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Cold-formed Steel Design Manual

American Iron and Steel Institute

COLD-FORMED STEEL DESIGN MANUAL

AMERICAN IRON AND STEEL INSTITUTE

COLD-FORMED STEEL DESIGN MANUAL

AMERICAN IRON AND STEEL INSTITUTE
1000 16th STREET, NW
WASHINGTON, DC 20036

1st Printing—March 1987

Produced by W. P. Reyman Associates, Inc. New York

PREFACE

This edition of the *Cold-Formed Steel Design Manual* is based on the *August 19, 1986, Edition of the Specification for the Design of Cold-Formed Steel Structural Members*. The *Manual* includes the following sections:

Part I—*Specification*

Part II—*Commentary*

Part III—*Supplementary Information*

Part IV—*Illustrative Examples*

Part V—*Charts and Tables*

Part VI—*Computer Aids*

Part VII—*Test Procedures*

The *Specification* and the *Commentary* are both also available as separately bound booklets.

American Iron and Steel Institute gratefully acknowledges the time and effort devoted to the preparation of the *Manual* by the Advisory Group on the Specification for the Design of Cold-Formed Steel Structural Members and its working subcommittees. A special thanks go to the following: the Cold-Formed Steel Design Manual Subcommittee—D. L. Johnson, Chairman, S. J. Errera, R. B. Haws, Herbert Klein, R. A. LaBoube, A. L. Johnson, and C. R. Clauer, Project Manager; the Editorial Subcommittee—C. W. Pinkham, Chairman, C. R. Clauer, D. A. Cuoco, S. J. Errera, A. L. Johnson, and T. B. Pekoz; W. R. Midgley, Clauer & Associates; and Jon Harrington and Associates.

American Iron and Steel Institute
August 1986

American Iron and Steel Institute

1133 15th STREET, N.W., WASHINGTON, D.C. 20005-2701 Phone (202) 452-7100 Fax (202) 463-6573

July 25, 1991

RE: Cold-Formed Steel Design Manual

Dear Friend of Cold-Formed Steel:

Enclosed please find a complete 1986 edition of the Cold-Formed Steel Design Manual which consists of:

Part I	-	1986 Edition of the Specification for the Design of Cold-Formed Steel Structural Members
Part II	-	Commentary to the 1986 Edition of the Specification for the Design of Cold-Formed Steel Structural Members
Part III	-	Supplementary Information
Part IV	-	Illustrative Examples
Part V	-	Charts and Tables
Part VI	-	Computer Aids
Part VII	-	Test Procedures

Also enclosed is a complimentary loose-leaf copy of the 1989 Addendum to the 1986 Edition of the Specification for the Design of Cold-Formed Steel Structural Members and its Commentary. The 1989 Addendum incorporates the results of research completed since the 1986 edition was issued. The Specification sections involving the more significant changes in the 1989 Addendum are:

A3	Material
C3.1.2	Lateral Buckling Strength
C3.1.2	Beams Having One Flange Through-Fastened to Deck or Sheathing
D4	Wall Studs and Wall Stud Assemblies
D5	Floor, Roof or Wall Steel Diaphragm Construction (New Section)
E2.2	Arc Spot Welds
E2.6	Resistance Welds

In an effort to make the Specification and Manual more responsive to its users, a questionnaire has been included. Please take a few minutes and complete the survey and return it to AISI. Your input is important in determining the content of future Specifications. Thank you for your interest in cold-formed steel design.

Sincerely yours,

Richard B. Haws, P.E.
Secretary
Committee on Specifications

RBH/klc

AMERICAN IRON AND STEEL INSTITUTE
1133 15th Street, NW
Washington, DC 20005

SURVEY OF USERS OF AISI "COLD-FORMED STEEL DESIGN MANUAL"

Please indicate the usefulness to you of the material in the COLD-FORMED STEEL DESIGN MANUAL by using the following rating system:

1 - Very Useful
2 - Somewhat Useful
3 - Present Version not Useful
4 - No Interest; have not reviewed material

Part I	**Specification**	1	2	3	4
Part II	**Commentary**	1	2	3	4
Part III	**Supplementary Information**				
	Section 1 - Linear Method	1	2	3	4
	Section 3 - Laterally Unbraced Compression Flanges	1	2	3	4
	Section 4 - Torsional - Flexural Buckling	1	2	3	4
Part IV	**Illustrative Examples**				
	Flexural Members - Examples 1-9	1	2	3	4
	Compression Members - Examples 10-14	1	2	3	4
	Beam-Column Members - Examples 15-17	1	2	3	4
	Tension Members & Connections - Examples 18-22	1	2	3	4
	Purlins - Examples 23-24	1	2	3	4
	Calculation of Section Properties - Example 25	1	2	3	4
Part V	**Charts and Tables**				
	Torsional-Flexural Design Charts	1	2	3	4
	Full Area Tables - Tables 1-9	1	2	3	4
	Effective Area Tables - Tables 10-15	1	2	3	4
Part VI	**Computer Aids (flow charts)**				
	Properties of Elements - Charts B2.1-B6.2	1	2	3	4
	Strength of Members - Charts C3.1-C5	1	2	3	4
	Capacity of a Wall Stud - Chart D4.1	1	2	3	4
	Connections - Charts E2.2-E3.2	1	2	3	4
Part VII	**Test Procedures**				
	Rotational-Lateral Stiffness	1	2	3	4
	Stub-Column	1	2	3	4

COMMENTS: Please use the back of this sheet to comment on the following questions.

1. What software do you use in applying these specifications?
2. What software would you desire related to these specifications?
3. What suggestions do you have for additional examples?
4. What suggestions do you have for improving the specification and manual?
5. Would additional section properties and load span tables be useful? **Y** **N**
   If yes, please list section profiles on reverse of this page.

Name:____

Address:____

____

Please return to the above address, attention Richard B. Haws.

American Iron and Steel Institute

1133 15TH STREET, N.W., WASHINGTON, D.C. 20005-2701

November 30, 1990
File No. SG-671

SUBJECT: Errata to the 1986 Cold-Formed Steel Design Manual

Dear Friend of Cold-Formed Steel:

The AISI Cold-Formed Steel Advisory Group has completed the errata to the 1986 Edition of the Specification for the Design of Cold-Formed Steel Structural Members, its Commentary and to the Cold-Formed Steel Design Manual. These updates are a result of user comments and questions. Please incorporate these revisions into your copies of the Specification, Commentary and Manual.

All users of the Specification, Commentary and Manual are invited to continue to offer their valuable comments and suggestions.

Sincerely yours,

Richard B. Haws

Richard B. Haws, P.E.
Program Manager
Cold-Formed Steel Construction

RBH/klc

Attachment

Errata to the 1986 AISI Cold–Formed Specification and Commentary

February 3, 1990
File No. SG671/2E

Page	Section	Line Number / Reference		Revision
I–8	Symbols and Definitions	12 from bottom	Change:	E Modulus of elasticity of steel (29,000 ksi)
			To:	E Modulus of elasticity of steel (29,500 ksi)
I–10	Symbols and Definitions	6 from bottom–I_{yc}	Change:	D3.1.1
			To:	C3.1.2
I–14	Symbols and Definitions	above Ω_w	Insert:	Ω_v Factor of safety for shear rupture E4
I–21	B2.1	1 from top	Change:	(3) If Section C3.1.2 is used, then the $f = \frac{M_c}{S_f}$
			To:	(3) If Section C3.1.2 is used, then $f = \frac{M_c}{S_f}$
I–23	B4	8 from bottom	Change:	C_1, C_2 = Coefficients defined in Figures B4–1 and B4–2
			To:	C_1, C_2 = Coefficients defined in Figure B4–2
I–27	B6.1	13 from bottom	Change:	the yield stress, f_y, and t_s the thickness of the stiffener
			To:	the yield stress, F_y, and t_s the thickness of the stiffener
I–28	B6.2	*Eq.* B6.2–4	Change:	$C_v = \frac{190}{h/t}\left(\frac{k_y}{\sqrt{F_y}}\right)$ when $C_v > 0.8$
			To:	$C_v = \frac{190}{h/t}\left(\sqrt{\frac{k_v}{F_y}}\right)$ when $C_v > 0.8$
I–30	C3.1.1	5 from top	Change:	Combined bending and web crippling shall be checked by provisions of Section C3.4
			To:	Combined bending and web crippling shall be checked by provisions of Section C3.5
I–32	C3.1.2	4 from top	Change:	bending moment (Section C5), C_b shall be taken as unity.
			To:	bending moment (Section C5), C_{TF} shall be taken as unity.
I–37	C5	21 from bottom M_{nxo} and M_{nyo}	Change:	Section C3.12 (lateral buckling)
			To:	Section C3.1.2 (lateral buckling)
I–41, I–42	D3.2.1	*Eq.* D3.2.1–1, *Eq.* D3.2.1–2, *Eq.* D3.2.1–3, *Eq.* D3.2.1–4, *Eq.* D3.2.1–5, *Eq.* D3.2.1–6	Change:	tan θ
			To:	sin θ

Errata to the 1986 AISI Cold–Formed Specification and Commentary

February 3, 1990
File No. SG671/2E

Page	Section	Line Number / Reference	Revision	
I–58	F1	*Eq.* F1–4	Change:	$R = 2.5D + 2.5L$
			To:	$R \geq 2.5D + 2.5L$
I–60	Appendices	1 from top	Change:	APPENDICIES
			To:	APPENDICES
I–61	Appendix B1.1(b)	*Eq.* B1.1b–1	Change:	$w_f = \sqrt{0.061tdE / f_{av}}\sqrt{(100c_f / d)}$
			To:	$w_f = \sqrt{0.061tdE / f_{av}}\sqrt[4]{(100c_f / d)}$
II–31	Commentary F	11 from bottom	Change:	Information on test
			To:	Information on tests

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Page	Section	Line Number / Reference		Revision	
III–13	1.2.3–9	14 from top	Change:	$\bar{c}$ = length of lip, see Figure 1.2.3–1	
			To:	$\bar{c}$ = length of lip, see Figure 1.2.2–1	
III–15	2.1	12 from top	Change:	$\sigma_t = \frac{\sigma_{to}\bar{a}^2}{t^2}\left(\frac{t}{\bar{\alpha}}\right)^2 + \frac{C_T}{\bar{\alpha}^2}\left(\frac{\bar{a}}{K_t L}\right)^2$	
			To:	$\sigma_t = \frac{\sigma_{to}\bar{a}^2}{t^2}\left(\frac{t}{\bar{a}}\right)^2 + \frac{C_T}{\bar{a}^2}\left(\frac{\bar{a}}{K_t L}\right)^2$	
IV–19	Example 4	14 from top	Change:	$D = d + 0.124 \tan(\theta/2) = 0.600 + 0.124 \tan(45°/2) = 0.651$	
			To:	$D = d + 0.154 \tan(\theta/2) = 0.600 + 0.154 \tan(45°/2) = 0.664$	
		21 from top	Change:	$D/w = 0.651 / 1.471 = 0.443,\ 0.25 < D/w = 0.443 < 0.8$	
			To:	$D/w = 0.664 / 1.471 = 0.451,\ 0.25 < D/w = 0.451 < 0.8$	
		24 from top	Change:	$k = [4.82 - 5(D/w)](I_s/I_a)^n + 0.43 = [4.82 - 5(0.443)](1.082)^{1/2} + 0.43 = 3.139$ $k \le 5.25 - 5(D/w) = 5.25 - 5(0.443) = 3.035$ $k = 3.035$	
			To:	$k = [4.82 - 5(D/w)](I_s/I_a)^n + 0.43 = [4.82 - 5(0.451)](1.082)^{1/2} + 0.43 = 3.097$ $k \le 5.25 - 5(D/w) = 5.25 - 5(0.451) = 2.994$ $k = 2.994$	
		16 from bottom	Change:	$= (1.052/\sqrt{3.035})(24.52)\sqrt{50/29500} = 0.609 < 0.673$	
			To:	$= (1.052/\sqrt{2.994})(24.52)\sqrt{50/29500} = 0.614 < 0.673$	
IV–24	Example 4A	5 from top	Change:	$k = 4 + 2(1-\psi)^3 + 2(1-4)$ $= 4 + 2[i - (-1.000)]^3 + 2[i - (-1.000)]$	
			To:	$k = 4 + 2(1-\psi)^3 + 2(1-\psi)$ $= 4 + 2[1 - (-1.000)]^3 + 2[1 - (-1.000)]$	
		8 from bottom	Change:	$0.064 \cos 45° + (0.600/2)\cos 45° = 0.257$	0.154
			To:	$(0.154 - 0.124 \cos 45°) + (0.600/2) \sin 45° = 0.278$	0.167
		3 from bottom	Change:	$9.5 - 0.257 = 9.243$	5.546
			To:	$9.5 - 0.278 = 9.221$	5.533

