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Supplement 2004 to the North American Specification for the Design of Cold-Formed Steel Structural Members

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- **ANSI Approved ANS (AISI/COS/NASPEC-SUP04) Supplement 2004 to the North American Specification for the Design of Cold-Formed Steel Structural Members, 2001 Edition**
- **Supplement 2004 to the Commentary on the North American Specification for the Design of Cold-Formed Steel Structural Members, 2001 Edition**
- **Appendix 1, Design of Cold-Formed Steel Structural Members Using Direct Strength Method**
- **Commentary on Appendix 1, Design of Cold-Formed Steel Structural Members Using Direct Strength Method**

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Supplement 2004 to the North American Specification for the Design of Cold-Formed Steel Structural Members

2001 EDITION

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CSA S136-01

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CANADIAN STANDARDS ASSOCIATION

CANACERO

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

A1.1 Scope and Limits of Applicability

- Change the second paragraph (including the bullets) as the follows:

  This *Specification* includes Symbols and Definitions, Chapters A through G, Appendices A through C, and Appendix 1, which shall apply as follows:
  - Appendix A shall apply only in the United States,
  - Appendix B shall apply only in Canada,
  - Appendix C shall apply only in Mexico, and
  - Appendix 1 provides alternative design provisions for several sections of Chapter C.

- In the fourth paragraph, revise the ending of the first sentence to "….Chapters B through G, Appendixes A through C, and Appendix 1 of the *Specification*."
- Add the following under item (b) and below the tables for factors of safety and the resistance factors:

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

A1.2 Terms

Add the following definitions under the "General Terms" on page 34:

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

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

A2.1 Applicable Steels

- Revise the last sentence in the first paragraph of the section to "…for sheet material as SS or, in the case of high-strength low-alloy steels, as HSLAS or HSLAS-F steels."
- Revise the list of ASTM A1003/A1003M as follows:

ASTM A1003/A1003M (ST Grades 50 (340) H, 40 (275) H, 37 (255) H, 33 (230) H), Steel Sheet, Carbon, Metallic- and Nonmetallic-Coated for Cold-Formed Framing Members

- Revise the metric value for Grade 60 of ASTM A1008/A1008M from "(450)" to "(410)".

A2.3 Ductility

- On line 10 from the bottom of page 39, change "For $w/t \le E/F_y$" to "For $w/t \le 0.067E/F_{sy}$".
- From the bottom of page 39, on lines 1, 3, 6, 7, 8 (two places), 14 and 16, change "F_y" to "F_{sy}".
- Change "reduced yield point" to "reduced specified minimum yield point" on line 15 from the bottom of page 39.
- On line 4 of page 40, change "Yield point" to "Specified minimum yield point".

A2.4 Delivered Minimum Thickness

Remove the point symbol, ☞B.

A4.1.1 ASD Requirements

Add "and Appendix 1" to the end of both definitions for R_n and Ω.

A5.1.1 LRFD Requirements

Add "and Appendix 1" to the end of both definitions for R_n and ϕ.

A6.1.1 LSD Requirements

Add "and Appendix 1" to the end of both definitions for R_n and ϕ.

A7.2 Strength Increase from Cold Work of Forming

Revise Eq. A7.2-1 as follows:

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

A9 Referenced Documents

- Add the following references and renumber the sequence of the subsequent references:
  1. American Iron and Steel Institute (AISI), 1140 Connecticut Avenue, NW, Washington, DC 20036:
     AISI TS-1-02, Rotational-Lateral Stiffness Test Method for Beam-to-Panel Assemblies
     AISI TS-6-04, Standard Procedures for Panel and Anchor Structural Tests
     AISI-TS-8-04, Base Test Method for Purlins Supporting a Standing Seam Roof System

- Change the reference numbering from "1" and "2" to "2" and "3", respectively.
- Add the following ASTM standard before ASTM F436-00:

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

- Add the following new references to the end of the section:

  4. U. S. Army Corps of Engineers, CEGS-07416, Guide Specification for Military Construction, Structural Standing Seam Metal Roof (SSSMR) System, 1995
  5. Factory Mutual, FM 4471, Approval Standard for Class 1 Metal Roofs, 1986

B2.2 Uniformly Compressed Stiffened Elements with Circular Holes

Revise Eq. B2.2-2 as follows:

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

B3.2 Unstiffened Elements and Edge Stiffeners with Stress Gradient

Replace the whole section as follows:

B3.2 Unstiffened Elements and Edge Stiffeners with Stress Gradient

The following notation is used 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 as given in this section

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, $f_1 \geq f_2$.

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, defined in Section B2.1(a)

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

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

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

(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, to be determined by 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 by this section.

(1) When both f_1 and f_2 are in compression (Fig. B3.2-1):

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

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

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

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

(2) When f_1 is in compression and f_2 in tension (Fig. B3.2-2):

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

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

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

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

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

For $\psi < 1$

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

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

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

For $\psi \geq 1$

$\rho = 1$

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

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

$$b = w \quad \text{when } \lambda \le 0.856 \qquad (Eq.\ B3.2\text{-}8)$$
$$b = \rho w \quad \text{when } \lambda > 0.856 \qquad (Eq.\ B3.2\text{-}9)$$

where

$$\rho = 0.925/\sqrt{\lambda} \qquad (Eq.\ B3.2\text{-}10)$$
$$k = 0.145(b_o/h_o) + 1.256 \qquad (Eq.\ B3.2\text{-}11)$$
$$0.1 \le b_o/h_o \le 1.0$$

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

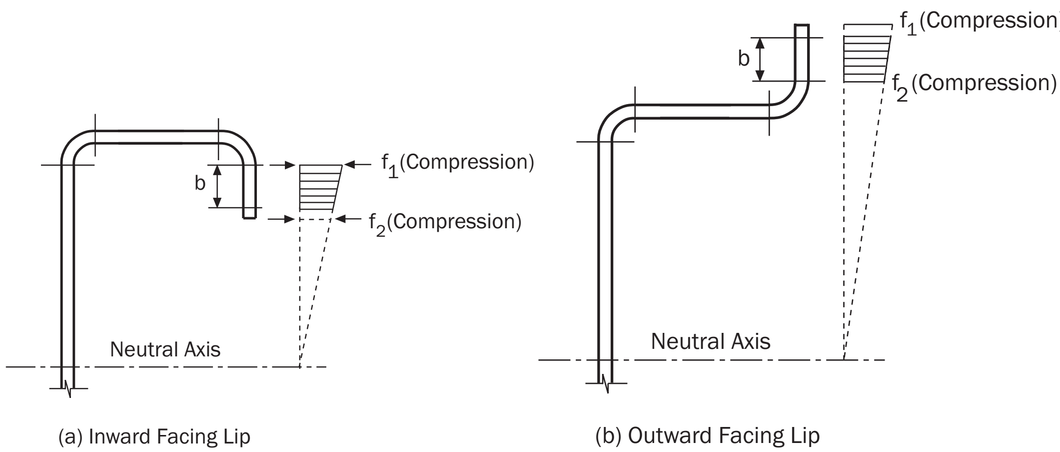

Figure B3.2-1 Unstiffened Elements under Stress Gradient, Both Longitudinal Edges in Compression

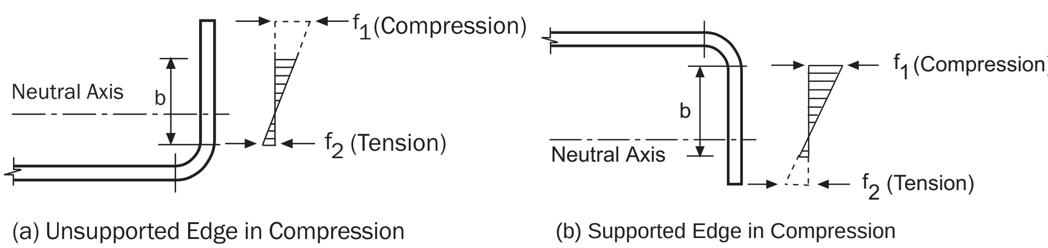

Figure B3.2-2 Unstiffened Elements under Stress Gradient, One Longitudinal Edge in Compression and the Other Longitudinal Edge in Tension

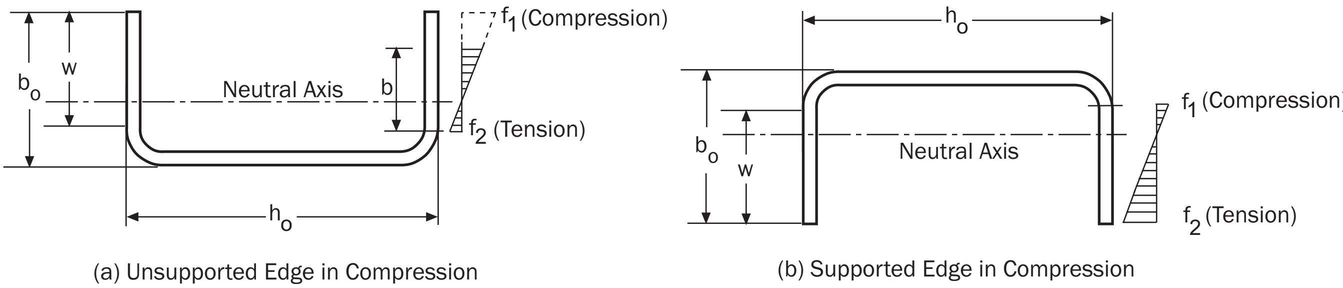

Figure B3.2-3 Unstiffened Elements of C-Section under Stress Gradient for Alternative Methods

In calculating the effective section modulus S_e in Section C3.1.1 or S_c in Section C3.1.2.1, the extreme compression fiber in Figures B3.2-1(b), B3.2-2(a) and B3.2-3(a) is 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) is 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.

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

Revise the last word in the title to "Stiffener(s)".

B5.2 Edge Stiffened Elements with Intermediate Stiffeners

Revise the last word in the title to "Stiffener(s)".

C3.1.1 Nominal Section Strength [Resistance]

- Replace item (4) in C3.1.1(b) with the following:

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

- Add the following definitions right before the definition for "e_y":

  h =Flat depth of web
  t =Base steel thickness of element
  w =Element flat width

- Replace items (a) to (c) for determining the compression strain factor, C_y, as follows:

(a) Stiffened compression elements without intermediate stiffeners

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

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

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

where

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

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

(b) Unstiffened compression elements

(i) Unstiffened compression elements under stress gradient causing compression at one longitudinal edge and tension at the other longitudinal edge:

$C_y = 3.0$ when $\lambda \le \lambda_3$ (*Eq*. C3.1.1-4)

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

$C_y = 1$ when $\lambda \ge \lambda_4$ (*Eq*. C3.1.1-6)

Where

$\lambda_3 = 0.43$ (*Eq*. C3.1.1-7)

$\lambda_4 = 0.673(1+\psi)$ (*Eq*. C3.1.1-8)

and ψ is defined in Section B3.2.

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

$C_y = 1$

(iii) Unstiffened compression elements under uniform compression:

$C_y = 1$

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

$C_y = 1$

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

- Add to the end of the first sentence, "subject to lateral-torsional buckling."
- Delete Eq. (C3.1.2.1-2) and replace with the following paragraph, and revise the sequence of all the subsequent equation numbers.

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

- Change the equation sequence for the remaining equations in this section.

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

Replace the first paragraph of the section by the followings:

Under gravity loading, the nominal strength [resistance] of standing seam roof panels shall be determined in accordance with Chapters B and C of the *Specification* or shall be tested in accordance with TS-6, "Standard Procedures for Panel and Anchor Structural Tests" as published by AISI. Under uplift loading, the nominal strength [resistance] of standing seam roof panel systems shall be determined by TS-6. Tests shall be performed according to TS-6 with the following exceptions:

1. The Uplift Pressure Test Procedure for Class 1 Panel Roofs in Factory Mutual Approval Standard 4471 shall be permitted.
2. Existing tests conducted according to the Corps of Engineers CEGS 07416 uplift test procedure prior to the adoption of these provisions shall be permitted.

The open-open end configuration although not prescribed by the ASTM E1592-01 test procedure shall be permitted provided the end conditions that are tested represent the installed condition and the test shall follow the requirements given in TS-6. All test results shall be evaluated according to this Section.

C3.3.1 ASD Method

Replace Eq. C3.3.1-1 with:

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

C3.3.2 LRFD and LSD Methods

Replace Eq. C3.3.2-1 with:

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

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

- On page 71, change the ending of the sentence in the fourth paragraph (starting with "One-flange loading…") as follows "…is equal to or greater than 1.5h."

- On the same page, change the ending of the sentence in the fifth paragraph (starting with "Two-flange loading…") as follows "…is less than 1.5h."
- Change the resistance factor, ϕ_w, for Canada LSD in Table C3.4.1-4 for the Unfastened support condition of Interior One-Flange Loading or Reaction from 0.75 to 0.70.
- The coefficients and safety and resistance factors are revised in Table C3.4.1-5, Multi-Web Deck Sections, for end one-flange loading or reaction cases with either fastened to or unfastened to support conditions:

TABLE C3.4.1-5
MULTI-WEB DECK SECTIONS

Support Conditions	Load Cases		C	C_R	C_N	C_h	USA and Mexico ASD Ω_w	USA and Mexico LRFD ϕ_w	Canada LSD ϕ_w	Limits
Fastened to Support	One-Flange Loading or Reaction	End	4	0.04	0.25	0.025	1.70	0.90	0.80	$R/t \le 20$
		Interior	8	0.10	0.17	0.004	1.75	0.85	0.75	$R/t \le 10$
	Two-Flange Loading or Reaction	End	9	0.12	0.14	0.040	1.80	0.85	0.70	$R/t \le 10$
		Interior	10	0.11	0.21	0.020	1.75	0.85	0.75	
Unfastened	One-Flange Loading or Reaction	End	3	0.04	0.29	0.028	2.45	0.60	0.50	$R/t \le 20$
		Interior	8	0.10	0.17	0.004	1.75	0.85	0.75	
	Two-Flange Loading or Reaction	End	6	0.16	0.15	0.050	1.65	0.90	0.80	$R/t \le 5$
		Interior	17	0.10	0.10	0.046	1.65	0.90	0.80	

Notes:
(1) The above coefficients apply when $h/t \le 200$, $N/t \le 210$, $N/h \le 3$.
(2) $45° \le \theta \le 90°$

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

Revise the requirement (6) on page 75 to "(6) Non-circular holes, corner radii $\ge 2t$".

C3.5 Combined Bending and Web Crippling

Replace the sub-sections C3.5.1, ASD Method, and C3.5.2, LRFD and LSD Methods, as follows:

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, shall satisfy $M \le M_{nxo}/\Omega_b$, and $P \le P_n/\Omega_w$. In addition, the following requirements shall be satisfied:

(a) For shapes having single unreinforced webs:

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

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

(b) For shapes having multiple unreinforced webs such as I-sections made of two C-sections connected back-to-back, or similar sections which provide a high degree of restraint against rotation of the web (such as I-sections made by welding two angles to a C-section);

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

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

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

Eq. C3.5.1-3 is valid for 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)
$R/t \leq 5.5$

The following conditions shall also be satisfied:

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

In the above equations:

Ω_b = Factor of safety for bending (See Section C3.1.1)
Ω_w = Factor of safety for web crippling (See Section C3.4)
Ω = Factor of safety for combined bending and web crippling
= 1.70
P = Required allowable strength for concentrated load or reaction in the presence of bending moment
P_n = Nominal strength for concentrated load or reaction in absence of bending moment determined in accordance with Section C3.4
M = Required allowable flexural strength at, or immediately adjacent to, the point of application of the concentrated load or reaction, P

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

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, $\overline{P}$, shall satisfy $\overline{M} \leq \phi_b M_{nxo}$, and $\overline{P} \leq \phi_w P_n$. In addition, the following requirements shall be satisfied:

(a) For shapes having single unreinforced webs:

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

where ϕ = 0.90 (LRFD)
= 0.75 (LSD)

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

(b) For shapes having multiple unreinforced webs such as I-sections made of two C-sections connected back-to-back, or similar sections which provide a high degree of restraint against rotation of the web (such as I-sections made by welding two angles to a C-section);

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

where ϕ = 0.90 (LRFD)
= 0.75 (LSD)

(c) For two nested Z-shapes

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

where ϕ =0.90 (LRFD)
=0.80 (LSD)

Eq. C3.5.2-3 is valid for shapes that meet the following limits:

$h/t \leq 150$
$N/t \leq 140$
$F_y \leq$ 70 ksi (480 MPa or 4910 kg/cm^2)
$R/t \leq 5.5$

The following conditions shall also be satisfied:

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

minimum of two 1/2 in. (12.7 mm) diameter A307 bolts through the flanges.

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

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

In the above equations:

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

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

$\overline{P}$ = Required strength for concentrated load or reaction [factored concentrated load or reaction] in presence of bending moment

$\overline{P} = P_u$ (LRFD)

$\overline{P} = P_f$ (LSD)

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

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

$\overline{M} = M_u$ (LRFD)

$\overline{M} = M_f$ (LSD)

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

C3.6 Stiffeners

Under this section, the title of Section C3.6.1 is changed to "Bearing Stiffeners"; a new section C3.6.2, Bearing Stiffeners in C-Section Flexural Members, is added; and the sequence of subsequent sections is changed accordingly.

