figure C-C3.1.2.1-4, unless

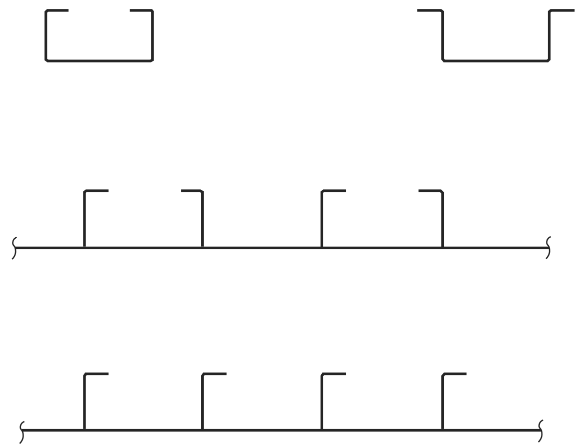


Figure C-C3.1.2.1-3 Combined Sheet-Stiffener Sections

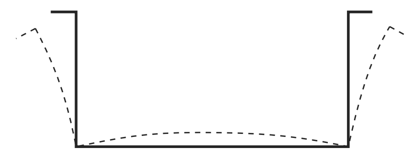


Figure C-C3.1.2.1-4 Lateral Buckling of U-Shaped Beam

adequate bracing is provided.

To analyze the lateral *buckling* of U-shaped beams, the design procedure outlined in Section 2 of Part V (Supplementary Information) of the AISI *Cold-Formed Steel Design Manual* (AISI, 2002) was used for determining the nominal *stress* for laterally unbraced compression *flanges*. This procedure is based on the considerable simplification of an analysis presented by Douty (1962).

In 1964, Haussler presented rigorous methods for determining the strength of elastically stabilized beams (Haussler, 1964). In his methods, Haussler also treated the unbraced compression *flange* as a column on an elastic foundation and maintained more rigor in his development.

A comparison of Haussler's method with Douty's simplified method indicates that the latter may provide a lower value of critical *stress*.

An additional study of laterally unbraced compression *flanges* has been made at Cornell University (Serrette and Peköz, 1992, 1994 and 1995). An analytical procedure has been developed for determining the *distortional buckling* strength of the standing seam roof panel. The predicted maximum capacities have been compared with experimental results.

With the introduction of distortional *buckling* analysis and design (AISI, 2004 and AISI, 2007), the above described *buckling* mode of U-shaped beams is classified as the *distortional buckling*, which can be analyzed in accordance with provisions provided in *Specification* Section C3.1.4 or Appendix 1.

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

Due to the high torsional stiffness of closed box sections, *lateral-torsional buckling* is not critical in typical design considerations, even for bending about the major axis.

Deflection limits will control most designs due to the large values of L_u. However, *lateral-torsional buckling* can control the design when the unbraced length is larger than L_u, which is determined by setting the inelastic *buckling stress* of *Specification* Equation C3.1.2.1-2 equal to F_y, with F_e set equal to *Specification* Equation C3.1.2.2-2.

In computing the *lateral-torsional buckling stress* of closed box sections, the warping constant, C_w, may be neglected since the effect of non-uniform warping of box sections is small. The development of *Specification* Equation C3.1.2.2-2 can be found in the *Guide to Stability Design Criteria for Metal Structures* (Galambos, 1998). As a result of adding Section C3.1.2.2 to the *Specification*, Section D3.3 of the 1996 edition of the *Specification* has been deleted.

The Saint-Venant torsional constant, J, of a box section, neglecting the corner radii, may be conservatively determined as follows:

$$J = \frac{2(ab)^2}{(a/t_1)+(b/t_2)} \tag{C-C3.1.2.2-1}$$

where

a = distance between *web* centerlines

b = distance between *flange* centerlines

t_1 = *thickness* of *flanges*

t_2 = *thickness* of *webs*

In 2001, the unbraced length, L, in *Specification* Equation C3.1.2.2-2 was replaced with K_yL_y, where K_y is the effective length factor for bending about the y-axis. The K_y factor provides for other than simply supported end conditions. Detailed discussions are provided in Section C3.1.2.1 of the *Commentary*.

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

The discussion on cylindrical tubular member behavior and *buckling* modes is provided in *Commentary* Section C4.1.5. It should be noted that the design provisions of *Specification* Sections C3.1.3 and C4.1.5 are applicable only for members having a ratio of outside diameter-to-wall *thickness*, D/t, not greater than $0.441E/F_y$ because the design of extremely thin tubes will be governed by elastic *local buckling* resulting in an uneconomical design. In addition, cylindrical tubular members with unusually large D/t ratios are very sensitive to geometric imperfections.

For thick cylinders in bending, the initiation of yielding does not represent a failure condition as is generally assumed for axial loading. Failure is at the plastic moment capacity, which is at least 1.29 times the moment at first yielding. In addition, the conditions for inelastic *local buckling* are not as severe as in axial compression due to the *stress* gradient.

Specification Equations C3.1.3-2, C3.1.3-3 and C3.1.3-4 are based upon the work reported by Sherman (1985) and an assumed minimum shape factor of 1.25. This slight reduction in the inelastic range has been made to limit the maximum bending *stress* to $0.75F_y$, a value typically used for solid sections in bending for the *ASD* method. The reduction also brings the criteria closer to a lower bound for inelastic *local buckling*. A small range of elastic *local buckling* has been included so that the upper D/t limit of $0.441E/F_y$ is the same as for axial compression.

All three equations for determining the *nominal flexural strength* [*resistance*] of closed cylindrical tubular members are shown in Figure C-C3.1.3-1. These equations have been used in the AISI *Specification* since 1986 and are retained in this edition. In 1999, the limiting D/t ratios for *Specification* Equations C3.1.3-2 and C3.1.3-3 were revised to provide an appropriate continuity. The *safety factor* Ω_b and the *resistance factor* ϕ_b are the same as that used in *Specification* Section C3.1.1 for sectional bending strength.

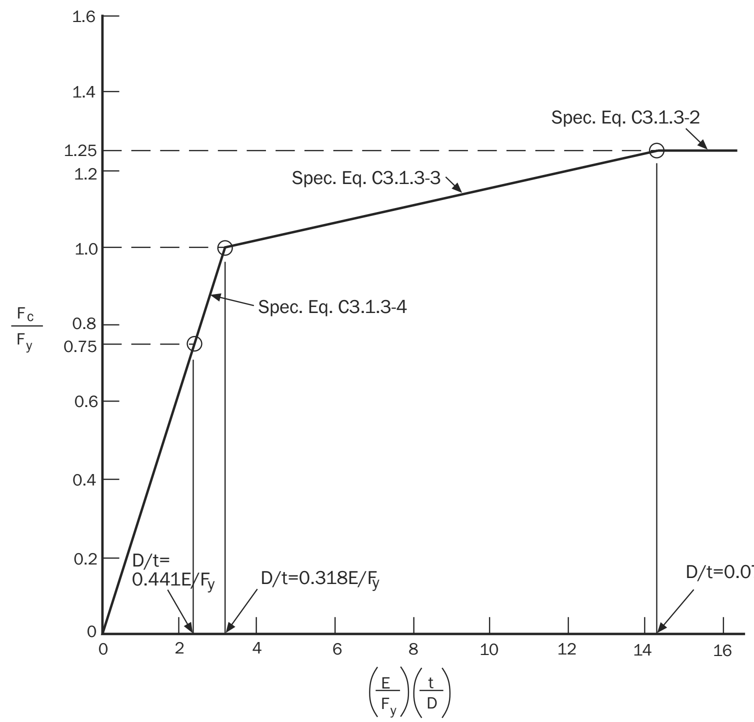


Figure C-C3.1.3-1 Nominal Flexural Strength of Cylindrical Tubular Members

C3.1.4 Distortional Buckling Strength [Resistance]

Distortional buckling is an *instability* that may occur in members with edge-stiffened *flanges*, such as lipped C- and Z-sections. As shown in Figure C-C3.1.4-1, this *buckling* mode is characterized by *instability* of the entire *flange*, as the *flange* along with the edge stiffener rotates about the junction of the compression *flange* and the *web*. The length of the *buckling* wave in *distortional buckling* is considerably longer than *local buckling*, and noticeably shorter than *lateral-torsional buckling*. The *Specification* provisions of Section B4 partially account for *distortional buckling*, but research has shown that a separate limit state check is required (Ellifritt, Sputo, and Haynes, 1992; Hancock, Rogers, and Schuster, 1996; Kavanagh and Ellifritt, 1994; Schafer and Peköz, 1999; Hancock, 1997; Yu and Schafer, 2003 and 2006). Thus, in 2007, *Specification* Section C3.1.4 was added to address *distortional buckling* as a separate limit state.

Determination of the *nominal strength* [*resistance*] in *distortional buckling* (*Specification* Equation C3.1.4-2) was validated by testing. Results of one such study (Yu and Schafer, 2006) are shown in Figure C-C3.1.4-2. The *Direct Strength Method* of Appendix 1 of the *Specification* also uses Equation C3.1.4-2. In addition, the Australian/New Zealand

Specification (AS/NZS 4600) has used Equation C3.1.4-2 since 1996. Calibration of the *safety* and *resistance factors* for *Specification* Equation C3.1.4-2 is provided in the commentary to Appendix 1.

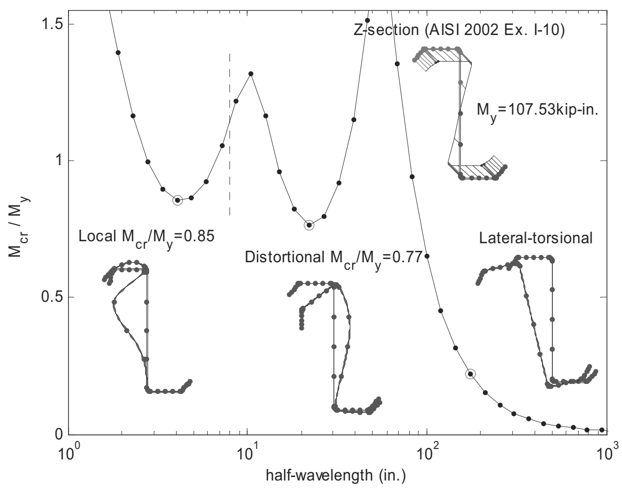


Figure C-C3.1.4-1 Rational Elastic Buckling Analysis of a Z-Section Under Restrained Bending Showing Local, Distortional, and Lateral-Torsional Buckling Modes

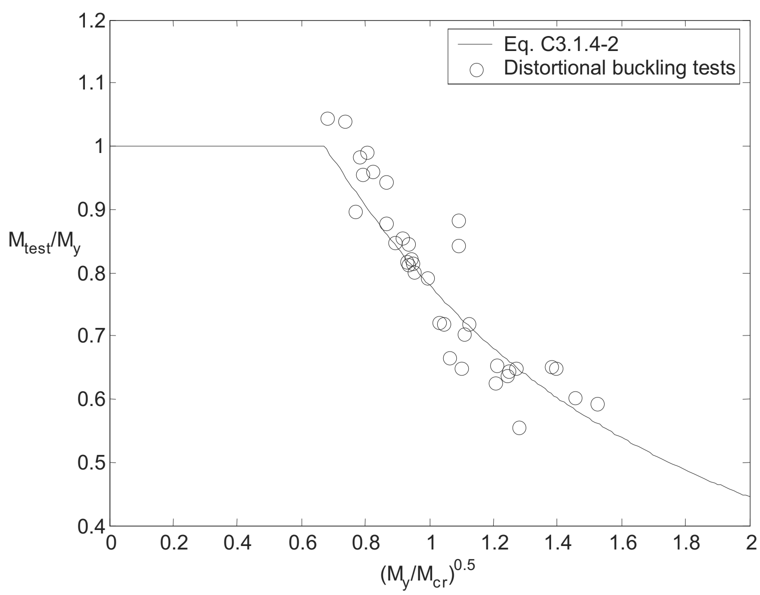


Figure C-C3.1.4-2 Performance of Distortional Buckling Prediction With Test Data on Common C- and Z-Sections in Bending (Yu and Schafer 2006)

Distortional buckling is unlikely to control the strength if: (a) edge stiffeners are sufficiently stiff and thus stabilize the *flange* (as is often the case for C-sections, but typically not for Z-sections due to the use of sloping lips), (b) unbraced lengths are long and *lateral-torsional buckling* strength limits the capacity, or (c) adequate rotational restraint is provided to the compression *flange* from attachments (panels, sheathing, etc.).

The primary difficulty in calculating the strength in *distortional buckling* is to efficiently estimate the elastic *distortional buckling stress*, F_d. Recognizing the complexity of this calculation, this *Specification* section provides two alternatives: C3.1.4(a) provides a comprehensive method for C- and Z-section members and any open section with a single *web* and single edge-stiffened compression *flange*, and C3.1.4(b) offers the option to use rational elastic *buckling* analysis, e.g., see the Appendix 1 commentary. In 2010, the *Simplified Provision for Unrestrained C- and Z-Sections With Simple Lip Stiffeners* was moved from the *Specification* to the *Commentary*. This provision provides a conservative approximation to the *distortional buckling* length, L_{cr}, and *stress*, F_d, for C- and Z-sections with simple lip stiffeners bent about an axis perpendicular to the *web*. The provision ignores any rotational restraint, which would restrain *distortional buckling*. The expressions were specifically derived as a conservative simplification to those provided in *Specification* Sections C3.1.4(a) and C3.1.4(b). For many common sections, the simplified method may be used to show that *distortional buckling* of the column will not control the capacity. *Specification* provisions C3.1.4(a) or C3.1.4 (b), however, should be used to obtain the *distortional buckling* strength if *distortional buckling* controls the design.

Simplified Method 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, Equation C-C3.1.4-1 can be used to calculate a conservative prediction of the *distortional buckling stress*, F_d. See *Specificatio*n Section C3.1.4(a) or C3.1.4(b) for alternative provisions and for members outside the dimensional limits.

The following dimensional limits should apply:

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

(2) $25 \le b_o/t \le 100$,

(3) $6.25 < D/t \le 50$,

(4) $45° \le \theta < 90°$,

(5) $2 \le h_o/b_o \le 8$, and

(6) $0.04 \le D \sin\theta/b_o \le 0.5$.

where

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

t = Base steel *thickness*

b_o = Out-to-out *flange* width as defined in *Specification* Figure B2.3-2

D = Out-to-out lip dimension as defined in *Specification* Figure B4-1

θ = Lip angle as defined in *Specification* Figure B4-1

The *distortional buckling stress*, F_d, can be calculated as follows:

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

where

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

$$= 1.0 \le 1 + 0.4(L/L_m)^{0.7}(1 + M_1/M_2)^{0.7} \le 1.3 \qquad \text{(C-C3.1.4-2)}$$

where

L = minimum of L_{cr} and L_m

where

$$L_{cr} = 1.2 h_o \left(\frac{b_o D \sin\theta}{h_o t} \right)^{0.6} \leq 10 h_o \qquad \text{(C-C3.1.4-3)}$$

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

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

$$k_d = 0.5 \leq 0.6 \left(\frac{b_o D \sin\theta}{h_o t} \right)^{0.7} \leq 8.0 \qquad \text{(C-C3.1.4-4)}$$

E = modulus of elasticity of steel

μ = Poisson's ratio of steel

Equations C-C3.1.4-1 to C-C3.1.4-4 assume the compression *flange* is unrestrained; however, the *Specification* methods of C3.1.4(a) and (b) allow for a rotational restraint, k_ϕ, to be included to account for attachments which restrict *flange* rotation.

k_ϕ Determination

While it is always conservative to ignore the rotational restraint, k_ϕ, in many cases it may be beneficial to include this effect. Due to the large variety of possible conditions, no specific method is provided for determining the rotational restraint. Instead, per Section A1.2 of the *Specification,* k_ϕ may be estimated by testing or *rational engineering analysis*. Test determination of k_ϕ may use AISI S901 (AISI, 2008). K from this method is a lower bound estimate of k_ϕ. The member lateral deformation may be removed from the measured lateral deformation to provide a more accurate estimate of k_ϕ.

Testing on 8 in. and 9.5 in. (203 and 241 mm) deep Z-sections with a *thickness* between 0.069 in. (1.75 mm) and 0.118 in. (3.00 mm), through-fastened 12 in. (205 mm) o.c., to a 36 in. (914 mm) wide, 1 in. (25.4 mm) and 1.5 in. (38.1 mm) high steel panels, with up to 6 in. (152 mm) of blanket insulation between the panel and the Z-section, results in a k_ϕ between 0.15 to 0.44 kip-in./rad./in. (0.667 to 1.96 kN-mm/rad./mm) (MRI 1981).

Additional testing on C- and Z-sections with pairs of through-fasteners provides considerably higher rotational stiffness: for 6 and 8 in. (152 and 203 mm) deep C-sections with a *thickness* between 0.054 and 0.097 in. (1.27 and 2.46 mm), fastened with pairs of fasteners on each side of a 1.25 in. (31.8 mm) high steel panel flute at 12 in. (305 mm) o.c., k_ϕ is 0.4 kip-in./rad./in. (1.78 kN-mm/rad./mm); and for 8.5 in. (216 mm) deep Z-sections with a *thickness* between 0.070 in. and 0.120 in. (1.78 mm to 3.05 mm), fastened with pairs of fasteners on each side of 1.25 in. (31.8 mm) high steel panel flute at 12 in. (305 mm) o.c., k_ϕ is 0.8 kip-in./rad./in. (3.56 kN-mm/rad./mm) (Yu and Schafer 2003, Yu 2005).

Examples of *rational engineering analysis* to estimate the rotational stiffness are provided in the *Direct Strength Method Design Guide* (AISI 2006). For a flexural member, k_ϕ can be approximated as:

$$k_\phi \approx EI/(W/2) \qquad \text{(C-C3.1.4-5)}$$

where E is the modulus of the attached material, I is the moment of inertia of the engaged attachment, and W is the member spacing. The primary complication in such a method is determining how much of the attachment (decking, sheathing, etc.) is engaged when the *flange* attempts to deform. For the Z-sections tested in Yu (2005), experimental k_ϕ is 0.8 kip-in./rad./in. (3.56 kN-mm/rad./mm). Using an estimate of EI/(W/2), the rational engineering values are k_ϕ of 9 kip-in./rad/in. (40.0 kN-mm/rad./mm) if the entire panel, flutes and all, are engaged; k_ϕ of 1.2 kip-in./rad/in. (5.34 kN-mm/rad./mm) if only the corrugated bottom panel, but not the flutes, is engaged; and k_ϕ of 0.003 kip-in./rad./in. (0.0133 kN-mm/rad./mm) if plate bending of the t = 0.019 in. (0.483 mm) panel occurs. The observed panel engagement is between the last two estimates, and assuming the corrugated bottom pan, but not the 1.25 in. (31.8 mm) high flutes is engaged is reasonable.

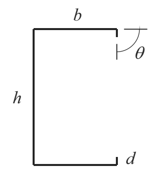
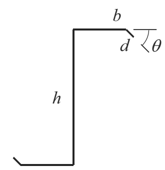
For members with wood sheathing attached, little experimental information is available. The problem has been studied numerically using the same paired fastener detail as in Yu (2005) and Yu and Schafer (2003) tests, but replacing the steel panel with a simulated wood member, *thickness* = 0.5 in. (12.7 mm), E = 1000 ksi (6900 MPa), and μ = 0.3. The calculated k_ϕ is 5.1 kip-in./rad./in. (22.7 kN-mm/rad./mm) for 6 and 8 in. (152 to 203 mm) deep C-sections with a *thickness* between 0.054 and 0.097 in. (1.37 and 2.46 mm); and k_ϕ is 4.1 kip-in./rad./in. (18.2 kN-mm/rad./mm) for 8.5 in. (216 mm) deep Z-sections with *thickness* between 0.070 and 0.120 in. (1.78 mm and 3.05 mm). From calculations assuming a fully engaged 1/2 in. (12.7 mm) thick wood sheet on top of C- or Z-section members spaced 12 in. (305 mm) apart, k_ϕ is predicted to be 1.7 kip-in./rad./in. (7.56 kN-mm/rad./mm). Thus, use of EI/(W/2) provides a reasonably conservative approximation, with I calculated assuming the full engagement of wood sheet.

The presence of moment gradient can also increase the *distortional buckling* moment (or equivalently *stress*, F_d). However, this increase is lessened if the moment gradient occurs over a longer length. Thus, in determining the influence of moment gradient, β, the ratio of the end moments, M_1/M_2, and the ratio of the critical *distortional buckling* length to the unbraced length, L/L_m, should both be accounted for. In 2010, the sign convention on the ratio of moments M_1 and M_2 was changed to be consistent with moment gradient expressions for C_{TF} (*Specification* Equation C3.1.2.1-12) and C_m (*Specification* Equation C5.2.1-8) used elsewhere in the *Specification*. *Specification* Equation C3.1.4-7 and *Commentary* Equation C-C3.1.4-2 were revised accordingly. Yu (2005) performed elastic *buckling* analysis with shell finite element models of C- and Z-sections under different moment gradients to examine this problem. Significant scatter exists in the results; therefore, a lower bound prediction (*Specification* Equation C3.1.4-7) for the increase was selected.

(a) 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 *Specification* Section C3.1.4(a) provide a general method for calculation of the *distortional buckling stress*, F_d, for any open section with an edge-stiffened compression *flange*, including complex edge stiffeners. The provisions of *Specification* Section C3.1.4(a) also provide a more refined answer for any C- and Z-section, including those meeting the dimensional criteria of the *Simplified Provision for Unrestrained C- and Z-Sections With Simple Lip Stiffeners* presented in this *Commentary*. The expressions employed here are derived in Schafer and Peköz (1999) and verified for complex stiffeners in Schafer et al. (2006). The equations used for the *distortional buckling stress*, F_d, in AS/NZS 4600 are also similar to those in *Specification* Section C3.1.4 (a), except that when the *web* is very slender and is restrained by the *flange*, AS/NZS 4600 uses a simpler, conservative treatment. Since the provided expressions can be complicated, solutions for the geometric properties of C- and Z-sections based on centerline dimensions are provided in Table C-C3.1.4(a)-1.

Table C-C3.1.4(a)-1
Geometric Flange Properties for C- and Z-Sections

	
$A_f = (b+d)t$	$A_f = (b+d)t$
$J_f = \frac{1}{3}bt^3 + \frac{1}{3}dt^3$	$J_f = \frac{1}{3}bt^3 + \frac{1}{3}dt^3$
$I_{xf} = \dfrac{t\left(t^2b^2 + 4bd^3 + t^2bd + d^4\right)}{12(b+d)}$	$I_{xf} = \dfrac{t\left(t^2b^2 + 4bd^3 - 4bd^3\cos^2(\theta) + t^2bd + d^4 - d^4\cos^2(\theta)\right)}{12(b+d)}$
$I_{yf} = \dfrac{t\left(b^4 + 4db^3\right)}{12(b+d)}$	$I_{yf} = \dfrac{t\left(b^4 + 4db^3 + 6d^2b^2\cos(\theta) + 4d^3b\cos^2(\theta) + d^4\cos^2(\theta)\right)}{12(b+d)}$
$I_{xyf} = \dfrac{tb^2d^2}{4(b+d)}$	$I_{xyf} = \dfrac{tbd^2\sin(\theta)(b + d\cos(\theta))}{4(b+d)}$
$C_{wf} = 0$	$C_{wf} = 0$
$x_{of} = \dfrac{b^2}{2(b+d)}$	$x_{of} = \dfrac{b^2 - d^2\cos(\theta)}{2(b+d)}$
$h_{xf} = \dfrac{-\left(b^2 + 2db\right)}{2(b+d)}$	$h_{xf} = \dfrac{-\left(b^2 + 2db + d^2\cos(\theta)\right)}{2(b+d)}$
$h_{yf} = y_o = \dfrac{-d^2}{2(b+d)}$	$h_{yf} = y_{of} = \dfrac{-d^2\sin(\theta)}{2(b+d)}$

Note: Other variables in Table C-C3.1.4(a)-1 are defined in *Specification* Section C3.1.4.

(b) Rational Elastic Buckling Analysis

Rational elastic *buckling* analysis consists of any method following the principles of mechanics to arrive at an accurate prediction of the elastic *distortional buckling stress* (moment). It is important to note that this is a rational elastic *buckling* analysis and not simply an arbitrary rational method to determine ultimate strength. A variety of rational computational and analytical methods can provide the elastic *buckling* moment with a high degree of accuracy. Complete details are provided in Section 1.1.2 of the commentary to Appendix 1 of the *Specification*. The *safety* and *resistance factors* of this section have been shown to apply to a wide variety of cross-sections undergoing *distortional buckling* (via the methods of Appendix 1). As long as the member falls within the geometric limits of main *Specification* Section B1.1, the same *safety* and *resistance factors* have been assumed to apply. Application of the β expression to account for moment gradient, as provided in *Specification* Section C3.1.4(a), is a rational extension to solutions which do not typically account for moment gradient such as the finite strip method.

C3.2 Shear

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

The shear strength of beam *webs* is governed by either yielding or *buckling*, depending on the h/t ratio and the mechanical properties of steel. For beam *webs* having small h/t ratios, the *nominal shear strength* [*resistance*] is governed by shear yielding, i.e.,

$$V_n = A_w \tau_y = A_w F_y / \sqrt{3} \approx 0.60 F_y ht \quad \text{(C-C3.2.1-1)}$$

in which A_w is the area of the beam *web* computed by (ht), and τ_y is the *yield stress* of steel in shear, which can be computed by $F_y / \sqrt{3}$.

For beam *webs* having large h/t ratios, the *nominal shear strength* [*resistance*] is governed by elastic *shear buckling* (Yu and LaBoube, 2010), i.e.,

$$V_n = A_w \tau_{cr} = \frac{k_v \pi^2 E A_w}{12(1-\mu^2)(h/t)^2} \quad \text{(C-C3.2.1-2)}$$

in which τ_{cr} is the critical *shear buckling stress* in the elastic range, k_v is the *shear buckling* coefficient, E is the modulus of elasticity, μ is the Poisson's ratio, h is the *web* depth, and t is the *web thickness*. By using μ = 0.3, the *nominal shear strength* [*resistance*], V_n, can be determined as follows:

$$V_n = 0.904 E k_v t^3 / h \quad \text{(C-C3.2.1-3)}$$

For beam *webs* having moderate h/t ratios, the *nominal shear strength* [*resistance*] is based on inelastic *shear buckling* (Yu and LaBoube, 2010), i.e.,

$$V_n = 0.64 t^2 \sqrt{k_v F_y E} \quad \text{(C-C3.2.1-4)}$$

The *Specification* provisions are applicable for the design of *webs* of beams and decks either with or without transverse *web* stiffeners.

The *nominal strength* [*resistance*] equations of Section C3.2.1 of the *Specification* are similar to the *nominal shear strength* [*resistance*] equations given in the AISI *LRFD Specification* (AISI, 1991). The acceptance of these *nominal strength* [*resistance*] equations for cold-formed steel sections has been considered in the study summarized by LaBoube and Yu (1978a).

Previous editions of the AISI *ASD Specification* (AISI, 1986) used three different *safety factors* when evaluating the *allowable shear strength* of an unreinforced *web* because it was intended to use the same *nominal strength* [*resistance*] equations for the AISI and AISC Specifications. To simplify the design of shear using only one *safety factor* for *ASD* and one *resistance factor* for *LRFD*, Craig (Craig, 1999) carried out a calibration using the data by LaBoube and Yu (LaBoube, 1978a). Based on this work, the constant used in *Specification* Equation C3.2.1-3 was reduced from 0.64 to 0.60. In addition, the *ASD safety factor* for yielding, elastic and inelastic *buckling* is now taken as 1.60, with a corresponding *resistance factor* of 0.95 for *LRFD* and 0.80 for *LSD*.

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

For C-section *webs* with holes, Schuster, et al. (1995) and Shan, et al. (1994) investigated the degradation in *web* shear strength due to the presence of a *web* perforation. The test program considered a constant shear distribution across the perforation, and included

d_0/h ratios ranging from 0.20 to 0.78, and h/t ratios of 91 to 168. Schuster's q_s equation was developed with due consideration for the potential range of both punched and field-cut holes. Three hole geometries–rectangular with corner fillets, circular, and diamond–were considered in the test program. Eiler (1997) extended the work of Schuster and Shan for the case of constant shear along the longitudinal axis of the perforation. He also studied linearly varying shear, but this case is not included in the *Specification*. The development of Eiler's reduction factor, q_s, utilized the test data of both Schuster et al. (1995) and Shan et al. (1994). The focus of the test programs was on the behavior of slender *webs* with holes. Thus, for stocky *web* elements with $h/t \le 0.96\sqrt{Ek_v/F_y}$, an anomaly exists; the calculated *available shear strength* [*factored resistance*] is independent of t when h is constant. In this region, the calculated *available shear strength* [*factored resistance*] is valid but may be somewhat conservative.

The provisions for circular and non-circular holes also apply to any hole pattern that fits within an equivalent virtual hole. Figure C-B2.4-1 illustrates the L_h and d_h that may be used for a multiple hole pattern that fits within a non-circular virtual hole. Figure C-B2.4-2 illustrates the d_h that may be used for a rectangular hole that fits within a circular virtual hole. For each case, the design provisions apply to the geometry of the virtual hole geometry, not to the actual hole or holes.

C3.3 Combined Bending and Shear

For cantilever beams and continuous beams, high bending *stresses* often combine with high shear *stresses* at the supports. Such beam *webs* must be safeguarded against *buckling* due to the combination of bending and shear *stresses*.

For disjointed flat rectangular plates, the critical combination of bending and shear *stresses* can be approximated by the following interaction equation (Bleich, 1952), which is part of a unit circle:

$$\left(\frac{f_b}{f_{cr}}\right)^2 + \left(\frac{\tau}{\tau_{cr}}\right)^2 = 1.0 \qquad \text{(C-C3.3-1)}$$

or

$$\sqrt{\left(\frac{f_b}{f_{cr}}\right)^2 + \left(\frac{\tau}{\tau_{cr}}\right)^2} = 1.0 \qquad \text{(C-C3.3-2)}$$

where f_b is the actual compressive bending *stress*, f_{cr} is the theoretical *buckling stress* in pure bending, τ is the actual shear *stress*, and τ_{cr} is the theoretical *buckling stress* in pure shear. The above equation was found to be conservative for beam *webs* with adequate shear stiffeners, for which a diagonal tension field action may be developed. Based on the studies made by LaBoube and Yu (1978b), Equation C-C3.3-3 was developed for beam *webs* with shear stiffeners satisfying the requirements of *Specification* Section C3.7.3.

$$0.6\frac{f_b}{f_{b_{max}}} + \frac{\tau}{\tau_{max}} = 1.3 \qquad \text{(C-C3.3-3)}$$

Equation C-C3.3-3 was added to the AISI *Specification* in 1980. The correlations between Equation C-C3.3-3 and the test results of beam *webs* having a diagonal tension field action are shown in Figure C-C3.3-1.

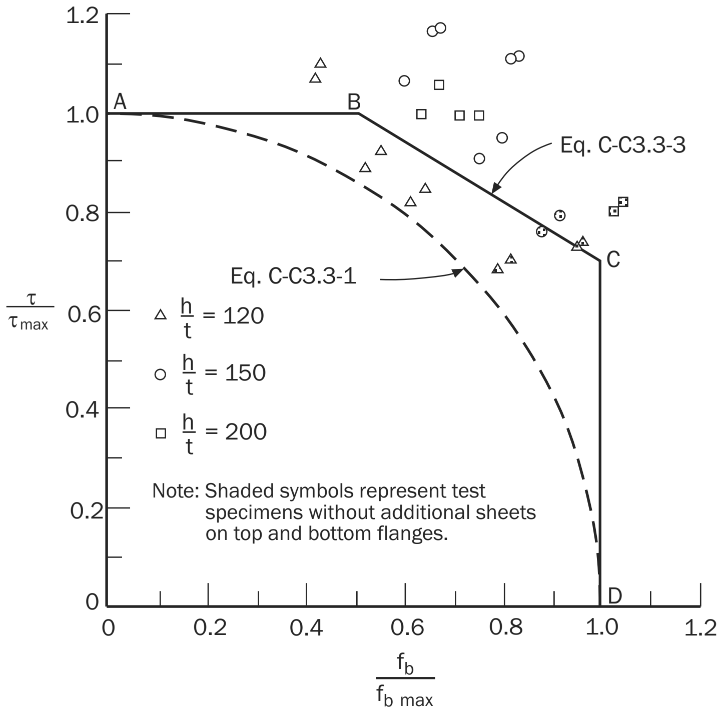


Figure C-C3.3-1 Interaction Diagram for τ/τ_{max} and f_b/f_{bmax}

C3.3.1 ASD Method

Since 1986, the AISI *ASD Specification* uses strength ratios (i.e., moment ratio for bending and force ratio for shear) instead of *stress* ratios for the interaction equations. *Specification* Equations C3.3.1-1 and C3.3.1-2 are based on Equations C-C3.3-2 and C-C3.3-3, respectively, by using the allowable moment, M_{nxo}/Ω_b, and the allowable shear force, V_n/Ω_v.

C3.3.2 LRFD and LSD Methods

For the *Load and Resistance Factor Design* and the *Limit States Design*, the interaction equations for combined bending and shear are also based on Equations C-C3.3-2 and C-C3.3-3 as given in *Specification* Equations C3.3.2-1 and C3.3.2-2 by using the *required strength* [forces and moments due to *factored loads*] and *design strength* [*factored resistance*]. In both equations, different symbols are used for the *required flexural strength* [moment due to *factored loads*] and the *required shear strength* [shear force due to *factored loads*] according to the *LRFD* and the *LSD* methods.

C3.4 Web Crippling

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

Since cold-formed steel flexural members generally have large *web* slenderness ratios, the *webs* of such members may cripple due to the high local intensity of the *load* or reaction. Figure C-C3.4.1-1 shows typical *web crippling* failure modes of unreinforced single hat sections (Figure C-C3.4.1-1(a)) and of I-sections (Figure C-C3.4.1-1(b)) unfastened to the support.

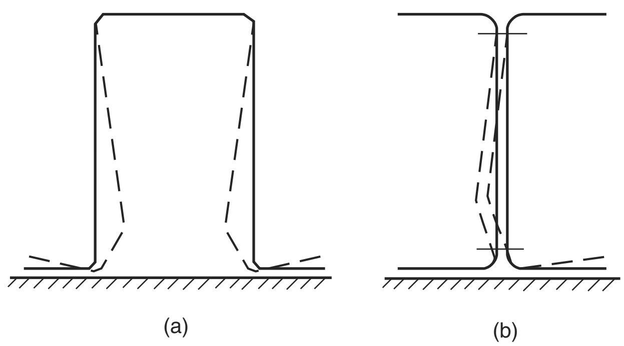


Figure C-C3.4.1-1 Web Crippling of Cold-Formed Steel Sections

In the past, the *buckling* problem of plates and the *web crippling* behavior of cold-formed steel members under locally distributed edge loading have been studied by numerous investigators (Yu and LaBoube, 2010). It has been found that the theoretical analysis of *web crippling* for cold-formed steel flexural members is rather complicated because it involves the following factors: (1) nonuniform *stress* distribution under the applied *load* and adjacent portions of the *web*, (2) elastic and inelastic stability of the *web* element, (3) *local yielding* in the immediate region of *load* application, (4) bending produced by eccentric *load* (or reaction) when it is applied on the bearing *flange* at a distance beyond the curved transition of the *web*, (5) initial out-of-plane imperfection of plate elements, (6) various edge restraints provided by beam *flanges* and interaction between *flange* and *web* elements, and (7) inclined *webs* for decks and panels.

For these reasons, the present AISI design provision for *web crippling* is based on the extensive experimental investigations conducted at Cornell University by Winter and Pian (1946) and Zetlin (1955a); at the University of Missouri-Rolla by Hetrakul and Yu (1978 and 1979), Yu (1981), Santaputra (1986), Santaputra, Parks and Yu (1989), Bhakta, LaBoube and Yu (1992), Langan, Yu and LaBoube (1994), Cain, LaBoube and Yu (1995) and Wu, Yu and LaBoube (1997); at the University of Waterloo by Wing (1981), Wing and Schuster (1982), Prabakaran (1993), Gerges (1997), Gerges and Schuster (1998), Prabakaran and Schuster (1998), Beshara (1999), and Beshara and Schuster (2000 and 2000a); and at the University of Sydney by Young and Hancock (1998). In these experimental investigations, the *web crippling* tests were carried out under the following four loading conditions for beams having single unreinforced *webs* and I-beams, single hat sections and multi-*web* deck sections:

1. End one-*flange* (EOF) loading
2. Interior one-*flange* (IOF) loading
3. End two-*flange* (ETF) loading

4. Interior two-*flange* (ITF) loading

All loading conditions are illustrated in Figure C-C3.4.1-2. In Figures (a) and (b), the distances between bearing plates were kept to no less than 1.5 times the *web* depth in order to avoid the two-*flange* loading action. Application of the various *load* cases is shown in Figure C-C3.4.1-3 and the assumed reaction or *load* distributions are illustrated in Figure C-C3.4.1-4.

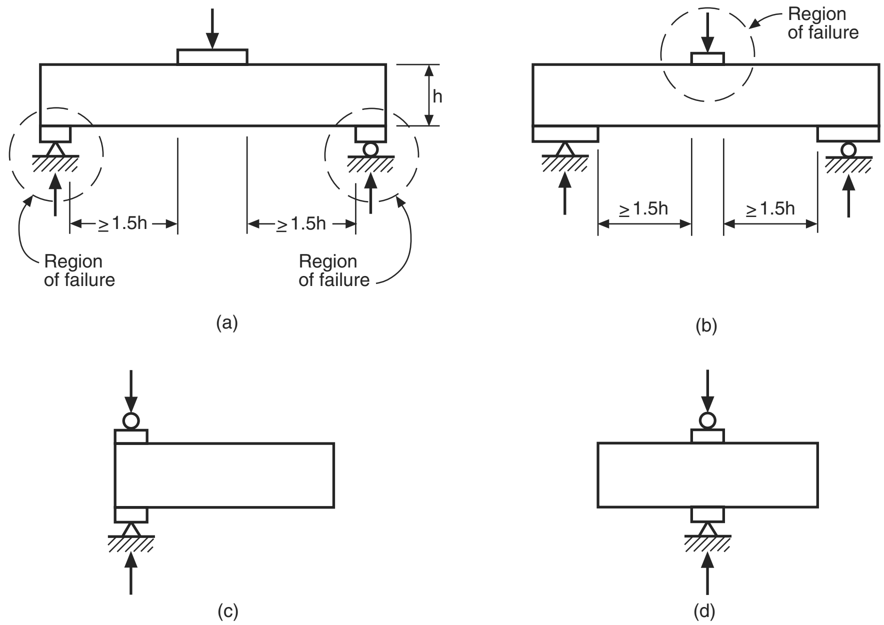


Figure C-C3.4.1-2 Loading Conditions for Web Crippling Tests:
(a) EOF Loading, (b) IOF Loading, (c) ETF Loading, (d) ITF Loading

In the 1996 edition of the AISI *Specification*, and in previous editions, different *web crippling* equations were used for the various loading conditions stated above. These equations were based on experimental evidence (Winter, 1970; Hetrakul and Yu, 1978) and the assumed distributions of *loads* or reactions acting on the *web* as shown in Figure C-C3.4.1-4. The equations were also based on the type of section geometry, i.e., shapes having single *webs* and I-sections (made of two channels connected back-to-back, by welding two angles to a channel, or by connecting three channels). C-and Z-sections, single hat sections and multi-*web* deck sections were considered in the single *web* member category. I-sections made of two channels connected back-to-back by a line of connectors near each *flange* or similar sections that provide a high degree of restraint against rotation of the *web* were treated separately. In addition, different equations were used for sections with stiffened or partially stiffened *flanges* and sections with unstiffened *flanges*.

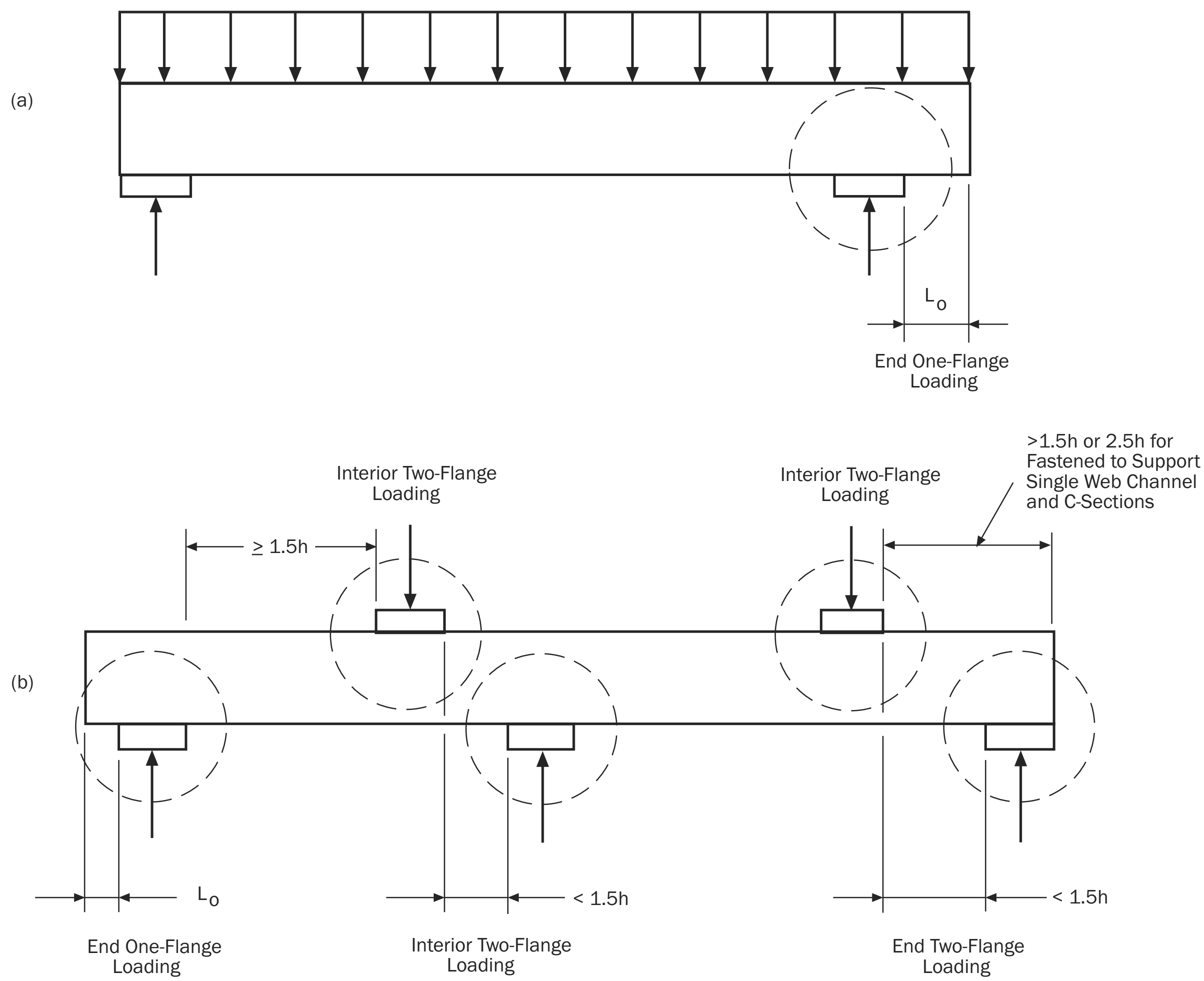


Figure C-C3.4.1-3 Application of Loading Cases

Prabakaran (1993) and Prabakaran and Schuster (1998) developed one consistent unified *web crippling* equation with variable coefficients (*Specification* Equation C3.4.1-1). These coefficients accommodate one- or two-*flange* loading for both end and interior loading conditions of various section geometries. Beshara (1999) extended the work of Prabakaran and Schuster (1998) by developing new *web crippling* coefficients using the available data as summarized by Beshara and Schuster (2000). The *web crippling* coefficients are summarized in Tables C3.4.1-1 to C3.4.1-5 of the *Specification* and the parametric limitations given are based on the experimental data that was used in the development of the *web crippling* coefficients. From *Specification* Equation C3.4.1-1, it can be seen that the *nominal web crippling strength* [*resistance*] of cold-formed steel members depends on an overall *web crippling* coefficient, C; the *web thickness*, t; the *yield stress*, F_y; the *web* inclination angle, θ; the inside bend radius coefficient, C_R; the inside bend radius ratio, R/t; the bearing length coefficient, C_N; the bearing length ratio, N/t; the *web* slenderness coefficient, C_h; and the *web* slenderness ratio, h/t.

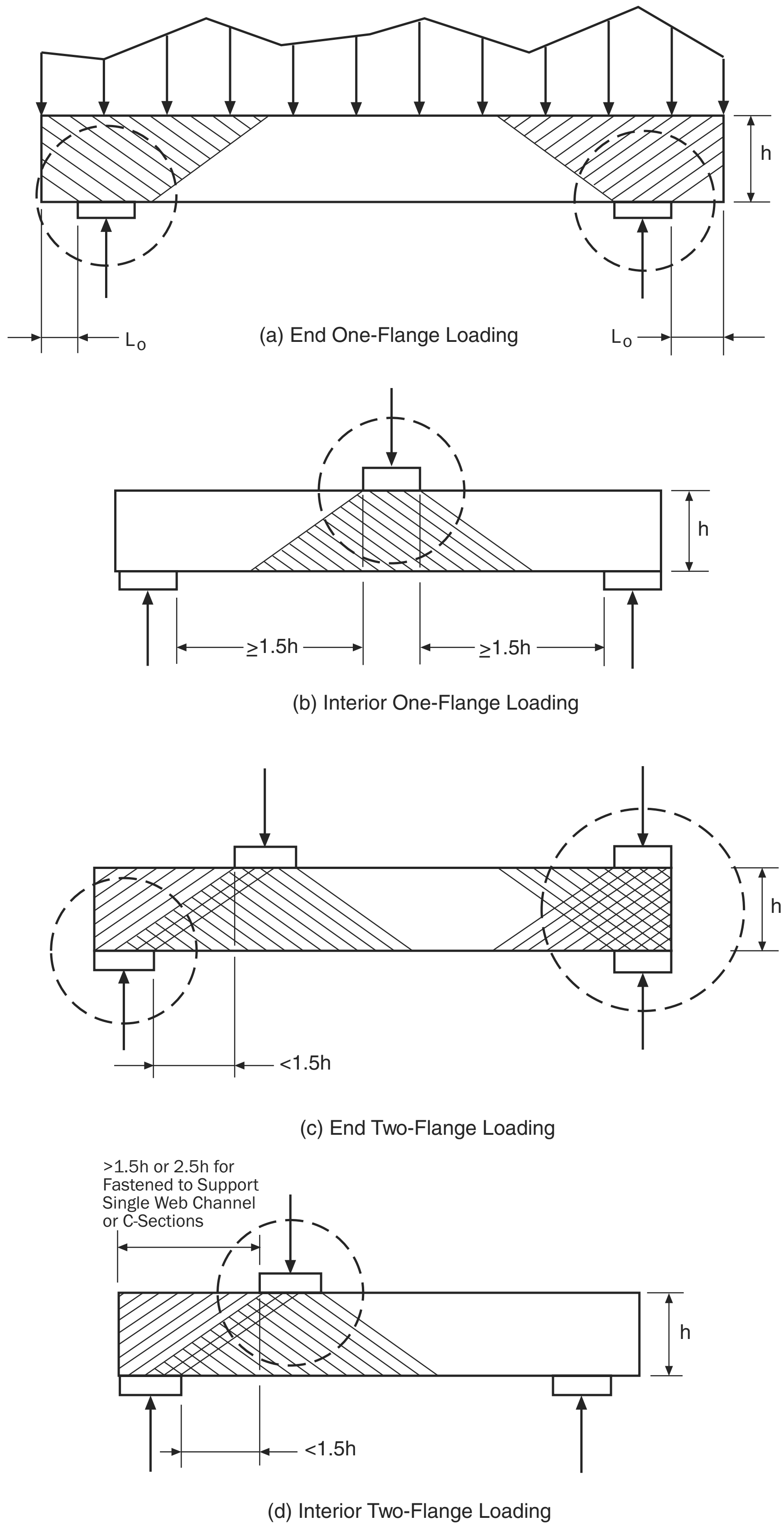


Figure C-C3.4.1-4 Assumed Distribution of Reaction or Load

This new equation is presented in a normalized format and is non-dimensional, allowing for any consistent system of measurement to be used. Consideration was given to whether or not the test specimens were fastened to the bearing plate/support during testing. It was discovered that some of the test specimens in the literature were not fastened to the bearing plate/support during testing, which can make a considerable difference in the *web crippling* capacity of certain sections and loading conditions. Therefore, it was decided to separate the data on the basis of members being fastened to the bearing plate/support and those not being fastened to the bearing plate/support. The fastened to the bearing plate/support data in the literature were primarily based on specimens being bolted to the bearing plate/support; hence, a few control tests were carried out by Schuster, the results of which are contained in Beshara (1999), using self-drilling screws to establish the *web crippling* integrity in comparison to the bolted data. Based on these tests, the specimens with self-drilling screws performed equally well in comparison to the specimens with bolts. Fastened to the bearing plate/support in practice can be achieved by either using bolts, self-drilling/self-tapping screws or by welding. What is important is that the *flange* elements are restrained from rotating at the location of *load* application. In fact, in most cases, the *flanges* are frequently completely restrained against rotation by some type of sheathing material that is attached to the *flanges*.

The data was further separated in the *Specification* based on section type, as follows:

1) Built-up sections (Table C3.4.1-1),

2) Single *web* channel and C-sections (Table C3.4.1-2),

3) Single *web* Z-sections (Table C3.4.1-3),

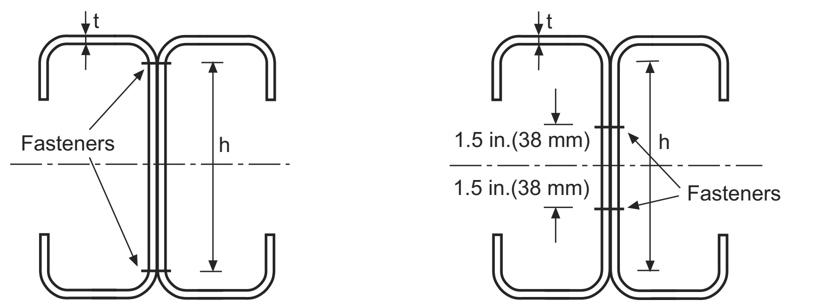
4) Single hat sections (Table C3.4.1-4), and

5) Multi-web deck sections (Table C3.4.1-5).

Calibrations were carried out by Beshara and Schuster (2000) in accordance with Supornsilaphachai, Galambos and Yu (1979) to establish the *safety factors*, Ω, and the *resistance factors*, ϕ, for each *web crippling* case. Based on these calibrations, different *safety factors* and corresponding *resistance factors* are presented in the *web crippling* coefficient tables for the particular *load* case and section type. In 2005, the *safety* and the *resistance factors* for built-up and single hat sections with interior one-*flange* loading case were revised based on a more consistent calibration. For the fastened built-up sections, the factors were revised from 1.65 to 1.75 (for *ASD*), 0.90 to 0.85 (for *LRFD*) and 0.80 to 0.75 (for *LSD*). For the fastened single hat section, the factors were revised from 1.90 to 1.80 (for *ASD*) and 0.80 to 0.85 (for *LRFD*). For the unfastened single hat sections, the factors were revised from 1.70 to 1.80 (for *ASD*), 0.90 to 0.80 (for *LRFD*) and 0.75 to 0.70 (for *LSD*). Also in 2005, the coefficients for built-up sections were revised to remove inconsistencies between unfastened and fastened conditions and to better reflect the calibration for the *safety factor* and the *resistance factors*. Also, a minimum bearing length of 3/4 in. (19 mm) was introduced based on the data used in the development of the *web crippling* coefficients. For fastened-to-support single *web* C- and Z-section members under interior two-*flange* loading or reaction, the distance from the edge of bearing to the end of the member (Figure C-C3.4.1-2(d)) must be extended at least 2.5h. This requirement is necessary because a total of 5h specimen length was used for the test setup shown in Figure C-C3.4.1-2(d) (Beshara, 1999). The 2.5h length is conservatively taken from the edge of bearing rather than the centerline of bearing.

The assumed distributions of *loads* or reactions acting on the *web* of a member, as shown in Figure C-C3.4.1-4, are independent of the flexural response of the member. Due to the flexural action, the point of bearing will vary relative to the plane of bearing, resulting in a non-uniform bearing *load* distribution on the *web*. The value of P_n will vary because of a transition from the interior one-*flange* loading (Figure C3.4.1-4(b)) to the end one-*flange* loading (Figure C3.4.1-4(a)) condition. These discrete conditions represent the experimental basis on which the design provisions were founded (Winter, 1970; Hetrakul and Yu, 1978). Based on additional updated calibrations, the *resistance factor* for Canada *LSD* for the unfastened interior one-*flange* loading (IOF) case in Table C3.4.1-4 was changed from 0.75 to 0.70 in 2004.

In the case of unfastened built-up members such as I-sections (not fastened to the bearing plate/support), the available data was for specimens that were fastened together with a row of fasteners near each *flange* line of the member (Winter and Pian, 1946) and Hetrakul and Yu (1978) as shown in Figure C-C3.4.1-5(a). For the fastened built-up member data of I-sections (fastened to the bearing plate/support), the specimens were fastened together with two rows of fasteners located symmetrically near the centerline length of the member, as shown in Figure C-C3.4.1-5(b) (Bhakta, LaBoube and Yu, 1992).



(a) Winter and Pian, 1946
Hetrakul and Yu, 1978

(b) Bhakta, LaBoube and Yu, 1992

Figure C-C3.4.1-5 Typical Bolt Pattern for I-Section Test Specimens

In *Specification* Table C3.4.1-1, the heading was changed in 2012 to indicate that the resulting *nominal web crippling strength* [*resistance*] is per *web*.

The research indicates that a Z-section having its end support *flange* bolted to the section's supporting member through two 1/2-in. (12.7 mm) diameter bolts will experience an increase in end one-*flange web crippling* capacity (Bhakta, LaBoube and Yu, 1992; Cain, LaBoube and Yu, 1995). The increase in *load*-carrying capacity was shown to range from 27 to 55 percent for the sections under the limitations prescribed in the *Specification*. A lower bound value of 30 percent increase was permitted in *Specification* Section C3.4 of the 1996 *Specification*. This is now incorporated under "Fastened to Support" condition.

In 2005, the R/t limit in Table C3.4.1-3 regarding interior one-*flange* loading for fastened Z-sections was changed from 5 to 5.5 to achieve consistency with *Specification* Equations C3.5.1-3 and C3.5.2-3, which stipulate a limit of R/t = 5.5.

For two nested Z-sections, the 1996 AISI *Specification* permitted the use of a slightly different *safety factor* and *resistance factor* for the interior one-*flange* loading condition. This is no longer required since the new *web crippling* approach now takes this into account in Table C3.4.1-3 of the *Specification* under the category of "Fastened to Support" for the interior one-*flange* loading case.

The coefficients in *Specification* Table C3.4.1-4 for one-*flange* loading or reaction with fastened to support condition are based on those with unfastened to support condition. For consistency, the R/t ratios for unfastened to support condition were revised in 2009 to be the same as the values of fastened to support condition. The table heading was changed to indicate that the resulting *nominal web crippling strength* [*resistance*] is per *web*.

The previous *web crippling* coefficients in Table C3.4.1-5 for end one-*flange* loading (EOF) of multi-*web* deck sections in the design provisions (AISI 2001) were based on limited data. This data was based on specimens that were not fastened to the support during testing; hence, the previous coefficients for this case were also being used conservatively for the case of fastened to the support. Based on extensive testing, *web crippling* coefficients were developed by James A. Wallace (2003) for both the unfastened and fastened case of EOF loading. Calibrations were also carried out to establish the respective *safety factors* and *resistance factors*. The R/t ratio for interior one-*flange* loading with fastened to support condition was revised in 2012 to be consistent with the corresponding interior one-*flange* loading value of the unfastened condition. The heading of Table C3.4.1-5 was changed to indicate that the resulting *nominal web crippling strength* [*resistance*] is per *web*. A note was also added to the table to indicate that multi-*web* deck sections are considered unfastened for any support fastener spacing greater than 18 in. (460 mm) (Wallace, 2004).

In 2004, the definitions of "one-*flange* loading" and "two-*flange* loading" were revised according to the test setup, specimen lengths, development of *web crippling* coefficients, and calibration of *safety factors* and *resistance factors*. In Figures C-C3.4.1-3 and C-C3.4.1-4 of the *Commentary*, the distances from the edge of bearing to the end of the member were revised to be consistent with the *Specification*.

Specification Equation C3.4.1-2 for single *web* C- and Z-sections with an overhang or overhangs is based on a study of the behavior and resultant failure *loads* from an end one-*flange* loading investigation performed at the University of Missouri-Rolla (Holesapple and LaBoube, 2002). This equation is applicable within the limits of the investigation. The UMR test results indicated that in some situations with overhangs, the interior one-*flange* loading capacity may not be achieved, and the interior one-*flange* loading condition was therefore removed from Figures C-C3.4.1-3 and C-C3.4.1-4.

Tests were conducted on fastened to support, stiffened *flange*, single *web* 3-½ in. (88.9 mm) C-sections subjected to interior two-*flange* loading or reactions (ITF) that indicate the *web crippling* equation is unconservative by about 25%. Therefore, in 2012, the application of the *web crippling* equation was limited to a *web* depth greater than or equal to 4-½ in. (110 mm) or more to be consistent with the tests conducted by Schuster and Bashera in 1999. This revision was based on the *web crippling* test observations (Yu, 2009 and 2009a).

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

Studies by Langan et al. (1994), Uphoff (1996) and Deshmukh (1996) quantified the reduction in *web crippling* capacity when a hole is present in a *web* element. These studies investigated both the end one-*flange* and interior one-*flange* loading conditions for h/t and d_h/h ratios as large as 200 and 0.81, respectively. The studies revealed that the reduction in *web crippling* strength is influenced primarily by the size of the hole as reflected in the d_h/h ratio and the location of the hole, x/h ratio.

The provisions for circular and non-circular holes also apply to any hole pattern that fits within an equivalent virtual hole. Figure C-B2.4-1 illustrates the L_h and d_h that may be used for a multiple hole pattern that fits within a non-circular virtual hole. Figure C-B2.4-2 illustrates the d_h that may be used for a rectangular hole that fits within a circular virtual hole. For each case, the design provisions apply to the geometry of the virtual hole geometry, not the actual hole or holes.

C3.5 Combined Bending and Web Crippling

C3.5.1 ASD Method

This *Specification* contains interaction equations for the combination of bending and *web crippling*. *Specification* Equations C3.5.1-1 and C3.5.1-2 are based on an evaluation of available experimental data using the *web crippling* equation included in the 2001 edition of the *Specification* (LaBoube, Schuster, and Wallace, 2002). The experimental data is based on research studies conducted at the University of Missouri-Rolla (Hetrakul and Yu, 1978 and 1980; Yu, 1981 and 2000), Cornell University (Winter and Pian, 1946), and the University of Sydney (Young and Hancock, 2000). For embossed *webs*, *crippling* strength should be determined by tests according to *Specification* Chapter F.

The exception clause included in *Specification* Section C3.5.1 for single unreinforced *webs* applies to the interior supports of continuous spans using decks and beams, as shown in Figure C-C3.5-1. Results of continuous beam tests of steel decks (Yu, 1981) and several independent studies by manufacturers indicate that, for these types of members, the post-*buckling* behavior of *webs* at interior supports differs from the type of failure mode occurring under concentrated *loads* on single-span beams. This post-*buckling* strength enables the member to redistribute the moments in continuous spans. For this reason, *Specification* Equation C3.5.1-1 is not applicable to the interaction between bending and the reaction at interior supports of continuous spans. This exception clause applies only to the members shown in Figure C-C3.5-1 and similar situations explicitly described in *Specification* Section C3.5.1.

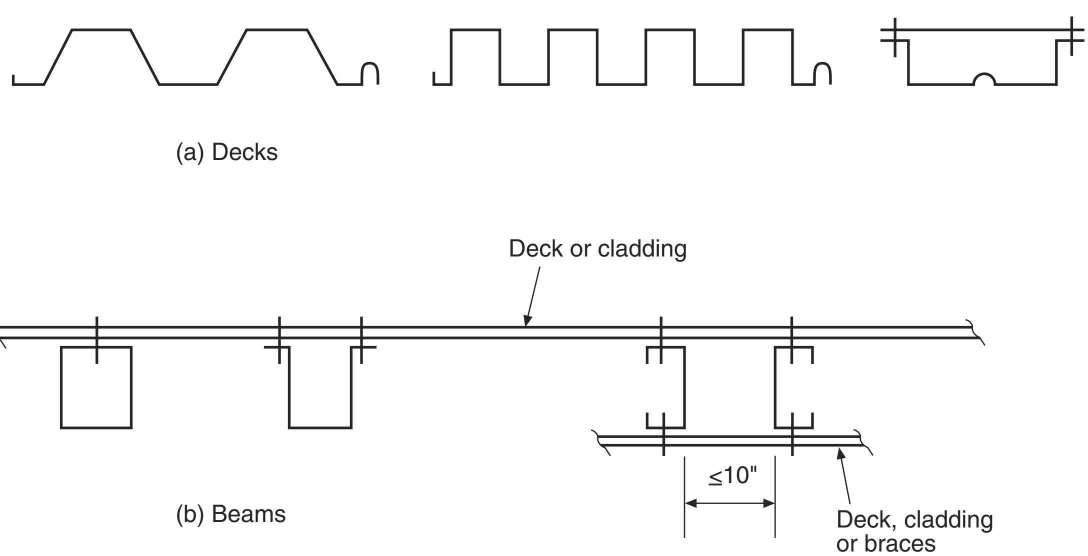


Figure C-C3.5-1 Sections Used for Exception Clause of Specification Section C3.5

The exception clause should be interpreted to mean that the effects of combined bending and *web crippling* need not be checked for determining *load*-carrying capacity. Furthermore, the positive bending resistance of the beam should be at least 90 percent of the negative bending resistance in order to ensure the safety implied by the *Specification*.

Using this procedure, the *service loads* may: (1) produce slight deformations in the member over the support, (2) increase the actual compressive bending *stresses* over the support to as high as 0.8 F_y, and (3) result in additional bending deflection of up to 22 percent due to elastic moment redistribution.

If *load*-carrying capacity is not the primary design concern because of the above behavior, the designer is urged to use *Specification* Equation C3.5.1-1.

In 1996, additional design information was added to *Specification* Section C3.5.1(c) for two nested Z-shapes. These design provisions are based on the research conducted at the University of Wisconsin-Milwaukee, University of Missouri-Rolla, and a metal building manufacturer (LaBoube, Nunnery and Hodges, 1994). The *web crippling* and bending behavior of unreinforced nested *web* elements is enhanced because of the interaction of the nested *webs*. The design equation is based on the experimental results obtained from 14 nested *web* configurations. These configurations are typically used by the metal building industry.

In 2003, based on the test data of LaBoube, Nunnery, and Hodges (1994), the interaction equation for the combined effects of bending and *web crippling* was reevaluated because a new *web crippling* equation was adopted for Section C3.4.1 of the *Specification*.

C3.5.2 LRFD and LSD Methods

For the *Load and Resistance Factor Design* and the *Limit States Design* methods, *Specification* Equations C3.5.2-1 and C3.5.2-2 are based on the same equations as used for *ASD* using the *required strength* [effect due to *factored loads*] and *design strength* [*factored resistance*]. In both equations, different symbols are used for the *required strength* [effect of *factored loads*] for the concentrated *load* or reaction due to *factored loads*, and the *required flexural strength* [moment due to *factored loads*] according to the *LRFD* and the *LSD* methods.