Errata to the 1986 AISI Cold–Formed Steel Design Manual

Page	Section	Line Number / Reference		Revision
IV–33	Example 5	top line	Change:	$V_a = (11800F_y/E)ht$ $= 11800(50/29500)(3.692)(0.06)$
			To:	$V_a = 0.4F_y ht$ $= (0.4)(50)(3.692)(0.06)$
		bottom line	Change:	Find I_{eff} at M_a = 31.3 kip–in.
			To:	Find I_{eff} at M_a = 31.1 kip–in.
IV–38	Example 5	entire page	Change:	Replace with replacement page
IV–46	Example 6	22 from top	Change:	$M_f = 0.6F_y = S_{eff}(0.6F_y)$
			To:	$M_f = S_{eff}(0.6F_y)$
IV–49	Example 7	drawing at bottom	Change:	2.629″
			To:	1.629″
IV–50	Example 7	top line	Change:	$f_1 = (1.048/2.629)(50) = 19.93$ ksi (compression) $f_2 = (1.306/2.629)(50) = 24.84$ ksi (tension) $\psi = f_1/f_2 = -24.84/19.93 = -1.246$
			To:	$f_1 = (1.048/1.629)(50) = 32.17$ ksi (compression) $f_2 = (1.306/1.629)(50) = 40.09$ ksi (tension) $\psi = f_1/f_2 = -40.09/32.17 = -1.246$
		7 from top	Change:	$\lambda = (1.052/\sqrt{k})(w/t)\sqrt{f/E}, \quad f = f_1$ $= (1.052/\sqrt{31.15})(17.44)\sqrt{19.93/29500}$ $= 0.085$
			To:	$\lambda = (1.052/\sqrt{k})(w/t)\sqrt{f/E}, \quad f = f_1$ $= (1.052/\sqrt{31.15})(17.44)\sqrt{32.17/29500}$ $= 0.108$
		20 from top	Change:	$S_e = I_x/(d - y_{cg}) = 2.47/(4 - 1.731) = 1.09$ in.^{3} $M_n = S_e F_y$ $= (1.09)(50)$ $= 54.5$ kip–in.
			To:	$S_e = I_x/(d - y_{cg}) = 2.47/(3 - 1.371) = 1.516$ in.^{3} $M_n = S_e F_y$ $= (1.516)(50)$ $= 75.8$ kip–in.

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Page	Section	Line Number / Reference		Revision
IV–51	Example 7	17 from bottom	Change:	M_n shall not exceed 1.25 $S_e F_y$ = 1.25(54.5) = 68.1 kip–in. Therefore $M_n = S_e F_y$ = 54.5 kip–in.
			To:	M_n shall not exceed 1.25 $S_e F_y$ = 1.25(75.8) = 94.8 kip–in. Therefore M_n = 1.25 $S_e F_y$ = 94.8 kip–in.
		3 from bottom	Change:	$M_a = M_n / \Omega_f$ = 68.1 / 1.67 = 40.8 kip–in.
			To:	$M_a = M_n / \Omega_f$ = 94.8 / 1.67 = 56.7 kip–in.
IV–69	Example 8	6 from top	Change:	For interior reaction: *Eq.* C3.4–3
			To:	For interior reaction: *Eq.* C3.4–4
IV–70	Example 9	entire page	Change:	Replace with replacement page
IV–71	Example 10	13 from bottom	Change:	a
			To:	$\bar{a}$
		12 from bottom	Change:	b
			To:	$\bar{b}$
		11 from bottom	Change:	c
			To:	$\bar{c}$
IV–73	Example 10	14 from top – #13	Change:	$f_y/2$ = 50/2 = 25.00 ksi
			To:	$F_y/2$ = 50/2 = 25.00 ksi
		15 from bottom–#14	Change:	(*Eq.* B4.3–4)
			To:	(*Eq.* B4.2–4)
		8 from bottom–#15	Change:	15. Determination of ρ_a:
			To:	15. Determination of P_a:
		7 from bottom–#15	Change:	$\rho_n = A_e F_n$
			To:	$P_n = A_e F_n$
		3 from bottom–#15	Change:	$\rho_a = \rho_n / \Omega_c$
			To:	$P_a = P_n / \Omega_c$

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Page	Section	Line Number / Reference		Revision
IV–75	Example 11	2 from top – #4	Change:	x
			To:	$\bar{x}$
		19 from top – #7	Change:	$x_o = \leq (\bar{x} + m) = -(0.757 + 1.194)$
			To:	$x_o = -(\bar{x} + m) = -(0.757 + 1.194)$
		27 from top – #9	Change:	$[8(\bar{b})^2(\bar{c}) + 2m[2\bar{c}(\bar{c} - \bar{a}) + \bar{b}(2\bar{c} - 3\bar{a})]]$
			To:	$\{8(\bar{b})^2(\bar{c}) + 2m[2\bar{c}(\bar{c} - \bar{a}) + \bar{b}(2\bar{c} - 3\bar{a})]\}$
		29 from top – #9	Change:	$C_w = [(0.105)^2/0.889]\{0.757 \times 0.889 \times (3.395)^2/0.105]$
			To:	$C_w = [(0.105)^2/0.889]\{[0.757 \times 0.889 \times (3.395)^2/0.105]$
		32 from top – #9	Change:	$-1.657(1.194)^2/0.105(2 \times 3.395 + 4 \times 0.848)$
			To:	$-1.657(1.194)^2/0.105\ (2 \times 3.395 + 4 \times 0.848)$ $+ 1.194\ (0.848)^2/3\ \{8(1.895)^2 \times 0.848$
		33 from top – #9	Change:	$+2{\times}1.194[2{\times}0.848(0.848 - 3.395) + 1.895(2{\times}0.848 - 3{\times}3.395)]\}$
			To:	$+2{\times}1.194[2{\times}0.848(0.848 - 3.395) + 1.895(2{\times}0.848 - 3{\times}3.395)]\}$
IV–77	Example 11	12 from top – #15	Change:	15. Determination of ρ_a:
			To:	15. Determination of P_a:
		13 from top – #15	Change:	$P\ = A_eF_n$
			To:	$P_n = A_eF_n$
	Example 12	3 from bottom	Change:	2. Section: $4 \times 4 \times 0.065$ Square Tube.
			To:	2. Section: $3.5 \times 3.5 \times 0.105$ channel with stiffened flanges.
IV–78	Example 12	4 from top – #1	Change:	from the sketch a = 2.914 in., b = 1.414 in., c = 0.607 in.
			To:	from the sketch a = 2.914 in., b = 2.914 in., c = 0.607 in.
		11 from top – #2	Change:	$A\ = t[a + 2b + 2u + 4c]$
			To:	$A\ = t[a + 2b + 2c + 4u]$
		8 from bottom – #9	Change:	$C_w = [(0.105)^2/1.204]\{1.445 \times 1.204 \times (3.395)^2/0.105]$
			To:	$C_w = [(0.105)^2/1.204]\{[1.445 \times 1.204 \times (3.395)^2/0.105]$
IV–79	Example 12	4 from top	Change:	r_o
			To:	r_o^2

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Page	Section	Line Number / Reference		Revision
IV–89	Example 15	17 from bottom	Change:	$A_e = 1.551 - 0.105(7.415 - 4.152) - 0.105(0.508 - 0.175)$ $= 1.173$ in.^{2} $P_{no} = 1.173 \times 50 = 58.65$ kips $\Omega_c = (5/3) + (3/8)R - (R^3/8))]$ $R = \sqrt{F_y / 2F_e}$, $F_e = \infty$ $R = 0$ $\Omega_c = 1.67$ $P_{ao} = 58.65/1.67$ $= 35.12$ kips
			To:	$A_e = 1.551 - 0.105(7.415 - 4.152) - 0.105(0.508 - 0.175)(2)$ $= 1.139$ in.^{2} $P_{no} = 1.139 \times 50 = 56.95$ kips $\Omega_c = 1.92$ since the section is not fully effective $P_{ao} = 56.95/1.92$ $= 29.66$ kips
IV–92	Example 15	16 from top	Change:	$M_y = S_f F_y$ (Yield Moment)
			To:	$M_y = S_f F_y$
IV–93	Example 15	13 from bottom	Change:	$C_{my} = 0.6 - 0.4(M_1 / M_2) \geq 0.4$
			To:	$C_{my} = 0.6 - 0.4(M_1 / M_2)$
		11 from bottom	Change:	$0.6 - 0.4(-1.00) = 1.00 > 0.4$
			To:	$0.6 - 0.4(-1.00) = 1.00$
IV–94	Example 15	5 from top	Change:	$= (2.50/30.55) + (5.00/24.34)$ $= 0.082 + 0.205$ $= 0.287 < 1.0$ *OK*
			To:	$= (2.50/29.66) + (5.00/24.34)$ $= 0.084 + 0.205$ $= 0.289 < 1.0$ *OK*
		16 from top–#4	Change:	$P_{ao} = 30.55$ K (calculated in part (a) 4).
			To:	$P_{ao} = 29.66$ kips (calculated in part (a) 4).
IV–98	Example 15	17 from top–#8	Change:	$C_{mx} = 0.6 - 0.4(-1.0) = 1.0 > 0.4$ *OK*
			To:	$C_{mx} = 0.6 - 0.4(-1.0) = 1.0$
		14 from bottom–#9	Change:	$d_x = 0.960$
			To:	$\alpha_x = 0.960$
		3 from bottom–#15	Change:	$(2.50/30.55) + (10.00/108.0) + (5.0/24.34)$ $0.082 + 0.093 + 0.205 = 0.380 < 1.0$ *OK*
			To:	$(2.50/29.66) + (10.00/108.0) + (5.0/24.34)$ $0.084 + 0.093 + 0.205 = 0.382 < 1.0$ *OK*

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Errata to the 1986 AISI Cold–Formed Steel Design Manual

Page	Section	Line Number / Reference		Revision
IV–100	Example 16	21 from bottom–(b)	Change:	Q_a
			To:	$\overline{Q}_a$
		19 from bottom–(b)	Change:	$\sigma_{CR} = (1/2\beta)\left[(\sigma_{ex} + \sigma_{tQ}) - \sqrt{(\sigma_{ex} + \sigma_{tQ})^2 - 4\beta\sigma_{ex} t_Q}\right]$
			To:	$\sigma_{CR} = (1/2\beta)\left[(\sigma_{ex} + \sigma_{tQ}) - \sqrt{(\sigma_{ex} + \sigma_{tQ})^2 - 4\beta\sigma_{ex}\sigma_{tQ}}\right]$
		17 from bottom–(b)	Change:	$\sigma_{tQ} = \sigma_t + Q_t$
			To:	$\sigma_{tQ} = \sigma_t + \overline{Q}_t$
		13 from bottom–(b)	Change:	$\overline{Q}_t = (Qd^2)/(4Ar_o^2)$
			To:	$\overline{Q}_t = (\overline{Q}d^2)/(4Ar_o^2)$
		9 from bottom–(b)	Change:	$\bar{q}_o = 2.0$ kip – in.
			To:	$\bar{q}_o = 2.0$ kip / in.
		8 from bottom–(b)	Change:	$S = 12$ in.
			To:	$s = 12$ in.
		7 from bottom–(b)	Change:	$\bar{q} = 2.0(2 - 12/12) = 2.0$ kip – in.
			To:	$\bar{q} = 2.0(2 - 12/12) = 2.0$ kip / in.
IV–101	Example 16	13 from bottom	Change:	$E_1 = (25)\{(68.5–25)[(3.57)^2(0.00257)–(2.03)(0.257)]$ $–(25)(2.03)[0.257–(2.03)(0.00257)]\}/(68.5–25)(3.57)^2(54.6–25)$ $–[(25)(2.03)]^2$
			To:	$E_1 = (25)\{(68.5–25)[(3.57)^2(0.00257)–(2.03)(0.257)]$ $–(25)(2.03)[0.257–(2.03)(0.00257)]\}/\{(68.5–25)(3.57)^2(54.6–25)$ $–[(25)(2.03)]^2\}$
IV–102	Example 16	9 from bottom	Change:	D/w = 0.275 k = [4.82–5(0.275)](0.000936/0.000542)$^{1/2}$+0.43 k = 4.96 5.25–5(0.275) = 3.88 < 4.96, so use k = 3.88 $\lambda = (1.052/\sqrt{3.88})(32.17)\sqrt{21.6/29500} = 0.465 (\leq 0.673)$
			To:	D/w = 0.290 k = [4.82–5(0.290)](0.000936/0.000542)$^{1/2}$ + 0.43 k = 4.86 5.25–5(0.290) = 3.80 < 4.86, so use k = 3.80 $\lambda = (1.052/\sqrt{3.80})(32.17)\sqrt{21.6/29500} = 0.470 (\leq 0.673)$