Changes to Section C3.6.1:

- Change the title of the Section from "Transverse Stiffeners" to "Bearing Stiffeners".
- In the first sentence of the first paragraph, change the "Transverse stiffeners" to "Bearing stiffeners".
- Revise the definitions for the variables as follows:

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

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

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

A_b = $b_1t + A_s$, for bearing stiffener at interior support or under concentrated load (*Eq*. C3.6.1-4)

A_b = $b_2t + A_s$, for bearing stiffener at end support (*Eq*. C3.6.1-5)

A_s =Cross sectional area of bearing stiffener

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

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

L_{st} =Length of bearing stiffener

t =Base steel thickness of beam web

- Change "transverse stiffeners" to "the bearing stiffener" on the second line of page 80.

Add Section C3.6.2:

C3.6.2 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 C3.6.1, the nominal strength, P_n, shall be determined as follows:

$$P_n = 0.7(P_{wc} + A_eF_y) \geq P_{wc} \quad (Eq.\ C3.6.2\text{-}1)$$

USA and Mexico		Canada
Ω (ASD)	ϕ (LRFD)	ϕ (LSD)
1.70	0.90	0.80

where

P_{wc} =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 point

F_y =Yield point of bearing stiffener steel

Eq. C3.6.2-1 applies within the following limits:

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

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

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

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

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

(6) The bearing width shall not be less than 1-$^1/_2$ in. (38 mm).

Changes in the Current Section C3.6.2

- Revise the section number from “C3.6.2” to “C3.6.3” in the title and in all the equation numbers.
- Change “transverse stiffeners” in the definition for “a” to “shear stiffeners”.

Changes in the Current Section C3.6.3

- Revise the section number from “C3.6.3” to “C3.6.4” in the title.
- Revise the sentence to “The design strength [factored resistance] of members with stiffeners that do not meet the requirements of Section C3.6.1, C3.6.2 or C3.6.3, such as stamped or rolled-in stiffeners, shall be determined…”

C4.5 Built-Up Members

Replace item (3) as follows:

(3) The intermediate fastener(s) or weld(s) at any longitudinal member tie location shall be capable of transmitting a force in any direction of 2.5% of the total force in the built-up member (determined in accordance with ASD, LRFD or LSD load combinations).

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

Revise the note on page 85 as follows:

Note:

*Further information on the test procedure should be obtained from AISI TS-1, “Rotational-Lateral Stiffness Test Methods for Beam-to-Panel Assemblies”, Part VI of AISI *Cold-Formed Steel Design Manual*, 2002 edition.

C4.7 Compression of Z-Section Members Having One Flange Fastened to a Standing Seam Roof

Add the following new section:

C4.7 Compression of Z-Section Members Having One Flange Fastened to a Standing Seam Roof

The provisions of this section are applicable only to the United States and Mexico and are given in Section C4.7 of Appendices A and C. ☜ A,C

C5.2.1 ASD Method

On page 87, revise the text of the definition for P_n to “… in accordance with Sections C4 and C6”. On the same page, revise the text of the definition for

P_{no} to "…in accordance with Sections C4 and C6, …"

C5.2.2 LRFD and LSD Methods

- On page 89, revise the definition for P_n to "… in accordance with Sections C4 and C6". On the same page, revise the text of the definition for P_{no} to "…in accordance with Sections C4 and C6, …"
- On page 89, change Eq. C5.2.2-4 and Eq. C5.2.2-5 to the following, accordingly:

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

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

C6.2 Compression

On page 92, add the parentheses to the denominator terms as follows:

$$R = F_y/(2F_e) \le 1.0 \qquad (Eq.\ C6.2\text{-}6)$$

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

Revise the first sentence in the first paragraph as follows "For C-sections and Z-sections having deck or sheathing attached to the top flanges (through fastened or standing seam systems), …"

D3.2.2 Neither Flange Connected to Sheathing

Replace the whole section as follows:

D3.2.2 Neither Flange Connected to Sheathing

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

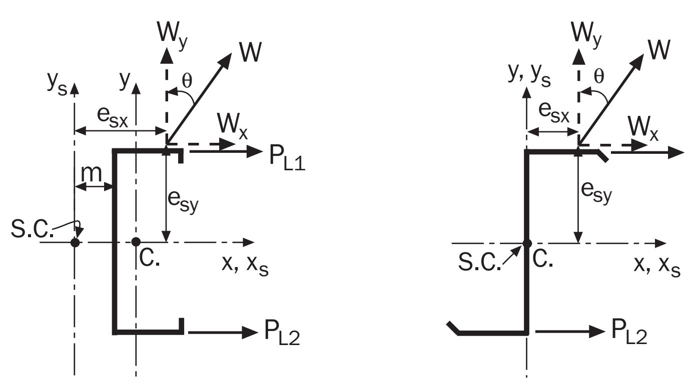

Figure D3.2.2-1 Coordinate Systems and Positive Force Directions and y axes positive, and P_{L2} is the brace force on the other flange. The x-axis is the centroidal axis perpendicular to the web, the y-axis is the centroidal axis parallel to the web. The x and y coordinates shall be oriented such that one of the flanges is located in the quadrant with both positive x and y axes. See Figure D3.2.2-1 for illustrations of coordinate systems and positive force directions.

(a) For uniform loads,

$$P_{L1} = 1.5[W_y K' - (W_x / 2) + (M_z / d)] \quad \text{(Eq. D3.2.2-1)}$$

$$P_{L2} = 1.5[W_y K' - (W_x / 2) - (M_z / d)] \quad \text{(Eq. D3.2.2-2)}$$

When a design load acts through the plane of the web, i.e. $W_y = W$:

$$P_{L1} = -P_{L2} = 1.5(m / d)W \qquad \text{For C-sections} \quad \text{(Eq. D3.2.2-3)}$$

$$P_{L1} = P_{L2} = 1.5(\frac{I_{xy}}{2I_x})W \qquad \text{For Z-sections} \quad \text{(Eq. D3.2.2-4)}$$

where

W_x, W_y = Components of design 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.

W = Design load (applied load determined in accordance with the most critical load combinations for ASD, LRFD or LSD, whichever is applicable) within a distance of 0.5a each side of the brace

a = Longitudinal distance between centerline of braces

d = Depth of section

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

M_z = $-W_x e_{sy} + W_y e_{sx}$, Torsional moment of design load W about shear center

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

K' = 0 for C-section

= $I_{xy}/(2I_x)$ for Z-section (Eq. D3.2.2-5)

I_{xy} = Product of inertia of full unreduced section

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

(b) For concentrated loads,

$$P_{L1} = P_y K' - (P_x / 2) + (M_z / d) \quad \text{(Eq. D3.2.2-6)}$$

$$P_{L2} = P_y K' - (P_x / 2) - (M_z / d) \quad \text{(Eq. D3.2.2-7)}$$

When a design load acts through the plane of the web, i.e. $P_y = P$:

$$P_{L1} = -P_{L2} = (m / d)P \qquad \text{For C-sections} \quad \text{(Eq. D3.2.2-8)}$$

$$P_{L1} = P_{L2} = (\frac{I_{xy}}{2I_x})P \qquad \text{For Z-sections} \quad \text{(Eq. D3.2.2-9)}$$

where

P_x, P_y = Components of design 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, correspondingly.

P = Design concentrated 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 concentrated design load is the applied load determined in accordance with the most critical load combinations for ASD, LRFD or LSD, whichever is applicable.

l = Distance from concentrated load to the brace

M_z = $-P_x e_{sy} + P_y e_{sx}$, Torsional moment of design load P about shear center

Other variables are defined under (a).

The bracing force, P_{L1} or P_{L2} is positive when restraint is required to prevent the movement of the corresponding flange in the negative x-direction.

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

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

D4 Wall Studs and Wall Stud Assemblies

Replace the whole section (including the subsections) with the followings:

D4 Wall Studs and Wall Stud Assemblies

Wall studs shall be designed either on the basis of an all steel system in accordance with Section D4.1 or on the basis of sheathing braced design in accordance with an appropriate theory, tests, or rational engineering analysis. Both solid and perforated webs shall be permitted. Both ends of the stud shall be connected to restrain rotation about the longitudinal stud axis and horizontal displacement perpendicular to the stud axis.

D4.1 All Steel Design

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

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

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

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

D5 Floor, Roof or Wall Steel Diaphragm Construction

Replace the whole section as follows:

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. Table D5 applies to both methods. If nominal shear strength is only established by test without defining all limit state thresholds, the factors of safety and resistance factors shall be limited by the values given in Table D5 for connection related failure modes.

Ω_d = As specified in Table D5 (ASD)

ϕ_d = As specified in Table D5 (LRFD and LSD)

TABLE D5
Factors of Safety and Resistance Factors for Diaphragms

Load Type or Combinations Including	Connection Type[1]	Limit State					
		Connection Related			Panel Buckling[2]		
		USA and Mexico		Canada	USA & Mexico		Canada
		Ω_d (ASD)	ϕ_d (LRFD)	ϕ_d (LSD)	Ω_d (ASD)	ϕ_d (LRFD)	ϕ_d (LSD)
Earthquake	Welds	3.00	0.55	0.50	2.00	0.80	0.75
	Screws	2.50	0.65				
Wind	Welds	2.35	0.70				
	Screws						
All Others	Welds	2.65	0.60				
	Screws	2.50	0.65				

Note:

1 When fastener combinations are used within a diaphragm system, the more severe factor is used.

2 Panel buckling is out of plane buckling and not local buckling at fasteners. The more severe factored limit state controls the design.

For mechanical fasteners other than screws: 1) Ω_d shall not be less than the Table D5 values for screws, and 2) 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 must 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 support thickness on the failure mode shall be considered.

E2 Welded Connections

- Change the thickness of the thinnest connected part from "0.18 in. (4.57 mm)" to "3/16 in. (4.76 mm)" two places in the first paragraph of the section.
- Add the sentence to the end of the second paragraph "For diaphragm applications, Section D5 shall be used."

E2.2 Arc Spot Welds

- Revise the first sentence in the first paragraph as follows: "Arc spot welds permitted by this *Specification* are for welding sheet steel to thicker supporting members or sheet-to-sheet in the flat position."
- Add the following sentence to the end of the second paragraph, "Sheet-to-sheet welds do not require weld washers."
- Section E2.2.1 has been reorganized and revised into two subsections: E2.2.1.1, Minimum Edge Distance, and E2.2.1.2, Shear Strength [Resistance] for Sheet(s) to a Thicker Supporting Member. And a new section E2.2.1.3, Shear Strength [Resistance] for Sheet-to-Sheet Connections, is added. The complete section is provided as follows:

E2.2.1 Shear

E2.2.1.1 Minimum Edge Distance

The distance measured in the line of force from the centerline of a weld to the nearest edge of an adjacent weld or to the end of the connected part toward which the force is directed shall not be less than the value of e_{min} as given below:

$$e_{min} = \frac{P\Omega}{F_u t} \quad \text{For ASD} \qquad (Eq.\ E2.2.1.1\text{-}1)$$

$$e_{min} = \frac{\overline{P}}{\phi F_u t} \quad \text{For LRFD and LSD} \qquad (Eq.\ E2.2.1.1\text{-}2)$$

When $F_u/F_{sy} \geq 1.08$

USA and Mexico		Canada
Ω(ASD)	ϕ(LRFD)	ϕ(LSD)
2.20	0.70	0.60

When $F_u/F_{sy} < 1.08$

USA and Mexico		Canada
Ω(ASD)	ϕ(LRFD)	ϕ(LSD)
2.55	0.60	0.50

where

P =Required shear strength (nominal force) transmitted by weld (ASD)

$\overline{P}$ =Required shear strength [shear force due to factored loads] transmitted by weld

$\overline{P} = P_u$ (LRFD)

$\overline{P} = P_f$ (LSD)

t =Total combined base steel thickness (exclusive of coatings) of sheet(s) involved in shear transfer above plane of maximum shear transfer

F_{sy} =Yield point as specified in Sections A2.1, A2.2 or A2.3.2

Note: See Figures E2.2.1.1-1 and E2.2.1.1-2 for edge distances of arc welds.

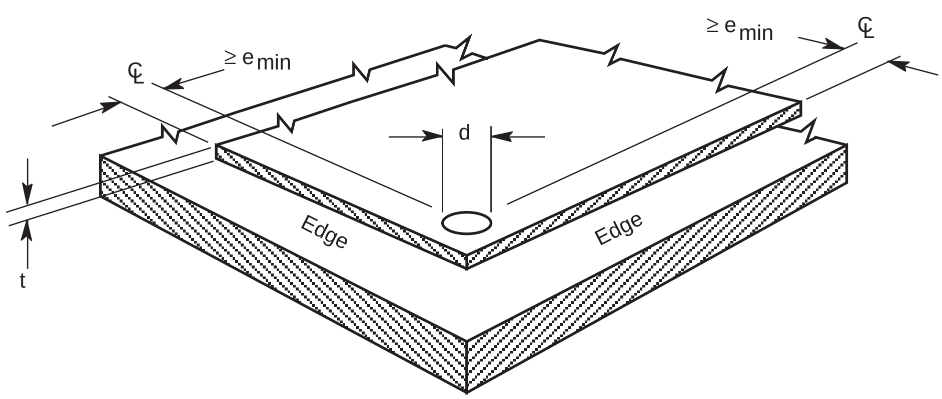

Figure E2.2.1.1-1 Edge Distance for Arc Spot Welds – Single Sheet

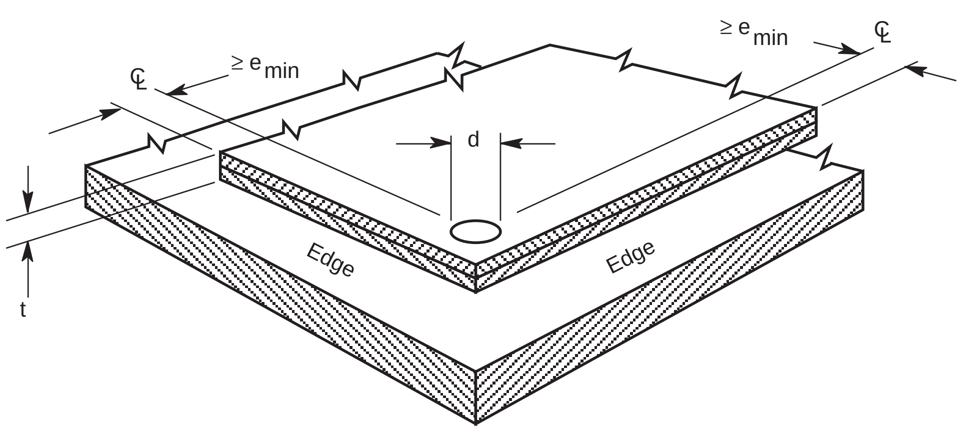

Figure E2.2.1.1-2 Edge Distance for Arc Spot Welds – Double Sheet

In addition, the distance from the centerline of any weld to the end or boundary of the connected member shall not be less than 1.5d. In no case shall the clear distance between welds and the end of member be less than 1.0d.

E2.2.1.2 Shear Strength [Resistance] for Sheet(s) Welded to a Thicker Supporting Member

The nominal shear strength [resistance], P_n, of each arc spot weld between the sheet or sheets and a thicker supporting member shall be determined by using the smaller of either

(a) $P_n = \frac{\pi d_e^2}{4} 0.75 F_{xx}$ (*Eq*. E2.2.1.2-1)

USA and Mexico		Canada
Ω(ASD)	ϕ(LRFD)	ϕ(LSD)
2.55	0.60	0.50

(b) For $(d_a/t) \leq 0.815 \sqrt{(E/F_u)}$

$P_n = 2.20\, t\, d_a\, F_u$ (*Eq*. E2.2.1.2-2)

USA and Mexico		Canada
Ω(ASD)	ϕ(LRFD)	ϕ(LSD)
2.20	0.70	0.60

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

$P_n = 0.280 \left[1 + 5.59 \frac{\sqrt{E/F_u}}{d_a/t} \right] t d_a F_u$ (*Eq*. E2.2.1.2-3)

USA and Mexico		Canada
Ω(ASD)	ϕ(LRFD)	ϕ(LSD)
2.80	0.55	0.45

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

$P_n = 1.40\, t\, d_a\, F_u$ (*Eq*. E2.2.1.2-4)

USA and Mexico		Canada
Ω(ASD)	ϕ(LRFD)	ϕ(LSD)
3.05	0.50	0.40

where

P_n = Nominal shear strength [resistance] of arc spot weld

d = Visible diameter of outer surface of arc spot weld

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

d_e = Effective diameter of fused area at plane of maximum shear transfer

= $0.7d - 1.5t \leq 0.55d$ (*Eq*. E2.2.1.2-5)

t = Total combined base steel thickness (exclusive of coatings) of sheets involved in shear transfer above plane of maximum shear transfer

F_{xx} =Tensile strength of electrode classification
F_u =Tensile strength as specified in Section A2.1, A2.2 or A2.3.2

Note: See Figures E2.2.1.2-1 and E2.2.1.2-2 for diameter definitions.