In the development of the original *LRFD* equations, a total of 551 tests were calibrated for combined bending and *web crippling* strength. Based on ϕ_w = 0.75 for single unreinforced *webs* and ϕ_w = 0.80 for I-sections, the values of reliability index vary from 2.5 to 3.3 as summarized in the AISI *Commentary* (AISI, 1991).

For two nested Z-shapes, *Specification* Equation C3.5.2-3 was derived from the same research work discussed in Section C3.5.1 for *Specification* Equation C3.5.1-3.

C3.6 Combined Bending and Torsional Loading

When the transverse *loads* applied to a bending member do not pass through the shear center of the cross-section of the member, twisting and torsional *stresses* can develop. The torsional *stresses* consist of pure torsional shear *stresses*, shear *stresses* due to warping, and normal *stresses* due to warping. References such as the AISC Steel Design Guide (AISC, 1997a) “Torsional Analysis of Structural Steel Members” describe the effect of torsion and how these *stresses* may be calculated.

Open cold-formed steel sections have little resistance to torsion, thus severe twisting and large *stresses* can develop. In many situations, however, the *connection* between a beam and the member delivering the *load* to the beam is such that it constrains twisting and in effect causes the resultant *load* to act as though it is delivered through the shear center. In such cases the torsional effects do not occur. Positive *connections* between the *load*-bearing *flange* and supported elements, in general, prevent torsional effects. An example of this is a *purlin* supporting a through-fastened roof panel that will prevent movement in the plane of the roof panel. It is important that the designer ensure that torsion is adequately constrained when evaluating a specific situation.

In situations where torsional loading cannot be avoided, discrete bracing will reduce the torsional effects. For most situations, the maximum torsional warping *stresses* will occur at discrete brace locations. Torsional bracing at the third points of the span would be adequate for most light construction applications. The bracing should be designed to prevent torsional twisting at the braced points.

Specification Section C3.6 provides design criteria for a *singly-* or *doubly*-symmetric member that is subjected to torsional loading. The provision uses a reduction factor, R, to reduce the *nominal moment strength* [*resistance*] as determined by *Specification* Section C3.1.1(a). This factor accounts for the normal *stresses* due to combined torsional warping and flexure. In 2012, the R factor was revised to accommodate situations where the maximum *stress* due to combined bending and torsional warping occurs at the tip of the *flange* stiffener. This R factor requires calculation of both the bending only *stresses* and the torsional warping *stresses* at critical points on the cross-section. The largest combination of these is the denominator of the reduction factor while the bending *stress* alone at the extreme fiber is the numerator. The member is then selected based on bending alone with the effect of torsion accounted for by the reduction in the *nominal moment strength* [*resistance*].

The largest combined *stresses* on the cross-section may occur at the junction of the *web* and *flange*, at the junction of the edge of *flange* and *flange* stiffener, or at the tip of the *flange* stiffener. The second and third conditions have a more severe effect on reducing the moment capacity of the member. These conditions can occur when the applied *load* is positioned off the member away from both the *web* and the shear center. This is shown from the test results reported in the referenced paper by Put et al. (1999). This is not a practical situation for structural assemblies; however, this location of the critical *stresses* would occur at the position of a torsional brace located at mid-span of a member. To allow for the more favorable situation, the provisions of *Specification* Section C3.6 permit the *nominal moment strength* [*resistance*] to be increased by 15% when the critical combination of *stresses* occurs at the junction of the *flange* and *web*. This is supported by tests on channels conducted by Winter, et al. (1950), which indicated that an overstress of 15% did not significantly affect the *load* -carrying capacity.

Rational engineering analysis should be used for sections, such as *point-symmetric* and non-symmetric *sections*, that are not covered by *Specification* Section C3.6. For these members, combined flexural with torsional warping *stresses* should be checked at both maximum tensile and compressive *stress* locations. A reasonable method would be to limit the combined bending *stress* and torsional warping *stress* to an allowable *stress* or factored *stress* using *safety factors* or *resistance factors*, respectively, provided in *Specification* Section C3.1.1. Any location on the cross-section that may control design should be considered.

The provisions of this section are intended as a separate limit state for *available flexural* *strength* [*factored resistance*]. It is still necessary to check the other limit states listed in *Specification* Section C3.1, but those limit states are calculated without the torsional R factor. In addition, the R factor is excluded from all interaction checks involving flexure such as combined bending and shear (*Specification* Section C3.3), combined bending and *web crippling* (*Specification* Section C3.5), and combined axial *load* and bending (*Specification* Section C5). The provisions of this section should not be used in combination with the bending provisions in *Specification* Sections D6.1.1 and D6.1.2, since these provisions are based on tests in which torsional effects are present.

C3.7 Stiffeners

C3.7.1 Bearing Stiffeners

Design requirements for attached bearing stiffeners (previously called transverse stiffeners) and for shear stiffeners were added in the 1980 AISI *Specification* and were unchanged in the 1986 *Specification*. The same design equations are retained in Section C3.7 of the current *Specification*. The term "transverse stiffener" was renamed to "bearing stiffeners" in 2004. The *nominal strength* [*resistance*] equation given in Item (a) of *Specification* Section C3.7.1 serves to prevent end crushing of the bearing stiffeners, while the *nominal strength* [*resistance*] equation given in Item (b) is to prevent column-type *buckling* of the *web*-stiffeners. The equations for computing the *effective areas* (A_b and A_c) and the *effective widths* (b_1 and b_2) were adopted from Nguyen and Yu (1978a) with minor modifications.

The available experimental data on cold-formed steel bearing stiffeners were evaluated by Hsiao, Yu and Galambos (1988a). A total of 61 tests were examined. The *resistance factor* of 0.85 used for the *LRFD* method was selected on the basis of the statistical data. The corresponding reliability indices vary from 3.32 to 3.41.

In 1999, the upper limit of w/t_s ratio for the unstiffened elements of cold-formed steel bearing stiffeners was revised from $0.37\sqrt{E/F_{ys}}$ to $0.42\sqrt{E/F_{ys}}$ for the reason that the former was calculated based on the *Allowable Strength Design* approach, while the latter is based on the *effective area* approach. The revision provided the same basis for the stiffened and unstiffened elements of cold-formed steel bearing stiffeners.

C3.7.2 Bearing Stiffeners in C-Section Flexural Members

The provisions of this section are based on the research by Fox and Schuster (2002), which investigated the behavior of stud and track type bearing stiffeners in cold-formed steel C-section flexural members. These stiffeners fall outside of the scope of *Specification* Section C3.7.1. The research program investigated bearing stiffeners subjected to two-*flange* loading at both interior and end locations, and with the stiffener positioned between the member *flanges* and on the back of the member. A total of 263 tests were carried out on different stiffened C-section assemblies. The design expression in *Specification* Section C3.7.2 is a simplified method applicable with the limits of the test program. A more detailed beam-column design method is described in Fox (2002).

C3.7.3 Shear Stiffeners

The requirements for shear stiffeners included in *Specification* Section C3.7.3 were primarily adopted from the AISC Specification (1978). The equations for determining the minimum required moment of inertia (*Specification* Equation C3.7.3-1) and the minimum required *gross area* (*Specification* Equation C3.7.3-2) of attached shear stiffeners are based on the studies summarized by Nguyen and Yu (1978a). In *Specification* Equation C3.7.3-1, the minimum value of $(h/50)^4$ was selected from the AISC Specification (AISC, 1978).

For the *LRFD* method, the available experimental data on the shear strength of beam *webs* with shear stiffeners were calibrated by Hsiao, Yu and Galambos (1988a). The statistical data used for determining the *resistance factor* were summarized in the AISI *Design Manual* (AISI, 1991). Based on these data, the reliability index was found to be 4.10 for $\phi = 0.90$.

C3.7.4 Non-Conforming Stiffeners

Tests on rolled-in stiffeners covered in *Specification* Section C3.7.4 were not conducted in the experimental program reported by Nguyen and Yu (1978). Lacking reliable information, the *available strength* [*factored resistance*] of stiffeners should be determined by special tests.

C4 Concentrically Loaded Compression Members

Axially loaded compression members should be designed for the following limit states depending on the configuration of the cross-section, *thickness* of material, unbraced length, and end restraint: (1) yielding, (2) overall column *buckling* (*flexural buckling*, *torsional buckling*, or *flexural-torsional buckling*), (3) *local buckling* of individual elements, and (4) *distortional buckling*. The first three limit states are discussed in Section C4.1 and *distortional buckling* limit state is discussed in Section C4.2. For the design tables and example problems on columns, see Parts I and III of the AISI *Cold-Formed Steel Design Manual* (AISI, 2013).

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

In this section, the limit states of yielding and overall column *buckling* are discussed.

A. Yielding

It is well known that a very short, compact column under an axial *load* may fail by yielding. The yield load is determined by Equation C-C4.1-1:

$$P_y = A_g F_y \tag{C-C4.1-1}$$

where A_g is the *gross area* of the column and F_y is the *yield stress* of steel.

B. Flexural Buckling of Columns

(a) Elastic Buckling Stress

A slender, axially loaded column may fail by overall *flexural buckling* if the cross-section of the column is a doubly-symmetric shape, closed shape (square or rectangular tube), cylindrical shape, or point-symmetric shape. For singly-symmetric shapes, *flexural buckling* is one of the possible failure modes. Wall studs connected with sheathing material can also fail by *flexural buckling*.

The elastic critical *buckling* load for a long column can be determined by the following Euler equation:

$$(P_{cr})_e = \frac{\pi^2 EI}{(KL)^2} \tag{C-C4.1-2}$$

where $(P_{cr})_e$ is the column *buckling* load in the elastic range, E is the modulus of elasticity, I is the moment of inertia, K is the effective length factor, and L is the unbraced length. Accordingly, the elastic column *buckling stress* is

$$(F_{cr})_e = \frac{(P_{cr})_e}{A_g} = \frac{\pi^2 E}{(KL/r)^2} \tag{C-C4.1-3}$$

in which r is the radius of gyration of the full cross-section, and KL/r is the effective slenderness ratio.

(b) *Inelastic Buckling Stress*

When the elastic column *buckling stress* computed by Equation C-C4.1-3 exceeds the proportional limit, F_{pr}, the column will buckle in the inelastic range. Prior to 1996, the following equation was used in the AISI *Specification* for computing the inelastic column *buckling stress*:

$$(F_{cr})_I = F_y\left(1 - \frac{F_y}{4(F_{cr})_e}\right) \tag{C-C4.1-4}$$

It should be noted that because the above equation is based on the assumption that $F_{pr} = F_y/2$, it is applicable only for $(F_{cr})_e \geq F_y/2$.

By using λ_c as the column slenderness parameter instead of slenderness ratio, KL/r, Equation C-C4.1-4 can be rewritten as follows:

$$(F_{cr})_I = \left(1 - \frac{\lambda_c{}^2}{4}\right)F_y \tag{C-C4.1-5}$$

where

$$\lambda_c = \sqrt{\frac{F_y}{(F_{cr})_e}} = \frac{KL}{r\pi}\sqrt{\frac{F_y}{E}} \tag{C-C4.1-6}$$

Accordingly, Equation C-C4.1-5 is applicable only for $\lambda_c \leq \sqrt{2}$.

(c) *Nominal Axial Strength [Resistance] for Locally Stable Columns*

If the individual components of compression members have small w/t ratios, *local buckling* will not occur before the compressive *stress* reaches the column *buckling stress* or the *yield stress* of steel. Therefore, the *nominal axial strength* [*resistance*] can be determined by the following equation:

$$P_n = A_g F_{cr} \tag{C-C4.1-7}$$

where

P_n = *nominal axial strength* [*resistance*]

A_g = *gross area* of cross-section

F_{cr} = column *buckling stress*

(d) *Nominal Axial Strength [Resistance] for Locally Unstable Columns*

For cold-formed steel compression members with large w/t ratios, *local buckling* of individual component plates may occur before the applied *load* reaches the *nominal axial strength* [*resistance*] determined by Equation C-C4.1-7. The interaction effect of the local and overall column *buckling* may result in a reduction of the overall column strength. From 1946 through 1986, the effect of *local buckling* on column strength was considered in the AISI *Specification* by using a form factor Q in the determination of allowable *stress* for the design of axially loaded compression members (Winter, 1970; Yu and LaBoube, 2010). Even though the Q-factor method was used successfully for the design of cold-formed steel compression members, research work conducted at Cornell University and other institutions have shown that this method is capable of improvement. On the basis of the test results and analytical studies of DeWolf, Peköz, Winter, and Mulligan (DeWolf, Peköz and Winter, 1974; Mulligan and Peköz, 1984) and Peköz's development of a unified approach for the design of cold-formed steel members (Peköz, 1986b), the Q-factor method was eliminated in the 1986 edition of the AISI *Specification*. In order to reflect the effect of *local buckling* on the reduction of column strength, the *nominal axial strength* [*resistance*] is determined by the critical column *buckling stress* and the *effective area*, A_e, instead of the full sectional area. When A_e cannot be calculated, such as when the compression member has dimensions or geometry beyond the range of applicability of the AISI *Specification*, the *effective area* A_e can be determined experimentally by stub column tests using AISI S902, *Stub-Column Test Method for Effective Area of Cold-Formed Steel Columns* (AISI, 2013c). For a more in-depth discussion of the background for these provisions, see Peköz (1986b). Therefore, the *nominal axial strength* [*resistance*] of cold-formed steel compression members can be determined by the following equation:

$$P_n = A_e F_{cr} \qquad \text{(C-C4.1-8)}$$

where F_{cr} is either elastic *buckling stress* or inelastic *buckling stress*, whichever is applicable, and A_e is the *effective area* at F_{cr}.

In the 1986 edition of the AISI *Specification*, the *nominal axial strength* [*resistance*] for C- and Z-sections and single angle sections was limited by Equation C-C4.1-9, which is determined by the *local buckling stress* of the unstiffened element and the area of the full cross-section:

$$P_n = \frac{A\pi^2 E}{25.7(w/t)^2} \qquad \text{(C-C4.1-9)}$$

This equation was deleted since the 1996 edition of the *Specification* based on a study conducted by Rasmussen at the University of Sydney (Rasmussen, 1994) and validated by Rasmussen and Hancock (1992).

In the 1996 AISI *Specification*, the design equations for calculating the inelastic and elastic *flexural buckling stresses* have been changed to those used in the AISC *LRFD Specification* (AISC, 1993). As given in the *Specification* Section C4.1(a), these design equations are as follows:

$$\text{For } \lambda_c \le 1.5: \quad F_n = \left(0.658^{\lambda_c^2}\right)F_y \qquad \text{(C-C4.1-10)}$$

$$\text{For } \lambda_c > 1.5: \quad F_n = \left[\frac{0.877}{\lambda_c^2}\right]F_y \qquad \text{(C-C4.1-11)}$$

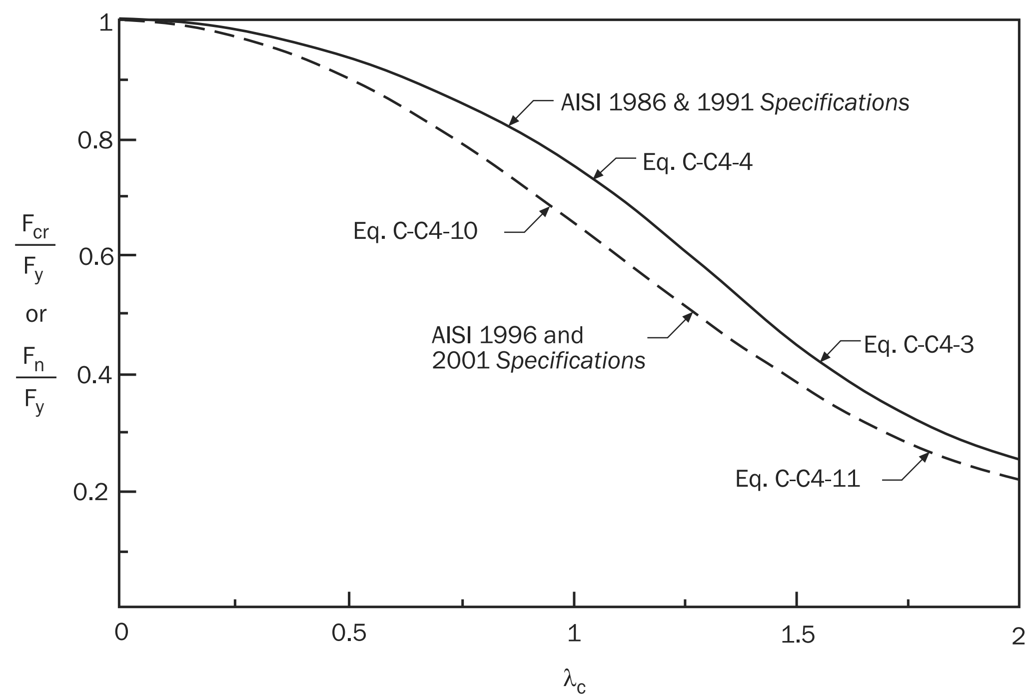


Figure C-C4.1-1 Comparison Between the Critical Buckling Stress Equations

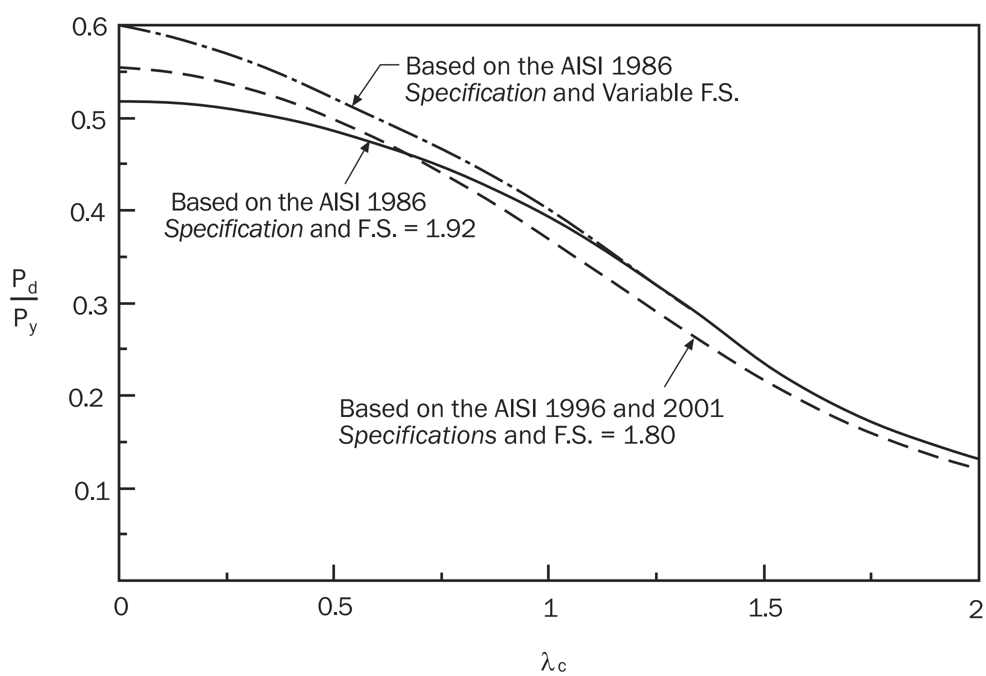


Figure C-C4.1-2 Comparison Between the Design Axial Strengths [Resistances], P_d

where F_n is the nominal *flexural buckling stress* which can be either in the elastic range or in the inelastic range depending on the value of $\lambda_c = \sqrt{F_y / F_e}$, and F_e is the elastic *flexural buckling stress* calculated by using Equation C-C4.1-3. Consequently, the equation for determining the *nominal axial strength* [*resistance*] can be written as

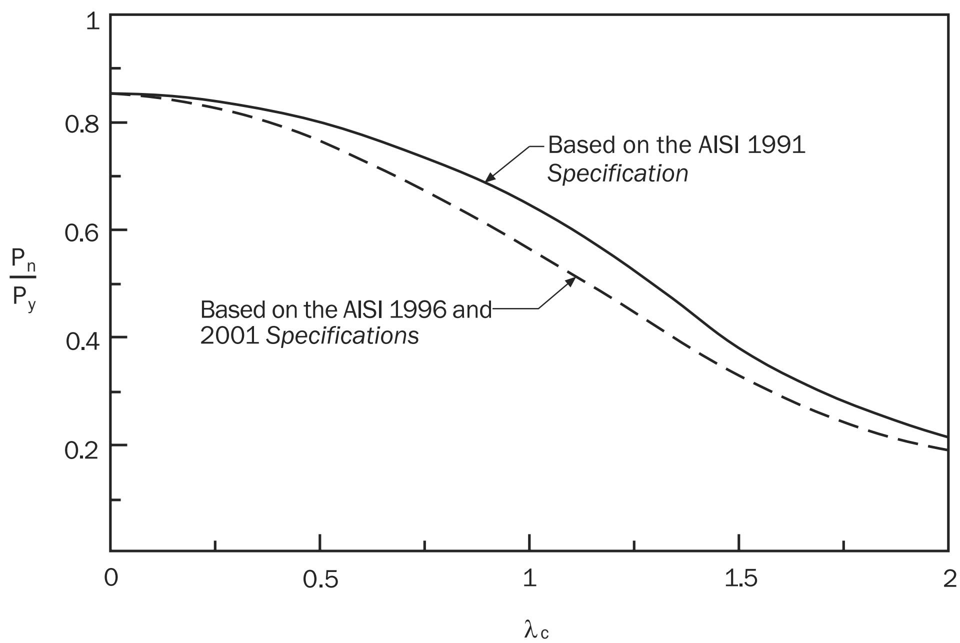


Figure C-C4.1-3 Comparison Between the Nominal Axial Strengths [Resistances], P_n

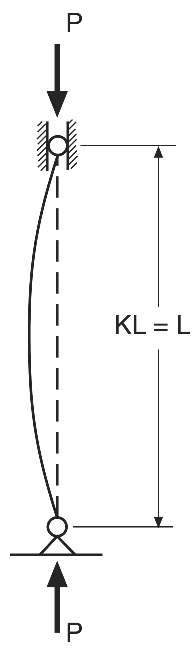


Figure C-C4.1-4 Overall Column Buckling

$$P_n = A_e F_n \tag{C-C4.1-12}$$

This is Equation C4.1-1 of the *Specification*.

The reasons for changing the design equations from Equation C-C4.1-4 to Equation C-C4.1-10 for inelastic *buckling stress* and from Equation C-C4.1-3 to Equation C-C4.1-11 for elastic *buckling stress* are:

1. The revised column design equations (Equations C-C4.1-10 and C-C4.1-11) are based

on a different basic strength model and were shown to be more accurate by Peköz and Sumer (1992). In this study, 299 test results on columns and beam-columns were evaluated. The test specimens included members with component elements in the post-*local buckling* range as well as those that were locally stable. The test specimens included members subject to *flexural buckling* as well as *flexural-torsional buckling*.

2. Because the revised column design equations represent the maximum strength with due consideration given to initial crookedness and can provide the better fit to test results, the required *safety factor* can be reduced. In addition, the revised equations enable the use of a single *safety factor* for all λ_c values even though the *nominal axial strength* [*resistance*] of columns decreases as the slenderness increases because of initial out-of-straightness. By using the selected *safety factor* and *resistance factor*, the results obtained from the *ASD* and *LRFD* approaches would be approximately the same for a live-to-dead *load* ratio of 5.0.

The design provisions included in the AISI *ASD Specification* (AISI, 1986), the *LRFD Specification* (AISI, 1991), the 1996 *Specification* and the current *Specification* (AISI, 2001, 2007) are compared in Figures C-C4.1-1, C-C4.1-2, and C-C4.1-3.

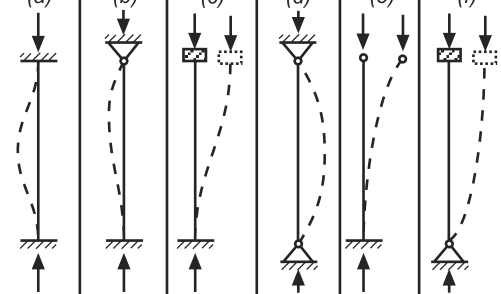
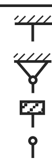
Figure C-C4.1-1 shows a comparison of the critical *flexural buckling stresses* used in the 1986, 1991, 1996, 2001 and 2007 *Specifications*. The equations used to plot these two curves are indicated in the figure. Because of the use of a relatively smaller *safety factor* in the 1996, 2001 and 2007 *Specifications*, it can be seen from Figure C-C4.1-2 that the design capacity is increased for thin columns with low slenderness parameters and decreased for high slenderness parameters. However, the differences would be less than 10%. For the *LRFD* method, the differences between the *nominal axial strengths* [*resistances*] used for the 1991, 1996, 2001 and the 2007 *LRFD* design provisions are shown in Figure C-C4.1-3. The curve for the *LSD* provisions would be the same as the curve for *LRFD*.

(e) Effective Length Factor, K

The effective length factor, K, accounts for the influence of restraint against rotation and translation at the ends of a column on its *load*-carrying capacity. For the simplest case, a column with both ends hinged and braced against lateral translation, *buckling* occurs in a single half-wave and the effective length KL, being the length of this half-wave, is equal to the actual physical length of the column (Figure C-C4.1-4); correspondingly, for this case, K = 1. This situation is approached if a given compression member is part of a structure which is braced in such a manner that no lateral translation (sidesway) of one end of the column relative to the other can occur. This is so for columns or studs in a structure with diagonal bracing, diaphragm bracing, shear-wall construction or any other provision which prevents horizontal displacement of the upper relative to the lower column ends. In these situations it is safe and only slightly, if at all, conservative to take K = 1.

If translation is prevented and abutting members (including foundations) at one or both ends of the member are rigidly connected to the column in a manner which provides substantial restraint against rotation, K-values smaller than 1 (one) are sometimes justified. Table C-C4.1-1 provides the theoretical K values for six idealized conditions in which *joint* rotation and translation are either fully realized or nonexistent. The same table also includes the K values recommended by the Structural Stability Research Council for design use (Galambos, 1998).

Table C-C4.1-1
Effective Length Factors K for Concentrically Loaded Compression Members

	(a)	(b)	(c)	(d)	(e)	(f)
Buckled shape of column is shown by dashed line						
Theoretical K value	0.5	0.7	1.0	1.0	2.0	2.0
Recommended K value when ideal conditions are approximated	0.65	0.80	1.2	1.0	2.10	2.0
End condition code		Rotation fixed, Translation fixed Rotation free, Translation fixed Rotation fixed, Translation free Rotation free, Translation free				

In trusses, the intersection of members provides rotational restraint to the compression members at *service loads*. As the collapse *load* is approached, the member *stresses* approach the *yield stress*, which greatly reduces the restraint they can provide. For this reason K value is usually taken as unity regardless of whether they are welded, bolted, or connected by screws. However, when sheathing is attached directly to the top *flange* of a continuous compression chord, research (Harper, LaBoube and Yu, 1995) has shown that the K values may be taken as 0.75 (AISI, 1995).

On the other hand, when no lateral bracing against sidesway is present, such as in the portal frame of Figure C-C4.1-5, the structure depends on its own bending stiffness for lateral stability. In this case, when failure occurs by *buckling* of the columns, it invariably takes place by the sidesway motion shown. This occurs at a lower *load* than the columns would be able to carry if they were braced against sidesway, and the figure shows that the half-wave length into which the columns buckle is longer than the actual column length. Hence, in this case K is larger than 1 (one) and its value can be read from the graph of Figure C-C4.1-6 (Winter et al., 1948a and Winter, 1970). Since column bases are rarely either actually hinged or completely fixed, K-values between the two curves should be estimated depending on actual base fixity.

Figure C-C4.1-6 can also serve as a guide for estimating K for other simple situations. For multi-bay and/or multi-story frames, simple alignment charts for determining K are given in the AISC Commentaries (AISC, 1989, 1999, 2005). For additional information on frame stability and *second-order effects*, see SSRC *Guide to Stability Design Criteria for Metal Structures* (Galambos, 1998) and the AISC Specifications and Commentaries.

If roof or floor slabs, anchored to *shear walls* or vertical plane bracing systems, are counted upon to provide lateral support for individual columns in a building system, their stiffness must be considered when functioning as horizontal *diaphragms* (Winter, 1958a).

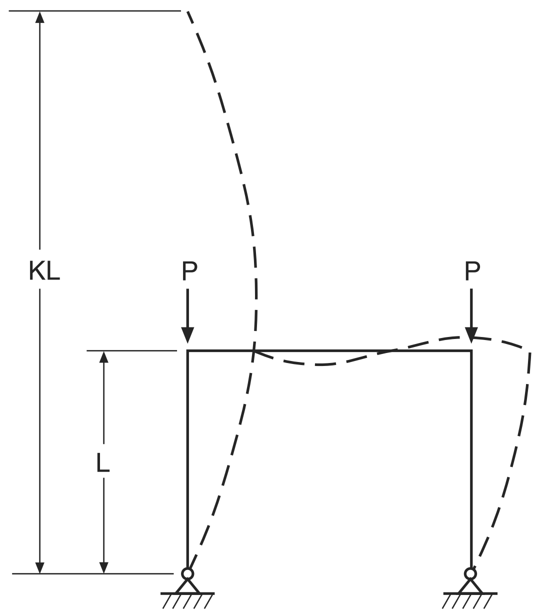


Figure C-C4.1-5 Laterally Unbraced Portal Frame

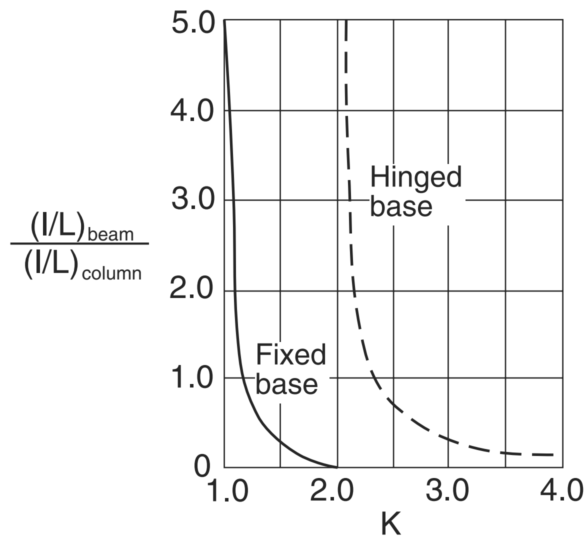


Figure C-C4.1-6 Effective Length Factor K in Laterally Unbraced Portal Frames

C. *Torsional Buckling of Columns*

It was pointed out at the beginning of this section that purely *torsional buckling*, i.e., failure by sudden twist without concurrent bending, is also possible for certain cold-formed open shapes. These are all point-symmetric shapes (in which shear center and centroid coincide), such as doubly-symmetric I-shapes, anti-symmetric Z-shapes, and such unusual sections as cruciforms, swastikas, and the like. Under concentric *load*, *torsional buckling* of such shapes very rarely governs design. This is so because such members of realistic slenderness will buckle flexurally or by a combination of flexural and *local buckling* at *loads* smaller than those which would produce *torsional buckling*. However, for relatively short members of this type, carefully dimensioned to minimize *local buckling*, such *torsional buckling* cannot be completely ruled out. If such *buckling* is elastic, it occurs at the critical *stress*, σ_t, calculated as follows (Winter, 1970):

$$\sigma_t = \frac{1}{Ar_o^2}\left[GJ + \frac{\pi^2 EC_w}{(K_t L_t)^2}\right] \quad \text{(C-C4.1-13)}$$

The above equation is the same as *Specification* Equation C3.1.2.1-9, in which A is the full cross-sectional area, r_o is the polar radius of gyration of the cross-section about the shear center, G is the shear modulus, J is Saint-Venant torsion constant of the cross-section, E is the modulus of elasticity, C_w is the torsional warping constant of the cross-section, and $K_t L_t$ is the effective length for twisting.

For inelastic *buckling*, the critical *torsional buckling stress* can also be calculated according to Equation C-C4.1-10 by using σ_t as F_e in the calculation of λ_c.

D. *Flexural-Torsional Buckling of Columns*

As discussed previously, concentrically loaded columns can buckle in the *flexural buckling* mode by bending about one of the principal axes; or in the *torsional buckling* mode by twisting about the shear center; or in the *flexural-torsional buckling* mode by simultaneous bending and twisting. For singly-symmetric shapes such as channels, hat sections, angles, T-sections, and I-sections with unequal *flanges*, for which the shear center and centroid do not coincide, *flexural-torsional buckling* is one of the possible *buckling* modes as shown in Figure C-C4.1-7. *Unsymmetric sections* will always buckle in the flexural-torsional mode.

It should be emphasized that one needs to design for *flexural-torsional buckling* only when it is physically possible for such *buckling* to occur. This means that if a member is so connected to other parts of the structure such as wall sheathing that it can only bend but

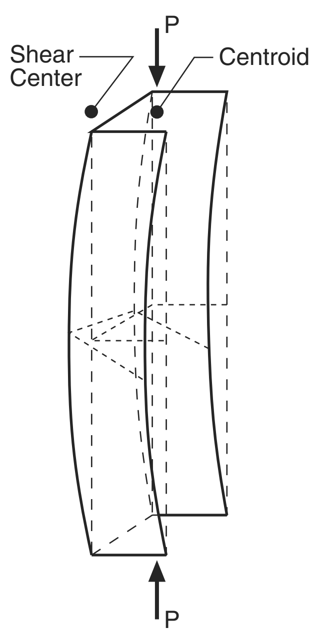


Figure C-C4.1-7 Flexural-Torsional Buckling of a Channel in Axial Compression

cannot twist, it needs to be designed for *flexural buckling* only. This may hold for the entire member or for individual parts. For instance, a channel member in a wall or the chord of a roof truss is easily connected to *girts* or *purlins* in a manner which prevents twisting at these connection points. In this case, *flexural-torsional buckling* needs to be checked only for the unbraced lengths between such *connections*. Likewise, a doubly-symmetric compression member can be made up by connecting two spaced channels at intervals by batten plates. In this case, each channel constitutes an "intermittently fastened component of a built-up shape." Here the entire member, being doubly-symmetric, is not subject to *flexural-torsional buckling* so that this mode needs to be checked only for the individual component channels between batten *connections* (Winter, 1970).

The governing elastic *flexural-torsional buckling* load of a column can be found from the following equation (Chajes and Winter, 1965; Chajes, Fang and Winter, 1966; Yu and LaBoube, 2010):

$$P_n = \frac{1}{2\beta}\left[(P_x + P_z) - \sqrt{(P_x + P_z)^2 - 4\beta P_x P_z}\right] \tag{C-C4.1-14}$$

If both sides of this equation are divided by the cross-sectional area A, one obtains the equation for the elastic, *flexural-torsional buckling stress* F_e as follows:

$$F_e = \frac{1}{2\beta}\left[(\sigma_{ex} + \sigma_t) - \sqrt{(\sigma_{ex} + \sigma_t)^2 - 4\beta\sigma_{ex}\sigma_t}\right] \tag{C-C4.1-15}$$

For this equation, as in all provisions which deal with *flexural-torsional buckling*, the x-axis is the axis of symmetry; $\sigma_{ex} = \pi^2 E/(K_x L_x/r_x)^2$ is the flexural Euler *buckling stress* about the x-axis; σ_t is the *torsional buckling stress* (Equation C-C4.1-13); and $\beta = 1-(x_o/r_o)^2$. It is worth noting that the *flexural-torsional buckling stress* is always lower than the Euler *stress* σ_{ex} for *flexural buckling* about the symmetry axis. Hence, for these *singly-symmetric sections*, *flexural buckling* can only occur, if at all, about the y-axis, which is the principal axis perpendicular to the axis of symmetry.

For inelastic *buckling*, the critical *flexural-torsional buckling stress* can also be calculated by using Equation C-C4.1-10.

An inspection of Equation C-C4.1-15 will show that in order to calculate β and σ_t, it is necessary to determine x_o = distance between shear center and centroid, J = Saint-Venant torsion constant, and C_w = warping constant, in addition to several other, more familiar cross-sectional properties. Because of these complexities, the calculation of the *flexural-torsional buckling stress* cannot be made as simple as that for *flexural buckling*. Formulas for typical C-, Z-sections, angle and hat sections are provided in Part I of the AISI *Design Manual* (AISI, 2013).

The above discussion refers to members subject to *flexural-torsional buckling*, but made up of elements whose w/t ratios are small enough so that no *local buckling* will occur. For shapes which are sufficiently thin, i.e., with w/t ratios sufficiently large, *local buckling* can combine with *flexural-torsional buckling* similar to the combination of local with *flexural buckling*. For this case, the effect of *local buckling* on the *flexural-torsional buckling* strength can also be handled by using the *effective area*, A_e, determined at the *stress* F_n for *flexural-torsional buckling*.

E. *Additional Design Consideration for Angles*

During the development of a unified approach to the design of cold-formed steel members, Peköz realized the possibility of a reduction in column strength due to initial sweep (out-of-straightness) of angle sections. Based on an evaluation of the available test results, an initial out-of-straightness of L/1000 was recommended by Peköz for the design of concentrically loaded compression angle members and beam-columns in the 1986 edition of the AISI *Specification*. Those requirements were retained in Sections C4.1, C5.2.1, and C5.2.2 of the 1996 edition of the *Specification*. A study conducted at the University of Sydney (Popovic, Hancock, and Rasmussen, 1999) indicated that for the design of singly-symmetric unstiffened angle sections under the axial compression *load*, the required additional moment about the minor principal axis due to initial sweep should only be applied to those angle sections for which the *effective area* at *stress* F_y is less than the *full, unreduced cross-sectional area*. Consequently, clarifications have been made in Sections C5.2.1 and C5.2.2 of the 2001 edition of the AISI *Specification* to reflect the research findings.

Equations C4.1-1 to C4.1-3 have been shown to be conservative in predicting the experimental failure loads obtained from tests of concentrically loaded pin-ended and fixed-ended angle columns. Tested columns exhibit end supports fixed with respect to warping and major-axis flexure, but pinned or fixed with respect to minor-axis flexure. Tests were performed by Popovic et al. (1999) and Chodraui et al. (2006) for columns with minor-axis pin-ends, and by Popovic et al. (1999) and Young (2004, 2005) for columns with fixed-ends. The above underestimation is essentially due to the fact that Equations C4.1-1 to C4.1-3: (i) account twice for the local/flexural-torsional effects (Rasmussen 2005), and (ii) disregard the beneficial effect of the warping fixity (Shifferaw and Schafer, 2011). Slivestre et al. (2012) investigated the mechanics of these phenomena and showed that the collapse of intermediate plain angle columns is governed by the interaction between major-axis *flexural-torsional buckling* (akin, but not identical, to *local bucking*) and minor-axis *flexural buckling* – due to effective centroid shift effects (Young and Rasmussen 1999). This interaction is much stronger in pin-ended columns. Several design methods/approaches have been proposed to estimate more accurately the angle column failure loads, thus accounting for the increased strength due to the warping fixity (e.g., Young 2004, Rasmussen 2005, Shifferaw and Schafer 2011, Silvestre et al. 2012).

F. *Slenderness Ratios*

The slenderness ratio, KL/r, of all compression members should preferably not exceed 200, except that during construction only, KL/r should not exceed 300. In 1999, the above recommendations were moved from the *Specification* to the *Commentary*.

The maximum slenderness ratios on compression and tension members have been stipulated in steel design standards for many years but are not mandatory in the AISI *Specification*.

The KL/r limit of 300 is still recommended for most tension members in order to control serviceability issues such as handling, sag and vibration. The limit is not mandatory, however, because there are a number of applications where it can be shown that such factors are not detrimental to the performance of the structure or assembly of which the member is a part. Flat strap tension bracing is a common example of an acceptable type of tension member where the KL/r limit of 300 is routinely exceeded.

The compression member KL/r limits are recommended not only to control handling, sag and vibration serviceability issues, but also to flag possible strength concerns. The AISI *Specification* provisions adequately predict the capacities of slender columns and beam-columns, but the resulting strengths are quite small and the members relatively inefficient. Slender members are also very sensitive to eccentrically applied axial *load* because the moment magnification factors given by $1/\alpha$ will be large.

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

If concentrically loaded compression members can buckle in the *flexural buckling* mode by bending about one of the principal axes, the *nominal flexural buckling strength* [*resistance*] of the column should be determined by using Equation C4.1-1 of the *Specification*. The elastic *flexural buckling stress* is given in Equation C4.1.1-1 of the *Specification*, which is the same as Equation C-C4.1-3 of the *Commentary*. This provision is applicable to *doubly-symmetric sections*, closed cross-sections and any other sections not subject to *torsional* or *flexural-torsional buckling*.

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

As discussed previously in Section C4.1, *torsional buckling* is one of the possible *buckling* modes for *doubly-* and *point-symmetric sections*. For *singly-symmetric sections*, *flexural-torsional buckling* is one of the possible *buckling* modes. The other possible *buckling* mode is *flexural buckling* by bending about the y-axis (i.e., assuming x-axis is the axis of symmetry).

For *torsional buckling*, the elastic *buckling stress* can be calculated by using Equation C-C4.1-13. For *flexural-torsional buckling*, Equation C-C4.1-15 can be used to compute the elastic *buckling stress*. The following simplified equation for elastic *flexural-torsional buckling* stress is an alternative permitted by the AISI *Specification*:

$$F_e = \frac{\sigma_t \sigma_{ex}}{\sigma_t + \sigma_{ex}} \tag{C-C4.1-16}$$

The above equation is based on the following interaction relationship given by Peköz and Winter (1969a):

$$\frac{1}{P_n} = \frac{1}{P_x} + \frac{1}{P_z} \tag{C-C4.1-17}$$

or

$$\frac{1}{F_e} = \frac{1}{\sigma_{ex}} + \frac{1}{\sigma_t} \tag{C-C4.1-18}$$

Research at the University of Sydney (Popovic, Hancock, and Rasmussen, 1999) has shown that singly-symmetric unstiffened cold-formed steel angles, which have a fully effective cross-section under *yield stress*, do not fail in a flexural-torsional mode and can be designed based on *flexural buckling* alone as specified in *Specification* Section C4.1.1. There is also no need to include a *load* eccentricity for these sections when using *Specification* Section C5.2.1 or Section C5.2.2 as explained in Item E of Section C4.1.

C4.1.3 Point-Symmetric Sections

This section of the *Specification* is for the design of discretely braced *point-symmetric sections* subjected to axial compression. An example of a *point-symmetric section* is a lipped or unlipped Z-section with equal *flanges*. The critical elastic *buckling stress* of *point-*

symmetric sections is the lesser of the two possible *buckling* modes, the elastic *torsional buckling stress*, σ_t, as defined in *Specification* Equation C3.1.2.1-9 or the elastic *flexural buckling stress* about its minor principal axis, as defined in *Specification* Equation C4.1.1-1. Figure C-D3.2.1-5 shows the relationship of the principal axes to the x and y axes of a lipped Z-section. The elastic *flexural buckling stress* should be calculated for axis 2.

C4.1.4 Nonsymmetric Sections

For nonsymmetric open shapes, the analysis for *flexural-torsional buckling* becomes extremely tedious unless its need is sufficiently frequent to warrant computerization. For one thing, instead of the quadratic equations, cubic equations have to be solved. For another, the calculation of the required section properties, particularly C_w, becomes quite complex. The method of calculation is given in Parts I and V of the AISI *Design Manual* (AISI, 2013) and the book by Yu and LaBoube (2010). Section C4.1.4 of the *Specification* states that calculation according to this section shall be used or tests according to Chapter F shall be made, when dealing with nonsymmetric open shapes.

C4.1.5 Closed Cylindrical Tubular Sections

Closed thin-walled cylindrical tubular members are economical sections for compression and torsional members because of their large ratio of radius of gyration to area, the same radius of gyration in all directions, and the large torsional rigidity. Like other cold-formed steel compression members, cylindrical tubes must be designed to provide adequate safety not only against overall column *buckling* but also against *local buckling*. It is well known that the classic theory of *local buckling* of longitudinally compressed cylinders overestimates the actual *buckling* strength, and that inevitable imperfections and residual *stresses* reduce the actual strength of compressed tubes radically below the theoretical value. For this reason, the design provisions for *local buckling* have been based largely on test results.

Local Buckling Stress

Considering the post-*buckling* behavior of the axially compressed cylinder and the important effect of the initial imperfection, the design provisions included in the AISI *Specification* were originally based on Plantema's graphic representation and the additional results of cylindrical shell tests made by Wilson and Newmark at the University of Illinois (Winter, 1970).

From the tests of compressed tubes, Plantema found that the ratio F_{ult}/F_y depends on the parameter $(E/F_y)(t/D)$, in which t is the wall *thickness*, D is the mean diameter of the tube, and F_{ult} is the ultimate *stress* or collapse *stress*. As shown in Figure C-C4.1-8, Line 1 corresponds to the collapse *stress* below the proportional limit, Line 2 corresponds to the collapse *stress* between the proportional limit and the *yield stress*, and Line 3 represents the collapse stress occurring at *yield stress*. In the range of Line 3, *local buckling* will not occur before yielding. In Ranges 1 and 2, *local buckling* occurs before the *yield stress* is reached. The cylindrical tubes should be designed to safeguard against *local buckling*.

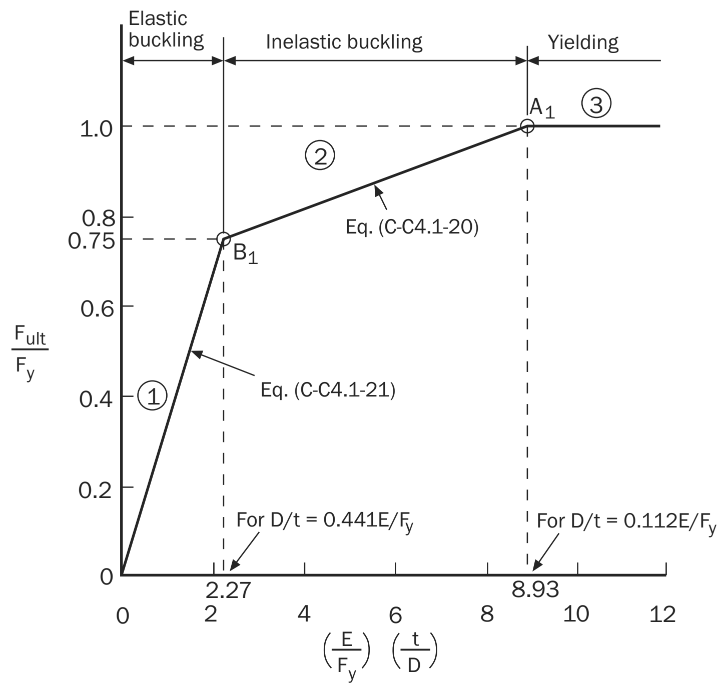


Figure C-C4.1-8 Critical Stress of Cylindrical Tubes for Local Buckling

Based on a conservative approach, the *Specification* specifies that when the D/t ratio is smaller than or equal to $0.112E/F_y$, the tubular member shall be designed for yielding. This provision is based on point A_1, for which $(E/F_y)(t/D) = 8.93$.

When $0.112E/F_y < D/t < 0.441E/F_y$, the design of tubular members is based on the inelastic *local buckling* criteria. For the purpose of developing a design equation for inelastic *buckling*, point B_1 was selected to represent the proportional limit. For point B_1,

$$\left(\frac{E}{F_y}\right)\left(\frac{t}{D}\right) = 2.27, \qquad \frac{F_{ult}}{F_y} = 0.75 \tag{C-C4.1-19}$$

Using line A_1B_1, the maximum *stress* of cylindrical tubes can be represented by

$$\frac{F_{ult}}{F_y} = 0.037\left(\frac{E}{F_y}\right)\left(\frac{t}{D}\right) + 0.667 \tag{C-C4.1-20}$$

When $D/t \geq 0.441E/F_y$, the following equation represents Line 1 for elastic *local buckling stress*:

$$\frac{F_{ult}}{F_y} = 0.328\left(\frac{E}{F_y}\right)\left(\frac{t}{D}\right) \tag{C-C4.1-21}$$

The correlations between the available test data and Equations C-C4.1-20 and C-C4.1-21 are shown in Figure C-C4.1-9. The definition of symbol "D" was changed from "mean diameter" to "outside diameter" in the 1986 AISI *Specification* in order to be consistent with the general practice.

As indicated in *Commentary* Section C3.1.3, *Specification* Section C4.1.5 is only applicable to members having a ratio of outside diameter-to-wall *thickness*, D/t, not greater than

$0.441E/F_y$ because the design of extremely thin tubes will be governed by elastic *local buckling* resulting in an uneconomical design. In addition, cylindrical tubular members with unusually large D/t ratios are very sensitive to geometric imperfections.

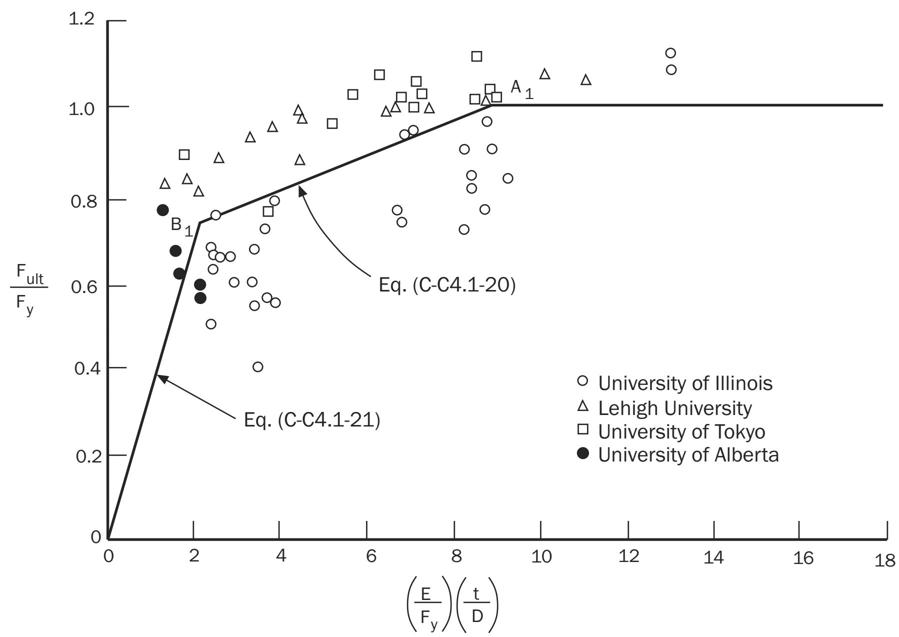


Figure C-C4.1-9 Correlation Between Test Data and AISI Criteria for Local Buckling of Cylindrical Tubes Under Axial Compression

When closed cylindrical tubes are used as concentrically loaded compression members, the *nominal axial strength* [*resistance*] is determined by the same equation as given in *Specification* Section C4.1, except that: (1) the nominal *buckling stress*, F_e, is determined only for *flexural buckling*, and (2) the *effective area*, A_e, is calculated by Equation C-C4.1-22:

$$A_e = [1-(1-R^2)(1-A_o/A)]A \qquad \text{(C-C4.1-22)}$$

where

$$R = \sqrt{F_y/2F_e} \qquad \text{(C-C4.1-23)}$$

$$A_o = \left[\frac{0.037}{DF_y/tE} + 0.667\right]A \le A \qquad \text{(C-C4.1-24)}$$

A = area of the unreduced cross-section.

Equation C-C4.1-24 is used for computing the reduced area due to *local buckling*. It is derived from Equation C-C4.1-20 for inelastic *local buckling stress* (Yu and LaBoube, 2010).

In 1999, the coefficient, R, was limited to one (1.0) so that the *effective area*, A_e, will always be less than or equal to the *unreduced cross-sectional area*, A. To simplify the equations, $R = F_y/(2F_e)$ is used rather than $R = \sqrt{F_y/(2F_e)}$ as in the previous edition of the AISI *Specification*. The equation for the *effective area* is simplified to $A_e = A_o + R(A - A_o)$ as given in Equation C4.1.5-1 of the *North American Specification*.

C4.2 Distortional Buckling Strength [Resistance]

Distortional buckling is an *instability* that may occur in members with edge-stiffened *flanges*, such as lipped C- and Z-sections. As shown in Figure C-C4.2-1, this *buckling* mode is characterized by *instability* of the entire *flange*, as the *flange* along with the edge stiffener rotates about the junction of the *flange* and the *web*. The length of the *buckling* wave in *distortional buckling* is considerably longer than *local buckling*, and noticeably shorter than *flexural* or *flexural-torsional buckling*. The *Specification* provisions of Section B4 partially account for *distortional buckling*, but research has shown that a separate limit state check is required (Schafer, 2002). Thus, in 2007, treating *distortional buckling* as a separate limit state, *Specification* Section C3.1.4 was added to address *distortional buckling* in beams and *Specification* Section C4.2 was added to address *distortional buckling* in columns. Note: As stated in the *Specification*, when a member is designed in accordance with Section D6.1.3, Compression Members Having One Flange Through-Fastened to Deck or Sheathing, the provisions of Section C4.2, Distortional Buckling Strength [Resistance], need not be applied since *distortional buckling* is inherently included as a limit state in the Section D6.1.3 strength prediction equations.

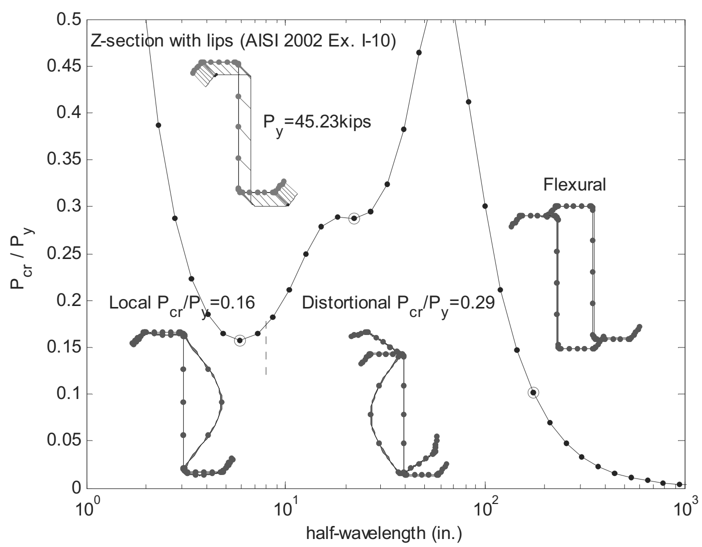


Figure C-C4.2-1 Rational Elastic Buckling Analysis of a Z-Section Under Compression Showing Local, Distortional, and Flexural Buckling Modes

Determination of the *nominal strength* [*resistance*] in *distortional buckling* (*Specification* Equation C4.2-2) was validated by testing. *Specification* Equation C4.2-2 was originally developed for the *Direct Strength Method* of Appendix 1 of the *Specification*. Calibration of the *safety* and *resistance factors* for *Specification* Equation C4.2-2 is provided in the commentary to Appendix 1. In addition, the Australian/New Zealand Specification (AS/NZS 4600) has used an expression of similar form to *Specification* Equation C4.2-2, but yielding slightly less conservative strength predictions than Equation C4.2-2, since 1996.

Distortional buckling is unlikely to control the strength of a column if: (a) the *web* is slender and triggers *local buckling* far in advance of *distortional buckling*, as is the case for many common C-sections, (b) edge stiffeners are sufficiently stiff and thus stabilize the *flange* (as is often the case for C-sections, but typically not for Z-sections due to the use of sloping lip stiffeners), (c) unbraced lengths are long and *flexural* or *flexural-torsional buckling* strength limits the capacity, or (d) adequate rotational restraint is provided to the *flanges* from attachments (panels, sheathing, etc.).

The primary difficulty in calculating the strength in *distortional buckling* is to efficiently estimate the elastic *distortional buckling stress*, F_d. Recognizing the complexity of this calculation, this section of the *Specification* provides two alternatives: *Specification* Section C4.2(a) provides a comprehensive method for C- and Z-section members and any open section with a single *web* and *flanges* of the same dimension, and Section C4.2(b) offers the option to use rational elastic *buckling* analysis. See the Appendix 1 commentary for further discussion. In 2010, the *Simplified Provision for Unrestrained C- and Z-Section With Simple Lip Stiffeners* was moved from the *Specification* to the *Commentary*. This simplified provision provides a conservative approximation to the *distortional buckling stress*, F_d, for C- and Z-sections with simple lip stiffeners. The expressions were specifically derived as a conservative simplification to those provided in *Specification* Sections C4.2(a) and (b). For many common sections, the simplified provision may be used to show that *distortional buckling* of the column will not control the capacity. *Specification* provisions C4.2(a) or (b), however, should be used to obtain the *distortional buckling* strength if *distortional buckling* controls the design.

Simplified Method for Unrestrained C- and Z-Sections With Simple Lip Stiffeners

For C- and Z-sections that have no rotational restraint of the *flange* and that are within the dimensional limits provided in this section, Equation C-C4.2-1 can be used to calculate a conservative prediction of *distortional buckling stress*, F_d. See *Specification* Section C4.2(a) or C4.2(b) for alternative provisions and for members outside the dimensional limits.

The following dimensional limits should 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 *Specification* Figure B2.3-2
b_o = out-to-out *flange* width as defined in *Specification* Figure B2.3-2
D = out-to-out lip dimension as defined in *Specification* Figure B4-1
t = base steel *thickness*
θ = lip angle as defined in *Specification* Figure B4-1

The *distortional buckling stress*, F_d, can be calculated in accordance with Eq. C-C4.2-1:

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

where

α = a value that accounts for the benefit of an unbraced length, L_m, shorter than L_{cr}, but can be conservatively taken as 1.0
= 1.0 for $L_m \ge L_{cr}$

$$= \left(L_m / L_{cr}\right)^{\ln\left(L_m / L_{cr}\right)} \qquad \text{for } L_m < L_{cr} \qquad \text{(C-C4.2-2)}$$

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 *Specification* Section C4.2 (a) or (b))

$$L_{cr} = 1.2 h_o \left(\frac{b_o D \sin\theta}{h_o t} \right)^{0.6} \le 10 h_o \qquad \text{(C-C4.2-3)}$$

$$k_d = 0.05 \le 0.1 \left(\frac{b_o D \sin\theta}{h_o t} \right)^{1.4} \le 8.0 \qquad \text{(C-C4.2-4)}$$

E = modulus of elasticity of steel

μ = Poisson's ratio of steel

Equations C-C4.2-1 to C-C4.2-4 assume the compression *flange* is unrestrained; however, the methods of *Specification* Sections C4.2(a) and (b) allow for a rotational restraint, k_ϕ, to be included to account for attachments which restrict *flange* rotation. Additional guidance on k_ϕ is provided in the *Commentary* Section C3.1.4.

(a) 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 *Specification* Section C4.2(a) provide a general method for calculation of the *distortional buckling stress*, F_d, for any open section with equal edge-stiffened compression *flanges*, including those with complex edge stiffeners. The provisions of *Specification* Section C4.2(a) also provide a more refined answer for any C- and Z-section, including those meeting the dimensional criteria of the *Simplified Provision for Unrestrained C- and Z-Sections With Simple Lip Stiffeners* presented in this *Commentary*. The expressions employed here are derived in Schafer (2002) and verified for complex stiffeners in Schafer et al. (2006). The equations used for the *distortional buckling stress*, F_d, in AS/NZS 4600 are also similar to those in *Specification* Section C4.2(a), except that when the *web* is very slender and is restrained by the *flange*, AS/NZS 4600 uses a simpler, conservative treatment. Since the provided expressions can be complicated, solutions for the geometric properties of C- and Z-sections based on centerline dimensions are provided in Table C-C3.1.4(a)-1.

(b) Rational Elastic Buckling Analysis

Rational elastic *buckling* analysis consists of any method following the principles of mechanics to arrive at an accurate prediction of the elastic *distortional buckling stress*. It is important to note that this is a rational elastic *buckling* analysis and not simply an arbitrary rational method to determine strength. A variety of rational computational and analytical methods can provide the elastic *buckling* moment with a high degree of accuracy. Complete details are provided in Section 1.1.2 of the commentary to Appendix 1 of the *Specification*. The *safety* and *resistance factors* of this section have been shown to apply to a wide variety of cross-sections undergoing *distortional buckling* (via the methods of Appendix 1). As long as the member falls within the geometric limits of main *Specification* Section B1.1, the same *safety* and *resistance factors* have been assumed to apply.

C5 Combined Axial Load and Bending

In the 1996 edition of the AISI *Specification*, the design provisions for combined axial *load* and bending were expanded to include expressions for the design of members subject to combined tensile axial *load* and bending. Since the 2001 edition, combined axial and bending for the *Limit States Design* (*LSD*) method has been added. The design approach of the *LSD* method is the same as the *LRFD* method.

C5.1 Combined Tensile Axial Load and Bending

These provisions apply to concurrent bending and tensile axial *load*. If bending can occur without the presence of tensile axial *load*, the member must also conform to the provisions of *Specification* Sections C3, D4 and D6.1. Care must be taken not to overestimate the tensile *load*, as this could be unconservative.

C5.1.1 ASD Method

Specification Equation C5.1.1-1 provides a design criterion to prevent yielding of the tension *flange* of a member under combined tensile axial *load* and bending. *Specification* Equation C5.1.1-2 provides a design criterion to prevent failure of the compression *flange*.

C5.1.2 LRFD and LSD Methods

Similar to the *ASD* method, two interaction equations are included in *Specification* Section C5.1.2 for the *LRFD* and the *LSD* methods. *Specification* Equations C5.1.2-1 and C5.1.2-2 are used to prevent the failure of the tension *flange* and compression *flange*, respectively. In both equations, different symbols are used for the *required tensile axial strength* [tensile axial force due to *factored loads*] and the *required flexural strength* [moment due to *factored loads*] according to the *LRFD* and the *LSD* methods.

C5.2 Combined Compressive Axial Load and Bending

Cold-formed steel members under a combination of compressive axial *load* and bending are usually referred to as beam-columns. The bending may result from eccentric loading, transverse *loads*, or applied moments. Such members are often found in framed structures, trusses, and exterior wall studs. For the design of such members, interaction equations have been developed for locally stable and unstable beam-columns on the basis of thorough comparison with rigorous theory and verified by the available test results (Peköz, 1986a; Peköz and Sumer, 1992).

The structural behavior of beam-columns depends on the shape and dimensions of the cross-section, the location of the applied eccentric *load*, the column length, the end restraint, and the condition of bracing. In this edition of the *Specification*, the *ASD* method is included in Section C5.2.1. *Specification* Section C5.2.2 is for the *LRFD* and the *LSD* methods.

In 2007, the *Specification* introduced the *second-order analysis* approach as an optional method of stability analysis. This new method is provided in Appendix 2 and specifies the use of a geometrically non-linear *second-order analysis* for determining the required moments and axial *loads* for use in *Specification* Sections C5.2.1 and C5.2.2. The moments and axial *loads* are the maximums in a member. Appendix 2 also specifies the values for K_x, K_y, α_x, α_y, C_{mx}

and C_{my} to be used. Detailed discussion is provided in the commentary on Appendix 2.

The previous effective length approach is still permitted. In this case, the required moments and axial forces for the *ASD* method and the *required strengths* [moments and axial forces due to *factored loads*] for the *LRFD* and *LSD* methods are derived from a *first-order* elastic *analysis* and stability effects are accounted for by choosing appropriate K-factors in combination with α_x, α_y, C_{mx} and C_{my} calculated in accordance with *Specification* Sections C5.2.1 and C5.2.2.

To avoid situations of the *load* $\Omega_c P$ (or $\overline{P}$) exceeding the Euler *buckling* load P_E, the amplification factor α is limited to a positive value in the 2007 *Specification*.