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Page	Section	Line Number / Reference		Revision
IV–125	Example 24	11 from top	Change:	$\theta = 4.76°$, $\tan\theta = 0.083$
			To:	$\theta = 4.76°$, $\sin\theta = 0.0830$
		Eq. D3.2.1–1, *Eq.* D3.2.1–2, *Eq.* D3.2.1–3	Change:	$\tan\theta$
			To:	$\sin\theta$
		12 from bottom, 8 from bottom, 4 from bottom	Change:	0.0833
			To:	0.0830
		11 from bottom	Change:	$P_L = 238$ lbs.
			To:	$P_L = 239$ lbs.
		5 from bottom, 4 from bottom	Delete:	0.5 before first bracket
IV–126	Example 25	8 from bottom	Change:	$I' = I'$ (straight portions)+I' (Arcs)=0.0045+0.0114 = 0.0159 in.^{3} $I = I't = 0.0159 \times 0.030 = 0.000477$ in.^{4}
			To:	$I' \geq I'$ (straight portions)+I' (Arcs)=0.0045+0.0114 = 0.0159 in.^{3} $I \geq I't = 0.0159 \times 0.030 = 0.000477$ in.^{4}
		4 from bottom	Change:	$I_{min} = [3.66t^4\sqrt{(w/t)^2 - 4000/F_y}\,]$ but not less than $(18t^4) = 0.000015$ in^4 $I_{min} = [3.66 \times (0.030)^4\sqrt{(79.8)^2 - 4000/50}\,] = 0.000235$ in^4 < 0.00477 in^4
			To:	$I_{min} = [3.66t^4\sqrt{(w/t)^2 - 0.136E/F_y}\,]$ but not less than $(18t^4) = 0.000015$ in^4 $I_{min} = [3.66 \times (0.030)^4\sqrt{(79.8)^2 - (0.136)(29500)/50}\,] = 0.0002$ < 0.000477 in^4

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Page	Section	Line Number / Reference		Revision
IV–127	Example 25	10 from bottom	Change:	$S = 1.28\sqrt{E/f} = 1.28\sqrt{29500/0.60(50)} = 40.138$
			To:	$S = 1.28\sqrt{E/f} = 1.28\sqrt{29500/(50)} = 31.091$
		7 from bottom	Change:	$I_a = (0.030)^4\,[115(86/40.138) + 5] = 0.000204$ $I_s = I'_1 = 0.003$ $k = 3.57(0.003/0.000204)^{1/3} + 0.43 = 9.176 > 4$
			To:	$I_a = (0.030)^4\,[115(86/31.091) + 5] = 0.000262$ $I_s = I_{x1} = 0.00090$ $k = 3.57(0.00090/0.000262)^{1/3} + 0.43 = 5.817 > 4$
		2 from bottom	Change:	$b = \rho w = 0.474[3 - 3(0.140)] = 1.233$ in. $A_s = A'_s = 0.348$ in.^{2} for $I_s \geq I_a$
			To:	$b = \rho w = 0.474[3 - 3(0.140)] = 1.223$ in. $A_s = A'_s = 0.0348$ in.^{2} for $I_s \geq I_a$
IV–128	Example 25	entire page	Change:	Replace with replacement page
IV–129	Example 25	entire page	Change:	Replace with replacement page
IV–130	Example 25	Ly column in table	Change:	3.490
			To:	3.492
		Ly column in table	Change:	4.909
			To:	4.911
		Ly^2 column in table	Change:	3.550
			To:	3.544
		Ly^2 column in table	Change:	1.700
			To:	1.697
		Ly^2 column in table	Change:	5.394
			To:	5.385
	Example 25	14 from bottom	Change:	$b_e = 0.030[(1.535/0.030) - 0.10(79.8 - 60)] = 1.476$ in.
			To:	$b_e = 0.030[(1.535/0.030) - 0.10(79.8 - 60)] = 1.476$ in. for $w/t > 60$
	Example 25	bottom line	Change:	$b_e = 0.030[(1.485/0.030) - 0.10(68.033 - 60)] = 1.458$ in.
			To:	$b_e = 0.030[(1.485/0.030) - 0.10(68.033 - 60)] = 1.461$ in.

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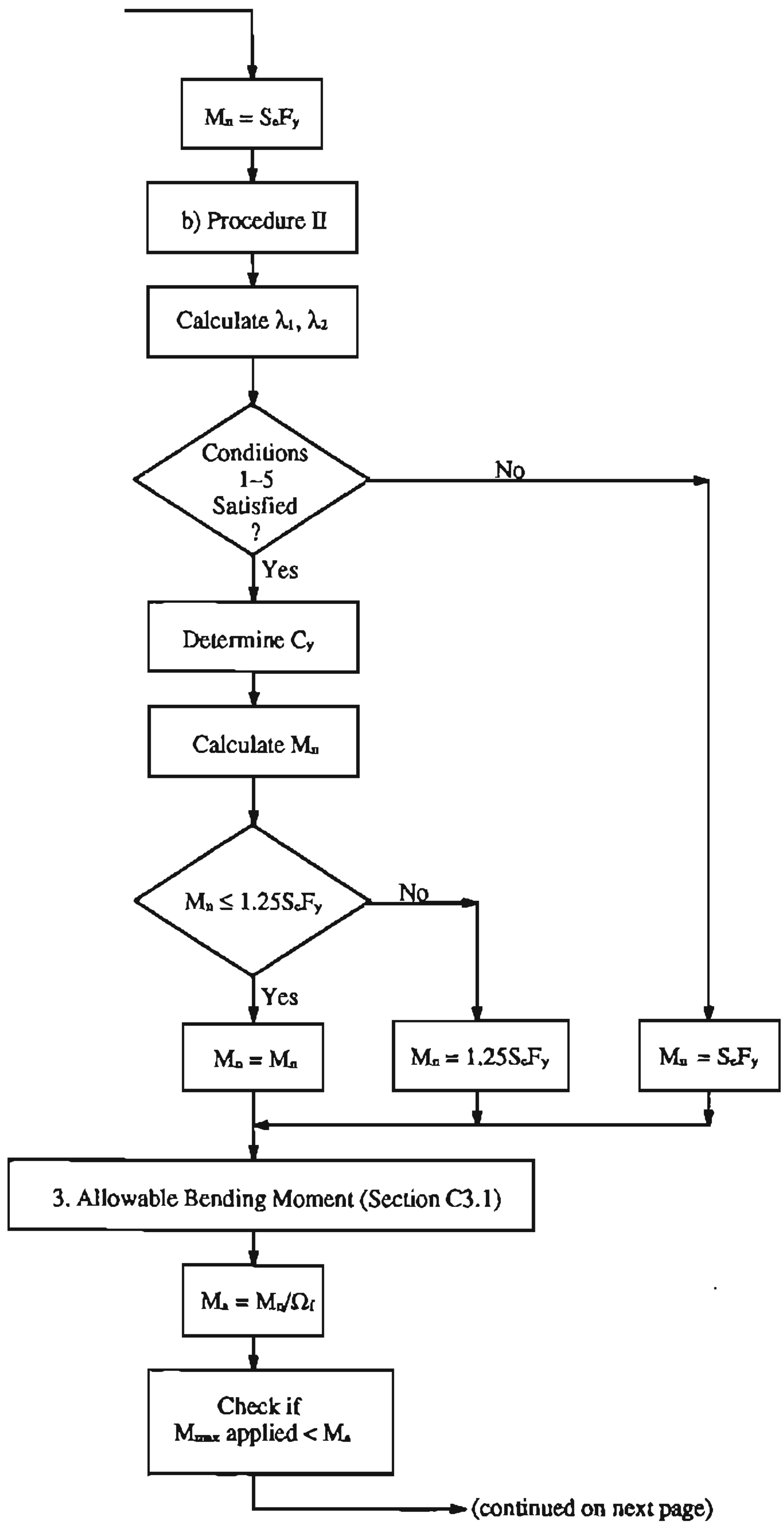
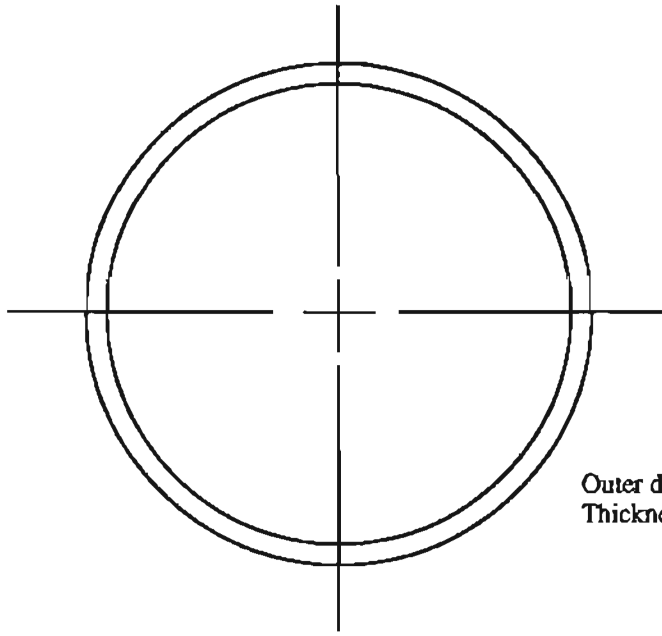
Page	Section	Line Number / Reference	Revision	
IV–131	Example 25	L column in table	Change:	1.458
			To:	1.461
		L column in table	Change:	16.460
			To:	16.463
		Ly column in table	Change:	3.490
			To:	3.492
		Ly column in table	Change:	2.938
			To:	2.944
		Ly column in table	Change:	17.089
			To:	17.097
	Example 25	8 from bottom	Change:	y_{cg} = 17.089/16.460 = 1.038 in.
			To:	y_{cg} = 17.097/16.463 = 1.039 in.
	Example 25	6 from bottom	Change:	$f = F_y$ since $y_{cg} < 2.030/2 = 1.015$
			To:	$f = F_y$ since $y_{cg} \cong 2.030/2 = 1.015$
V–25	Tables – General Notes	3 from bottom	Change:	(h) Tables 1–9 incl. are Full Area Tables. Tables 10–15 incl. are Effective Area Tables. (F_y = 50 ksi)
			To:	(h) All tables assume F_y = 50 ksi.
V–26	Table 1	entire page	Change:	Replace with replacement page
V–27	Table 1 (cond.)	entire page	Change:	Replace with replacement page
V–28	Table 2	entire page	Change:	Replace with replacement page
V–29	Table 3	entire page	Change:	Replace with replacement page
V–30	Table 4	entire page	Change:	Replace with replacement page
V–35 to V–38	Tables 10 to 13	entire page	Change:	Remove pages
VI–25	Computer Aids	diamond	Change:	$F_{e1} < F_e$
			To:	$F_{e1} < F_{e2}$


$M_n = S_eF_y$
b) Procedure II
Calculate λ_1, λ_2
Conditions 1–5 Satisfied ?
No
Yes
Determine C_y
Calculate M_n
$M_n \leq 1.25S_eF_y$
No
Yes
$M_n = M_n$
$M_n = 1.25S_eF_y$
$M_n = S_eF_y$
3. Allowable Bending Moment (Section C3.1)
$M_a = M_n/\Omega_f$
Check if M_{max} applied < M_a
(continued on next page)

EXAMPLE NO. 9

CYLINDRICAL TUBULAR SECTION



Outer diameter = 8.000 in.
Thickness = 0.125 in.

Given: Steel: $F_y = 50$ ksi.
Section: Shown in sketch above.

Required: Allowable bending moment

Solution:

$S_f = \pi[(O.D.)^4 - (I.D.)^4]/[32(O.D.)]$
$= \pi[(8.00)^4 - (7.75)^4]/[32(8.00)]$
$= 5.995$ in.^{3}

Ratio of outside diameter to wall thickness,

$D/t = 8.00/0.125 = 64.00$

$D/t < 0.441\ E/F_y = 0.441(29500/50) = 260.2$ *OK*

$0.070\ E/F_y = 0.070(29500/50) = 41.30$

$0.319\ E/F_y = 0.319(29500/50) = 188.2$

For $0.070\ E/F_y < D/t/(0.319E/F_y)$

$M_n = [0.97 + 0.02(E/F_y)/(D/t)]F_yS_f$ (*Eq.* C6.1–4)
$= [0.97 + 0.02(29500/50)/64.00](50)(5.995)$
$= 343.0$ kip–in.

$M_a = M_n/\Omega_f$ (*Eq.* C6.1–1)
$= 343.0/1.67$
$= 205.4$ kip–in.