E2.2.1.3 Shear Strength [Resistance] for Sheet-to-Sheet Connections

The nominal shear strength [resistance] for each weld between two sheets of equal thickness shall be determined as follows:

$P_n = 1.65td_aF_u$ (*Eq*. E2.2.1.3-1)

USA and Mexico		Canada
Ω (ASD)	ϕ (LRFD)	ϕ (LSD)
2.20	0.70	0.60

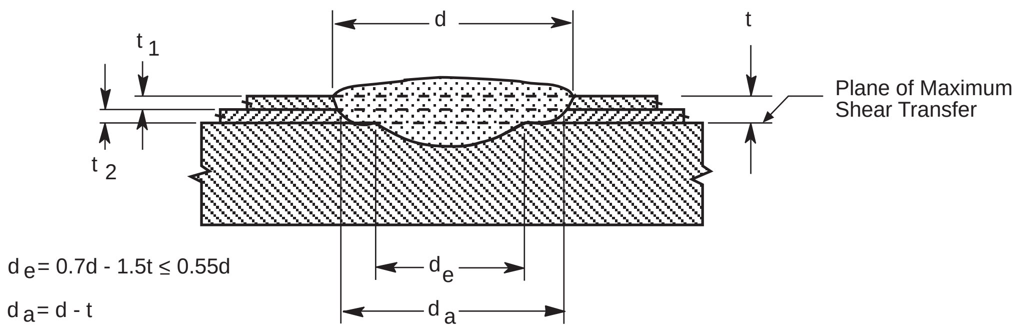

Figure E2.2.1.2-2 Arc Spot Weld – Double Thickness of Sheet

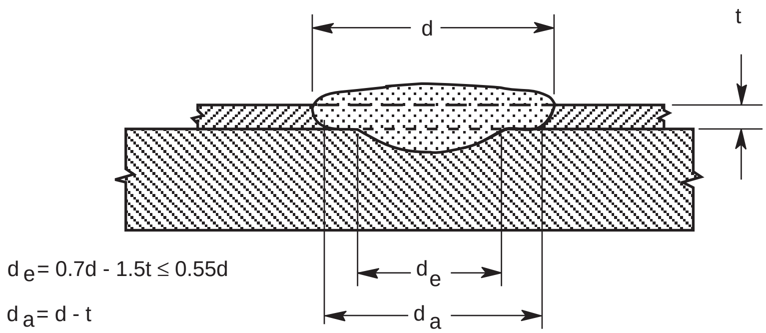

Figure E2.2.1.2-1 Arc Spot Weld – Single Thickness of Sheet

where

P_n = Nominal shear strength [resistance] of sheet-to-sheet connection
d = Visible diameter of the outer surface of arc spot weld
d_a = Average diameter of arc spot weld at mid-thickness of t
= (d - t)
d_e = Effective diameter of fused area at plane of maximum shear transfer
= $0.7d - 1.5t \leq 0.55d$ (*Eq*. E2.2.1.3-2)
F_u = Tensile strength of sheet as specified in Section A2.1 or A2.2

In addition, the following limits shall apply:

$F_u \leq 59$ ksi (407 MPa or 4150 kg/cm^2)
$F_{xx} > F_u$
0.028 in. (0.71 mm) $\leq t \leq$ 0.0635 in. (1.61 mm)

Note: See Figure E2.2.1.3-1 for diameter definitions.

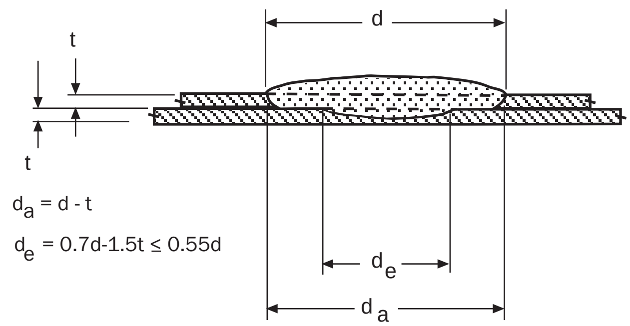

Figure E2.2.1.3-1 Arc Spot Weld – Sheet-to-Sheet

E2.2.2 Tension

On page 109, add the unit to constant "3" and the corresponding conversion as follows:

$t\, d_a\, F_u \leq 3$ kips (13.34 kN)

E4 Screw Connections

- Revise the fourth and fifth paragraphs as follows:

The following factor of safety or resistance factor shall be used for the sub-sections of Section E4, except as otherwise indicated.

USA and Mexico		Canada
Ω(ASD)	ϕ(LRFD)	ϕ(LSD)
3.00	0.50	0.40

Alternatively, design values for a particular application shall be permitted to be based on tests, with the factor of safety, Ω, and the resistance factor, ϕ, determined according to Chapter F.

- Delete the definition for "P_{nt}" on page 119.

E4.3.3 Shear in Screws

Replace the whole section as follows:

E4.3.3 Shear in Screws

The nominal shear strength [resistance] of the screw shall be taken as P_{ss}.

In lieu of the value provided in Section E4, the factor of safety or the resistance factor shall be permitted to be determined in accordance with Section F1 and shall be taken as $1.25\Omega \leq 3.0$ (ASD), $\phi/1.25 \geq 0.5$ (LRFD) or $\phi/1.25 \geq 0.4$ (LSD).

E4.4.3 Tension in Screws

Replace the whole section as follows:

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 factor of safety or the resistance factor shall be permitted to be determined in accordance with Section F1 and shall be taken as $1.25\Omega \leq 3.0$ (ASD), $\phi/1.25 \geq 0.5$ (LRFD) or $\phi/1.25 \geq 0.4$ (LSD).

E4.5 Combined Shear and Pull-Over

Add the following new section and the subsections:

E4.5 Combined Shear and Pull-Over

E4.5.1 ASD Method

For screw connections subjected to a combination of shear and tension forces, the following requirement shall be met:

$$\frac{Q}{P_{ns}} + 0.71\frac{T}{P_{nov}} \leq \frac{1.10}{\Omega} \qquad (Eq.\ E4.5.1\text{-}1)$$

In addition, Q and T shall not exceed the corresponding allowable design strength determined by Sections E4.3 and E4.4, respectively.

where

P_{ns} = Nominal shear strength of connection

$= 2.7 t_1 d F_{u1}$ (*Eq*. E4.5.1-2)

P_{nov} = Nominal pull-over strength of connection

$= 1.5 t_1 d_w F_{u1}$ (*Eq*. E4.5.1-3)

T = Required allowable tension strength of connection

Q = Required allowable shear strength of connection

d_w = Larger of screw head diameter or washer diameter

t_1 = Thickness of the member in contact with the screw head

F_{u1} = Tensile strength of the member in contact with the screw head

Ω = 2.35

Eq. E4.5.1-1 is valid for connections that meet the following limits:

(1) 0.0285 in. (0.724 mm) $\leq t_1 \leq$ 0.0445 in. (1.130 mm)

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

(3) $d_w \le 0.75$ in. (19.1 mm)

(4) $F_{u1} \le 70$ ksi (483 MPa or 4920 kg/cm^2)

(5) $t_2/t_1 \ge 2.5$

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

E4.5.2 LRFD and LSD Methods

For screw connections subjected to a combination of shear and tension forces the following requirements shall be met:

$$\frac{\overline{Q}}{P_{ns}} + 0.71\frac{\overline{T}}{P_{nov}} \le 1.10\phi \qquad (Eq.\ E4.5.2\text{-}1)$$

In addition, $\overline{Q}$ and $\overline{T}$ shall not exceed the corresponding design strength determined by Sections E4.3 and E4.4, respectively.

where

P_{ns} = Nominal shear strength [resistance] of connection

$= 2.7t_1dF_{u1}$ (*Eq*. E4.5.2-2)

P_{nov} = Nominal pull-over strength [resistance] of connection

$= 1.5t_1d_wF_{u1}$ (*Eq*. E4.5.2-3)

$\overline{T}$ = Required tension strength [factored tensile force] of connection

$\overline{T} = T_u$ for LRFD

$\overline{T} = T_f$ for LSD

$\overline{Q}$ = Required shear strength [factored shear force] of connection

$\overline{Q} = V_u$ for LRFD

$\overline{Q} = V_f$ for LSD

ϕ = 0.65 (LRFD)

= 0.55 (LSD)

Eq. E4.5.2-1 is valid for connections that meet the following limits:

(1) 0.0285 in. (0.724 mm) $\le t_1 \le$ 0.0445 in. (1.13 mm)

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

(3) $d_w \le 0.75$ in. (19.1 mm)

(4) $F_{u1} \le 70$ ksi (483 MPa or 4920 kg/cm^2)

(5) $t_2/t_1 \ge 2.5$

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

F1.1 Load and Resistance Factor Design and Limit States Design

In Table F1, the statistic data of "Bearing Strength" of "Screw Connections" (on page 125) were revised from "0.10" to "0.08" for V_M and from "0.10" to "0.05" for V_F, respectively.

CHANGES AND UPDATES IN APPENDICES A AND C

A2.2 Other Steels

Replace the whole section as follows:

A2.2 Other Steels

The listing in Section A2.1 does not exclude the use of steel up to and including 1 in. (25.4 mm) in thickness, ordered or produced to other than the listed specifications, provided the following requirements are met:

(1) The steel shall conform to the chemical and mechanical requirements of one of the listed specifications or other published specification.

(2) The chemical and mechanical properties shall be determined by the producer, the supplier, or the purchaser, in accordance with the following specifications. For coated sheets, ASTM A924/A924M; for hot-rolled or cold-rolled sheet and strip, ASTM A568/A568M; for plate and bar, ASTM A6/A6M; for hollow structural sections, such tests shall be made in accordance with the requirements of A500 (for carbon steel) or A847 (for HSLA steel).

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

(4) The steel shall meet the requirements of Section A2.3.

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

If the identification and documentation of the production of the steel have not been established, then in addition to requirements (1) through (5), the manufacturer of the cold-formed product shall establish that the yield point and tensile strength of the master coil are at least 10 percent greater than specified in the referenced published specification.

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

Revise the definition for R as follows:

R = Reduction factor determined by AISI TS-8, "Base Test Method for Purlins Supporting a Standing Seam Roof System" published by AISI.

C4.7 Compression of Z-Section Members Having One Flange Fastened to a Standing Seam Roof

Add the following new section:

C4.7 Compression of Z-Section Members Having One Flange Fastened to a Standing Seam Roof

These provisions are applicable to Z-sections concentrically loaded along their longitudinal axis, with only one flange attached to standing seam roof panels. Alternatively, design values for a particular system shall be permitted to be based on discrete point bracing locations, or on tests according to Chapter F.

The nominal axial strength of simple span or continuous Z-sections shall be calculated as follows:

(a) For weak axis nominal strength

$$P_n = k_{af}RF_yA \quad (Eq.\ C4.7\text{-}1)$$

Ω = 1.80 (ASD)

ϕ = 0.85 (LRFD)

where

For $d/t \le 90$

k_{af} = 0.36

For $90 < d/t \le 130$

$$k_{af} = 0.72 - \frac{d}{250t} \quad (Eq.\ C4.7\text{-}2)$$

For $d/t > 130$

k_{af} = 0.20

R = The reduction factor determined from uplift tests performed using AISI TS-8, "Base Test Method for Purlins Supporting a Standing Seam Roof System", published by AISI.

A = The full unreduced cross-sectional area of Z-section.

d = Z-section depth

t = Z-section thickness

F_y is defined in Section C3.1.1.

Eq. C4.7-1 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) $2.8 \le d/b < 5$
(6) $16 \le \dfrac{\text{flange flat width}}{t} < 50$
(7) Both flanges are prevented from moving laterally at the supports
(8) Yield point, $F_y \le$ 70 ksi (483 MPa or 4920 kg/cm^2)

where b = Z-section flange width.

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

E2a Welded Connections

Change the thickness of connected part from "0.18 in. (4.57 mm)" to "3/16 in. (4.76 mm)" one place in the first paragraph and one place in the second paragraph.

E3a Bolted Connections

- Add the following content to the end of the third paragraph:

In the situation where the hole occurs within the lap of lapped and nested zee members, the above requirements regarding the direction of the slot and the use of washers do not apply, subject to the following restrictions:

1) 1/2 in. (12.7 mm) diameter bolts only
2) Maximum slot size is 9/16 in. x 7/8 in. (14.3 mm x 22.2 mm) slotted vertically
3) Maximum oversize hole is 5/8 in. (15.9 mm) diameter
4) Minimum member thickness is 0.060 in. (1.52 mm) nominal
5) Maximum member yield stress is 60 ksi (410 MPa, and 4220 kg/cm^2)
6) Minimum lap length measured from center of frame to end of lap is 1.5 times the member depth.

E5.3 Block Shear Rupture

Replace the whole section with the followings:

E5.3 Block Shear Rupture

The block shear rupture nominal strength, R_n, shall be determined as follows when the thickness of the thinnest connected part is less than 3/16 in. (4.76 mm). For connections in which the thickness of the thinnest connected part is equal to or greater than 3/16 in. (4.76 mm), refer to AISC "Specification for Structural Steel Buildings, Allowable Stress Design and Plastic Design", or the "Load and Resistance Factor Design Specification for Structural Steel Buildings".

The nominal block shear rupture strength, R_n, shall be determined as the lesser of the following:

$$R_n = 0.6F_yA_{gv} + F_uA_{nt} \qquad (Eq.\ E5.3\text{-}1)$$

$$R_n = 0.6F_uA_{nv} + F_uA_{nt} \qquad (Eq.\ E5.3\text{-}2)$$

For bolted connections:

Ω =2.22 (ASD)

ϕ =0.65 (LRFD)

For welded connections:

Ω =2.50 (ASD)

ϕ =0.60 (LRFD)

where

A_{gv} = Gross area subject to shear

A_{nv} = Net area subject to shear

A_{nt} = Net area subject to tension

CHANGES AND UPDATES IN APPENDIX B

A2.2.1 Other Structural Quality Steels

On the third line in the section, change "published material Specification" to "published Specification".

A2.4a Delivered Minimum Thickness

Delete the entire section.

A3.1 Specified Loads

Replace the whole section as follows:

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 the mass of 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

L live load, a variable load due to intended use and occupancy, including loads due to movable equipment, cranes, 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, temperature, ground settlement, or combination thereof

W wind load, a variable load due to wind

H a permanent load due to lateral earth pressure, including groundwater

A3.2 Temperature Effects

Replace the whole section as follows:

A3.2 Temperature, Earth and Hydrostatic Pressure Effects

Where the effects due to lateral earth pressure H and imposed deformation T affect the structural safety, they shall be taken into account in the calculations, H with a load factor of 1.5 and T with a load factor of 1.25.

A6.1.2 Load Factors and Load Combinations for LSD

Replace the whole section, including the subsections A6.1.2.1, Load Factors (α), A6.1.2.2, Load Combination Factor (ψ), and A6.1.2.3, Importance Factors (γ), as follows:

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 combinations listed in Table B-A6.1.2-1, the applicable combination being that which results in the most critical effect.

Table B-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(4)) + 1.5L(2)	0.5S or 0.4W
3	(1.25D(4) or 0.9D(4)) + 1.5S	0.5L(3) or 0.4W
4	(1.25D(4) or 0.9D(4)) + 1.4W	0.5L(3) or 0.5S
5	1.0D(1) + 1.0E(5)	0.5L(3) + 0.25S

Notes to Table B-A6.1.2-1:

(1) Except for rocking footings, the counteracting factored dead load, 0.9D in load combinations (2), (3) and (4) and 1.0D in load combination (5), shall be used when dead load acts to resist overturning, uplift, sliding, failure due to stress reversal, and to determine anchorage requirements and factored member resistances.

(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 occupancies, and equipment areas and service rooms.

(4) The load factor 1.25 for dead load D for soil, superimposed earth, plants and trees shall be increased to 1.5, except that when the soil depth exceeds 1.2m, 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 (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 B-A6.1.2.1-1.

Table A6.1.2.1-1
Importance Categories for Buildings

USE AND OCCUPANCY	Importance Category
Buildings that represent a low direct or indirect hazard to human life in the event of failure including: • Low human-*occupancy buildings*, where it can be shown that collapse is not likely to cause injury or other serious consequences • Minor storage *buildings*(1)	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: • Elementary, middle and secondary schools • Community centres Manufacturing and storage facilities containing toxic, explosive or other hazardous substances in sufficient quantities to be dangerous to the public if released(1)	High
Post-disaster buildings including: • 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 • *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 having a Low Importance Category, a factor of 0.8 may be applied to the live load, L.

A6.1.2.2 Importance Factor (I)

The importance factor for snow, wind and earthquake shall be as provided for in Table B-A6.1.2.2-1

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

Change the publication year for *National Building Code of Canada* from "1995" to "2005".

C2.2 Fracture of Net Section

- Revise Eq. C2.2-4 to "$L_c = 0.6L_{nv}$".
- Add the following after Eq. C2.2-6:

(d) For failure of coped beams:

$L_c = 0.5L_t + 0.6L_v$ not involving stagger (*Eq*. C2.2-7)

$L_c = 0.45(L_t + L_s) + 0.6L_v$ involving stagger (*Eq*. C2.2-8)

where in (c) and (d), L_v is the lesser of CL_{gv} or L_{nv}

$C = F_y/F_u$ (*Eq*. C2.2-9)

- Delete the definition for L_v and add the following two new definitions:

L_{gv} = Gross failure path length parallel to force (i.e., in shear)

L_{nv} = Net failure path length parallel to force (i.e., in shear)

E2a Welded Connections

On the first and the third lines of the second paragraph, change the thickness limit from "4.57 mm" to "4.76 mm".