C5.2.1 ASD Method

When a beam-column is subject to an axial *load* P and end moments M as shown in Figure C-C5.2-1(a), the combined axial and bending *stress* in compression is given in Equation C-C5.2.1-1 as long as the member remains straight:

$$f = \frac{P}{A} + \frac{M}{S} \qquad \text{(C-C5.2.1-1)}$$

$$= f_a + f_b$$

where

f = combined *stress* in compression
f_a = axial compressive *stress*
f_b = bending stress in compression
P = applied axial *load*
A = cross-sectional area
M = bending moment
S = section modulus

It should be noted that in the design of such a beam-column by using the *ASD* method, the combined *stress* should be limited by certain allowable *stress* F, that is,

$$f_a + f_b \le F$$

or

$$\frac{f_a}{F} + \frac{f_b}{F} \le 1.0 \qquad \text{(C-C5.2.1-2)}$$

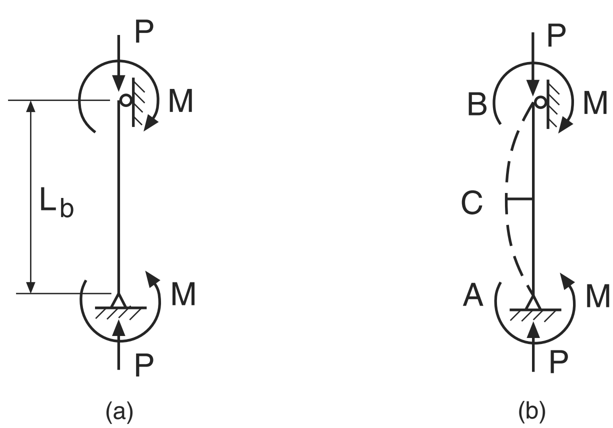


Figure C-C5.2-1 Beam-Column Subjected to Axial Loads and End Moments

As specified in Sections C3.1, D6.1 and C4 of the *Specification*, the *safety factor* Ω_c for the design of compression members is different from the *safety factor* Ω_b for beam design. Therefore, Equation C-C5.2.1-2 may be modified as follows:

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \le 1.0 \qquad \text{(C-C5.2.1-3)}$$

where

F_a = allowable *stress* for the design of compression members

F_b = allowable *stress* for the design of beams

If the strength ratio is used instead of the *stress* ratio, Equation C-C5.2.1-3 can be rewritten as follows:

$$\frac{P}{P_a} + \frac{M}{M_a} \le 1.0 \qquad \text{(C-C5.2.1-4)}$$

where

P = applied axial *load* = Af_a

P_a = allowable axial load = AF_a

M = applied moment = Sf_b

M_a = allowable moment = SF_b

According to Equation C-A4.1.1-1,

$$P_a = \frac{P_n}{\Omega_c}$$

$$M_a = \frac{M_n}{\Omega_b}$$

In the above equations, P_n and Ω_c are given in *Specification* Sections C4 and D6.1, while M_n and Ω_b are specified in *Specification* Sections C3.1 and D6.1. Substituting the above expressions into Equation C-C5.2.1-4, the following interaction equation (*Specification* Equation C5.2.1-3) can be obtained:

$$\frac{\Omega_c P}{P_n} + \frac{\Omega_b M}{M_n} \le 1.0 \qquad \text{(C-C5.2.1-5)}$$

Equation C-C5.2.1-4 is a well-known interaction equation which has been adopted in several specifications for the design of beam-columns. It can be used with reasonable accuracy for short members and members subjected to a relatively small axial *load*. It should be realized that in practical applications, when end moments are applied to the member, it will be bent as shown in Figure C-C5.2-1(b) due to the applied moment M and the secondary moment resulting from the applied axial *load* P and the deflection of the member. The maximum bending moment at mid-length (point C) can be represented by

$$M_{max} = \Phi M \qquad \text{(C-C5.2.1-6)}$$

where

M_{max} = maximum bending moment at mid-length

M = applied end moments

Φ = amplification factor

It can be shown that the amplification factor Φ may be computed by

$$\Phi = \frac{1}{1 - P/P_E} \qquad \text{(C-C5.2.1-7)}$$

where P_E = elastic column *buckling* load (Euler load) = $\pi^2EI/(KL_b)^2$. Applying a *safety factor* Ω_c to P_E, Equation C-C5.2.1-7 may be rewritten as

$$\Phi = \frac{1}{1 - \Omega_c P/P_E} \qquad \text{(C-C5.2.1-8)}$$

If the maximum bending moment M_{max} is used to replace M, the following interaction equation can be obtained from Equations C-C5.2.1-5 and C-C5.2.1-8:

$$\frac{\Omega_c P}{P_n} + \frac{\Omega_b M}{(1 - \Omega_c P/P_E)M_n} \leq 1.0 \qquad \text{(C-C5.2.1-9)}$$

It has been found that Equation C-C5.2.1-9, developed for a member subjected to an axial compressive *load* and equal end moments, can be used with reasonable accuracy for braced members with unrestrained ends subjected to an axial *load* and a uniformly distributed transverse *load*. However, it could be conservative for compression members in unbraced frames (with sidesway), and for members bent in reverse curvature. For this reason, the interaction equation given in Equation C-C5.2.1-9 should be further modified by a coefficient, C_m, as shown in Equation C-C5.2.1-10, to account for the effect of end moments:

$$\frac{\Omega_c P}{P_n} + \frac{\Omega_b C_m M}{\alpha M_n} \leq 1.0 \qquad \text{(C-C5.2.1-10)}$$

The above equation is *Specification* Equation C5.2.1-1, in which $\alpha = 1 - \Omega_c P/P_E$.

In Equation C-C5.2.1-10, C_m can be determined for one of the three cases defined in *Specification* Section C5.2.1. For Case 1, C_m is given as 0.85. In Case 2, it can be computed by Equation C-C5.2.1-11 for restrained compression members braced against *joint* translation and not subject to transverse loading:

$$C_m = 0.6 - 0.4\frac{M_1}{M_2} \qquad \text{(C-C5.2.1-11)}$$

where M_1/M_2 is the ratio of smaller to the larger end moments. For Case 3, C_m may be approximated by using the value given in the AISC Commentaries for the applicable condition of transverse loading and end restraint (AISC, 1989, 1999, 2005, and 2010).

Figure C-C5.2-2 illustrates the interaction relation. In order to simplify the illustration, bending about only one axis is considered in Figure C-C5.2-2 and the *safety factors*, Ω_c and Ω_b, are taken as unity. The ordinate is the compressive axial *load* on the member and the abscissa is the bending moment. When the moment is zero, the limiting axial load is P_n determined in accordance with *Specification* Section C4, which is based on column *buckling* and *local buckling*. When the axial *load* is zero, the limiting moment, M_n, is determined in accordance with *Specification* Sections C3 and D6.1 and is the lowest of the effective *yield moment*, the moment based on inelastic reserve capacity (if applicable) or the moment based on *lateral-torsional buckling*. The interaction relation cannot exceed either of these limits.

When *Specification* Equation C5.2.1-1 is plotted in Figure C-C5.2-2, the axial load limit is P_n and the moment limit is M_n/C_m, which will exceed M_n when $C_m < 1$. Therefore, *Specification* Equation C5.2.1-2 is used as a mathematical stratagem to limit the moment to M_n and match the rigorous solution at low axial *loads*. The interaction limit is the lower of the two equations as shown by hash marks. *Specification* Equation C5.2.1-2 is a linear relation between the *nominal axial yield strength* [*resistance*] $P_{no} = F_yA_e$ and M_n, and does not represent a failure state over its whole range. If *Specification* Equation C5.2.1-2 uses the moment capacity based only on yield or *local buckling*, $M_{no} = F_yS_{eff}$, it would be represented by the dashed line, which could exceed an M_n limit based on *lateral-torsional buckling*. Clearly, *load* combinations in the shaded region would be unconservative. If M_n is determined by M_{no}, the relation in Figure C-C5.2-2 still applies. If $C_m/\alpha \geq 1$, *Specification* Equation C5.2.1-1 controls.

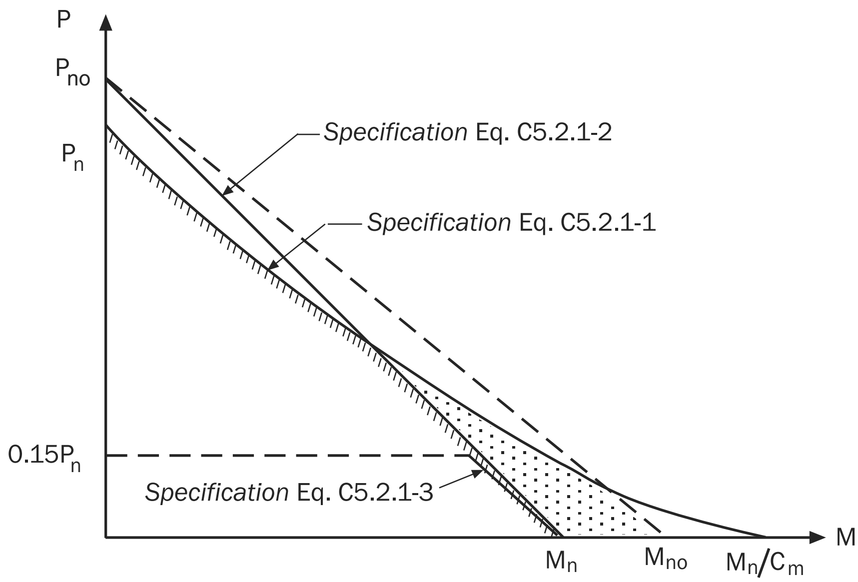


Figure C-C5.2-2 Interaction Relations

For low axial *loads*, *Specification* Equation C5.2.1-3 may be used. This is a conservative simplification of the interaction relation defined by *Specification* Equations C5.2.1-1 and C5.2.1-2.

In 2001, a requirement of each individual ratio in *Specification* Equations C5.2.1-1 to C5.2.1-3 not exceeding unity was added to avoid situations of the *load* Ω_cP exceeding the Euler *buckling* load P_E, which leads to amplification factor Φ (given in Equation C-C5.2.1-8) negative.

For the design of angle sections using the *ASD* method, the required additional bending moment of PL/1000 about the minor principal axis is discussed in Item E of Section C4 of the *Commentary*.

C5.2.2 LRFD and LSD Methods

The *LRFD* and the *LSD* methods use the same interaction equations as the *ASD* method, except that ϕ_cP_n and ϕ_bM_n are used for *design strengths* [*factored resistances*]. In addition, the *required axial strength* [compressive force due to *factored loads*], P_u or P_f, and the *required flexural strength* [moment due to *factored loads*], M_u or M_f, are to be determined from *factored loads* according to the requirements of Section A5.1.2 of the *Specification* Appendix A for U.S. and Mexico, and Appendix B for Canada. In *Specification* Equations C5.2.2-1 through C5.2.2-3, symbols $\overline{P}$ and $\overline{M}$ are used for the *required compressive axial strength* [compressive force due to *factored loads*] and the *required flexural strength* [moment due to *factored loads*] for both the *LRFD* and the *LSD* methods.

It should be noted that, as compared with the 1991 edition of the AISI *LRFD Specification*, the definition of factor α was changed in the AISI 1996 by eliminating the ϕ_c term because the term P_E is a deterministic value and hence does not require a *resistance factor*.

The interaction equations used in *Specification* Section C5.2.2 are the same as those used in the AISI *LRFD* Specification (AISI, 1991), but they are different as compared with the AISC Specifications (AISC, 1999, 2005, and 2010) due to the lack of sufficient evidence for cold-formed steel columns to adopt the AISC criteria.

Similar to *Specification* Section C5.2.1, ASD Method, the requirement of each individual ratio in *Specification* Equations C5.2.2-1 to C5.2.2-3 not exceeding unity was added in 2001.

For the design of angle sections using the *LRFD* and the *LSD* methods, the required additional bending moment of PL/1000 about the minor principal axis was discussed in Item E of Section C4 of the *Commentary*.

D. STRUCTURAL ASSEMBLIES AND SYSTEMS

D1 Built-Up Sections

I-sections made by connecting two C-sections back-to-back are one type of built-up section that is often used as either flexural or compression members. Cases (2) and (8) of Figure C-A1.3-2 and Cases (3) and (7) of Figure C-A1.3-3 show several built-up I-sections. For built-up flexural members, the *Specification* is limited to two back-to-back C-sections. For built-up compression members, other sections can be used.

D1.1 Flexural Members Composed of Two Back-to-Back C-Sections

For the I-sections to be used as flexural members, the longitudinal spacing of connectors is limited by Equation D1.1-1 of the *Specification*. The first requirement is an arbitrarily selected limit to prevent any possible excessive distortion of the top *flange* between connectors. The second requirement is based on the strength and arrangement of connectors and the intensity of the *load* acting on the beam (Yu and LaBoube, 2010).

The second requirement for maximum spacing of connectors required by *Specification* Equation D1.1-1 is based on the fact that the shear center of the C-section is neither coincident with nor located in the plane of the *web*; and that when a *load*, Q, is applied in the plane of the *web*, it produces a twisting moment, Qm, about its shear center, as shown in Figure C-D1.1-1. The tensile force of the top connector T_s can then be computed from the equality of the twisting moment, Qm, and the resisting moment, $T_s g$, that is:

$$Qm = T_s g \tag{C-D1.1-1}$$

$$T_s = \frac{Qm}{g} \tag{C-D1.1-2}$$

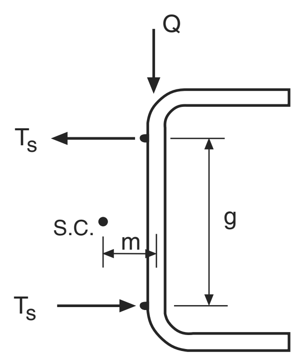


Figure C-D1.1-1 Tensile Force Developed in the Connector for C-Section

Considering that q is the intensity of the *load* and that s is the spacing of connectors as shown in Figure C-D1.1-2, the applied *load* is Q=qs/2. The maximum spacing, s_{max}, used in the *Specification* can easily be obtained by substituting the above value of Q into Equation C-D1.1-2 of this *Commentary*. The determination of the *load* intensity q is based upon the type of loading applied to the beam. The requirement of three times the uniformly distributed *load* is applied to reflect that the assumed uniform *load* will not really be uniform. The *Specification* prescribes a conservative estimate of the applied loading to account for the likely concentration of *loads* near the welds or other connectors that join the two C-sections.

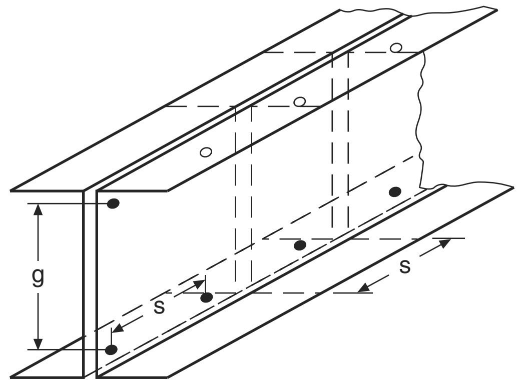


Figure C-D1.1-2 Spacing of Connectors

For simple C-sections without stiffening lips at the outer edges,

$$m = \frac{w_f^{\,2}}{2w_f + d/3} \qquad \text{(C-D1.1-3)}$$

For C-sections with stiffening lips at the outer edges,

$$m = \frac{w_f dt}{4I_x}\left[w_f d + 2D\left(d - \frac{4D^2}{3d}\right)\right] \qquad \text{(C-D1.1-4)}$$

where

w_f = projection of *flanges* from the inside face of the *web* (For C-sections with *flanges* of unequal width, w_f should be taken as the width of the wider *flange*)

d = depth of C-section or beam

D = overall depth of lip

I_x = moment of inertia of one C-section about its centroidal axis normal to the *web*

In addition to the above considerations on the *required strength* [force due to *factored loads*] of *connections*, the spacing of connectors should not be so great as to cause excessive distortion between connectors by separation along the top *flange*. In view of the fact that C-sections are connected back-to-back and are continuously in contact along the bottom *flange*, a maximum spacing of L/3 may be used. Considering the possibility that one connection may be defective, a maximum spacing of s_{max} = L/6 is the first requirement in *Specification* Equation D1.1-1.

D1.2 Compression Members Composed of Two Sections in Contact

Compression members composed of two shapes joined together at discrete points have a reduced shear rigidity. The influence of this reduced shear rigidity on the *buckling stress* is taken into account by modifying the slenderness ratio used to calculate the elastic critical *buckling stress* (Bleich, 1952). The overall slenderness and the local slenderness between connected points both influence the compressive resistance. The combined action is expressed by the modified slenderness ratio given by the following:

$$\left(\frac{KL}{r}\right)_m = \sqrt{\left(\frac{KL}{r}\right)_o^2 + \left(\frac{a}{r_i}\right)^2} \qquad \text{(C-D1.2-1)}$$

Note that in this expression, the overall slenderness ratio, $(KL/r)_o$, is computed about the same axis as the modified slenderness ratio, $(KL/r)_m$. Further, the modified slenderness ratio, $(KL/r)_m$, replaces KL/r in the *Specification* Section C4 for both flexural and *flexural-torsional buckling*.

This modified slenderness approach is used in other steel standards, including the AISC (AISC, 1999, 2005 and 2010), CSA S136 (CSA S136, 1994), and CAN/CSA S16.1 (CAN/CSA S16.1-94, 1994).

To prevent the *flexural buckling* of the individual shapes between intermediate connectors, the intermediate fastener spacing, a, is limited such that a/r_i does not exceed one-half the governing slenderness ratio of the built-up member (i.e. $a/r_i \leq 0.5(KL/r)_o$). This intermediate fastener spacing requirement is consistent with the previous edition of the AISI *Specification* with the one-half factor included to account for any one of the connectors becoming loose or ineffective. Note that the previous edition of S136 (S136, 1994) had no limit on fastener spacing.

The importance of preventing shear slip in the end connection is addressed by the prescriptive requirements in *Specification* Section D1.2(b) adopted from the AISC (AISC, 1999) and CAN/CSA S16.1 (CAN/CSA S16.1-94, 1994). These provisions were added to the *North American Specification* since 2001.

The intermediate fastener(s) or weld(s) at any longitudinal member tie location is required, as a group, to transmit a force equal to 2.5 percent of the *nominal axial strength* [*resistance*] of the built-up member. A longitudinal member tie is defined as a location of interconnection of the two members in contact. In the 2001 edition of the *Specification*, a 2.5 percent total force determined in accordance with appropriate *load* combinations was used for design of the intermediate fastener(s) or weld(s). This requirement was adopted from CSA S136-94. In 2004, the requirement was changed to be a function of the *nominal axial strength* [*resistance*]. This change ensures that the *nominal axial strength* [*resistance*] of the built-up member is valid and is not compromised by the strength of the member interconnections. To avoid confusion for different design methods, the minimum *required strength* [force due to *factored loads*] of the interconnection changed to 2.5 percent of the *available strength* [*factored resistance*] of the built-up member.

Note that the provision in *Specification* Section D1.2 has been substantially taken from research in hot-rolled built-up members connected with bolts or welds. These hot-rolled provisions have been extended to include other fastener types common in cold-formed steel construction (such as screws) provided they meet the 2.5 percent requirement for shear strength and the conservative spacing requirement $a/r_i \leq 0.5(KL/r)_o$.

D1.3 Spacing of Connections in Cover-Plated Sections

When compression elements are joined to other parts of built-up members by intermittent *connections*, these connectors must be closely spaced to develop the required strength of the connected element. Figure C-D1.3-1 shows a box-shaped beam made by connecting a flat sheet to an inverted hat section. If the connectors are appropriately placed, this flat sheet will act as a stiffened compression element with a width, w, equal to the distance between rows of connectors, and the sectional properties can be calculated accordingly. This is the intent of the provisions in Section D1.3 of the *Specification*.

Section D1.3(a) of the *Specification* requires that the necessary shear strength be provided by the same standard structural design procedure that is used in calculating *flange connections* in bolted or welded plate girders or similar structures.

Section D1.3(b) of the *Specification* ensures that the part of the flat sheet between two adjacent connectors will not buckle as a column (see Figure C-D1.3-1) at a *stress* less than $1.67f_c$, where f_c is the *stress* at *service load* in the connected compression element (Winter, 1970; Yu and LaBoube, 2010). The AISI requirement is based on the following Euler equation for column *buckling*:

$$\sigma_{cr} = \frac{\pi^2 E}{(KL/r)^2}$$

by substituting $\sigma_{cr} = 1.67f_c$, $K = 0.6$, $L = s$, and $r = t/\sqrt{12}$. This provision is conservative because the length is taken as the center distance instead of the clear distance between connectors, and the coefficient K is taken as 0.6 instead of 0.5, which is the theoretical value for a column with fixed end supports.

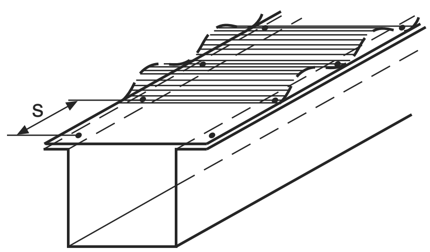


Figure C-D1.3-1 Spacing of Connectors in Composite Section

Section D1.3(c) ensures satisfactory spacing to make a row of connectors act as a continuous line of stiffening for the flat sheet under most conditions (Winter, 1970; Yu and LaBoube, 2010).

Specification Section B2.5 extends the limits of this section and uses the post-*buckling* strength of the edge-stiffened compression plate. *Specification* Section B2.5 specifies the parameter ranges that are validated by the research (Luttrell and Balaji, 1992; Snow and Easterling, 2008).

D2 Mixed Systems

When cold-formed steel members are used in conjunction with other construction materials, the design requirements of the other material specifications must also be satisfied.

D3 Lateral and Stability Bracing

Bracing design requirements were expanded in the 1986 AISI *Specification* to include a general statement regarding bracing for symmetrical beams and columns and specific requirements for the design of roof systems subjected to gravity *load*. These requirements are retained in this *Specification.* ☜B

Brace points are to provide lateral and torsional restraints to the top and bottom *flanges* of C- and Z-sections to resist the tendency of Z-sections to translate laterally, and the tendency of both Z- and C-sections to twist due to eccentrically applied *loads*. By restraining both lateral displacement and torsional rotation, *second-order effects* are minimized. Lateral bracing may be provided by lateral bracing, torsional bracing or a combination of the two.

D3.1 Symmetrical Beams and Columns

There are no simple, generally accepted techniques for determining the *required strength* [effect due to *factored loads*] and stiffness for discrete braces in steel construction. Winter (1960) offered a partial solution and others have extended this knowledge (Haussler, 1964; Haussler and Pahers, 1973; Lutz and Fisher, 1985; Salmon and Johnson, 1990; Yura, 1993; SSRC, 1993). The design engineer is encouraged to seek out the stated references to obtain guidance for design of a brace or brace system.

☜B

D3.2 C-Section and Z-Section Beams

C-sections and Z-sections used as beams to support transverse *loads* applied in the plane of the *web* may twist and deflect laterally unless adequate lateral supports are provided. Section D3.2 of the *Specification* includes the requirements for spacing and design of braces, when neither *flange* of the beam is braced by deck or sheathing material. The bracing requirements for members having one *flange* connected to deck or sheathing materials are provided in D6.3.1.

☜B

D3.2.1 Neither Flange Connected to Sheathing That Contributes to the Strength and Stability of the C- or Z- Section

(a) Bracing of C-Section Beams

If C-sections are used singly as beams, rather than being paired to form I-sections, they should be braced at intervals so as to prevent them from rotating in the manner indicated in Figure C-D3.2.1-1. Figure C-D3.2.1-2, for simplicity, shows two C-sections braced at intervals against each other. The situation is evidently much the same as in the composite I-section of Figure C-D1.1-2, except that the role of the connectors is now played by the braces. The difference is that the two C-sections are not in contact, and that the spacing of braces is generally considerably larger than the connector spacing. In consequence, each C-section may actually rotate very slightly between braces, and this will cause some additional *stresses*, which superimpose on the usual, simple bending *stresses*. Bracing should be so arranged that: (1) these additional *stresses* are small enough not to reduce the *load*-carrying capacity of the C-section (as compared to what it would be in the continuously braced condition), and (2) rotations should be kept small enough to be unobjectionable on the order of 1 to 2 degrees.

In order to obtain the information for developing bracing provisions, different C-section shapes were tested at Cornell University (Winter, 1970). Each of these was tested with full, continuous bracing; without any bracing; and with intermediate bracing at two different spacings. In addition to this experimental work, an approximate method of analysis was developed and checked against the test results. A condensed account of this work was given by Winter, Lansing and McCalley (1949b). It is indicated in the reference that the above requirements are satisfied for most distributions of beam *load* if between supports not less than three equidistant braces are placed (i.e., at quarter-points of the span, or closer). The exception is the case where a large part of the total *load* of the beam is concentrated over a short portion of the span; in this case, an additional brace should be placed at such a *load*. Correspondingly, previous editions of the AISI *Specification* (AISI, 1986; AISI, 1991) provided that the distance between braces should not be greater than one-quarter of the span and

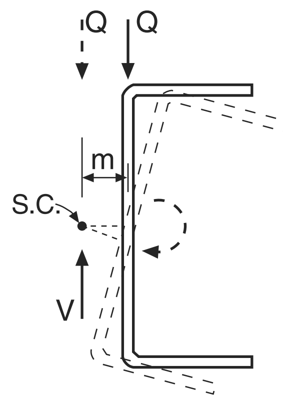


Figure C-D3.2.1-1 Rotation of C-Section Beams

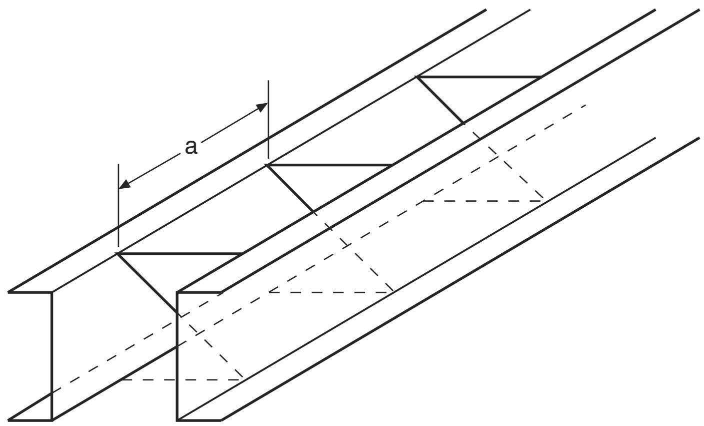


Figure C-D3.2.1-2 Two C-Sections Braced at Intervals Against Each Other

defined the conditions under which an additional brace should be placed at a *load* concentration.

For such braces to be effective, it is not only necessary that their spacing be appropriately limited and their strength should suffice to provide the force required to prevent the C-section from rotating. It is, therefore, also necessary to determine the forces that will act in braces, such as those forces shown in Figure C-D3.2.1-3. These forces are found if one considers that the action of a *load* applied in the plane of the *web* (which causes a torque Qm) is equivalent to that same *load* when applied at the shear center (where it causes no torque) plus two forces P = Qm/d which, together, produce the same torque Qm. As is sketched in Figure C-D3.2.1-4 and shown in some detail by Winter, Lansing and McCalley (1949b), each half of the channel can then be regarded as a continuous beam loaded by the horizontal forces and supported at the brace points. The horizontal brace force is then, simply, the appropriate reaction of this continuous beam. The provisions of *Specification* Section D3.2.1 provide expressions for determining bracing forces P_{L1} and P_{L2}, which the braces are required to resist at each *flange*.

(b) Bracing of Z-Section Beams

Most Z-sections are anti-symmetrical about the vertical and horizontal centroidal axes; i.e., they are point-symmetrical. In view of this, the centroid and the shear center coincide and are located at the midpoint of the *web*. A *load* applied in the plane of the *web* has, then, no lever arm about the shear center (m = 0) and does not tend to produce the kind of rotation that a similar *load* would produce on a C-section. However, in Z-sections the principal axes are oblique to the *web* (Figure C-D3.2.1-5). A *load* applied in the plane of the *web*, resolved in the direction of the two axes, produces deflections along each of them. By projecting these deflections onto the horizontal and vertical planes, it is found that a Z-beam loaded vertically in the plane of the *web* deflects not only vertically but also horizontally. If such deflection is permitted to occur, then the *loads*, moving sideways with the beam, are no longer in the same plane with the reactions at the ends. In consequence, the *loads* produce a twisting moment about the line connecting the reactions. In this manner it is seen that a Z-beam, unbraced between ends and loaded in the plane of the *web*, deflects laterally and also twists. Not only are these deformations likely to interfere with the proper functioning of the beam, but the additional *stresses* caused by them produce failure at a *load* considerably lower than when the same beam is used fully braced.

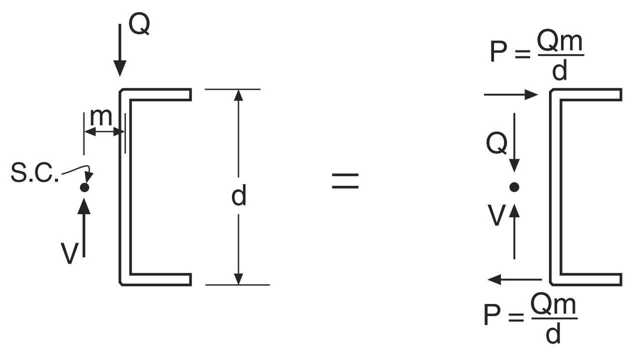


Figure C-D3.2.1-3 Lateral Forces Applied to C-Section

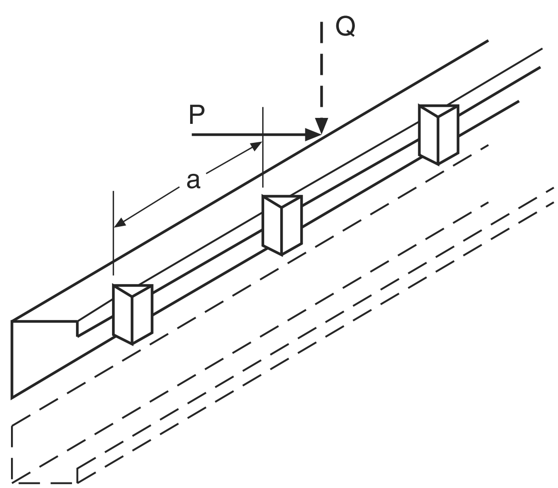


Figure C-D3.2.1-4 Half of C-Section Treated as a Continuous Beam Loaded by Horizontal Forces

In order to obtain information for developing appropriate bracing provisions, tests have been carried out on three different Z-sections at Cornell University, unbraced as well as with variously spaced intermediate braces. In addition, an approximate method of analysis has been developed and checked against the test results. An account of this work was given by Zetlin and Winter (1955b). Briefly, it is shown that intermittently braced Z-beams can be analyzed in much the same way as intermittently braced C-beams. It is merely necessary, at the point of each actual vertical *load* Q, to apply a fictitious horizontal *load*, $Q(I_{xy}/I_x)$ or $Q[I_{xy}/(2I_x)]$, to each *flange*. One can then compute the vertical and horizontal deflections, and the corresponding *stresses*, in conventional ways by utilizing the convenient axes x and y (rather than 1 and 2, Figure C-D3.2.1-5), except that certain modified section properties have to be used. To control the lateral deflection, brace forces, P, must statically balance the fictitious force.

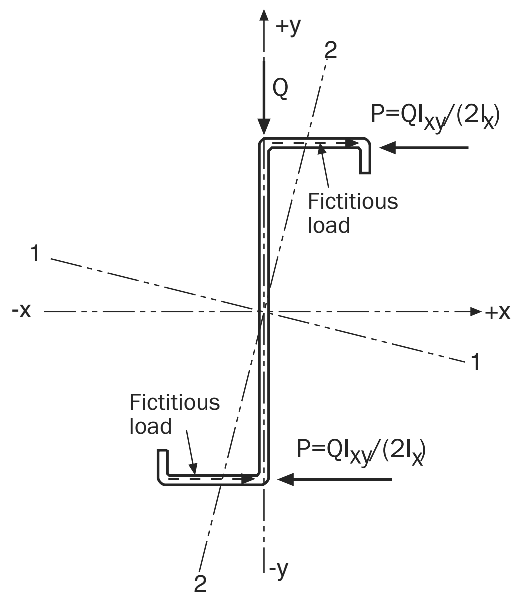


Figure C-D3.2.1-5 Principal Axis of Z-Section

In this manner it has been shown that as to location of braces, the same provisions that apply to C-sections are also adequate for Z-sections. Likewise, the forces in the braces are again obtained as the reactions of continuous beams horizontally loaded by fictitious loads, P. It should, however, be noted that the direction of the bracing forces in Z-beams is different from the direction in C-beams. In the Z-beam, the bracing forces are acting in the same direction, as shown in Figure C-D3.2.1-5, in order to constrain bending of the section about the axis x-x. The directions of the bracing forces in the C-beam *flanges* are in the opposite direction, as shown in Figure C-D3.2.1-3, in order to resist the torsion caused by the applied *load*. In the previous edition of the *Specification*, the magnitude of the Z-beam bracing force was shown as $P = Q(I_{xy}/I_x)$ on each *flange*. In 2001, this force was corrected to $P = Q[I_{xy}/(2I_x)]$.

(c) *Slope Effect and Eccentricity*

For a C- or Z-section member subjected to an arbitrary *load*, bracing forces, P_{L1} and P_{L2}, on *flanges* need to resist: (1) force component P_x that is perpendicular to the *web*, (2) the torsion caused by eccentricity about the shear center, and (3) for the Z-section member, the lateral movement caused by component P_y, that is parallel to the *web*.

To develop a set of equations applicable to any loading conditions, the x and y axes are oriented such that one of the *flanges* is located in the quadrant with both x and y axes positive. Since the torsion should be calculated about the shear center, coordinates

x_s and y_s, that go through the shear center and parallel to x and y axes, are established. *Load* eccentricities e_x and e_y should be measured based on x_s and y_s coordinate system.

For the C-section member as shown in Figure C-D3.2.1-6, the bracing forces on both *flanges* are given in Equations C-D3.2.1-1 and C-D3.2.1-2.

$$P_{L1} = -\frac{P_x}{2} + \frac{M_z}{d} \quad \text{(C-D3.2.1-1)}$$

$$P_{L2} = -\frac{P_x}{2} - \frac{M_z}{d} \quad \text{(C-D3.2.1-2)}$$

$$M_z = -P_x e_{sy} + P_y e_{sx} \quad \text{(C-D3.2.1-3)}$$

where d = overall depth of the *web*; e_{sx}, e_{sy} = eccentricities of *design load* about the shear center in x_s- and y_s-direction, respectively; P_x, P_y = components of *design load* in x- and y-direction, respectively; M_z = torsional moment about the shear center; and P_{L1} = bracing force applied to the *flange* located in the quadrant with both positive x and y axes, and P_{L2} = bracing force applied on the other *flange*. Positive P_{L1} and P_{L2} indicate that a restraint is required to prevent the movement of the corresponding *flange* in the negative x-direction.

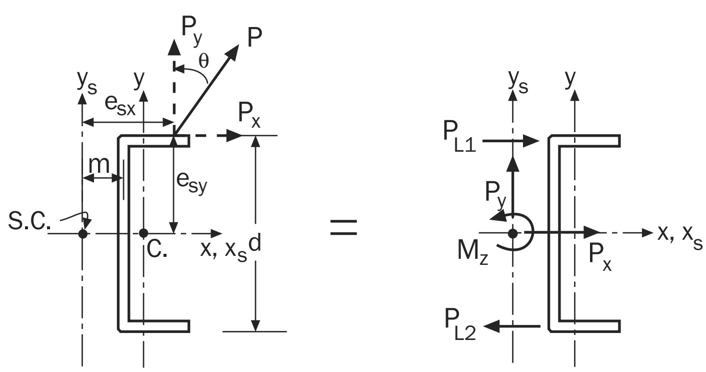


Figure C-D3.2.1-6 C-Section Member Subjected to a Concentrated Load

For a special case where *design load*, Q, is through the *web*, as shown in Figure C-D3.2.1-3, P_y = -Q, P_x = 0; e_{sx} = m, e_{sy} = d/2, and from Equation C-D3.2.1-3, M_z = -Qm. Therefore:

P_{L1} = -Qm/d (C-D3.2.1-4)

P_{L2} = Qm/d (C-D3.2.1-5)

In which, m = distance from centerline of *web* to the shear center.

For the Z-section member as shown in Figure C-D3.2.1-7, bracing forces, P_{L1} and P_{L2}, are given in Equations C-D3.2.1-6 and C-D3.2.1-7.

$$P_{L1} = P_y\left(\frac{I_{xy}}{2I_x}\right) - \frac{P_x}{2} + \frac{M_z}{d} \quad \text{(C-D3.2.1-6)}$$

$$P_{L2} = P_y\left(\frac{I_{xy}}{2I_x}\right) - \frac{P_x}{2} - \frac{M_z}{d} \quad \text{(C-D3.2.1-7)}$$

where I_x, I_{xy} = unreduced moment of inertia and product of inertia, respectively. Other variables are defined under the discussion for C-section members.

Assuming that a gravity *load*, P, acts at 1/3 of the top *flange* width, b_f, and the Z-section member rests on a sloped roof with an angle of θ, P_x = -Psinθ; P_y = -Pcosθ; e_{sx} = b_f/3; e_{sy} = d/2 and M_z = Psinθ(d/2) - Pcosθ(b_f/3). Substituting the above expressions into Equations C-D3.2.1-6 and C-D3.2.1-7 results in:

$$P_{L1} = -P\cos\theta\left(\frac{I_{xy}}{2I_x}\right) + P\sin\theta - \frac{Pb_f\cos\theta}{3d}$$

$$P_{L2} = -P\cos\theta\left(\frac{I_{xy}}{2I_x}\right) + \frac{Pb_f\cos\theta}{3d}$$

In considering the distribution of *loads* and the braces along the member length, it is required that the resistance at each brace location along the member length be greater than or equal to the *design load* within a distance of 0.5a on each side of the brace for distributed loads. For concentrated loads, the resistance at each brace location should be greater than or equal to the concentrated *design load* within a distance 0.3a on each side of the brace, plus 1.4(1-*l*/a) times each *design load* located farther than 0.3a but not farther than 1.0a from the brace. In the above, a is the distance between centerline of braces along the member length and *l* is the distance from concentrated *load* to the brace to be considered.

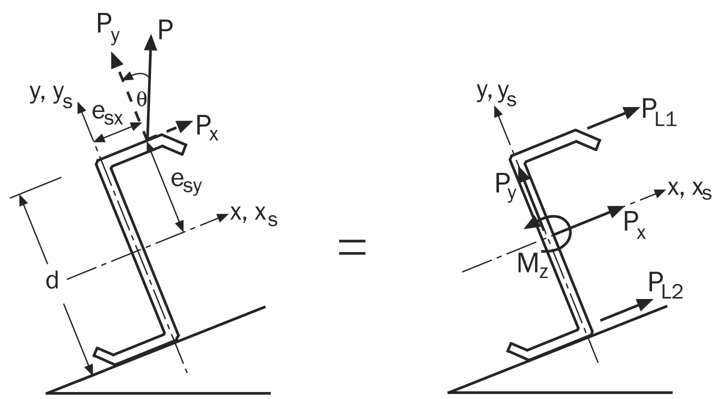


Figure C-D3.2.1-7 A Z-Section Member Subjected to an Arbitrary Load

(d) Spacing of Braces

During the period from 1956 through 1996, the AISI *Specification* required that braces be attached both to the top and bottom *flanges* of the beam, at the ends and at intervals not greater than one-quarter of the span length, in such a manner as to prevent tipping at the ends and lateral deflection of either *flange* in either direction at intermediate braces. The *lateral-torsional buckling* equations provided in *Specification* Section C3.1.2.1 can be used to predict the moment capacity of the member. Beam tests conducted by Ellifritt, Sputo and Haynes (1992) have shown that for typical sections, a mid-span brace may reduce *service load* horizontal deflections and rotations by as much as 80 percent when compared to a completely unbraced beam. However, the restraining effect of braces may change the failure mode from *lateral-torsional buckling* to *distortional buckling* of the *flange* and lip at a brace point. The natural tendency of the member under vertical *load* is to twist and translate in such a manner as to relieve the compression on the lip. When such movement is restrained by intermediate braces, the compression on the stiffening lip is not relieved, and may increase. In this case, local *distortional buckling* may occur at *loads* lower than that predicted by the *lateral-torsional buckling* equations of *Specification* Section C3.1.2.1.

Research (Ellifritt, Sputo and Haynes, 1992) has also shown that the *lateral-torsional buckling* equations of *Specification* Section C3.1.2.1 predict *loads*, which are conservative for cases where one mid-span brace is used but may be unconservative where more than one intermediate brace is used. Based on such research findings, Section D3.2.1 of the *Specification* was revised in 1996 to eliminate the requirement of quarter-point bracing. It is suggested that, minimally, a mid-span brace be used for C-section and Z-section beams to control lateral deflection and rotation at *service loads*. The *lateral-torsional buckling* strength of an open cross-section member should be determined by *Specification* Section C3.1.2.1 using the distance between centerlines of braces "a" as the unbraced length of the member "L" in all design equations. In any case, the user is permitted to perform tests, in accordance with *Specification* Section F1, as an alternative, or use a rigorous analysis, which accounts for biaxial bending and torsion.

Section D3.2.1 of the *Specification* provides the lateral forces for which these discrete braces must be designed.

The *Specification* permits omission of discrete braces when all *loads* and reactions on a beam are transmitted through members that frame into the section in such a manner as to effectively restrain the member against torsional rotation and lateral displacement. Frequently, this occurs in the end walls of metal buildings.

In 2007, the title of this section was changed to clarify that it is and was formerly anticipated that the C- and Z-sections covered by these provisions would be supporting sheathing and be loaded as a result of providing this support function. The revised title reflects that the supported sheathing is not contributing to the strength and stiffness of these members by virtue of the nature of its connection to the C- and Z-sections.

D3.3 Bracing of Axially Loaded Compression Members

The requirements for bracing a single compression member were developed from a study by Green et. al (2004) and adaptation of requirements in the AISC Specification (AISC, 2010). These bracing provisions are developed to ensure that an individual concentrically loaded compression member can develop the *required compressive axial strength* [compressive axial force due to *factored loads*]; however, they do not necessarily allow individual concentrically loaded compression members to develop their fully braced capacity at an effective length equal to the length between braces. The required bracing stiffness ensures that the translation at the brace point is limited until the axial *loads* equal the *required strength* [compressive axial force due to *factored loads*], P_{ra}, which is determined in accordance with the applied *load* combinations for the corresponding design method of *ASD*, *LRFD*, or *LSD*. The engineer should recognize that a column braced to these provisions has an *available strength* [*factored resistance*] equal to the *required strength* [compressive axial force due to *factored loads*], but not in excess of the *required strength* [compressive axial force due to *factored loads*]. If the engineer desires the *available column strength* [*factored resistance*] to exceed P_{ra} then the *required brace strength* [brace force due to *factored loads*] designed for P_{ra} should be increased. If the engineer desires the *available column strength* [*factored resistance*] to equal the fully braced column strength, the *required axial compressive strength* [compressive axial force due to *factored loads*], P_{ra}, in *Specification* Equations D3.3-1, D3.3-2a and D3.3-2b should be replaced by the fully braced column *available strength* [*factored resistance*], P_n/Ω_c for *ASD* or $\phi_c P_n$ for *LRFD* or *LSD*.

The requirements for brace stiffness for a single compression member are similar to the AISC provisions, with the exception that the number of braces is accounted for by including the term 2(4-(2/n)). As a simplification, AISC assumes n = infinity, but this simplification is considered too conservative for cold-formed steel structures. Analytical modeling by Sputo and Beery (2006) has shown that these provisions may be applied to members of varied cross-sections. The *safety factor* (Ω=2.0) and *resistance factor* (ϕ=0.75) for calculating required brace stiffness in *Specification* Equations D3.3-2a and D3.3-2b are the same as those used in the AISC provisions (AISC, 2010).

The brace provisions for lateral translation assume that the braces are perpendicular to the compression member being braced and located in the plane of *buckling*. For inclined brace members, the *required brace strength* [brace force due to *factored loads*] and stiffness should be increased as follows:

$$P'_{rb} = \frac{P_{rb}}{\cos\theta} \qquad \text{(C-D3.3-1)}$$

where

P'_{rb} = *Required strength* [brace force due to *factored loads*] of the inclined brace

θ = Angle of brace from perpendicular

The required stiffness is

$$\beta_{rb} = \frac{P_{rb}}{\Delta} \qquad \text{(C-D3.3-2)}$$

And the required stiffness of the inclined brace, β'_{rb}, is

$$\beta'_{rb} = \frac{P'_{rb}}{\Delta'} \qquad \text{(C-D3.3-3)}$$

$$\Delta' = \Delta\cos\theta \qquad \text{(C-D3.3-4)}$$

where

Δ' = deformation of inclined brace, and

Δ = lateral movement of brace point.

Substituting Equations C-D3.3-1, C-D3.3-2, and C-D3.3-4 into Equation C-D3.3-3,

$$\beta'_{rb} = \frac{\beta_{rb}}{\cos^2\theta} \qquad \text{(C-D3.3-5)}$$

The stiffness requirements include the contributions of the bracing members, connections, and anchorage details.

Additional bracing or additional brace strength and stiffness may be required to brace members that may also be subject to bending, torsion, or torsional-flexural *stresses*. Bracing for these effects are not accounted for in Section D3.3 and should be determined through rational analysis or other methods.

Once the *required brace strength* [brace force due to *factored loads*] and required stiffness are determined in accordance with Specification Equations D3.3-1 and D3.3-2, the brace member should then be designed in accordance with *Specification* Section A4, A5, or A6, as appropriate, and with the *safety* and *resistance factors* determined in accordance with the applicable *Specification* section.

In 2012, *second-order analysis* was introduced as an alternative method for establishing the *required strength* [brace force due to *factored loads*] and stiffness for column bracing. The analysis includes consideration of the initial out-of-straightness of the compression member as well as the bracing member properties, *connections* and anchoring details. Specific requirements are provided in Appendix 2.

D4 Cold-Formed Steel Light-Frame Construction

In 2007, the scope of Section D4 on Wall Studs and Wall Stud Assemblies of the 2001 edition of the *Specification* with 2004 Supplement was broadened to include light-frame construction. This was done in order to recognize the growing use of cold-formed steel framing in a broader range of residential and light commercial framing applications and to provide a means for either requiring or accepting use of the various ANSI-approved standards that have been developed by the AISI Committee on Framing Standards.

AISI S200, *North American Standard for Cold-Formed Steel Framing – General Provisions* addresses requirements for construction with cold-formed steel framing that are common to prescriptive and engineered design. Use of AISI S200 is mandatory for the design and installation of *structural members* utilized in cold-formed steel repetitive framing applications where the specified minimum base steel *thickness* is not greater than 118 mils (0.1180 inches) (2.997 mm) because certain requirements, such as corrosion protection, product designators, manufacturing and installation tolerances, are not addressed adequately by the *Specification*. In 2012, the reference to nonstructural members was removed from Section D4 because the provisions for nonstructural members were moved from AISI S200 to the newly developed AISI S220, *North American Standard for Cold-Formed Steel Framing – Nonstructural Members*.

The other referenced standards include the following:

(a) AISI S210, *North American Standard for Cold-Formed Steel Framing – Floor and Roof System Design* provides technical information and specifications for designing floor and roof systems made from cold-formed steel. Use of AISI S210 is optional for the design and installation of cold-formed steel framing for floor and roof systems in buildings because individual *structural members* of a floor and roof system assembly can be designed fully, albeit often less efficiently, using the *Specification* alone.

(b) AISI S211, *North American Standard for Cold-Formed Steel Framing–Wall Stud Design* provides technical information and specifications for designing wall studs made from cold-formed steel. Use of AISI S211 is optional for the design and installation of cold-formed steel studs for both structural and non-structural walls in buildings because individual *structural members* of a wall stud assembly can be designed fully, albeit often less efficiently, using the *Specification* alone. For more comments on the design and use of wall studs, see Section D4.1 of this *Commentary*.

(c) AISI S212, *North American Standard for Cold-Formed Steel Framing–Header Design* provides technical information and specifications for designing headers made from cold-formed steel. Use of AISI S212 is optional for the design and installation of cold-formed steel box and back-to-back headers, as well as double and single L-headers for *load*-carrying purposes in buildings, because individual *structural members* of a header assembly can be designed fully, albeit often less efficiently, using the *Specification* alone.

(d) AISI S213, *North American Standard for Cold-Formed Steel Framing–Lateral Design* addresses the design of lateral force-resisting systems to resist wind and seismic forces in a wide range of buildings constructed with cold-formed steel framing. Use of AISI S213 is mandatory for the design and installation of cold-formed steel 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* because certain requirements, such as design requirements specific to shear walls and diaphragms sheathed with wood structural panels, gypsum board, fiberboard and steel sheet, as well as special seismic requirements for these and systems using diagonal strap bracing, are not adequately addressed by the *Specification*.

(e) AISI S214, *North American Standard for Cold-Formed Steel Framing – Truss Design* provides technical information and specifications on cold-formed steel truss construction. Use of AISI S214 is mandatory for the design of cold-formed steel trusses for *load*-carrying purposes in buildings because certain requirements, such as design responsibilities, design requirements specific to truss assemblies using C-shape, hat-shape and Z-shape sections and gusset plates, as well as manufacturing, quality criteria, installation and testing as they relate to the design of cold-formed steel trusses, are not addressed adequately by the *Specification*.

These framing standards are available for adoption and use in the United States, Canada and Mexico, and provide an integrated treatment of *Allowable Strength Design* (*ASD*), *Load and Resistance Factor Design* (*LRFD*), and *Limit States Design* (*LSD*). These framing standards do not preclude the use of other materials, assemblies, structures or designs not meeting the criteria herein when the other materials, assemblies, structures or designs demonstrate equivalent performance for the intended use to those specified in the standards.

D4.1 All Steel Design of Wall Stud Assemblies

It is well known that column strength can be increased considerably by using adequate bracing, even though the bracing is relatively flexible. This is particularly true for those sections generally used as *load*-bearing wall studs which have large I_x/I_y ratios.

Cold-formed I-, C-, Z-, or box-type studs are generally used in walls with their *webs* placed perpendicular to the wall surface. The walls may be made of different materials such as fiberboard, pulp board, plywood, or gypsum board. If the wall material is strong enough and there is adequate attachment provided between wall material and studs for lateral support of the studs, then the wall material can contribute to the structural economy by increasing the usable strength of the studs substantially.

In order to determine the necessary requirements for adequate lateral support of the wall studs, theoretical and experimental investigations were conducted in the 1940s by Green, Winter, and Cuykendall (1947). The study included 102 tests on studs and 24 tests on a variety of wall material. Based on the findings of this earlier investigation, specific AISI provisions were developed for the design of wall studs.

In the 1970s, the structural behavior of columns braced by steel *diaphragms* was a special subject investigated at Cornell University and other institutions. The renewed investigation of wall-braced studs has indicated that the bracing provided for studs by steel panels is of the shear diaphragm type rather than the linear type, which was considered in the 1947 study. Simaan (1973) and Simaan and Peköz (1976), which are summarized by Yu (2000), contain procedures for computing the strength of C- and Z-section wall studs that are braced by sheathing materials. The bracing action is due to both the shear rigidity and the rotational restraint supplied by the sheathing material. The treatment by Simaan (1973) and Simaan and

Peköz (1976) is quite general and includes the case of studs braced on one as well as on both *flanges*. However, the provisions of Section D4 of the 1980 AISI *Specification* dealt only with the simplest case of identical sheathing material on both sides of the stud. For simplicity, only the restraint due to the shear rigidity of the sheathing material was considered.

The 1989 Addendum to the AISI 1986 *Specification* included the design limitations from the *Commentary* and introduced stub column tests and/or rational analysis for the design of studs with perforations (Davis and Yu, 1972; Rack Manufacturers Institute, 1990).

In 1996, the design provisions were revised to permit: (a) all steel design, and (b) sheathing braced design of wall studs with either solid or perforated *webs*. For sheathing braced design, in order to be effective, sheathing must retain its design strength and integrity for the expected service life of the wall. Of particular concern is the use of gypsum sheathing in a moist environment.

In 2004, the sheathing braced design provisions were removed from the *Specification* and a requirement added that sheathing braced design be based on appropriate theory, tests, or *rational engineering analysis* that can be found in AISI (2004a); Green, Winter, and Cuykendall (1947); Simaan (1973); and Simaan and Peköz (1976).

In 2007, in addition to the revisions of *Specification* Section D4 as discussed in this *Commentary*, the provisions for non-circular holes were moved from *Specification* Section D4.1 to Section B2.2 on Uniformly Compressed Stiffened Elements With Circular or Non-Circular Holes. Within the limitations stated for the size and spacing of perforations and section depth, the provisions were deemed appropriate for members with uniformly compressed stiffened elements, not just wall studs.

D5 Floor, Roof or Wall Steel Diaphragm Construction

In building construction, it has been a common practice to provide a separate bracing system to resist horizontal *loads* due to wind load, blast force, or earthquake. However, steel floor and roof panels, with or without concrete fill, are capable of resisting horizontal *loads* in addition to the bending strength for gravity loads if they are adequately interconnected to each other and to the supporting frame. The effective use of steel floor and roof decks can therefore eliminate separate bracing systems and result in a reduction of building costs. For the same reason, wall panels can provide not only enclosure surface and support normal *loads*, but they can also provide *diaphragm* action in their own planes.

The structural performance of a *diaphragm* construction can be evaluated by either calculations or tests. Several analytical procedures exist, and are summarized in the literature (Steel Deck Institute, 2004; Metal Construction Association, 2004; Department of Army, 1982[1]; and ECCS, 1977). Analytical methods depend on the capacity of the connections between the panels and structural supports. The support thickness and mechanical properties must be considered. As an example, the tilting potential of screws is discussed in *Specification* Section E4.3 and is distinct from the bearing capacity controlled by panels. When using analytical methods, refer to the applicability limits. Tested performance is measured using the procedures of ASTM E455, *Standard Method for Static Load Testing of Framed Floor, Roof and Wall Diaphragm Construction for Buildings*. AISI S907, *Test Standard for Cantilever Test Method for Cold-Formed Steel*

[1] In 2010, the reference to Department of Army, 1992 edition was reverted back to the 1982 edition due to errors that are related to deck design found in the 1992 edition.

Diaphragms (AISI, 2013e), provides the test procedures with commentary for cold-formed steel *diaphragms*. Yu and LaBoube (2010) provide a general discussion of structural *diaphragm* behavior.

The *safety factors* and *resistance factors* listed in the *Specification* are based on a recalibration of the full-scale test data summarized in the Steel Deck Institute (SDI) *Diaphragm Design Manual*, First Edition. The recalibration used the method of *Specification* Section A5.1.1 and F1.1 and the *load factors* in ASCE 7-98. The most probable *diaphragm* D/L *load* ratio is zero and this was used in the recalibration. The dominant *diaphragm* limit state is *connection*-related. Consistent with *Commentary* Section A5.1.1(b), the calibration used β_o = 3.5 for all *load effects* except wind *load*. The U.S. *LRFD* method allows β_o = 2.5 for *connections* subjected to wind *loads*. For both weld and screw calibration, using β_o = 2.5 suggests factors less severe than ϕ = 0.8 and Ω = 2.0. Because of concerns over weld quality control and to avoid significant departures from the SDI historically accepted values and the previous edition's Table D5, ϕ = 0.70 and Ω = 2.35 were conservatively selected for wind *loads*. These values more closely equate to a calibration using $\beta_o \geq 3.0$. Since *diaphragm* stiffness is typically determined from the test data at 0.4 times the *nominal load*, this selection also avoids inconsistencies between strength and stiffness service determinations.

Consistent with confidence in construction quality control and the test data, the recalibration provides a distinction between screw fasteners and welded connections for *load* combinations not involving wind loading. The calibration of resistance to seismic *loads* is based on a *load factor* of 1.6 and is consistent with AISC provisions. The *safety factor* for welded *diaphragms* subjected to earthquake loading is slightly larger than those for other loading types. That factor is also slightly larger than the recalibration suggested. The increase is due to the greater toughness demands required by seismic loading, uncertainty over *load* magnitudes, and concern over weld quality control. When the *load factor* for earthquake loading is one, the 0.7 multiplier of ASCE 7 - 98 is allowed in *ASD* and the *safety factors* of Table D5 apply. If a local code requires a seismic *load factor* of 1.6, the factors of Table D5 still apply.

The Steel Deck Institute (1987) and the Department of Army (1982) have consistently recommended a *safety factor* of two to limit "out-of-plane *buckling*" of *diaphragms*. Out-of-plane *buckling* is related to panel profile, while the other *diaphragm* limit state is *connection*-related. The remainder of the *Specification* requires different *safety* and *resistance factors* for the two limit states and larger *safety factors* for *connection*-controlled states. The *safety* and *resistance factors* for panel *buckling* were changed and the limit state being considered was clarified relative to the previous edition. The prescribed factors for out-of-plane panel *buckling* are constants for all loading types.

The *Specification* allows mechanical fasteners other than screws. The *diaphragm* shear value using any fastener must not be based on a *safety factor* less than the individual fastener shear strength *safety factor* unless: 1) sufficient data exists to establish a system effect, 2) an analytical method is established from the tests, and 3) test limits are stated.

D6 Metal Roof and Wall Systems

For members with one *flange* connected to deck or metal sheathing, the member flexural and compression strengths as well as bracing requirements are provided in *Specification* Section D6.

D6.1 Purlins, Girts and Other Members

D6.1.1 Flexural Members Having One Flange Through-Fastened to Deck or Sheathing

For beams having the tension *flange* attached to deck or sheathing and the compression *flange* unbraced, e.g., a roof *purlin* or wall girt subjected to wind suction, the bending capacity is less than a fully braced member, but greater than an unbraced member. This partial restraint is a function of the rotational stiffness provided by the panel-to-*purlin connection*. The *Specification* contains factors that represent the reduction in capacity from a fully braced condition. These factors are based on experimental results obtained for both simple and continuous span *purlins* (Peköz and Soroushian, 1981 and 1982; LaBoube, 1986; Haussler and Pahers, 1973; LaBoube, et al., 1988; Haussler, 1988; Fisher, 1996).

The R factors for simple span C-sections and Z-sections up to 8.5 inches (216 mm) in depth have been increased from the 1986 *Specification,* and a member design *yield stress* limit is added based on the work by Fisher (1996).

As indicated by LaBoube (1986), the rotational stiffness of the panel-to-*purlin connection* is primarily a function of the member *thickness*, sheet *thickness*, fastener type and fastener location. To ensure adequate rotational stiffness of the roof and wall systems designed using the AISI provisions, *Specification* Section D6.1.1 explicitly states the acceptable panel and fastener types.

Continuous beam tests were made on three equal spans and the R values were calculated from the failure loads using a maximum positive moment, $M = 0.08\ wL^2$.

The provisions of *Specification* Section D6.1.1 apply to beams for which the tension *flange* is attached to deck or sheathing and the compression *flange* is completely unbraced. Beams with discrete point braces on the compression *flange* may have a bending capacity greater than those completely unbraced. Available data from simple span tests (Peköz and Soroushian, 1981 and 1982; LaBoube and Thompson, 1982a; LaBoube, et al., 1988; LaBoube and Golovin, 1990) indicate that for members having a lip edge stiffener at an angle of 75 degrees or greater with the plane of the compression *flange* and braces to the compression *flange* located at third points or more frequently, member capacities may be increased over those without discrete braces.

For the *LRFD* method, the use of the reduced *nominal flexural strength* [*resistance*] (*Specification* Equation D6.1.1-1) with a *resistance factor* of $\phi_b = 0.90$ provides the β values varying from 1.5 to 1.60 which are satisfactory for the target value of 1.5. This analysis was based on the *load* combination of 1.17 W - 0.9D using a reduction factor of 0.9 applied to the *load factor* for the *nominal* wind *load*, where W and D are nominal wind and dead *loads*, respectively (Hsiao, Yu and Galambos, 1988a; AISI, 1991).

In 2007, the panel depth was reduced from 1-1/4 inch (32 mm) to 1-1/8 inch (29 mm). This reduction in depth was justified because the behavior during full-scale tests indicated that the panel deformation was restricted to a relatively small area around the screw attachment of the panel to the *purlin*. Also, tests by LaBoube (1986) demonstrated that the panel depth did not influence the rotational stiffness of the panel-to-*purlin* attachment.

Prior to the 2001 edition, the *Specification* specifically limited the applicability of these provisions to continuous *purlin* systems in which any given span length did not vary from any other span length by more than 20 percent. This limitation was included in recognition of the fact that the research was based on systems with equal bay spacing. In 2007, the *Specification* was revised to permit *purlin* systems with adjacent span lengths varying more than 20 percent to use the reduction factor, R, for the simply supported condition. The revision allows a row of continuous *purlins* to be treated with a continuous beam condition R-factor in some bays and a simple span beam condition R-factor in others. The 20 percent span variation rule is a local effect and as such, only variation in adjacent spans is relevant.

In 2012, based on tests reported by Wibbenmeyer (2009), the limitation on the member depth was increased to 12 in. (305 mm), the ratio of depth-to-*flange* width was increased to 5.5, and a minimum *flange* width of 2.125 in. (54.0 mm) was added. The ratio of *tensile strength* to *yield stress* of 1.08 was added based on research at the University of Sydney (Pham and Hancock, 2009), which is also consistent with the applicable steels listed in *Specification* Section A9. The average depth-to-*flange* width ratio based on measured properties in the research by Wibbenmeyer (2009) was 5.3. However, the limit was increased to 5.5 in the *Specification*. This increased value was justified because the smallest measured *purlin flange* width for any of the members tested by Wibbenmeyer (2009) was 2.1875 in. (71.56 mm), which resulted in a ratio of depth-to-*flange* width of 5.5. Also, the reported value of R for the 12-in. (305-mm) deep *purlins* significantly exceed those previously stipulated for 11.5-in. (292-mm) deep members.

D6.1.2 Flexural Members Having One Flange Fastened to a Standing Seam Roof System

The design provision of this section is only applicable to the United States and Mexico. The discussion for this section is provided in the commentary on Appendix A. ☜A

D6.1.3 Compression Members Having One Flange Through-Fastened to Deck or Sheathing

For axially loaded C- or Z-sections having one *flange* attached to deck or sheathing and the other *flange* unbraced, e.g., a roof *purlin* or wall girt subjected to wind- or seismic-generated compression forces, the axial load capacity is less than a fully braced member, but greater than an unbraced member. The partial restraint relative to weak axis *buckling* is a function of the rotational stiffness provided by the panel-to-*purlin connection*. *Specification* Equation D6.1.3-1 is used to calculate the weak axis capacity. This equation is not valid for sections attached to standing seam roofs. The equation was developed by Glaser, Kaehler and Fisher (1994) and is also based on the work contained in the reports of Hatch, Easterling and Murray (1990), and Simaan (1973).

A limitation on the maximum *yield stress* of the C- or Z-section is not given in the *Specification* since *Specification* Equation D6.1.3-1 is based on elastic *buckling* criteria. A limitation on minimum length is not contained in the *Specification* because Equation D6.1.3-1 is conservative for spans less than 15 feet. The *gross area*, A, has been used rather than the *effective area*, A_e, because the ultimate axial *stress* is generally not large enough to result in a significant reduction in the *effective area* for common cross-section geometries.

As indicated in the *Specification*, the strong axis axial load capacity is determined by assuming that the weak axis of the strut is braced.

The controlling axial capacity (weak or strong axis) is suitable for usage in the combined axial *load* and bending equations in Section C5 of the *Specification* (Hatch, Easterling, and Murray, 1990).

D6.1.4 Compression of Z-Section Members Having One Flange Fastened to a Standing Seam Roof

The design provision of this section is only applicable to the United States and Mexico. The discussion for this section is provided in the commentary on Appendix A. ☜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 many panels can be calculated accurately. Under uplift loading, *nominal strength* [*resistance*] of standing seam roof panels and their attachments or anchors cannot be calculated with accuracy. Therefore, it is necessary to determine the *nominal strength* [*resistance*] by testing. Three test protocols have been used in this effort: FM 4471 developed by Factory Mutual, CEGS 07416 by the Corps of Engineers and ASTM E1592. In Supplement No. 1 to the 1996 edition of the *Specification*, (AISI, 1999), only the ASTM E1592-95 procedure was approved. In 2004, the Factory Mutual and Corps of Engineers protocols were also approved, provided that testing was in accordance with the AISI test procedure defined in S906 (AISI, 2002). While these test procedures have a common base, none defines a *design strength* [*factored resistance*]. *Specification* Section D6.2.1 and AISI S906, *Standard Procedures for Panel and Anchor Structural Tests*, adopted in 1999, added closure to the question by defining appropriate *resistance* and *safety factors*. The *safety factors* determined in Section D6.2.1 will vary depending on the characteristics of the test data. In 2006, limits were placed on the *safety factor* and *resistance factor* determined in this section to require a minimum *safety factor* of 1.67 and a maximum *resistance factor* of 0.9.