$S_e = I_y/(3.000 - x_{cg})$
$= 1.628/(3.000 - 0.998)$
$= 0.813 \text{ in.}^3$
$M_{ny} = S_eF_y$ (*Eq.* C3.1.1–1)
$= 0.813\,(50)$
$= 40.65$ kip–in.

(b) Section C3.1.2:
M_{ny} will be calculated on the basis of the lateral buckling strength.(y–axis is the axis of bending)
For the full section:
$I_y = 1.786 \text{ in.}^4$
$x_{cg} = \bar{x} + t/2 = 0.820 + 0.105/2$
$= 0.873$ in.
$S_f = I_y/x_{cg} = 1.786 / 0.873$
$= 2.046 \text{ in.}^3$
$M_y = S_fF_y$ (*Eq.* C3.1.2–5)
$= 2.046\,(50)$
$= 102.3$ kip–in.
$C_s = +1.00$
$A = 1.551 \text{ in.}^2$
$\sigma_{ex} = 76.93$ ksi
$\sigma_t = 10.26$ ksi
$M_1/M_2 = -1.00$ (single curvature)
$C_{TF} = 0.6 - 0.4\,(M_1/M_2)$
$= 0.6 - 0.4(-1.00) = 1.00$
$r_o = 3.961$ in.
$j = 4.568$
$M_e = C_sA\sigma_{ex}\left[j + C_s\sqrt{j^2 + r_o^2(\sigma_t / \sigma_{ex})}\right] / C_{TF}$ (*Eq.* C3.1.2–11)
$= 1.0(1.551)(76.93)[4.568 + 1.00\sqrt{(4.568)^2 + (3.961)^2(10.26 / 76.93}] / 1.00$
$= 1116.77$ kip–in.
$M_e = 1116.77 \text{ kip–in.} > 0.5M_y = 21.0$ kip–in.
$M_c = M_y[1 - (M_y/4M_e)]$ (*Eq.* C3.1.2–8)
$= 42.0[1 - 42.0/(4 \times 1116.77)]$
$= 41.61$ kip–in.
$M_c/S_f = 41.61/0.840 = 49.54$ ksi

To calculate effective section properties to obtain S_c at stress 49.54 ksi, we assume that the webs are fully effective.

Compression flange:
$\lambda = \left(1.052 / \sqrt{4.00}\right)(70.62)\sqrt{49.54 / 29500} = 1.522 > 0.673$
$\rho = [1 - (0.22/1.522)]/1.522 = 0.562$
$b = 0.562\,(7.415) = 4.167$ in.

Element	L Effective Length (in.)	x Distance from Top Fiber (in.)	Lx (in.^{2})	Lx^2 (in.^{3})	I'_1 About Own Axis (in.^{3})
Webs	$2 \times 2.415 = 4.830$	1.500	7.245	10.868	2.347
Upper Corners	$2 \times 0.377 = 0.754$	0.140	0.106	0.015	–
Lower Corners	$2 \times 0.377 = 0.754$	2.860	2.156	6.167	–
Compression Flange	4.167	0.053	0.221	0.012	–
Tension Flange	$2 \times 0.508 = 1.016$	2.948	2.995	8.830	–
Sum	11.521		12.723	25.892	2.347

Distance from top fiber to y–axis is $x_{cg} = 12.723/11.521 = 1.104$ in.
To check if the webs are fully effective (Section B2.3):

$f_1 = [(1.104 - 0.293)/1.896]\ 49.54 = 21.19$ ksi (compression)
$f_2 = -[(1.104 - 0.293)/1.104)\ 49.54 = -36.39$ ksi (tension)
$\psi = -36.39/21.19 = -1.717$
$k = 4 + 2[1 - (-1.717)]^3 + 2[1 - (-1.717)]$
$= 49.548$
$\lambda = \left(1.052 / \sqrt{49.548}\right)(70.62)\sqrt{21.19 / 29500} = 0.092 < 0.673$
$b_e = 2.415$ in.
$b_2 = 2.415/2 = 1.208$ in.
$b_1 = 2.415/[3 - (-1.717)] = 0.512$ in.

Compression portion of each web calculated on the basis of the effective section = 1.104 – 0.293 = 0.811

Since $b_1 + b_2 = 1.720$ in. > 0.811 in., $b_1 + b_2$ shall be taken as 0.811 in. This verifies the assumption that the web is fully effective.

$I'_y = 25.892 + 2.347 - 11.521\ (1.104)^2$
$= 14.197$ in.^{3}
Actual $I_y = 14.197\ (0.105)$
$= 1.491$ in.^{4}
$S_c = I_y/(3.000 - x_{cg}) = 1.491/(3.000 - 1.104)$
$= 0.786$ in.^{3}
$M_{ny} = M_c\ S_c/S_f$ (*Eq.* C3.1.2–1)
$= 41.61\ (0.786)/0.840$
$= 38.94$ kip–in.

M_{ny} shall be the smaller of 40.65 kip–in. and 38.94 kip–in.
Thus

$M_{ny} = 38.94$ kip–in.
$\Omega_f = 1.67$
$M_{ay} = M_{ny}/\Omega_f$ (*Eq.* C3.1–1)
$= 38.94/1.67$
$= 23.32$ kip–in.

7. Determination of M_{ayo}:

M_{ayo} is the allowable moment about the centroidal axes determined in accordance with Section C3.1 excepting the provisions of Section C3.1.2 (excluding lateral buckling).

Therefore

$M_{nyo} = 40.65$ kip–in.
$\Omega_f = 1.67$
$M_{ayo} = 40.65/1.67$
$= 24.34$ kip–in.

8. $C_{my} = 0.6 - 0.4\ (M_1/M_2)$
$M_1/M_2 = -1.00$ (single curvature)
$0.6 - 0.4\ (-1.00) = 1.00$
$C_{my} = 1.00$

9. Determination of $1/\alpha_y$:
$\Omega_c = 1.92$
$P_{cr} = \pi^2 EI_y/(K_yL_y)^2$ (*Eq.*C5–5)
$I_y = 1.786$ in.^{4}
$K_yL_y = 1.0\ (16 \times 12) = 192$ in.
$P_{cr} = [\pi^2\ (29500)\ (1.786)]/(192)^2 = 14.11$ kips
$1/\alpha_y = 1/[1 - (\Omega_c\ P/P_{cr})]$ (*Eq.* C5–4)
$= 1/[1 - (1.92 \times 2.5/14.11)] = 1.516$
$\alpha_y = 0.660$

Element 9 from Section B3.2(a)

$w = 0.415 - 0.030 - 0.125 = 0.26$ in.

$k = 0.43$

$f < F_y$ Use F_y as conservative value.

$\lambda = \left(1.052 / \sqrt{0.43}\right)(0.26 / 0.030)\sqrt{50 / 29500} = 0.572$ (*Eq.* B2.1–4)

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

$b = w = 0.26$ in. (*Eq.* B2.1–1)

Element 3 from Section B4.2(a)

$w/t = [2 - 2(0.140)]/0.030 = 57.333$ $f = F_y$ [see B2.1a(1)] (*Eq.* B4–1)

$S = 1.28\sqrt{E / f} = 1.28\sqrt{29500 / (50)} = 31.091$

$D/w = [0.415 - 0.5(0.030)]/[2 - 2(0.140)] = 0.233$

$n = 1/3$ for $w/t > S$

$I_a = (0.030)^4\ [(57.333/31.091)(115) + 5] = 0.000176$ for $w/t \geq S$ (*Eq.* B4.2–13)

$I_s = (1/12)bh^3 = (1/12)(0.030)(0.415 - 0.125 - 0.030)^3 = 0.000044$

$k = 3.57(0.000044/0.000176)^{1/3} + 0.43 = 2.679 < 4$ (*Eq.* B4.2–10)

$\lambda = \left(1.052 / \sqrt{2.769}\right)(57.333)\sqrt{50 / 29500} = 1.517$ (*Eq.* B2.1–4)

$\rho = [1 - (0.22/1.517)](1/1.517) = 0.564$ (*Eq.* B2.1–3)

$b = \rho w = 0.564[2 - 2(0.140)] = 0.970$ in.

$d_s = (I_s/I_a)d'_s = (0.000044/0.000176)(0.26) = 0.065$ in. (*Eq.* B4.2–11)

Element	L	y	Ly	Ly²	I'₁
1	1.160	0.321	0.372	0.120	0.030
2	1.223	0.015	0.018	–	–
3	0.970	0.015	0.015	–	–
4	0.660	0.066	0.044	0.003	–
5	3.440	1.015	3.492	3.544	0.849
6	4.788	2.015	9.648	19.440	–
7	2.396	1.840	4.409	8.112	–
8	2.068	2.015	4.167	8.397	–
9	0.065	0.188	0.012	0.002	–
10	0.440	1.964	0.864	1.697	–
	17.210		23.041	41.315	0.879

$y_{cg} = 23.041/17.210 = 1.339$ in.

$I'_x = 41.315 + 0.879 - 17.210(1.339)^2 = 11.338$ in.^{3}

$I_x = I'_x t = (11.338)(0.030) = 0.340$ in.^{4}

$S_x = I_x/y_{cg} = 0.340/1.339 = 0.254$ in.^{3}

$M_n = S_e F_y = 0.254(50) = 12.700$ kip–in.

$M_a = M_n/\Omega_f = 12.700/1.67 = 7.605$ kip–in. (*Eq.* C3.1–1)

Element 5 from Section B2.3(a)

$y_{cg} = 1.339$ in.

$f_1 = [(1.339 - 0.125 - 0.030)/1.339](50) = 44.212$

$f_2 = -[(2.030 - 0.125 - 0.030 - 1.339)/1.339](50) = -20.015$

$\psi = f_2/f_1 = -20.015/44.212 = -0.453$

$k = 4 + 2(1 + 0.453)^3 + 2(1 + 0.453) = 13.041 > 4$ (*Eq.* B2.3–4)

$\lambda = \left(1.052 / \sqrt{13.041}\right)\left\{[2.030 - 2(0.155)] / 0.030\right\}\sqrt{44.212 / 29500} = 0.647 < 0.673$ (*Eq.* B2.1–4)

$b_o = w = 1.72$ (*Eq.* B2.1–2)

Thus element 5 is fully effective so properties above are correct. If $b_o < 1.72$ then properties should be recomputed for an exact solution.

3. Moment of Inertia for Deflection Determination–Positive Bending

Element 2 from Section B4.2(b)

$f = F_y/1.67 = 30$ ksi

$w = 3 - 3(0.140) = 2.580$ in.

$\lambda_c = 0.256 + 0.328(2.580 / 0.030)\sqrt{50 / 29500} = 1.417$ (*Eq.* B2.1–10)

$k = 4$

$\lambda = (1.052 / \sqrt{4.00})(2.580 / 0 / 030)\sqrt{30 / 29500} = 1.443$ (*Eq.* B2.1–4)

$\rho = \left[0.41 + 0.59\sqrt{50 / 30} - (0.22 / 1.443)\right](1 / 1.443) = 0.706$ (*Eq.* B2.1–9)

$b = \rho w = 0.706(2.580) = 1.821$ in (*Eq.* B2.1–2)

$A_s = A'_s = 0.348$ in.^{2} (*Eq.* B4.2–12)

Element 3 from Section 4.2(b)

$f = 50/1.67 = 30$ ksi

$w = 2 - 2(0.140) = 1.720$ in.

$\lambda_c = 0.256 + 0.328(1.720 / 0.030)\sqrt{50 / 29500} = 1.030$ (*Eq.* B2.1–10)

$S = 1.28\sqrt{29500 / 30} = 40.14$ (*Eq.* B4–1)

$I_a = [115(57.33/40.14) + 5](0.030)^4 = 0.000137$ in.^{4} (*Eq.* B4.2–13)

$k = 3.57(0.000044/0.000137)^{1/3} + 0.43 = 2.875$ (*Eq.* B4.2–10)

$\lambda = (1.052 / \sqrt{2.875})(1.720 / 0.030)\sqrt{30 / 29500} = 1.134$ (*Eq.* B2.1–4)

$\rho = \left[0.41 + 0.59\sqrt{50 / 30} - (0.22 / 1.134)\right](1 / 1.134) = 0.862$ (*Eq.* B2.1–9)

$b = \rho w = 0.862(1.720) = 1.483$ in. (*Eq.* B2.1–2)

$d_s = (0.000044/0.000137)(0.260) = 0.084$ in. (*Eq.* B4.2–11)

Element	L	y	Ly	Ly^2	I'$_1$
1	1.160	0.321	0.372	0.120	0.030
2	1.821	0.015	0.027	–	–
3	1.483	0.015	0.022	–	–
4	0.660	0.066	0.044	0.003	–
5	3.440	1.015	3.492	3.544	0.849
6	4.788	2.015	9.648	19.440	–
7	2.396	1.840	4.409	8.112	–
8	2.068	2.015	4.167	8.397	–
9	0.084	0.197	0.017	0.003	–
10	0.440	1.964	0.864	1.697	–
	18.340		23.062	41.316	0.879

$y_{cg} = 23.062/18.340 = 1.257$ in.