E3a Bolted Connections

Add the following to the end of the section:

Slotted of oversized holes may be used when the hole occurs within the lap of lapped or nested Z-members, subject to the following restrictions:

(1) 12.7 mm diameter bolts only, with or without washers
(2) Maximum slot size is 14.3 x 22.2 mm slotted vertically
(3) Maximum oversize hole is 15.9 mm diameter
(4) Minimum member thicknesses is 1.52 mm nominal
(5) Maximum member yield stress is 410 MPa
(6) Minimum lap length measured from centre of frame to end of lap is 1.5 times the member depth.

E3.4 Shear and Tension in Bolts

In the first line of the section, change "less than or equal to" to "less than".

Appendix 1, Design of Cold-Formed Steel Structural Members Using the Direct Strength Method

Appendix 1 is a newly added appendix.

American Iron and Steel Institute

Appendix 1

Design of Cold-Formed Steel Structural Members Using the Direct Strength Method

2004 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 through C (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, nor iteration, and instead 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.

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APPENDIX 1: DESIGN OF COLD-FORMED STEEL STRUCTURAL MEMBERS USING THE DIRECT STRENGTH METHOD

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 applicable for determination of the nominal axial (P_n) and flexural (M_n) strengths of cold-formed steel members. Sections 1.2.1 and 1.2.2 present a method applicable to all cold-formed steel beams and columns. Those members meeting the geometric and material limitations of Section 1.1.1.1 for columns and section 1.1.1.2 for beams have been pre-qualified for use, and the calibrated safety factor, Ω, and resistance factor, ϕ, given in 1.2.1 and 1.2.2 apply. Other beams and columns shall be permitted to use the provisions of Sections 1.2.1 and 1.2.2, but the standard Ω and ϕ factors for rational analysis (Section A1.1(b) of the main *Specification**) apply.

Currently, the Direct Strength Method provides no explicit provisions for members in tension, shear, combined bending and shear, web crippling, combined bending and web crippling, or combined axial load and bending (beam-columns). Further, no provisions are given for structural assemblies or connections and joints. As detailed in main *Specification* Section A1.1, the provisions of the main *Specification*, when applicable, shall be used for all cases listed above.

It shall be permitted to substitute the nominal strengths [resistances], resistance factors and factors of safety from this Appendix for the corresponding values from Sections C3.1, C4.1, C4.2, C4.3 and C4.4 of the main *Specification*.

For members or situations to which the main *Specification* is not applicable, obvious extensions to the Direct Strength Method of this Appendix may exist. Users who choose to employ such extensions to the Direct Strength Method are subject to the same provisions as any other rational analysis procedure as detailed in Section A1.1(b) of the main *Specification*: (1) applicable provisions of the main *Specification* must be followed when they exist, and (2) increased safety factors, Ω, and reduced resistance factors, ϕ, are employed for strength when rational analysis is conducted.

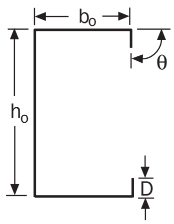
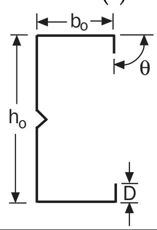
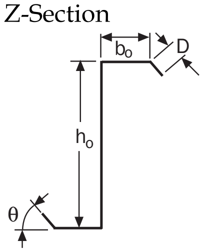
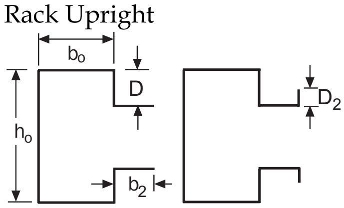
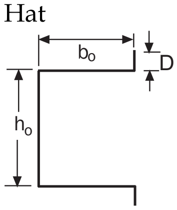
Note:

* The *North American Specification for the Design of Cold-Formed Steel Structural Members*, Chapters A through G and Appendices A through C is herein referred to as the main *Specification*.

1.1.1.1 Pre-qualified Columns

Unperforated columns that fall within the geometric and material limitations given in Table 1.1.1-1 shall be permitted to be designed using the safety factor, Ω, and resistance factor, ϕ, defined in Section 1.2.1.

Table 1.1.1-1 Limits for Pre-qualified Columns(1)

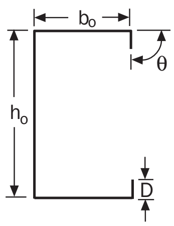
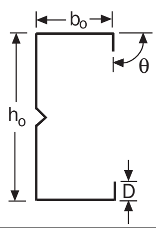
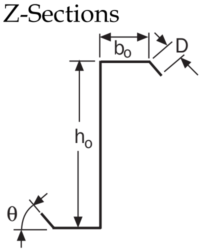
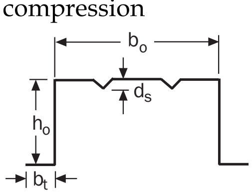
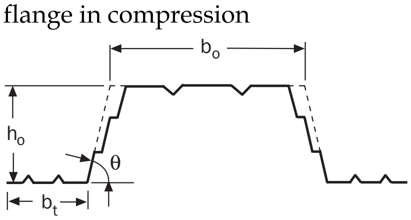
Lipped C-Section 	$h_o/t < 472$ $b_o/t < 159$ $4 < D/t < 33$ $0.7 < h_o/b_o < 5.0$ $0.05 < D/b_o < 0.41$ $\theta = 90$ deg $E/F_y > 340$ [$F_y < 86$ ksi (593 MPa or 6050 kg/cm^2)]
Lipped C-Section with Web Stiffener(s) 	$h_o/t < 489$ $b_o/t < 160$ $6 < D/t < 33$ $1.3 < h_o/b_o < 2.7$ $0.05 < D/b_o < 0.41$ one or two intermediate stiffeners $E/F_y > 340$ [$F_y < 86$ ksi (593 MPa or 6050 kg/cm^2)]
Z-Section 	$h_o/t < 137$ $b_o/t < 56$ $0 < D/t < 36$ $1.5 < h_o/b_o < 2.7$ $0.00 < D/b_o < 0.73$ $\theta = 50$ deg $E/F_y > 590$ [$F_y < 50$ ksi (345 MPa or 3520 kg/cm^2)]
Rack Upright 	$h_o/t < 51$ $b_o/t < 22$ $5 < D/t < 8$ $2.1 < h_o/b_o < 2.9$ $1.6 < b_2/D < 2.0$ (b_2 = stiffener parallel to b_o) $D_2/D = 0.3$ (D_2 = second lip parallel to D) $E/F_y = 340$ [$F_y < 86$ ksi (593 MPa or 6050 kg/cm^2)]
Hat 	$h_o/t < 50$ $b_o/t < 20$ $4 < D/t < 6$ $1.0 < h_o/b_o < 1.2$ $D/b_o = 0.13$ $E/F_y > 428$ [$F_y < 69$ ksi (476 MPa or 4850 kg/cm^2)]

(1) $r/t < 10$, where r is the centerline bend radius
b_o = overall width
D = overall lip depth
t = base metal thickness
h_o = overall depth

1.1.1.2 Pre-qualified Beams

Unperforated beams that fall within the geometric and material limitations given in Table 1.1.1-2 shall be permitted to be designed using the safety factor, Ω, and resistance factor, ϕ, defined in Section 1.2.2.

Table 1.1.1-2 Limitations for Pre-qualified Beams(1)

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 deg < θ < 90 deg $E/F_y > 421$ [$F_y < 70$ ksi (483 MPa or 4920 kg/cm²)]
Lipped C-Sections with Web Stiffener 	$h_o/t < 358$ $b_o/t < 58$ $14 < D/t < 17$ $5.5 < h_o/b_o < 11.7$ $0.27 < D/b_o < 0.56$ θ = 90 deg $E/F_y > 578$ [$F_y < 51$ ksi (352 MPa or 3590 kg/cm²)]
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 deg < θ < 90 deg $E/F_y > 440$ [$F_y < 67$ ksi (462 MPa or 4710 kg/cm²)]
Hats (Decks) with stiffened flange in compression 	$h_o/t < 97$ $b_o/t < 467$ $0 < d_s/t < 26$ (depth of stiffener) $0.14 < h_o/b_o < 0.87$ $0.88 < b_o/b_t < 5.4$ $0 < n \le 4$ (number of compression flange stiffeners) $E/F_y > 492$ [$F_y < 60$ ksi (414 MPa or 4220 kg/cm²)]
Trapezoids (Decks) with stiffened flange in compression 	$h_o/t < 203$ $b_o/t < 231$ $0.42 < (h_o/\sin\theta)/b_o < 1.91$ $1.10 < b_o/b_t < 3.38$ $0 < n_c \le 2$ (number of compression flange stiffeners) $0 < n_w \le 2$ (number of web stiffener/folds) $0 < n_t \le 2$ (number of tension flange stiffeners) 52 deg < θ < 84 deg (angle between web and horizontal plane) $E/F_y > 310$ [$F_y < 95$ ksi (655 MPa or 6680 kg/cm²)]

(1) $r/t < 10$, where r is the centerline bend radius.
See section 1.1.1.1 for definitions of other variables given in Table 1.1.1-2.

1.1.2 Elastic Buckling

Analysis is required for determination of the elastic buckling loads and or moments used in this Appendix. For columns, this includes the local, distortional and overall buckling loads: $P_{cr\ell}$, P_{crd}, and P_{cre} of Section 1.2.1. For beams, this includes the local, distortional and overall buckling moments: $M_{cr\ell}$, M_{crd}, and M_{cre} of Section 1.2.2. For a given column or beam, all three modes may not exist. In this case, the non-existent mode shall be ignored in the calculations of Sections 1.2.1 and 1.2.2. The commentary to this Appendix provides guidance on appropriate analysis procedures for elastic buckling determination.

1.1.3 Serviceability Determination

The bending deflection at any moment (M) due to nominal loads, shall be permitted to be determined by reducing the gross moment of inertia, I_g, to an effective moment of inertia for deflection, as given in Eq. 1.1.3-1:

$$I_{eff} = I_g(M_d/M) \le I_g \qquad (Eq.\ 1.1.3\text{-}1)$$

where

M_d = Nominal strength M_n defined in Section 1.2.2, but with M_y replaced by M in all formulas of Section 1.2.2.

M = Moment due to nominal loads [specified moments] on member to be considered ($M \le M_y$)

1.2 MEMBERS

1.2.1 Column Design

The nominal axial strength, P_n, is 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 are as follows:

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

For all other columns, Ω and ϕ of main *Specification* Section A1.1(b) apply.

1.2.1.1 Flexural, Torsional, or Torsional-Flexural Buckling

The nominal axial strength, P_{ne}, for flexural, torsional, or torsional-flexural buckling is

for $\lambda_c \le 1.5$

$$P_{ne} = \left(0.658^{\lambda_c^2}\right)P_y \qquad (Eq.\ 1.2.1\text{-}1)$$

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

$$\lambda_c = \sqrt{P_y/P_{cre}} \qquad (Eq.\ 1.2.1\text{-}3)$$

$$P_y = A_gF_y \qquad (Eq.\ 1.2.1\text{-}4)$$

P_{cre} = Minimum of the critical elastic column buckling load in flexural, torsional, or torsional-flexural buckling determined in accordance with Section 1.1.2

1.2.1.2 Local Buckling

The nominal axial strength, $P_{n\ell}$, for local buckling is

for $\lambda_\ell \le 0.776$

$$P_{n\ell} = P_{ne} \quad (Eq.\ 1.2.1\text{-}5)$$

for $\lambda_\ell > 0.776$

$$P_{n\ell} = \left[1 - 0.15\left(\frac{P_{cr\ell}}{P_{ne}}\right)^{0.4}\right]\left(\frac{P_{cr\ell}}{P_{ne}}\right)^{0.4} P_{ne} \quad (Eq.\ 1.2.1\text{-}6)$$

where $\lambda_\ell = \sqrt{P_{ne}/P_{cr\ell}}$ (*Eq*. 1.2.1-7)

$P_{cr\ell}$ = Critical elastic local column buckling load determined in accordance with Section 1.1.2

P_{ne} is defined in Section 1.2.1.1.

1.2.1.3 Distortional Buckling

The nominal axial strength, P_{nd}, for distortional buckling is

for $\lambda_d \le 0.561$

$$P_{nd} = P_y \quad (Eq.\ 1.2.1\text{-}8)$$

for $\lambda_d > 0.561$

$$P_{nd} = \left(1 - 0.25\left(\frac{P_{crd}}{P_y}\right)^{0.6}\right)\left(\frac{P_{crd}}{P_y}\right)^{0.6} P_y \quad (Eq.\ 1.2.1\text{-}9)$$

where $\lambda_d = \sqrt{P_y/P_{crd}}$ (*Eq*. 1.2.1-10)

P_{crd} = Critical elastic distortional column buckling load determined in accordance with Section 1.1.2

P_y is given in Eq. 1.2.1-4.

1.2.2 Beam Design

The nominal flexural strength, M_n, is the minimum of M_{ne}, $M_{n\ell}$ and M_{nd} as given in Sections 1.2.2.1 to 1.2.2.3. For beams meeting the geometric and material criteria of Section 1.1.1.2, Ω_b and ϕ_b are as follows:

USA and Mexico		Canada
Ω_b (ASD)	ϕ_b (LRFD)	ϕ_b (LSD)
1.67	0.90	0.85

For all other beams, Ω and ϕ of main *Specification* Section A1.1(b) apply.

1.2.2.1 Lateral-Torsional Buckling

The nominal flexural strength, M_{ne}, for lateral-torsional buckling is

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

where

M_y = S_fF_y, where S_f is the gross section modulus referenced to the extreme fiber in first yield (*Eq*. 1.2.2-4)

M_{cre} = Critical elastic lateral-torsional buckling moment determined in accordance with Section 1.1.2

1.2.2.2 Local Buckling

The nominal flexural strength, $M_{n\ell}$, for local buckling is

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (*Eq*. 1.2.2-7)

$M_{cr\ell}$ = Critical elastic local buckling moment determined in accordance with Section 1.1.2

M_{ne} is defined in Section 1.2.2.1.

1.2.2.3 Distortional Buckling

The nominal flexural strength, M_{nd}, for distortional buckling is

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (*Eq*. 1.2.2-10)

M_{crd} = Critical elastic distortional buckling moment determined in accordance with Section 1.1.2.

M_y is given in Eq. 1.2.2-4.

American Iron and Steel Institute

Supplement 2004 to the Commentary on the North American Specification for the Design of Cold-Formed Steel Structural Members

2001 EDITION

The material contained herein has been developed by a joint effort of the American Iron and Steel Institute Committee on Specifications, the Canadian Standards Association Technical Committee on Cold Formed Steel Structural Members (S136), and Camara Nacional de la Industria del Hierro y del Acero (CANACERO) in Mexico. The organizations and the Committees have made a diligent effort to present accurate, reliable, and useful information on cold-formed steel design. The Committees acknowledge and are grateful for the contributions of the numerous researchers, engineers, and others who have contributed to the body of knowledge on the subject. Specific references are included in the *Supplement* to 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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DECEMBER, 2004

CHANGES AND UPDATES IN CHAPTERS A THROUGH G

A1.1 Scope and Limits of Applicability

- On line 22 of page 11, add a sentence as follows "…sound engineering judgment. Safety and resistance factors are provided for ease of use, but these factors should not be used if applicable safety factors or resistance factors in the main *Specification**** are more conservative."

- Add the following as the last paragraph of the section:

In 2004, Appendix 1, Design of Cold-Formed Steel Structural Members Using the Direct Strength Method, was introduced. The Appendix provides an alternative design procedure for several Sections of Chapters 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 based on the elastic buckling behavior. The procedure does not require effective width calculations, nor iteration, and instead uses gross section properties and the elastic buckling behavior of the cross-section to predict the strength. The applicability of the provided provisions is detailed in the General Provisions of Appendix 1.

- Add the following to the end of the section as the third note:

*** The *Specification* Chapters A through G and Appendices A through C is herein referred to as the main *Specification*.

A2.1 Applicable Steels

Add the following content to the end of the last paragraph:

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 A2.3.1.

A2.3 Ductility

- On line 11 from the bottom of page 18, change "Part VIII" to "Part VI"

- Revise the fifth paragraph of the section as follows:

In the past, the yield point used in design was limited to 75 percent of the specified minimum yield point, 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 factor of safety, but still made low ductility steels, such as SS Grade 80 and Grade E, useful for the named applications.

- In the sixth paragraph of the section, delete the word "recent" on the first line, delete "Clause" on the third line, change "R_bF_y" to "R_bF_{sy}" on the same line, and change "is" to "was" on the sixth line.
- Delete the word "Clause" on the first line of both seventh and eighth paragraphs.
- In the ninth paragraph, change "F_y" to "F_{sy}" on the fifth line.

A2.4 Delivered Minimum Thickness

- Delete the point symbol, ☞B .
- Add the following sentence to the end of the section:

In 2004, the country specific section, A2.4a, was deleted from Appendix B.

A7.2 Strength Increase from Cold Work of Forming

Add the following as the first paragraph page 33:

The limitation $F_{ya} \leq F_{uv}$ places an upper bound on the average yield point. The intent of the upper bound is to limit stresses in flat elements that may not see significant increases in yield point and tensile strength as compared to the virgin steel properties.

B2.2 Uniformly Compressed Stiffened Elements with Circular Holes

Add the following paragraph to the end of the section on page 46:

In 2004, the *Specification* Equation B2.2-2 was revised to provide continuity at $\lambda = 0.673$.