The *Specification* permits end conditions other than those prescribed by ASTM E1592-01. Areas of the roof plane that are sufficiently far enough away from crosswise restraint can be simulated by testing the open/open condition that was permitted in the 1995 edition of ASTM E1592. In addition, eave and ridge configurations that do not provide crosswise restraint can be evaluated.

The relationship of strength to serviceability limits may be taken as strength limit/serviceability limit = 1.25, or

$$\Omega_{serviceability} = \Omega_{strength}/1.25 \quad \text{(C-D6.2.1-1)}$$

It should be noted that the purpose of the test procedure specified in *Specification* Section D6.2.1 is not to set up guidelines to establish the serviceability limit. The purpose is to define the method of determining the *available strength* [*factored resistance*] whether based on the serviceability limit or on the *nominal strength* [*resistance*]. The Corps of Engineers Procedure CEGS 07416 (1991) requires a *safety factor* of 1.65 on strength and 1.3 on serviceability. A *buckling* or crease does not have the same consequences as a failure of a clip. In the latter case, the roof panel itself may become detached and expose the contents of a building to the elements of the environment. Further, Galambos (1988a) recommended a value of 2.0 for the target reliability index, β_o, when slight damage is expected and a value of 2.5 when moderate damage is expected. The resulting ratio is 1.25.

In *Specification* Section D6.2.1, a target reliability index of 2.5 is used for *connection* limits. It is used because the consequences of a panel fastener failure (β_o = 2.5) are not nearly as severe as the consequences of a primary frame *connection* failure (β_o = 3.5). The intermittent nature of wind *load* as compared to the relatively long duration of snow *load* further justifies the use of β_o = 2.5 for panel anchors. In *Specification* Section D6.2.1, the coefficient of variation of the material factor, V_M, is recommended to be 0.08 for failure limited by anchor or *connection* failure, and 0.10 for limits caused by flexural or other modes of failure. *Specification* Section D6.2.1 also eliminates the limit on coefficient of variation of the test results, V_p, because consistent test results often lead to V_p values lower than the 6.5 percent value set in *Specification* Section F1. The elimination of the limit will be beneficial when test results are consistent.

The value for the number of tests for fasteners is set as the number of anchors tested with the same tributary area as the anchor that failed. This is consistent with design practice where anchors are checked using a *load* calculated based on tributary area. Actual anchor *loads* are not calculated from a stiffness analysis of the panel in ordinary design practice.

Commentary for load combinations including wind uplift is provided in Appendix A.

☞A

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

In metal roof systems utilizing C- or Z-*purlins*, the application of gravity *loads* will cause torsion in the *purlin* and lateral displacements of the roof system. These effects are due to the slope of the roof, the loading of the member eccentric to its shear center, and for Z-*purlins*, the inclination of the principal axes. The torsional effects are not accounted for in the design provisions of Sections C3.1 and D6.1, and lateral displacements may create *instability* in the system. Lateral restraint is typically provided by the roof sheathing and lateral anchorage devices to minimize the lateral movement and the torsional effects. The anchorage devices are designed to resist the lateral anchorage force and provide the appropriate level of stiffness to ensure the overall stability of the *purlins*.

The calculation procedure in *Specification* Equations D6.3.1-1 through D6.3.1-6 determines the anchorage force by first calculating an upper bound force for each *purlin*, P_i, at the line of anchorage. This upper bound force is then distributed to anchorage devices and reduced due to the system stiffness based on the relative effective stiffness of each component. For the calculation procedure, the anchorage devices are modeled as linear springs located at the top of the *purlin web*. The stiffness of anchorage devices that do not attach at this location must be adjusted, through analysis or testing, to an equivalent lateral stiffness at the top of the *web*. This adjustment must include the influence of the attached *purlin* but not include any reduction due to the flexibility of the sheathing to *purlin* connection. *Specification* Equation D6.3.1-4 establishes an effective lateral stiffness for each anchorage device, relative to each *purlin*, that has been adjusted for the flexibility of the roof system between the *purlin* location and the anchorage location. It is important to note that the units of A_p are area per unit width. Therefore the bay length, L, in this equation must have units consistent with the unit width used for establishing A_p. The resulting product, LA_p, has units of area. The total effective stiffness for a given *purlin* is then calculated with *Specification* Equation D6.3.1-5 by summarizing the effective stiffness relative to each anchorage device and the system stiffness from *Specification* Equation

D6.3.1-6. The force generated by an individual *purlin* is calculated by Equation D6.3.1-2, and then distributed to an anchorage device based on the relative stiffness ratio in *Specification* Equation D6.3.1-1.

Lateral bracing forces will accumulate within the roof sheathing and must be transferred into the anchorage devices. The strength of the elements in this *load* path must be verified. AISI S912, *Test Procedures for Determining a Strength Value for a Roof Panel-to-Purlin-to-Anchorage Device Connection*, provides a means to determine a lower bound strength for the complete *load* path. For through-fastened roof systems, this strength value can be reasonably estimated by rational analysis by assuming that the roof fasteners within 12 inches (305 mm) of the anchorage device participate in the force transfer.

The 1986 through 2001 *Specifications* included brace force equations that were based on the work by Murray and Elhouar (1985) with various extensions from subsequent work. The original work assumed the applied loading was parallel to the *purlin webs*. The later addition of the "cosθ" and "sinθ" terms attempted to account for the roof slope, but it failed to correctly model the system effect for higher-sloped roofs. Tests by Lee and Murray (2001) and Seek and Murray (2004) showed generally that the brace force equations conservatively predicted the lateral anchorage forces at slopes less than 1:12, but predicted unconservative lateral anchorage forces at steeper slopes. The new procedure outlined in *Specification* Section D6.3.1 was formulated to correlate better with test results. Also, the original work was based on the application of one anchorage device to a group of *purlins*. Until the work of Sears and Murray (2007), a generally accepted manual technique to extend this procedure to roofs with multiple anchors was not available.

Prior to the work by Seek and Murray (2006, 2007) and Sears and Murray (2007), the anchorage devices were assumed to have a constant and relatively high lateral stiffness. The current provisions recognize the finite stiffness of the anchorage device, and the corresponding decrease in anchorage forces for more flexible anchorage devices. *Specification* Equation D6.3.1-7 establishes a minimum effective stiffness that must be provided to limit the lateral displacement at the anchorage device to d/20. This required stiffness does not represent the required stiffness of each anchorage device, but instead the total stiffness provided by the stiffness of the *purlin* system (K_{sys}) and the anchorage devices relative to the most remote *purlin*.

Several alternative rational analysis methods have been developed to predict lateral anchorage forces for Z-section roof systems. A method for calculating lateral anchorage forces is presented by Seek and Murray (2006, 2007). The method is similar to the procedure outlined in *Specification* Section D6.3.1 but uses a more complex method derived from mechanics to determine the lateral force introduced into the system at each Z-section, P_i, and distributes the force to the components of the system according to the relative lateral stiffness of each of the components. The method is more computationally intensive, but allows for analysis of more complex bracing configurations such as supports plus third points lateral anchorage and supports plus third points torsional braces.

A method to predict lateral anchorage forces using the finite element method is presented in Seek and Murray (2004). The model uses shell finite elements to model the Z-sections and sheathing in the roof system. The model accurately represents Z-section behavior and is capable of handling configurations other than lateral anchorage applied at the top *flange*. However, the computational complexity limits the size of the roof system that can be modeled by this method.

Rational analysis may also be performed using the elastic stiffness model developed by Sears and Murray (2007) upon which the provisions of *Specification* Section D6.3.1 are based. The model uses frame finite elements to represent the Z-sections and a truss system to represent the *diaphragm*. The model is computationally efficient, allowing for analysis of large systems.

Anchorage is most commonly applied along the frame lines due to the effectiveness and ease in which the forces are transferred out of the system. In the absence of substantial *diaphragm* stiffness, anchorage may be required along the interior of the span to prevent large lateral displacements. Torsional braces applied along the span of a Z- or C-section provide an alternative to interior anchorage.

D6.3.2 Alternative Lateral and Stability Bracing for Purlin Roof Systems

Tests (Shadravan and Ramseyer, 2007) have shown that C- and Z-sections can reach the capacity determined by *Specification* Section C3.1 through the application of torsional braces along the span of the member. Torsional braces applied between pairs of *purlins* prevent twist of the section at a discrete location. The moments developed due to the torsional brace can be resolved by forces in the plane of the *web* of each section and do not require external anchorage at the location of the brace. The vertical forces should, however, be accounted for when determining the applied *load* on the section.

Torsional braces should be applied at or near each *flange* of the Z- or C-section to prevent deformation of the *web* of the section and ensure the effectiveness of the brace. When twist of the section is thus prevented, a section may deflect laterally and retain its strength. Second-order moments can be resisted by the rotational restraints. Therefore, a more liberal lateral deflection of L/180 between the supports is permitted for a C- or Z-section with torsional braces. Anchorage is required at the frame line to prevent excessive deformation at the support location that undermines the strength of the section. A lateral displacement limit, therefore, is imposed along the frame lines to ensure that adequate restraint along the frame lines is provided.

E. CONNECTIONS AND JOINTS

E1 General Provisions

Welds, bolts, screws, rivets, and other special devices such as metal stitching and adhesives are generally used for cold-formed steel connections (Brockenbrough, 1995). The 2012 edition of the *Specification* contains provisions in Chapter E for welded *connections*, bolted *connections*, screw *connections*, and *power–actuated fastener connections*. Among these commonly used types of connections, the design provisions for using screws were developed in 1993 and were included in the 1996 AISI *Specification* for the first time, and the design provisions for *power-actuated fasteners* were added in the 2012 *Specification*. The following brief discussions deal with the application of rivets and other special devices:

(a) Rivets

While hot rivets have little application in cold-formed steel construction, cold rivets find considerable use, particularly in special forms such as blind rivets (for application from one side only), tubular rivets (to increase bearing area), high shear rivets, and explosive rivets. For the design of connections using cold rivets, the provisions for bolted connections may be used as a general guide, except that the shear strength of rivets may be quite different from that of bolts. Additional design information on the strength of rivets should be obtained from manufacturers or from tests.

(b) Special Devices

Special devices include: (1) metal stitching, achieved by tools that are special developments of the common office stapler, and (2) connecting by means of special clinching tools that draw the sheets into interlocking projections.

Most of these *connections* are proprietary devices for which information on strength of *connections* must be obtained from manufacturers or from tests carried out by or for the user. Guidelines provided in *Specification* Chapter F are to be used in these tests.

The plans or specifications are to contain information and design requirement data for the adequate detailing of each *connection* if the *connection* is not detailed on the engineering design drawings.

In the 2001 edition of the *Specification*, the *ASD*, *LRFD* and *LSD* design provisions for welded and bolted connections were based on the 1996 edition of the AISI *Specification,* with some revisions and additions which will be discussed in subsequent sections.

E2 Welded Connections

Welds used for cold-formed steel construction may be classified as fusion welds (or arc welds) and resistance welds. Fusion welding is used for connecting cold-formed steel members to each other as well as connecting such members to heavy, hot-rolled steel framing (such as floor panels to beams of the steel frame). It is used in groove welds, arc spot welds, arc seam welds, fillet welds, and flare-groove welds.

The design provisions contained in this *Specification* section for fusion welds have been based primarily on experimental evidence obtained from an extensive test program conducted at Cornell University. The results of this program are reported by Peköz and McGuire (1979) and summarized by Yu and LaBoube (2010). All possible failure modes are covered in the *Specification* since 1996, whereas the earlier *Specification* mainly dealt with shear failure.

For most of the *connection* tests reported by Peköz and McGuire (1979), the onset of yielding was either poorly defined or followed closely by failure. Therefore, in the provisions of this section, rupture rather than yielding is used as a more reliable criterion of failure.

The welded connection tests, which served as the basis of the provisions given in *Specification* Sections E2.1 through E2.7, were conducted on sections with single and double sheets (see *Specification* Figures E2.2-1 and E2.2-2). The largest total sheet *thickness* of the cover plates was approximately 0.15 inch (3.81 mm). However, within this *Specification*, the validity of the equations was extended to welded connections in which the *thickness* of the thinnest connected part is 3/16 inch (4.76 mm) or less. For arc spot welds, the maximum *thickness* of a single sheet (*Specification* Figure E2.2.2.1-1) and the combined *thickness* of double sheets (*Specification* Figure E2.2.2.1-2) are set at 0.15 inch (3.81 mm).

In 2001, the *safety factors* and *resistance factors* in this section were modified for consistency based on the research work by Tangorra, Schuster, and LaBoube (2001).

For design tables and example problems on welded connections, see Part IV of the *Design Manual* (AISI, 2013).

See Appendix A or B for additional commentary. ☞A,B

E2.1 Groove Welds in Butt Joints

The design equations for determining *nominal strength* [*resistance*] for groove welds in butt *joints* have been taken from the AISC *LRFD* Specification (AISC, 1993). Therefore, the AISC definition for the effective throat thickness, t_e, is equally applicable to this section of the *Specification*. Prequalified *joint* details are given in AWS D1.3-98 (AWS, 1998) or other equivalent weld standards.

In 2010, *Specification* Section E2.1(a) was revised to delete the case for tension or compression parallel to the axis of the weld, so that *Specification* Equation E2.1-1 is applicable only to tension or compression normal to the *effective area* of the weld. For tension or compression parallel to the weld axis, the computation of the weld strength is not required (AISC, 2005 and 2010).

E2.2 Arc Spot Welds

Arc spot welds (puddle welds) used for connecting thin sheets are similar to plug welds used for relatively thicker plates. The difference between plug welds and arc spot welds is that the former are made with pre-punched holes, but no pre-punched holes are required for the latter. Instead, a hole is burned in the top sheet by the arc and then filled with weld metal to fuse it to the bottom sheet or a framing member. The provisions of Section E2.2 apply to plug welds as well as spot welds.

E2.2.1 Minimum Edge and End Distance

In the 2001 and 2007 editions of the *Specification*, the distance measured in the line of force from the centerline of weld to the nearest edge of an adjacent weld or to the end of the connected part toward which the force is directed was required to not be less than e_{min}, which is equal to *required strength* [forces due to *factored loads*] divided by ($F_u t$). In 2010, an equivalent resistance is determined by the use of Section E6.1.

E2.2.2 Shear

E2.2.2.1 Shear Strength [Resistance] for Sheet(s) Welded to a Thicker Supporting Member

The Cornell tests (Peköz and McGuire, 1979) identified four modes of failure for arc spot welds, which are addressed in this *Specification* section. They are: (1) shear failure of welds in the fused area, (2) tearing of the sheet along the contour of the weld with the tearing spreading the sheet at the leading edge of the weld, (3) sheet tearing combined with *buckling* near the trailing edge of the weld, and (4) shearing of the sheet behind the weld. It should be noted that many failures, particularly those of the plate tearing type, may be preceded or accompanied by considerable inelastic out-of-plane deformation of the type indicated in Figure C-E2.2.2.1-1. This form of behavior is similar to that observed in wide, pin-connected plates. Such behavior should be avoided by closer spacing of welds. When arc spot welds are used to connect two sheets to a framing member as shown in *Specification* Figure E2.2.2.1-2, consideration should also be given to possible shear failure between thin sheets.

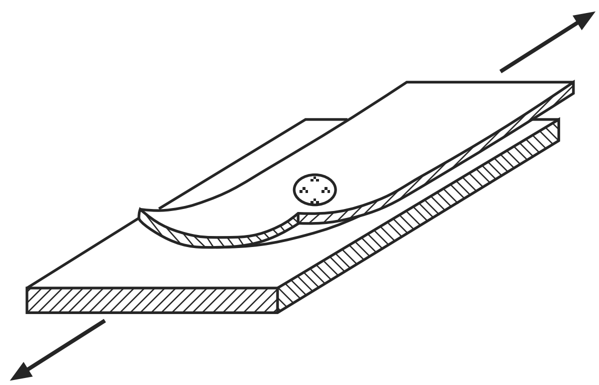


Figure C-E2.2.2.1-1 Out-of-Plane Distortion of Welded Connection

The *thickness* limitation of 0.15 inch (3.81 mm) is due to the range of the test program that served as the basis of these provisions. On sheets below 0.028 inch (0.711 mm) thick, weld washers are required to avoid excessive burning of the sheets and, therefore, inferior quality welds.

In the AISI 1996 *Specification*, Equation E2.2-1 was revised to be consistent with the research report (Peköz and McGuire, 1979).

In 2001, the equation used for determining d_a for multiple sheets was revised to be (d-t).

E2.2.2.2 Shear Strength [Resistance] for Sheet-to-Sheet Connections

The Steel Deck Institute *Diaphragm Design Manual* (SDI, 1987 and 2004) stipulates that the shear strength for a sheet-to-sheet arc spot weld *connection* be taken as 75% of the strength of a sheet-to-structural *connection*. SDI further stipulates that the sheet-to-structural connection strength be defined by *Specification* Equation E2.2.2.1-2. This design provision was adopted by the *Specification* in 2004. Prior to accepting the SDI design recommendation, a review of the pertinent research by Luttrell (SDI, 1987) was performed by LaBoube (LaBoube, 2001). The tested sheet *thickness* range that is reflected in the *Specification* documents is based on the scope of Luttrell's test program. SDI suggests that sheet-to-sheet welds are problematic for *thicknesses* of less than 0.0295 in. (0.75 mm). Such welds result in "blow holes," but the perimeter must be fused to be effective.

Quality control for sheet–to-sheet connections is not within the purview of AWS D1.3. However, using AWS D1.3 as a guide, the following quality control/assurance guidelines are suggested:

(1) Measure the visible diameter of the weld face,

(2) Ensure no cracks in the welds,

(3) Maximum undercut = 1/8 of the weld circumference, and

(4) Sheets are to be in contact with each other.

E2.2.3 Tension

For tensile capacity of arc spot welds, the design provisions in the AISI 1989 *Specification* Addendum were based on the tests reported by Fung (1978) and the study made by Albrecht (1988). Those provisions were limited to sheet failure with restrictive limitations on material properties and sheet *thickness*. These design criteria were revised in 1996 because the tests conducted at the University of Missouri-Rolla (LaBoube and Yu, 1991 and 1993) have shown that two potential limit states may occur. The most common failure mode is that of sheet tearing around the perimeter of the weld. This failure condition was found to be influenced by the sheet *thickness*, the average weld diameter, and the material *tensile strength*. In some cases, it was found that tensile failure of the weld can occur. The strength of the weld was determined to be a function of the cross-section of the fused area and *tensile strength* of the weld material. Based on analysis by LaBoube (LaBoube, 2001), the *nominal strength* [*resistance*] equation was changed in 2001 to reflect the ductility of the sheet, F_u/F_y, and the sheet *thickness*, the average weld diameter, and the material *tensile strength*.

The multiple *safety factors* and *resistance factors* recognize the behavior of a panel system with many *connections* versus the behavior of a member connection and the potential for a catastrophic failure in each application. In *Specification* Section E2.2.3, a target reliability index of 3.0 for the United States and Mexico and 3.5 for Canada is used for the panel connection limit, whereas a target reliability index of 3.5 for the United States and Mexico and 4 for Canada is used for the other *connection* limit. Precedence for the use of a smaller target reliability index for systems was established in Section D6.2.1 of the *Specification*.

Tests (LaBoube and Yu, 1991 and 1993) have also shown that when reinforced by a weld washer, thin sheet weld connections can achieve the *design strength* [*factored resistance*] given by *Specification* Equation E2.2.3-2 using the *thickness* of the thinner sheet.

The equations given in the *Specification* were derived from the tests for which the applied tension *load* imposed a concentric *load* on the weld, as would be the case, for example, for the interior welds on a roof system subjected to wind uplift. Welds on the perimeter of a roof or floor system would experience an eccentric tensile loading due to wind uplift. Tests have shown that as much as a 50 percent reduction in *nominal connection strength* [*resistance*] could occur because of the eccentric *load* application (LaBoube and Yu, 1991 and 1993). Eccentric conditions may also occur at *connection* laps as depicted by Figure C-E2.2.3-1.

At a lap *connection* between two deck sections as shown in Figure C-E2.2.3-1, the length of the unstiffened *flange* and the extent of the encroachment of the weld into the unstiffened *flange* have a measurable influence on the strength of the welded *connection* (LaBoube and Yu, 1991). The *Specification* recognizes the reduced capacity of this connection detail by imposing a 30 percent reduction on the calculated *nominal strength* [*resistance*].

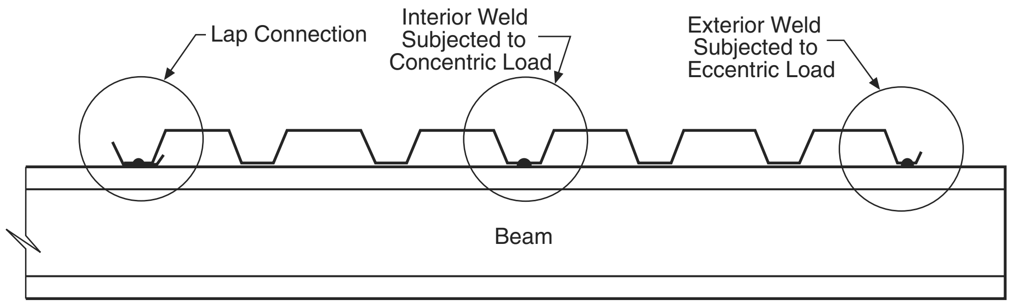


Figure C-E2.2.3-1 Interior Weld, Exterior Weld and Lap Connection

E2.2.4 Combined Shear and Tension on an Arc Spot Weld

The Steel Deck Institute *Diaphragm Design Manual* (2004) provides a design equation for evaluating the strength of an arc spot weld *connection* subject to combined shear and tension forces. An experimental investigation was conducted at the University of Missouri–Rolla to study the behavior and to develop design recommendations for the relationship (interaction) of the tension and shear forces on an arc spot weld *connection* (Stirnemann and LaBoube, 2007).

The experimental study focused on six variables that were deemed to be the key parameters that could influence the strength of the arc spot weld connection. These variables were the sheet *thickness*; sheet material properties including *yield stress*, *tensile strength* and ductility of the sheet; visible diameter of the arc spot weld; and the relationship between the magnitude of the shear force and tension force. Based on an analysis of the test results, the Steel Deck Institute's interaction equation was found to provide an acceptable estimate of the strength of the arc spot weld *connection*.

E2.3 Arc Seam Welds

The general behavior of arc seam welds is similar to that of arc spot welds. In 2010, Section E2.3 was reorganized to be consistent with provisions provided for arc spot welds.

E2.3.2 Shear

E2.3.2.1 Shear Strength [Resistance] for Sheet(s) Welded to a Thicker Supporting Member

No simple shear failures of arc seam welds were observed in the Cornell tests (Peköz and McGuire, 1979). Therefore, *Specification* Equation E2.3.2.1-1, which accounts for shear failure of welds, is adopted from the AWS welding provisions for sheet steel (AWS, 1998).

Specification Equation E2.3.2.1-2 is intended to prevent failure through a combination of tensile tearing plus shearing of the cover plates.

E2.3.2.2 Shear Strength [Resistance] for Sheet-to-Sheet Connections

In 2010, the provisions for determining the shear strength of sheet-to-sheet arc spot weld connections were adopted for arc seam weld connections. This is conservative because the length of the seam weld is not considered.

E2.4 Top Arc Seam Sidelap Welds

Top arc seam sidelap welds (often referred to as TSWs) have commonly been used to attach the edges of standing seam steel roof and floor deck panels, particularly those used for *diaphragms*. The top arc seam sidelap connection is formed by a vertical sheet leg (edge stiffener of deck) inside an overlapping sheet hem, or by two vertical sheet legs back-to-back. *Top arc seam welds* have been referenced in some historical *diaphragm* design standards as part of a system without defining the strength of individual connections. Similarly, AWS D1.3 has shown the weld as a possible variation of an arc seam weld, without clear provisions to determine weld strength. The research to develop the design provisions for the *top arc seam welds* is presented in the S. B. Barnes Associates (Nunna and Pinkham, 2012; Nunna, et al., 2012) report.

E2.4.1 Shear Strength [Resistance] of Top Arc Seam Sidelap Welds

The design limitations are due to the scope of the test program that served as the basis for these provisions. The tests included typical weld spacing of approximately 12 in. (305 mm) o.c. and this established the strength of the welds with the stated limits. All testing was performed on *joints* with a vertical sheet leg inside an overlapping sheet hem configuration, but the behavior of connections with back-to-back vertical sheet legs is assumed to be similar.

Testing was performed in general accordance with AISI S905 (AISI, 2008), with the specimen dimensions in S905 Table 2 modified as required to address the described deck edge configuration. The ductility of the tested steels ranged from F_u/F_{sy} = 1.01 to F_u/F_{sy} = 1.52. The limits were extended to permit the use of the full range of recognized steels. Application should be based on the specified F_u/F_{sy} for steels recognized in Section A2 of the *Specification*. The exclusion of the *connection* design restrictions for *top arc seam welds* used in *diaphragms* considers that the shear in the side lap welds is flowing from the sheet into each weld such that each weld is loaded as if it were a singular weld by its tributary length. This mitigates the concern over *load* sharing in brittle connections, and the strength reduction of lower ductility steels is based on the tests and built into *Specification* Equation E2.4.1-1.

The impact of shear rupture in the sheet can be calculated based on *Specification* Section E6 and this can be used to determine minimum acceptable weld spacing. The distance from the centerline of any weld and the centerline of adjacent weld can be checked by using Equation C-E2.4.1-1. Equation C-E2.4.1-1 is derived by equating the *nominal shear strength* [*resistance*] expression from *Specification* Section E6 (*Eq*. E6.1-1 with A_{nv} = st) to the *nominal shear strength* [*resistance*] expression from *Specification* Section E2.4.1.

$$s = [6.67(F_u/F_{sy})-2.53]L_w(t/L_w)^{0.33} \qquad \text{(C-E2.4.1-1)}$$

where

s = minimum distance from centerline of any weld to centerline of adjacent weld

s/2 = minimum distance from centerline of weld to end of connected member

L_w = specified weld length

t = base steel *thickness* (exclusive of coatings) of the thinner connected sheet

F_u = minimum *tensile strength* of connected sheets as determined in accordance with *Specification* Section A2.3.1, A2.3.2 or A2.3.3

F_{sy} = *minimum specified yield stress* of connected sheets as determined in accordance with *Specification* Section A2.3.1, A2.3.2 or A2.3.3

The steel deck sheets at the sidelap need to be tightly interlocked by crimping or pinching the sidelap prior to welding. When using the *joint* variation shown in *Specification* Figure E2.4.1-1(b), contact must be maintained between the two vertical legs while welding. For sidelaps with overlapping hem, *Specification* Figure E2.4.1-1(a) illustrates a crimped area nominally longer than the length of fusion, and the top of the overlapping hem sidelap must be burned through to allow fusion with the top of the inner vertical leg. Holes are commonly present at either or both ends of the completed welds. The holes do not necessarily indicate deficient welds or poor workmanship provided the specified length of fusion is obtained. Holes may aid in determining proper fusion with the inner vertical leg.

E2.5 Fillet Welds

For fillet welds on the lap *joint* specimens tested in the Cornell research (Peköz and McGuire, 1979), the dimension, w_1, of the leg on the sheet edge generally was equal to the sheet *thickness*; the other leg, w_2, often was two or three times longer than w_1 (see *Specification* Figure E2.5-1). In *connections* of this type, the fillet weld throat is commonly larger than the throat of conventional fillet welds of the same size. Usually, ultimate failure of fillet-welded *joints* has been found to occur by the tearing of the plate adjacent to the weld (see Figure C-E2.5-1).

In most cases, the higher strength of the weld material prevents weld shear failure; therefore, the provisions of this *Specification* section are based on sheet tearing. Because specimens up to 0.15 inch (3.81 mm) *thickness* were tested in the Cornell research (Peköz and McGuire, 1979), the last provision in this section covers the possibility that for sections thicker

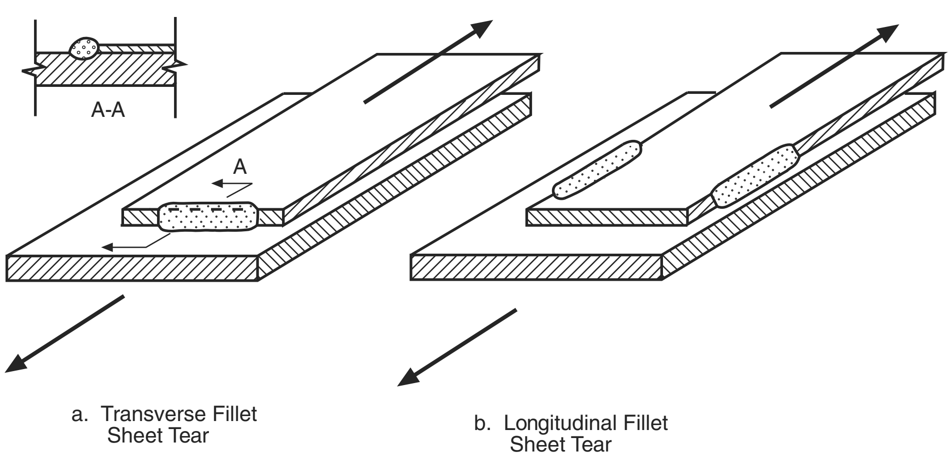


Figure C-E2.5-1 Fillet Weld Failure Modes

than 0.15 inch (3.81 mm), the throat dimension may be less than the *thickness* of the cover plate and the tear may occur in the weld rather than in the plate material. Additional research at the University of Sydney (Zhao and Hancock, 1995) has further indicated that weld throat failure may even occur between the *thicknesses* of 0.10 in. (2.54 mm) to 0.15 in. (3.81 mm). Accordingly, the *Specification* was revised in 2001 to require weld strength check when the plate *thickness* is greater than 0.10 in. (2.54 mm). For high-strength materials with *yield stress* of 65 ksi (448 MPa) or higher, research at the University of Sydney (Teh and Hancock, 2000) has shown that weld throat failure does not occur in materials less than 0.10 in. (2.54 mm) thick and that the AISI *Specification* provisions based on sheet strength are satisfactory for high-strength material less than 0.10 in. (2.54 mm) thick. Prequalified fillet welds are given in AWS D1.3-98 (AWS, 1998) or other equivalent weld standards.

In 2012, the design provisions were modified to take into consideration that the connected parts may have different *tensile strengths*.

E2.6 Flare Groove Welds

The primary mode of failure in cold-formed steel sections welded by flare groove welds, loaded transversely or longitudinally, was found to be sheet tearing along the contour of the weld (see Figure C-E2.6-1).

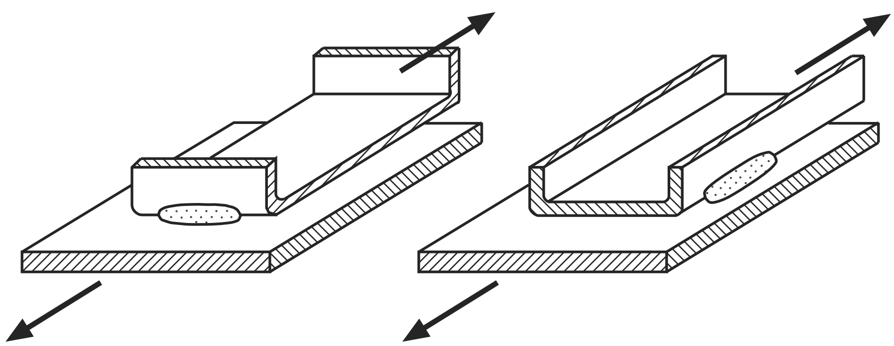


Figure C-E2.6-1 Flare Groove Weld Failure Modes

Except for *Specification* Equation E2.6-4, the provisions of this *Specification* section are intended to prevent shear tear failure. *Specification* Equation E2.6-4 covers the possibility that thicker sections may have effective throats less than the *thickness* of the channel and weld failure may become critical.

In 2001, the *Specification* was revised to require that weld strength be checked when the plate *thickness* is greater than 0.10 in. (2.54 mm) based on the research by Zhao and Hancock (1995).

In 2010, the former *Specification* Figures E2.6-4 through E2.6-7 were replaced by two new drawings showing reference dimensions for flare-bevel groove welds and flare V-groove welds, respectively. *Specification* Equations E2.6-5 and E2.6-7 were added to more accurately define the effective throat of these welds. Filled flush throat depths were modified to match those specified in AWS D1.1-2006 Section 2.3.1.4 and Table 2.1. Welding process designations in *Specification* Tables E2.6-1 and E2.6-2 were based on AWS D1.1 Annex K, where SMAW stands for "shielded metal arc welding," FCAW-S stands for "flux cored arc welding-self shielded," GMAW stands for "gas metal arc welding," FCAW-G stands for "flux cored arc welding-gas shielded," and SAW stands for "submerged arc welding." No change was needed in the *Specification* requirements from previous editions except in the definitions of the effective throat for use in *Specification* Equation E2.6-4.

E2.7 Resistance Welds

The shear values for outside sheets of 0.125 inch (3.18 mm) or less in *thickness* are based on "Recommended Practice for Resistance Welding Coated Low-Carbon Steels," AWS C1.3-70 (Table 2.1 - Spot Welding Galvanized Low-Carbon Steel). Shear values for outside sheets thicker than 0.125 inch (3.18 mm) are based upon "Recommended Practices for Resistance Welding," AWS C1.1-66 (Table 1.3 - Pulsation Welding Low-Carbon Steel) and apply to pulsation welding as well as spot welding. They are applicable for all structural grades of low-carbon steel, uncoated or galvanized with 0.90 oz/ft^2 (275 g/m^2) of sheet or less, and are based on values selected from AWS C1.3-70 (Table 2.1), and AWS C1.1-66 (Table 1.3). These values may also be applied to medium carbon and low-alloy steels. Spot welds in such steels give somewhat higher shear strengths than those upon which these values are based; however, they may require special welding conditions. In view of the fact that AWS C1.1-66 and AWS C1.3-70 Standards were incorporated in AWS C1.1-2000, resistance welds should be performed in accordance with AWS C1.1-2000 (AWS, 2000).

In the 2001 edition of the *Specification*, a design equation is used to determine the *nominal shear strength* [*resistance*] that replaces the tabulated values given in the previous specifications. The upper limit of *Specification* Equations E2.7-1, E2.7-3 and E2.7-5 is selected to best fit the data provided in AWS C1.3-70, Table 2.1 and AWS C1.1-66, Table 1.3. Shear strength values for welds with the *thickness* of the thinnest outside sheet greater than 0.180 in. (4.57 mm) have been excluded in *Specification* Equations E2.7-2, E2.7-4 and E2.7-6 due to the *thickness* limit set forth in *Specification* Section E2.

E3 Bolted Connections

The structural behavior of bolted connections in cold-formed steel construction is somewhat different from that in hot-rolled heavy construction, mainly because of the thinness of the connected parts. Prior to 1980, the provisions included in the AISI *Specification* for the design of bolted connections were developed on the basis of the Cornell tests (Winter, 1956a, 1956b). These provisions were updated in 1980 to reflect the results of additional research performed in the United States (Yu, 1982) and to provide better coordination with the specifications of the Research Council on Structural Connections (RCSC, 1980) and AISC (1978). In 1986, design provisions for the maximum size of bolt holes and the allowable tension *stress* for bolts were added to the AISI *Specification* (AISI, 1986). In the 1996 edition of the AISI *Specification*, minor changes to the *safety factors* were made for computing the *allowable* and *design tensile* and *shear strengths* [*factored resistances*] of bolts. The allowable tensile *stress* for the bolts subject to the combination of shear and tension was determined by the equations provided in *Specification* Table E3.4-2 with the applicable *safety factor*.

(a) Scope

Previous studies and practical experiences have indicated that the structural behavior of bolted *connections* used for joining *relatively thick* cold-formed steel members is similar to that for connecting hot-rolled shapes and built-up members. The AISI *Specification* criteria are applicable only to cold-formed steel members or elements 3/16 inch (4.76 mm) or less in *thickness*. For materials greater than 3/16 inch (4.76 mm), reference is made to the specifications or standards stipulated in Section E3a of Appendix A or B.

Because of the lack of appropriate test data and the use of numerous surface conditions, this *Specification* does not provide design criteria for slip-critical (also called friction-type) *connections*. When such *connections* are used with cold-formed steel members where the *thickness* of the thinnest connected part is 3/16 inch (4.76 mm) or less, it is recommended that tests be conducted to confirm their design capacity. The test data should verify that the specified design capacity for the *connection* provides sufficient safety against initial slip at least equal to that implied by the provisions of the specifications or standards listed in Section E3a of Appendix A or B. In addition, the safety against ultimate capacity should be at least equal to that implied by this *Specification* for bearing-type connections.

The *Specification* provisions apply only when there are no gaps between plies. The designer should recognize that the *connection* of a rectangular tubular member by means of bolt(s) through such members may have less strength than if no gap existed. Structural performance of *connections* containing unavoidable gaps between plies would require tests in accordance with *Specification* Section F1.

(b) Materials

This section lists five different types of fasteners which are normally used for cold-formed steel construction. In view of the fact that A325 and A490 bolts are available only for diameters of 1/2 inch (12.7 mm) and larger, A449 and A354 Grade BD bolts should be used as an equivalent of A325 and A490 bolts, respectively, whenever smaller bolts (less than 1/2 inch (12.7 mm) in diameter) are required.

During recent years, other types of fasteners, with or without special washers, have been widely used in steel structures using cold-formed steel members. The design of these fasteners should be determined by tests in accordance with Chapter F of this *Specification*.

(c) Bolt Installation

Bolted connections in cold-formed steel structures use either mild or high-strength steel bolts and are designed as a bearing-type *connection*. Bolt pre-tensioning is not required because the ultimate strength of a bolted *connection* is independent of the level of bolt preload. Installation must ensure that the bolted assembly will not come apart during service. Experience has shown that bolts installed to a snug tight condition do not loosen or "back-off" under normal building conditions and are not subject to vibration or *fatigue*.

Bolts in slip-critical *connections*, however, must be tightened in a manner which ensures the development of the fastener tension forces required by the Research Council on Structural Connections (1985 and 2000) for the particular size and type of bolts. Turn-of-nut rotations specified by the Research Council on Structural Connections may not be applicable because such rotations are based on larger grip lengths than are encountered in usual cold-formed construction. Reduced turn-of-the-nut values would have to be established for the actual combination of grip and bolt. A similar test program (RCSC, 1985 and 1988) could establish a cut-off value for calibrated wrenches. Direct tension indicators (ASTM F959), whose published clamping forces are independent of grip, can be used for tightening slip-critical *connections*.

(d) Hole Sizes

For bolts having diameters less than 1/2 inch (12.7 mm), the diameter of a standard hole is the diameter of bolt plus 1/32 inch (0.794 mm). This maximum size of bolt holes is based on previous editions of the AISI *Specification*.

An alternative short-slotted hole size was added to Table E3 as a result of a research project done by Yu and Xu (2010), who investigated bolted *connections* having various hole dimensions.

When using oversized holes or short-slotted holes, care must be exercised by the designer to ensure that excessive deformation due to slip will not occur at working *loads*. Excessive deformations, which can occur in the direction of the slots, may be prevented by requiring bolt pretensioning.

Short-slotted holes are usually treated in the same manner as oversized holes. Washers or back-up plates should be used over oversized or short-slotted holes in an outer ply when the bolt hole deformation is considered in design. For *connections* using long-slotted holes, *Specification* Section E3 requires that the washers or back-up plates be used and that the shear capacity of bolts be determined by tests because a reduction in strength may be encountered.

Design information for oversized and slotted holes is included in Section E3.3.1 because such holes are often used in practice to meet dimensional tolerances during erection.

When the bolt hole deformation is considered in design, standard holes should be used in bolted *connections*. Oversized holes and slotted holes are only permitted as approved by the designer. An exception to the provisions for slotted holes is made in the case of slotted holes in lapped and nested zees. Resistance is provided in this situation partially by the nested components, rather than direct bolt shear and bearing. An oversize or slotted hole is required for proper fit-up due to offsets inherent in nested parts. Research (Bryant and Murray, 2001) has shown that lapped and nested zee members with 1/2-in. (12.7-mm) diameter bolts without washers and 9/16 in. x 7/8 in. (14.3 mm x 22.2 mm) slotted holes can develop the full moment in the lap.

E3.3 Bearing

Previous bolted *connection* tests have shown that *bearing* strength of bolted *connections* depends on: (1) the *tensile strength*, F_u, of the connected parts, (2) the *thickness* of connected parts, (3) the diameter of bolt, (4) *joints* with single shear and double shear conditions, (5) the F_u/F_y ratio, and (6) the use of washers (Winter, 1956a and 1956b; Chong and Matlock, 1974; Yu, 1982 and 2000). These design parameters were used in the 1996 and earlier editions of the AISI *Specification* for determining the *bearing* strength between bolt and connected parts (AISI, 1996).

In the Canadian Standard (CSA, 1994), the d/t ratio was also used in the design equation for determining the *bearing* strength of bolted *connections*.

☜B

E3.3.1 Strength [Resistance] Without Consideration of Bolt Hole Deformation

Rogers and Hancock (1998) developed the design equation for bearing of bolted *connections* with washers (*Specification* Table E3.3.1-1). Based on research at the University of Waterloo (Wallace, Schuster, and LaBoube, 2001a), the Rogers and Hancock equation was extended to bolted *connections* without washers and to the inside sheet of double shear *connections* with or without washers (*Specification* Table E3.3.1-2). In *Specification* Table E3.3.1-1, the bearing factor, C, depends on the ratio of bolt diameter to member *thickness*, d/t. The design equations in *Specification* Section E3.3.1 are based on available test data. Thus, for sheets thinner than 0.024 in. (0.61 mm), tests must be performed to determine the structural performance.

The *safety factor* and *resistance factors* are based on calibration of available test data (Wallace, Schuster, and LaBoube, 2001b).

Yu and Xu (2010) conducted testing of bolted *connections* without washers on oversized and short-slotted holes. Based on the test data, Yu and Xu developed new equations for bearing factor, C, and new values for modification factor, m_f. The hole dimensions investigated in Yu and Xu (2010) are consistent with those in Table E3. The added provisions for oversized and short-slotted holes do not apply to the slotted holes in lapped and nested zees. The *safety factor* and *resistance factors* are verified by Yu and Xu (2010) to be applicable for bolted *connections* using oversized and short-slotted holes.

E3.3.2 Strength [Resistance] With Consideration of Bolt Hole Deformation

Based on research at the University of Missouri-Rolla (LaBoube and Yu, 1995), design equations have been developed that recognize the presence of hole elongation prior to reaching the limited *bearing* strength of a bolted *connection*. The researchers adopted an elongation of 0.25 in. (6.4 mm) as the acceptable deformation limit. This limit is consistent with the permitted elongation prescribed for hot-rolled steel.

Since the *nominal strength* [*resistance*] value with consideration of bolt hole deformation should not exceed the *nominal strength* [*resistance*] without consideration of the hole deformation, this limit was added in 2004.

E3.4 Shear and Tension in Bolts

The design provisions of this section are given in Section E3.4 of Appendix A or B. In Appendix A, the commentary is provided for Section E3.4. ☜A

E4 Screw Connections

The results of over 3500 tests worldwide were analyzed to formulate screw *connection* provisions (Peköz, 1990). European Recommendations (1987) and British Standards (1992) were considered and modified as appropriate. Since the provisions apply to many different screw *connections* and fastener details, a greater degree of conservatism is implied than is otherwise typical within this *Specification*. These provisions are intended for use when a sufficient number of test results are not available for the particular application. A higher degree of accuracy can be obtained by testing any particular *connection* geometry (AISI, 1992).

Over 450 elemental *connection* tests and eight *diaphragm* tests were conducted in which compressible fiberglass insulation, typical of that used in metal building roof systems (MBMA, 2002), was placed between steel sheet samples in the elemental *connection* tests and between the deck and *purlin* in the *diaphragm* tests (Lease and Easterling, 2006a, 2006b). The results indicate that the equations in Section E4 of the *Specification* are valid for applications that incorporate 6-3/8 in. (162 mm) or less of compressible fiberglass insulation.

Screw *connection* tests used to formulate the provisions included single fastener specimens as well as multiple fastener specimens. However, it is recommended that at least two screws should be used to connect individual elements. This provides redundancy against under-torquing, over-torquing, etc., and limits lap shear *connection* distortion of flat unformed members such as straps.

Proper installation of screws is important to achieve satisfactory performance. Power tools with adjustable torque controls and driving depth limitations are usually used.

For the convenience of designers, Table C-E4-1 gives the correlation between the common number designation and the nominal diameter for screws. See Figure C-E4-1 for the measurement of nominal diameters.

Table C-E4-1 Nominal Diameter for Screws

Number Designation	Nominal Diameter, d in.	mm
0	0.060	1.52
1	0.073	1.85
2	0.086	2.18
3	0.099	2.51
4	0.112	2.84
5	0.125	3.18
6	0.138	3.51
7	0.151	3.84
8	0.164	4.17
10	0.190	4.83
12	0.216	5.49
1/4	0.250	6.35

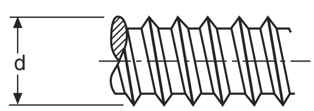


Figure C-E4-1 Nominal Diameter for Screws

E4.1 Minimum Spacing

Minimum spacing is the same as specified for bolts.

E4.2 Minimum Edge and End Distances

In 2001, the minimum edge distance was decreased from 3d to 1.5d.

E4.3 Shear

E4.3.1 Shear Strength [Resistance] Limited by Tilting and Bearing

Screw *connections* loaded in shear can fail in one mode or in combination of several modes. These modes are screw shear, edge tearing, tilting and subsequent pull-out of the screw, and bearing of the joined materials.

Tilting of the screw followed by threads tearing out of the lower sheet reduces the *connection* shear capacity from that of the typical *connection bearing* strength (Figure C-E4.3-1).

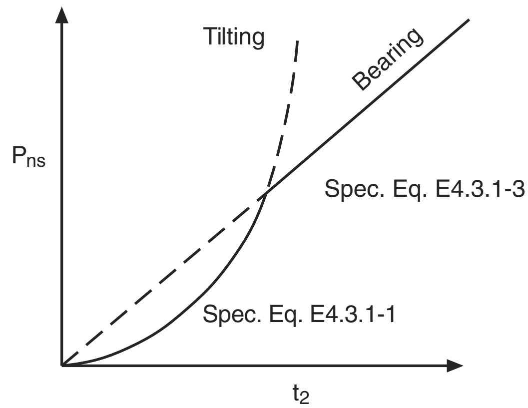


Figure C-E4.3-1 Comparison of Tilting and Bearing

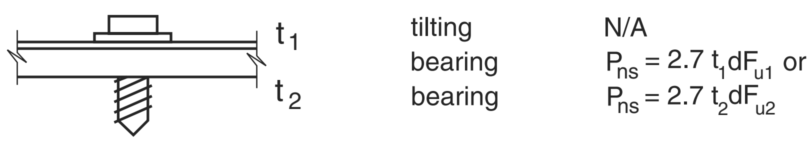


Figure C-E4.3-2 Design Equations for $t_2/t_1 \geq 2.5$

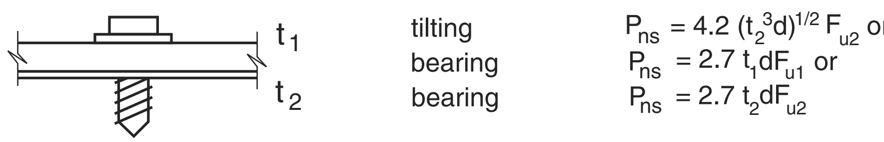


Figure C-E4.3-3 Design Equations for $t_2/t_1 \leq 1.0$

These provisions are focused on the tilting and bearing failure modes. Two cases are given depending on the ratio of *thicknesses* of the connected members. Normally, the head of the screw will be in contact with the thinner material as shown in Figure C-E4.3-2. However, when both members are the same *thickness*, or when the thicker member is in contact with the screw head, tilting must also be considered as shown in Figure C-E4.3-3.

It is necessary to determine the lower *bearing* capacity of the two members based on the product of their respective *thicknesses* and *tensile strengths*.

E4.3.2 Shear in Screws

Shear strength of the screw fastener itself should be known and documented from testing. Screw strength should be established and published by the manufacturer. In order to prevent the brittle and sudden shear fracture of the screw, the *Specification* applies a 25 percent adjustment to the *safety factor* or the *resistance factor* where determined in accordance with *Specification* Section F1.

E4.4 Tension

Screw *connections* loaded in tension can fail either by the screw pulled out from the plate (pull-out); material pulled over the screw head and the washer, if a washer is present (pull-over); or by tensile fracture of the screw. The serviceability concerns of gross distortion are not covered by the equations given in *Specification* Section E4.4.

Diameter and rigidity of the fastener head assembly as well as sheet *thickness* and *tensile strength* have a significant effect on the pull-over failure load of a *connection*.

There are a variety of washers and head styles in use. Washers must be sufficiently thick to withstand bending forces with little or no deformation. In 2010, the minimum washer *thickness* requirement of 0.050 in. (1.27 mm) was relaxed for the washers in *connections* where t_1 does not exceed 0.027 in. (0.686 mm), with the evidence that the washer *thickness* of as low as 0.024 in. (0.610 mm) does not adversely impact the pull-over strength of the *connection* for such top substrate *thicknesses* (Mujagic, 2008). In 2012, the washer dimension requirements were modified to harmonize the limitations of *Specification* Sections E4.5 with E4.4, given similar pull-over models in the two sections. Based on the findings of Zwick and LaBoube (2002), washers with outside diameter of 5/8 to 3/4 in. (15.9 mm to 19.1 mm) and a minimum *thickness* of 0.063 in. (1.60 mm) were included in the scope of *Specification* Section E4.4. Designers should include minimum required washer *thickness* in project documents.

E4.4.1 Pull-Out Strength [Resistance]

For the limit state of pull-out, *Specification* Equation E4.4.1-1 was derived on the basis of the modified European Recommendations and the results of a large number of tests. The statistic data on pull-out design considerations were presented by Peköz (1990).

E4.4.2 Pull-Over Strength [Resistance]

For the limit state of pull-over, *Specification* Equation E4.4.2-1 was derived on the basis of the modified British Standard and the results of a series of tests as reported by Peköz (1990). In 2007, a rational allowance was included to cover the contribution of steel washers beneath screw heads. For the special case of screws with domed washers (washers that are not solid or do not seat flatly against the sheet metal in contact with the washer), the calculated *nominal pull-over strength* [*resistance*] should not exceed $1.5t_1d'_wF_{u1}$ with d'_w = 5/8 in. (15.9 mm). The 5/8 in. (15.9 mm) limit does not apply to solid steel washers in full contact with the sheet metal. In accordance with *Specification* Section E4, testing is allowed as an alternative method to determine fastener capacity. To use test data in design, the tested material should be consistent with the design. When a polygon-shaped washer is used and capacity is determined using *Specification* Equation E4.4.2-1, the washer should have rounded corners to prevent premature tearing.

In 2010, the pancake head washer screws and domed washers integral with the screw head were added and defined to assist the designer in proper determination of computational variables.

E4.4.3 Tension in Screws

Tensile strength of the screw fastener itself should be known and documented from testing. Screw strength should be established and published by the manufacturer. In order to prevent the brittle and sudden tensile fracture of the screw, the *Specification* applies a 25 percent adjustment to the *safety factor* or the *resistance factor* where determined in accordance with Section F1.

E4.5 Combined Shear and Tension

Section E4.5 checks three failure modes where shear and tension are present at a *connection*: *connection* failures due to combined shear and pull-over and combined shear and pull-out, as well as screw failure in the shank due to combined shear and tension.

E4.5.1 Combined Shear and Pull-Over

Research pertaining to the behavior of a screw *connection* has been conducted at West Virginia University (Luttrell, 1999). Based on the review and analysis of West Virginia University's data for the behavior of a screw *connection* subject to combined shear and tension (Zwick and LaBoube, 2002), equations were derived that enable the evaluation of the strength of a screw *connection* when subjected to combined shear and tension. The tests indicated that at failure, the sheet beneath the screw head pulled over the head of the screw or the washer. Therefore, the *nominal tensile strength* [*resistance*] is based solely on P_{nov}. Although both non-linear and linear equations were developed for ease of computation and because the linear equation provides regions of Q/P_{ns} and T/P_{nov} equal to unity, the linear equation was adopted for the *Specification*. The proposed equation is based on the following test program limits:

0.0285 in. (0.724 mm) $\le t_1 \le 0.0445$ in. (1.13 mm)

No. 12 and No. 14 self-drilling screws with or without washers

$d_w \le 0.75$ in. (19.1 mm)

62 ksi (427 MPa or 4360 kg/cm^2) $\le F_{u1} \le 70.7$ ksi (487 MPa or 4970 kg/cm^2)

$t_2 / t_1 \ge 2.5$

The limit $t_2 / t_1 \ge 2.5$ reflects the fact that the test program (Luttrell, 1999) focused on *connections* having sheet *thicknesses* that precluded the tilting limit state from occurring. Thus, this limit ensures that the design equations will only be used when tilting limit state is not the controlling limit state.

The standard washer with outside diameter of 3/4 in. (19.1 mm) has a minimum *thickness* of 0.063 in. (1.60 mm). In 2011, the washer dimension limitations of *Specification* Sections E4.4 and E4.5 were harmonized, given similar pull-over models in the two sections.

The linear form of the equation as adopted by the *Specification* is similar to the following more conservative linear design equation that has been used by engineers:

$$Q/P_{ns} + T/P_{nov} \le 1.0$$

An eccentric *load* on a clip *connection* may create a non-uniform *stress* distribution around the fastener. For example, tension tests on roof panel welded *connections* have shown that under an eccentrically applied tension force, the resulting *connection* capacity is 50 percent of the tension capacity under a uniformly applied tension force. Thus, the

Specification stipulates that the pull-over strength shall be taken as 50 percent of P_{nov}. If the eccentric *load* is applied by a rigid member such as a clip, the resulting tension force on the screw may be uniform; thus the force in the screw can be determined by mechanics, and the capacity of the fastener should be reliably estimated by P_{nov}. Based on the field performance of screw-attached panels, the 30 percent reduction associated with welds at side-laps need not be applied when evaluating the strength of side-lap screw *connections* at supports or for sheet-to-sheet. The reduction is due to transverse prying or peeling. It is acceptable to apply the 50 percent reduction at panel ends due to longitudinal prying.

E4.5.2 Combined Shear and Pull-Out

Research pertaining to the behavior of a screw *connection* has been conducted at the Missouri University of Science and Technology (Francka and LaBoube, 2010). Based on the findings of this research, equations were derived that enable the evaluation of the strength of a screw *connection* when subjected to combined shear and tension. The tests indicated that at failure, the screw pulled out of the bottom sheet of the *connection*. Therefore, the *nominal tensile strength* [*resistance*] is based solely on the tilting and tearing failure mode, *Specification* Equation E4.5.2.1-2. Although both non-linear and linear equations were developed, the reliability of the non-linear and linear equations was comparable. Therefore, for ease of computation, the linear equation was adopted for the *Specification*. The proposed equation is based on the test program limits as defined in the *Specification*. Evaluation of the *connection* for the combined shear and pull-out does not negate the need to evaluate the shear alone and pull-out alone limit states.

E4.5.3 Combined Shear and Tension in Screws

In 2012, new provisions were added to account for shear and tension interaction in screws. Based on the engineering rational analysis, the same strength interaction as that used for bolts, *Specification* Equation E3.4-2 (but in a different form) is used for screws.

E5 Power-Actuated Fasteners

In 2011, Section E5 was added to address *connections* with *power-actuated fasteners* (*PAFs*) connecting steel elements in non-diaphragm applications. These provisions do not preclude evaluation of any limit state on any *power-actuated fastener* through manufacturer or independent laboratory testing. The *safety* and *resistance factors* for any *nominal strength* [*resistance*] established through testing should be determined using provisions of Chapter F of the *Specification*.

In *Specification* Section E5, the provisions for determining the *available strengths* [*factored resistance*] were developed based on the study by Mujagic et al. (2010). Applicability constraints of these provisions correspond to the limitations of data available in the study (Mujagic et al., 2010).

In the provisions, the term "near side of the embedment material" refers to the surface of the embedment material from which the *PAF* is driven. The term "far side of the embedment material" refers to the embedment material surface from which the driven fastener exits.

E5.1 Minimum Spacing, Edge and End Distances

The minimum center-to-center spacing of the *PAFs* and the edge distances in the *Specification* are those stipulated by Table 2 of ASTM E1190 (ASTM, 2008). While larger spacing and edge distances are frequently found in test reports, the minimum distances given in ASTM E1190 (ASTM, 2008) are deemed sufficient in eliminating the detrimental effects of inadequate edge distance or fastener grouping.

E5.2 Power Actuated Fasteners in Tension

Applicable limit states in tension include tension fracture, pull-out, and pull-over. The determination of *available strength* [*factored resistance*] due to any particular limit state for the fasteners depicted in *Specification* Figure E5 should be accomplished through appropriate testing. Alternatively, the *available strength* [*factored resistance*] should be determined using Sections E5.2.1 through E5.2.3 of the *Specification*.

E5.2.1 Tension Strength [Resistance]

Power-actuated fasteners typically possess the Rockwell hardness (HRC) values of 49 to 58. Adequate HRC values represent one of the most critical design, installation and behavioral features of *PAFs*. The HRC values can be properly related to *tensile strength* in most ranges of HRC. The study by Mujagic et al. (2010) showed that the *nominal tensile fracture strength* [*resistance*] can be determined using the value of 260,000 psi (1790 MPa) for the HRC range in excess of 52. The user is cautioned to distinguish between the strength properties and HRC of pre-hardened steel from which a fastener is made and those of the hardened steel representing the final fastener product.

Specification Equation E5.2.1-1 was developed with the *PAF* driven such that no part of the length ℓ_{dp}, as illustrated in *Specification* Figure E5, is located above the near side of the embedment material.

E5.2.2 Pull-Out Strength [Resistance]

The *nominal pull-out strength* [*resistance*] of *PAFs* greatly depends on minute metallurgical, geometric, installation, and other design (often proprietary) features. *Power-actuated fasteners* develop their pull-out strength through partial fusion to the embedment material and friction resulting from the confinement *stresses* imposed by the displaced embedment material. Mechanical interlock or keying with *PAF* shank knurling and brazing effects due to zinc plating of the *PAF* also contribute to strength. While various behavioral trends can be established, it is not possible to develop a generic prediction model for *power-actuated fasteners*, which captures the above-mentioned, often proprietary, specific design features. Consequently, it was decided to stipulate testing as the only viable method of determining the pull-out strength. This approach is similar to how the pull-out strength is addressed in the EN 1993-1-3 (CEN 2006). The currently available testing protocols for determining the pull-out strength are given in AISI S905 (AISI, 2008) and ASTM E1190 (ASTM, 2008).

The tabulated *nominal pull-out strengths* [*resistances*] in Table C-E5.2.2-1 are provided for informational purposes. The table is extracted from the study by Mujagic et al. (2010), and it represents lower bound values from a limited selection of industry fastener and embedment plate combinations available to the study. Table C-E5.2.2-1 is only applicable to fasteners embedded in steel plate for which manufacturer applicability guidelines stipulate embedment condition whereby no part of the length ℓ_{dp} of *PAF point*, as illustrated in *Specification* Figure E5, is located above the near side of the embedment material. The values in Table C-E5.2.2-1 were scaled such that a *safety factor* of 3.0 computed in accordance with Chapter F of the *Specification* can be justified for the *nominal strength* [*resistance*] value of each of the considered fasteners. Since these are lower bound solutions, the actual *safety factor* for some of the fasteners would be higher than 3.0. The table is only applicable to fastener types and geometries depicted in *Specification* Figure E5. The current design practice generally involves reliance on tested capacities established per International Code Council Evaluation Services (ICC-ES) Acceptance Criteria 70 (AC70) (ICC-ES, 2010). The AC70 stipulates a minimum *safety factor* of 5.0, thus in many cases resulting in lower *allowable strength* values than those implied by Table C-E5.2.2-1. The approaches for establishing the *safety factor* stipulated by Chapter F of the *Specification* and by ICC-ES AC70 are not consistent. However, the values in Table C-E5.2.2-1 can be conservatively related to the current practice by reducing the *nominal strength* [*resistance*] values given therein by a factor of 0.6 (i.e., 3/5).

Table C-E5.2.2-1
Nominal Tensile Pull-Out Strength of PAFs, P_{not}, lbs (N)

	Embedment Thickness, in. (mm)		
PAF Shank Diameter, d_s, in. (mm)	1/8 (3.18)	3/16 (4.76)	1/4 (6.35)
0.106 (2.69) ≤ d_s < 0.146 (3.71)	450 (2000)	915 (4070)	1230 (5470)
0.177 (4.50) ≤ d_s < 0.206 (5.23)	-	-	1970 (8760)

Where statistical indices required to compute the *safety* and *resistance factors* in accordance with *Specification* Chapter F are not given for a pull-out strength provided by a manufacturer, a *safety factor* of 4.0 and *a resistance factor* of 0.40 (0.35 for *LSD*) can be applied to the nominal strengths provided in Table C-E5.2.2-1. This option was provided based on the study by Mujagic et al. (2010) which shows that 4.0 represents a conservative lower bound value of *safety factor* for a variety of fastener types and models, when the computed *safety factor* or data required for its computation is not available to the user.

E5.2.3 Pull-Over Strength [Resistance]

The pull-over limit state in *PAF connections* is fundamentally the same as that in screw *connections*. The *Specification* addresses the screw-like *PAFs* in an identical manner that screw *connections* are dealt with in *Specification* Section E4. The two notable exceptions represent *connections* with tapered-head fasteners that consistently yield about 20% lower pull-over strength than screw-like *PAF connections*, and *connections* with collapsible spring washers that consistently yield about 30% higher strength than screw-like *PAF connections*. The *Specification* addresses the two special cases by varying the constant multiplier of the pull-over equation.

E5.3 Power Actuated Fasteners in Shear

Applicable limit states in shear are shear fracture, bearing and tilting, pull-out, net section checks, and *nominal shear strength* [*resistance*] limited by edge distance.

E5.3.1 Shear Strength [Resistance]

Nominal shear strength [*resistance*] is determined by relating the ultimate *tensile strength* in tension to that in shear by a factor of 0.6.

E5.3.2 Bearing and Tilting Strength [Resistance]

The *nominal bearing strength* [*resistance*] is based on the equation proposed in the study by Mujagic et al. (2010) based on the data for which $t_2/t_1 \geq 2.0$ and $t_2 \geq 1/8$ in. (3.2 mm). While some decrease in calculated strength was observed with decreasing t_2/t_1 ratio, thus suggesting the presence of tilting at lower ratios of t_2/t_1, it was noted that the bearing and tilting strength can be predicted by setting the constant multiplier in the bearing equation to 3.7. Since the study by Mujagic et al. (2010) was based only on the types of fasteners shown in *Specification* Figures E5(c) and E5(d), the ENV 1993-1-3 (ECS, 2006) equation constant of 3.2 is conservatively adopted for other types of *PAFs*.

E5.3.3 Pull-Out Strength [Resistance] in Shear

Pull-out in shear is essentially a derivative of fastener tilting. The pull-out failures were reported at wide range of t_2/t_1 ratios. The *bearing* strength equation of *Specification* Section E5.3.2 considers the effect of tilting deformation on *bearing* failures at low ratios of t_2/t_1. However, as expected, it does not accurately predict the *connection* strength where tilting is the predicted failure mode. The *Specification*, therefore, stipulates a separate pull-out check over the entire range of t_2/t_1 ratios and *thicknesses* covered by the *Specification*.