$I'_x = 41.316 + 0.879 - 18.340(1.257)^2 = 13.217$ in.^{3}

$I_x = I'_x t = 13.217(0.030) = 0.397$ in.^{4}

$S_e = 0.395/1.258 = 0.314$ in.^{3}

$M = f_x S_e = 30.0 \times 0.314 = 9.426$ kip–in.

M_a (from stress calculation) = 7.605 kip–in.

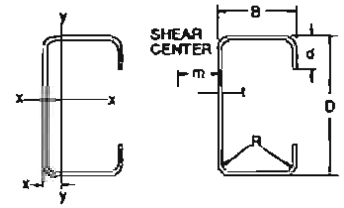
For deflection calculations, a stress level, f, should be used such that $f_x S_e$ will equal M_a from stress calculations (see Examples 2 and 3). In this example, further iterations should be made, reducing f until $f_x S_e = M_a$. This will be left to the user to complete.

4. Section Modulus for Load Determination–Negative Bending

Since the N.A. may be closer to the compression flange than to the tension flange, the compression stress is unknown, and therefore the effective width of the compression flange and section properties must be determined by an iterative method.

Elements 1, 2, 3, 4, 5, 9 and 10 do not vary with stress level.

CHANNEL WITH STIFFENED FLANGES

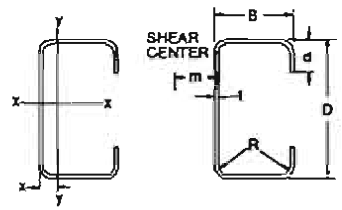


See notes on page V–25

Size							Properties of Full Section													Eff. Section Properties		Moment Capacity
						Wgt.	Axis x–x			Axis y–y												
D	B	t	d	R	Area	per Foot	I_x	S_x	r_x	I_y	S_y	r_y	x	m	J	C_w	j	r_o	x_o	I_x	S_x	M_a
In.	In.	In.	In.	In.	In.^{2}	Lb.	In.^{4}	In.^{3}	In.	In.^{4}	In.^{3}	In.	In.	In.	In.^{4}	In.^{6}	In.	In.	In.	In.^{4}	In.^{3}	Kip–In.
12.000	3.500	0.135	1.010	0.188	2.706	9.199	56.266	9.378	4.560	4.037	1.560	1.222	0.912	1.487	0.01644	117.191	6.777	5.266	−2.332	55.387	9.135	273.488
		0.105	0.900	0.188	2.097	7.129	43.836	7.306	4.572	3.090	1.181	1.214	0.883	1.461	0.00771	88.556	6.844	5.257	−2.291	40.904	6.546	195.994
10.000	3.500	0.135	1.010	0.188	2.436	8.281	36.526	7.305	3.872	3.823	1.533	1.253	1.006	1.582	0.01480	78.535	5.591	4.787	−2.520	35.957	7.110	212.889
		0.105	0.900	0.188	1.887	6.415	28.505	5.701	3.887	2.929	1.160	1.246	0.975	1.553	0.00693	59.102	5.638	4.774	−2.476	26.513	5.071	151.837
		0.075	0.720	0.094	1.344	4.568	20.533	4.107	3.909	2.035	0.792	1.231	0.932	1.473	0.00252	39.268	5.580	4.733	−2.367	17.571	3.201	95.829
9.000	3.250	0.135	1.000	0.188	2.230	7.584	27.157	6.035	3.489	3.072	1.344	1.174	0.965	1.507	0.01355	52.171	5.035	4.397	−2.404	27.157	6.035	180.688
		0.105	0.840	0.188	1.717	5.836	21.077	4.684	3.504	2.300	0.987	1.157	0.921	1.458	0.00631	37.785	5.105	4.362	−2.326	20.030	4.307	128.960
		0.075	0.700	0.094	1.228	4.176	15.287	3.397	3.528	1.627	0.689	1.151	0.888	1.393	0.00230	25.652	5.031	4.337	−2.244	13.551	2.807	84.034
		0.060	0.610	0.094	0.976	3.319	12.182	2.707	3.533	1.265	0.530	1.138	0.862	1.364	0.00117	19.637	5.061	4.313	−2.196	9.703	1.875	56.123
8.000	3.000	0.135	0.930	0.188	2.009	6.831	19.371	4.843	3.105	2.356	1.127	1.083	0.909	1.409	0.01221	31.896	4.514	3.985	−2.250	19.371	4.843	144.996
		0.105	0.810	0.188	1.553	5.279	15.125	3.781	3.121	1.794	0.844	1.075	0.876	1.375	0.00571	23.639	4.563	3.966	−2.198	14.727	3.613	108.164
		0.075	0.700	0.094	1.116	3.793	11.045	2.761	3.147	1.290	0.600	1.075	0.851	1.322	0.00209	16.341	4.484	3.952	−2.135	9.965	2.341	70.098
		0.060	0.600	0.094	0.885	3.008	8.791	2.198	3.152	0.997	0.458	1.061	0.822	1.288	0.00106	12.357	4.515	3.923	−2.080	7.397	1.667	49.907
8.000	1.625	0.105	0.820	0.188	1.266	4.305	10.647	2.662	2.900	0.402	0.332	0.564	0.412	0.709	0.00465	5.656	4.971	3.141	−1.068	10.647	2.662	79.693
		0.075	0.820	0.094	0.927	3.153	7.996	1.999	2.936	0.316	0.261	0.584	0.414	0.706	0.00174	4.364	4.686	3.184	−1.083	7.996	1.999	59.851
		0.060	0.600	0.094	0.720	2.447	6.190	1.548	2.932	0.222	0.177	0.555	0.369	0.645	0.00086	2.925	4.992	3.143	−0.984	6.190	1.548	46.335
		0.048	0.500	0.094	0.569	1.935	4.889	1.222	2.931	0.167	0.131	0.542	0.347	0.618	0.00044	2.168	5.131	3.126	−0.941	4.447	1.041	31.157
7.000	2.750	0.135	0.880	0.188	1.793	6.097	13.266	3.790	2.720	1.777	0.940	0.996	0.859	1.320	0.01089	18.778	3.992	3.584	−2.112	13.266	3.790	113.483
		0.105	0.880	0.188	1.410	4.794	10.553	3.015	2.736	1.438	0.761	1.010	0.859	1.335	0.00518	15.341	3.995	3.618	−2.141	10.548	3.013	90.203
		0.075	0.700	0.094	1.003	3.411	7.659	2.188	2.763	1.000	0.517	0.999	0.815	1.251	0.00188	9.940	3.950	3.570	−2.029	7.033	1.904	57.020
		0.060	0.600	0.094	0.795	2.702	6.098	1.742	2.770	0.772	0.393	0.986	0.786	1.217	0.00095	7.469	3.977	3.541	−1.973	5.368	1.422	42.571
6.000	2.500	0.135	0.820	0.188	1.574	5.353	8.575	2.858	2.334	1.293	0.764	0.906	0.808	1.229	0.00956	10.264	3.485	3.185	−1.969	8.575	2.858	85.580
		0.105	0.820	0.188	1.240	4.215	6.843	2.281	2.349	1.051	0.621	0.921	0.808	1.244	0.00456	8.436	3.491	3.219	−1.999	6.843	2.281	68.296
		0.075	0.820	0.094	0.909	3.089	5.124	1.708	2.375	0.805	0.478	0.941	0.814	1.232	0.00170	6.425	3.408	3.249	−2.008	4.944	1.607	48.103
		0.060	0.600	0.094	0.705	2.396	4.012	1.337	2.386	0.583	0.333	0.909	0.752	1.148	0.00085	4.257	3.456	3.164	−1.869	3.606	1.125	33.692
6.000	1.625	0.105	0.820	0.188	1.056	3.591	5.247	1.749	2.229	0.370	0.324	0.592	0.483	0.798	0.00388	3.074	3.304	2.613	−1.229	5.247	1.749	52.364
		0.075	0.820	0.094	0.777	2.643	3.972	1.324	2.260	0.291	0.255	0.611	0.486	0.791	0.00146	2.388	3.154	2.650	−1.240	3.972	1.324	39.641
		0.060	0.600	0.094	0.600	2.039	3.085	1.028	2.268	0.206	0.173	0.585	0.437	0.722	0.00072	1.555	3.330	2.600	−1.129	3.085	1.028	30.793
		0.048	0.500	0.094	0.473	1.608	2.441	0.814	2.271	0.155	0.128	0.573	0.412	0.692	0.00036	1.140	3.412	2.580	−1.081	2.334	0.757	22.677

TABLE 1 (continued)

CHANNEL WITH STIFFENED FLANGES



See notes on page V–25

Size							Properties of Full Section													Eff. Section Properties		Moment Capacity
							Axis x–x			Axis y–y												
D	B	t	d	R	Area	Wgt. per Foot	I_x	S_x	r_x	I_y	S_y	r_y	x	m	J	C_w	j	r_o	x_o	I_x	S_x	M_a
In.	In.	In.	In.	In.	In.^{2}	Lb.	In.^{4}	In.^{3}	In.	In.^{4}	In.^{3}	In.	In.	In.	In.^{4}	In.^{6}	In.	In.	In.	In.^{4}	In.^{3}	Kip–In.
5.000	2.000	0.135	0.700	0.188	1.272	4.325	4.683	1.873	1.919	0.648	0.478	0.714	0.644	0.987	0.00773	3.610	2.895	2.576	–1.563	4.684	1.873	56.090
		0.105	0.700	0.188	1.005	3.416	3.761	1.504	1.935	0.533	0.393	0.728	0.644	1.002	0.00369	3.007	2.897	2.610	–1.593	3.761	1.504	45.036
		0.075	0.600	0.094	0.726	2.467	2.797	1.119	1.963	0.389	0.283	0.733	0.622	0.949	0.00136	2.065	2.834	2.596	–1.533	2.760	1.093	32.713
		0.060	0.500	0.094	0.573	1.948	2.227	0.891	1.972	0.298	0.212	0.721	0.594	0.915	0.00069	1.515	2.864	2.568	–1.478	2.053	0.780	23.365
		0.048	0.500	0.094	0.461	1.568	1.804	0.722	1.978	0.244	0.173	0.727	0.594	0.921	0.00035	1.246	2.865	2.581	–1.491	1.632	0.614	18.377
4.000	2.000	0.135	0.700	0.188	1.137	3.866	2.751	1.376	1.556	0.598	0.464	0.725	0.712	1.054	0.00671	2.305	2.520	2.415	–1.699	2.751	1.376	41.189
		0.105	0.700	0.188	0.900	3.059	2.219	1.109	1.570	0.492	0.382	0.740	0.712	1.069	0.00331	1.930	2.534	2.450	–1.729	2.219	1.109	33.215
		0.075	0.600	0.094	0.651	2.212	1.664	0.832	1.599	0.361	0.275	0.745	0.689	1.009	0.00122	1.306	2.478	2.423	–1.661	1.644	0.813	24.341
		0.060	0.500	0.094	0.513	1.744	1.330	0.665	1.610	0.277	0.206	0.734	0.660	0.972	0.00062	0.943	2.489	2.387	–1.602	1.223	0.578	17.303
		0.048	0.500	0.094	0.413	1.404	1.079	0.539	1.616	0.226	0.169	0.740	0.660	0.979	0.00032	0.777	2.495	2.401	–1.614	0.972	0.454	13.593
4.000	1.625	0.075	0.600	0.094	0.594	2.021	1.448	0.724	1.561	0.219	0.201	0.607	0.539	0.818	0.00111	0.802	2.260	2.132	–1.319	1.448	0.724	21.670
		0.060	0.500	0.094	0.468	1.591	1.155	0.577	1.571	0.167	0.150	0.597	0.511	0.785	0.00056	0.577	2.291	2.104	–1.266	1.143	0.566	16.961
		0.048	0.500	0.094	0.377	1.282	0.938	0.469	1.577	0.137	0.123	0.603	0.511	0.791	0.00029	0.477	2.294	2.118	–1.278	0.895	0.433	12.973
		0.036	0.500	0.094	0.285	0.969	0.715	0.357	1.584	0.106	0.095	0.609	0.511	0.797	0.00012	0.370	2.296	2.132	–1.290	0.663	0.316	9.454
3.625	1.625	0.075	0.600	0.094	0.566	1.925	1.148	0.633	1.424	0.211	0.199	0.611	0.564	0.841	0.00106	0.660	2.126	2.067	–1.367	1.148	0.633	18.965
		0.060	0.500	0.094	0.445	1.514	0.918	0.506	1.436	0.162	0.148	0.602	0.536	0.807	0.00053	0.471	2.148	2.037	–1.313	0.908	0.497	14.876
		0.048	0.500	0.094	0.359	1.221	0.746	0.412	1.442	0.133	0.122	0.608	0.536	0.813	0.00028	0.390	2.153	2.050	–1.325	0.712	0.380	11.372
		0.036	0.500	0.094	0.271	0.923	0.569	0.314	1.448	0.102	0.094	0.614	0.536	0.819	0.00012	0.303	2.157	2.064	–1.337	0.527	0.276	8.277
3.500	2.000	0.135	0.700	0.188	1.069	3.636	2.003	1.144	1.368	0.568	0.456	0.729	0.753	1.093	0.00650	1.790	2.386	2.359	–1.778	2.003	1.144	34.265
		0.105	0.700	0.188	0.847	2.880	1.620	0.926	1.383	0.468	0.375	0.743	0.753	1.107	0.00311	1.504	2.404	2.394	–1.808	1.620	0.926	27.714
		0.075	0.600	0.094	0.613	2.085	1.222	0.698	1.412	0.344	0.271	0.749	0.729	1.043	0.00115	1.005	2.349	2.358	–1.734	1.208	0.683	20.444
		0.060	0.500	0.094	0.483	1.642	0.979	0.559	1.424	0.264	0.203	0.739	0.699	1.005	0.00058	0.717	2.351	2.318	–1.673	0.899	0.484	14.506
		0.048	0.500	0.094	0.389	1.323	0.795	0.454	1.430	0.216	0.166	0.745	0.699	1.011	0.00030	0.592	2.358	2.333	–1.686	0.715	0.380	11.389
3.000	1.750	0.105	0.700	0.188	0.742	2.523	1.017	0.678	1.171	0.318	0.300	0.654	0.689	1.016	0.00273	0.835	2.100	2.128	–1.652	1.017	0.678	20.297
		0.075	0.530	0.094	0.528	1.794	0.767	0.512	1.206	0.224	0.202	0.651	0.644	0.918	0.00099	0.486	2.045	2.050	–1.525	0.767	0.512	15.316
		0.060	0.530	0.094	0.426	1.450	0.628	0.418	1.213	0.185	0.167	0.658	0.644	0.926	0.00051	0.406	2.054	2.068	–1.540	0.617	0.405	12.132
		0.048	0.410	0.094	0.332	1.130	0.499	0.332	1.225	0.138	0.120	0.644	0.607	0.877	0.00026	0.272	2.051	2.011	–1.460	0.442	0.272	8.146
2.500	1.625	0.075	0.600	0.094	0.482	1.638	0.481	0.384	0.999	0.184	0.190	0.618	0.656	0.923	0.00090	0.337	1.859	1.937	–1.541	0.481	0.384	11.511
		0.060	0.500	0.094	0.378	1.285	0.388	0.311	1.014	0.141	0.141	0.611	0.626	0.885	0.00045	0.230	1.856	1.896	–1.481	0.385	0.305	9.143
		0.048	0.500	0.094	0.305	1.037	0.317	0.254	1.019	0.116	0.116	0.617	0.626	0.891	0.00023	0.191	1.864	1.910	–1.493	0.303	0.233	6.991
		0.036	0.500	0.094	0.231	0.785	0.242	0.194	1.025	0.089	0.090	0.622	0.627	0.897	0.00010	0.149	1.873	1.924	–1.505	0.224	0.169	5.063