B3.2 Unstiffened Elements and Edge Stiffeners with Stress Gradient

Replace the whole section with the following:

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

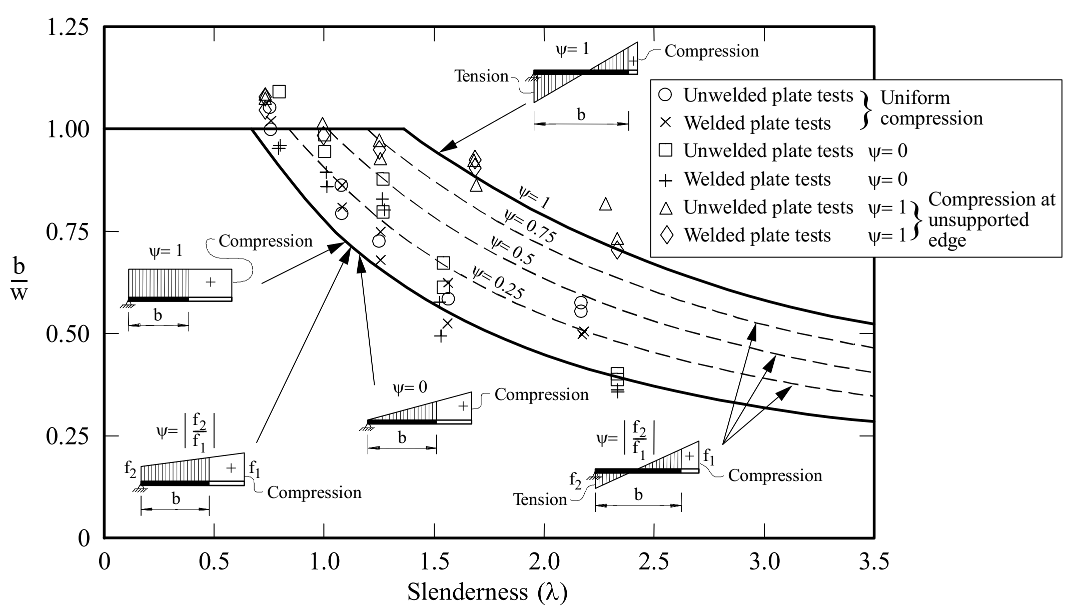

Figure C-B3.2-1 Effective Width vs. Plate Slenderness unsupported edge is in compression, both the effective width curve and the plate buckling coefficient depend on the stress ratio, as per Equations B3.2-4 and B3.2-5 of the *Specification*.

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 Pekoz (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*.

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

Change the last word in the title from “Stiffeners” to “Stiffener(s)”.

B5.2 Edge Stiffened Elements with Intermediate Stiffeners

- Change the last word in the title from “Stiffeners” to “Stiffener(s)”.
- Revise the first sentence of the section to “The buckling modes for edge stiffened elements with one or more intermediate stiffeners include:…”
- Add the following paragraph to the end of the section:

Stub compression testing performed in 2003 demonstrates the adequacy of this approach (Yang and Hancock, 2003).

C3.1.1 Nominal Section Strength

Add the following paragraph to the end of the section:

In 2004, additional *Specification* equations are 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*.

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

- On lines 14 and 15 from the bottom of page 64, change the equation numbers from "C3.1.2.1-14 and C3.1.2.1-15" to "C3.1.2.1-13 and C3.1.2.1-14" and on line 11 from the bottom of the same paragraph, change "C3.1.2.1-14" to "C3.1.2.1-13".
- On line 20 of page 65, change the equation numbers from "C3.1.2.1-5 and C3.1.2.1-6" to "C3.1.2.1-4 and C3.1.2.1-5". On line 11 from the bottom of the same page, change "C3.1.2.1-8 and C3.1.2.1-9" to "C3.1.2.1-7 and C3.1.2.1-8".
- Revise Equations C-C3.1.2.1-15 and C-C3.1.2.1-16 on page 67 as follows, respectively:

$$L_u = \frac{1}{K_y}\left[\frac{0.36C_b\pi^2EdI_{yc}}{F_yS_f}\right]^{0.5} \qquad \text{(C-C3.1.2.1-15)}$$

$$L_u = \frac{1}{K_y}\left[\frac{0.18C_b\pi^2EdI_{yc}}{F_yS_f}\right]^{0.5} \qquad \text{(C-C3.1.2.1-16)}$$

- Add the following paragraph right below Equation C-C3.1.2.1-16 on page 67:

  For members with unbraced length, $L \le L_u$, or elastic lateral-torsional buckling stress, $F_e \ge 2.78F_y$, the flexural strength [moment resistance] is determined in accordance with C3.1.1(a).

- On line 5 from the bottom of page 67, change "…nominal lateral buckling strength…" to "nominal lateral-torsional buckling strength…"
- On line 5 of page 69, change "Part VII" to "Part V".

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

On line 13 from bottom of page 69, change "C3.1.2.1-3" to "C3.1.2.1-2".

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

- Replace the first and the second paragraphs with the followings:

  Under uplift loading, nominal strength [nominal 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 [nominal resistance] by testing. Under gravity loading, the nominal strength [nominal resistance] of many panels can be calculated accurately. Three test protocols have been used in this effort: FM 4471 developed by Factory Mutual, CEGS 07416 by the Corps of Engineers and E1592 by ASTM. 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 TS-6. While these test procedures have a common base, none defines a design strength [factored resistance]. *Specification* Section C3.1.5 and AISI TS-6, "Standard Procedures for Panel and Anchor Structural Tests", adopted in 1999, added closure to the question by defining appropriate resistance and safety factors.

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.

- In the fourth paragraph of the section change "controlling allowable load" to "design strength" on the fourth line, and change "ultimate load" to "nominal strength" on the fifth line.

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

- Revise the sentence in the second paragraph to: "…by the following interaction equation (Bleich, 1952), which is part of a unit circle:".
- Add the following after Equation C-C3.3-1:

or

$$\sqrt{\left(\frac{f_b}{f_{cr}}\right)^2 + \left(\frac{\tau}{\tau_{cr}}\right)^2} = 1.0 \qquad \text{(C-C3.3-2)}$$

- On page 74, revise the subsequent equation number "C-C3.3-2" to "C-C3.3-3" on line 12, in Figure C-C3.3-1, lines 2 and 4 from bottom of the page; and on page 75, revise the equation number "C-C3.3-2" to "C-C3.3-3" on lines 4, 5, 11 and 12.

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

- In Figures C-C3.4.1-3 and C-C3.4.1-4, the distances from the edge of bearing to the end of the member have been revised as shown in the next two pages.
- Add the following sentence after the second sentence in the second to the last paragraph on page 80, "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."

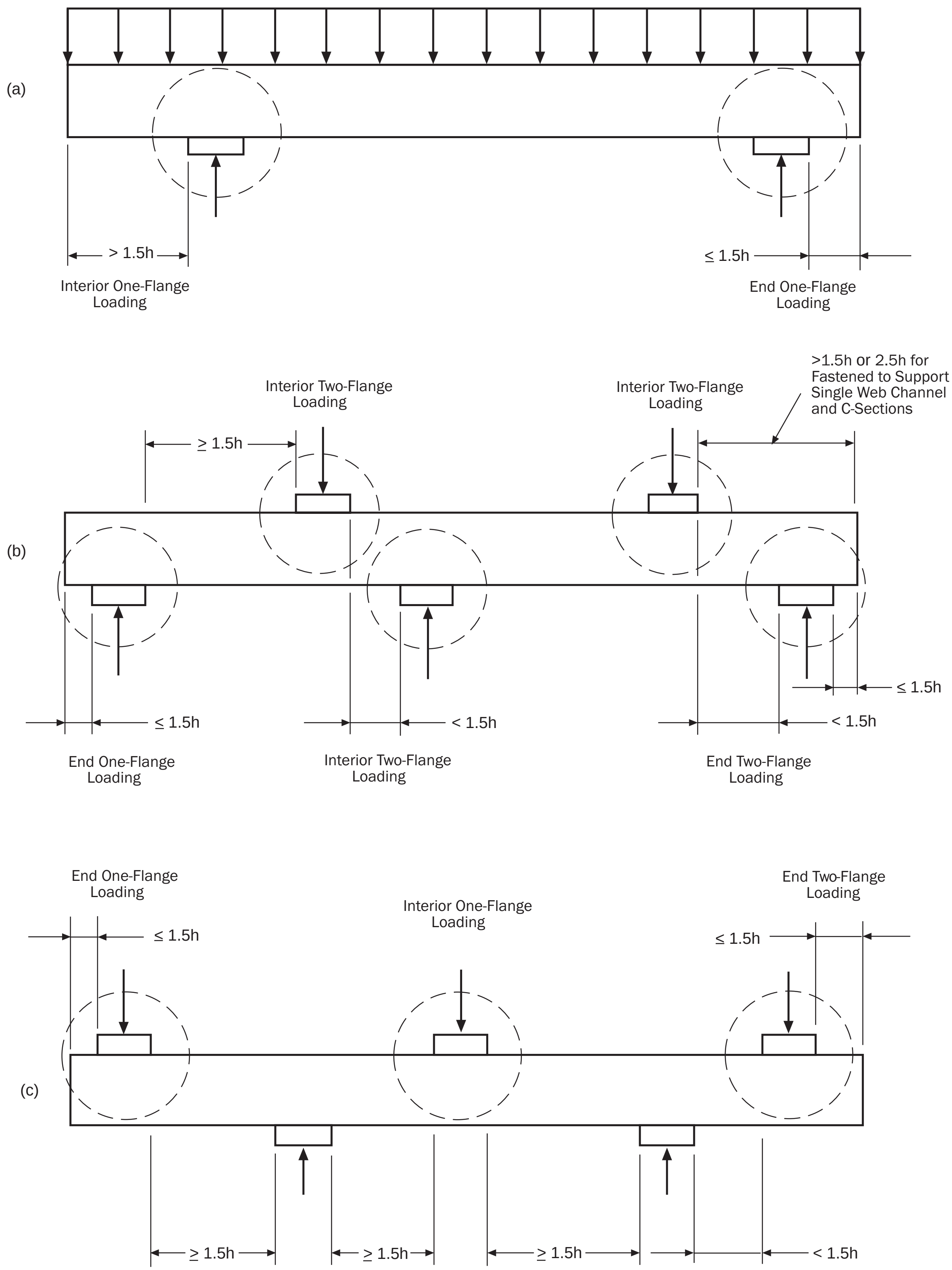

Figure C-C3.4.1-3 Application of Loading Cases

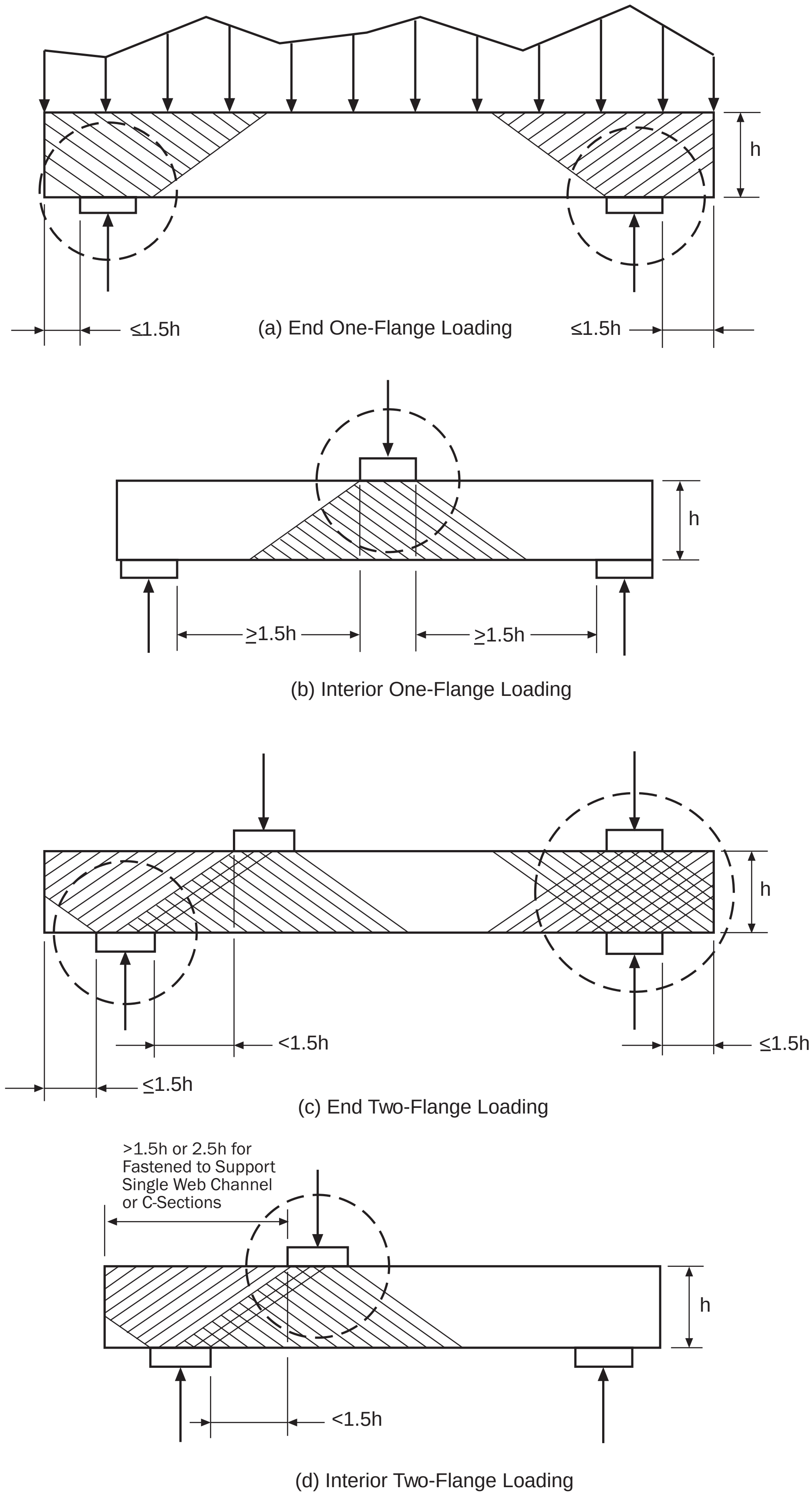

Figure C-C3.4.1-4 Assumed Distribution of Reaction or Load

- Add the following two paragraphs to the end of the section on page 81:

  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 (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 factors of safety and resistance factors.

  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 factors of safety 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*.

C3.5.1 ASD Method

- Replace the first paragraph of the section with the following:

  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 [resistance] should be determined by tests according to *Specification* Chapter F.

- Delete the third paragraph on page 83.
- Replace the paragraph starting with "In 2001, …" and the rest of the section with the following:

  Based on the test data of LaBoube, Nunnery, and Hodges (1994), in 2004, the interaction equation for the combined effects of bending and web crippling was re-evaluated because a new web crippling equation was adopted for Section C3.4.1 of the *Specification*.

C3.5.2 LRFD and LSD Methods

- Revise the first sentence in the first paragraph of the section as follows "… are based on the same equations as used for ASD using the required and design strengths."

- Revise the first sentence of the second paragraph of the section as follows: "In the development of the original LRFD equations, …"
- Delete the second sentence (to the end of the section) in the last paragraph of the section.

C3.6 Stiffeners

Under this section, the title of Section C3.6.1 has been changed to "Bearing Stiffeners". A new section C3.6.2, Bearing Stiffeners in C-Section Flexural Members, is added. Accordingly, the subsequent sections are renumbered.

Changes in Section C3.6.1

- Change the title of the Section from "Transverse Stiffeners" to "Bearing Stiffeners".
- Revise the first sentence in the first paragraph to "Design requirements for attached bearing stiffeners (previously called transverse stiffeners) and …"
- Add the following sentence as the third sentence in the first paragraph:

  The term "transverse stiffener" was renamed to "bearing stiffeners" in 2004.

- Change "transverse stiffeners" to "bearing stiffeners" on lines 6, 11, 17 and 21 in the section.

Add a New Section:

C3.6.2 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 Section C3.6.1. 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 Section C3.6.2 is a simplified method applicable with the limits of the test program. A more detailed beam-column design method is described in Fox (2002).

Changes in the Current Section C3.6.2

Revise the section number from "C3.6.2" to "C3.6.3" in the section title, on line 1 from the bottom of page 84, and lines 2, 3, and 4 on page 85.

Changes in the Current Section C3.6.3

- Revise the section number from "C3.6.3" to "C3.6.4" in the section title.

- Delete the word "transverse" on the first line and change "C3.6.3" to "C3.6.4" on the second line of the section.

C4 Concentrically Loaded Compression Members

- On line 25 of page 87, change "Part VIII" to "Part VI".
- On page 95, change "VII" to "V" on lines 5, 10 and 18 from the bottom of the page.

C4.4 Nonsymmetric Sections

On line 6 of the section, change "VII" to "V".

C4.5 Built-Up Members

Replace the six paragraph with the following:

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% of the total axial force in the built-up member determined in accordance with ASD, LRFD or LSD load combinations, whichever applicable. A longitudinal member tie is defined as a location of interconnection of the two members in contact. This requirement has been adopted from CSA S136-94 and is new to the AISI *Specification*.

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

Add a sentence to the end of the second paragraph in the section: "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."

C4.7 Compression of Z-Section Members Having One Flange Fastened to a Standing Seam Roof

Add the following new section:

C4.7 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 Appendices A and C. ☜A,C

C6.2 Compression

Revise the last sentence of the section and add an additional sentence as follows "…, $R = F_y/(2F_e)$ 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 Eq. C6.2-5 of the *North American Specification*."