E5.3.4 Net Section Rupture Strength [Resistance]

Based on the recommendations of Beck and Engelhardt (2002), the *PAF* hole is required to be calculated based on a width of 1.10 times the *PAF* diameter. The effect of partially driven *PAFs* (i.e., where the *PAF point length*, ℓ_{dp}, is fully or partially located in the embedment material) on net section properties of a *connection* are not presently known. The *Specification*, therefore, stipulates that the *PAF* shank diameter, d_s, be used in determination of net section properties.

E5.3.5 Shear Strength [Resistance] Limited by Edge Distance

The *Specification* presently stipulates the application of the same criteria given for screws in *Specification* Section E6.1, recognizing fundamental similarities in behavior and application of screw and *PAF connections*. Favorable local effects of sheath folding and local hardening of the sheathing near the *PAF* hole may render the screw *connection* criteria slightly conservative when applied to *PAF connections*. The effect of partially driven *PAFs* (i.e., where the *PAF point length*, ℓ_{dp}, is fully or partially located in the embedment material) on edge distance properties of a *connection* are not presently known. The *Specification*, therefore, stipulates that the *PAF* shank diameter, d_s, be used in edge distance checks.

E5.4 Combined Tension and Shear

Combined tension and shear in the *PAF connection* should include the interaction of combined shear and pull-over, combined shear and pull-out, and fracture due to combined shear and tension on the *PAF* fastener itself. Currently available research does not address *PAF connections* subject to combined tension and shear. Consequently, the *Specification* does not at present provide equations for consideration of such *connections*. The ICC-ES AC 70 (ICC-ES, 2010) criteria can be used to consider combined tension and shear through testing. Alternatively, such a condition can be evaluated in accordance with *Specification* Section A1.2. Based upon fundamental principles of fastener mechanics, Equation C-E5.4-1 represents an exact interaction between tension and shear when fastener fracture governs. Since the actual interaction curve is not presently known for other combinations of tension and shear limit states, the power coefficient of one, rendering the Equation C-E5.4-1 a linear interaction, can be used as a conservative check when both shear and tension are not limited by fracture.

$$\left(\frac{T_r}{T_c}\right)^n + \left(\frac{V_r}{V_c}\right)^n \leq 1.0 \qquad \text{(C-E5.4-1)}$$

where

T_r = *required tension strength* [force due to *factored loads*]

T_c = tension strength determined in accordance with *Specification* Section E5.2

V_r = *required shear strength* [shear force due to *factored loads*]

V_c = shear strength determined in accordance with *Specification* Section E5.3

n = 2 when both tension and shear are governed by the fracture limit state

= 1 in all other cases

E6 Rupture

The provisions contained in *Specification* Section E6 and its subsections are applicable only when the thinnest connected part is 3/16 inch (4.76 mm) or less in *thickness*. For materials thicker than 3/16 inch (4.76 mm), the design should follow the specifications or standards stipulated in *Specification* Section E6a of Appendix A or B. A,B

Significant changes were made to the format of *Specification* Section E6 in 2010. *Connections* may be subject to shear rupture, tension rupture, block failure in tension, block failure, or any combinations of these failures in shear depending upon the relationship of the connectors to the *connection* geometry and loading direction. *Specification* Table E6.2-1 provides adjustment factors consistent with prior editions of the *Specification* to cover shear lag factors. Other adjustment factors provide allowances for staggered connector patterns and non-uniform *stress* distribution on the tensile plane. In 2012, the committee added a reference to *PAFs* in Table E6-1, permitting the use of the same *safety* and *resistance factors* as for screws. This step was taken recognizing inherent similarities in configurations and behavior of screw and *PAF connections* as they relate to net fracture of connected elements. Furthermore, partial fusion occurring between the embedment steel and *PAF* should result in a conservative design with respect to application of *resistance* and *safety factors* for screw *connections*.

(a) Shear Lag for Flat Sheet Connections

Previous tests showed that for flat sheet *connections* using a single bolt or a single row having multiple bolts perpendicular to the force (Chong and Matlock, 1975; Carill, LaBoube and

Yu, 1994), the *joint* rotation and out-of-plane deformation of flat sheets are excessive. The strength reduction due to tearing of steel sheets in the net section is considered by *Specification* Equations E6.2-4, E6.2-5, and E6.2-6 contained in Table E6.2-1 according to the d/s ratio and the use of washers (AISI, 1996; Fox and Schuster, 2007). For flat sheet *connections* using multiple connectors in the line of force and having less out-of-plane deformations, the strength reduction is not required in the 2012 edition of the *Specification* (Rogers and Hancock, 1998).

(b) Staggered Holes

The presence of staggered or diagonal hole patterns in a bolted *connection* has long been recognized as increasing the net section area for the limit state of rupture in the net section. LaBoube and Yu (1995) summarized the findings of a limited study of the behavior of bolted *connections* having staggered hole patterns. The research showed that when a staggered hole pattern is present, the width of a rupture plane could be adjusted by use of $s'^2/4g$ with an additional 10 percent reduction factor. More recent testing on the critical tensile path involving stagger has been carried out by Fox and Schuster (2010), the results of which indicate that the 10 percent reduction is not required. Based on this study, the 10 percent reduction factor has been removed in the 2012 edition of the *Specification*.

(c) Shear Lag for Other Than Flat Sheet Connections

Shear lag has a debilitating effect on the tensile capacity of a cross-section. Based on The University of Missouri-Rolla research (LaBoube and Yu, 1995), design equations have been developed that can be used to estimate the influence of the shear lag. The research demonstrated that the shear lag effect differs for an angle and a channel. For both cross-sections, however, the key parameters that influence shear lag are the distance from the shear plane to the center of gravity of the cross-section and the length of the *connection* (See Figures C-E6-1 and C-E6-2). The research showed that for cold-formed steel sections using single-bolt *connections*, *bearing* usually controlled the *nominal strength* [*resistance*], not rupture in the net section.

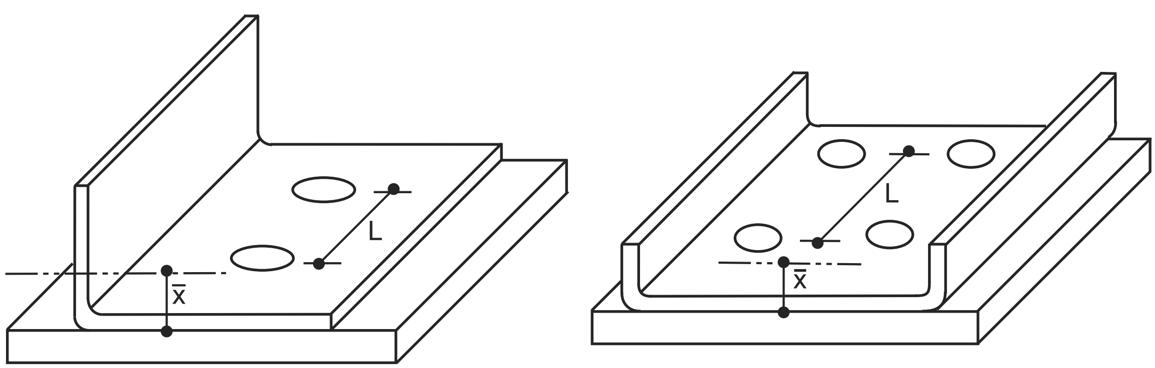


Figure C-E6-1 $\bar{x}$ Definition for Sections With Bolted Connections

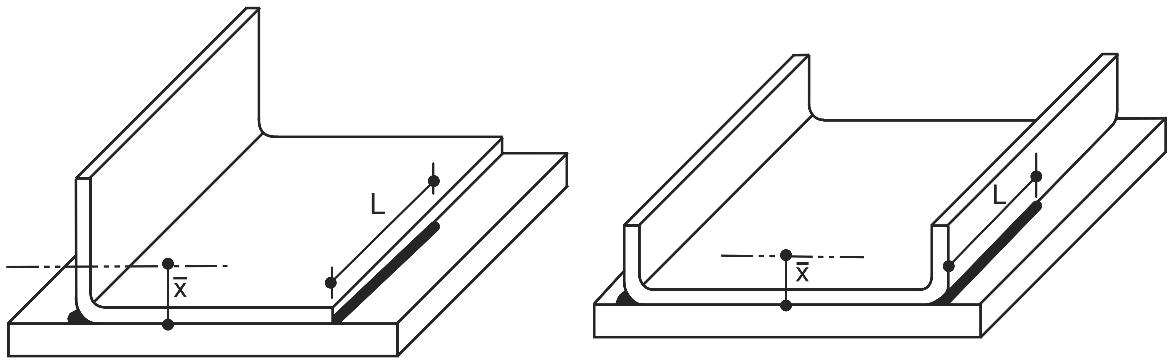


Figure C-E6-2 $\bar{x}$ Definition for Sections With Fillet Welding

(d) Block Shear

Block shear is a limit state in which the resistance is determined by the sum of the shear strength on a failure path(s) parallel to the force and the tensile strength on the segment(s) perpendicular to the force. A comprehensive test program does not exist regarding block shear for cold-formed steel members. However, a limited study conducted at the University of Missouri-Rolla indicates that the AISC equations may be applied to cold-formed steel members.

Block shear is a rupture or tearing phenomenon, not a yielding limit state. However, gross yielding on the shear plane can occur when tearing on the tensile plane. *Specification* Equations E6.3-1 and E6.3-2 check both conditions.

Connection tests conducted by Birkemoe and Gilmor (1978) have shown that on coped beams, a tearing failure mode as shown in Figure C-E6-5 can occur along the perimeter of the holes. Hardash and Bjorhovde (1985) have demonstrated these effects for tension members as illustrated in Figure C-E6-4. The research paper "AISC *LRFD* Rules for Block Shear in Bolted Connections – A Review" (Kulak and Grondin, 2001) provides a summary of test data for block shear *rupture strength*.

The distribution of tensile *stresses* is not always uniform (Ricles and Yura, 1983; Kulak and Grondin, 2001). For shear forces on coped beams, an additional multiplier, U_{bs}, of 0.5 is used when more than one row of bolts is present. This approach is consistent with the provisions of ANSI/AISC 360 (AISC, 2005 and 2010).

Tests performed at the University at Missouri-Rolla have indicated that the current design equations for shear and tilting provide a reasonably good estimate of the *connection* performance for multiple screws in a pattern (LaBoube and Sokol, 2002).

Examples of failure paths can be found in Figures C-E6-3 through C-E6-7.

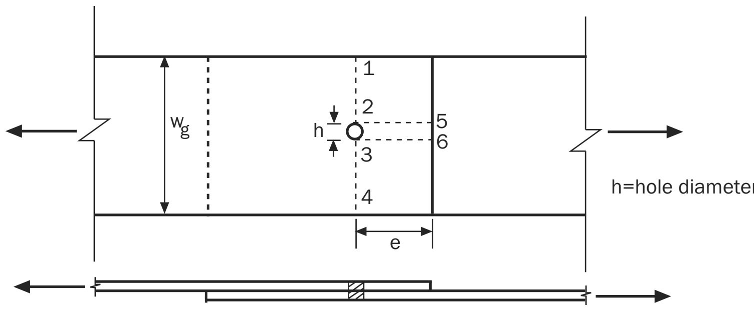


Figure C-E6-3 Potential Failure Paths of Single Lap Joint

(Tension Failure)

Failure Path 1, 2, 3, 4

Specification Section E6.2 applies

$A_e = U_{sl}A_{nt}$

U_{sl} in accordance with *Specification* Equations E6.2-4, E6.2-5, or E6.2-6

$A_{nt} = (w_g - h)\ t$

(Shear Failure)

Failure Path 5, 2, 3, 6

Specification Section E6.1 applies

$A_{nv} = 2n(e - 1/2h)\ t$

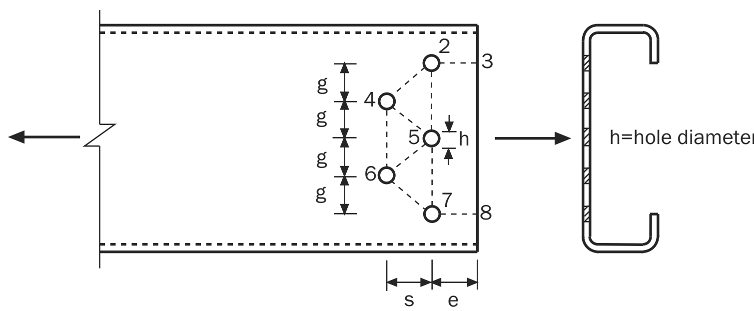


Figure C-E5-4 Potential Failure Paths of Stiffened Channel (Block Shear)

n = 1 as there is only a single fastener

Failure Path 3, 2, 4, 5, 6, 7, 8

Specification Section E6.3 applies

$A_{gv} = 2et$

$A_{nv} = 2(e - 1/2h)\ t$

$A_{nt} = 4(g + s^2/4g - h)\ t$

$U_{bs} = 1.0$

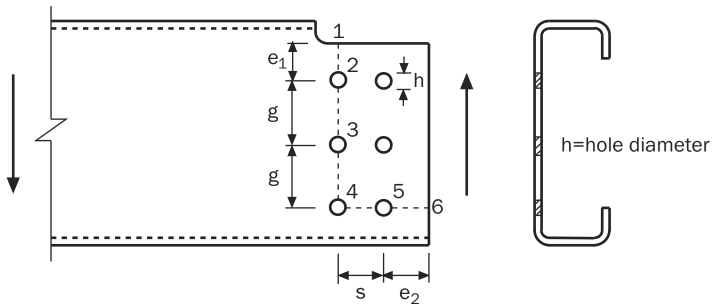


Figure C-E6-5 Potential Failure Path of Coped Stiffened Channel (Block Shear)

Failure Path 1, 2, 3, 4, 5, 6

Specification Section E6.3 applies

$A_{gv} = (2g + e_1)\ t$

$A_{nv} = A_{gv} - 2.5ht$

$A_{nt} = [(s + e_2) - 1.5h]\ t$

$U_{bs} = 0.5$

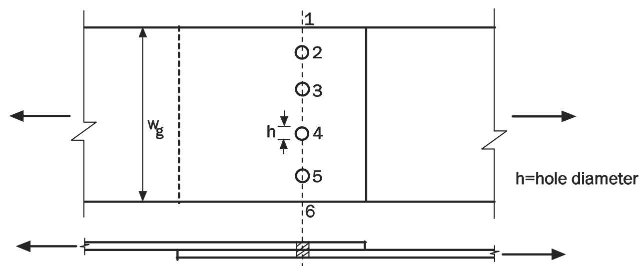


Figure C-E6-6 Potential Failure Path of Multiple-Fastener Lap Joint (Tension)

Failure Path 1, 2, 3, 4, 5, 6

Specification Section E6.2 applies

$A_e = U_{sl}A_{nt}$

U_{sl} in accordance with *Specification* Eq. E6.2-4, E6.2-5, or E6.2-6

$A_{nt} = (w_g - 4h)\ t$

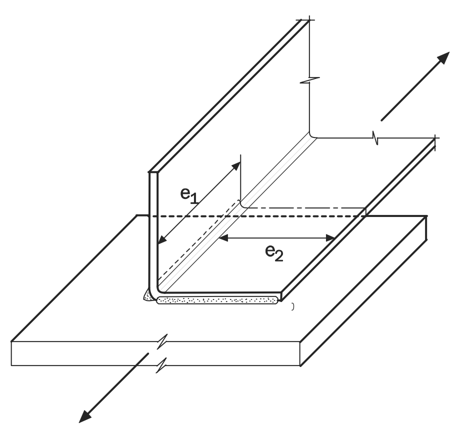


Figure C-E6-7 Potential Failure Path of Fillet-Welded Joint

Specification Section E6.2 applies

$U_{sl} = 1.0 - 1.20\ \bar{x}\ /e_1 \leq 0.9$ (*Specification Eq*. E6.2-7)

E7 Connections to Other Materials

E7.1 Bearing

The design provisions for the *nominal bearing strength* [*resistance*] on the other materials should be derived from appropriate material specifications.

E7.2 Tension

This section is included in the *Specification* to raise the awareness of the design engineer regarding tension on fasteners and the connected parts.

E7.3 Shear

This section is included in the *Specification* to raise the awareness of the design engineer regarding the transfer of shear forces from steel components to adjacent components of other materials.

F. TESTS FOR SPECIAL CASES

All tests for: (1) the determination and confirmation of structural performance, and (2) the determination of mechanical properties must be made by an independent testing laboratory or by a manufacturer's testing laboratory. Information on tests for cold-formed steel *diaphragms* can be found in *Design of Light Gage Steel Diaphragms* (AISI, 1967). A general discussion of structural *diaphragms* is given in *Cold-Formed Steel Design* (Yu and LaBoube, 2010).

F1 Tests for Determining Structural Performance

This *Specification* section contains provisions for proof of structural adequacy by *load* tests. This section is restricted to those cases permitted under Section A1.2 of the *Specification* or specifically permitted elsewhere in the *Specification*.

F1.1 Load and Resistance Factor Design and Limit States Design

The determination of *load*-carrying capacity of the tested elements, assemblies, *connections*, or members is based on the same procedures used to calibrate the *LRFD* design criteria, for which the ϕ factor can be computed from Equation C-A5.1.1-15. The correction factor, C_P, is used in *Specification* Equation F1.1-2 for determining the ϕ factor to account for the influence due to a small number of tests (Peköz and Hall, 1988b and Tsai, 1992). It should be noted that when the number of tests is large enough, the effect of the correction factor is negligible. In the 1996 edition of the AISI *Specification*, Equation F1.1-4 was revised because the old formula for C_P could be unconservative for combinations of a high V_P and a small sample size (Tsai, 1992). This revision enables the reduction of the minimum number of tests from four to three identical specimens. Consequently, the ±10 percent deviation limit was relaxed to ±15 percent. The use of C_P with a minimum V_P reduces the need for this restriction. In *Specification* Equation F1.1-4, a numerical value of $C_P = 5.7$ was found for $n = 3$ by comparison with a two-parameter method developed by Tsai (1992). It is based on the given value of V_Q and other statistics listed in *Specification* Table F1, assuming that V_P will be no larger than about 0.20. The requirements of *Specification* Section F1.1(a) for $n = 3$ help to ensure this.

The 0.065 minimum value of V_P, when used in *Specification* Equation F1.1-2 for the case of three tests, produces *safety factors* similar to those of the 1986 edition of the AISI *ASD Specification*, i.e. approximately 2.0 for members and 2.5 for *connections*. The *LRFD* calibration reported by Hsiao, Yu and Galambos (1988a) indicates that V_P is almost always greater than 0.065 for common cold-formed steel components, and can sometimes reach values of 0.20 or more. The minimum value for V_P helps to prevent potential unconservatism compared to values of V_P implied in *LRFD* design criteria.

In evaluating the coefficient of variation V_P from test data, care must be taken to use the coefficient of variation for a sample. This can be calculated as follows:

$$V_P = \frac{\sqrt{s^2}}{R_n} \qquad \text{C-F1.1-1}$$

where

s^2 = sample variance of all test results

$$= \frac{1}{n-1}\sum_{i=1}^{n}\left(R_i - R_n\right)^2 \qquad \text{C-F1.1-2}$$

R_n = mean of all test results

R_i = test result i of n total results

Alternatively, V_P can be calculated as the sample standard deviation of n ratios R_i/R_n.

If the *nominal strength* [*resistance*] is determined in accordance with a *rational engineering analysis* while the *safety* and *resistance factors* are calculated based on tests, the coefficient of variation, V_P, is determined in accordance with *Specification* Equation F1.1-6 with P_m determined in accordance with *Specification* Equation F1.1-3.

For beams having tension *flange* through-fastened to deck or sheathing and with compression *flange* laterally unbraced (subject to wind uplift), the calibration is based on a *load* combination of 1.17W-0.9D with D/W = 0.1 (see Section D6.1.1 of this *Commentary* for detailed discussion).

The statistical data needed for the determination of the *resistance factor* are listed in *Specification* Table F1. The data listed for screw *connections* were added in 1996 on the basis of the study of bolted *connections* reported by Rang, Galambos, and Yu (1979b). The same statistical data of M_m, V_M, F_m, and V_F have been used by Peköz in the development of the design criteria for screw *connections* (Peköz, 1990).

In 1999, two entries were added to Table F1–one for “Structural Members Not Listed Above” and the other for “Connections Not Listed Above.” It was considered necessary to include these values for members and *connections* not covered by one of the existing classifications. The statistical values were taken as the most conservative values in the existing table.

In 2004, the statistic data, V_M, for screw *bearing* strength was revised from 0.10 to 0.08. This revision is based on the *tensile strength* statistic data provided in the University of Missouri-Rolla research report (Rang, Galambos, and Yu, 1979b). In addition, V_f was revised from 0.10 to 0.05 to reflect the tolerance of the cross-sectional area of the screw.

In 2007, additional entries were made to Table F1 to provide statistical data for all limit states included within the *Specification* for the standard *connection* types. The entry "Connections Not Listed Above" is intended to provide statistical data for *connections* other than welded, bolted, or screwed.

Also in 2007, the *Specification* more clearly defined the appropriate material properties that are to be used when evaluating test results by specifying that supplier provided properties are not to be used.

In 2012, statistical data of M_m, V_M, F_m, and V_F were added for *power-actuated fasteners* to accompany the newly created *Specification* Section E5, based on the study by Mujagic et al. (2010).

In 2012, Section F1.1(c) was revised to permit the use of mill certificates to establish the mechanical properties of small connectors and devices. As a general practice, the *yield stress*, F_y, is determined by testing a tensile specimen that is either cut from the test specimen, or the steel coil or sheet used to produce the test specimen. However, for some cold-formed steel components such as small hurricane ties and clips, it is often impossible to cut a standard size or sub-size tensile specimen that would meet the requirements of ASTM A370 (ASTM, 2012). Since mill certificate tensile specimens are taken from the lead or tail of the *master coil* which may not be representative of the entire coil, and because coiling and uncoiling operations can alter mechanical properties, it is necessary to reduce M_m. When using mill certificates instead of tensile specimens for a range of 21 coils (Stauffer and McEntee, 2012), it has been shown that using M_m = 0.85 will provide corresponding ϕ and Ω values that are on average 15% more conservative. In order to use mill certificates to establish material properties, it is important to maintain proper records and procedures that can trace the connector or device to the *master coil*. The use of mill certificates is not permitted for members. In addition, although mill certificates are routinely used to establish the raw material properties for fasteners such as screws or *power-actuated fasteners*, they should not be used to establish the final material properties. This is because the raw steel undergoes secondary operations such as heat treating that alters its final properties.

In 2012, Section A1.2(b) and Section F1.1(b) were added as an optional method to calibrate *safety* and *resistance factors* for a proposed strength theory using test data. In order to use this optional method, sufficient correlation must exist between the proposed strength theory and the test data. The correlation coefficient, C_c, used in this section is a statistical measure of the agreement between the strength predictions ($R_{n,i}$) and test results ($R_{t,i}$):

$$C_c = \frac{n\sum R_{t,i}R_{n,i} - \left(\sum R_{t,i}\right)\left(\sum R_{n,i}\right)}{\sqrt{n\left(\sum R_{t,i}^2\right) - \left(\sum R_{t,i}\right)^2}\sqrt{n\left(\sum R_{n,i}^2\right) - \left(\sum R_{n,i}\right)^2}} \quad \text{(C-F1.1-3)}$$

where

$R_{t,i}$ = tested *strength* [*resistance*], corresponding to test i

$R_{n,i}$ = predicted *nominal strength* [*resistance*], corresponding to test i.

The value of the correlation coefficient reveals information about the potential quality of the proposed strength theory, namely:

(1) High or moderately high positive correlation indicates that the theory and tests either agree substantially as they are, or can be brought into good agreement by using a constant factor. This means that bias factor, P_m, will compensate for the bias, as intended, in the calibration procedure to determine the *resistance factor*.

(2) Low or nearly zero correlation is an indicator of independence; in other words, no relationship between the tests and theory can be discerned. Using the theory will produce bad results and it should be rejected.

(3) Negative correlation indicates that the theory and test data not only disagree but actually have opposite relationships. For example, when the theory says the strength increases, it actually decreases. Using the theory will produce bad results and it should be rejected.

The square of the correlation coefficient is referred to as the coefficient of determination. It gives the proportion of the variance (fluctuation) of one variable (tested *strength* [*resistance*]) that is predicted by the other variable (strength theory). For example, for $C_c^2 = (0.8)^2$, 64% of the variance is accounted for by the theory. Alternative values for the minimum correlation coefficient could be used, but values above C_c = 0.707 have the desirable characteristic that $C_c^2 \geq 0.5$, that is, more than 50% of the variance is explained by the theory.

In general, higher values of the correlation coefficient are desirable, and indicate a better agreement with the theory, lower V_P, and a better result for the product of the *resistance factor* times the *nominal strength* [*resistance*] given by the theory.

Another advantage of a correlation coefficient criterion is that it is less restrictive and easier to satisfy than alternative criteria based on individual deviations, such as a 15% deviation restriction. C_c is obtained from the full data set and does not apply to individual values. Also, there are multiple ways to obtain a good correlation coefficient. For example, if the test data and strength theory differ by a constant factor, i.e., they are proportional; one will still get a correlation coefficient of 1.0 as if they had agreed directly. This advantage also holds for moderately high correlation coefficients as well. As mentioned above, this will improve the effectiveness of bias factor, P_m, and the *resistance factor*.

It is important that users not only test at the upper and lower bounds of the desired parameter range, but that even coverage of tests is provided throughout the range. This is emphasized in the *Specification* in order to ensure that potential minima or maxima within the test range are detected and that the *resistance factor* and *safety factor* calibrated using the test data properly reflect any variation from the minima/maxima.

The *Specification* provides methods for determining the deflection of some members for serviceability consideration, but the *Specification* does not provide serviceability limits. Justification is discussed in Section A8 of the *Commentary*.

F1.2 Allowable Strength Design

The equation for the *safety factor* Ω (*Specification* Equation F1.2-2) converts the *resistance factor* ϕ from *LRFD* test procedures in *Specification* Section F1.1 to an equivalent *safety factor* for the A*llowable Strength Design*. The average of the test results, R_n, is then divided by the *safety factor* to determine an *allowable strength*. It should be noted that *Specification* Equation F1.2-2 is identical with Equation C-A5.1.1-16 for D/L = 0.

F2 Tests for Confirming Structural Performance

Members, *connections* and assemblies that can be designed according to the provisions of Chapters A through E of the *Specification* need no confirmation of calculated results by test. However, special situations may arise where it is desirable to confirm by test the results of calculations. Tests may be called for by the manufacturer, the engineer, or a third party.

Since design is in accordance with the *Specification*, all that is needed is that the tested specimen or assembly demonstrates the strength is not less than the applicable *nominal resistance*, R_n.

F3 Tests for Determining Mechanical Properties

F3.1 Full Section

Explicit methods for utilizing the effects of cold work are incorporated in Section A7.2 of the *Specification*. In that section, it is specified that as-formed mechanical properties, in particular the *yield stress*, can be determined either by full-section tests or by calculating the strength of the corners and computing the weighted average for the strength of corners and flats. The strength of flats can be taken as the virgin strength of the steel before forming, or can be determined by special tension tests on specimens cut from flat portions of the formed section. This *Specification* section spells out in considerable detail the types and methods of these tests, and their number as required for use in *connection* with *Specification* Section A7.2. For details of testing procedures which have been used for such purposes, but which in no way should be regarded as mandatory, see AISI *Specification* (1968), Chajes, Britvec and Winter (1963), and Karren (1967). AISI S902, *Stub-Column Test Method for Effective Area of Cold-Formed Steel Columns,* provides testing procedures (AISI, 2013c).

F3.2 Flat Elements of Formed Sections

Specification Section F3.2 provides the basic requirements for determining the mechanical properties of flat elements of formed sections. These tested properties are to be used in *Specification* Section A7.2 for calculating the average *yield stress* of the formed section by considering the strength increase from cold work of forming.

F3.3 Virgin Steel

For steels other than the ASTM Specifications listed in *Specification* Section A2.1, the tensile properties of the *virgin steel* used for calculating the increased *yield stress* of the formed section should also be determined in accordance with the Standard Methods of ASTM A370 (2012).

G. DESIGN OF COLD-FORMED STEEL STRUCTURAL MEMBERS AND CONNECTIONS FOR CYCLIC LOADING (FATIGUE)

Fatigue in a cold-formed steel member or connection is the process of initiation and subsequent growth of a crack under the action of a cyclic or repetitive *load*. The *fatigue* process commonly occurs at a *stress* level less than the static failure condition.

When *fatigue* is a design consideration, its severity is determined primarily by three factors: (1) the number of cycles of loading, (2) the type of member and connection detail, and (3) the *stress* range at the detail under consideration (Fisher et al., 1998).

Fluctuation in *stress*, which does not involve tensile *stress*, does not cause crack propagation and is not considered to be a *fatigue* situation.

When fabrication details involving more than one category occur at the same location in a member, the design *stress* range at the location must be limited to that of the most restrictive category. By locating notch-producing fabrication details in regions subject to a small range of *stress*, the need for a member larger than required by static loading will often be eliminated.

For axially stressed angle members, the *Specification* allows the effects of eccentricity on the weld group to be ignored provided the weld lengths L_1 and L_2 are proportional such that the centroid of the weld group falls between "$\bar{x}$" and "b/2" in Figure C-G1(a). When the weld lengths L_1 and L_2 are so proportioned, the effects of eccentric *loads* causing moment about x-x in Figure C-G1(b) also need not be considered.

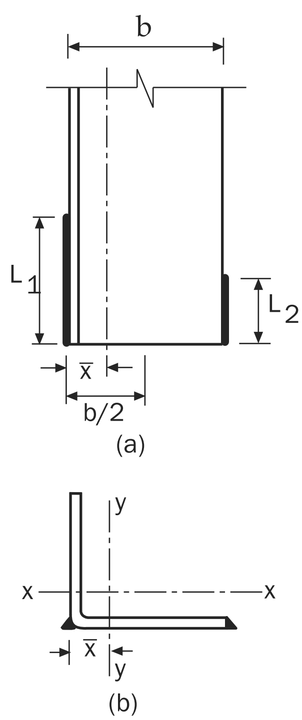


Figure C-G1 Welded Angle Members

Research by Barsom et al. (1980) and Klippstein (1980, 1981, 1985, 1988) developed *fatigue* information on the behavior of sheet and plate steel weldments and mechanical connections. Although research indicates that the values of F_y and F_u do not influence *fatigue* behavior, the *Specification* provisions are based on tests using ASTM A715 (Grade 80), ASTM A607 Grade 60, and SAE 1008 (F_y = 30 ksi). Using regression analysis, mean *fatigue* life curves (S-N curves) with the corresponding standard deviation were developed. The *fatigue* resistance S-N curve has been expressed as an exponential relationship between *stress* range and life cycle (Fisher et al, 1970). The general relationship is often plotted as a linear log-log function, Equation C-G1.

$$\log N = C_f - m \log F_{SR} \quad \text{(C-G1)}$$

$$C_f = b - (n\,s) \quad \text{(C-G2)}$$

where

N = number of full *stress* cycles

m = slope of the mean *fatigue* analysis curve

F_{SR} = effective *stress* range

B = intercept of the mean *fatigue* analysis curve from Table C-G1

n = number of standard deviations to obtain a desired confidence level
= 2 for C_f given in Table G1 of the *Specification*

s = approximate standard deviation of the *fatigue* data
= 0.25 (Klippstein, 1988)

The database for these design provisions is based upon cyclic testing of real *joints*; therefore, *stress* concentrations have been accounted for by the categories in Table G1 of the *Specification*. It is not intended that the allowable *stress* ranges should be compared to “hot-spot” *stresses* determined by finite element analysis. Also, calculated *stresses* computed by ordinary analysis need not be amplified by stress concentration factors at geometrical discontinuities and changes of cross-section. All categories were found to have a common slope with m = -3. Equation G3-1 of the *Specification* is to be used to calculate the design *stress* range for the chosen design life, N. Table G1 of the *Specification* provides a classification system for the various *stress* categories. This also provides the constant, C_f, that is applicable to the *stress* category that is required for calculating design *stress* range, F_{SR}.

Table C-G1 Intercept for Mean Fatigue Curves

Stress Category	b
I	11.0
II	10.5
III	10.0
IV	9.5

The provisions for bolts and threaded parts were taken from the AISC Specification (AISC, 1999).

REFERENCES

Acharya, V.V. and R.M. Schuster (1998), "Bending Tests of Hat Section With Multiple Longitudinal Stiffeners," *Proceedings of the Fourteenth International Specialty Conference on Cold-Formed Steel Structures,* University of Missouri-Rolla, Rolla, MO, October 1998.

Albrecht, R. E. (1988), "Developments and Future Needs in Welding Cold-Formed Steel," *Proceedings of the Ninth International Specialty Conference on Cold-Formed Steel Structures,* University of Missouri-Rolla, Rolla, MO, November 1988.

Allen, D. E. and T. M. Murray (1993), "Designing Criterion for Vibrations Due to Walking," *Engineering Journal,* AISC, Fourth Quarter, 1993.

American Institute of Steel Construction (1978), *Specification for the Design, Fabrication and Erection of Structural Steel for Buildings,* Chicago, IL, November 1978.

American Institute of Steel Construction (1986), *Load and Resistance Factor Design Specification For Structural Steel Buildings,* Chicago, IL, 1986.

American Institute of Steel Construction (1989), *Specification for Structural Steel Buildings - Allowable Stress Design and Plastic Design,* Chicago, IL, 1989.

American Institute of Steel Construction (1993), *Load and Resistance Factor Design Specification for Structural Steel Buildings,* Chicago, IL, December 1993.

American Institute of Steel Construction (1997a), *Steel Design Guide Series 9: Torsional Analysis of Structural Steel Members*, Chicago, IL, 1997.

American Institute of Steel Construction (1997b), *AISC/CISC Steel Design Guide Series 11: Floor Vibration Due to Human Activity*, Chicago, IL, 1997.

American Institute of Steel Construction (1999), *Load and Resistance Factor Design Specification for Structural Steel Buildings*, Chicago, IL, 1999.

American Institute of Steel Construction (2005), *Specification for Structural Steel Buildings*, Chicago, IL, 2005.

American Institute of Steel Construction (2010), *Specification for Structural Steel Buildings*, Chicago, IL, 2010.

American Iron and Steel Institute (1946), *Specification for the Design of Light Gage Steel Structural Members,* New York, NY, 1946.

American Iron and Steel Institute (1949), *Light Gage Steel Design Manual,* New York, NY, 1949.

American Iron and Steel Institute (1956), *Light Gage Cold-Formed Steel Design Manual* (Part I - Specification, Part II - Supplementary Information, Part III - Illustrative Examples, Part IV - Charts and Tables of Structural Properties, and Appendix), New York, NY, 1956.

American Iron and Steel Institute (1960), *Specification for the Design of Light Gage Cold-Formed Steel Structural Members,* New York, NY, 1960.

American Iron and Steel Institute (1961), *Light Gage Cold-Formed Steel Design Manual* (Part I - Specification, Part II - Supplementary Information, Part III - Illustrative Examples, Part IV - Charts and Tables of Structural Properties, and Appendix), New York, NY, 1961.

American Iron and Steel Institute (1962), *Light Gage Cold-Formed Steel Design Manual* (Part I - Specification, Part II - Supplementary Information, Part III - Illustrative Examples, Part IV - Charts and Tables of Structural Properties, Appendix, and Commentary on the 1962 Edition of the Specification by George Winter), New York, NY, 1962.

American Iron and Steel Institute (1967), *Design of Light Gage Steel Diaphragms,* First Edition, New York, NY, 1967.

American Iron and Steel Institute (1968), *Specification for the Design of Cold-Formed Steel Structural Members,* New York, NY, 1968.

American Iron and Steel Institute (1977), *Cold-Formed Steel Design Manual* (Part I - Specification, 1968 Edition; Part II - Commentary by George Winter, 1970 Edition; Part IV - Illustrative Examples, 1972 Edition, March 1977; and Part V - Charts and Tables, 1977 Edition), Washington, DC, 1977.

American Iron and Steel Institute (1983), *Cold-Formed Steel Design Manual* (Part I - Specification, 1980 Edition, Part II - Commentary, Part III - Supplementary Information, Part IV - Illustrative Examples, Part V - Charts and Tables), Washington, DC, 1983.

American Iron and Steel Institute (1986), *Cold-Formed Steel Design Manual* (Part I - Specification, 1986 Edition With the 1989 Addendum, Part II - Commentary, 1986 Edition with the 1989 Addendum, Part III - Supplementary Information, Part IV - Illustrative Examples, Part V - Charts and Tables, Part VI - Computer Aids, Part VII - Test Procedures), Washington, DC, 1986.

American Iron and Steel Institute (1991), *LRFD Cold-Formed Steel Design Manual* (Part I - Specification, Part II - Commentary, Part III - Supplementary Information, Part IV - Illustrative Examples, Part V - Charts and Tables, Part VI - Computer Aids, Part VII - Test Procedures), Washington, DC, 1991.

American Iron and Steel Institute (1992), "Test Methods for Mechanically Fastened Cold-Formed Steel Connections," Research Report CF92-2, Washington, DC, 1992.

American Iron and Steel Institute (1995), "Design Guide for Cold-Formed Steel Trusses," Publication RG-95-18, Washington, DC, 1995.

American Iron and Steel Institute (1996), *Cold-Formed Steel Design Manual,* Washington, DC, 1996.

American Iron and Steel Institute (1999), *Specification for the Design of Cold-Formed Steel Structural Members With Commentary,* 1996 Edition, Supplement No. 1, Washington, DC, 1999.

American Iron and Steel Institute (2001), *North American Specification for the Design of Cold-Formed Steel Structural Members With Commentary,* Washington, DC, 2001.

American Iron and Steel Institute (2002), *Cold-Formed Steel Design Manual*, Washington, DC, 2002.

American Iron and Steel Institute (2004a), *Standard for Cold-Formed Steel Framing – Wall Stud Design*, Washington, DC, 2004.

American Iron and Steel Institute (2004b), *Supplement 2004 to the North American Specification for the Design of Cold-Formed Steel Structural Members, 2001 Edition*, Washington, DC, 2004.

American Iron and Steel Institute (2005), AISI S905, *Test Procedure for Determining a Strength Value for a Roof Panel-to-Purlin-to-Anchorage Device Connection*, Washington, DC, 2005.

American Iron and Steel Institute (2006), *Direct Strength Method (DSM) Design Guide*, Design Guide 06-1, Washington, DC, 2006.

American Iron and Steel Institute (2007a), *North American Specification for the Design of Cold-Formed Steel Structural Members*, Washington, DC, 2007.

American Iron and Steel Institute (2007b), *Commentary on North American Specification for the Design of Cold-Formed Steel Structural Members*, Washington, DC, 2007.

American Iron and Steel Institute (2008), *Cold-Formed Steel Design Manual*, Washington, DC, 2008.

American Iron and Steel Institute (2012a), *North American Specification for the Design of Cold-Formed Steel Structural Members*, Washington, DC, 2012.

American Iron and Steel Institute (2012b), *Commentary on North American Specification for the Design of Cold-Formed Steel Structural Members*, Washington, DC, 2012.

American Iron and Steel Institute (2013), *Cold-Formed Steel Design Manual*, Washington, DC, 2013.

American Iron and Steel Institute (2013b), AISI S903, *Standard Methods for Determination of Uniform and Local Ductility*, Washington, DC, 2013.

American Iron and Steel Institute (2013c), AISI S902, *Stub-Column Test Method for Effective Area of Cold-Formed Steel Columns*, Washington, DC, 2013.

American Iron and Steel Institute (2013d), AISI S906, *Standard Procedures for Panel and Anchor Structural Tests*, Washington, DC, 2013.

American Iron and Steel Institute (2013e), AISI S907, *Test Standard for Cantilever Test Method for Cold-Formed Steel Diaphragms*, Washington, DC, 2013.

American Iron and Steel Institute (2013f), AISI S908, *Base Test Method for Purlins Supporting a Standing Seam Roof System*, Washington, DC, 2013.

American Society of Civil Engineers (1991), *Specification for the Design and Construction of Composite Slabs and Commentary on Specifications for the Design and Construction of Composite Slabs*, ANSI/ASCE 3-91, 1991.

American Society of Civil Engineers (1998), *Minimum Design Loads for Buildings and Other Structures,* ASCE Standard 7-98, 1998.

American Society of Civil Engineers (2005), *Minimum Design Loads for Buildings and Other Structures*, ASCE/SEI 7-05, Reston, VA, 2005.

American Society of Civil Engineers (2010), *Minimum Design Loads for Buildings and Other Structures*, ASCE Standard ASCE/SEI 7-10, 2010.

American Welding Society (1966), *Recommended Practice for Resistance Welding,* AWS C1.1-66, Miami, FL, 1966.

American Welding Society (1970), *Recommended Practice for Resistance Welding Coated Low Carbon Steels,* AWS C1.3-70, (Reaffirmed 1987), Miami, FL, 1970.

American Welding Society (1996), *Structural Welding Code - Steel*, ANSI/AWS D1.1-96, Miami, FL, 1996.

American Welding Society (1998), *Structural Welding Code - Sheet Steel,* ANSI/AWS D1.3-98, Doral, FL, 1998.

American Welding Society (2000), *Recommended Practices for Resistance Welding*, ANSI/AWS C1.1/C1.1M-2000, Miami, FL, 2000.

Applied Technology Council (1999), *ATC Design Guide 1: Minimizing Floor Vibration*, Redwood City, CA, 1999.

ASTM International (2012), A370-12a, *Standard Methods and Definitions for Mechanical Testing of Steel Products*, 2012.

ASTM International (1995), E1592-95, *Standard Test Method for Structural Performance of Sheet Metal Roof and Siding Systems by Uniform Static Air Pressure Difference*, 1995.

ASTM International (2008), E1190-95(Reapproved 2007), *Standard Test Methods for Strength of Power-Actuated Fasteners Installed in Structural Members*, 2007.

ASTM International (2008), A29-05, *Standard Specification for Steel Bars, Carbon and Alloy, Hot-Wrought, General Requirements for*," *ASTM Standards in Building Codes*, 2008.

Standards Australia and the Australian Institute of Steel Construction (1996), AS/NZS (1996), AS/NZS 4600: *1996 Cold-Formed Steel Structures*, 1996.

Bambach, M. R., and K. J. R. Rasmussen (2002a), "Tests on Unstiffened Elements Under Combined Bending and Compression" *Research Report R818*, Department of Civil Engineering, University of Sydney, Australia, May 2002.

Bambach, M.R. and K.J.R. Rasmussen (2002b), "Elastic and Plastic Effective Width Equations for Unstiffened Elements," *Research Report R819*, Department of Civil Engineering, University of Sydney, Australia, 2002.

Bambach, M.R. and K.J.R. Rasmussen (2002c), "Design Methods for Thin-Walled Sections Containing Unstiffened Elements," *Research Report R820*, Department of Civil Engineering, University of Sydney, Australia, 2002.

Barsom, J. M., K. H. Klippstein and A. K. Shoemaker (1980), "Fatigue Behavior of Sheet Steels for Automotive Applications," Research Report SG 80-2, American Iron and Steel Institute, Washington, DC, 1980.

Beck, H. and M.D. Engelhardt (2002), "Net Section Efficiency of Steel Coupons with Power Actuated Fasteners," *ASCE Journal of Structural Engineering*, Vol. 128, Number 1, pp. 12-21, 2002.

Beshara, B. (1999), "Web Crippling of Cold-Formed Steel Members," M.A.Sc. Thesis, University of Waterloo, Waterloo, Canada, 1999.

Beshara, B. and R.M. Schuster (2000), "Web Crippling Data and Calibrations of Cold-Formed Steel Members," Final Report, University of Waterloo, Waterloo, Canada, 2000.

Beshara, B. and R.M. Schuster (2000a), "Web Crippling of Cold-Formed C- and Z-Sections," *Proceedings of the Fifteenth International Specialty Conference on Cold-Formed Steel Structures*, University of Missouri-Rolla, Rolla, MO, October 2000.

Bhakta, B.H., R.A. LaBoube and W.W. Yu (1992), "The Effect of Flange Restraint on Web Crippling Strength," Final Report, Civil Engineering Study 92-1, University of Missouri-Rolla, Rolla, MO, March 1992.

Birkemoe, P. C. and M. I. Gilmor (1978), "Behavior of Bearing-Critical Double-Angle Beam Connections," *Engineering Journal*, AISC, Fourth Quarter, 1978.

Bleich, F. (1952), *Buckling Strength of Metal Structures,* McGraw-Hill Book Co., New York, NY, 1952.

British Standards Institution (1992), *British Standard: Structural Use of Steelwork in Building,* "Part 5 - Code of Practice for Design of Cold-Formed Sections," BS 5950: Part 5: CF92-2, 1992.

Brockenbrough, R. L. (1995), *Fastening of Cold-Formed Steel Framing*, American Iron and Steel Institute, Washington, DC, September 1995.

Bryant, M.R. and T.M. Murray (2001), "Investigation of Inflection Points as Brace Points in Multi-Span Purlin Roof Systems," Report No. CE/VPI-ST 99/08, Virginia Polytechnic Institute and State University, Blackburg, VA, 2001.

Bulson, P. S. (1969), *The Stability of Flat Plates*, American Elsevier Publishing Company, New York, NY, 1969.

Cain, D.E., R.A. LaBoube and W.W. Yu (1995), "The Effect of Flange Restraint on Web Crippling Strength of Cold-Formed Steel Z- and I-Sections," Final Report, Civil Engineering Study 95-2, University of Missouri-Rolla, Rolla, MO, May 1995.

Camara Nacional de la Industria del Hierro y del Acero (1965), *Manual de Diseno de Secciones Estructurales de Acero Formadas en Frio de Calibre Ligero,* Mexico, 1965.

Canadian Standards Association (1994a), *Limit States Design of Steel Structures*, CAN/CSA-S16.1-94, Rexdale, Ontario, Canada, 1994.

Canadian Standards Association (1994b), *Cold Formed Steel Structural Members*, S136-94, Rexdale, Ontario, Canada, 1994.

Canadian Standards Association (1995), *Commentary on CSA Standard S136-94, Cold Formed Steel Structural Members,* S136.1-95, Rexdale, Ontario, Canada, 1995.

Carril, J.L., R. A. LaBoube and W. W. Yu (1994), "Tensile and Bearing Capacities of Bolted Connections," First Summary Report, Civil Engineering Study 94-1, University of Missouri-Rolla, Rolla, MO, May 1994.

CEN (2006), "Eurocode 3 - Design of Steel Structures – Part 1-3: General Rules - Supplementary Rules for Cold Formed Thin Gauge Members and Sheeting (EN 1993-1-3)," ECS, Brussels, Belgium, 2006.

Chajes, A., S. J. Britvec and G. Winter (1963), "Effects of Cold-Straining on Structural Steels," *Journal of the Structural Division,* ASCE, Vol. 89, No. ST2, February 1963.

Chajes, A. and G. Winter (1965), "Torsional-Flexural Buckling of Thin-Walled Members," *Journal of the Structural Division*, ASCE, Vol. 91, No. ST4, August 1965.

Chajes, A., P.J. Fang, and G. Winter (1966), "Torsional-Flexural Buckling, Elastic and Inelastic, of Cold-Formed Thin-Walled Columns," *Engineering Research Bulletin*, No. 66-1, Cornell University, 1966.

Chodraui, G.M.B., Y. Shifferaw, M. Malite and B.W. Schafer (2006), "Cold-Formed Steel Angles Under Axial Compression," *Proceedings of 18th International Specialty Conference on Cold-Formed Steel Structures*, University of Missouri-Rolla, Rolla, MO, October, 2006.

Chong, K. P. and R. B. Matlock (1975), "Light Gage Steel Bolted Connections Without Washers," *Journal of the Structural Division,* ASCE, Vol. 101, No. ST7, July 1975.

Cohen, J. M. and T. B. Peköz (1987), "Local Buckling Behavior of Plate Elements," *Research Report*, Department of Structural Engineering, Cornell University, 1987.

Craig, B. (1999), "Calibration of Web Shear Stress Equations," Canadian Cold Formed Steel Research Group, University of Waterloo, December 1999.

Davis, C. S. and W. W. Yu (1972), "The Structural Performance of Cold-Formed Steel Members With Perforated Elements," Final Report, University of Missouri-Rolla, Rolla, MO, May 1972.

Department of Army (1985), *Seismic Design for Buildings,* U.S. Army Technical Manual 5-809-10, Washington, DC, 1985.

Deshmukh, S. U. (1996), "Behavior of Cold-Formed Steel Web Elements With Web Openings Subjected to Web Crippling and a Combination of Bending and Web Crippling for Interior-One-Flange Loading," Thesis presented to the faculty of the University of Missouri-Rolla in partial fulfillment for the degree of Master of Science, 1996.

Desmond, T. P., T. B. Peköz and G. Winter (1981a), "Edge Stiffeners for Thin-Walled Members," *Journal of the Structural Division*, ASCE, Vol. 107, No. ST2, February 1981.

Desmond, T. P., T. B. Peköz and G. Winter (1981b), "Intermediate Stiffeners for Thin-Walled Members," *Journal of the Structural Division,* ASCE, Vol. 107, No. ST4, April 1981.

DeWolf, J.T., T. B. Peköz and G. Winter (1974), "Local and Overall Buckling of Cold-Formed Steel Members," *Journal of the Structural Division*, ASCE, Vol. 100, October 1974.

Dhalla, A. K., S. J. Errera and G. Winter (1971), "Connections in Thin Low-Ductility Steels," *Journal of the Structural Division,* ASCE, Vol. 97, No. ST10, October 1971.

Dhalla, A. K. and G. Winter (1974a), "Steel Ductility Measurements," *Journal of the Structural Division,* ASCE, Vol. 100, No. ST2, February 1974.

Dhalla, A. K. and G. Winter (1974b), "Suggested Steel Ductility Requirements," *Journal of the Structural Division,* ASCE, Vol. 100, No. ST2, February 1974.

Dinovitzer, A.S., M. Sohrabpour and R.M. Schuster (1992), "Observations and Comments Pertaining to CAN/CSA-S136-M89," *Proceedings of the Eleventh International Specialty Conference on Cold-Formed Steel Structures*, University of Missouri-Rolla, Rolla, MO, October 1992.

Dinis, P.B., D. Camotim and N. Silvestre (2012), "On the Mechanics of Angle Column Instability," *Thin-Walled Structures*, 52(March), 80-89, 2012.

Douty, R. T. (1962), "A Design Approach to the Strength of Laterally Unbraced Compression Flanges," *Bulletin* No. 37, Cornell University, Ithaca, NY, 1962.

Eiler, M. R., R. A. LaBoube and W.W. Yu (1997), "Behavior of Web Elements With Openings Subjected to Linearly Varying Shear," Final Report, Civil Engineering Series 97-5, Cold-Formed Steel Series, Department of Civil Engineering, University of Missouri-Rolla, Rolla, MO, 1997.

Elhouar, S. and T.M. Murray (1985), "Adequacy of Proposed AISI Effective Width Specification Provisions for Z- and C-Purlin Design," Fears Structural Engineering Laboratory, FSEL/MBMA 85-04, University of Oklahoma, Norman, Oklahoma, 1985.

Ellifritt, D. S. (1977), "The Mysterious 1/3 Stress Increase," *Engineering Journal,* AISC, Fourth Quarter, 1977.

Ellifritt, D. S., T. Sputo and J. Haynes (1992), "Flexural Capacity of Discretely Braced C's and Z's," *Proceedings of the Eleventh International Specialty Conference on Cold-Formed Steel Structures,* University of Missouri-Rolla, Rolla, MO, October 1992.

Ellifritt, D. S., R. L. Glover and J. D. Hren (1998), “A Simplified Model for Distortional Buckling of Channels and Zees in Flexure,” *Proceedings of the Fourteenth International Specialty Conference on Cold-Formed Steel Structures*, University of Missouri-Rolla, Rolla, MO, October 1998.

Ellingwood, B., T. V. Galambos, J. G. MacGregor and C. A. Cornell (1980), “Development of a Probability Based Load Criterion for American National Standard A58: Building Code Requirements for Minimum Design Loads in Buildings and Other Structures,” U.S. Department of Commerce, National Bureau of Standards, *NBS Special Publication 577,* June 1980.

Ellingwood, B., J. G. MacGregor, T. V. Galambos and C. A. Cornell (1982), “Probability Based Load Criteria: Load Factors and Load Combinations,” *Journal of the Structural Division,* ASCE, Vol. 108, No. ST5, May 1982.

Ellingwood, B. (1989), “Serviceability Guidelines for Steel Structures,” *Engineering Journal,* AISC, First Quarter, 1989.

European Convention for Constructional Steelwork (1977), “European Recommendations for the Stressed Skin Design of Steel Structures,” ECCS-XVII-77-1E, CONSTRADO, London, March 1977.

European Convention for Constructional Steelwork (1987), “European Recommendations for the Design of Light Gage Steel Members,” First Edition, Brussels, Belgium, 1987.

Fisher, J. M. and M.A. West (1990), *Serviceability Design Considerations for Low-Rise Buildings,* Steel Design Guide Series, AISC, 1990.

Fisher, J. M. (1996), “Uplift Capacity of Simple Span Cee and Zee Members With Through-Fastened Roof Panels,” Final Report, MBMA 95-01, Metal Building Manufacturers Association, 1996.

Fisher, J.W., K. H. Frank, M. A. Hirt and B.M. McNamee (1970), “Effect of Weldments on the Fatigue Strength of Steel Beams,” National Cooperative Highway Research Program *Report 102*, Highway Research Board, Washington, DC, 1970.

Fisher, J. W., G. L. Kulak and I. F.C. Smith (1998), “A Fatigue Primer for Structural Engineers,” National Steel Bridge Alliance, 1998.

Fox, D.M. and R. M. Schuster (2010), “Cold Formed Steel Tension Members with Two and Three Staggered Bolts,” *Proceedings of the Twentieth International Specialty Conference on Cold-Formed Steel Structural Members*, Missouri University of Science and Technology, Rolla, MO, November 2010.

Fox, S.R. (2002), “Bearing Stiffeners in Cold Formed Steel C-Sections,” Ph.D. Thesis, Department of Civil Engineering, University of Waterloo, Waterloo, Ontario, Canada, 2002.

Fox, S.R. and R.M. Schuster (2002), “Bearing Stiffeners in Cold-Formed Steel C-Sections,” Final Report, American Iron and Steel Institute, Washington, DC, 2002.

Francka, R.M. and R.A. LaBoube (2010), “Screw Connections Subject to Tension Pull-Out and Shear Forces,” *Proceedings of the Twentieth International Specialty Conference on Cold-Formed Steel Structures*, Missouri University of Science and Technology, Rolla, MO, October 2010.

Fung, C. (1978), "Strength of Arc-Spot Welds in Sheet Steel Construction," Final Report to Canadian Steel Industries Construction Council (CSICC), Westeel-Rosco Limited, Canada, 1978.

Galambos, T. V. (1963), "Inelastic Buckling of Beams," *Journal of the Structural Division,* ASCE, Vol. 89, No. ST5, October 1963.

Galambos, T. V., B. Ellingwood, J. G. MacGregor and C. A. Cornell (1982), "Probability Based Load Criteria: Assessment of Current Design Practice," *Journal of the Structural Division,* ASCE, Vol. 108, No. ST5, May 1982.

Galambos, T.V. and B. Ellingwood (1986), "Serviceability Limit States: Deflection," ASCE, *Journal of Structural Engineering*, 112 (1) 67-84.

Galambos, T. V. (Editor) (1988a), *Guide to Stability Design Criteria for Metal Structures,* Fourth Edition, John Wiley and Sons, New York, NY, 1988.

Galambos, T. V. (1988b), "Reliability of Structural Steel Systems," Report No. 88-06, American Iron and Steel Institute, Washington, DC, 1988.

Galambos, T. V. (1998), *Guide to Stability Design Criteria for Metal Structures*, Fifth Edition, John Wiley & Sons, Inc., 1998.

Gerges, R.R. (1997) "Web Crippling of Single Web Cold-Formed Steel Members Subjected to End One-Flange Loading," M.A.Sc. Thesis, University of Waterloo, Waterloo, Canada, 1997.

Gerges, R.R. and R.M. Schuster (1998), "Web Crippling of Members Using High-Strength Steels," *Proceedings of the Fourteenth International Specialty Conference on Cold-Formed Steel Structures*, University of Missouri-Rolla, October 1998.

Glaser, N.J., R. C. Kaehler and J. M. Fisher (1994), "Axial Load Capacity of Sheeted C and Z Members," *Proceedings of the Twelfth International Specialty Conference on Cold-Formed Steel Structures*, University of Missouri-Rolla, October 1994.

Green, G. G., G. Winter and T. R. Cuykendall (1947), "Light Gage Steel Columns in Wall-Braced Panels," *Bulletin*, No. 35/2, Cornell University Engineering Experimental Station, 1947.

Green, P.S., T. Sputo and V. Urala (2004), "Bracing Strength and Stiffness Requirements for Axially Loaded Lipped Cee Studs," *Proceedings of the Seventeenth International Specialty Conference on Cold-Formed Steel Structures*, University of Missouri-Rolla, 2004.

Hancock, G. J., Y. B. Kwon and E. S. Bernard (1994), "Strength Design Curves for Thin-Walled Sections Undergoing Distortional Buckling," *Journal of Constructional Steel Research*, Vol. 31, 1994.

Hancock, G. J. (1995), "Design for Distortional Buckling of Flexural Members," *Proceedings of the Third International Conference on Steel and Aluminum Structures*, Istanbul, Turkey, May 1995.

Hancock, G.J. (1997), "Design for Distortional Buckling of Flexural Members," *Thin-Walled Structures*, Vol. 27, No.1, 1997.

Hancock, G.J., C. A. Rogers and R.M. Schuster (1996), "Comparison of the Distortional Buckling Method for Flexural Members with Tests," *Proceedings of the Thirteenth International Specialty Conference on Cold-Formed Steel Structures*, University of Missouri-Rolla, MO, 1996.

Hardash, S. G. and R. Bjorhovde (1985), "New Design Criteria for Gusset Plates in Tension," AISC *Engineering Journal*, Vol. 22, No. 2, 2nd Quarter, 1985.

Harper, M.M., R.A. LaBoube and W. W. Yu (1995), "Behavior of Cold-Formed Steel Roof Trusses," Summary Report, Civil Engineering Study 95-3, University of Missouri-Rolla, Rolla, MO, May 1995.

Harris, P. S. and R. A. LaBoube (1985), "Understanding the Engineering Safety Factor in Building Design," *Plant Engineering,* August 1985.

Hatch, J., W. S. Easterling and T. M. Murray (1990), "Strength Evaluation of Strut-Purlins," *Proceedings of the Tenth International Specialty Conference on Cold-Formed Steel Structures*, University of Missouri-Rolla, October 1990.

Haussler, R. W. (1964), "Strength of Elastically Stabilized Beams," *Journal of Structural Division,* ASCE, Vol. 90, No. ST3, June 1964; also ASCE *Transactions,* Vol. 130, 1965.

Haussler, R. W. and R. F. Pahers (1973), "Connection Strength in Thin Metal Roof Structures," *Proceedings of the Second Specialty Conference on Cold-Formed Steel Structures,* University of Missouri-Rolla, Rolla, MO, October 1973.

Haussler, R. W. (1988), "Theory of Cold-Formed Steel Purlin/Girt Flexure," *Proceedings of the Ninth International Specialty Conference on Cold-Formed Steel Structures,* University of Missouri-Rolla, Rolla, MO, November 1988.

Hetrakul, N. and W. W. Yu (1978), "Structural Behavior of Beam Webs Subjected to Web Crippling and a Combination of Web Crippling and Bending," Final Report, Civil Engineering Study 78-4, University of Missouri-Rolla, Rolla, MO, June 1978.

Hetrakul, N. and W. W. Yu (1980), "Cold-Formed Steel I-Beams Subjected to Combined Bending and Web Crippling," *Thin-Walled Structures - Recent Technical Advances and Trends in Design, Research and Construction,* Rhodes, J. and A. C. Walker (Eds.), Granada Publishing Limited, London, 1980.

Hill, H. N. (1954), "Lateral Buckling of Channels and Z-Beams," *Transactions,* ASCE, Vol. 119, 1954.

Holcomb, B.D., R.A. LaBoube and W. W. Yu (1995), "Tensile and Bearing Capacities of Bolted Connections," Second Summary Report, Civil Engineering Study 95-1, University of Missouri-Rolla, Rolla, MO, May 1995.

Holesapple, M.W. and R.A. LaBoube (2002), "Overhang Effects on End-One-Flange Web Crippling Capacity of Cold-Formed Steel Members," Final Report, Civil Engineering Study 02-1, Cold-Formed Steel Series, University of Missouri-Rolla, MO, 2002.

Hsiao, L. E., W. W. Yu and T. V. Galambos (1988a), "Load and Resistance Factor Design of Cold-Formed Steel: Calibration of the AISI Design Provisions," Ninth Progress Report, Civil Engineering Study 88-2, University of Missouri-Rolla, Rolla, MO, February 1988.

Hsiao, L. E., W. W. Yu and T. V. Galambos (1988b), "Load and Resistance Factor Design of Cold-Formed Steel: Comparative Study of Design Methods for Cold-Formed Steel," Eleventh Progress Report, Civil Engineering Study 88-4, University of Missouri-Rolla, Rolla, MO, February 1988.

Hsiao, L. E. (1989), "Reliability Based Criteria for Cold-Formed Steel Members," Thesis presented to the University of Missouri-Rolla, Rolla, MO, in partial fulfillment of the requirements for the Degree of Doctor of Philosophy, 1989.

Hsiao, L. E., W. W. Yu, and T. V. Galambos (1990), "AISI LRFD Method for Cold-Formed Steel Structural Members," *Journal of Structural Engineering,* ASCE, Vol. 116, No. 2, February 1990.

ICC-ES (2010), "Acceptance Criteria for Fasteners Power-Driven into Concrete, Steel and Masonry Elements (AC70)," Whittier, CA, 2010.

Jones, M. L., R. A. LaBoube and W. W. Yu (1997), "Spacing of Connections in Compression Elements for Cold-Formed Steel Members," Summary Report, Civil Engineering Study 97-6, University of Missouri-Rolla, MO, December 1997.

Johnston, B. G. (Editor) (1976), *Guide to Stability Design Criteria for Metal Structures,* Third Edition, John Wiley and Sons, New York, NY, 1976.

Joint Departments of the Army, Navy, Air Force, USA (1982), Chapter 13, *Seismic Design for Buildings*, TM 5-809-10/NAVFACP-355/AFM 88-3, Washington, DC, February 1986.

Kalyanaraman, V., T. Peköz and G. Winter (1977), "Unstiffened Compression Elements," *Journal of the Structural Division*, ASCE, Vol. 103, No. ST9, September 1977.

Kalyanaraman, V. and T. Peköz (1978), "Analytical Study of Unstiffened Elements," *Journal of the Structural Division*, ASCE, Vol. 104, No. ST9, September 1978.

Karren, K. W. (1967), "Corner Properties of Cold-Formed Steel Shapes," *Journal of the Structural Division,* ASCE, Vol. 93, No. ST1, February 1967.

Karren, K. W. and G. Winter (1967), "Effects of Cold Work on Light Gage Steel Members," *Journal of the Structural Division,* ASCE, Vol. 93, No. ST1, February 1967.

Kavanagh, K. T. and D. S. Ellifritt (1993), "Bracing of Cold-Formed Channels Not Attached to Deck or Sheeting," *Is Your Building Suitably Braced?,* Structural Stability Research Council, April 1993.

Kavanagh, K. T. and D. S. Ellifritt (1994), "Design Strength of Cold-Formed Channels in Bending and Torsion," *Journal of Structural Engineering*, ASCE, Vol. 120, No. 5, May 1994.