CHANNEL WITH UNSTIFFENED FLANGES

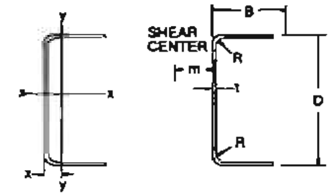


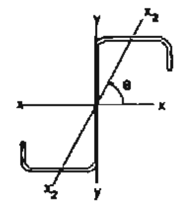
TABLE 2

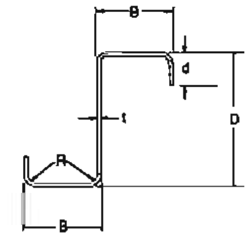
See notes on page V–25

Size						Properties of Full Section													Eff. Section Properties		Moment Capacity M_a
					Wgt. per Foot	Axis x–x			Axis y–y												
D	B	t	R	Area		I_x	S_x	r_x	I_y	S_y	r_y	x	m	J	C_w	j	r_o	x_o	I_x	S_x	
In.	In.	In.	In.	In.^{2}	Lb.	In.^{4}	In.^{3}	In.	In.^{4}	In.^{3}	In.	In.	In.	In.^{4}	In.^{6}	In.	In.	In.	In.^{4}	In.^{3}	Kip–In.
8.000	2.000	0.135	0.188	1.554	5.284	13.075	3.269	2.901	0.485	0.302	0.559	0.393	0.596	0.00944	5.522	4.843	3.094	–0.922	12.691	3.124	93.519
		0.105	0.188	1.216	4.135	10.335	2.584	2.915	0.386	0.238	0.563	0.381	0.600	0.00447	4.426	4.856	3.111	–0.929	9.581	2.305	69.002
		0.075	0.094	0.880	2.993	7.599	1.900	2.938	0.283	0.173	0.567	0.366	0.597	0.00165	3.264	4.887	3.132	–0.925	6.519	1.513	45.289
		0.060	0.094	0.706	2.402	6.127	1.532	2.945	0.229	0.140	0.569	0.360	0.599	0.00085	2.650	4.894	3.140	–0.929	4.820	1.066	31.924
7.000	1.500	0.135	0.188	1.284	4.366	7.840	2.240	2.471	0.204	0.168	0.399	0.284	0.416	0.00780	1.805	4.611	2.582	–0.633	7.840	2.240	67.067
		0.105	0.188	1.006	3.421	6.218	1.777	2.486	0.164	0.133	0.404	0.272	0.421	0.00370	1.459	4.619	2.598	–0.640	6.120	1.734	51.929
		0.075	0.094	0.730	2.483	4.603	1.315	2.511	0.121	0.097	0.407	0.257	0.418	0.00137	1.086	4.653	2.622	–0.638	4.219	1.155	34.594
		0.060	0.094	0.586	1.994	3.718	1.062	2.518	0.098	0.079	0.410	0.251	0.420	0.00070	0.885	4.656	2.630	–0.641	3.278	0.882	26.413
6.000	1.500	0.135	0.188	1.149	3.907	5.334	1.778	2.155	0.197	0.166	0.414	0.310	0.447	0.00698	1.250	3.607	2.300	–0.689	5.334	1.778	53.238
		0.105	0.188	0.901	3.064	4.240	1.413	2.169	0.158	0.132	0.419	0.298	0.451	0.00331	1.013	3.620	2.316	–0.696	4.168	1.377	41.229
		0.075	0.094	0.655	2.228	3.150	1.050	2.192	0.117	0.096	0.423	0.283	0.447	0.00123	0.755	3.653	2.338	–0.692	2.868	0.912	27.298
		0.060	0.094	0.526	1.790	2.547	0.849	2.200	0.095	0.078	0.425	0.277	0.449	0.00063	0.616	3.660	2.346	–0.696	2.223	0.693	20.748
		0.048	0.094	0.423	1.437	2.055	0.685	2.205	0.077	0.063	0.427	0.272	0.451	0.00032	0.501	3.665	2.352	–0.698	1.716	0.524	15.697
5.000	1.250	0.105	0.188	0.744	2.529	2.399	0.960	1.796	0.089	0.090	0.347	0.256	0.376	0.00273	0.395	3.001	1.919	–0.580	2.399	0.960	28.730
		0.075	0.094	0.543	1.846	1.797	0.719	1.820	0.067	0.066	0.350	0.241	0.372	0.00102	0.297	3.036	1.940	–0.575	1.700	0.661	19.782
		0.060	0.094	0.436	1.484	1.456	0.583	1.827	0.054	0.054	0.353	0.235	0.374	0.00052	0.244	3.042	1.949	–0.579	1.320	0.503	15.054
		0.048	0.094	0.351	1.192	1.177	0.471	1.832	0.044	0.043	0.355	0.230	0.376	0.00027	0.199	3.047	1.955	–0.582	1.024	0.382	11.451
4.000	1.125	0.105	0.188	0.613	2.083	1.286	0.643	1.449	0.062	0.071	0.319	0.251	0.356	0.00225	0.172	2.267	1.584	–0.555	1.286	0.643	19.251
		0.075	0.094	0.449	1.527	0.973	0.486	1.472	0.047	0.053	0.323	0.235	0.351	0.00084	0.131	2.300	1.604	–0.549	0.938	0.460	13.777
		0.060	0.094	0.361	1.229	0.790	0.395	1.479	0.038	0.043	0.325	0.229	0.353	0.00043	0.108	2.308	1.612	–0.553	0.730	0.350	10.479
		0.048	0.094	0.291	0.988	0.640	0.320	1.484	0.031	0.034	0.327	0.225	0.355	0.00022	0.088	2.314	1.618	–0.555	0.565	0.265	7.946
3.000	1.125	0.105	0.188	0.508	1.726	0.636	0.424	1.120	0.057	0.069	0.336	0.292	0.398	0.00187	0.086	1.598	1.331	–0.637	0.636	0.424	12.704
		0.075	0.094	0.374	1.272	0.487	0.324	1.141	0.043	0.051	0.340	0.275	0.390	0.00070	0.066	1.625	1.345	–0.627	0.467	0.304	9.116
		0.060	0.094	0.301	1.025	0.397	0.265	1.147	0.035	0.041	0.342	0.269	0.392	0.00036	0.054	1.634	1.353	–0.631	0.363	0.230	6.888
		0.048	0.094	0.243	0.825	0.322	0.215	1.153	0.029	0.033	0.344	0.264	0.393	0.00019	0.045	1.641	1.360	–0.634	0.280	0.173	5.181
2.000	1.125	0.105	0.188	0.403	1.369	0.241	0.241	0.773	0.050	0.064	0.351	0.355	0.451	0.00148	0.032	1.175	1.135	–0.753	0.241	0.241	7.209
		0.075	0.094	0.299	1.017	0.187	0.187	0.792	0.038	0.048	0.356	0.335	0.438	0.00056	0.025	1.191	1.138	–0.736	0.179	0.174	5.211
		0.060	0.094	0.241	0.821	0.154	0.154	0.798	0.031	0.039	0.358	0.329	0.440	0.00029	0.021	1.200	1.145	–0.738	0.139	0.130	3.907
		0.048	0.094	0.195	0.661	0.126	0.126	0.804	0.025	0.032	0.360	0.324	0.441	0.00015	0.017	1.207	1.151	–0.741	0.107	0.097	2.907