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

Add the following content to the end of the second line on page 111: "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."

D3.2.2 Neither Flange Connected to Sheathing

- Change "of" to "on" on the last line of the first paragraph of this section.
- Revise the first sentence of the second paragraph of this section as follows "In order to obtain the information for developing bracing provisions,…"
- Revise the sentence starting from line 3 from the bottom of page 115 to "The horizontal brace force is then, simply, the appropriate reaction of this continuous beam. The provisions of *Specification* Section D3.2.2 provide expressions for determining bracing forces P_{L1} and P_{L2}, which the braces are required to resist at each flange."
- On line 8 from the bottom of page 116, change "…in each of them." to "…along each of them."
- On line 13 on page 117, change "…intermittently C-beams." to "…intermittently braced C-beams."
- On line 14 on page 117, change "…$P=Q(I_{xy}/I_x)$ or $P=Q[I_{xy}/(2I_x)]$…" to "…$Q(I_{xy}/I_x)$ or $Q[I_{xy}/(2I_x)]$…"
- At the end of the sentence on line 19 of page 117, add the sentence "To control the lateral deflection, brace forces, P, must statically balance the fictitious force."
- Revise Figure C-D3.2.2-5 on page 117 as shown below:

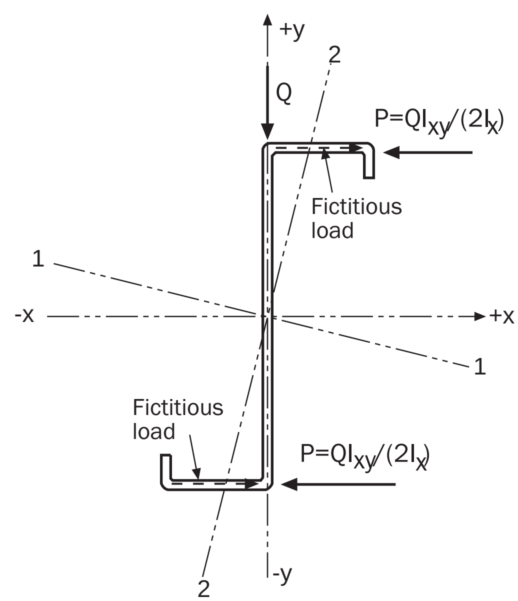

Figure C-D3.2.2-5 Principal Axis of Z-Section

- Add the following new content before item *(c) Spacing of Braces* on page 118:

(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_{sx} and e_{sy} should be measured based on x_s and y_s coordinate system.

For the C-section member as shown in Figure C-D3.2.2-6, the bracing forces on both flanges are given in Equations C-D3.2.2-1 and C-D3.2.2-2.

$$P_{L1} = -\frac{P_x}{2} + \frac{M_z}{d} \quad \text{(C-D3.2.2-1)}$$

$$P_{L2} = -\frac{P_x}{2} - \frac{M_z}{d} \quad \text{(C-D3.2.2-2)}$$

$$M_z = -P_x e_{sy} + P_y e_{sx} \quad \text{(C-D3.2.2-3)}$$

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.

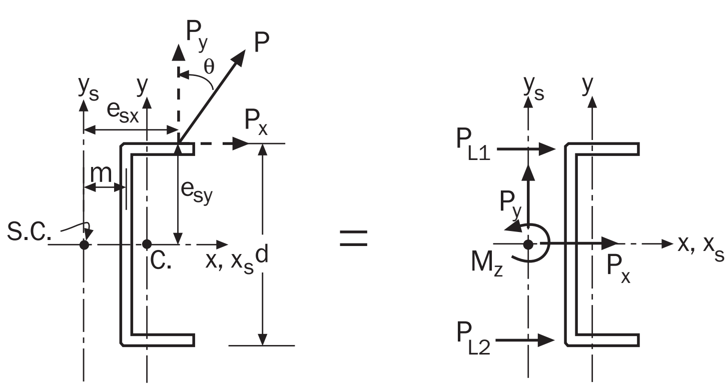

Figure C-D3.2.2-6 C-Section Member Subjected to a Concentrated Load

For a special case where design load, Q, is through the web, as shown in Figure C-D3.2.2-3, P_y = -Q, P_x = 0; e_{sx} = m, e_{sy} = d/2, and from Equation C-D3.2.2-3, M_z = -Qm. Therefore

$$P_{L1} = -Qm/d \quad \text{(C-D3.2.2-4)}$$

$$P_{L2} = Qm/d \quad \text{(C-D3.2.2-5)}$$

In which, m = distance from centerline of web to the shear center.

For the Z-section member as shown in Figure C-D3.2.2-7, bracing forces, P_{L1} and P_{L2}, are given in Equations C-D3.2.2-6 and C-D3.2.2-7.

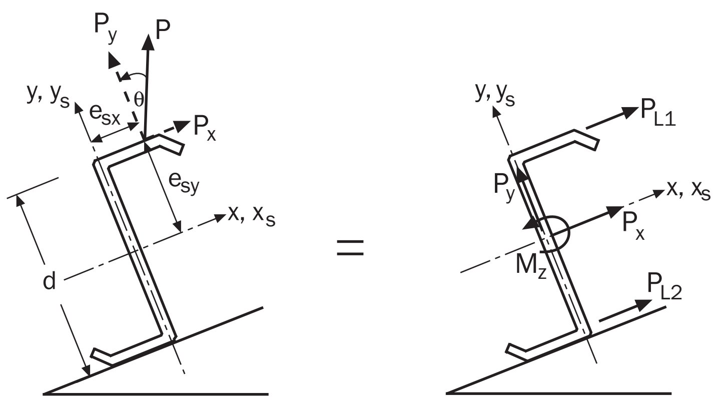

Figure C-D3.2.2-7 A Z-Section Member Subjected to an Arbitrary Load

$$P_{L1} = P_y\left(\frac{I_{xy}}{2I_x}\right) - \frac{P_x}{2} + \frac{M_z}{d} \quad \text{(C-D3.2.2-6)}$$

$$P_{L2} = P_y\left(\frac{I_{xy}}{2I_x}\right) - \frac{P_x}{2} - \frac{M_z}{d} \quad \text{(C-D3.2.2-7)}$$

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-D3.2.2-6 and C-D3.2.2-7 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 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.

- Change "*(c) Spacing of Braces*" to "*(d) Spacing of Braces*".

D4 Wall Studs and Wall Stud Assemblies

- Delete the last sentence of the last paragraph of the section.

- Add a new paragraph to the end of the section as follows:

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, 2004; Green, Winter, and Cuykendall, 1947; Simaan, 1973; and Simaan and Pekoz, 1976.

- Replace Section D4.1 with the following:

D4.1 All Steel Design

The approach of determining effective areas in accordance with *Specification* Section D4.1 is currently being used in the RMI *Specification* (Rack Manufacturers Institute, 1997) for the design of perforated rack columns and was verified extensively for such structures as reported by Pekoz (1988a). The validity of this approach for wall studs was verified in a Cornell University project on wall studs reported by Miller and Pekoz (1989 and 1994).

The limitations included in *Specification* Section D4.1 for the size and spacing of perforations and the depth of studs are based on the parameters used in the test program. For sections with perforations which do not meet these limits, the effective area, A_e, can be determined by stub column tests.

In the *Specification*, the web is defined as the component element of the section perpendicular to the wall and the flange is parallel to the plane of the wall.

- Delete Sections D4.2 and D4.3.

D5 Floor, Roof or Wall Steel Diaphragm Construction

Replace the second, third and fourth paragraphs of the section with the followings:

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, 1987; Department of Army, 1992; 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 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 the Standard Method for Static Load Testing of Framed Floor, Roof and Wall Diaphragm Construction for Buildings, ASTM E455. Part VI of the AISI *Design Manual* (AISI, 2002) contains the Test Procedure with Commentary on Cantilever Test Method for Cold-Formed Diaphragms. Yu (2000) provides a general discussion of structural diaphragm behavior.

The factors of safety and resistance factors listed in the *Specification* are based on a recalibration of the full-scale test data summarized in the Steel Deck Institute Diaphragm Design Manual, First Edition. The recalibration used the method of 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 A 5.1.1(b), the calibration used $\beta_0 = 3.5$ for all load effects except wind load. The US LRFD method allows $\beta_0 = 2.5$ for connections subjected to wind loads. For both welds and screws calibration using 2.5 suggests factors less severe than $\phi = 0.8$ and $\Omega = 2.0$. 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, $\phi = 0.70$ and $\Omega = 2.35$ were conservatively selected for wind loads. These values more closely equate to a calibration using $\beta_0 \geq 3.0$. 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. The factor of safety 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 (1992) 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 factor 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.

E2.2.1 Shear

This section has been reorganized and expanded. The complete section is provided as follows:

E2.2.1 Shear

E2.2.1.1 Minimum Edge Distance

The edge distance requirements provided in the *Specification* Section E2.2.1.1 are to ensure the connection provides the sufficient strength for preventing shear failure of connected part in the direction of stress. Compared with previous editions of the AISI *Specification*, the limiting F_u/F_{sy} ratio was revised to be consistent with *Specification* Section A2.3.1.

E2.2.1.2 Shear Strength [Resistance] for Sheet(s) Welded to a Thicker Supporting Member

The Cornell tests (Pekoz 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 weld. It should be noted that many failures, particularly those of the plate tearing type, may be preceded or accompanied by considerable

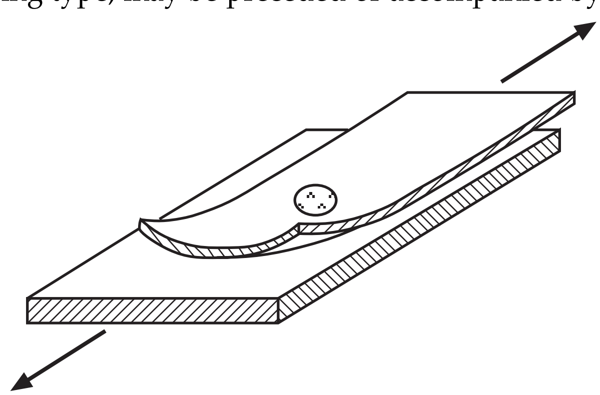

Figure C-E2.2.1.2-1 Out of Plane Distortion of Welded Connection inelastic out-of-plane deformation of the type indicated in Figure C-E2.2.1.2-1. This form of behavior is similar to that observed in wide, pin-connected plates. Such behavior should be avoided by closer spacing of welds. When arc spot welds are used to connect two sheets to a framing member as shown in *Specification* Figure E2.2.1.1-2, consideration should also be given to the possible shear failure between thin sheets.

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 E2.2.1.2-1 was revised to be consistent with the research report (Pekoz and McGuire, 1979).

In 2001, the equation used for determining d_a for multiple sheets was revised to be (d-t).

E2.2.1.3 Shear Strength [Resistance] for Sheet to Sheet Connections

The Steel Deck Institute Design Manual (SDI, 1987) 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 [resistance] be defined by *Specification* Equation E2.2.1.2-2. 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 test data thickness range that is reflected in the *Specification* documents the scope of Luttrell's test program. SDI suggests that sheet-to-sheet welds are problematic for thickness 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.

E4 Screw Connections

Revise the last sentence of the first paragraph in the section to "A higher degree of accuracy can be obtained by testing any particular connection geometry (AISI, 1992)."

E4.3.3 Shear in Screws

Revise the last sentence to "In order to prevent the brittle and sudden shear fracture of the screw, the *Specification* applies a 25 percent adjustment to the factor of safety or the resistance factor where determined in accordance with

Section F1."

E4.4.3 Tension in Screws

Revise the last sentence to "In order to prevent the brittle and sudden tensile fracture of the screw, the *Specification* applies a 25 percent adjustment to the factor of safety or the resistance factor where determined in accordance with Section F1."

E4.5 Combined Shear and Pull-Over

Add the following new section:

E4.5 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 a 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 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) $\leq t_1 \leq$ 0.0445 in. (1.13 mm)

No. 12 and No. 14 self-drilling screws with or without washers

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

62 ksi (427 MPa or 4360 kg/cm^2)$\leq F_{u1} \leq$ 70.7 ksi (487 MPa or 4970 kg/cm^2)

$t_2 / t_1 \geq 2.5$

The limit $t_2 / t_1 \geq 2.5$ 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 does not the control limit state.

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:

$$Q/P_{ns} + T/P_{nov} \leq 1.0$$

An eccentric load on a connection may create a non-uniform stress distribution around the fastener. For example, tension tests on roof panel connections have shown that under an eccentrically applied tension force the resulting connection capacity is 50% of the tension capacity under a uniformly applied tension force. Thus, the *Specification* stipulates that the pull-over strength shall be taken as 50% 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}.

F1.1 Load and Resistance Factor Design and Limit States Design

Add the following paragraph to the end of the section:

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 UMR research report (Rang and Galambos, 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.

F3.1 Full Section

On the second line to the end of the section, change "Part VIII" to "Part VI".

G. DESIGN OF COLD-FORMED STEEL STRUCTURAL MEMBERS AND CONNECTIONS FOR CYCLIC LOADING (FATIGUE)

In the second paragraph on page 143 of the *Commentary*, a reference of "(LaBoube and Yu, 1999)" is added to the end of the first sentence.

REFERENCES

- On lines 9 and 10 from top of page 159, change "(1988)" to "1987".

- Add the following references:

American Iron and Steel Institute (2004), *Standard for Cold-Formed Steel Framing – Wall Stud Design*, Washington, D. C., 2004.

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 Models for Thin-Walled Sections in Bending containing Unstiffened Elements," *Research Report R820*, Department of Civil Engineering, University of Sydney, Australia, 2002.

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, VA, 2001.

Fox, S.R. and R. M. Schuster (2002), “Bearing Stiffeners in Cold-Formed Steel C-Sections,” Final Report, American Iron and Steel Institute, Washington, D.C., 2002.

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.

Joint Departments of the Army, Navy, Air Force, USA (1992), Chapter 13, *Seismic Design for Buildings*, TM 5-809-10/NAVFACP-355/AFM 88-3, Washington, D.C., 20 October 1992.

Kulak, G.L., and G.Y. Grondin, (2001), “AISC LRFD Rules for Block Shear in Bolted Connections – A Review,” *Engineering Journal*, Fourth Quarter, 2001, American Institute of Steel Construction, IL, 2001.

LaBoube, R. A. (2001), “Arc Spot Welds in Sheet-to-Sheet Connections,” Department of Civil Engineering, University of Missouri-Rolla, MO, 2001.

LaBoube and M.A. Sokol (2002), “Behavior of Screw Connections in Residential Construction,” *Journal of Structural Engineering*, ASCE, Vol. 128, No. 1, January 2002.

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, Canada, 2002.

Luttrell, L.D. (1999), “Metal Construction Association Diaphragm Test Program,” West Virginia University, WV, 1999.

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, MO, October 2002.

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.

Yang, D. and G. J. Hancock (2003), “Compression Tests of Cold-Reduced High Strength Steel Channel Columns Failing in the Distortional Mode.” University of Sydney, Department of Civil Engineering, Research Report R825, Australia.

Yiu, F., and T. Pekoz (2001), "Design of Cold-Formed Steel Plain Channels. Final Report," Cornell University, Ithaca, NY, 2001.

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, MO, 2000.

Zwick, K. and R. A. LaBoube (2002), “Self-Drilling Screw Connections Subject to Combined Shear and Tension,” Wei-Wen Yu Center for Cold-Formed Steel Structures, University of Missouri-Rolla, MO, 2002.

CHANGES AND UPDATES IN APPENDICES A AND C

A2.2 Other Steels

Replace the whole section with the following:

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 this provision.

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, A1.2. 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 accord 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 point 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 point 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 A2.3.

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.

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

Revise the second to the last sentence of the section to "In *Specification* Equation C3.1.4-1, the reduction factor, R, can be determined by AISI TS-6-04 published by AISI (AISI, 2004).

C4.7 Compression of Z-Section Members Having One Flange Fastened to a Standing Seam Roof

Add the following new section:

C4.7 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 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 C4.7-1 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 TS-8, Base Test Method for Purlins Supporting a Standing Seam Roof System, as published by AISI. This equation, developed by Stolarczyk (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 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.

Equation C4.7-1 may be used with the results of uplift "Base Tests" conducted with and without discreet point bracing. There is no limitation on the minimum length because Equation C4.7-1 is conservative for spans that are smaller than that 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 Section C4 and C4.1 should also be checked for buckling about the strong axis.

E2a Welded Connections

- Add the following as the first paragraph of the section:

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).

- Change "0.18 in. (4.57 mm)" to "3/16 in. (4.76 mm)" in the last paragraph.

E3a Bolted Connections

Add the following content to the end of the last paragraph: "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. Recent 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 in the direction of stress can develop the full moment in the lap."

E5 Rupture

Add the following as a new paragraph to the end of the section:

The summary 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. In 2004, Equations E5.3-1 and E5.3-2 were adopted for the limit state of block shear rupture for bolted cold-formed steel connections because eccentricity in cold-formed steel sections is usually small. In theory provisions for block shear could also be applied to screw connections. However, because the final placement location of self-drilling screws cannot be assured, a block shear check is of little significance. Also, tests performed at the University of Missouri-Rolla have indicated that the current design equations for shear and tilting provide a reasonably good estimate of the connection performance for multiple screws in a pattern (LaBoube and Sokol, 2002).