Kian, T. and T. B. Peköz (1994), "Evaluation of Industry-Type Bracing Details for Wall Stud Assemblies," Final Report, Submitted to American Iron and Steel Institute, Cornell University, January 1994.

Kirby, P. A. and D. A. Nethercot (1979), *Design for Structural Stability*, John Wiley and Sons, Inc., New York, NY, 1979.

Klippstein, K. H. (1980), "Fatigue Behavior of Sheet Steel Fabrication Details," *Proceedings of the Fifth International Specialty Conference on Cold-Formed Steel Structures*, University of Missouri-Rolla, Rolla, MO, November 1980.

Klippstein, K. H. (1981), "Fatigue Behavior of Steel-Sheet Fabrication Details," SAE Technical Paper Series 810436, International Congress and Exposition, Detroit, MI.

Klippstein, K. H. (1985), "Fatigue of Fabricated Steel-Sheet Details - Phase II," SAE Technical Paper Series 850366, International Congress and Exposition, Detroit, MI.

Klippstein, K. H. (1988), "Fatigue Design Curves for Structural Fabrication Details Made of Sheet and Plate Steel," Unpublished AISI research report.

Koka, E.N., W. W. Yu and R. A. LaBoube (1997), "Screw and Welded Connection Behavior Using Structural Grade 80 of A653 Steel (A Preliminary Study)," Fourth Progress Report, Civil Engineering Study 97-4, University of Missouri-Rolla, Rolla, MO, June 1997.

Kulak, G.L., and G.Y. Grondin, (2001), "AISC LRFD Rules for Block Shear in Bolted Connections – A Review," *Engineering Journal*, AISC, Fourth Quarter, 2001.

LaBoube, R.A., and W. W. Yu (1978), "Structural Behavior of Beam Webs Subjected to Bending Stress," *Civil Engineering Study Structural Series*, 78-1, Department of Civil Engineering, University of Missouri-Rolla, Rolla, MO, 1978.

LaBoube, R. A. and W. W. Yu (1978a), "Structural Behavior of Beam Webs Subjected Primarily to Shear Stress," Final Report, Civil Engineering Study 78-2, University of Missouri-Rolla, Rolla, MO, June 1978.

LaBoube, R. A. and W. W. Yu (1978b), "Structural Behavior of Beam Webs Subjected to a Combination of Bending and Shear," Final Report, Civil Engineering Study 78-3, University of Missouri-Rolla, Rolla, MO, June 1978.

LaBoube, R. A. and M. B. Thompson (1982a), "Static Load Tests of Braced Purlins Subjected to Uplift Load," Final Report, Midwest Research Institute, Kansas City, MO, 1982.

LaBoube, R. A. and W. W. Yu (1982b), "Bending Strength of Webs of Cold-Formed Steel Beams," *Journal of the Structural Division,* ASCE, Vol. 108, No. ST7, July 1982.

LaBoube, R. A. (1983), "Laterally Unsupported Purlins Subjected to Uplift," Final Report, Metal Building Manufacturers Association, 1983.

LaBoube, R. A. (1986), "Roof Panel to Purlin Connection: Rotational Restraint Factor," *Proceedings of the IABSE Colloquium on Thin-Walled Metal Structures in Buildings,* Stockholm, Sweden, 1986.

LaBoube, R. A., M. Golovin, D. J. Montague, D. C. Perry, and L. L. Wilson (1988), "Behavior of Continuous Span Purlin Systems," *Proceedings of the Ninth International Specialty Conference on Cold-Formed Steel Structures,* University of Missouri-Rolla, Rolla, MO, November 1988.

LaBoube, R. A. and M. Golovin (1990), "Uplift Behavior of Purlin Systems Having Discrete Braces," *Proceedings of the Tenth International Specialty Conference on Cold-Formed Steel Structures,* University of Missouri-Rolla, Rolla, MO, October 1990.

LaBoube, R. A. and W. W. Yu (1991), "Tensile Strength of Welded Connections," Final Report, Civil Engineering Study 91-3, University of Missouri-Rolla, Rolla, MO, June 1991.

LaBoube, R. A. and W. W. Yu (1993), "Behavior of Arc Spot Weld Connections in Tension," *Journal of Structural Engineering,* ASCE, Vol. 119, No. 7, July 1993.

LaBoube, R. A., J. N. Nunnery, and R. E. Hodges (1994), "Web Crippling Behavior of Nested Z-Purlins," *Engineering Structures* (G.J. Hancock, Guest Editor), Vol. 16, No. 5, Butterworth-Heinemann Ltd., London, July 1994.

LaBoube, R. A., and W. W. Yu (1995), "Tensile and Bearing Capacities of Bolted Connections," Final Summary Report, Civil Engineering Study 95-6, Cold-Formed Steel Series, Department of Civil Engineering, University of Missouri-Rolla, Rolla, MO, 1995.

LaBoube, R. A., and W. W Yu (1999), "Design of Cold-Formed Steel Structural Members and Connections for Cyclic Loading (Fatigue)," Final Report, Civil Engineering Study 99-1, Cold-Formed Steel Series, Department of Civil Engineering, University of Missouri-Rolla, Rolla, MO, 1999.

LaBoube, R.A. (2001a), "Tension on Arc Spot Welded Connections – AISI Section E2.2.2," University of Missouri-Rolla, Rolla, MO, 2001.

LaBoube, R.A. (2001b), "Arc Spot Welds in Sheet-to-Sheet Connections," Department of Civil Engineering, University of Missouri-Rolla, Rolla, MO, 2001.

LaBoube, R. A., R.M. Schuster, and J. Wallace (2002), "Web Crippling and Bending Interaction of Cold-Formed Steel Members," Final Report, University of Waterloo, Waterloo, Ontario, Canada, 2002.

Langan, J. E., R. A LaBoube, and W. W Yu (1994), "Structural Behavior of Perforated Web Elements of Cold-Formed Steel Flexural Members Subjected to Web Crippling and a Combination of Web Crippling and Bending," Final Report, Civil Engineering Series 94-3, Cold-Formed Steel Series, Department of Civil Engineering, University of Missouri-Rolla, Rolla, MO, 1994.

Lau, S. C. W. and G. J. Hancock (1987), "Distortional Buckling Formulas for Channel Columns," *Journal of Structural Engineering*, ASCE, Vol. 113, No. 5, May 1987.

Lease, A. R. and W. S. Easterling (2006a), "Insulation Impact on Shear Strength of Screw Connections and Shear Strength of Diaphragms," Report No. CE/VPI – 06/01, Virginia Polytechnic Institute and State University, Blacksburg, VA, 2006.

Lease, A. and W.S. Easterling (2006b), "The Influence of Insulation on the Shear Strength of Screw Connections," *Proceedings of the Eighteenth International Specialty Conference on Cold-Formed Steel Structures*, Orlando, FL, 2006.

Lee, S. and T. M. Murray (2001), "Experimental Determination of Required Lateral Restraint Forces for Z-Purlin Supported, Sloped Metal Roof Systems," CE/VPI-ST 01/09, Virginia Polytechnic Institute and State University, Blacksburg, VA, 2001.

Luttrell, L.D. (1999), "Metal Construction Association Diaphragm Test Program," West Virginia University, WV, 1999.

Luttrell, L.D. and K. Balaji (1992), "Properties of Cellular Decks in Negative Bending," *Proceedings of the Eleventh International Specialty Conference on Cold-Formed Steel Structures*, University of Missouri – Rolla, Rolla, MO, 1992.

Lutz, L. A. and J. M. Fisher (1985), "A Unified Approach for Stability Bracing Requirements," *Engineering Journal,* AISC, 4th Quarter, Vol. 22, No. 4, 1985.

Macadam, J. N., R. L. Brockenbrough, R. A. LaBoube, T. Peköz, and E. J. Schneider (1988), "Low-Strain-Hardening Ductile-Steel Cold-Formed Members," *Proceedings of the Ninth International Specialty Conference on Cold-Formed Steel Structures,* University of Missouri-Rolla, Rolla, MO, November 1988.

Metal Building Manufacturers Association (2002), *Metal Building Systems Manual,* Metal Building Manufacturers Association, Cleveland, OH, 2002.

Metal Construction Association (2004), *A Primer on Diaphragm Design*, Glenview, IL, 2004.

Midwest Research Institute (1981), "Determination of Rotational Restraint Factor 'F' for Panel to Purlin Connection Rigidity," Observer's Report, MRI Project No. 7105-G, Midwest Research Institute for Metal Building Manufacturers Association, 1981.

Miller, T. H. and T. Peköz (1989), "Studies on the Behavior of Cold-Formed Steel Wall Stud Assemblies," Final Report, Cornell University, Ithaca, NY, 1989.

Miller, T. H. and T. Peköz (1994), "Unstiffened Strip Approach for Perforated Wall Studs," *Journal of the Structural Engineering,* ASCE, Vol. 120, No. 2, February 1994.

Moreyra, M.E. (1993), "The Behavior of Cold-Formed Lipped Channels Under Bending," M.S. Thesis, Cornell University, Ithaca, NY, 1993.

Mujagic, J.R.U. (2008), "Effect of Washer Thickness on the Pull-Over Strength of Screw Connections Covered Under AISI S100-2007 Chapter E," Wei-Wen Yu Center for Cold-Formed Steel Structures, Rolla, MO, 2008.

Mujagic, J.R.U., P. S. Green, and W.G. Gould, (2010), "Strength Prediction Model for Power Actuated Fasteners Connecting Steel Members in Tension and Shear – North American Applications," Wei-Wen Yu Center for Cold-Formed Steel Structures, Missouri University of Science and Technology, Rolla, MO, 2010.

Mulligan, G. P. and T. B. Peköz (1984), "Locally Buckled Thin-Walled Columns," *Journal of the Structural Division*, ASCE, Vol. 110, No. ST11, November 1984.

Murray, T. M. and S. Elhouar (1985), "Stability Requirements of Z-Purlin Supported Conventional Metal Building Roof Systems," *Annual Technical Session Proceedings,* Structural Stability Research Council, 1985.

Murray, T. M. (1991), "Building Floor Vibrations," *Engineering Journal,* AISC, Third Quarter, 1991.

Nguyen, P. and W. W. Yu (1978a), "Structural Behavior of Transversely Reinforced Beam Webs," Final Report, Civil Engineering Study 78-5, University of Missouri-Rolla, Rolla, MO, July 1978.

Nguyen, P. and W. W. Yu, (1978b), "Structural Behavior of Longitudinally Reinforced Beam Webs," Final Report, Civil Engineering Study 78-6, University of Missouri-Rolla, Rolla, MO, July 1978.

Ortiz-Colberg, R. and T. B. Peköz (1981), "Load Carrying Capacity of Perforated Cold-Formed Steel Columns," Research Report No. 81-12, Cornell University, Ithaca, NY, 1981.

Pan, L.C. and W. W. Yu (1988), "High Strength Steel Members With Unstiffened Compression Elements," *Proceedings of the Ninth International Specialty Conference on Cold-Formed Steel Structures*, University of Missouri-Rolla, Rolla, MO, November 1988.

Papazian, R.P., R.M. Schuster and M. Sommerstein (1994), "Multiple Stiffened Deck Profiles," *Proceedings of the Twelfth International Specialty Conference on Cold-Formed Steel Structures,* University of Missouri-Rolla, Rolla, MO, October 1994.

Peköz, T. B. and G. Winter (1969a), "Torsional-Flexural Buckling of Thin-Walled Sections Under Eccentric Load," *Journal of the Structural Division,* ASCE, Vol. 95, No. ST5, May 1969.

Peköz, T. B. and N. Celebi (1969b), "Torsional-Flexural Buckling of Thin-Walled Sections Under Eccentric Load," *Engineering Research Bulletin* 69-1, Cornell University, 1969.

Peköz, T. B. and W. McGuire (1979), "Welding of Sheet Steel," Report SG-79-2, American Iron and Steel Institute, January 1979.

Peköz, T. B. and P. Soroushian (1981), "Behavior of C- and Z-Purlins Under Uplift," *Report* No. 81-2, Cornell University, Ithaca, NY, 1981.

Peköz, T. B. and P. Soroushian (1982), "Behavior of C- and Z-Purlins Under Wind Uplift," *Proceedings of the Sixth International Specialty Conference on Cold-Formed Steel Structures,* University of Missouri-Rolla, Rolla, MO, November 1982.

Peköz, T. B. (1986a), "Combined Axial Load and Bending in Cold-Formed Steel Members," *Thin-Walled Metal Structures in Buildings,* IABSE Colloquium, Stockholm, Sweden, 1986.

Peköz, T. B. (1986b), "Development of a Unified Approach to the Design of Cold-Formed Steel Members," Report SG-86-4, American Iron and Steel Institute, 1986.

Peköz, T. B. (1986c), "Developments of a Unified Approach to the Design of Cold-Formed Steel Members," *Proceedings of the Eighth International Specialty Conference on Cold-Formed Steel Structures*, University of Missouri-Rolla, Rolla, MO, November 1986.

Peköz, T. B. (1988a), "Design of Cold-Formed Steel Columns," *Proceedings of the Ninth International Specialty Conference on Cold-Formed Steel Structures,* University of Missouri-Rolla, Rolla, MO, November 1988.

Peköz, T. B. and W. B. Hall (1988b), "Probabilistic Evaluation of Test Results," *Proceedings of the Ninth International Specialty Conference on Cold-Formed Steel Structures,* University of Missouri-Rolla, Rolla, MO, November 1988.

Peköz, T. B. (1990), "Design of Cold-Formed Steel Screw Connections," *Proceedings of the Tenth International Specialty Conference on Cold-Formed Steel Structures,* University of Missouri-Rolla, Rolla, MO, October 1990.

Peköz, T. B. and O. Sumer (1992), "Design Provisions for Cold-Formed Steel Columns and Beam-Columns," Final Report, Submitted to American Iron and Steel Institute, Cornell University, September 1992.

Pham, CH and G.J. Hancock (2009), "Direct Strength Design of Cold-Formed Purlins," *Journal of Structural Engineering*, ASCE, Vol 135, No. 3, pp. 229 – 238, 2009.

Popovic, D., G.J. Hancock, and K.J.R. Rasmussen (1999), "Axial Compression Tests of Cold-Formed Angles," *Journal of Structural Engineering*, ASCE, Vol. 125, No.5, May 1999.

Prabakaran, K. (1993), "Web Crippling of Cold-Formed Steel Sections," M.S. Thesis, University of Waterloo, Waterloo, Canada, 1993.

Prabakaran, K. and R.M. Schuster (1998), "Web Crippling of Cold-Formed Steel Members," *Proceedings of the Fourteenth International Specialty Conference on Cold-Formed Steel Structures,* University of Missouri-Rolla, Rolla, MO, October, 1998.

Put, B.M., Y.L. Pi and N.S. Trahair (1999), "Bending and Torsion of Cold-Formed Channel Beams," *Journal of the Structural Engineering*, ASCE, Vol. 125, No. 5, May 1999.

Rack Manufacturers Institute (1997), *Specification for the Design, Testing, and Utilization of Industrial Steel Storage Racks,* Charlotte, NC, 1997.

Rang, T. N., T. V. Galambos, W. W. Yu, and M. K. Ravindra (1978), "Load and Resistance Factor Design of Cold-Formed Steel Structural Members," *Proceedings of the Fourth International Specialty Conference on Cold-Formed Steel Structures*, University of Missouri-Rolla, Rolla, MO, June 1978.

Rang, T. N., T. V. Galambos, and W. W. Yu (1979a), "Load and Resistance Factor Design of Cold-Formed Steel: Study of Design Formats and Safety Index Combined With Calibration of the AISI Formulas for Cold Work and Effective Design Width," First Progress Report, Civil Engineering Study 79-1, University of Missouri-Rolla, Rolla, MO, January 1979.

Rang, T. N., T. V. Galambos and W. W. Yu (1979b), "Load and Resistance Factor Design of Cold-Formed Steel: Statistical Analysis of Mechanical Properties and Thickness of Material Combined With Calibration of the AISI Design Provisions on Unstiffened Compression Elements and Connections," Second Progress Report, Civil Engineering Study 79-2, University of Missouri-Rolla, Rolla, MO, January 1979.

Rang, T. N., T. V. Galambos and W. W. Yu (1979c), "Load and Resistance Factor Design of Cold-Formed Steel: Calibration of the Design Provisions on Connections and Axially Loaded Compression Members," Third Progress Report, Civil Engineering Study 79-3, University of Missouri-Rolla, Rolla, MO, January 1979.

Rang, T. N., T. V. Galambos and W. W. Yu (1979d), "Load and Resistance Factor Design of Cold-Formed Steel: Calibration of the Design Provisions on Laterally Unbraced Beams and Beam-Columns," Fourth Progress Report, Civil Engineering Study 79-4, University of Missouri-Rolla, Rolla, MO, January 1979.

Rasmussen, K. J. R. and G. J. Hancock (1992), "Nonlinear Analyses of Thin-Walled Channel Section Columns," *Thin Walled Structures* (J. Rhodes and K.P. Chong, Eds.), Vol. 13, Nos. 1-2, Elsevier Applied Science, Tarrytown, NY, 1992.

Rasmussen, K. J. R. (1994), "Design of Thin-Walled Columns With Unstiffened Flanges," *Engineering Structures* (G. J. Hancock, Guest Editor), Vol. 16, No. 5, Butterworth-Heinmann Ltd., London, July 1994.

Ravindra, M. K. and T. V. Galambos (1978), "Load and Resistance Factor Design for Steel," *Journal of the Structural Division*, ASCE, Vol. 104, No. ST9, September 1978.

Reck, H. P., T. Peköz and G. Winter (1975), "Inelastic Strength of Cold-Formed Steel Beams," *Journal of the Structural Division,* ASCE, Vol. 101, No. ST11, November 1975.

Research Council on Structural Connections (1980), *Specification for Structural Joints Using ASTM A325 or A490 Bolts*, 1980.

Research Council on Structural Connections (1985), *Allowable Stress Design Specification for Structural Joints Using ASTM A325 or A490 Bolts,* 1985.

Research Council on Structural Connections (2000), *Specification for Structural Joints Using ASTM A325 or A490 Bolts*, 2000.

Research Council on Structural Connections (2004), *Specification for Structural Joints Using ASTM A325 or A490 Bolts*, 2004.

Rivard, P. and T.M. Murray (1986), "Anchorage Forces in Two Purlin Line Standing Seam Z-Purlin Supported Roof Systems," Research Report, University of Oklahoma, Norman, OK, December 1986.

Roark, R. J. (1965), *Formulas for Stress and Strain,* Fourth Edition, McGraw-Hill Book Company, New York, NY, 1965.

Rogers, C.A., and R. M. Schuster (1995), "Interaction Buckling of Flange, Edge Stiffener and Web of C-Sections in Bending," *Research Into Cold Formed Steel, Final Report of CSSBI/IRAP Project*, Department of Civil Engineering, University of Waterloo, Waterloo, Ontario, 1995.

Rogers, C. and R.M. Schuster (1996), "Cold-Formed Steel Flat Width Ratio Limits, d/t, and d_i/w," *Proceedings of the Thirteenth International Specialty Conference on Cold-Formed Steel Structures*, University of Missouri-Rolla, Rolla, MO, October 1996.

Rogers, C.A. and G.J. Hancock, (1997), "Ductility of G550 Sheet Steels in Tension," *Journal of Structural Engineering*, Vol. 123 (12), pp. 1586-1594, 1997.

Rogers, C. A. and G. J. Hancock (1998), "Bolted Connection Tests of Thin G550 and G300 Sheet Steels," *Journal of Structural Engineering*, ASCE, Vol. 124, No. 7, pp. 798-808, 1998.

Rogers, C.A. and G.J. Hancock (1999a), "Screwed Connection Tests of Thin G550 and G300 Sheet Steels," *Journal of Structural Engineering*, ASCE, Vol. 125, No. 2, pp. 128-136, 1999.

Rogers, C.A. and G.J. Hancock (1999b), "Bolted Connection Design for Sheet Steels Less Than 1.0 mm Thick," *Journal of Constructional Steel Research*, Vol. 51, No. 2, 1999, pp. 123-146, 1999.

Rogers, C.A. and G.J. Hancock (2000), "Fracture Toughness of G550 Sheet Steels Subjected to Tension," *Journal of Constructional Steel Research*, Vol. 57, No. 1, pp. 71-89, 2000.

Salmon, C. G. and J.E. Johnson (1990), *Steel Structures: Design and Behavior,* Third Edition, Harper & Row, New York, NY, 1990.

Santaputra, C. (1986), "Web Crippling of High Strength Cold-Formed Steel Beams," Ph.D. Thesis, University of Missouri-Rolla, Rolla, MO, 1986.

Santaputra, C., M. B. Parks, and W. W. Yu (1989), "Web Crippling Strength of Cold-Formed Steel Beams," *Journal of Structural Engineering,* ASCE, Vol. 115, No. 10, October 1989.

Sarawit, A. and T. Peköz (2006), "Notional Load Method for Industrial Steel Storage Racks," *Thin-Walled Structures*, Elserier, Vol. 44, No. 12, December 2006.

S. B. Barnes Associates (2012), "Top Arc Seam Welds (Arc Seam Weld on Standing Seam Hem) Shear Strength [Resistance] and Flexibility for Sheet-to-Sheet Connections," Report No. 11-01 by R. Nunna and C. W. Pinkham, Wei-Wen Yu Center for Cold-Formed Steel Structures Library, Los Angeles, CA, 2012.

Schafer, B.W. and T. Peköz (1998), "Cold-Formed Steel Members With Multiple Longitudinal Intermediate Stiffeners in the Compression Flange," *Journal of Structural Engineering*, ASCE, Vol. 124, No. 10, October 1998.

Schafer, B.W. and T. Peköz (1999), "Laterally Braced Cold-Formed Steel Flexural Members With Edge Stiffened Flanges," *Journal of Structural Engineering*, ASCE, Vol. 125, No. 2, February 1999.

Schafer, B.W. (2000), "Distortional Buckling of Cold-Formed Steel Columns," Final Report, Sponsored by the American Iron and Steel Institute, Washington, DC, 2000.

Schafer, B.W. (2002), "Local, Distortional, and Euler Buckling in Thin-Walled Columns," *Journal of Structural Engineering,* ASCE, Vol. 128, No. 3, March 2002.

Schafer, B. W. (2008), "Review: The Direct Strength Method of Cold-Formed Steel Member Design," *Journal of Constructional Steel Research*, 64 (7/8), pp. 766-778, 2008.

Schafer, B.W. (2009), "Improvement to AISI Section B5.1.1 for Effective Width of Elements With Intermediate Stiffeners," CCFSS Technical Bulletin, February 2009.

Schafer, B.W., R.H. Sangree Y. Guan (2007), "Experiments on Rotational Restraint of Sheathing," Final Report, American Iron and Steel Institute – Committee on Framing Standards, July 2007.

Schafer, B.W., R.H. Sangree and Y. Guan (2008), "Floor System Design for Distortional Buckling Including Sheathing Restraint," *Proceedings of the Nineteenth International Specialty Conference on Cold-Formed Steel Structures*, St Louis, MO, October 14-15, 2008.

Schafer, B.W., A. Sarawit, and T. Peköz (2006), "Complex Edge Stiffeners for Thin-Walled Members," *Journal of Structural Engineering*, ASCE, Vol. 132, No. 2, February 2006.

Schafer, B.W. and T. Trestain (2002), "Interim Design Rules for Flexure in Cold-Formed Steel Webs," *Proceedings of the Sixteenth International Specialty Conference on Cold-Formed Steel Structures*, Orlando, FL, 2002.

Schardt, R. W. and Schrade (1982), "Kaltprofil-Pfetten," Institut Für Statik, Technische Hochschule Darmstadt, Bericht Nr. 1, Darmstadt, 1982.

Schuster, R.M. (1992), "Testing of Perforated C-Stud Sections in Bending," Report for the Canadian Sheet Steel Building Institute, University of Waterloo, Waterloo, Ontario, 1992.

Schuster, R. M., C. A. Rogers, and A. Celli (1995), "Research Into Cold-Formed Steel Perforated C-Sections in Shear," Progress Report No. 1 of Phase I of CSSBI/IRAP Project, Department of Civil Engineering, University of Waterloo, Waterloo, Ontario, Canada, 1995.

Sears, J. M. and T. M. Murray (2007), "Proposed Method for the Prediction of Lateral Restraint Forces in Metal Building Roof Systems," *Annual Stability Conference Proceedings*, Structural Stability Research Council, 2007.

Seek, M. W. and T. M. Murray (2004), "Computer Modeling of Sloped Z-Purlin Supported Roof Systems to Predict Lateral Restraint Force Requirements," *Proceedings of the Seventeenth International Specialty Conference on Cold-Formed Steel Structures,* Department of Civil Engineering, University of Missouri-Rolla, Rolla, MO, 2004.

Seek, M. W. and T. M. Murray (2006), "Component Stiffness Method to Predict Lateral Restraint Forces in End Restrained Single Span Z-Section Supported Roof Systems With One Flange Attached to Sheathing," *Proceedings of the Nineteenth International Specialty Conference on Cold-Formed Steel Structures,* University of Missouri-Rolla, Rolla, MO, 2006.

Seek, M.W. and T.M. Murray (2007), "Lateral Brace Forces in Single Span Z-Section Roof Systems with Interior Restraints Using the Component Stiffness Method," *Annual Stability Conference Proceedings*, Structural Stability Research Council, 2007.

Serrette, R. L. and T. B. Peköz (1992), "Local and Distortional Buckling of Thin-Walled Beams," *Proceedings of the Eleventh International Specialty Conference on Cold-Formed Steel Structures*, University of Missouri-Rolla, Rolla, MO, October 1992.

Serrette, R. L. and T. B. Peköz (1994), "Flexural Capacity of Continuous Span Standing Seam Panels: Gravity Load," *Proceedings of the Twelfth International Specialty Conference on Cold-Formed Steel Structures*, University of Missouri-Rolla, Rolla, MO, October 1994.

Serrette, R. L. and T. B. Peköz (1995), "Behavior of Standing Seam Panels," *Proceedings of the Third International Conference on Steel and Aluminum Structures,* Bogazici University, Istanbul, Turkey, May 1995.

Shadravan, S. and C. Ramseyer (2007), "Bending Capacity of Steel Purlins With Torsional Bracing Using the Base Test," *Annual Stability Conference Proceedings*, Structural Stability Research Council, 2007.

Shan, M. Y., R. A. LaBoube, and W. W. Yu (1994), "Behavior of Web Elements With Openings Subjected to Bending, Shear and the Combination of Bending and Shear," Final Report, Civil Engineering Series 94-2, Cold-Formed Steel Series, Department of Civil Engineering, University of Missouri-Rolla, MO, 1994.

Sherman, D. R. (1976), "Tentative Criteria for Structural Applications of Steel Tubing and Pipe," American Iron and Steel Institute, Washington, DC, 1976.

Sherman, D. R. (1985), "Bending Equations for Circular Tubes," *Annual Technical Session Proceedings,* Structural Stability Research Council, 1985.

Simaan, A. (1973), "Buckling of Diaphragm-Braced Columns of Unsymmetrical Sections and Applications to Wall Studs Design," Report No. 353, Cornell University, Ithaca, NY, 1973.

Simaan, A. and T. Peköz (1976), "Diaphragm-Braced Members and Design of Wall Studs," *Journal of the Structural Division,* ASCE, Vol. 102, ST1, January 1976.

Snow, G. L. and Easterling, W. S. (2008), "Section Properties for Cellular Decks Subjected to Negative Bending," Report No. CE/VPI – 08/06, Virginia Polytechnic Institute and State University, Blacksburg, VA.

Sputo, T., and K. Beery (2006), "Accumulation of Bracing Strength and Stiffness Demand in Cold-Formed Steel Stud Walls," *Proceedings of the Eighteenth International Specialty Conference on Cold-Formed Steel Structures*, University of Missouri-Rolla, Rolla, MO, 2006.

Standards Australia (2001), "Steel Sheet and Strip - Hot-Dipped Zinc-Coated or Aluminium/Zinc Coated -AS 1397-2001," Sydney, Australia, 2006.

Stauffer, T. M. and P. B. McEntee (2012), "Use of Mill Certificates to Establish Material Properties in Testing of Cold-Formed Steel Components," Report published by Center for Cold-Formed Steel Structures, Missouri University of Science and Technology, Rolla, MO, 2012.

Steel Deck Institute, Inc. (1981), *Steel Deck Institute Diaphragm Design Manual,* First Edition, Canton, OH, 1981.

Steel Deck Institute, Inc. (1987), *Steel Deck Institute Diaphragm Design Manual*, Canton, OH, 1987.

Steel Deck Institute, Inc. (2004), *Steel Deck Institute Diaphragm Design Manual,* Third Edition, Fox River Grove, IL, 2004.

Steel Deck Institute, Inc. (2007), *Design Manual for Composite Decks, Form Decks, Roof Decks, and Cellular Deck Floor Systems With Electrical Distribution*, SDI Publication No. 31, 2007.

Steel Deck Institute, Inc. (2011), ANSI/SDI C-2011, *Standard for Composite Steel Floor Deck-Slabs*, 2011.

Stirnemann, L.K. and R. A. LaBoube (2007), "Behavior of Arc Spot Weld Connections Subjected to Combined Shear and Tension Forces," Research Report, University of Missouri-Rolla, Rolla, MO, 2007.

Stolarczyk, J. A., J. M. Fisher and A. Ghorbanpoor (2002), "Axial Strength of Purlins Attached to Standing Seam Roof Panels," *Proceedings of the Sixteenth International Specialty Conference on Cold-Formed Steel Structures*, University of Missouri-Rolla, Rolla, MO, October 2002.

Structural Stability Research Council (1993), *Is Your Structure Suitably Braced?*, Lehigh University, Bethlehem, PA, April 1993.

Supornsilaphachai, B., T. V. Galambos and W. W. Yu (1979), "Load and Resistance Factor Design of Cold-Formed Steel: Calibration of the Design Provisions on Beam Webs," Fifth Progress Report, Civil Engineering Study 79-5, University of Missouri-Rolla, Rolla, MO, September 1979.

Supornsilaphachai, B. (1980), "Load and Resistance Factor Design of Cold-Formed Steel Structural Members," Thesis presented to the University of Missouri-Rolla, MO, in partial fulfillment of the requirements for the Degree of Doctor of Philosophy, 1980.

Surry, D., R. R. Sinno, B. Nail, T.C.E. Ho, S. Farquhar and G. A. Kopp (2007), "Structurally-Effective Static Wind Loads for Roof Panels," *Journal of Structural Engineering*, ASCE, Vol. 133, No. 6, June 2007.

Tangorra, F. M., R. M. Schuster and R. A. LaBoube (2001), "Calibrations of Cold Formed Steel Welded Connections," Research Report, University of Waterloo, Waterloo, Ontario, Canada, 2001.

Teh, L.H. and G.J. Hancock (2000), "Strength of Fillet Welded Connections in G450 Sheet Steels," Research Report R802, Centre for Advanced Structural Engineering, University of Sydney, July 2000.

Tsai, M. (1992), "Reliability Models of Load Testing," Ph.D. Dissertation, Dept. of Aeronautical and Astronautical Engineering, University of Illinois at Urbana-Champaign, 1992.

United States Army Corps of Engineers (1991), *Guide Specification for Military Construction, Standing Seam Metal Roof Systems*, October 1991.

Uphoff, C. A. (1996), "Structural Behavior of Circular Holes in Web Elements of Cold-Formed Steel Flexural Members Subjected to Web Crippling for End-One-Flange Loading," Thesis presented to the faculty of the University of Missouri-Rolla in partial fulfillment for the degree Master of Science, 1996.

von Karman, T., E. E. Sechler, and L.H. Donnell (1932), "The Strength of Thin Plates in Compression," *Transactions*, ASME, Vol. 54, 1932.

Wallace, A.W. (2003), "Web Crippling of Cold-Formed Steel Multi-Web Deck Sections Subjected to End One-flange Loading," Final Report, Canadian Cold Formed Steel Research Group, Department of Civil Engineering, University of Waterloo, Waterloo, Ontario, Canada, May 2003.

Wallace, J. A. and R.M. Schuster (2004), "Web Crippling of Cold Formed Steel Multi - Web Deck Sections Subjected to End One-Flange Loading," *Proceedings of the Seventeenth International Specialty Conference on Cold-Formed Steel Structures*, University of Missouri-Rolla, Rolla, MO, October 2004, Page 171-185.

Wallace, J. A., R.M. Schuster, and R.A. LaBoube (2001a), "Testing of Bolted Cold-Formed Steel Connections in Bearing", University of Waterloo, Waterloo, Canada, 2001.

Wallace, J. A., R.A. LaBoube and R.M. Schuster (2001b), "Calibration of Bolted Cold-Formed Steel Connections in Bearing (With and Without Washers)," University of Waterloo, Waterloo, Canada, 2001.

Weng, C. C. and T. B. Peköz (1986), "Subultimate Behavior of Uniformly Compressed Stiffened Plate Elements," Research Report, Cornell University, Ithaca, NY, 1986.

Wibbenmeyer, K. (2009), "Determining the R Values for 12 Inch Deep Z-Purlins and Girts With Through-Fastened Panels Under Suction Load," Thesis presented to the Missouri University of Science and Technology in partial fulfillment of the requirements for the degree Master of Science in Civil Engineering, Rolla, MO, 2010.

Willis, C.T. and B. Wallace (1990), "Behavior of Cold-Formed Steel Purlins Under Gravity Loading," *Journal of Structural Engineering*, ASCE, 116 No. 8, 1990.

Wing, B.A. (1981), "Web Crippling and the Interaction of Bending and Web Crippling of Unreinforced Multi-Web Cold-Formed Steel Sections," M.A.Sc. Thesis, University of Waterloo, Waterloo, Canada, 1981.

Wing, B.A. and R.M. Schuster (1982), "Web Crippling of Decks Subjected to Two-Flange Loading," *Proceedings of the Sixth International Specialty Conference on Cold-Formed Steel Structures*, University of Missouri-Rolla, Rolla, MO, November 1982.

Winter, G. (1940), "Stress Distribution in and Equivalent Width of Flanges of Wide, Thin-Walled Steel Beams," Technical Note No. 784, National Advisory Committee for Aeronautics, Washington, DC, 1940.

Winter, G. (1943), "Lateral Stability of Unsymmetrical I-Beams and Trusses," *Transactions,* ASCE, Vol. 198, 1943.

Winter, G. (1944), "Strength of Slender Beams," *Transactions,* ASCE, Vol. 109, 1944.

Winter, G. and R. H. J. Pian (1946), "Crushing Strength of Thin Steel Webs," Cornell *Bulletin* 35, pt. 1, April 1946.

Winter, G. (1947a), Discussion of "Strength of Beams as Determined by Lateral Buckling," by Karl deVries, *Transactions,* ASCE, Vol. 112, 1947.

Winter, G. (1947b), "Strength of Thin Steel Compression Flanges," (with Appendix), *Bulletin* No. 35/3, Cornell University, Ithaca, NY, 1947.

Winter, G. (1947c), "Strength of Thin Steel Compression Flanges," *Transactions,* ASCE, Vol. 112, 1947.

Winter, G., P. T. Hsu, B. Koo and M. H. Loh (1948a), "Buckling of Trusses and Rigid Frames," *Bulletin* No. 36, Cornell University, Ithaca, NY, 1948.

Winter, G. (1948b), "Performance of Thin Steel Compression Flanges," *Preliminary Publication,* The Third Congress of the International Association of Bridge and Structural Engineers, Liege, Belgium, 1948.

Winter, G. (1949a), "Performance of Compression Plates as Parts of Structural Members," *Research, Engineering Structures Supplement* (Colston Papers, Vol. II), 1949.

Winter, G., W. Lansing, and R. B. McCalley, Jr. (1949b), "Performance of Laterally Loaded Channel Beams," *Research, Engineering Structures Supplement*, (Colston Papers, Vol. II), 1949.

Winter, G., W. Lansing and R. McCalley (1950), *Performance of Laterally Loaded Channel Beams,* Four papers on the performance of Thin Walled Steel Structures, Cornell University, Engineering Experiment Station, Reprint No. 33, November 1, 1950.

Winter, G. (1956a), "Light Gage Steel Connections With High-Strength, High-Torqued Bolts," *Publications,* IABSE, Vol. 16, 1956.

Winter, G. (1956b), "Tests on Bolted Connections in Light Gage Steel," *Journal of the Structural Division,* ASCE, Vol. 82, No. ST2, February 1956.

Winter, G. (1958a), "Lateral Bracing of Columns and Beams," *Journal of the Structural Division,* ASCE, Vol. 84, No. ST2, March 1958.

Winter, G. (1958b), *Commentary on the 1956 Edition of the Light Gage Cold-Formed Steel Design Manual,* American Iron and Steel Institute, New York, NY, 1958.

Winter, G. (1959a), "Development of Cold-Formed, Light Gage Steel Structures," AISI Regional Technical Papers, October 1, 1959.

Winter, G. (1959b), "Cold-Formed, Light Gage Steel Construction," *Journal of the Structural Division,* ASCE, Vol. 85, No. ST9, November 1959.

Winter, G. (1960), "Lateral Bracing of Columns and Beams," *Transactions,* ASCE, Vol. 125, 1960.

Winter, G. and J. Uribe (1968), "Effects of Cold-Work on Cold-Formed Steel Members," *Thin-Walled Steel Structures - Their Design and Use in Buildings,* K. C. Rockey and H. V. Hill (Eds.), Gordon and Breach Science Publishers, United Kingdom, 1968.

Winter, G. (1970), *Commentary on the 1968 Edition of the Specification for the Design of Cold-Formed Steel Structural Members,* American Iron and Steel Institute, New York, NY, 1970.

Yang, D., G.J. Hancock and K.J.R. Rasmussen, (2004), "Compression Tests of Cold-Reduced High Strength Steel Long Columns," *Journal of Structural Engineering*, Vol. 130, No. 1, pp. 1782–1789, 2004.

Yang, D. and G.J. Hancock, (2004a), "Compression Tests of Cold-Reduced High Strength Steel Stub Columns," *Journal of Structural Engineering*, Vol. 130, No. 11, pp. 1772–1781, 2004.

Yang, D. and G.J. Hancock, (2004b), "Compression Tests of Cold-Reduced High Strength Steel Channel Columns," *Journal of Structural Engineering*, Vol. 130, No. 12, pp. 1954–1963, 2004.

Yu, C., and B.W. Schafer (2003), "Local Buckling Tests on Cold-Formed Steel Beams," ASCE, *Journal of Structural Engineering,* 129 (12) pp. 1596-1606, 2003.

Wu, S., W. W. Yu and R. A. LaBoube (1996), "Strength of Flexural Members Using Structural Grade 80 of A653 Steel (Deck Panel Tests)," Second Progress Report, Civil Engineering Study 96-4, University of Missouri-Rolla, Rolla, MO, November 1996.

Wu, S., W. W. Yu and R. A. LaBoube (1997), "Strength of Flexural Members Using Structural Grade 80 of A653 Steel (Web Crippling Tests)," Third Progress Report, Civil Engineering Study 97-3, University of Missouri-Rolla, Rolla, MO, February 1997.

Yang, D and G.J. Hancock (2002), "Compression Tests of Cold-Reduced High Strength Steel Stub Columns," Research Report R815, Center for Advanced Structural Engineering, Department of Civil Engineering, University of Sydney, Australia, March 2002.

Yang, D, G.J. Hancock and Rasmussen (2002), "Compression Tests of Cold-Reduced High Strength Steel Long Columns," Research Report R816, Center for Advanced Structural Engineering, Department of Civil Engineering, University of Sydney, Australia, March 2002.

Yang, D. and G.J. Hancock (2003), "Compression Tests of Cold-Reduced High Strength Steel Channel Columns Failing in the Distortional Mode," Research Report R825, Department of Civil Engineering, University of Sydney, Australia, 2003.

Yang, H. and B.W. Schafer (2006), "Comparison of AISI Specification Methods for Members With Single Intermediate Longitudinal Stiffeners," Report to American Iron and Steel Institute, Washington, DC, 2006.

Yener, M. and T. B. Peköz (1985a), "Partial Stress Redistribution in Cold-Formed Steel," *Journal of Structural Engineering*, ASCE, Vol. 111, No. 6, June 1985.

Yener, M. and T. B. Peköz (1985b), "Partial Moment Redistribution in Cold-Formed Steel," *Journal of Structural Engineering*, ASCE, Vol. 111, No. 6, June 1985.

Yiu, F. and T. Peköz (2001), "Design of Cold-Formed Steel Plain Channels," Cornell University, Ithaca, NY, 2001.

Young, B. and G.J. Hancock (1998), "Web Crippling Behaviour of Cold-Formed Unlipped Channels," *Proceedings of the Fourteenth International Specialty Conference on Cold-Formed Steel Structures*, University of Missouri-Rolla, Rolla, MO, October 1998.

Young, B. and G.J. Hancock (2000), "Experimental Investigation of Cold-Formed Channels Subjected to Combined Bending and Web Crippling," *Proceedings of the Fifteenth International Specialty Conference on Cold-Formed Steel Structures*, University of Missouri-Rolla, Rolla, MO, October 2000.

Yu, C. (2005), "Distortional Buckling of Cold-Formed Steel Members in Bending," Ph.D. Thesis, Johns Hopkins University, Baltimore, MD, 2005.

Yu, C. (2009), "Web Crippling Strength of Cold-Formed Steel NUFRAME Members," Report No. 20090112-01, University of North Texas, Denton, TX, 2009.

Yu, C. (2009a), "Web Crippling Strength of 43 Mil Cold-Formed Steel NUFRAME Members," Report No. 20090217-01, University of North Texas, Denton, TX, 2009.

Yu, C. and B.W. Schafer (2003), "Local Buckling Tests on Cold-Formed Steel Beams," *Journal of Structural Engineering*, ASCE, Vol. 129, No. 12, December 2003.

Yu, C. and B.W. Schafer (2006), "Distortional Buckling Tests on Cold-Formed Steel Beams," *Journal of Structural Engineering*, ASCE, Vol. 132, No. 4, April 2006.

Yu, C. and K. Xu, (2010), "Cold-Formed Steel Bolted Connections Using Washers on Oversized and Slotted Holes – Phase 2," Research Report RP10-2, American Iron and Steel Institute, Washington, DC, 1020.

Yu, W. W. and C. S. Davis (1973a), "Cold-Formed Steel Members With Perforated Elements," *Journal of the Structural Division,* ASCE, Vol. 99, No. ST10, October 1973.

Yu, W. W., V. A. Liu, and W. M. McKinney (1973b), "Structural Behavior and Design of Thick, Cold-Formed Steel Members," *Proceedings of the Second Specialty Conference on Cold-Formed Steel Structures,* University of Missouri-Rolla, Rolla, MO, October 1973.

Yu, W. W., V. A. Liu, and W. M. McKinney (1974), "Structural Behavior of Thick Cold-Formed Steel Members," *Journal of the Structural Division,* ASCE, Vol. 100, No. ST1, January 1974.

Yu, W. W. (1981), "Web Crippling and Combined Web Crippling and Bending of Steel Decks," Civil Engineering Study 81-2, University of Missouri-Rolla, Rolla, MO, April 1981.

Yu, W. W. (1982), "AISI Design Criteria for Bolted Connections," *Proceedings of the Sixth International Specialty Conference on Cold-Formed Steel Structures,* University of Missouri-Rolla, Rolla, MO, November 1982.

Yu, W. W. (1985), *Cold-Formed Steel Design,* Wiley-Interscience, New York, NY, 1985.

Yu, W.W. (1996), *Commentary on the 1996 Edition of the Specification for the Design of Cold-Formed Steel Structural Members*, American Iron and Steel Institute, Washington, DC, 1996.

Yu, W. W. and R. A. LaBoube (2010), *Cold-Formed Steel Design,* Fourth Edition, John Wiley & Sons, New York, NY, 2010.

Yura, J.A. (1993), "Fundamentals of Beam Bracing," *Is Your Structure Suitably Braced?*, Structural Stability Research Council, April 1993.

Zetlin, L. (1955a), "Elastic Instability of Flat Plates Subjected to Partial Edge Loads," *Journal of the Structural Division*, ASCE, Vol. 81, September 1955.

Zetlin, L. and G. Winter (1955b), "Unsymmetrical Bending of Beams With and Without Lateral Bracing," *Journal of the Structural Division*, ASCE, Vol. 81, 1955.

Zhao, X.L. and G.J. Hancock (1995), "Butt Welds and Transverse Fillet Welds in Thin Cold-Formed RHS Members," *Journal of Structural Engineering*, ASCE, Vol. 121, No. 11, November 1995.

Zeinoddini, V. and B. W. Schafer (2010), "Impact of Cornier Radius on Cold-Formed Steel Member Strength,", *Proceedings of the Twentieth International Specialty Conference on Cold-Formed Steel Structures*, Missouri University of Science and Technology, Rolla, MO, pp. 1-15, November, 2010.

Zwick, K. and R. A. LaBoube (2002), "Self-Drilling Screw Connections Subject to Combined Shear and Tension," Center for Cold-Formed Steel Structures, University of Missouri-Rolla, Rolla, MO, 2002.

Appendix 1:

Commentary on Appendix 1

Design of Cold-Formed Steel

Structural Members

Using the Direct Strength Method

2012 EDITION

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APPENDIX 1: COMMENTARY ON APPENDIX 1–DESIGN OF COLD-FORMED STEEL STRUCTURAL MEMBERS USING THE DIRECT STRENGTH METHOD

1.1 GENERAL PROVISIONS

1.1.1 Applicability

The Direct Strength Method (DSM) of Appendix 1 is an alternative procedure for determining the strength and stiffness of cold-formed steel members (beams and columns). The reliability of Appendix 1 is ensured by using calibrated *safety factor*, Ω, and *resistance factor*, ϕ, within set geometric limits, and conservative Ω and ϕ for other configurations. The applicability of Appendix 1 to all beams and columns implies that in some situations competing methods may exist for strength determination of a member: the main *Specification** and Appendix 1. In this situation there is no preferred method. Either method may be used to determine the strength. The fact that one method may give a greater or lower strength prediction in a given situation does not imply an increased accuracy for either method. The Ω and ϕ factors are designed to ensure that both methods reach their target reliability.

The method of Appendix 1 provides solutions for beams and columns only, but these solutions can be combined with the regular provisions of the main *Specification* to cover other cases. For example, a beam-column may be designed using the interaction equations of the main *Specification*, but replacing the beam and column *available strength* [*factored resistance*] with the provisions of this Appendix. Beam-columns may also be analyzed using the actual *stress* state in a rational analysis extension of DSM (Schafer, 2002b).

In 2012, DSM was extended to include cold-formed steel columns and beams with holes. The method is not constrained by limits on hole shape, size or pattern. Research has shown the validity of the method, even for members with general holes, if: (a) elastic *buckling* is properly evaluated for the presence of such holes, and (b) inelastic *buckling* and yielding are properly evaluated for net section yielding (Moen and Schafer, 2009a). This is in contrast to the main *Specification* where *local buckling* strength equations for members with holes are empirically derived, and therefore require dimensional limits to retain their applicability. Design examples for columns with holes (Moen and Schafer, 2010a) and beams with holes (Moen and Schafer, 2010b) are available. See also the AISI *Cold-Formed Steel Design Manual* (AISI, 2013).

Note:

* The *North American Specification for the Design of Cold-Formed Steel Structural Members*, Chapters A through G and Appendices A and B and Appendix 2, are herein referred to as the main *Specification*.

1.1.1.1 Prequalified Columns

An extensive amount of testing has been performed on concentrically loaded, pin-ended, cold-formed steel columns (Kwon and Hancock, 1992; Lau and Hancock, 1987; Loughlan, 1979; Miller and Peköz, 1994; Mulligan, 1983; Polyzois et al., 1993; Thomasson, 1978). Data from these researchers were compiled and used for calibration of the Direct Strength Method. The geometric limitations listed in Appendix 1 are based on these experiments. In 2006, the prequalified category of Lipped C-Section and Rack Upright were merged, as a rack upright is a C-section with a complex stiffener. In addition, the complex stiffener limits from the original Rack Upright category were relaxed to match those found for C-section beams with complex stiffeners (Schafer, et al., 2006). In 2011, the inside bend radius-to-*thickness* ratio limit for prequalified columns was increased to 20 based on the study by Zeinoddini and Schafer (2010).

It is intended that as more cross-sections are verified for use in the Direct Strength Method, these tables and sections will be augmented. Companies with proprietary sections may wish to perform their own testing and follow Chapter F of the main *Specification* to justify the use of the prequalified Ω and ϕ factors for a particular cross-section. When such testing is performed, the provisions of *Specification* Section 1.1.1.1 provide some relief from the sample size correction factor, C_P, of *Specification* Chapter F. Based on the existing data, the largest observed V_P for the prequalified categories is 15% (AISI, 2006; Schafer, 2008). Therefore, as long as the tested section, over at least three tests, exhibits a $V_P < 15\%$, then the section is assumed to be similar to the much larger database of tested sections used to calibrate the Direct Strength Method and the correction for small sample sizes is not required, and, therefore, C_p is set to 1.0. If the ϕ generated from *Specification* Chapter F is higher than that of Section 1.2.1 of Appendix 1, this is evidence that the section behaves as a prequalified section.

It is not anticipated that member testing is necessarily required for all relevant limit states: *local*, *distortional* and global *buckling*. An engineer may only require testing to reflect a single common condition for the member, with a minimum of three tests in that condition. However, beams and columns should be treated as separate entities. A manufacturer who cannot establish a common condition for a product may choose to perform testing in each of the limit states to ensure reliable performance in any condition. Engineering judgment is required. Note that for the purposes of this section, the test results in *Specification* Chapter F are replaced by test-to-predicted ratios. The prediction is that of the Direct Strength Method (this Appendix) using the actual material and cross-sectional properties from the tests. The P_m parameter, taken as equal to one in *Specification* Chapter F, is taken instead as the mean of the test-to-predicted ratios, and V_P is the accompanying coefficient of variation.

Alternatively, member geometries that are not prequalified may still use the method of Appendix 1, but with the increased Ω and reduced ϕ factors consistent with any rational analysis method as prescribed in A1.2 of the main *Specification*.

1.1.1.2 Prequalified Beams

An extensive amount of testing has been performed on laterally braced beams (Cohen, 1987; Ellifritt et al., 1997; LaBoube and Yu, 1978; Moreyara, 1993; Phung and Yu, 1978; Rogers, 1995; Schardt and Schrade, 1982; Schuster, 1992; Shan et al., 1994; Willis and Wallace, 1990) and on hats and decks (Acharya and Schuster, 1998; Bernard, 1993; Desmond, 1977; Höglund, 1980; König, 1978; Papazian et al., 1994). Data from these researchers were compiled and used for calibration of the Direct Strength Method. The geometric limitations listed in the Appendix are based on the experiments performed by these researchers. The original geometric limits were extended to cover C- and Z-section beams with complex lip stiffeners based on the work of Schafer et al. (2006). In 2012, the inside bend radius-to-*thickness* ratio limit for prequalified beams was increased to 20 based on the study by Zeinoddini and Schafer (2010). For rounded edge stiffeners or other edge stiffeners that do not meet the geometric criteria either for prequalified simple or complex stiffeners, one may still use the method of Appendix 1, but instead with the rational analysis Ω and ϕ factors prescribed in A1.2 of the main *Specification*. See the note on prequalified columns for further commentary on members that do not meet the prequalified geometric limits.

For beams that do not meet the material and geometric requirements defined by the prequalified categories, similar to column design, provisions are provided to potentially permit those members to use the Ω and ϕ factors of the prequalified members by using *Specification* Chapter F as discussed in detail in *Commentary* Section 1.1.1.1 above.

Users of this Appendix should be aware that prequalified beams with large *flat width-to-thickness ratios* in the compression *flange* will be conservatively predicted by the method of this Appendix when compared to the main *Specification* (Schafer and Peköz, 1998). However, the same beam with small longitudinal stiffeners in the compression *flange* will be well-predicted using this Appendix.

1.1.2 Elastic Buckling

The elastic *buckling* load is the load in which the equilibrium of the member is neutral between two alternative states: buckled and straight. Thin-walled cold-formed steel members have at least three relevant elastic *buckling* modes: local, distortional, and global (Figure C-1.1.2-1). The global *buckling* mode includes *flexural*, *torsional*, or *flexural-torsional buckling* for columns, and *lateral-torsional buckling* for beams. Traditionally, the main *Specification* has only addressed *local* and global *buckling*. Further, the main *Specification*'s approach to *local buckling* is to conceptualize the member as a collection of "elements" and investigate *local buckling* of each element separately.

The method of this Appendix provides a means to incorporate all three relevant *buckling* modes into the design process. Further, all *buckling* modes are determined for the member as a whole rather than element by element. This ensures that compatibility and equilibrium are maintained at element junctures. Consider, as an example, the lipped C-section shown in pure compression in Figure C-1.1.2-1(a). The member's *local* elastic *buckling* load from the analysis is:

$P_{cr\ell}$ = 0.12 x 48.42 kips = 5.81 kips (25.84 kN)

The column has a *gross area* (A_g) of 0.881 in^2 (568.4 mm^2); therefore,

$f_{cr\ell} = P_{cr\ell}/A_g$ = 6.59 ksi (45.44 MPa)

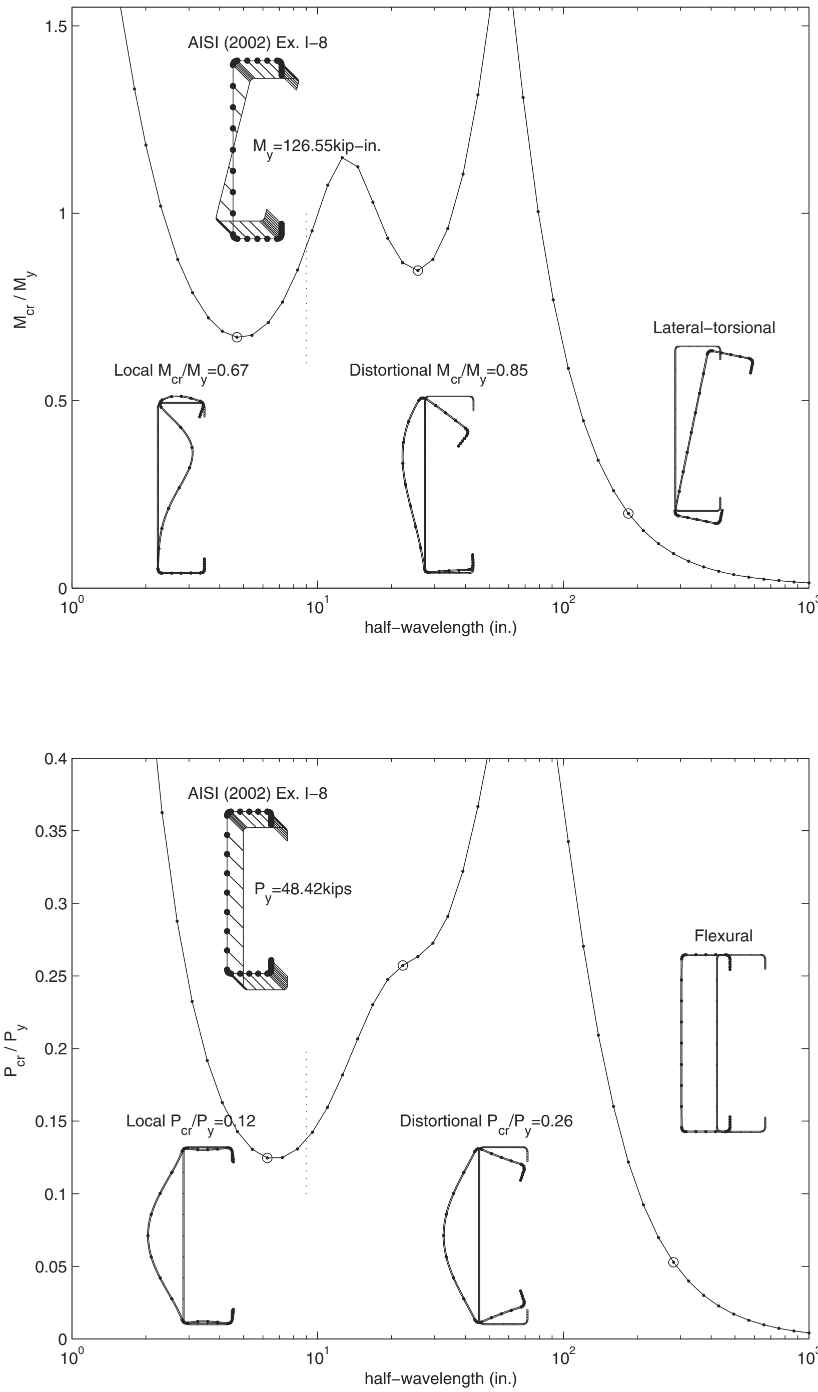
The main *Specification* determines a plate *buckling* coefficient, k, for each element, then f_{cr}, and finally the *effective width*. The centerline dimensions (ignoring corner radii) are h = 8.94 in. (227.1 mm), b = 2.44 in. (62.00 mm), d = 0.744 in. (18.88 mm), and t = 0.059 in. (1.499 mm), the critical *buckling stress*, f_{cr} of each element as determined from the main *Specification*:

lip: k = 0.43, $f_{cr\ell\text{-lip}} = 0.43[\pi^2E/(12(1-\mu^2))](t/d)^2$ = 72.1 ksi (497 MPa)

flange: k = 4, $f_{cr\ell\text{-flange}} = 4.0[\pi^2E/(12(1-\mu^2))](t/b)^2$ = 62.4 ksi (430 MPa)

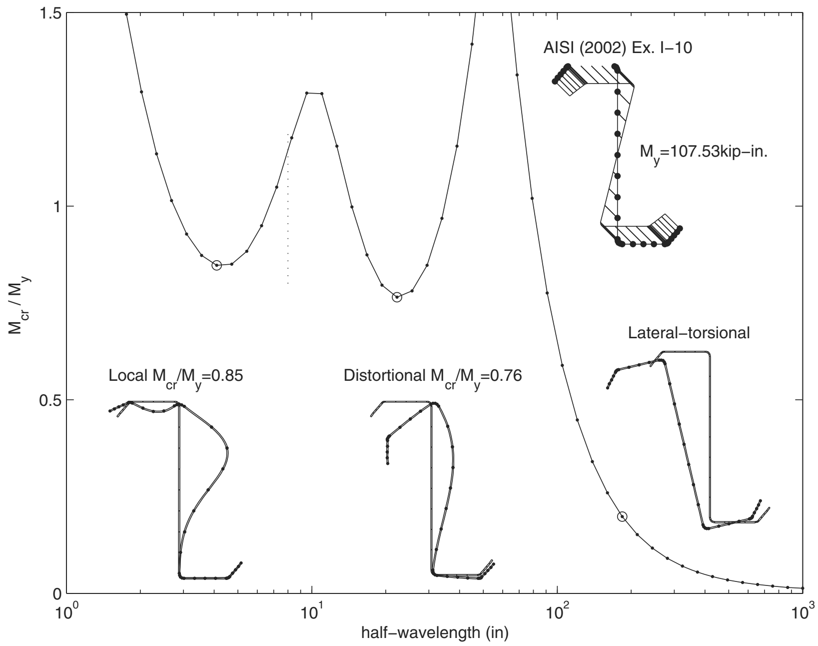
web: k = 4, $f_{cr\ell\text{-web}} = 4.0[\pi^2E/(12(1-\mu^2))](t/h)^2$ = 4.6 ksi (32.0 MPa)

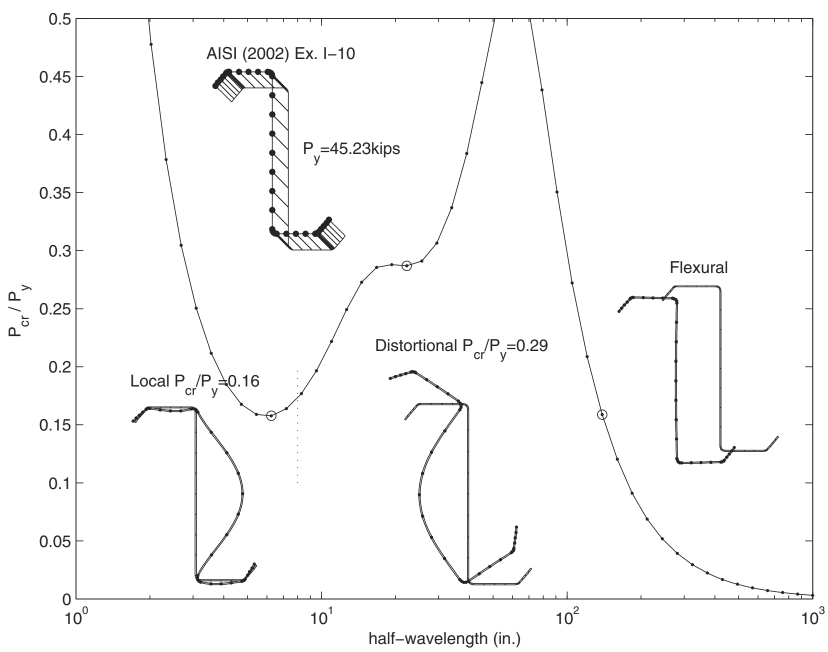
Each element predicts a different *buckling stress*, even though the member is a connected group. These differences in the *buckling stress* are ignored in the main *Specification*. The high *flange* and lip *buckling stresses* have little relevance given the low *web buckling stress*. The finite strip analysis, which includes the interaction amongst the elements, shows that the *flange* aids the *web* significantly in *local buckling*, increasing the *web buckling stress* from 4.6 ksi (32.0 MPa) to 6.59 ksi (45.4 MPa), but the *buckling stress* in the *flange* and lip are much reduced due to the same interaction. Comparisons to the *distortional buckling stress* (f_{crd}) using k from B4.2 of the main *Specification* do no better (Schafer and Peköz, 1999; Schafer, 2002).