TABLE 3

Z-SECTION WITH STIFFENED FLANGES

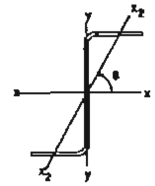




See notes on page V–25

Size							Properties of Full Section											Eff. Section Properties		Moment Capacity M_a
						Wgt.	Axis x–x			Axis y–y			Axis							
D	B	t	d	R	Area	per Foot	I_x	S_x	r_x	I_y	S_y	r_y	I_{xy}	$x_2 - x_2$ r_{min}	90°–θ	J	C_w	I_x	S_x	
In.	In.	In.	In.	In.	In.^{2}	Lb.	In.^{4}	In.^{3}	In.	In.^{4}	In.^{3}	In.	In.^{4}	In.	Deg.	In.^{4}	In.^{6}	In.^{4}	In.^{3}	Kip–In.
12.000	3.500	0.135	1.010	0.188	2.706	9.199	56.267	9.378	4.560	5.967	1.738	1.485	13.151	1.006	13.803	0.01644	162.474	55.387	9.135	273.488
		0.105	0.900	0.188	2.097	7.129	43.836	7.306	4.573	4.535	1.315	1.471	10.096	0.999	13.596	0.00771	123.250	40.904	6.546	195.994
10.000	3.500	0.135	1.010	0.188	2.436	8.281	36.526	7.305	3.873	5.967	1.738	1.565	10.866	1.013	17.709	0.01480	108.445	35.957	7.110	212.889
		0.105	0.900	0.188	1.887	6.415	28.506	5.701	3.887	4.535	1.315	1.550	8.355	1.006	17.440	0.00693	82.180	26.513	5.071	151.837
		0.075	0.720	0.094	1.344	4.568	20.533	4.107	3.909	3.109	0.898	1.521	5.842	0.996	16.923	0.00252	54.811	17.571	3.201	95.029
9.000	3.250	0.135	1.000	0.188	2.230	7.584	27.158	6.035	3.489	4.868	1.530	1.477	8.505	0.945	18.674	0.01355	71.593	27.157	6.035	180.688
		0.105	0.840	0.188	1.717	5.836	21.077	4.684	3.504	3.594	1.124	1.447	6.422	0.931	18.151	0.00631	52.550	20.030	4.307	128.960
		0.075	0.700	0.094	1.228	4.176	15.287	3.397	3.528	2.516	0.783	1.431	4.557	0.927	17.756	0.00230	35.747	13.551	2.807	84.034
		0.060	0.610	0.094	0.976	3.319	12.182	2.707	3.533	1.941	0.603	1.410	3.566	0.918	17.427	0.00117	27.515	9.703	1.875	56.123
8.000	3.000	0.135	0.930	0.188	2.009	6.831	19.372	4.843	3.105	3.778	1.288	1.371	6.357	0.868	19.596	0.01221	43.700	19.371	4.843	144.996
		0.105	0.810	0.188	1.553	5.279	15.125	3.781	3.121	2.845	0.965	1.354	4.868	0.860	19.203	0.00571	32.762	14.727	3.613	108.164
		0.075	0.700	0.094	1.116	3.793	11.046	2.761	3.147	2.028	0.685	1.348	3.500	0.862	18.909	0.00209	22.664	9.965	2.341	70.098
		0.060	0.600	0.094	0.885	3.008	8.791	2.198	3.152	1.552	0.523	1.324	2.725	0.851	18.488	0.00106	17.277	7.397	1.667	49.907
7.000	2.750	0.135	0.880	0.188	1.793	6.097	13.266	3.790	2.720	2.900	1.081	1.272	4.636	0.794	20.907	0.01089	25.576	13.266	3.790	113.483
		0.105	0.880	0.188	1.410	4.794	10.553	3.015	2.736	2.355	0.873	1.292	3.727	0.805	21.140	0.00518	20.824	10.548	3.013	90.203
		0.075	0.700	0.094	1.003	3.411	7.659	2.188	2.763	1.607	0.592	1.266	2.612	0.796	20.399	0.00188	13.678	7.033	1.904	57.020
		0.060	0.600	0.094	0.795	2.702	6.099	1.742	2.770	1.227	0.451	1.242	2.033	0.785	19.922	0.00095	10.388	5.368	1.422	42.571
6.000	2.500	0.135	0.820	0.188	1.574	5.353	8.575	2.858	2.334	2.155	0.886	1.170	3.235	0.716	22.609	0.00956	13.873	8.575	2.858	85.580
		0.105	0.820	0.188	1.240	4.215	6.843	2.281	2.349	1.758	0.718	1.191	2.610	0.728	22.874	0.00456	11.356	6.843	2.281	68.296
		0.075	0.820	0.094	0.909	3.089	5.124	1.708	2.375	1.353	0.549	1.220	1.977	0.746	23.180	0.00170	8.532	4.944	1.607	48.103
		0.060	0.600	0.094	0.705	2.396	4.012	1.337	2.386	0.950	0.384	1.161	1.463	0.718	21.845	0.00085	5.865	3.606	1.125	33.692
5.000	2.000	0.135	0.700	0.188	1.272	4.325	4.684	1.874	1.919	1.070	0.554	0.917	1.680	0.568	21.456	0.00773	4.904	4.684	1.873	56.090
		0.105	0.700	0.188	1.005	3.416	3.761	1.504	1.935	0.883	0.454	0.938	1.368	0.579	21.780	0.00369	4.063	3.761	1.504	45.036
		0.075	0.600	0.094	0.726	2.467	2.797	1.119	1.963	0.637	0.325	0.937	0.998	0.583	21.376	0.00136	2.805	2.760	1.093	32.713
		0.060	0.500	0.094	0.573	1.948	2.227	0.891	1.972	0.480	0.244	0.915	0.772	0.573	20.726	0.00069	2.094	2.053	0.780	23.365
		0.048	0.500	0.094	0.461	1.568	1.804	0.722	1.978	0.393	0.199	0.924	0.629	0.577	20.860	0.00035	1.719	1.632	0.614	18.377
4.000	2.000	0.135	0.700	0.188	1.137	3.866	2.752	1.376	1.556	1.070	0.554	0.970	1.313	0.556	28.685	0.00671	3.000	2.751	1.376	41.189
		0.105	0.700	0.188	0.900	3.059	2.219	1.110	1.571	0.883	0.454	0.991	1.071	0.567	29.031	0.00331	2.496	2.219	1.109	33.215
		0.075	0.600	0.094	0.651	2.212	1.664	0.832	1.599	0.637	0.325	0.990	0.786	0.570	28.426	0.00122	1.716	1.644	0.813	24.341
		0.060	0.500	0.094	0.513	1.744	1.330	0.665	1.610	0.480	0.244	0.967	0.610	0.561	27.572	0.00062	1.274	1.223	0.578	17.303
		0.048	0.500	0.094	0.413	1.404	1.079	0.539	1.616	0.393	0.199	0.976	0.498	0.565	27.717	0.00032	1.047	0.972	0.454	13.593
3.500	2.000	0.135	0.700	0.188	1.069	3.636	2.003	1.145	1.369	1.070	0.554	1.000	1.130	0.542	33.783	0.00650	2.242	2.003	1.144	34.265
		0.105	0.700	0.188	0.847	2.880	1.620	0.926	1.383	0.883	0.454	1.021	0.923	0.552	34.120	0.00311	1.871	1.620	0.926	27.714
		0.075	0.600	0.094	0.613	2.085	1.222	0.698	1.412	0.637	0.325	1.020	0.680	0.556	33.367	0.00115	1.282	1.208	0.683	20.444
		0.060	0.500	0.094	0.483	1.642	0.979	0.559	1.424	0.480	0.244	0.997	0.529	0.546	32.376	0.00058	0.947	0.899	0.484	14.506
		0.048	0.500	0.094	0.389	1.323	0.795	0.454	1.430	0.393	0.199	1.005	0.432	0.550	32.520	0.00030	0.780	0.715	0.380	11.389
3.000	1.750	0.105	0.700	0.188	0.742	2.523	1.017	0.678	1.171	0.618	0.364	0.912	0.610	0.487	35.937	0.00273	0.990	1.017	0.678	20.297
		0.075	0.530	0.094	0.528	1.794	0.767	0.512	1.206	0.418	0.244	0.890	0.437	0.480	34.107	0.00099	0.617	0.767	0.512	15.316
		0.060	0.530	0.094	0.426	1.45	0.628	0.418	1.213	0.346	0.201	0.900	0.360	0.485	34.302	0.00051	0.514	0.617	0.405	12.132
		0.048	0.410	0.094	0.332	1.130	0.499	0.332	1.225	0.251	0.145	0.868	0.274	0.472	32.822	0.00026	0.362	0.442	0.272	8.146

Z–SECTION WITH UNSTIFFENED FLANGES



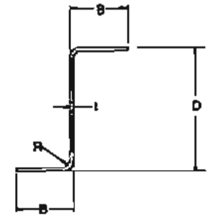


TABLE 4

See notes on page V–25

Size						Properties of Full Section											Eff. Section Properties		Moment Capacity
						Axis x–x			Axis y–y				Axis						
D	B	t	R	Area	Wgt. per Foot	I_x	S_x	r_x	I_y	S_y	r_y	I_{xy}	x_2-x_2 r_{min}	90°–θ	J	C_w	I_x	S_x	M_a
In.	In.	In.	In.	In.^{2}	Lb.	In.^{4}	In.^{3}	In.	In.^{4}	In.^{3}	In.	In.^{4}	In.	Deg.	In.^{4}	In.^{6}	In.^{4}	In.^{3}	Kip–In.
8.000	2.000	0.135	0.188	1.554	5.284	13.076	3.269	2.901	0.649	0.336	0.646	1.987	0.467	8.868	0.00944	7.562	12.691	3.124	93.519
		0.105	0.188	1.216	4.135	10.335	2.584	2.915	0.517	0.265	0.652	1.575	0.471	8.895	0.00447	6.061	9.581	2.305	69.002
		0.075	0.094	0.880	2.993	7.599	1.900	2.938	0.378	0.193	0.655	1.145	0.477	8.800	0.00165	4.460	6.519	1.513	45.289
		0.060	0.094	0.706	2.402	6.127	1.532	2.945	0.306	0.155	0.658	0.925	0.479	8.814	0.00085	3.621	4.820	1.066	31.924
7.000	1.500	0.135	0.188	1.284	4.366	7.840	2.240	2.471	0.264	0.184	0.453	0.955	0.337	7.074	0.00780	2.429	7.840	2.240	67.067
		0.105	0.188	1.006	3.421	6.218	1.777	2.486	0.212	0.146	0.459	0.761	0.341	7.112	0.00370	1.964	6.120	1.734	51.929
		0.075	0.094	0.730	2.483	4.603	1.315	2.511	0.156	0.107	0.463	0.556	0.347	7.021	0.00137	1.458	4.219	1.155	34.594
		0.060	0.094	0.586	1.994	3.718	1.062	2.518	0.127	0.086	0.465	0.450	0.349	7.041	0.00070	1.188	3.278	0.882	26.413
6.000	1.500	0.135	0.188	1.149	3.907	5.334	1.778	2.155	0.264	0.184	0.479	0.816	0.344	8.918	0.00698	1.715	5.334	1.778	53.238
		0.105	0.188	0.901	3.064	4.240	1.413	2.169	0.212	0.146	0.485	0.651	0.348	8.952	0.00331	1.389	4.168	1.377	41.229
		0.075	0.094	0.655	2.228	3.150	1.050	2.193	0.156	0.107	0.488	0.476	0.355	8.816	0.00123	1.032	2.868	0.912	27.298
		0.060	0.094	0.526	1.790	2.547	0.849	2.200	0.127	0.086	0.491	0.385	0.357	8.835	0.00063	0.842	2.223	0.693	20.748
		0.048	0.094	0.423	1.437	2.055	0.685	2.205	0.103	0.070	0.493	0.311	0.359	8.850	0.00032	0.685	1.716	0.524	15.697
5.000	1.250	0.105	0.188	0.744	2.529	2.399	0.960	1.796	0.120	0.100	0.401	0.370	0.287	9.003	0.00273	0.543	2.399	0.960	28.730
		0.075	0.094	0.543	1.846	1.797	0.719	1.820	0.089	0.073	0.405	0.272	0.294	8.830	0.00102	0.407	1.700	0.661	19.782
		0.060	0.094	0.436	1.484	1.456	0.583	1.827	0.073	0.060	0.408	0.221	0.296	8.853	0.00052	0.333	1.320	0.503	15.054
		0.048	0.094	0.351	1.192	1.177	0.471	1.832	0.059	0.048	0.410	0.179	0.298	8.870	0.00027	0.272	1.024	0.382	11.451
4.000	1.500	0.060	0.094	0.406	1.382	0.965	0.483	1.541	0.127	0.086	0.559	0.256	0.369	15.693	0.00049	0.335	0.819	0.376	11.245
4.000	1.125	0.105	0.188	0.613	2.083	1.286	0.643	1.449	0.086	0.080	0.375	0.237	0.259	10.764	0.00225	0.240	1.286	0.643	19.251
		0.075	0.094	0.449	1.527	0.973	0.486	1.472	0.064	0.059	0.378	0.174	0.267	10.500	0.00084	0.181	0.938	0.460	13.777
		0.060	0.094	0.361	1.229	0.791	0.395	1.479	0.052	0.048	0.381	0.142	0.269	10.521	0.00043	0.149	0.730	0.350	10.47
		0.048	0.094	0.291	0.988	0.640	0.320	1.485	0.043	0.039	0.383	0.115	0.271	10.538	0.00022	0.122	0.565	0.265	7.946
3.000	1.500	0.060	0.094	0.346	1.178	0.494	0.329	1.194	0.127	0.086	0.606	0.191	0.364	23.054	0.00040	0.172	0.412	0.248	7.417
3.000	1.125	0.105	0.188	0.508	1.726	0.636	0.424	1.120	0.086	0.080	0.412	0.176	0.261	16.288	0.00187	0.123	0.636	0.424	12.704
		0.075	0.094	0.374	1.272	0.487	0.324	1.141	0.064	0.059	0.414	0.130	0.271	15.802	0.00070	0.094	0.467	0.304	9.116
		0.060	0.094	0.301	1.025	0.397	0.265	1.147	0.052	0.048	0.417	0.106	0.273	15.800	0.00036	0.077	0.363	0.230	6.888
		0.048	0.094	0.243	0.825	0.322	0.215	1.153	0.043	0.039	0.420	0.086	0.275	15.799	0.00019	0.063	0.280	0.173	5.181
2.000	1.125	0.105	0.188	0.403	1.369	0.241	0.241	0.773	0.086	0.080	0.462	0.115	0.248	28.036	0.00148	0.047	0.241	0.241	7.209
		0.075	0.094	0.299	1.017	0.188	0.188	0.792	0.064	0.059	0.464	0.086	0.262	27.111	0.00056	0.036	0.179	0.174	5.211
		0.060	0.094	0.241	0.821	0.154	0.154	0.798	0.052	0.048	0.466	0.070	0.264	27.020	0.00029	0.030	0.139	0.130	3.907
		0.048	0.094	0.195	0.661	0.126	0.126	0.804	0.043	0.039	0.468	0.057	0.266	26.949	0.00015	0.025	0.107	0.097	2.907
1.500	1.500	0.048	0.094	0.207	0.702	0.084	0.112	0.639	0.103	0.070	0.706	0.076	0.287	41.503	0.00016	0.027	0.062	0.070	2.089
		0.036	0.094	0.156	0.530	0.065	0.086	0.644	0.078	0.053	0.708	0.058	0.289	41.686	0.00007	0.021	0.044	0.047	1.395

SPECIFICATION

SPECIFICATION FOR THE DESIGN OF COLD–FORMED STEEL STRUCTURAL MEMBERS

AUGUST 19, 1986, EDITION
WITH
DECEMBER 11, 1989 ADDENDUM

Cold–Formed Steel Design Manual – Part I

AMERICAN IRON AND STEEL INSTITUTE
1133 15th STREET, NW
WASHINGTON, DC 20005–2701

1st Printing – January, 1991

Produced by Computerized Structural Design, Inc.
Milwaukee, Wisconsin

PREFACE
TO 1989 ADDENDUM

Several changes and additions to the 1986 Edition of AISI's Specification for the Design of Cold–Formed Steel Structural Members are contained in the 1989 Addendum to the Specification. The results of continuing research, advances in design techniques, development of new steels, and the needs of the design profession and consuming industries have all given impetus to the Addendum.