CHANGES AND UPDATES IN APPENDIX B

A2.4a Delivered Minimum Thickness

Delete the entire section.

A3 Loads

Add the following as a new section:

A3 Loads

The load provisions contained in Appendix B of CSA S136-01 were changed to be compatible with the changes to be incorporated in Part 4 of the *National Building Code of Canada* (NBC) 2005. This entails the following:

(1) The version of Limit States Design to appear in NBC 2005 is based on the companion action format, which is being adopted world-wide 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. It was decided to harmonize the approach used, and so the importance factor methodology was chosen for snow, wind and earthquake loads.

C2.2 Fracture of Net Section

Add the following paragraph to the end of the section:

The provision regarding block tear-out of Section C2.2 was rewritten in accordance with the latest research by Kulak and Grondin (2001). A new section on coped beams was also added as per the recommendations by these authors. Based on the actual test data, an additional correction was made by only applying the 0.9 factor to the direct tension and stagger failure paths. In CSA S136-01, the 0.9 factor was also applied to the shear failure path.

COMMENTARY ON APPENDIX 1, DESIGN OF COLD-FORMED STEEL STRUCTURAL MEMBERS USING THE DIRECT STRENGTH METHOD

The following commentary is newly added.

American Iron and Steel Institute

Commentary on Appendix 1

Design of Cold-Formed Steel Structural Members Using the Direct Strength Method

2004 EDITION

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COMMENTARY ON APPENDIX 1
DESIGN OF COLD-FORMED STEEL STRUCTURAL MEMBERS
USING THE DIRECT STRENGTH METHOD
2004 EDITION

1.1 GENERAL PROVISIONS

1.1.1 Applicability

The Direct Strength Method of Appendix 1 is an alternative procedure for determining the strength [resistance] and stiffness of cold-formed steel members (beams and columns). The reliability of Appendix 1 is insured 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 [resistance]. The fact that one method may give a greater or lower strength [resistance] prediction in a given situation does not imply an increased accuracy for either method. The Ω and ϕ factors are designed to insure that both methods reach their target reliability.

The method of Appendix 1 provides solutions for beams and columns only, but these solutions must be combined with the regular provisions of the main *Specification* to cover other cases: shear, beam-columns, etc. For example, an application to purlin design was completed using the provisions of this Appendix for the bending strength, and then those calculations were augmented by shear, and shear and bending interaction calculations, in line with the main *Specification* (Quispe and Hancock, 2002). Further, beam-columns may be conservatively examined using the provisions of the main *Specification*, by replacing the beam and column design strength [factored resistance] with the provisions of this Appendix, or beam-columns may be analyzed using the actual stress state (Schafer, 2002b).

The pre-qualified columns and beams only include members without perforations (punchouts). Members with perforations generally may be designed by the main *Specification*. For perforated members not covered by the *Specification* one may want to consider a rational analysis method which partially employs the methods of this Appendix. The key issue in such a rational analysis is the accurate determination of the elastic local, distortional, and global buckling loads (or moments) for the member with perforations. Numerical (e.g., finite element) analysis where the holes are explicitly considered is one option in this case.

Note:

* The *North American Specification for the Design of Cold-Formed Steel Structural Members*, Chapters A through G and Appendices A through C, is herein referred to as the main *Specification*.

1.1.1.1 Pre-qualified 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.

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 lower Ω and higher ϕ factors for a particular cross-section. Alternatively, member geometries that are not pre-qualified 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 A1.1 of the main *Specification*.

1.1.1.2 Pre-qualified 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. Please see the note on pre-qualified columns for further commentary on members which do not meet the pre-qualified geometric limits.

Users of this Appendix should be aware that pre-qualified 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 3 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 insures 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)

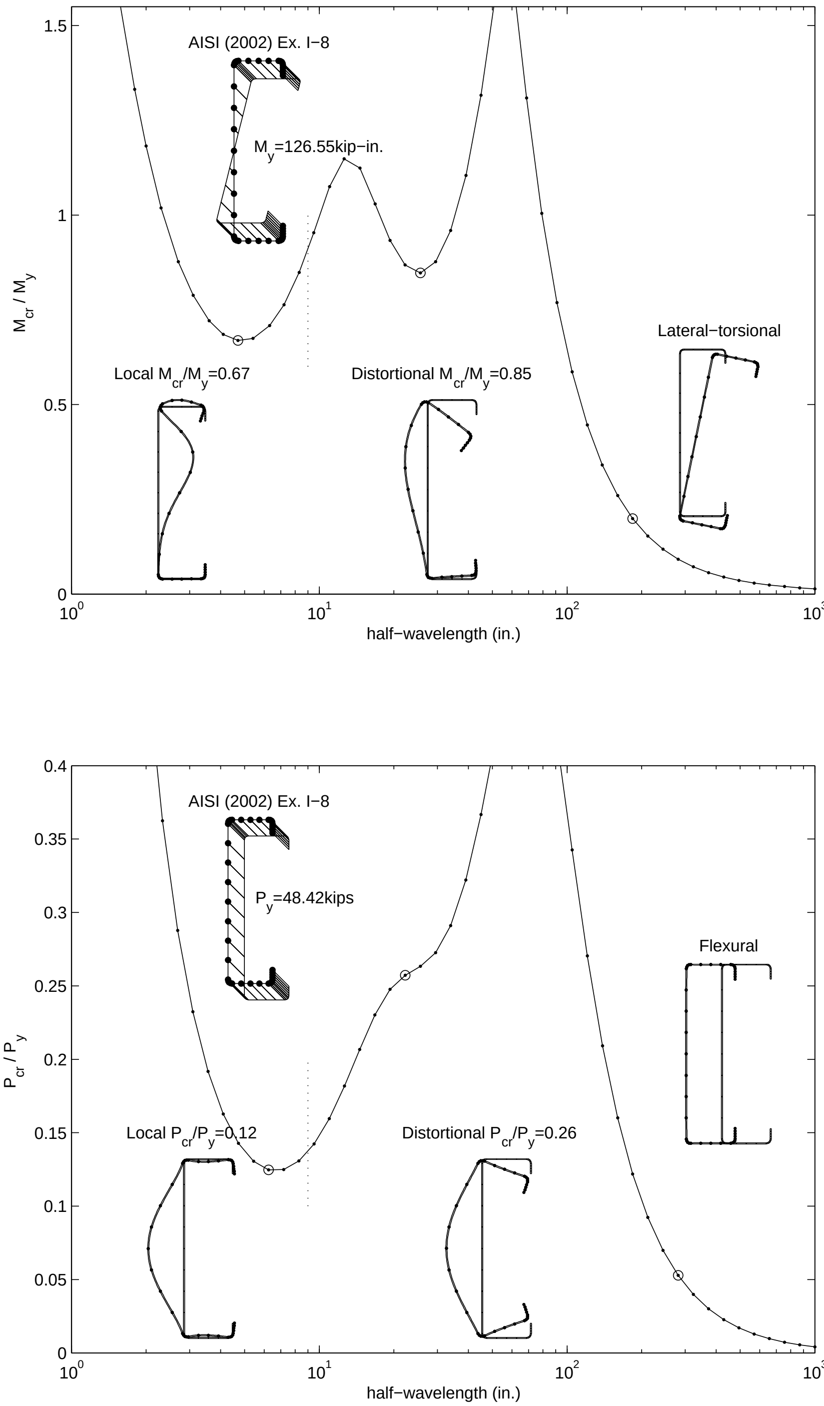
flange: k = 4, $f_{cr\ell\text{-flange}} = 4.0[\pi^2E/(12(1-\mu^2))](t/b)^2$ = 62.4 ksi (430 MPa)

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).

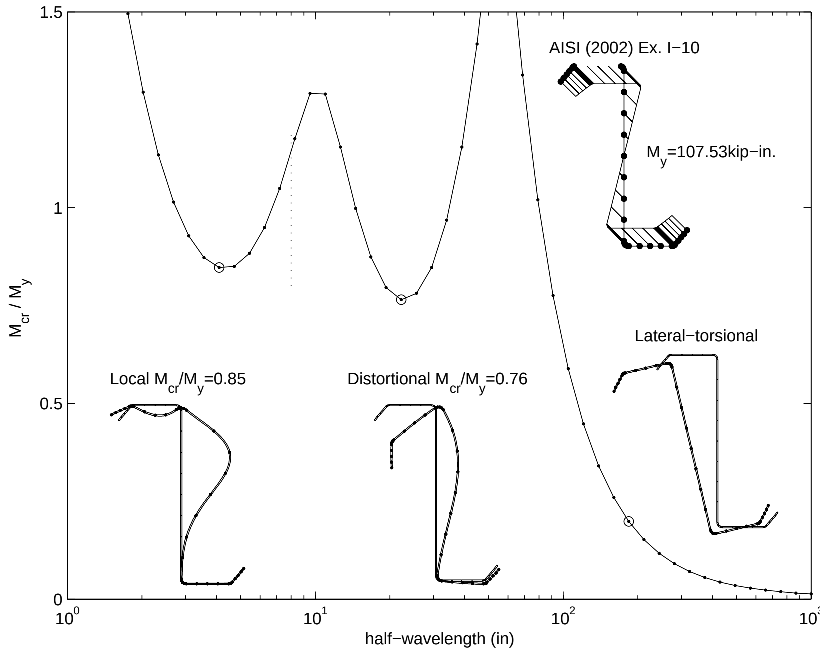
The method of this Appendix allows rational analysis to be used for determining the local, distortional and global buckling load or moment. Specific guidance on elastic buckling determination follows. Users are reminded that the strength of a member is not equivalent to the elastic buckling load (or 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.

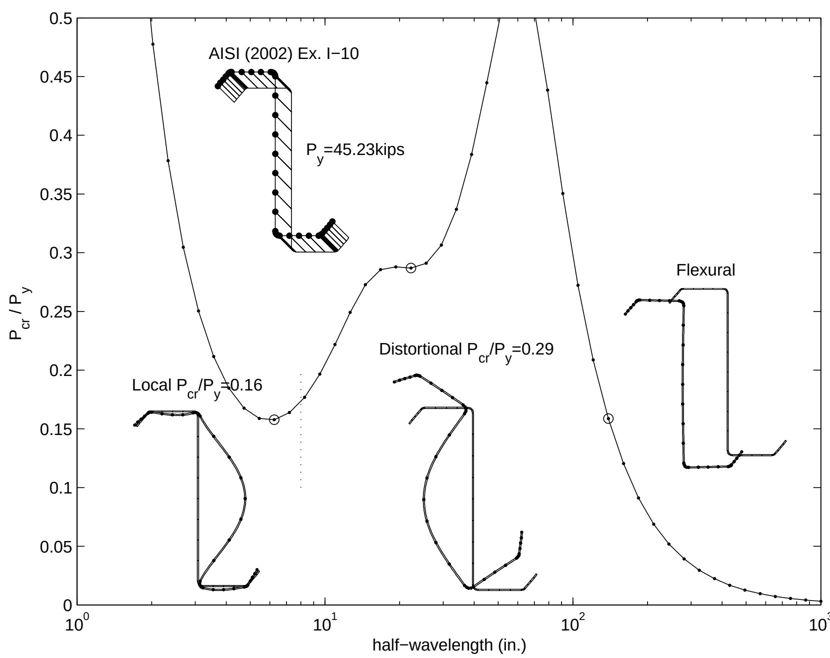
Manual and numerical solutions for elastic buckling prediction are covered in the following sections. It is permissible to mix the manual and numerical methods; in some cases it is even advantageous. For example, numerical solutions for member local and distortional buckling are particularly convenient; 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.



(a) 9CS2.5x059 of AISI 2002 Cold-Formed Steel Design Manual Example I-8

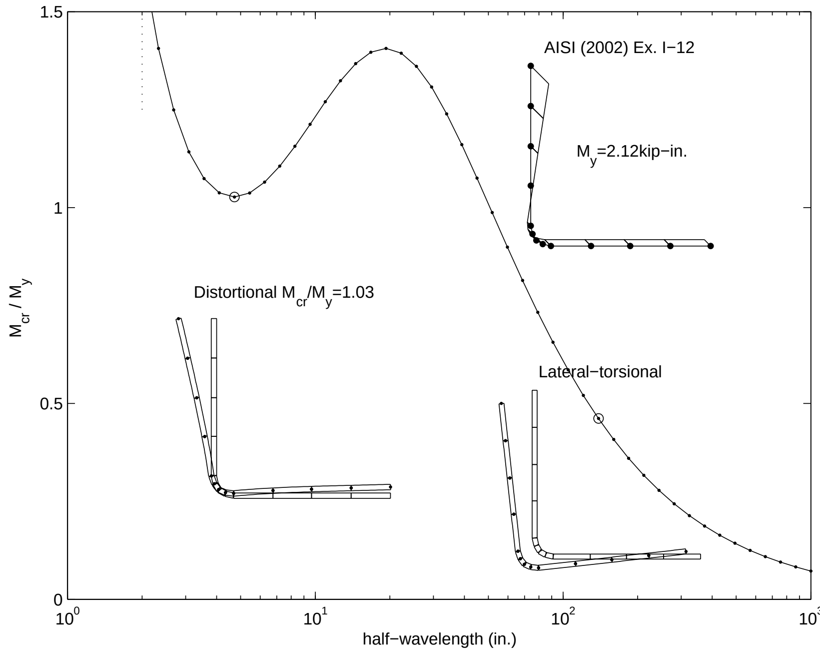
Figure C-1.1.2-1 Examples of Bending and Compression Elastic Buckling Analysis with Finite Strip Method

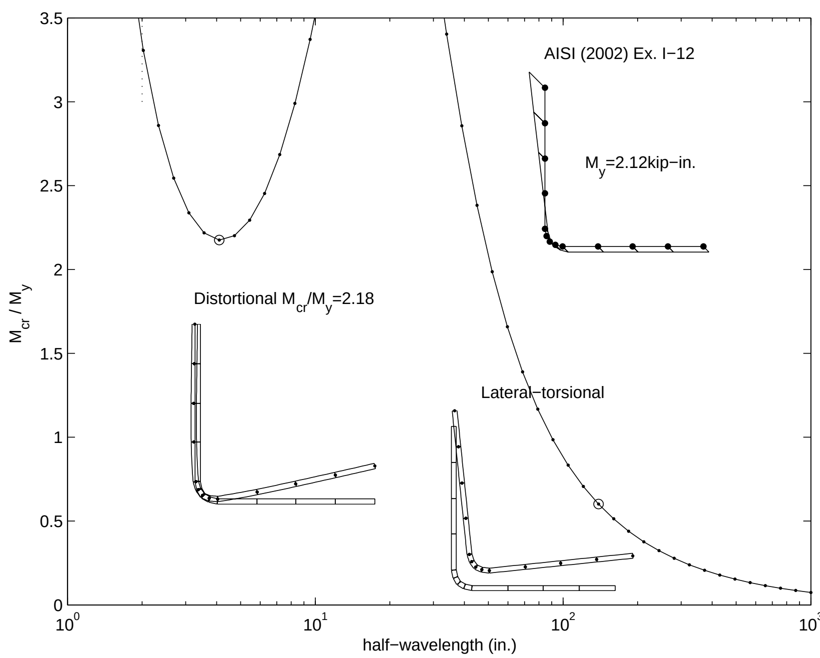




(b) 8ZS2.25x059 of AISI 2002 Cold-Formed Steel Design Manual Example I-10

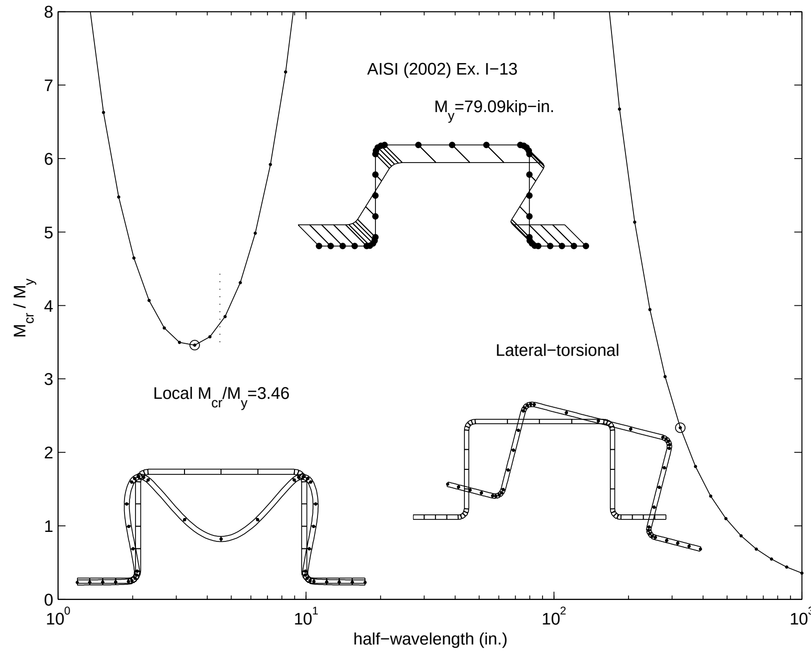
Figure C-1.1.2-1 Examples of Bending and Compression Elastic Buckling Analysis with Finite Strip Method (cont.)