(a) 9CS2.5x059 of AISI 2002 Cold-Formed Steel Design Manual, Example I-8

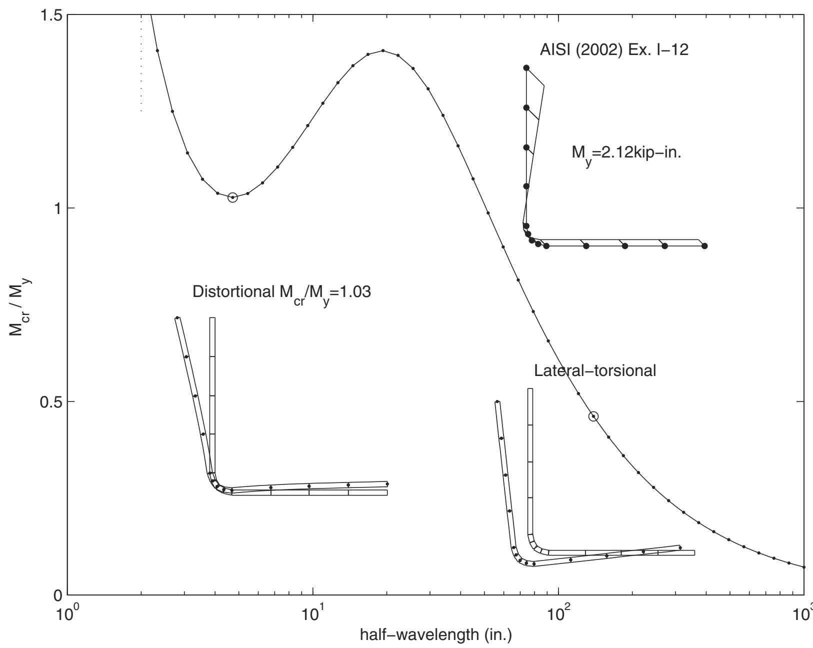
Figure C-1.1.2-1 Examples of Bending and Compression Elastic Buckling Analysis With Finite Strip Method

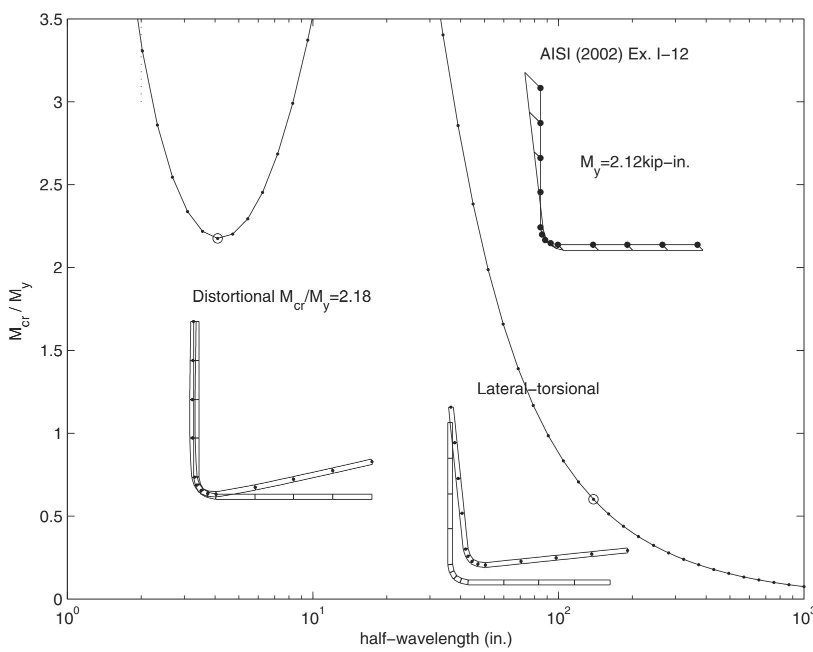




(b) 8ZS2.25x059 of AISI 2002 Cold-Formed Steel Design Manual, Example I-10

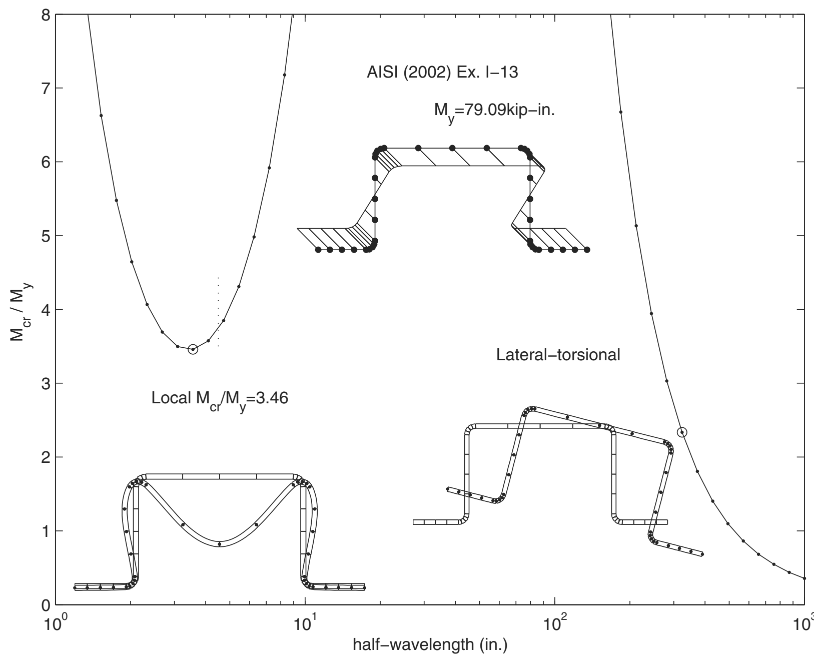
Figure C-1.1.2-1 Examples of Bending and Compression Elastic Buckling Analysis With Finite Strip Method (cont.)

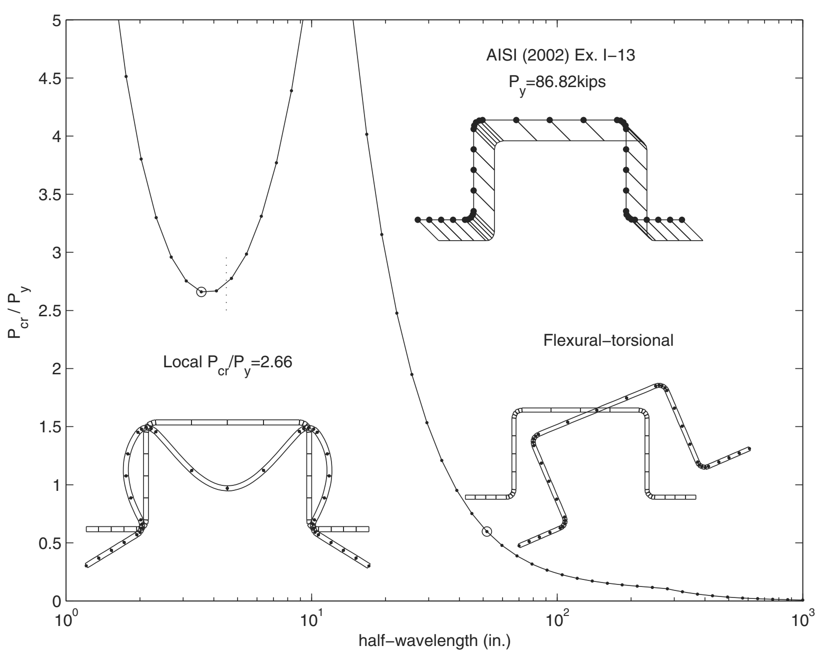




(c) 2LU2x060 of AISI 2002 Cold-Formed Steel Design Manual, Example I-12

Figure C-1.1.2-1 Examples of Bending and Compression Elastic Buckling Analysis With Finite Strip Method (cont.)





(d) 3HU4.5x135 of AISI 2002 Cold-Formed Steel Design Manual, Example I-13

Figure C-1.1.2-1 Examples of Bending and Compression Elastic Buckling Analysis With Finite Strip Method (cont.)

The method of this appendix allows rational analysis to be used for determining the *local*, *distortional* and global *buckling* load or moment. Specific guidance on elastic *buckling* determination follows. Users are reminded that the strength of a member is not equivalent to the elastic *buckling* load (or moment) of the member. In fact, the elastic *buckling* load can be lower than the actual strength, for slender members with considerable post-*buckling* reserve; or the elastic *buckling* load can be fictitiously high due to ignoring inelastic effects. Nonetheless, the elastic *buckling* load is a useful reference load for determining a member's slenderness and ultimately its strength.

Manual and numerical solutions for elastic *buckling* prediction are covered in the following sections. It is permissible to mix the manual and numerical methods; in some cases it is even advantageous. For example, numerical solutions for member *local* and *distortional buckling* are particularly convenient; however, unusual long column bracing conditions $(KL)_x \neq (KL)_y \neq (KL)_t$ may often be handled with less confusion using the traditional manual formulas. Use of the numerical solutions is generally encouraged, but verification with the manual solutions can aid in building confidence in the numerical solution.

Members with holes were added to Appendix 1 of the *Specification* in 2012. For members with holes, the determination of the elastic *local*, *distortional*, and global *buckling* loads including the influence of the holes can be obtained with numerical (e.g., finite element) analysis where the holes are explicitly considered or with the approximate methods provided in this *Commentary*. The following two cases are identified:

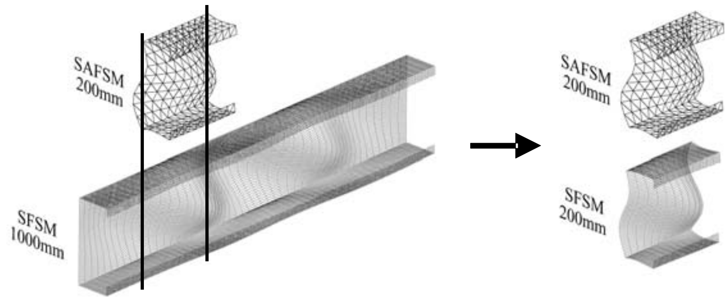
1) Commonly found perforations/holes in industry

To establish the elastic *buckling* values, simplified approximate elastic *buckling* methods are presented in this *Commentary* for the case of flat-punched discrete holes in the *web* or *flange* elements (or both).

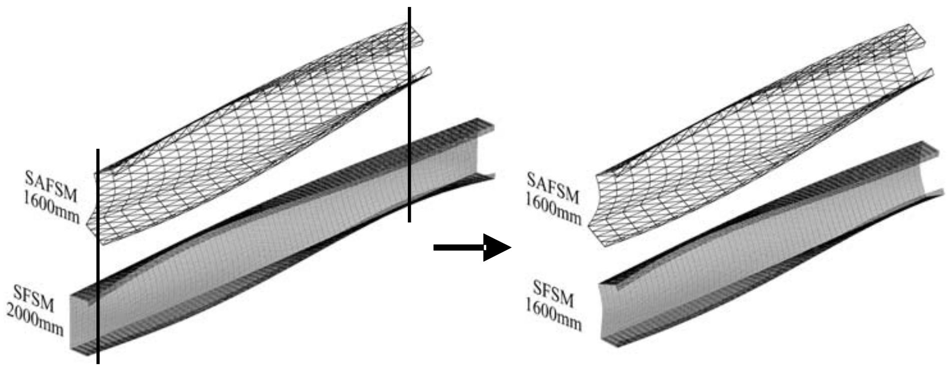
2) Flange-stiffened and/or pattern-type holes

Members with *flange*d or stiffened holes and members with patterned holes (storage racks) currently require a thin shell finite element Eigen-buckling analysis to establish the elastic *buckling* values. In this case, no specific solutions are given in the *Commentary*. Work is ongoing to provide general simplified methods for these cases in the near future (Grey and Moen, 2011; Casafont et al., 2012; Smith and Moen, 2013).

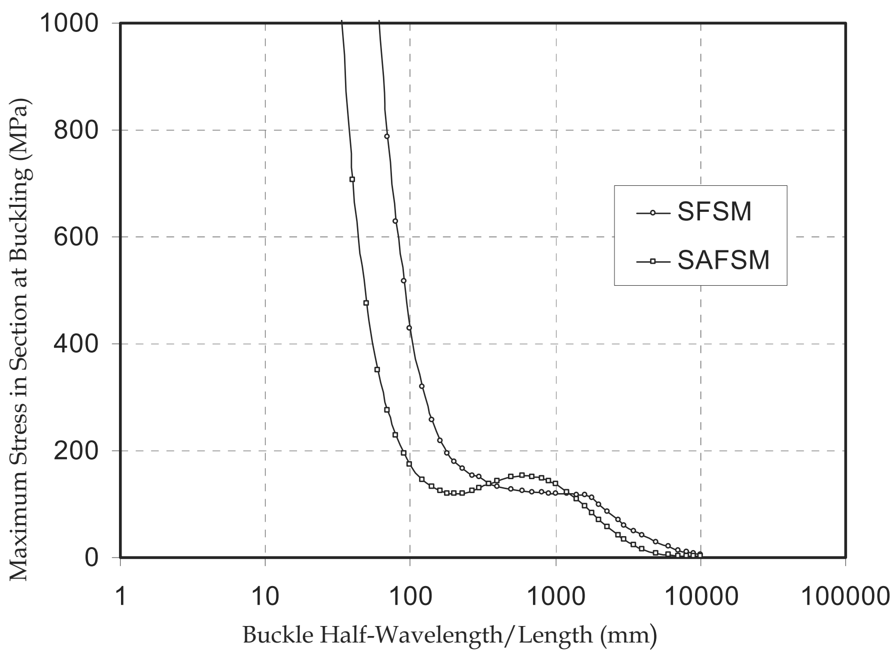
Members in pure shear were added to Appendix 1 of the *Specification* in 2012. Members in pure shear can also undergo *buckling* of the whole section in the form of *local buckling* as shown in Figure C-1.1.2-2(a) or *distortional buckling* as shown in Figure C-1.1.2-2(b) depending on the geometry of the section, loading and restraint. Pure *shear buckling* is different from that for compression or bending in that the nodal lines are not perpendicular to the axis of the section as shown for the *shear local buckling* mode in Figure C-1.1.2-2(a). The modes shown as Semi-Analytical Finite Strip Method (SAFSM) apply to a single half-wavelength of an infinitely long section, and those designated as Spline Finite Strip Method (SFSM) apply to a section of finite length with simply supported ends. The SAFSM and SFSM methods are discussed further in *Commentary* Section 1.1.2.1. Typically, the local mode dominates at short half-wavelengths, and *distortional buckling* is evident at longer half-wavelengths in some instances. The *buckling stress* versus half-wavelength curves from Hancock and Pham (2011) are shown in Figure C-1.1.2-2(c). The minimum on the SAFSM curve corresponds to the value on the SFSM curve at longer half-wavelengths where end conditions do not affect the *buckling*.



(a) Local Buckling Modes in Pure Shear



(b) Distortional Buckling Modes in Pure Shear



(c) SAFSM and SFSM Curves of Buckling Stress Versus Half-Wavelength/Length for Plain Lipped Channel

Figure C-1.1.2-2 Examples of Shear Elastic Buckling Analysis With Finite Strip Method

1.1.2.1 Elastic Buckling - Numerical Solutions

A variety of numerical methods–finite element, finite differences, boundary element, generalized beam theory, finite strip analysis, and others–may provide accurate elastic *buckling* solutions for cold-formed steel beams and columns.

Traditional finite element analysis using thin plate or shell elements may be used for elastic *buckling* prediction. Due to the common practice of using polynomial shape functions, the number of elements required for reasonable accuracy can be significant. Finite element analysis texts such as Cook et al. (1989) and Zienkiewicz and Taylor (1989, 1991) explain the basic theory, while a number of commercial implementations can provide accurate elastic *buckling* answers if implemented with care. Finite difference solutions for plate stability are implemented by Harik et al. (1991) and others. The boundary element method may also be used for elastic stability (Elzein, 1991).

Generalized beam theory–developed by Schardt (1989), extended by Davies et al. (1994), and implemented by Davies and Jiang (1996, 1998), and Silvestre and Camotim (2002a, 2002b)–has been shown to be a useful tool for elastic stability analysis of cold-formed steel members. The ability to separate the different *buckling* modes makes the method especially amenable to design methods.

Finite strip analysis is a specialized variant of the finite element method. For elastic stability of cold-formed steel structures, it is one of the most efficient and popular methods. Cheung and Tham (1998) explain the basic theory while Hancock et al. (2001) and Ádány and Schafer (2006) provide specific details for stability analysis with this method. Hancock and his researchers pioneered the use of finite strip analysis for stability of cold-formed steel members and convincingly demonstrated the important potential of finite strip analysis in both cold-formed steel design and behavior.

The Direct Strength Method of this Appendix emphasizes the use of finite strip analysis for elastic *buckling* determination. Finite strip analysis is a general tool that provides accurate elastic *buckling* solutions with a minimum of effort and time. Finite strip analysis, as conventionally implemented, does have limitations. The two most important are

1. The model assumes the ends of the member are simply supported, and
2. The cross-section may not vary along its length.

These limitations preclude some analysis from readily being used with the finite strip method; but despite these limitations, the tool is useful and a major advancement over plate *buckling* solutions and plate *buckling* coefficients (k) that only partially account for the important stability behavior of cold-formed steel members.

The American Iron and Steel Institute has sponsored research that, in part, has led to the development of the freely available program, CUFSM, which employs the finite strip method for elastic *buckling* determination of any cold-formed steel cross-section. The program is available at www.ce.jhu.edu/bschafer/cufsm and runs on both Windows and MAC platforms. Tutorials and examples are available online at the same address. Current versions of CUFSM expand the application of finite strip analysis to general end boundary conditions (Li and Schafer, 2010).

For sections in bending and compression, the variant of the finite strip method (i.e., the Semi-Analytical Finite Strip Method (SAFSM)) encompassed in CUFSM can be readily used. However, for sections in pure shear, the phase shifts of the *buckling* mode around the section as shown in Figure C-1.1.2-2 require greater sophistication. Available numerical solutions include: 1) a generalized version of SAFSM developed by Plank and Wittrick (1974) and implemented in Hancock and Pham (2011), 2) the Spline Finite Strip Method (SFSM) as developed by Lau and Hancock (1986) (this method can also handle general end boundary conditions), or 3) finite element models with shell elements as discussed above.

1.1.2.1.1 Local Buckling via Finite Strip ($P_{cr\ell}$, $M_{cr\ell}$)

In the Finite Strip Method, members are loaded with a reference *stress* distribution: pure compression for finding P_{cr}, and pure bending for finding M_{cr} (see Figure C-1.1.2-1). Determination of the *buckling* mode requires consideration of the half-wavelength and mode shape of the member. Special attention is given to the half-wavelength and mode shape for *local*, *distortional*, and global *buckling* via finite strip analysis in the following sections.

Half-Wavelength

Local buckling minima occur at half-wavelengths that are less than the largest characteristic dimension of the member under compressive *stresses*. For the examples of Figure C-1.1.2-1, this length has been demarcated with a short vertical dashed line. For instance, the largest out-to-out dimension for the lipped channel of Figure C-1.1.2-1(a) is 9 in. (229 mm); therefore, the cutoff for *local buckling* is at 9 in. (229 mm). Minima in the *buckling* curves that fall at half-wavelengths less than this length are considered as *local buckling* modes. *Buckling* modes occurring at longer lengths are either distortional or global in nature.

The criteria of limiting the half-wavelength for *local buckling* to less than the largest outside dimension under compressive *stresses* is based on the following. *Local buckling* of a simply supported plate in pure compression occurs in square waves, i.e., it has a half-wavelength that is equal to the plate width (the largest outside dimension). If any *stress* gradient exists on the plate, or any beneficial restraint is provided to the edges of the plate by other elements, the critical half-wavelength will be less than the width of the plate. Therefore, *local buckling*, with the potential for stable post-*buckling* response, is assumed to occur only when the critical half-wavelength is less than the largest potential "plate" (i.e., outside dimension with compressive *stresses* applied) in a member.

Mode Shape

Local buckling involves significant distortion of the cross-section, but this distortion involves only rotation, not translation, at the fold lines of the member. The mode shapes for members with edge-stiffened *flanges* such as those of the lipped cee or zee provide a direct comparison between the difference between *local buckling* and *distortional buckling*. Note the behavior at the *flange*/lip junction – for *local buckling,* only rotation occurs; for *distortional buckling,* translation occurs.

Discussion

Local buckling may be indistinct from *distortional buckling* in some members. For example, *buckling* of the unlipped angle may be considered as *local buckling* by the main *Specification*, but is considered as *distortional buckling* as shown in Figure C-1.1.2-1(c) because of the half-wavelength of the mode and the characteristics of the mode shape. By the definitions of this Appendix, no *local buckling* mode exists for this member. *Local buckling* may be at half-wavelengths much less than the characteristic dimension if intermediate stiffeners are in place, or if the element undergoes large tension and small compressive *stress*.

Users may encounter situations where they would like to consider the potential for bracing to retard *local buckling*. Springs may be added to a numerical model to include the effect of external bracing. Care should be used if the bracing only provides support in one direction (such as a deck on a compression *flange*), as the increase of the *local buckling* strength is limited in such a case. In general, since *local buckling* occurs at short wavelengths, it is difficult to effectively retard this mode by external bracing. Changes to the geometry of the member (stiffeners, change of thickness, etc.) should be pursued instead.

Members With Holes

Researchers have observed that holes can change the *local buckling* mode shapes of thin plates and cold-formed steel columns and beams (Kumai, 1952; Schlack, Jr., 1964; Kawai and Ohtsubo, 1968; Vann 1971; Kesti, 2000; El-Sawy and Nazmy, 2001; Sarawit, 2003; and Schafer, 2009b). A finite strip approximate method for predicting $P_{cr\ell}$ and $M_{cr\ell}$ including the influence of holes is described in Moen and Schafer (2009c). The method assumes that *local buckling* occurs as either *buckling* of the unstiffened strip(s) adjacent to a hole at the net section or as *local buckling* of the gross section between holes. This approach is an improvement over element-based methods because the interaction between the unstiffened strip and the connected cross-section is explicitly considered. For a column with holes:

$$P_{cr\ell} = \min(P_{cr\ell nh}, P_{cr\ell h}) \qquad \text{(C-1.1.2-1)}$$

where

$P_{cr\ell nh}$ = *local buckling* load of the gross section by a finite strip analysis

$P_{cr\ell h}$ = *local buckling* load of the net section by a finite strip analysis (e.g., in CUFSM), but restraining the deformations to *local buckling* and examining only those *buckling* half-wavelengths shorter than the length of the hole

To calculate $P_{cr\ell h}$, a finite strip analysis of the net section is performed as shown in Figure C-1.1.2-3. To ensure a consistent comparison of $P_{cr\ell h}$ and $P_{cr\ell nh}$, the reference *stress* used in the net section and gross section finite strip analyses should be calculated with the same reference load (e.g., 1 kip (4.45 kN) on the net section, 1 kip (4.45 kN) on the gross section).

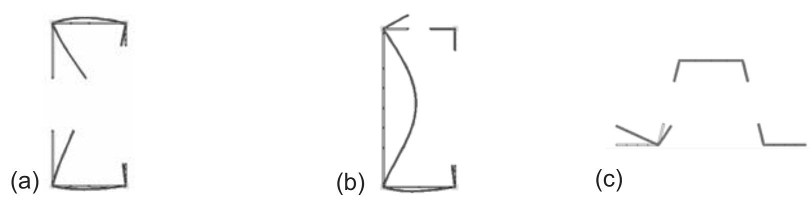


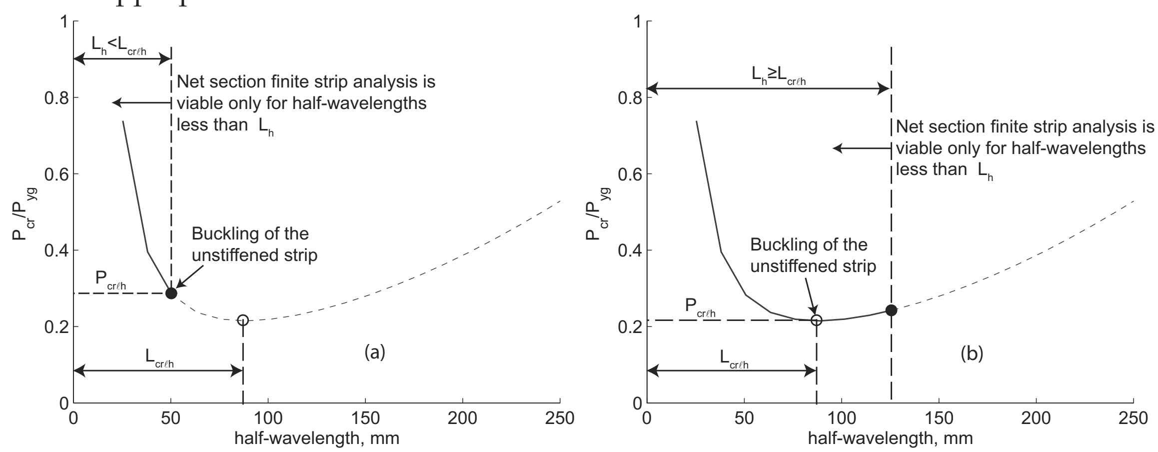
Figure C-1.1.2-3 Modeling a Column Net Cross-Section in the Finite Strip Method (e.g., CUFSM): (a) C-Section With a Web Hole, (b) C-Section With a Flange Hole, (c) Hat Section With Web Holes

Eigen-*buckling* analysis of the restrained cross-section results in an elastic *buckling* curve similar to Figure C-1.1.2-4, where the buckled half-wavelength at the minimum *buckling* load is $L_{cr\ell h}$. When the hole length, L_h, is less than $L_{cr\ell h}$, as shown in Figure C-1.1.2-4(a),

$P_{cr\ell h}$ is equal to the *buckling* load for a single half-wave forming over the length of the hole. (This case is common for circular and square holes, where L_h is less than the width of the cross-sectional element containing the hole.) If $L_h \geq L_{cr\ell h}$ (Figure C-1.1.2-4(b)), $P_{cr\ell h}$ is the minimum on the *buckling* curve, corresponding to a single half-wave forming within the length of the hole. Note that use of the net cross-section for *buckling* half-wavelengths greater than L_h is conservative by failing to reflect the stiffness contributions of the gross section. Knowledge of the specific *buckling* half-wavelength of interest allows the finite strip method to be extended by utilizing the net section, but only for half-waves less than the length of the hole, L_h.

The same approach described previously for columns is also applicable to beams, i.e., $M_{cr\ell}$ =min($M_{cr\ell nh}$, $M_{cr\ell h}$). In this case, the applied reference *stress* in the finite strip analysis should represent as a moment, i.e., 1 kip-in. (113 kN-mm) on the net section and 1 kip-in. (113 kN-mm) on the gross section. See Moen and Schafer (2010b).

The finite strip elastic *buckling* simplified methods presented herein are only appropriate for the case of flat-punched discrete holes in the *web* or *flange* (or both), but not *flange*-stiffened holes or pattern-type holes as would be typical for a rack post. For *flange*-stiffened holes and pattern-type holes, a general finite element elastic *buckling* approach is more appropriate.



(a) Hole Length Less Than $L_{cr\ell h}$ **(b) Hole Length Greater Than $L_{cr\ell h}$**

Figure C-1.1.2-4 Local Elastic Buckling Curve of Net Cross-Section

1.1.2.1.2 Distortional Buckling via Finite Strip (P_{crd}, M_{crd})

Half-Wavelength

Distortional buckling occurs at a half-wavelength intermediate to *local* and global *buckling* modes, as shown in the figures given in C-1.1.2-1. The half-wavelength is typically several times larger than the largest characteristic dimension of the member. The half-wavelength is highly dependent on both the loading and the geometry.

Mode Shape

Distortional buckling involves both translation and rotation at the fold line of a member. *Distortional buckling* involves distortion of one portion of the cross-section and predominantly rigid response of a second portion. For instance, the edge-stiffened *flanges* of the lipped cee and zee are primarily responding as one rigid piece while the *web* is distorting.

Discussion

Distortional buckling may be indistinct (without a minimum) even when *local buckling* and long half-wavelength (global) *buckling* are clear. The lipped cee and zee in bending show this basic behavior. For some members, *distortional buckling* may not occur.

Bracing can be effective in retarding *distortional buckling* and boosting the strength of a member. Continuous bracing may be modeled by adding a continuous spring in a finite strip model. For discrete bracing of *distortional buckling*, when the unbraced length is less than the critical distortional half-wavelength, best current practice is to use the *buckling* load (or moment) at the unbraced length. The key consideration for distortional bracing is limiting the rotation at the compression *flange*/*web* juncture.

Members With Holes

The *distortional buckling* loads P_{crd} and M_{crd} are, at least in part, dictated by the bending stiffness provided by the *web* of an open cross-section as it restrains the attached *flange* from rotating (see Figure C-1.1.2-3). If a hole with length L_h is introduced into the *web* of an open cross-section, the rotational restraint provided by the *web* is decreased, resulting in a lower critical *distortional buckling* load (Kesti, 2000; Moen and Schafer, 2009a). An approximate method for calculating P_{crd} and M_{crd} including the influence of flat-punched unstiffened *web* holes has been developed by Moen and Schafer (2009c). To implement the method, a finite strip analysis is performed with the gross cross-section to identify the *distortional buckling* half-wavelength, L_{crd}. Then, the *web thickness* is modified from t to t_r to simulate the reduction in bending stiffness caused by the presence of a *web* hole:

$$t_r = \left(1 - \frac{L_h}{L_{crd}}\right)^{1/3} t \qquad \text{(C-1.1.2-2)}$$

Note that the cross-section *thickness* is modified over the full depth of the *web*, not just at the location of the hole in the cross-section. The *buckling* load P_{crd} or M_{crd} (including the influence of holes) is obtained with another finite strip analysis of the modified cross-section performed just at L_{crd} of the gross cross-section with the reduced *thickness*. The second analysis is only conducted at L_{crd} as this is the only length for which the reduced *thickness* t_r has any relevance. This finite strip elastic *buckling* simplified method is only appropriate for the case of flat-punched discrete holes in the *web* or *flange* (or both) but not *flange*-stiffened holes or pattern-type holes as would be typical for a rack post. For *flange*-stiffened holes and pattern-type holes, a general finite element elastic *buckling* approach is more appropriate.

1.1.2.1.3 Global (Euler) Buckling via Finite Strip (P_{cre}, M_{cre})

Global *buckling* modes for columns include *flexural*, *torsional* and *flexural-torsional buckling*. For beams bent about their strong-axis, *lateral-torsional buckling* is the global *buckling* mode of interest.

Half-Wavelength

Global (or "Euler") *buckling* modes–*flexural*, *torsional*, or *flexural-torsional buckling* for columns; *lateral-torsional buckling* for beams–occur as the minimum mode at long half-wavelengths.

Mode Shape

Global *buckling* modes involve translation (flexure) and/or rotation (torsion) of the entire cross-section. No distortion exists in any of the elements in the long half-wavelength *buckling* modes.

Discussion

Flexural and *distortional buckling* may interact at relatively long half-wavelengths, making it difficult to determine long column modes at certain intermediate to long lengths. When long column end conditions are not simply supported, or when they are dissimilar for flexure and torsion, higher modes are needed for determining the appropriate *buckling* load. By examining higher modes in a finite strip analysis, distinct flexural and flexural-torsional modes may be identified. Based on the boundary conditions, the effective length, KL, for a given mode can be determined. With KL known, then P_{cre} (or M_{cre}) for that mode may be read directly from the finite strip at a half-wavelength of KL by using the curve corresponding to the appropriate mode. For beams, C_b of the main *Specification* may be employed to account for the moment gradient. Mixed flexural and torsional boundary conditions may not be directly treated. Alternatively, traditional manual solutions may be used for global *buckling* modes with different bracing conditions.

1.1.2.1.4 Shear Buckling via Finite Strip (V_{cr})

Methods for computing the *shear buckling* for plain lipped channels by the Semi-Analytical Finite Strip Method (SAFSM) are given in Hancock and Pham (2011) and by the Spline Finite Strip Method (SFSM) in Pham and Hancock (2009a). *Shear buckling* requires the sections to be subjected to a shear flow associated with pure shear as shown in Figure C-1.1.2-5.

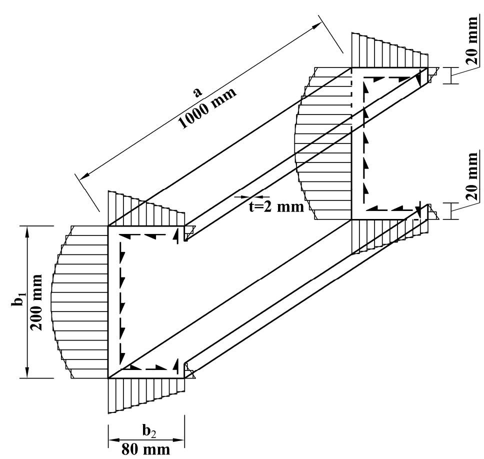


Figure C-1.1.2-5 Shear Flow Distributions in a Lipped Channel

1.1.2.2 Elastic Buckling – Manual Solutions

Local Buckling

Manual solutions for member *local buckling* rely on the use of element plate *buckling* coefficients, as given below.

For columns,

$$P_{cr\ell} = A_g f_{cr\ell} \quad \text{(C-1.1.2-3)}$$

where

A_g = *gross area* of cross-section

$f_{cr\ell}$ = *local buckling stress*

For beams,

$$M_{cr\ell} = S_g f_{cr\ell} \quad \text{(C-1.1.2-4)}$$

where

S_g = gross section modulus to the extreme compression fiber

$f_{cr\ell}$ = *local buckling stress* at the extreme compression fiber

$$= k\frac{\pi^2 E}{12(1-\mu^2)}\left(\frac{t}{w}\right)^2 \quad \text{(C-1.1.2-5)}$$

where

E = modulus of elasticity of steel

μ = Poisson's ratio of steel

t = element *thickness*

w = element *flat width*

k = element (plate) *buckling* coefficient. *Local* plate *buckling* coefficients for an isolated element may be predicted through use of *Commentary* Table C-B2-1. Schafer and Peköz (1999) present additional expressions for stiffened and unstiffened elements under a *stress* gradient. Elastic *local buckling* of a member may be conservatively approximated by using the minimum of the *local buckling stress* of the elements which make up the member. However, using the minimum element solution and ignoring interaction may be excessively conservative for predicting member *local buckling*. To alleviate this, hand methods that account for the interaction of two elements are available. Solutions include two stiffened or edge-stiffened elements (a *flange* and a *web*) under a variety of loading cases (Schafer, 2001 and 2002); and *local buckling* of an edge-stiffened element, including lip/*flange* interaction (Schafer and Peköz, 1999).

Elements With Holes

Moen and Schafer (2009b) provide approximations for $f_{cr\ell}$, including the influence of discrete flat-punched unstiffened hole(s), for stiffened and unstiffened elements in uniaxial compression, and stiffened elements under a *stress* gradient. The approximations include a conversion of the *buckling stress* at the net section to that of the gross section to allow for direct implementation with Equations C-1.1.2-3 and C-1.1.2-4.

Distortional Buckling

Distortional buckling of members with edge-stiffened *flanges* may also be predicted by manual solutions. Unfortunately, the complicated interaction that occurs between the edge-stiffened *flange* and the *web* leads to cumbersome and lengthy formulas.

For Columns,

$$P_{crd} = A_g f_{crd} \qquad \text{(C-1.1.2-6)}$$

A_g = *gross area* of the member

f_{crd} = *distortional buckling stress* (see below)

For Beams,

$$M_{crd} = S_f f_{crd} \qquad \text{(C-1.1.2-7)}$$

S_f = gross section modulus to the extreme compression fiber

f_{crd} = *distortional buckling stress* at the extreme compression fiber. Solutions and design aids for f_{crd} are available for beams (Hancock et al., 1996; Hancock, 1997; Schafer and Peköz, 1999) and for columns (Lau and Hancock, 1987; Schafer 2002). Design aids for *flanges* with unusual edge stiffeners (e.g., Bambach et al., 1998) or flexural members with a longitudinal stiffener in the *web* (Schafer, 1997) are also available. See the *Commentary* on the main *Specification* Sections C3.1.4 and C4.2 for additional information.

Members With Holes

The modified *web thickness*, t_r, in Equation C-1.1.2-2 can be used directly with the hand methods in *Specification* Section C3.1.4 and *Specification* Section C4.2 to approximate the influence of unstiffened *web* holes on P_{crd} and M_{crd}. For beams, t_r replaces t in *Specification* Equation C3.1.4-10 (elastic stiffness contribution of the *web*) and *Specification* Equation C3.1.4-16 (geometric stiffness contribution of the *web*). For columns, t_r replaces t in *Specification* Equations C4.2-7 and C4.2-8.

Global Buckling

Global *buckling* of members is calculated in the main *Specification*. Therefore, for both beams and columns, extensive closed-form expressions are already available and may be used for manual calculation. See the *Commentary* to *Specification* Sections C4 and C3 for additional details.

For Columns,

$$P_{cre} = A_g f_{cre} \qquad \text{(C-1.1.2-8)}$$

A_g = *gross area* of the member

f_{cre} = minimum of the elastic critical *flexural*, *torsional*, or *flexural-torsional buckling stress*. f_{cre} is equal to F_e of Section C4 of the main *Specification*. The hand methods presented in *Specification* Sections C4.1.1 through C4.1.4 provide all necessary formula. Note: Section C4.1.4 specifically addresses the long-standing practice that F_e (or f_{cre}) may be calculated by rational analysis. Rational analysis hand solutions to long column *buckling* are available - see the *Commentary* for main *Specification* Section C4.1.4, as well as Yu (2000) or Hancock et al. (2001). The hand calculations may be quite lengthy, particularly if member properties x_o and C_w are unknown.

For Beams,

$$M_{cre} = S_f f_{cre} \qquad \text{(C-1.1.2-9)}$$

S_f = gross section modulus to the extreme compression fiber

f_{cre} = elastic critical *lateral-torsional buckling stress*. f_{cre} is equal to F_e of main *Specification* Section C3.1.2.1 for open cross-section members and C3.1.2.2 for closed cross-section members. Hand solutions are well-established for *doubly-* and *singly-symmetric sections*, but not so for *point-symmetric sections* (zees). F_e of *point-symmetric sections* is taken as half of the value for *doubly-symmetric sections*. Rational numerical analysis may be desirable in cases where a close to exact solution is required.

Members With Holes

The global (Euler) *buckling* load, P_{cre} for columns and M_{cre} for beams, decreases when holes are present (Sarawit, 2003; Moen and Schafer, 2009a). A "weighted average" approach employing the net section and gross section properties can be utilized to approximate P_{cre} (Moen and Schafer, 2009c). For weak-axis or strong-axis *flexural buckling* of a column with *n* arbitrarily spaced holes or net section regions:

$$P_{cre} = \frac{\pi^2 E I_{avg}}{(KL)^2} \qquad \text{(C-1.1.2-10)}$$

where

KL = effective column length

I_{avg} = "weighted average" of moment of inertia

$$= \left[\frac{I_g L_g + I_{net} L_{net} + T\left(I_g - I_{net}\right)}{L}\right] \qquad \text{(C-1.1.2-11)}$$

where

I_g = gross section moment of inertia

L_g = total segment length without holes

I_{net} = net section moment of inertia

L_{net} = total length of holes or net section regions

L = total member length

= $L_g + L_{net}$

$$T = \frac{L}{2\pi}\sum_{j=1}^{n}\cos\left(\frac{2\pi c_j}{L}\right)\sin\left(\frac{\pi L_{h,j}}{L}\right) \qquad \text{(C-1.1.2-12)}$$

where

$L_{h,j}$ = length of hole or net section region j

c_j = distance from top of column to hole centerline or net section region; see Figure C-1.1.2-6

$$L_{net} = \sum_{j=1}^{n} L_{h,j} \qquad \text{(C-1.1.2-13)}$$

If the holes or net section regions are spaced symmetrically about the longitudinal mid-height of the column, then T=0. For example, T=0 for a column with two holes located in a column at $c_1 = L/3$ and $c_2 = 2L/3$.

The "weighted average" approach for *flexural buckling* can be extended to the general case of calculating P_{cre} for *flexural*, *torsional* and *flexural-torsional buckling* in columns as

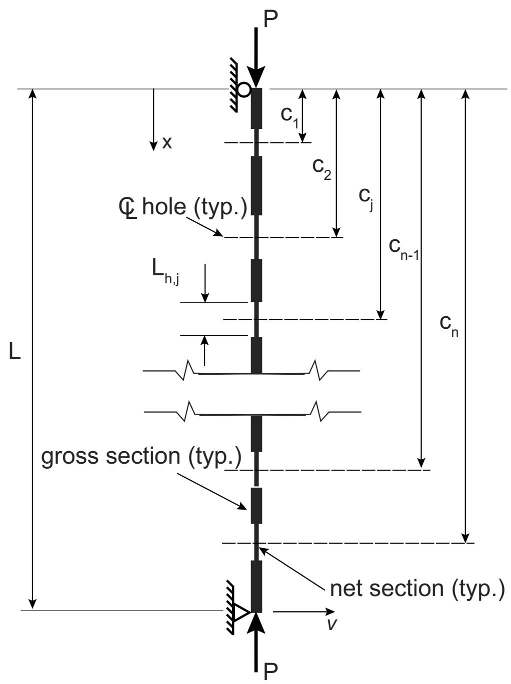


Figure C-1.1.2-6 A Column with j = 1, 2, ..., n Holes or Net Section Regions

described in Moen and Schafer (2009c). For *doubly-* or *singly-symmetric sections* subjected to *torsional* or *flexural-torsional buckling*, the elastic *flexural-torsional buckling* load, P_{cre}, including the influence of holes, can be calculated with the following modifications to equations in *Specification* Section C4.1.2:

$$P_{cre} = \frac{A_g}{2\beta}\left[(\sigma_{ex}+\sigma_t)-\sqrt{(\sigma_{ex}+\sigma_t)^2-4\beta\sigma_{ex}\sigma_t}\right] \quad \text{(C-1.1.2-14)}$$

where

$$\beta = 1-\left(\frac{x_{o,avg}}{r_{o,avg}}\right)^2 \quad \text{(C-1.1.2-15)}$$

$$\sigma_{ex} = \frac{\pi^2 E I_{x,avg}}{A_g (K_x L_x)^2} \quad \text{(C-1.1.2-16)}$$

$$\sigma_t = \frac{1}{A_g r_{o,avg}^2}\left[G J_{avg} + \frac{\pi^2 E C_{w,net}}{(K_t L_t)^2}\right] \quad \text{(C-1.1.2-17)}$$

The radius of gyration about the shear center is defined as $r_{o,avg}=(r_{x,avg}^2 + r_{y,avg}^2 + x_{o,avg}^2)^{0.5}$, where the “weighted average” x distance from the shear center to the centroid of the cross-section is $x_{o,avg}$:

$$x_{o,avg} = \frac{x_{o,g}L_g + x_{o,net}L_{net}}{L} \quad \text{(C-1.1.2-18)}$$

Note that the form of Equation C-1.1.2-18 is applicable to a column with holes spaced evenly about its mid-height, and can be a viable approximation even when the holes are not evenly spaced.

The radii of gyration about the centroidal axes are $r_{x,avg} = (I_{x,avg} / A_{avg})^{0.5}$ and $r_{y,avg} = (I_{y,avg}/A_{avg})^{0.5}$, where $I_{x,avg}$, $I_{y,avg}$, and A_{avg} are calculated using the same form of Equation C-1.1.2-18. Note that the *gross cross-sectional area*, A_g, in Equations C-1.1.2-14, C-1.1.2-16, and C-1.1.2-17 converts the uniform compressive *stress* at the ends of the column to a force and should not be confused with A_{avg}.

The St. Venant torsional constant, J_{avg}, including the influence of holes, can be calculated using the weighted average approach with the same form of Equation C-1.1.2-18. However, the warping torsion constant, C_w, does not follow the weighted average approximation, as the presence of holes prevents warping resistance from developing (Moen and Schafer, 2009c). A viable approximation for warping stiffness at the net section is $C_{w,net}$. Note that all net section properties, e.g., $I_{x,net}$, $I_{y,net}$, A_{net}, $x_{o,net}$, $y_{o,net}$, J_{net}, and $C_{w,net}$, can be readily calculated with the built-in section property calculator in the freely available open source program CUFSM (Schafer and Ádàny, 2006) by setting the element *thicknesses* to zero at the hole(s). See Moen and Schafer (2010a).

The "weighted average" method is also applicable to cold-formed steel beams with holes. For example, an equivalent form of *Specification* Equation C3.1.2.1-4 can be written to calculate the critical elastic *buckling* moment of beams with *singly-* and *doubly-symmetric cross-sections* bending about the symmetry axis:

$$M_{cre} = F_e S_f = C_b \frac{\pi}{K_y L_y} \sqrt{E I_{y,avg}\left(G J_{avg} + E C_{w,net} \frac{\pi^2}{(K_t L_t)^2}\right)} \quad \text{(C-1.1.2-19)}$$

Note that holes are incorporated by replacing I_y and J with $I_{y,avg}$ and J_{avg} and by calculating $C_{w,net}$ assuming the cross-section *thickness* is zero at the hole(s).

Shear Buckling

$$V_{cr} = \frac{\pi^2 E A_w k_v}{12(1-\mu^2)(h/t)^2} \quad \text{(C-1.1.2-20)}$$

V_{cr} = elastic *shear buckling* force of the *web*
E = modulus of elasticity of steel
μ = Poisson's ratio of steel
h = depth of the flat portion of *web* measured along the plane of the *web*
t = *web thickness*
k_v = *shear buckling* coefficient calculated in accordance with main *Specification* Equations C3.2.1-6 and C3.2.1-7. Alternatively, Pham and Hancock (2011) give k_v values for a range of lipped channel section geometries calculated using the Spline Finite Strip Method (SFSM), or Aswegan and Moen (2012) provide k_v values via an energy solution.

1.1.3 Serviceability Determination

The provisions of this Appendix use a simplified approach to deflection calculations that assume the moment of inertia of the section for deflection calculations is linearly proportional to the strength of the section, determined at the allowable *stress* of interest. This approximation avoids lengthy effective section calculations for deflection determination.

1.2 MEMBERS

1.2.1 Column Design

Commentary Section C4 provides a complete discussion on the behavior of cold-formed steel columns as it relates to the main *Specification*. This commentary addresses the specific issues raised by the use of the Direct Strength Method of Appendix 1 for the design of cold-formed steel columns. The thin-walled nature of cold-formed columns complicates behavior and design. Elastic *buckling* analysis reveals at least three *buckling* modes: *local*, *distortional*, and Euler (*flexural*, *torsional*, or *flexural-torsional*) *buckling* that must be considered in design. Therefore, in addition to usual considerations for steel columns–material non-linearity (e.g., yielding), imperfections, and residual *stresses*–the individual role and potential for interaction of *buckling* modes must also be considered. The Direct Strength Method of this Appendix emerged through

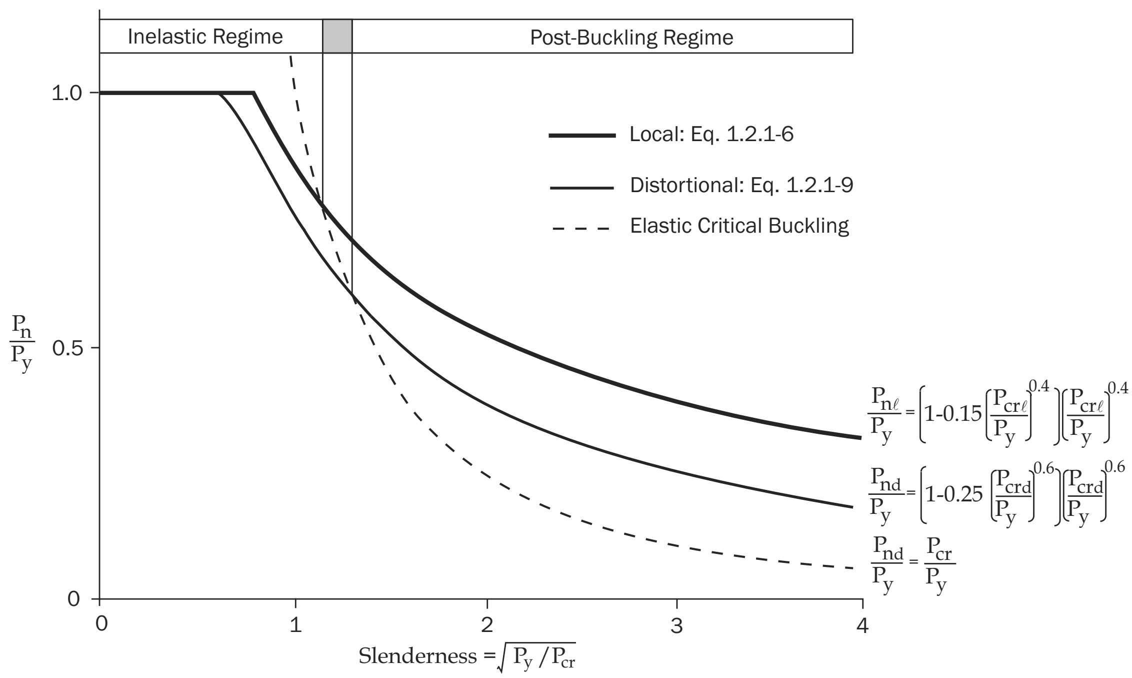


Figure C-1.2.1-1 Local and Distortional Direct Strength Curves for a Braced Column ($P_{ne} = P_y$)

the combination of more refined methods for *local* and *distortional buckling* prediction, improved understanding of the *post-buckling* strength and imperfection sensitivity in distortional failures, and the relatively large amount of available experimental data.

Fully effective or compact columns are generally well-predicted by conventional column curves (AISC, 2001; Galambos, 1998, etc.). Therefore, the long column *nominal strength* [*resistance*], P_{ne}, follows the same practice as the main *Specification* and uses the AISC (2001) curves for strength prediction. The main *Specification* provides the long column strength in terms of a *stress*, F_n (Equations C4.1-2 and C4.1-3). In the Direct Strength Method, this is converted from *stress* to strength by multiplying the *gross area*, A_g, resulting in the formulas for P_{ne} given in Appendix 1.

In the main *Specification*, column *nominal strength* [*resistance*] is calculated by multiplying the nominal column *buckling stress*, F_n, by the *effective area*, A_e, calculated at F_n. This accounts for *local buckling* reductions in the actual column strength (i.e., local-global interaction). In the Direct Strength Method, this calculation is broken into two parts: the long column strength

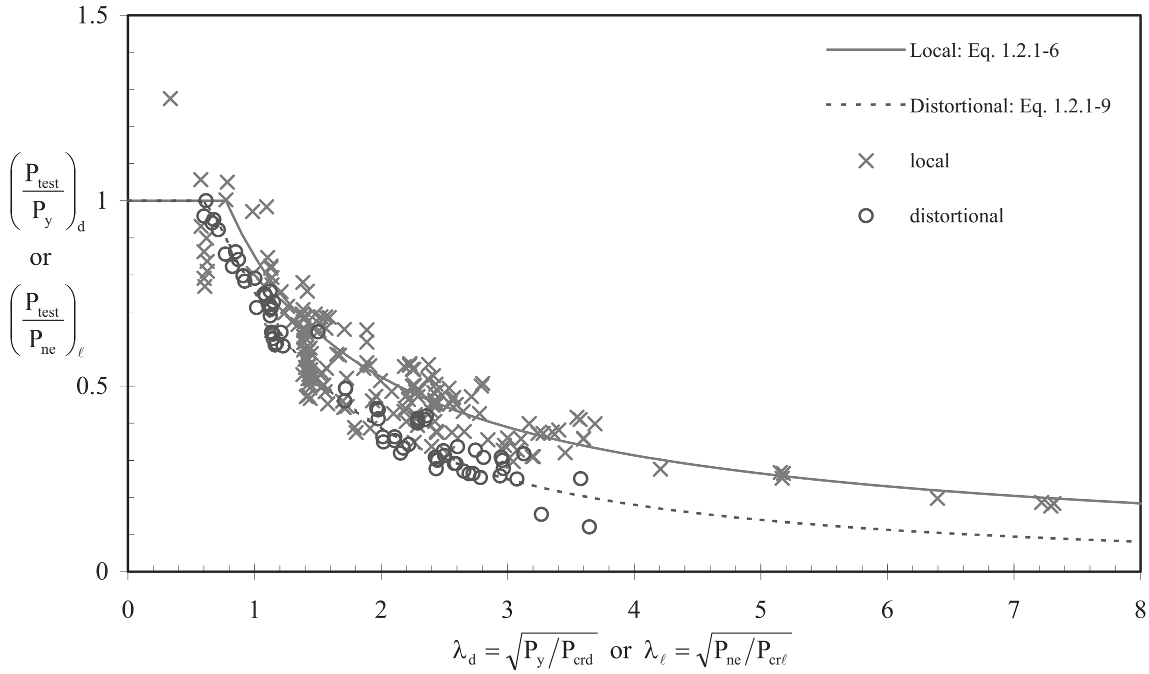


Figure C-1.2.1-2 Direct Strength Method for Concentrically Loaded Pin-Ended Columns

without any reduction for *local buckling* (P_{ne}), and the long column strength considering local-global interaction ($P_{n\ell}$).

The strength curves for *local* and *distortional buckling* of a fully braced column are presented in Figure C-1.2.1-1. The curves are presented as a function of slenderness, which in this case refers to slenderness in the local or distortional mode, as opposed to traditional long column slenderness. Inelastic and post-*buckling* regimes are observed for both *local* and *distortional buckling* modes. The magnitude of the post-*buckling* reserve for the *distortional buckling* mode is less than the *local buckling* mode, as may be observed by the location of the strength curves in relation to the critical elastic *buckling* curve.

The development and calibration of the Direct Strength Method provisions for columns are reported in Schafer (2000, 2002). The reliability of the column provisions was determined using the test data of Appendix 1 Section 1.1.1.1 and the provisions of Chapter F of the main *Specification*. Based on a target reliability, β, of 2.5, a *resistance factor*, ϕ, of 0.84 was calculated for all the investigated columns. Based on this information, the *safety* and *resistance factors* of Appendix 1 Section 1.2.1 were determined for the prequalified members. For the United States and Mexico, ϕ = 0.85 was selected; while for Canada, ϕ = 0.80, since a slightly higher reliability, β, of 3.0 is employed. The *safety factor*, Ω, was back-calculated from ϕ at an assumed dead-to-live *load* ratio of 1 to 5. Since the range of prequalified members is relatively large, extensions of the Direct Strength Method to geometries outside the prequalified set is allowed. Given the uncertain nature of this extension, increased *safety factors* and reduced *resistance factors* are applied in that case, per the *rational engineering analysis* provisions of Section A1.2(c) of the main *Specification*.

The provisions of Appendix 1, applied to the columns of Section 1.1.1.1, are summarized in Figure C-1.2.1-2. The controlling strength is either by Appendix 1 Section 1.2.1.2, which considers *local buckling* interaction with long column *buckling*, or by Section 1.2.1.3, which considers the distortional mode alone. The controlling strength (minimum predicted of the two modes) is highlighted for the examined members by the choice of marker. Overall performance of the method can be judged by examination of Figure C-1.2.1-2. Scatter exists throughout the data set, but the trends in strength are clearly shown, and further, the scatter (variance) is similar to that of the main *Specification*.

The extension of the DSM approach to columns with holes utilizes the elastic *buckling* properties of a cold-formed steel column ($P_{cr\ell}$, P_{crd}, and P_{cre}), including the influence of holes to predict ultimate strength. In most cases, holes decrease the elastic *buckling* properties, $P_{cr\ell}$, P_{crd}, and P_{cre}, which increases a column's local (λ_ℓ), distortional (λ_d) and global (λ_c) slenderness and lowers the predicted strength. Simplified methods for predicting $P_{cr\ell}$, P_{crd}, and P_{cre} including holes are presented in Section 1.1.2. Alternatively, full finite element elastic Eigen-*buckling* analysis can be performed.

The DSM strength prediction expressions have been modified to limit the maximum strength of a column with holes to the capacity of the net cross-section, P_{ynet} (Moen and Schafer, 2011). A transition from P_{ynet}, through the inelastic regime, to the elastic *buckling* portion of the *distortional buckling* strength curve has also been included in the design provisions. The transition slope is dictated by the ratio of the net section capacity to gross section capacity, P_{ynet}/P_y, which was derived based on observed trends in column simulations to collapse, reported in Moen and Schafer (2009a). If a member contains mostly holes, then the critical elastic *buckling* loads and the net section capacity approach zero. The DSM strength equations are written such that when the net section goes to zero, predicted capacity also degrades to zero.

The development and calibration of the Direct Strength Method provisions for columns with holes was performed with experimental and simulation databases as reported in Moen and Schafer (2009a) and summarized in Moen and Schafer (2011). Note that both databases contain only lipped Cee cross-sections with discrete *web* holes because this is what was available in the research literature at the time. However, the philosophy of employing elastic *buckling* parameters ($P_{cr\ell}$, P_{crd}, P_{cre}) to predict the ultimate strength of cold-formed steel columns with holes was thoroughly validated in Moen and Schafer (2009a), and is assumed to hold true for other cross-section shapes.

The generality of the DSM approach for holes was demonstrated across experiments and nonlinear finite element analysis collapse simulations across a wide range of spacing, shape, and size of holes for both cold-formed steel columns and beams. Based on a target reliability, β, of 2.5, the *resistance factor*, ϕ, was calculated as 0.94 (experiments) and 0.89 (simulations) for columns with holes predicted to fail from local-global *buckling* interaction. For columns with holes predicted to experience a *distortional buckling* failure mode, ϕ was calculated as 0.96 (experiments) and 0.91 (simulations). The prediction accuracy for DSM is higher than the main *Specification* for members with holes (Ganesan and Moen, 2012). In addition, DSM takes into account the influence of holes on global *buckling* and *distortional buckling*, neither of which is currently dealt with in the main *Specification*.

1.2.1.1 Flexural, Torsional, or Flexural-Torsional Buckling

As discussed in detail above, the strength expressions for long wavelength *buckling* of columns follow directly from Section C4 of the main *Specification*. These provisions are identical to those used for compact section hot-rolled columns in the AISC Specification (2001) and are fully discussed in the *Commentary* to Section C4. The axial elastic strength, P_{ne}, calculated in this section represents the upper bound capacity for a given column. Actual column strength is determined by considering reductions that may occur due to *local buckling*, and performing a separate check on the distortional mode. See Section 1.1.2 for information on *rational engineering analysis* methods for calculation of P_{cre} considering columns with or without hole(s).

1.2.1.2 Local Buckling

The expression selected for *local buckling* of columns is shown in Figure C-1.2.1-1 and Figure C-1.2.1-2 and is discussed in Section 1.2.1. The potential for local-global interaction is presumed; thus the column strength in *local buckling* is limited to a maximum of the long column strength, P_{ne}. See Section 1.1.2 for information on *rational engineering analysis* methods for calculation of $P_{cr\ell}$. For columns with holes, $P_{n\ell}$ is limited to P_{ynet} to reflect yielding and collapse of the net section when both local and global column slenderness are low.

1.2.1.3 Distortional Buckling

The expression selected for *distortional buckling* of columns is shown in Figure C-1.2.1-1 and Figure C-1.2.1-2 and is discussed in Section 1.2.1. Based on experimental test data and on the success of the Australian/New Zealand code (see Hancock et al., 2001 for discussion and Hancock et al., 1994 for further details), the *distortional buckling* strength is limited to P_y instead of P_{ne}. This presumes that *distortional buckling* failures are independent of long-

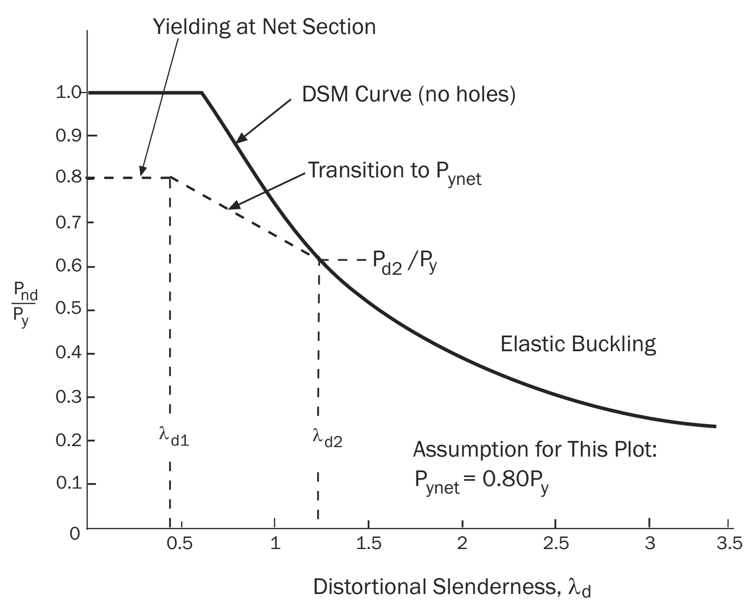


Figure C-1.2.1-3 DSM Distortional Buckling Strength Curve for a Column With Holes

column behavior, i.e., little if any distortional-global interaction exists. See Section 1.1.2 for information on rational analysis methods for calculation of P_{crd}.

Figure C-1.2.1-3 compares the *distortional buckling* strength prediction curve for a column without holes to the prediction curve for the same column with holes, where $P_{ynet} = 0.80P_y$. For the column with holes, P_{nd} is limited to a maximum strength of P_{ynet}. As distortional slenderness increases, the prediction transitions from P_{ynet} to the same strength curve used for columns without holes. The transition is implemented to reflect the change in failure mode as slenderness increases, from yielding at the net section to elastic *distortional buckling* along the column.

1.2.2 Beam Design

The determination of flexural strength using the Direct Strength Method was introduced in 2004. The shear strength and the combined bending and shearing check were introduced in 2012.

1.2.2.1 Bending

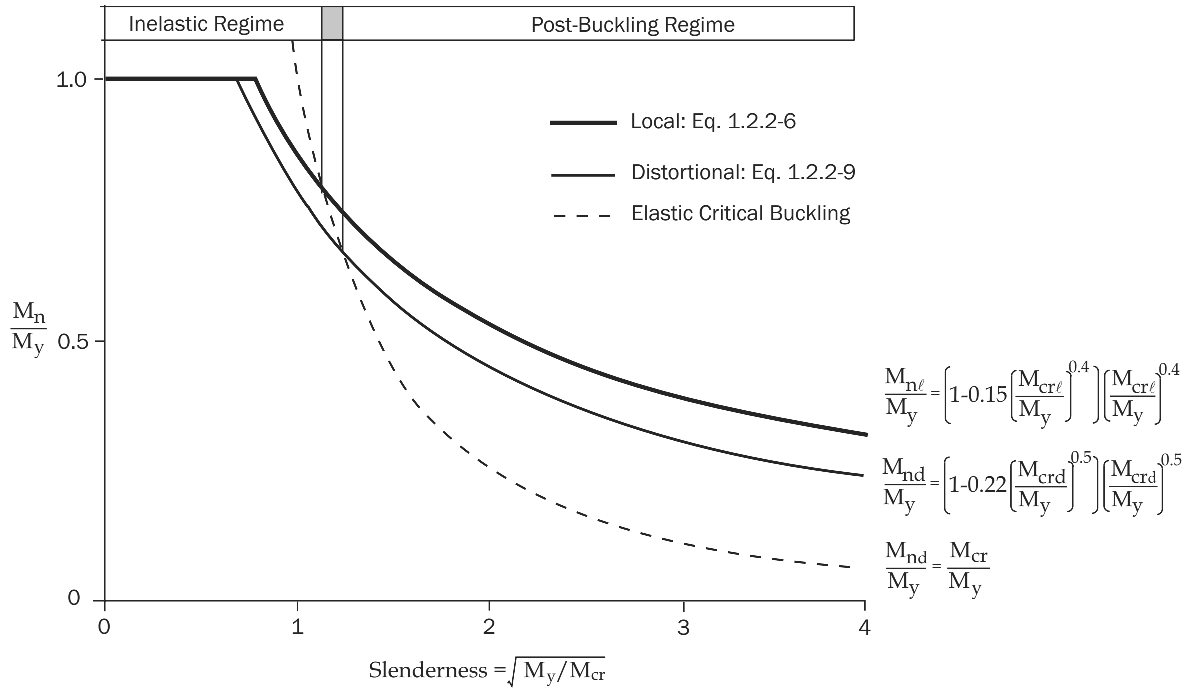
Commentary Section C3 provides a complete discussion on the behavior of cold-formed steel beams as it relates to the main *Specification*. This commentary addresses the specific issues raised by the use of the Direct Strength Method of Appendix 1 for the design of cold-formed steel beams.

The thin-walled nature of cold-formed beams complicates behavior and design. Elastic *buckling* analysis reveals at least three *buckling* modes: *local*, *distortional*, and *lateral-torsional buckling* (for members in strong-axis bending) that must be considered in design. The Direct Strength Method of this Appendix emerged through the combination of more refined methods for *local* and *distortional buckling* prediction, improved understanding of the post-*buckling* strength and imperfection sensitivity in distortional failures, and the relatively large amount of available experimental data.

The *lateral-torsional buckling* strength, M_{ne}, follows the same practice as the main *Specification*. The main *Specification* provides the *lateral-torsional buckling* strength in terms of a *stress*, F_c (Equation C3.1.2.1-8). In the Direct Strength Method, this is converted from a *stress* to a moment by multiplying by the gross section modulus, S_f, resulting in Equation 1.2.2-2 for M_{ne} as given in Appendix 1.

In the main *Specification*, for beams that are not fully braced and locally unstable, beam strength is calculated by multiplying the predicted *stress* for failure in *lateral-torsional buckling*, F_c, by the effective section modulus, S_c, determined at *stress* F_c. This accounts for *local buckling* reductions in the *lateral-torsional buckling* strength (i.e., local-global interaction). In the Direct Strength Method, this calculation is broken into two parts: the *lateral-torsional buckling* strength without any reduction for *local buckling* (M_{ne}), and the strength considering local-global interaction ($M_{n\ell}$).