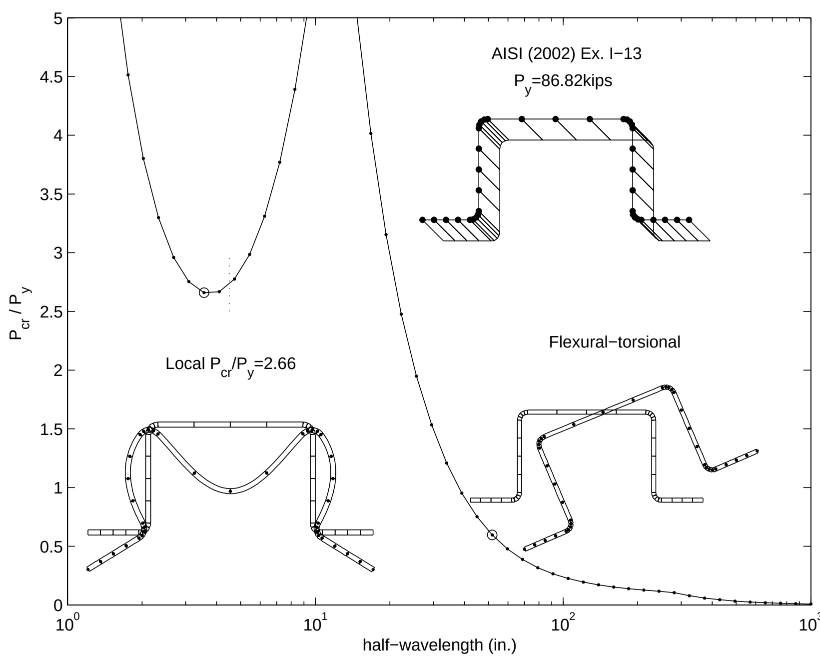




(c) 2LU2x060 of AISI 2002 Cold-Formed Steel Design Manual Example I-12

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.)

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 books 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) explains the basic theory while Hancock et al. (2001) and Schafer (1997) provide specific details for stability analysis with this method. Hancock and his researchers (see Hancock et al., 2001 for full references and descriptions) 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 implemented in conventional programs, does have limitations, the two most important ones are

- the model assumes the ends of the member are simply supported, and
- 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 advance over plate buckling solutions and plate buckling coefficients (k's) 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 any PC with Windows 9x, NT, 2000, XP. Tutorials and examples are available online at the same address.

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.

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 predominately 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 [resistance] 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.

1.1.2.1.3 Global (Euler) Buckling via Finite Strip (P_{cre}, M_{cre})

Global bucking 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 for columns, lateral-torsional 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.

1.1.2.2 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,

$$P_{cr\ell} = A_g f_{cr\ell} \quad \text{(C-1.1.2-1)}$$

A_g = gross area

$f_{cr\ell}$ = local buckling stress

For beams,

$$M_{cr\ell} = S_g f_{cr\ell} \quad \text{(C-1.1.2-2)}$$

S_g = gross section modulus to the extreme compression fiber

$f_{cr\ell}$ = local buckling stress at the extreme compression fiber

and

$$f_{cr\ell} = k\frac{\pi^2 E}{12(1-\mu^2)}\left(\frac{t}{w}\right)^2 \quad \text{(C-1.1.2-3)}$$

where

E = Young's Modulus

μ = Poisson's ratio

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, 2002); and local buckling of an edge stiffened element, including lip/flange interaction (Schafer and Peköz, 1999).

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,

$$P_{crd} = A_g f_{crd} \qquad \text{(C-1.1.2-3)}$$

A_g = gross area of the member

f_{crd} = distortional buckling stress (see below)

For beams,

$$M_{crd} = S_f f_{crd} \qquad \text{(C-1.1.2-4)}$$

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.

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 main *Specification* Sections C4 and C3 for additional details.

For columns,

$$P_{cre} = A_g f_{cre} \qquad \text{(C-1.1.2-5)}$$

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 through C4.4 provide all necessary formula. Note, C4.4 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 C4.4 as well as Yu (2000) or Hancock et al. (2001). The hand calculations may be quite lengthy, particular if member properties x_o and C_w are unknown.

For beams,

$$M_{cre} = S_f f_{cre} \quad \text{(C-1.1.2-6)}$$

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.

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 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 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) 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 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 strength, P_{ne}, follows the same practice as the main *Specification* and uses the current AISC (2001) curves for strength prediction. The main *Specification* provides the long column strength in terms of a stress, F_n (Equations C4-2 and C4-3). In the Direct Strength Method this is converted from a stress to a strength by multiplying the gross area, A_g, resulting in the formulas for P_{ne} given in Appendix 1.

In the main *Specification*, column strength is calculated by multiplying the long column 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 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 provisions for columns are reported in Schafer (2000, 2002). The reliability of the column provisions was determined using the test data of Appendix 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 Section 1.2.1 were determined for the pre-qualified 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 pre-qualified members is relatively large, extensions of the Direct Strength Method to geometries outside the pre-qualified 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 analysis provisions of A1.1(b) 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 below. 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

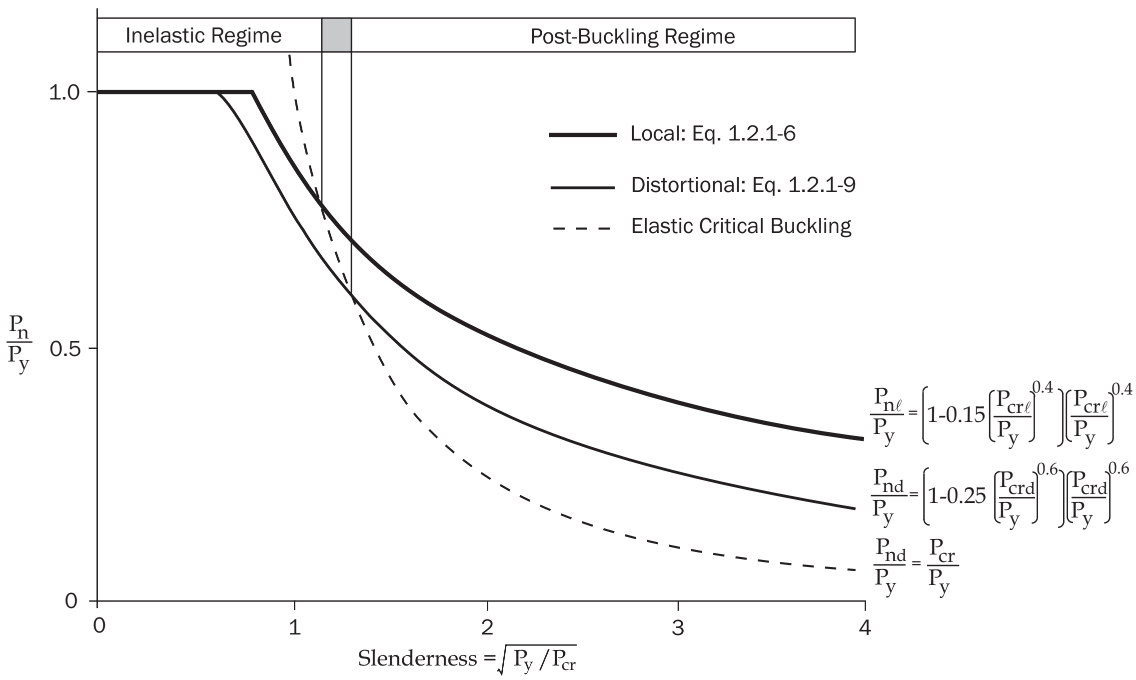


Figure C-1.2.1-1 Local and Distortional Direct Strength Curves for a Braced Column ($P_{ne} = P_y$)

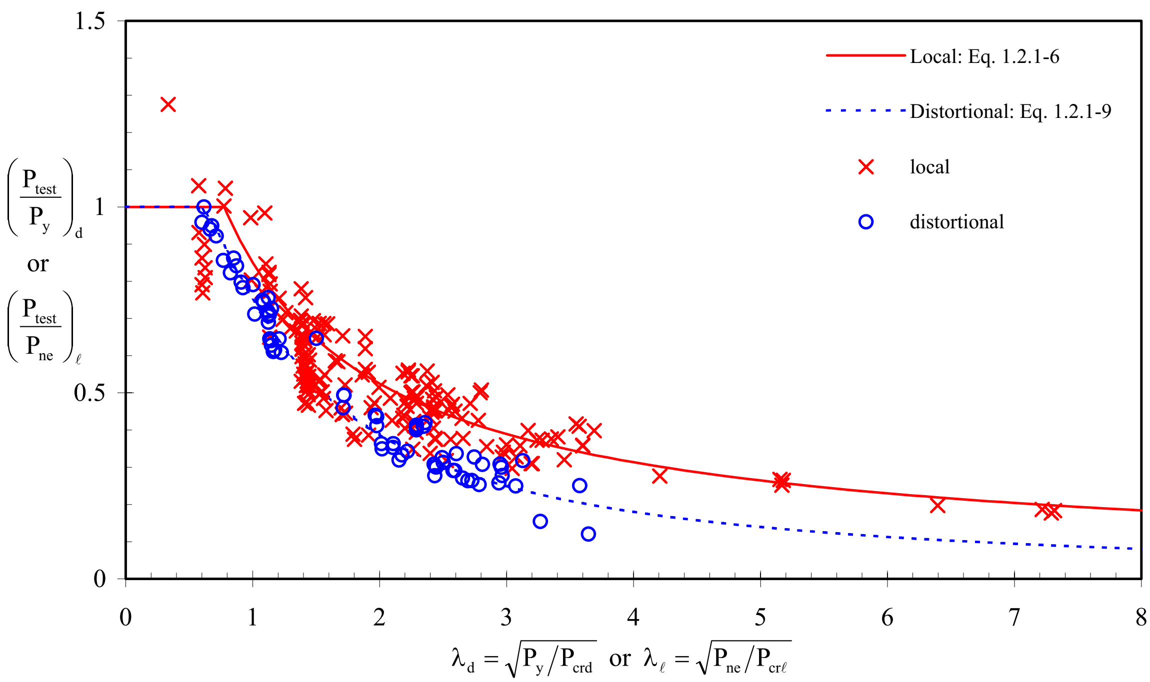


Figure C-1.2.1-2 Direct Strength Method for Concentrically Loaded Pin-Ended Columns

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*. Since the main *Specification* has no rules for distortional buckling, the Direct Strength Method actually provides better agreement than the main *Specification* when compared with this test database for many members.

1.2.1.1 Flexural, Torsional, or Torsional-Flexural 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 analysis methods for calculation of P_{cre}.

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 analysis methods for calculation of $P_{cr\ell}$.

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-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}.

1.2.2 Beam Design

Commentary Section C3 provides a complete discussion on the behavior of cold-formed 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 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 (Equations C3.1.2.1-2, -3, and -4). 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 the formulas for M_{ne} 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-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 ($M_{n\ell}$).

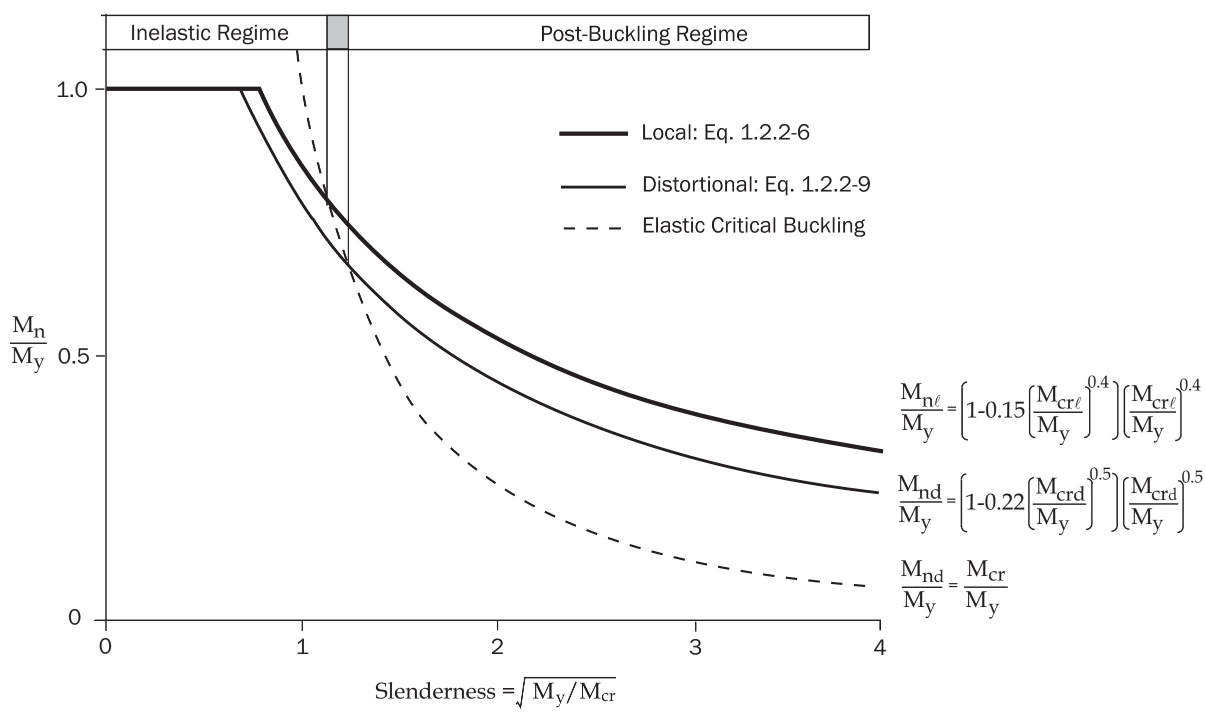


Figure C-1.2.2-1 Local and Distortional Direct Strength Curves for a Braced Beam ($M_{ne} = M_y$)

The strength curves for local and distortional buckling of a fully braced beam are presented in Figure C-1.2.2-1 and compared to the critical elastic buckling curve. While the strength in both the local and distortional modes exhibit both an inelastic regime and a post-buckling regime, the post-buckling reserve for the local mode is predicted to be greater than that of the distortional mode.

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 the investigated beams. Based on this information the safety and resistance factors of Section 1.2.2 were determined for the pre-qualified members. For the United States and Mexico ϕ = 0.90; while for Canada ϕ = 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 pre-qualified members is relatively large, extensions of the Direct Strength Method to geometries outside the pre-qualified set is allowed. However, given the uncertain nature of this extension, increased safety factors and reduced resistance factors are applied in that case, per the rational analysis provisions of A1.1(b) of the main *Specification*.

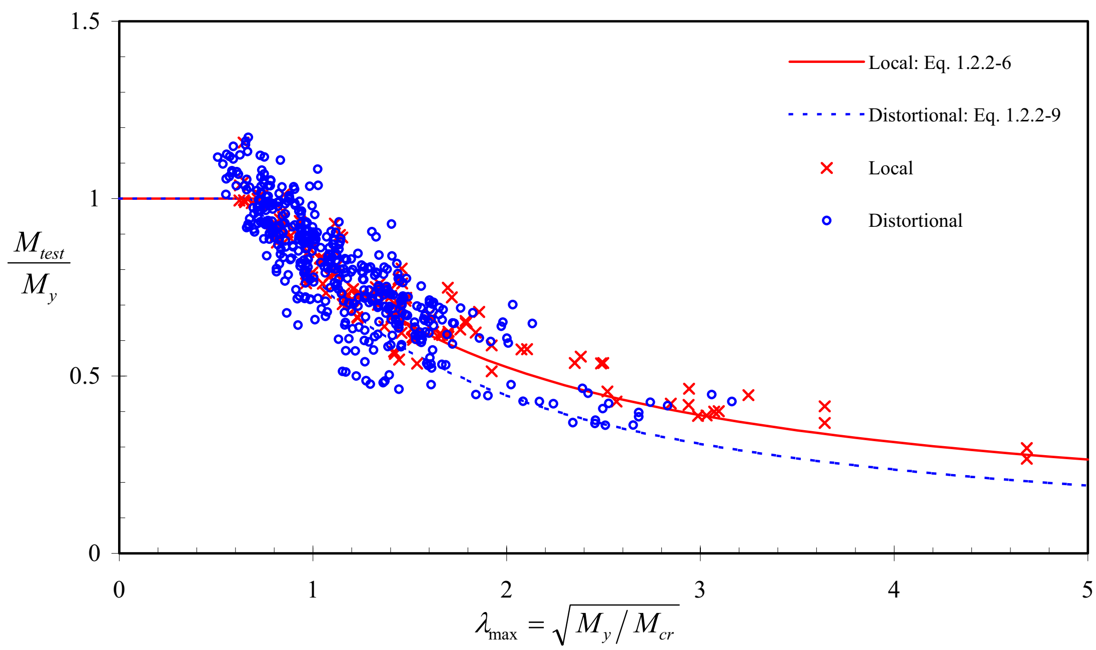


Figure C-1.2.2-2 Direct Strength Method for laterally braced beams

The provisions of Appendix 1, applied to the beams of Section 1.1.1.2, are summarized in Figure C-1.2.2-2. The controlling strength is determined either by Section 1.2.2.2, which considers local buckling interaction with lateral-torsional buckling, or by Section 1.2.2.3, 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*, and since the main *Specification* has no rules for distortional buckling, the Direct Strength Method actually provides better agreement than the main *Specification* when compared with this test database for many members.

1.2.2.1 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 bending elastic buckling strength, M_{ne}, calculated in this section represents the upperbound 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 analysis methods for calculation of M_{cre}.

1.2.2.2 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 is 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 lateral-torsional buckling strength, 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 analysis methods for calculation of $M_{cr\ell}$.

1.2.2.3 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 distortional buckling strength 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}.

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