The strength curves for *local* and *distortional buckling* of a beam fully braced against *lateral-torsional buckling* are presented in Figure C-1.2.2-1 and compared to the critical elastic *buckling* curve. The post-*buckling* reserve for the local mode is predicted to be greater than that of the distortional mode. As depicted in Figure C-1.2.2-1, provisions were added in 2012 for inelastic reserve capacity in bending, i.e. where $M_n > M_y$.



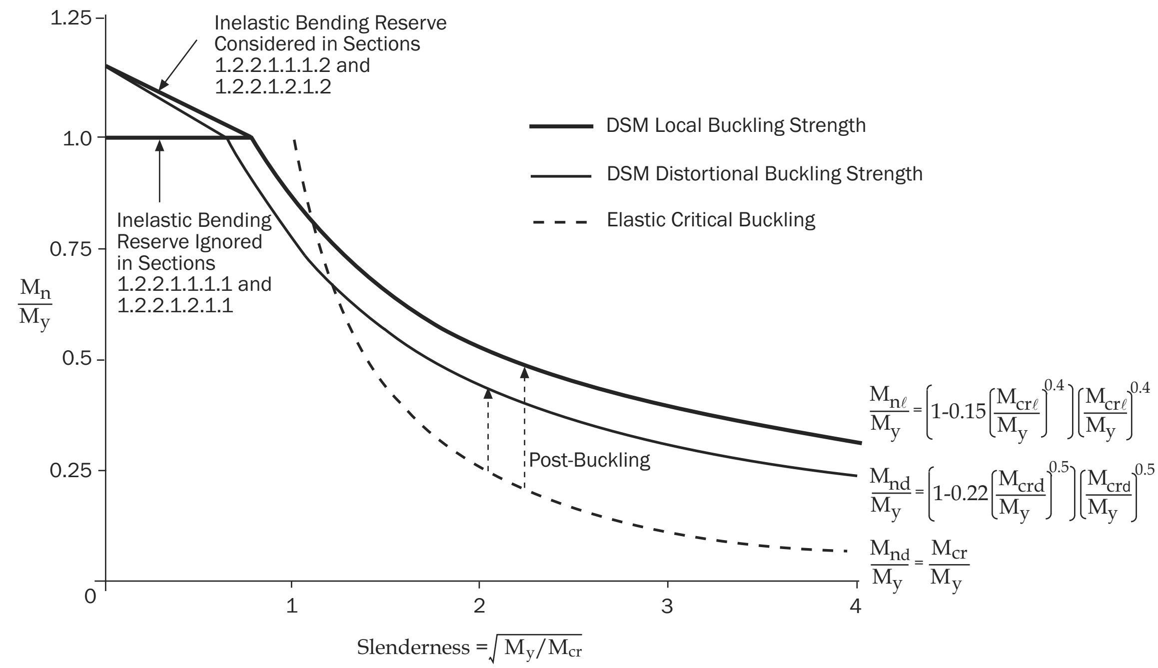


Figure C-1.2.2-1 Local and Distortional Direct Strength Curves for a Beam Braced Against Lateral-Torsional Buckling ($M_{ne} = M_y$)

The reliability of the beam provisions was determined using the test data of Section 1.1.1.2 and the provisions of Chapter F of the main *Specification*. Based on a target reliability, β, of 2.5, a *resistance factor*, ϕ, of 0.90 was calculated for all of the investigated beams. Based on this information, the *safety* and *resistance factors* of Appendix 1 Section 1.2.2 were determined for the prequalified members. For the United States and Mexico, ϕ = 0.90; while for Canada,

$\phi = 0.85$ because Canada employs a slightly higher reliability, β, of 3.0. The *safety factor*, Ω, is back-calculated from ϕ at an assumed dead-to-live *load* ratio of 1 to 5. Since the range of prequalified members is relatively large, extensions of the Direct Strength Method to geometries outside the prequalified set are allowed. However, given the uncertain nature of this extension, increased *safety factors* and reduced *resistance factors* are applied in that case, per the *rational engineering analysis* provisions of Section A1.2(c) of the main *Specification*.

The provisions of Appendix 1, applied to the beams of *Specification* Section 1.1.1.2, are summarized in Figure C-1.2.2-2. The controlling strength is determined either by *Specification* Section 1.2.2.1.2, which considers *local buckling* interaction with *lateral-torsional buckling*, or by *Specification* Section 1.2.2.1.3, which considers the distortional mode alone. The controlling strength (minimum predicted of the two modes) is highlighted for the examined members by the choice of marker. Overall performance of the method can be judged by examination of Figure C-1.2.2-2. The scatter shown in the data is similar to that of the main *Specification.*

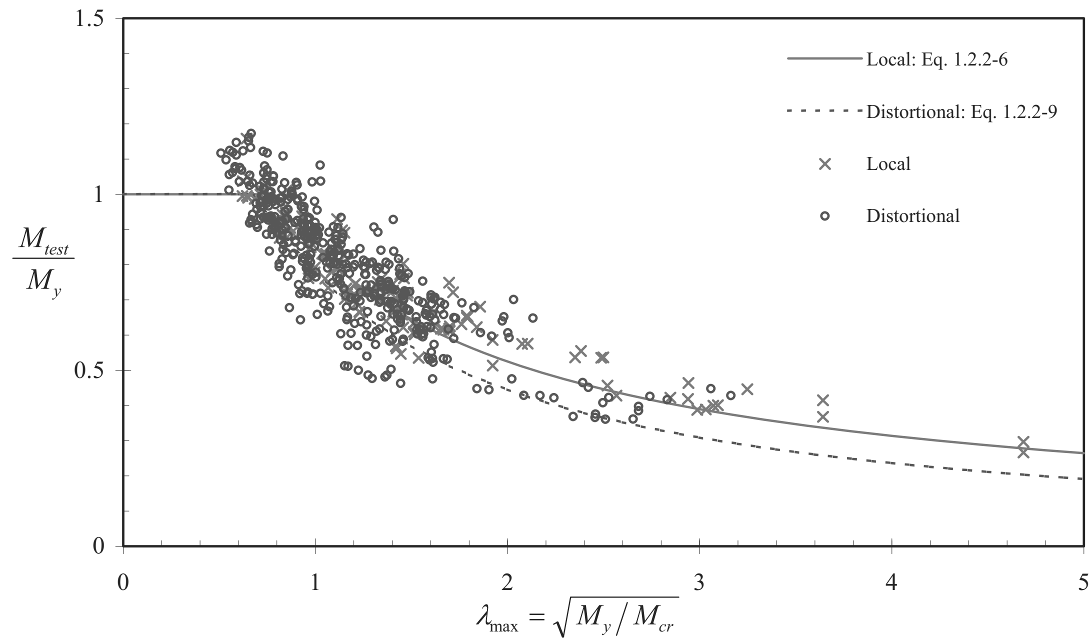


Figure C-1.2.2-2 Direct Strength Method for Laterally Braced Beams

The extension of the DSM approach to beams with holes utilizes the elastic *buckling* properties of a cold-formed steel beam ($M_{cr\ell}$, M_{crd}, and M_{cre}) including the influence of holes to predict ultimate strength. In most cases, holes decrease $M_{cr\ell}$, M_{crd}, and M_{cre}, which increases the column's local (λ_ℓ), distortional (λ_d) and global (λ_c) slenderness and lowers the predicted strength. Simplified methods for predicting $M_{cr\ell}$, M_{crd}, and M_{cre} including holes are presented in Section 1.1.2. Alternatively, full finite element elastic Eigen-*buckling* analysis can be performed.

The DSM strength prediction expressions have been modified to limit the maximum strength of a beam with holes to the capacity of the net cross-section, M_{ynet} (Moen and Schafer, 2009b). A transition from M_{ynet}, through the inelastic regime, to the elastic *buckling* portion of the *distortional buckling* strength curve is also included in the design provisions as shown in Figure C-1.2.2-3. The transition slope is dictated by the ratio of the net section capacity to gross section capacity, M_{ynet}/M_y, which was derived based on observed trends in beam simulations to collapse reported in Moen and Schafer (2009b) and experiments (Moen et al., 2012).

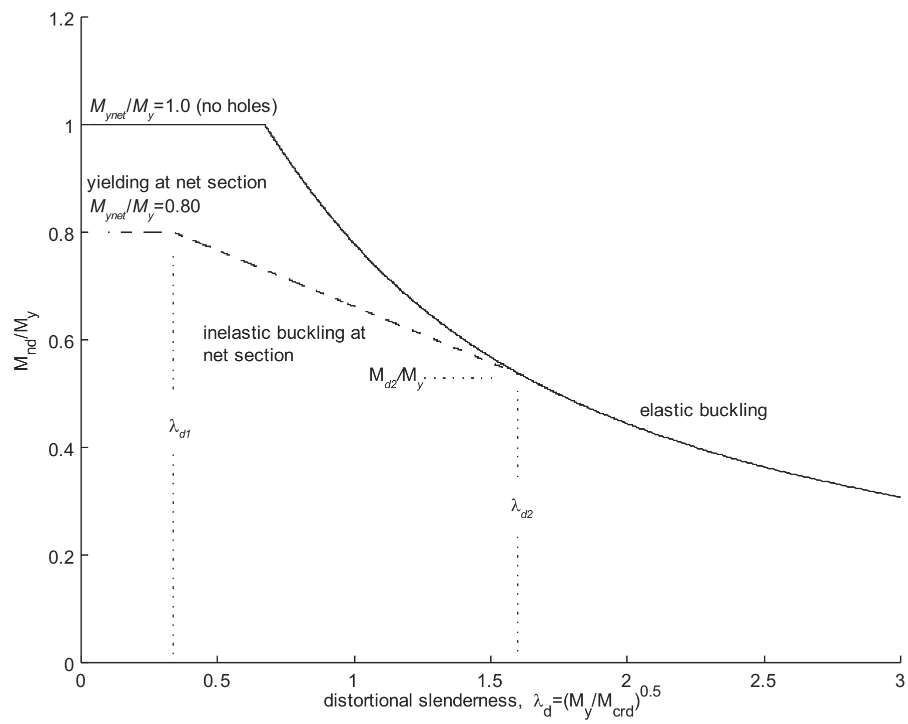


Figure C-1.2.2-3 DSM Distortional Buckling Strength Curve for Beams With Holes

The development and calibration of the Direct Strength Method provisions for beams with holes was performed with a simulation database as reported in Moen and Schafer (2009a) and a set of 12 beam experiments summarized in Moen et al. (2012). Note that the simulations and experiments only considered lipped Cee cross-sections with discrete *web* holes. However, the philosophy of employing elastic *buckling* parameters ($M_{cr\ell}$, M_{crd}, M_{cre}) to predict the ultimate strength of cold-formed steel beams with holes, validated in Moen and Schafer (2009a), is assumed to hold true for other cross-section shapes.

Resistance factors were calculated by limit state with Chapter F of the main *Specification*. Based on a target reliability, β, of 2.5, the *resistance factor*, ϕ, was calculated with the simulation database as 0.95 for laterally braced beams predicted to fail from *local buckling*. For beams predicted to experience a *distortional buckling* failure mode, ϕ was calculated with the simulation database as 0.91 and with the Moen et al. (2012) experiments as 0.94.

In 2012, provisions were added (*Specification* Sections 1.2.2.1.1.1.2, 1.2.2.1.2.1.2, and 1.2.2.1.3.1.2) to take advantage of the inelastic reserve strength for members that are stable enough to allow partial plastification of the cross-section. Such sections have capacities in excess of M_y and potentially as high as M_p (though practically, this upper limit is rarely achievable). As Figure C-1.2.2-1 shows, the inelastic reserve capacity is assumed to linearly increase with decreasing slenderness.

1.2.2.1.1 Lateral-Torsional Buckling

As discussed in detail above, the strength expressions for *lateral-torsional buckling* of beams follow directly from Section C3 of the main *Specification* and are fully discussed in Section C3 of the *Commentary*. The *nominal lateral-torsional buckling strength* [*resistance*], M_{ne}, calculated in this section represents the upper bound capacity for a given beam. Actual beam strength is determined by considering reductions that may occur due to *local buckling* and performing a separate check on the distortional mode. See Section 1.1.2 for information on *rational engineering analysis* methods for calculation of M_{cre}.

The hot-rolled steel design specification (AISC 2005 and 2010) has long provided expressions for inelastic reserve *lateral-torsional buckling* of compact sections. The expression provided in *Specification* Equation 1.2.2-5 is a conservative extension of the AISC approach: first, the M_y/M_{cre} required to develop M_p may be shown equivalent to 1/2 L_p, as employed in AISC; second, the moment gradient factor (C_b) is only used in the elastic *buckling* approximation (for M_{cre}) and not to linearly increase the reserve strength, as in the AISC Specification (Shifferaw and Schafer, 2010).

1.2.2.1.2 Local Buckling

The expression selected for *local buckling* of beams is shown in Figures C-1.2.2-1 and C-1.2.2-2 and is discussed in Section 1.2.2. The use of the Direct Strength Method for *local buckling* and the development of the empirical strength expression are given in Schafer and Peköz (1998). The potential for local-global interaction is presumed; thus, the beam strength in *local buckling* is limited to a maximum of the *nominal lateral-torsional buckling strength* [*resistance*], M_{ne}. For fully braced beams, the maximum M_{ne} value is the yield moment, M_y. See Section 1.1.2 for information on *rational engineering analysis* methods for calculation of $M_{cr\ell}$. For beams with holes, $M_{n\ell}$ is limited to M_{ynet} to reflect yielding and collapse of the net section when both local and global column slenderness are low.

Unique expressions were derived for inelastic bending reserve in *local buckling*. This reserve is only allowed in cross-sections that are predicted to have inelastic bending reserve in *lateral-torsional buckling* (i.e., $M_{ne} > M_y$). The compressive strain which the cross-section may sustain in *local bucking*, $C_{y\ell}\varepsilon_y$, is shown to increase as specified in *Specification* Equation 1.2.2-13 in both back-calculated strains from tested sections and average membrane strains from finite element models (Shifferaw and Schafer, 2010). Local strains in the corners and at the surface of the plates (comprising the cross-section) as they undergo bending may be significantly in excess of $C_{y\ell}\varepsilon_y$ (Shifferaw and Schafer, 2010). As a result, and consistent with the main *Specification*, $C_{y\ell}$ is limited to 3.

For sections with first yield in tension, the potential for inelastic reserve capacity is great, but the design calculations are more complicated. *Specification* Equation 1.2.2-5 only applies after the cross-section begins to yield in compression, i.e., when the moment reaches M_{yc}. Calculation of M_{yc} requires the use of basic mechanics to determine the moment strength in the partially plastfied cross-section. M_y may be used in place of M_{yc}, but this is conservative (excessively so if the tensile strain demands are much higher than the compressive strain demands). Based on experience and past practice, it has also been determined that the tensile strain should not exceed 3 times the yield strain; thus the moment is also limited by this value, i.e., M_{yt3}.

Note: The slenderness λ_ℓ utilizes M_y, instead of M_{ne}, for simplicity in the inelastic reserve regime and to provide continuity with the expressions of *Specification* Section 1.2.2.1.2.1.1. Further, the elastic *buckling* moment, $M_{cr\ell}$, is determined based on the elastic bending *stress* distribution, not the plastic *stress* distribution. These simplifications were shown to be sufficiently accurate when compared with existing tests and a parametric study using rigorous nonlinear finite element analysis (Shifferaw and Schafer, 2010).

1.2.2.1.3 Distortional Buckling

The expression selected for *distortional buckling* of beams is shown in Figures C-1.2.2-1 and C-1.2.2-2 and is discussed in Section 1.2.2. Based on experimental test data and on the success of the Australian/New Zealand code (see Hancock, 2001 for discussion), the *nominal distortional buckling strength* [*resistance*] is limited to M_y instead of M_{ne}. This presumes that *distortional buckling* failures are independent of *lateral-torsional buckling* behavior, i.e., little if any distortional-global interaction exists. See Section 1.1.2 for information on rational analysis methods for calculation of M_{crd}.

For beams with holes, M_{nd} is limited to a maximum *nominal strength* [*resistance*] of M_{ynet}. As distortional–slenderness increases, the prediction transitions from M_{ynet} to the same strength curve used for beams without holes. The transition is implemented to reflect the change in failure mode as slenderness increases, from yielding at the net section to elastic *distortional buckling* along the beam.

The approach for strength prediction in inelastic reserve *distortional buckling* is similar to that of inelastic reserve *local buckling*. Use of the same form for C_{yd} in *Specification* Equation 1.2.2-26 as that of $C_{y\ell}$ in *Specification* Equation 1.2.2-29 results in slightly more conservative strength predictions for inelastic *distortional buckling* (Shifferaw and Schafer, 2010). *Specification* simplicity and greater concern with post-collapse response in *distortional buckling* is used as justification for this additional conservatism.

1.2.2.2 Shear

The Direct Strength Method (DSM) equations for shear are based on those in main *Specification* Section C3.2.1. Validation for the *local buckling* equations in DSM format has been confirmed (Pham and Hancock, 2012a) by tests on high-strength steel C-sections in shear, and combined bending and shear, and the tests of LaBoube and Yu (1978a). The Pham and Hancock tests show that considerable tension field action is available for *local buckling* if the *web* is fully restrained at the loading and support points over its full depth by bolted connections. This post-local *buckling* has been included in the DSM equations as a higher tier alternative (*Specification* Equations 1.2.2-36 and 1.2.2-37) for aspect ratios up to 2:1 based on the testing and FEM analyses (Pham and Hancock, 2012b). The DSM equations allow elastic local critical *shear buckling* force, V_{cr}, to be determined by an elastic *buckling* analysis of the whole section or *web* in pure shear including longitudinal intermediate stiffeners. Experimental justification for inclusion of small longitudinal intermediate stiffeners in the value of V_{cr} in the DSM shear equations is given in Pham and Hancock (2012a). *Distortional buckling* in shear has been ignored at this stage.

1.2.2.3 Combined Bending and Shear

The DSM equations for combined bending and shear are based on those in main *Specification* Section C3.3. The *nominal flexural strength* [*resistance*], $M_{n\ell}$, for *local buckling* from *Specification* Section 1.2.2.1.2 has been used in the interaction equations since combined bending and shear occur in regions of high moment gradient where *distortional buckling* is unlikely to play a significant role. *Distortional buckling* is checked independently in *Specification* Section 1.2.2.1. Validation of this approach has been confirmed from tests of lapped *purlins* (Pham and Hancock, 2009b) and tests on high-strength steel C-sections in combined bending and shear (Pham and Hancock, 2012a). However, where tension field action given by *Specification* Equations 1.2.2-36 and 1.2.2-37 is used to compute V_n, then *flange* distortion of unrestrained *flanges* requires that *distortional buckling* be considered when computing M_{nxo} (Pham and Hancock, 2012a).

APPENDIX 1 REFERENCES

Acharya, V.V. and R.M. Schuster (1998), "Bending Tests of Hat Section With Multiple Longitudinal Stiffeners," *Proceedings of the Fourteenth International Specialty Conference on Cold-Formed Steel Structures*, University of Missouri-Rolla, Rolla, MO, October 1998.

American Institute of Steel Construction (2001), *Manual of Steel Construction: Load and Resistance Factor Design*, 3rd Edition, American Institute of Steel Construction, Chicago, IL, 2001.

American Iron and Steel Institute (2002), *Cold-Formed Steel Design Manual*, American Iron and Steel Institute, Washington, DC, 2002.

Awegan, K. and C. D. Moen (2012), "Critical Elastic Shear Buckling Stress Hand Solution for C- and Z-Sections Including Cross-Section Connectivity," *Proceedings of the Twenty-first International Specialty Conference on Cold-Formed Steel Structures*, Missouri University of Science and Technology, Rolla, MO, October 2012.

Bambach, M.R., J.T. Merrick and G.J. Hancock (1998), "Distortional Buckling Formulae for Thin Walled Channel and Z-Sections With Return Lips," *Proceedings of the fourteenth International Specialty Conference on Cold-Formed Steel Structures*, University of Missouri-Rolla, Rolla, MO, October 1998, pp. 21-38.

Bernard, E.S. (1993), "Flexural Behavior of Cold-Formed Profiled Steel Decking," Ph.D. Thesis, University of Sydney, Australia, 1993.

Casafont, M., M. Pastor, F. Roure, J. Bonada, and T. Peköz (2012), "An Investigation on the Design of Steel Storage Rack Columns via the Direct Strength Method," *Journal of Structural Engineering*, ASCE Vol. 139, 2012.

Chajes, A. (1974), *Principles of Structural Stability*, Prentice Hall College Div, Englewood Cliffs, NJ, 1974.

Cheung, Y.K. and L.G. Tham (1998), *Finite Strip Method*, CRC Press, 1998.

Cohen, J. M. (1987), "Local Buckling Behavior of Plate Elements," Department of Structural Engineering Report, Cornell University, Ithaca, NY, 1987.

Cook, R.D., D.S. Malkus and M.E. Plesha (1989), *Concepts and Applications of Finite Element Analysis,* John Wiley & Sons, Third Edition, 1989.

Davies, J.M. and C. Jiang (1996), "Design of Thin-Walled Beams for Distortional Buckling," *Proceedings of the Thirteenth International Specialty Conference on Cold-Formed Steel Structures*, University of Missouri-Rolla, Rolla, MO, October 1996, pp. 141-154.

Davies, J.M., C. Jiang and V. Ungureanu (1998), "Buckling Mode Interaction in Cold-Formed Steel Columns and Beams," *Proceedings of the Fourteenth International Specialty Conference on Cold-Formed Steel Structures*, University of Missouri-Rolla, Rolla, MO, October 1998, pp. 53-68.

Davies, J.M., P. Leach and D. Heinz (1994), "Second-Order Generalised Beam Theory," *Journal of Constructional Steel Research*, Elsevier, 31 (2-3), pp. 221-242.

Desmond, T.P. (1977), "The Behavior and Design of Thin-Walled Compression Elements with Longitudinal Stiffeners," Ph.D. Thesis, Cornell University, Ithaca, NY, 1977.

Ellifritt, D., B. Glover and J. Hren (1997), "Distortional Buckling of Channels and Zees Not Attached to Sheathing," Report for the American Iron and Steel Institute, Washington, DC, 1997.

El-Sawy, K.M. and A. S. Nazmy (2001), "Effect of Aspect Ratio on the Elastic Buckling of Uniaxially Loaded Plates With Eccentric Holes," *Thin-Walled Structures*, 39(12), pp. 983-998.

Elzein, A. (1991), *Plate Stability by Boundary Element Method*, Springer-Verlag, NY, 1991.

Galambos, T.V. (1998), *Guide to Stability Design Criteria for Metal Structures,* John Wiley & Sons, the Fifth Edition, 1998.

Ganesan, K. and C.D. Moen (2012), "LRFD Resistance Factor for Cold-Formed Steel Compression Members," *Journal of Constructional Steel Research*, 72, pp. 261-266.

Grey, C.N. and C.D. Moen (2011), "Elastic Buckling Simplified Methods for Cold-formed Steel Columns and Beams With Edge-Stiffened Holes," *2011 Annual Technical Session and Meeting, Structural Stability Research Council,* Pittsburgh, PA, 2011.

Hancock, G.J. (1997), "Design for Distortional Buckling of Flexural Members," *Thin-Walled Structures*, 27(1), 3-12, Elsevier Science Ltd, 1997.

Hancock, G.J., Y.B. Kwon and E.S. Bernard (1994), "Strength Design Curves for Thin-Walled Sections Undergoing Distortional Buckling," *Journal of Constructional Steel Research*, Elsevier, 31(2-3), pp. 169-186.

Hancock, G.J., T.M. Murray and D.S. Ellifritt (2001), *Cold-Formed Steel Structures to the AISI Specification,* Marcell-Dekker, New York, NY, 2001.

Hancock, G.J. and C.H. Pham (2011), "A Signature Curve for Cold-Formed Channel Sections in Pure Shear," Research Report R919, University of Sydney, School of Civil Engineering, July 2011.

Hancock, G.J., C.A. Rogers and R.M. Schuster (1996), "Comparison of the Distortional Buckling Method for Flexural Members With Tests," *Proceedings of the Thirteenth International Specialty Conference on Cold-Formed Steel Structures*, University of Missouri-Rolla, Rolla, MO, October 1998, pp. 125-140.

Harik, I.E., X. Liu and R. Ekambaram (1991), "Elastic Stability of Plates With Varying Rigidities," *Computers and Structures*, 38 (2), pp. 161-168.

Höglund, T. (1980), "Design of Trapezoidal Sheeting Provided with Stiffeners in the Flanges and Webs," *Swedish Council for Building Research*, Stockholm, Sweden, D28:1980.

Kawai, T., and H. Ohtsubo (1968), "A Method of Solution for the Complicated Buckling Problems of Elastic Plates with Combined Use of Rayleigh-Ritz's Procedure in the Finite Element Method," *Proceedings of the Second Conference on Matrix Methods in Structural Mechanics*, AFFDL-TR-68-150, Wright-Patterson Air Force Base, OH, pp. 967-994.

König, J. (1978), "Transversally Loaded Thin-Walled C-Shaped Panels With Intermediate Stiffeners," *Swedish Council for Building Research*, Stockholm, Sweden, D7:1978.

Kwon, Y.B. and G.J. Hancock (1992), "Strength Tests of Cold-Formed Channel Sections Undergoing Local and Distortional Buckling," *Journal of Structural Engineering*, ASCE, Vol. 117, No. 2, pp. 1786 – 1803, 1992.

Kumai, T. (1952), "Elastic Stability of the Square Plate With a Central Circular Hole Under Edge Thrust," *Reports of Research Institute for Applied Mechanics*, I(2).

LaBoube, R.A. and W. W. Yu (1978), "Structural Behavior of Beam Webs Subjected to Bending Stress," *Civil Engineering Study Structural Series*, 78-1, Department of Civil Engineering, University of Missouri-Rolla, Rolla, MO, 1978.

Lau, S.C.W. and G.J Hancock (1987), "Distortional Buckling Formulas for Channel Columns," *Journal of Structural Engineering*, ASCE, Vol. 113, No. 5, pp. 1063 – 1078.

Li, Z. and B.W. Schafer (2010), "Buckling Analysis of Cold-Formed Steel Members With General Boundary Conditions Using CUFSM: Conventional and Constrained Finite Strip Methods," *Proceedings of the Twentieth International Specialty Conference on Cold-Formed Steel Structures*, St. Louis, MO, November 2010.

Loughlan, J. (1979), "Mode Interaction in Lipped Channel Columns Under Concentric or Eccentric Loading," Ph.D. Thesis, University of Strathclyde, Glasgow, 1979.

Miller, T.H. and T. Peköz (1994), "Load-Eccentricity Effects on Cold-Formed Steel Lipped-Channel Columns," *Journal of Structural Engineering*, ASCE, Vol. 120, No. 3, pp. 805-823, 1994.

Moen, C.D. and B. W. Schafer (2009a), "Direct Strength Design for Cold-Formed Steel Members with Holes," Final Report, American Iron and Steel Institute, Washington, DC, 2009

Moen, C. D. and B. W. Schafer (2009b), "Elastic Buckling of Thin Plates With Holes in Compression or Bending," *Thin-Walled Structures*, 47(12), pp. 1597-1607, 2009.

Moen, C. D. and B. W. Schafer (2009c), "Elastic Buckling of Cold-Formed Steel Columns and Beams With Holes," *Engineering Structures*, 31(12), pp. 2812-2824, 2009.

Moen, C. D. and B. W. Schafer (2010a), "Direct Strength Design of Cold-Formed Steel Columns With Holes," 2010 Annual Technical Session and Meeting, Structural Stability Research Council, Orlando, FL, 2010.

Moen, C. D. and B. W. Schafer (2010b), "Extending Direct Strength Design to Cold-Formed Steel Beams With Holes," *Proceedings of the Twentieth International Specialty Conference on Cold-Formed Steel Structures*, St. Louis, MO, 2010.

Moen, C. D. and B. W. Schafer (2011), "Direct Strength Method for Design of Cold-Formed Steel Columns with Holes," *ASCE Journal of Structural Engineering,* 137(5), pp. 559-570.

Moreyra, M.E. (1993), "The Behavior of Cold-Formed Lipped Channels Under Bending," M.S. Thesis, Cornell University, Ithaca, NY, 1993.

Mulligan, G.P. (1983), "The Influence of Local Buckling on the Structural Behavior of Singly-Symmetric Cold-Formed Steel Columns," Ph.D. Thesis, Cornell University, Ithaca, NY, 1983.

Papazian, R.P., R.M. Schuster and M. Sommerstein (1994), "Multiple Stiffened Deck Profiles," *Proceedings of the Twelfth International Specialty Conference on Cold-Formed Steel Structures*, University of Missouri-Rolla, Rolla, MO, October, 1994, pp. 217-228.

Pham, C. H. and G. J. Hancock (2009a), "Shear Buckling of Thin-Walled Channel Sections," *Journal of Constructional Steel Research*, Volume 65, No 3, pp. 578-585, 2009.

Pham, C. H. and G. J. Hancock (2009b), "Direct Strength Design of Cold-Formed Purlins," *Journal of Structural Engineering*, American Society of Civil Engineers, Volume 135, Issue 3, pp. 229-238, 2009.

Pham, C.H. and Hancock, G.J. (2011), "Elastic Buckling of Cold-Formed Channel Sections in Shear," *Proceedings of the International Conference on Thin-Walled Structures*, Timisoara, Romania, September 2011, pp. 205-212.

Pham, C. H. and G. J. Hancock (2012a), "Direct Strength Design of Cold-Formed C-Sections for Shear and Combined Actions," *Journal of Structural Engineering*, American Society of Civil Engineers, Volume 138, No. 6, 2012.

Pham, C.H. and Hancock, G.J. (2012b), "Tension Field Action for Cold-Formed Channel Sections in Shear", *Journal of Constructional Steel Research*, Vol. 72, pp. 168-178, 2012.

Phung, N. and W.W. Yu (1978), "Structural Behavior of Longitudinally Reinforced Beam Webs," *Civil Engineering Study Structural Series*, Department of Civil Engineering, 78-6, University of Missouri-Rolla, MO, 1978.

Plank, R.J. and W.H. Wittrick (1974), "Buckling Under Combined Loading of Thin, Flat-Walled Structures by a Complex Finite Strip Method," *International Journal for Numerical Methods in Engineering*, Vol. 8, No. 2, pp. 323 - 329.

Polyzois, D. and P. Charnvarnichborikarn (1993), "Web-Flange Interaction in Cold-Formed Steel Z-Section Columns," *Journal of Structural Engineering*, ASCE, Vol. 119, No. 9, pp. 2607-2628.

Quispe, L. and G.J. Hancock (2002), "Direct Strength Method for the Design of Purlins," *Proceedings of the Sixteenth International Specialty Conference on Cold-Formed Steel Structures*, University of Missouri-Rolla, Rolla, MO, October 2002, pp. 561-572.

Rogers, C.A. (1995), "Interaction Buckling of Flange, Edge Stiffener and Web of C-Sections in Bending," M.S. Thesis, University of Waterloo, Ontario, Canada, 1995.

Sarawit, A. (2003). "Cold-Formed Steel Frame and Beam-Column Design," Ph.D. Thesis, Cornell University, Ithaca, 2003.

Schafer, B.W. (1997), "Cold-Formed Steel Behavior and Design: Analytical and Numerical Modeling of Elements and Members With Longitudinal Stiffeners," Ph.D. Thesis, Cornell University, Ithaca, NY, 1997.

Schafer, B.W. (2000), "Distortional Buckling of Cold-Formed Steel Columns: Final Report," Sponsored by the American Iron and Steel Institute, Washington, DC, 2000.

Schafer, B.W. (2001), "Progress Report 2: Test Verification of the Effect of Stress Gradient on Webs of Cee and Zee Sections," Submitted to the AISI and MBMA, July 2001.

Schafer, B.W. (2002), "Local, Distortional, and Euler Buckling in Thin-Walled Columns," *Journal of Structural Engineering*, ASCE, Vol. 128, No. 3, pp. 289-299.

Schafer, B.W. (2002b), "Progress on the Direct Strength Method," *Proceedings of the Sixteenth International Specialty Conference on Cold-Formed Steel Structures*, Orlando, FL, pp. 647-662.

Schafer, B.W. and S. Ádány (2006), "Buckling Analysis of Cold-Formed Steel Members Using CUFSM: Conventional and Constrained Finite Strip Methods," *Proceeding of Eighteenth International Specialty Conference on Cold-Formed Steel Structures*, Orlando, FL, 2006.

Schafer, B.W., A. Sarawit and T. Peköz (2006), "Complex Edge Stiffeners for Thin-Walled Members," *Journal of Structural Engineering, ASCE*, Vol. 132, No. 2, pp. 212-226, 2006.

Schafer, B.W. and T. Peköz (1998), "Direct Strength Prediction of Cold-Formed Steel Members Using Numerical Elastic Buckling Solutions," *Proceedings of the Fourteenth International Specialty Conference on Cold-Formed Steel Structures,* University of Missouri-Rolla, Rolla, MO, October 1998.

Schafer, B.W. and T. Peköz (1999), "Laterally Braced Cold-Formed Steel Flexural Members With Edge Stiffened Flanges," *Journal of Structural Engineering,* ASCE, Vol. 125, No. 2, 1999.

Schardt, R. (1989), *Verallgemeinerte Technische Biegetheorie [Generalized Beam Theory]*, Springer-Verlag, Berlin.

Schardt, R. and W. Schrade (1982), "Kaltprofil-Pfetten," Institut Für Statik, Technische Hochschule Darmstadt, Bericht Nr. 1, Darmstadt.

Schlack Jr., A.L. (1964), "Elastic Stability of Pierced Square Plates," *Experimental Mechanics*, 4(6), pp. 167-172, 1964.

Schuster, R.M. (1992), "Testing of Perforated C-Stud Sections in Bending," Report for the Canadian Sheet Steel Building Institute, University of Waterloo, Waterloo Ontario, Canada, 1992.

Shan, M., R.A. LaBoube and W. W. Yu (1994), "Behavior of Web Elements with Openings Subjected to Bending, Shear and the Combination of Bending and Shear," *Civil Engineering Study Structural Series*, 94-2, Department of Civil Engineering, University of Missouri-Rolla, Rolla, MO, 1994.

Shifferaw, Y. and B. W. Schafer (2010), "Inelastic Bending Capacity in Cold-Formed Steel Members," Submitted to ASCE *Journal of Structural Engineering*, Vol. 138, No. 4, pp. 468-480, 2012.

Silvestre, N. and D. Camotim (2002a), "First-Order Generalised Beam Theory for Arbitrary Orthotropic Materials," *Thin-Walled Structures*, Elsevier, Vol. 40, pp. 755-789.

Silvestre, N. and D. Camotim (2002b), "Second-Order Generalised Beam Theory for Arbitrary Orthotropic Materials," *Thin-Walled Structures*, Elsevier, Vol. 40, pp. 791-820.

Thomasson, P. (1978), "Thin-Walled C-Shaped Panels in Axial Compression," *Swedish Council for Building Research,* D1:1978, Stockholm, Sweden.

Timoshenko, S.P. and J. M. Gere (1961), *Theory of Elastic Stability*, McGraw-Hill, NY, 1961.

Willis, C.T. and B. Wallace (1990), "Behavior of Cold-Formed Steel Purlins Under Gravity Loading," *Journal of Structural Engineering*, ASCE, Vol. 116, No. 8, 1990.

Yu, W.W. (2000), *Cold-Formed Steel Design*, John Wiley & Sons, Inc., 2000.

Zienkiewicz, O.C. and R.L. Taylor (1989), *The Finite Element Method: Volume 1 Basic Formulations and Linear Problems*, McGraw Hill, Fourth Edition, 1989.

Zienkiewicz, O.C. and R.L. Taylor (1991), *The Finite Element Method: Volume 2 Solid and Fluid Mechanics Dynamics and Non-Linearity*, McGraw-Hill, Fourth Edition, 1991.

Appendix 2:

Commentary on Appendix 2

Second-Order Analysis

2012 EDITION

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APPENDIX 2: COMMENTARY ON APPENDIX 2 SECOND-ORDER ANALYSIS

The provisions of this Appendix are based on Sarawit (2003), Sarawit and Peköz (2006) and AISC (2005). The provisions here are supported by an extensive study on Industrial Steel Storage Racks (2006) sponsored at Cornell University by the Rack Manufacturers Institute and the American Iron and Steel Institute. The subject of *notional loads* is discussed fully in the commentary to Appendix 7 of AISC (2005). The application to cold-formed steel structures has to include the frequently encountered *flexural-torsional buckling*, semi-rigid *joints* and local *instabilities*. In Sarawit (2003) and Sarawit and Peköz (2006), it is shown that the *second-order analysis* gives more accurate results than the effective length approach.

2.1 General Requirements

Required strengths [effect of *factored loads*] are determined by analysis according to *Specification* Appendix 2 and the members have to satisfy the provisions of Section C5 of the main body of the *Specification*. In checking the strength by *Specification* Section C5, magnification of the moments does not need to be included since the *second-order analysis* gives the magnified moments.

Since the frame stability is considered by the *second-order analysis*, *nominal axial strength* [*resistance*] in *Specification* Section C5.2 should be determined with an effective length coefficient equal to 1.0.

2.2 Design and Analysis Constraints

Second-order frame *analysis* is permitted either on the out-of-plumb geometry without *notional loads* or on the plumb geometry by applying *notional loads* or minimum lateral *loads* as defined in *Specification* Appendix 2. If *second-order* elastic *analysis* is used, namely inelasticity effects are not modeled explicitly; axial and flexural stiffnesses are to be reduced as specified in *Specification* Appendix 2.

It is required to carry out a *second-order analysis* that considers both the effect of *loads* acting on the deflected shape of a member between *joints* or nodes (*P-δ effects*) and the effect of *loads* acting on the displaced location of *joints* or nodes in a structure (*P-Δ effects*). On a member level, *P-δ effects* need to be modeled explicitly. Adding a node or nodes along the length of the member will suffice. These intermediate nodes do not need to account for the initial out-of-straightness for the member. This is because for members, the design equations used include the presence of δ imperfection and thus member strength is already calibrated to include the effect of P-δ.

The 20 percent reduction in member stiffness EI, namely multiplying EI by 0.8, that is used in the AISC Specification (2005) is applied only to E for convenience in analysis. The reasoning for the 20 percent reduction in EI as well as the inelastic *buckling* factor τ_b is provided in the commentary to the AISC Specification (2005). Part of the justification for 20 percent reduction in member stiffness is based on a *resistance factor* of 0.9 used in the AISC Specification (AISC, 2005) for columns. However, in the AISI *Specification*, the *resistance factor* is less than 0.9. For this reduced *resistance factor*, the adequacy of 20 percent reduction in member stiffness for cold-formed steel frames can be deduced from the studies described in Sarawit and Peköz (2006), which is based on Sarawit (2003). Sarawit and Peköz (2006) show that for typical industrial storage rack frames with a wide variety of section properties, configurations, and behavior modes, a reduction of 10 percent in member stiffnesses results in an increased conservatism of 10 percent in the calculated *load*-carrying capacity. A 20 percent reduction in member stiffnesses would lead to an increased conservatism of 20 percent in the calculated *load*-carrying capacity. A parametric study of individual columns in Sarawit and Peköz (2006) shows that some unconservative results can be obtained in a few instances if the stiffness of members is not reduced in the analysis. Reducing the stiffness by 20 percent gives satisfactory results for these cases.

It should be noted that the *nominal axial* and *flexural strengths* [*resistances*] used in the interaction equations of Section C5.2 do not need to be calculated based on reduced value of E.

APPENDIX 2 REFERENCES

American Institute of Steel Construction (2005), *Specification for Structural Steel Buildings*, March 9, 2005.

Sarawit, A. (2003), "Cold-Formed Steel Frame and Beam-Column Design," Ph.D. Thesis, and Research Report 03-03, Department of Civil and Environmental Engineering, Cornell University, Ithaca, New York, March 2003.

Sarawit, A. and T. Peköz (2006), "Notional Load Method for Industrial Steel Storage Racks," *Thin-Walled Structures*, Elserier, Vol. 44, No. 12, December 2006.

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Appendix A:

Commentary on Provisions Applicable to the United States and Mexico

2012 EDITION

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APPENDIX A: COMMENTARY ON PROVISIONS APPLICABLE TO THE UNITED STATES AND MEXICO

This commentary on Appendix A provides a record of reasoning behind, and justification for, provisions that are applicable to the United States and Mexico. The format used herein is consistent with that used in Appendix A of the *Specification*.

A1.1a Scope

In the 2007 edition of the *Specification*, both the *Allowable Strength Design* and the *Load and Resistance Factor Design* are permitted to be used in a design.

A2.2 Other Steels

Although the use of ASTM-designated steels listed in *Specification* Section A2.1 is encouraged, other steels may also be used in cold-formed steel structures, provided they satisfy the requirements stipulated in *Specification* Section A2.2.

In 2004, these requirements were clarified and revised. The *Specification* has long required that such "other steels" conform to the chemical and mechanical requirements of one of the listed specifications or "other *published specification*." Specific requirements for a *published specification* have been detailed in the definitions under General Terms, A1.3. It is important to note that, by this definition, published requirements must be established before the steel is ordered, not by a post-order screening process. The requirements must include minimum tensile properties, chemical composition limits, and for coated sheets, coating properties. Testing procedures must be in accordance with the referenced ASTM specifications. A proprietary specification of a manufacturer, purchaser, or producer could qualify as a *published specification* if it meets the definition requirements.

As an example of these *Specification* provisions, it would not be permissible to establish a minimum *yield stress* or minimum *tensile strength* greater than that ordered to a standard ASTM grade by reviewing mill test reports or conducting additional tests. However, it would be permissible to publish a manufacturer's or producer's specification before the steel is ordered requiring that such enhanced properties be furnished as a minimum. Testing to verify that the minimum properties are achieved could be done by the manufacturer or the producer. The intent of these provisions is to ensure that the material factor M_m (see Chapter F) will be maintained at about 1.10, corresponding to an assumed typical 10 percent overrun in tensile properties for ASTM grades.

Special additional requirements have been added to qualify unidentified material. In such a case, the manufacturer must run tensile tests sufficient to establish that the *yield stress* and *tensile strength* of each *master coil* are at least 10 percent greater than the applicable *published specification*. As used here, *master coil* refers to the coil being processed by the manufacturer. Of course, the testing must always be adequate to ensure that specified minimum properties are met, as well as the ductility requirements of *Specification* Section A2.3.1, A2.3.2, A2.3.3, or A2.3.4 as desired.

Where the material is used for fabrication by welding, care must be exercised in selection of chemical composition or mechanical properties to ensure compatibility with the welding process and its potential effect on altering the tensile properties.

A2.3.5a Ductility Requirements of Other Steels

The low-ductility steel application is limited for *curtain wall stud* application in heavy weight exterior walls in areas with Seismic Design Categories D, E and F.

A3 Loads

A3.1 Nominal Loads

The *Specification* does not establish the dead, live, snow, wind, earthquake or other loading requirements for which a structure should be designed. These *loads* are typically covered by the *applicable building code*. Otherwise, the American Society of Civil Engineers Standard, ASCE/SEI 7 (ASCE, 2005) should be used as the basis for design.

Recognized engineering procedures should be employed to reflect the effect of impact *loads* on a structure. For building design, reference may be made to AISC publications (AISC, 1989, 1999, 2005, and 2010).

When gravity and lateral *loads* produce forces of opposite sign in members, consideration should be given to the minimum gravity *loads* acting in combination with wind or earthquake *loads*.

A4.1.2 Load Combinations for ASD

In 2001, the *Specification* was revised to specify that all *loads* and *load* combinations were required to follow the *applicable building code*. In the absence of an *applicable building code*, *loads* and *load* combinations should be determined according to the American Society of Civil Engineers Standard, *Minimum Design Loads for Buildings and Other Structures*, ASCE/SEI 7, with the edition adopted in *Specification* Section A9a.

When steel decks are used for roof and floor composite construction, steel decks should be designed to carry the concrete dead *load*, the steel dead *load*, and the construction live *load*. The construction *load* is based on the sequential loading of concrete as specified in the ANSI/ASCE Standard 3-91 (ASCE, 1991) and in the *Design Manual* of Steel Deck Institute (SDI, 2006).

A5.1.2 Load Factors and Load Combinations for LRFD

In 2001, the *Specification* was revised to specify that all *loads* and *load* combinations were required to follow the *applicable building code*. In the absence of an *applicable building code*, *loads* and *load* combinations should be determined according to the American Society of Civil Engineers Standard, *Minimum Design Loads for Buildings and Other Structures*, ASCE/SEI 7 (ASCE, 2010).

In view of the fact that building codes and ASCE/SEI 7 do not provide *load factors* and *load* combinations for roof and floor composite construction using cold-formed steel deck, the following *load* combination may be used for this type of composite construction:

$1.2D_S + 1.6C_W + 1.4C$

where

D_s = weight of steel deck

C_W = weight of wet concrete during construction

C = construction *load*, including equipment, workmen and formwork, but excluding the weight of the wet concrete.

The above *load* combination provides safety construction practices for cold-formed steel decks and panels which otherwise may be damaged during construction. The *load factor* used for the weight of wet concrete is 1.6 because of delivery methods and an individual sheet can be subjected to this *load*. The use of a *load factor* of 1.4 for the construction *load* is comparable to the *Allowable Strength Design* method.

D6.1.2 Flexural Members Having One Flange Fastened to a Standing Seam Roof System

For beams supporting a standing seam roof system, e.g. a roof *purlin* subjected to dead plus live *load*, or uplift from wind *load*, the bending capacity is greater than the bending strength of an unbraced member and may be equal to the bending strength of a fully braced member. The bending capacity is governed by the nature of the loading, gravity or uplift, and the nature of the particular standing seam roof system. Due to the availability of many different types of standing seam roof systems, an analytical method for determining positive and negative bending capacities has not been developed at the present time. However, in order to resolve this issue relative to the gravity loading condition, Section D6.1.2 was added in the 1996 edition of the AISI *Specification* for determining the *nominal flexural strength* [*resistance*] of beams having one *flange* fastened to a standing seam roof system. In *Specification* Equation D6.1.2-1, the reduction factor, R, can be determined by AISI S908 published by AISI (AISI, 2013f). Application of the base test method for uplift loading was subsequently validated after further analysis of the research results.

D6.1.4 Compression of Z-Section Members Having One Flange Fastened to a Standing Seam Roof

The strength of axially loaded Z-sections having one *flange* attached to a standing seam roof may be limited by either a combination of *torsional buckling* and lateral *buckling* in the plane of the roof, or by *flexural buckling* in a plane perpendicular to the roof. As in the case of Z-sections carrying gravity or wind *loads* as beams, the roof diaphragm and *purlin* clips provide a degree of torsional and lateral bracing restraint that is significant, but not necessarily sufficient, to develop the full strength of the cross-section.

Specification Equation D6.1.4-1 predicts the lateral *buckling* strength using an ultimate axial *buckling stress* ($k_{af}RF_y$) that is a percentage of the ultimate flexural *stress* (RF_y) determined from uplift tests performed using AISI S908, *Base Test Method for Purlins Supporting a Standing Seam Roof System*, as published by AISI (2013f). This equation, developed by Stolarczyk, el al. (2002), was derived empirically from elastic finite element *buckling* studies and calibrated to the results of a series of tests comparing flexural and axial strengths using the uplift "Base Test" setup. The *full unreduced cross-sectional area*, A, has been used rather than the *effective area*, A_e, because the ultimate axial *stress* is generally not large enough to result in a significant reduction in the *effective area* for common cross-section geometries.

Specification Equation D6.1.4-1 may be used with the results of uplift "Base Tests" conducted with and without discrete point bracing. There is no limitation on the minimum length because Equation D6.1.4-1 is conservative for spans that are smaller than those tested under the "Base Test" provisions.

The strength of longer members may be governed by *axial buckling* perpendicular to the roof; consequently, the provisions of *Specification* Sections C4.1 and C4.1.1 should also be checked for *buckling* about the strong axis.

D6.2.1a Strength [Resistance] of Standing Seam Roof Panel Systems

The introduction of the wind uplift loading *required strength* factor of 0.67 was a result of research conducted to correlate the static uplift capacity represented by tests performed in accordance with S906 (AISI, 2013d) and the dynamic behavior of real wind, by Surry et al. (2007). This research utilized two separate methods of comparison. The first method utilized full-scale tests conducted at Mississippi State University (MSU) using simulated wind l*oads* on a portion of a standing seam metal roof. The second method utilized model-scale wind tunnel tests carried out at the University of Western Ontario of an aeroelastic "failure" model of the same roof system. In spite of these significantly different approaches, the results obtained were very consistent. It was found that the ASTM E1592 uniform pressure test contains conservatism of about 50 percent for the roof system tested by both approaches, and up to about 80 percent for the other roof systems tested only at MSU. This conservatism arises if the roof system is required to withstand the code-recommended pressure applied as uniform pressure in the ASTM E1592 test, without accounting for the reality of the dynamic spatially-varying properties of the wind-induced pressures. The limits of applicability of this factor (panel *thickness* and width) are conservatively listed based on the scope of the research. The failure mode is restricted to those failures associated with the *load* in the clip because this was how the research measured and compared the static and dynamic capacities. Therefore, the 2012 *Specification* was clarified with respect to the strength factor of 0.67 applying to the clips and fasteners as well as the standing seam roof panels. The required strength factor of 0.67 is not permitted to be used with other observed failures. In addition, the research does not support or confirm whether interpolation would be appropriate between ASTM E1592 tests of the same roof system with different spans, where one test meets the requirements, such as a clip failure, and another test does not, such as a panel failure.

It was determined that the strength factor, 0.67, when applied to the corner and edge zones of steeper slope roofs (greater than 27-degree slope) could yield a nominal wind *load* less than that in the field of the roof, based on ASCE 7 (2010). So, the limiting value of the wind *load* in the field of the roof was introduced in the 2012 *Specification*.

An AISI interpretation was issued in 2012 that clarified that the strength factor, 0.67, that was based on research that compared the static and dynamic capacities of these types of roof systems is justified to be used with the *loads* or *load* combinations in the International Building Code (IBC), since this strength factor is based on structural behavior caused by rate or duration of *load*. Therefore, this 0.67 factor is not duplicative of the consideration given for multiple *variable loads* in both the strength design *load* combinations and the allowable *stress load* combinations used in IBC and ASCE 7 (ASCE, 2010). It would be appropriate to utilize the 0.67 factor on the *nominal* wind *load* for any *load* combination that includes wind uplift as long as all of the conditions stated in *Specification* Section D6.2.1a (Appendix A) are met.

It is recognized that there are other analytical tools available, especially advanced finite element analyses, that have made strides in replicating the behavior of standing seam roof systems and determining their dynamic uplift capacity. Therefore, alternative means of analysis may be available to compare the dynamic and static behavior that could be used to extend the applicability of this method, provided it was sufficiently calibrated to the existing test data. Any alternative method should also comply with the *rational engineering analysis* requirements of Section A1.2, including the appropriate *safety factor* and *resistance factor* for members and connections.

E2a Welded Connections

The upper limit of the *Specification* applicability was revised in 2004 from 0.18 in. (4.57 mm) to 3/16 in. (4.76 mm). This change was made to be consistent with the limit given in the AWS D1.3 (1998).

The design provisions for welded *connections* were developed based primarily on experimental evidence obtained from an extensive test program conducted at Cornell University. In addition, the Cornell research provided the experimental basis for the AWS *Structural Welding Code for Sheet Steel* (AWS, 1998). In most cases, the provisions of the AWS code are in agreement with this *Specification* section.

The terms used in this *Specification* section agree with the standard nomenclature given in the AWS *Welding Structural Code for Sheet Steel* (AWS, 1998).

For welded material *thicknesses* greater than 3/16 in. (4.76 mm), AISC Specification (2010) should be followed.

E3.4 Shear and Tension in Bolts

For the design of bolted *connections*, the allowable shear *stresses* for bolts have been provided in the AISI *Specification* for cold-formed steel design since 1956. However, the allowable tension *stresses* were not provided in *Specification* Section E3.4 for bolts subjected to tension until 1986. In *Specification* Table E3.4-1, the allowable *stresses* specified for A307 (d ≥ 1/2 inch (12.7 mm)), A325, and A490 bolts were based on Section 1.5.2.1 of the AISC Specification (AISC, 1978). It should be noted that the same values were also used in Table J3.2 of the AISC *ASD* Specification (AISC, 1989). For A307, A449, and A354 bolts with diameters less than 1/2 inch (12.7 mm), the allowable tension *stresses* were reduced by 10 percent, as compared with these bolts having diameters not less than 1/2 inch (12.7 mm), because the average ratio of (tensile-*stress* area)/(gross-area) for 1/4-inch (6.35 mm) and 3/8-inch (9.53 mm) diameter bolts is 0.68, which is about 10 percent less than the average area ratio of 0.75 for 1/2-inch (12.7 mm) and 1-inch (25.4 mm) diameter bolts. In the AISI *ASD/LRFD Specification* (AISI, 1996), Table E3.4-1 provided *nominal tensile strengths* [*resistance*] for various types of bolts with applicable *safety factors*. The allowable tension *stresses* computed from F_{nt}/Ω were approximately the same as those permitted by the AISI 1986 *ASD Specification*. The same table also gave the *resistance factor* to be used for the *LRFD* method. In 2012, the table values were realigned with the AISC Specification (AISC, 2010).

The design provisions for bolts subjected to a combination of shear and tension were added in AISI *Specification* Section E3.4 in 1986. Those design equations were based on Section 1.6.3 of the AISC Specification (AISC, 1978) for the design of bolts used for bearing-type *connections*.

In 1996, tables which listed the equations for determining the reduced nominal tension *stress*, F'_{nt}, for bolts subjected to the combination of shear and tension were included in the

Specification and were retained in the 2001 edition. In 2007, those tables were replaced by *Specification* Equations E3.4-2 and E3.4-3 to determine the reduced tension *stress* of bolts subjected to the combined tension and shear. *Specification* Equations E3.4-2 and E3.4-3 were adopted to be consistent with the AISC Specification (AISC, 2005).

Note that when the required *stress*, f, in either shear or tension, is less than or equal to 20 percent of the corresponding available *stress*, the effects of combined *stress* need not be investigated.

For bolted connection design, the possibility of pull-over of the connected sheet at the bolt head, nut, or washer should also be considered when bolt tension is involved, especially for thin sheathing material. For unsymmetrical sections, such as C- and Z-sections used as *purlins* or *girts*, the problem is more severe because of the prying action resulting from rotation of the member which occurs as a consequence of loading normal to the sheathing. The designer should refer to applicable product code approvals, product specifications, other literature, or tests.

For design tables and example problems on bolted connections, see Part IV of the *Design Manual* (AISI, 2013).

Appendix B:

Commentary on Provisions Applicable to Canada

2012 EDITION

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APPENDIX B: COMMENTARY ON PROVISIONS APPLICABLE TO CANADA

This commentary on Appendix B of the *Specification* provides a record of reasoning behind, and justification for, provisions that are applicable only to Canada. Only those sections of Appendix B of the *Specification* are addressed herein or where additional commentary is required beyond what is already contained in the *Commentary on the 2013 Edition of the North American Specification for the Design of Cold-Formed Steel Structural Members* (hereinafter referred to as the *Commentary*). The format used herein is consistent with that used in Appendix B of the *Specification*.

In comparison to Appendix B of the 2007 edition of CSA Standard S136, a few changes have been incorporated into this *Specification*. The most significant ones are as follows:

a) The entire Section A3 on *loads* has been updated in accordance with the 2010 Edition of the *National Building Code of Canada.*
b) The referenced standards in *Specification* Section A9a are updated.
c) *Specification* Section C2, Tension Members, has been moved to the main body of the *Specification*.
d) *Specification* Sections E2.2a and E2.3a have been incorporated into the main body of the *Specification*.
e) Provisions related to rupture of net section have been incorporated into the main body of the *Specification*.

A2.1.1a Applicable Steels

CSA Standard G40.20/G40.21 is referenced because it is widely used in Canada for structural quality bars and plate.

A2.2.2 Other Steels

Provisions are included for determining the mechanical properties of unidentified structural steels.

A2.3.5a Ductility Requirements of Other Steels

The use of low-ductility steel has been limited to *curtain wall stud* applications in specific low seismic areas.

A3 Loads

The *load* provisions contained in Appendix B of CSA S136-07 were changed to be compatible with the changes that are incorporated in Part 4 of the *National Building Code of Canada* (NBC) 2010. This entails the following:

(1) The version of Limit States Design in NBC 2010 is based on the companion action format, which is being adopted worldwide and is a more rational method of combining *loads* than the previous version.
(2) NBC 1995 distinguished wind *load* for different categories of buildings using a return period approach, an increase in design *loads* for earthquake based on building use by means of an

importance factor, and made no allowance for different snow *loads* based on the occupancy of the structure. In NBC 2010, it was decided to harmonize the approach used, and so the importance factor methodology was chosen for snow, wind and earthquake *loads*.

A6 Limit States Design

In *Limit States Design*, the resistance of a *structural component* is checked against the various *limit states*. For the ultimate *limit states* resistance, the structural member must retain its *load*-carrying capacity up to the *factored load* levels. For *serviceability limit states*, the performance of the structure must be satisfactory at *specified load* levels. *Specified loads* are those prescribed by the *National Building Code of Canada*. Examples of serviceability requirements include deflections and the possibility of vibrations.

Section A6 of the *Specification* sets forth the fundamental safety criterion that must be met, namely:

Factored resistance ≥ effect of *factored loads*

The *factored resistance* is given by the product ϕR_n, where ϕ is the *resistance factor* which is applied to the *nominal* member *resistance*, R_n. The *resistance factor* is intended to take into account the fact that the resistance of the member may be less than anticipated, due to variability of the material properties, dimensions, and workmanship, and also to take into account the type of failure and uncertainty in the prediction of the resistance.

The *resistance factor* does not, however, cover gross human errors. Human errors cause most structural failures and typically these human errors are "gross" errors. Gross errors are completely unpredictable and are not covered by the overall *safety factor* inherent in buildings.

In *limit states design*, structural reliability is specified in terms of a safety index, β, determined through a statistical analysis of the *loads* and resistances. The safety index is directly related to the structural reliability of the design; hence, increasing β increases the reliability, and decreasing β decreases the reliability. The safety index, β, is also directly related to the *load* and *resistance factors* used in the design.

The *National Building Code of Canada* defines a set of *load factors*, *load* combination factors, and specified minimum *loads* to be used in the design, hence fixing the position of the nominal *load* distribution and the *factored load* distribution. The design Standard is then obligated to specify the appropriate resistance function.

Those responsible for writing a design *Standard* are given the *load* distribution and *load factors*, and must calibrate the *resistance factors*, ϕ, such that the safety index, β, reaches a certain target value. The technical committee responsible for CSA Standard S136 elected to use a target safety index of 3.0 for members and 4.0 for *connections*.

In order to determine the loading for calibration, it was assumed that 80% of cold-formed steel is used in panel form (e.g., roof or floor deck, wall panels, etc.) and the remaining 20% for structural sections (*purlins*, *girts*, studs, etc.). An effective *load factor* was arrived at by assuming live-to-dead *load* ratios and their relative frequencies of occurrence.

Probabilistic studies show that consistent probabilities of failure are determined for all live-to-dead *load* ratios when a live *load factor* of 1.50 and a dead *load factor* of 1.25 are used.

A6.1.2 Load Factors and Load Combinations for LSD

The load factors and the load combinations provided in the *Specification* are from Division B, Part 4, Structural Design of the *National Building Code of Canada* (NBCC). Refer to the Structural Commentaries (Part 4 of Division B) for more information.

D3a Lateral and Stability Bracing

The provisions of this section cover members loaded in the plane of the *web*. Conditions may occur that cause a lateral component of the *load* to be transferred through the bracing member to supporting structural members. In such a case, these lateral forces shall be additive to the requirements of this section. The provisions in the *Specification* recognize the distinctly different behavior of the members to be braced, as defined in Section D3.1 and D3.2 of this Appendix. The term "discrete braces" is used to identify those braces that are only connected to the member to be braced for this express purpose.

D3.1a Symmetrical Beams and Columns

D3.1.1 Discrete Bracing for Beams

This section was revised to retain the 2% requirement for the compressive force in the compressive *flange* of a flexural member at the braced location only. The discrete bracing provisions for columns are provided in Section D3.3.

D3.2a C-Section and Z-Section Beams

This section covers bracing requirements of channel and Z-sections and any other section in which the applied *load* in the plane of the *web* induces twist.

D3.2.2 Discrete Bracing

This section provides for brace intervals to prevent the member from rotating about the shear centre for channels or from rotating about the point of symmetry for Z-sections. The spacing must be such that any *stresses* due to the rotation tendency are small enough so that they will not significantly reduce the *load*-carrying capacity of the member. The rotation must also be small enough (in the order of 2°) to be not objectionable as a service requirement.

Based on tests and the study by Winter et al. (1949b), it was found that these requirements are satisfied for any type of *load* if braces are provided at intervals of one-quarter of the span, with the exception of concentrated *loads* requiring braces near the point of application.

Fewer brace points may be used if it can be shown to be acceptable by rational analysis or testing in accordance with Chapter F of the *Specification*, recognizing the variety of conditions, including the case where *loads* are applied out of the plane of the *web*.

For sections used as *purlins* with a standing seam roof, the number of braces per bay is often determined by rational analysis and/or testing. The requirement for a minimum number of braces per bay is to recognize that predictability of the lateral support and rotational restraint is limited on account of the many variables such as fasteners, insulation, friction coefficients, and distortion of roof panels under *load*.

D3.2.3 One Flange Braced by Deck, Slab, or Sheathing

Forces generated by the tendency for lateral movement and/or twist of the beams, whether cumulative or not, must be transferred to a sufficiently stiff part of the framing system. There are several ways in which this transfer may be accomplished:

(a) by the deck, slab, or sheathing providing a rigid diaphragm capable of transferring the forces to the supporting structure;

(b) by arranging equally loaded pairs of members facing each other;

(c) by direct axial force in the covering material that can be transferred to the supporting structure or balanced by opposing forces;

(d) by a system of sag members such as rods, angles, or channels that transfer the forces to the supporting structure; or

(e) by any other method that designers may select to transfer forces to the supporting structure.

For all types of single *web* beams, the *flange* that is not attached to the deck or sheathing material may be subject to compressive *stresses* under certain loading arrangements, such as beams continuous over supports or under wind *load*. The elastic lateral support to this *flange* provided through the *web* may allow an increase in limit *stress* over that calculated by assuming that the compressive *flange* is a column, with pinned ends at points of lateral bracing. Research indicates that the compressive limit *stress* is also sensitive to the rotational flexibility of the joint between the beam and the deck or sheathing material.

This section is intended to apply even when the *flange* that is not attached to the sheathing material is in tension.

E2a Welded Connections

The section has been revised and expanded and replaces Clause 7.2 of CSA Standard S136-94. See *Commentary* for detailed information. Both fabricators and erectors must be certified under CSA Standard W47.1 for arc welding and CSA Standard W55.3 for resistance welding. This provision extends the certification requirements to the welding of cold-formed members or components to other construction, e.g., welding steel deck to structural steel framing.

E3 Bolted Connections

E3.3 Bearing

Improvements have been made to this section in comparison to Clause 7.3.5.1 of CSA Standard S136-94. Section E3.3.2 has been added, giving consideration to bolt hole deformation. See *Commentary* for detailed information.

Research findings in Yu and Xu (2010) regarding the use of oversized holes and short-slotted holes are adopted for situations where the bolt hole deformation is not a concern in design.

E6a Rupture

As can be observed in Table E6-1, there is a difference in *resistance factors* between *LSD* and *LRFD*. In Canada, rupture has been traditionally assumed to be a member type failure and not a connection type. Therefore, the *resistance factor* in *Specification* Table E6-1 is the same regardless of the type of connector and is consistent with rupture of the net section provisions in *Specification* Section C2.2. In the U.S. and Mexico, rupture in *Specification* Table E6-1 is treated as a *connection* type of failure with the resulting lower *resistance factors*.