AISI acknowledges the continuing support and hard work by the members of Advisory Group on the Specification and its subcommittees. The current membership lists follow this preface.

Development and publication of the Specification is sponsored by AISI's Construction Marketing Committee, Light Construction Subcommittee, under the auspices of AISI's Committee on Construction Codes and Standards.

All users of the Specification are encouraged to continue to provide their invaluable recommendations for improvement.

American Iron and Steel Institute
December 11, 1989

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PREFACE

In memory of George Winter, in
recognition of his many contributions
and achievements to the enhancements
of cold–formed steel design.

The newly published Edition of AISI's Specification for the Design of Cold–Formed Steel Structural Members represents a major revision, with many changes made to keep the Specification responsive to the needs of users. It reflects the results of research projects and improvements in design techniques. Moreover, it embodies the results of efforts to simplify the use of the Specification by changes in its format, organization, and content. To accomplish this simplification, relevant sections needed to design a particular member, such as a beam or a column, have been collected together as much as possible.

AISI acknowledges the devoted efforts of the members of the Advisory Group on the Specification of the Design of Cold–Formed Steel Structural Members. This group, comprised of consulting engineers, researchers, designers from companies manufacturing cold–formed steel members, components, assemblies, and complete structures, and specialists from the steel producing industry, has met two to three times per year since its establishment in 1973. Its current members, who have made extensive contributions of time and effort in developing and reaching consensus on the changes which have been described above, are:

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The activities of the Advisory Group are sponsored by AISI's Committee of Sheet Steel Producers. The Specification is issued under the auspices of AISI's Committee on Construction Codes and Standards.

Users of the Specification are invited to continue to offer their valuable comments and suggestions. The cooperation of all involved, the users as well as the writers, is needed to continue to keep the Specification up to date and a useful tool for the designer.

American Iron and Steel Institute
August 19, 1986

*Past Chairman

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A_b	$b_1t + A_s$, for transverse stiffeners at interior support and under concentrated load, and $b_2t + A_s$, for transverse stiffeners at end support	B6.1, E3.4
A_c	$18t^2 + A_s$, for transverse stiffeners at interior support and under concentrated load, and $10t^2 + A_s$, for transverse stiffeners at end support	B6.1
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SYMBOLS AND DEFINITIONS

Symbol	Definition	Section
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C_y	Compression strain factor	C3.1.1
C_0	Initial column imperfection	D4.1
C_1	Term used to compute shear strain in wall board	B4, B4.1, D4.2
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E_0	Initial column imperfection; a measure of the initial twist of the stud from the initial, ideal, unbuckled location	D4.1

SYMBOLS AND DEFINITIONS

Symbol	Definition	Section
E_1	Term used to compute shear strain in wallboard	D4.1
E'	Inelastic modulus of elasticity	D4.1
e_{min}	Minimum allowable distance measured in the line of force from the centerline of a weld to the nearest edge of an adjacent weld or to the end of the connected part which the force is directed	E2.2
e_{min}	The distance e measured in the line of force from the center of a standard hole to the nearest edge of an adjacent hole or to the end of the connected part toward which the force is directed	E3.1
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F_L	Live load factor	F1
F_n	Nominal buckling stress	C4, C6.2, D4.2
F_p	Allowable bearing stress	E3.3, E5.1
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SYMBOLS AND DEFINITIONS

Symbol	Definition	Section
F_{ya}	Average yield point of section	A5.2.2
F_{yc}	Tensile yield point of corners	A5.2.2
F_{yf}	Weighted average tensile yield point of the flat portions	F3.2, A5.2.2
F_{ys}	Yield point of stiffener steel	B6.1
F_{yv}	Tensile yield point of virgin steel specified by Section A3 or established in accordance with Section F3.3	A5.2.2
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f_b	Maximum bending stress equal to the bending moment divided by appropriate section modulus of member	C3.1.3
f_c	Computed stress at design load in the cover plate or sheet	D1.2
f'_c	Specified compression stress of concrete	E5.1
f_d	Computed compressive stress in the element being considered. Calculations are based on the effective section at the load for which deflections are determined.	B2.1, B2.2, B3.1, B4.1, B4.2
f_{d1}, f_{d2}	Computed stresses f_1 and f_2 as shown in Figure B2.3–1. Calculations are based on the effective section at the load for which deflections are deteremined	B2.3
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f_v	Computed shear stress on a bolt	E4
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G′	Inelastic shear modulus	D4.1
g	Vertical distance between two rows of connections nearest to the top and bottom flanges	D1.1

SYMBOLS AND DEFINITIONS

Symbol	Definition	Section
h	Depth of flat portion of web measured along the plane of web	B1.2, B6.2, C3.2, C3.4, C3.5.2
I_a	Adequate moment of inertia of stiffener so that each component will behave as a stiffened element	B1.1, B4, B4.1, B4.2
I_b	Moment of inertia of the full unreduced section about the bending axis	C5
I_o	Moment of inertia of effective section about its major axis	C3.1.3
I_s	Actual moment of inertia of the full stiffener about its own centroidal axis parallel to the element to be stiffened	B1.1, B4, B4.1, B4.2, B5
I_{sf}	Moment of inertia of the full area of the multiple stiffened element, including the intermediate stiffeners, about its own centroidal axis parallel to the element to be stiffened	B5
I_{yc}	Moment of inertia of the compression portion of a section about the centroidal axis of the entire section parallel to the web, using the full unreduced section	C3.1.2
I_x, I_y	Moment of inertia of full section about principal axis	D3.2.2, D1.1
I_{xy}	Product of inertia of full section about major and minor centroidal axes	D3.2.2, D4.1
J	St. Venant torsion constant	C3.1.1
j	Section property for torsional–flexural buckling	C3.1.1
K	Effective length factor	C3.1.3, C4, C4.1, C5
K′	A constant	D3.2.2
K_b	Effective length factor in the plane of bending	C5
K_t	Effective length factor for torsion	C3.1.2
K_x	Effective length factor for bending about x–axis	C3.1.2
K_y	Effective length factor for bending about y–axis	C3.1.2
k	Plate buckling coefficient	B2.1, B2.3, B3.1, B3.2, B4 B4.1, B4.2
k_v	Shear buckling coefficient	B6.2, C3.2
L	Full span for simple beams, distance between inflection points for continuous beams, twice the length of cantilever beams	B1.1c, D3.2.1

SYMBOLS AND DEFINITIONS

Symbols	Definition	Section
L	Length of seam weld not including the circular ends	E2.3
L	Length of fillet weld	E2.4, E2.5
L	Unbraced length of member	C3.1.2, C3.1.3, C4.1, D1.1, D4, D4.1
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L_a	Length of the portion of the span between supports where the flange that is not connected to the sheathing is in compression	C3.1.3
L_{st}	Length of transverse stiffener	B6.1
L_t	Unbraced length of compression member for torsion	C3.1.1
L_x	Unbraced length of compression member for bending about x–axis	C3.1.1
L_y	Unbraced length of compression member for bending about y–axis	C3.1.1
M	Applied bending moment	C3.3, C3.5.1, C3.5.2
M_a	Allowable bending moment permitted if bending stress only exists	C3.1, C3.3, C3.5.1, C3.5.2, C6.1
M_{ax}, M_{ay}	Allowable moments about the centroidal axes determined in accordance with Section C3	C5
M_{axo} M_{ayo}	Allowable moments about the centroidal axes determined in accordance with Section 3.1 excluding the provisions of Section 3.1.2	C3.3, C3.5
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M_e	Elastic critical moment	C3.1.2
M_n	Nominal moment strength	C3.1, C3.1.1, C3.1.2, C3.1.3, C6.1
M_x, M_y	Applied moments about the centroidal axes determined in accordance with Section C3	C5
M_y	Moment causing a maximum strain of e_y	B2.1, C3.1
M_1	Smaller end moment	C3.1.1, C5
M_2	Larger end moment	C3.1.1, C5
m	Distance from the shear center of one channel to the mid–plane of its web	D3.2.2, D1.1

SYMBOLS AND DEFINITIONS

Symbol	Definition	Section
m	$0.192\,(F_{uv}/F_{yv}) - 0.068$	A5.2.2
N	Actual length of bearing	D3.6
n	Number of holes	E4
n_p	Number of parallel purlin lines	D3.2.1
P	Concentrated load or reaction	C3.5
P	Applied axial load	C5, D4.1
P	Force transmitted by bolt	E3, E3.1
P	Force transmitted by weld	E2, E2.2
P_a	Allowable concentrated load or reaction for one transverse stiffener	B6.1
P_{ao}	Allowable axial load determined in accordance with Section C4 for L = 0	C5
P_L	Force to be resisted by intermediate beam brace	B3.2.2
P_n	Nominal axial strength of member	C4, C6.2
P_n	Nominal strength of connection component	E2, E2.2, E2.3, E2.4, E2.5
$\overline{Q}$	Design shear rigidity for sheathing on both side of the wall assembly	D4.1
q	Uniformly distributed load in the plane of the web	C3.1.3, D1.1
q_w	Allowable uniform load	C3.1.3
$\overline{q}$	Design shear rigidity for sheathing per inch of stud spacing	D4.1
$\overline{q}_o$	Factor used to determine design shear rigidity	D4.1
q_u	Maximum uniformly distributed load in the plane of the web	C3.1.3
R	Inside bend radius	A5.2.2, C3.4
|| R	Reduction Factor	C3.1.3
R	Coefficient	C4, C6.2
R	Required load carrying capacity	F1
r	Radius of gyration of full unreduced cross section	C3.1.1, C4, C4.1
r	Force transmitted by the bolt or bolts at the section considered, divided by the tension force in member at that section	E3.2
r_{cy}	Radius of gyration of one channel about its centroidal axis parallel to web	D1.1
r_o	Polar radius of gyration of cross section about the shear center	C3.1.1, C4.2, D4.1
r_x, r_y	Radius of gyration of cross section about centroidal principal axis	C3.1.1

SYMBOLS AND DEFINITIONS

	Symbol	Definition	Section
	r_I	Radius of gyration of I–section about the axis perpendicular to the direction in which buckling would occur for the given conditions of end support and intermediate bracing	D1.1
	S	$1.28\sqrt{E/f}$	B4, B4.1
||	S_a	In–plane load carrying capacity for diaphragms	D5
	S_c	Elastic section modulus of the effective section calculated at a stress M_c/S_f in the extreme compression fiber	C3.1.1, C3.1.2, C4
||	S_e	Elastic section modulus of the effective section calculated with extreme compression or tension fiber at F_y	C3.1.1, C3.1.3
	S_f	Elastic seciton modulus of full unreduced section for the extreme compression fiber	C3.1.1, C3.1.2, C6.1
||	S_f	Nominal diaphragm shear strength	D5
	s_{max}	Maximum permissible longitudinal spacing of welds or other connectors joining two channels to form an I–section	D1.1
	s	Fastener spacing	D1.2, D4.1
	s	Spacing in line of stress of welds, rivets, or bolts connecting a compression coverplate or sheet to a non–integral stiffener or other element	E3.2
	s	Weld spacing	D1.1
	T_a	Allowable tensile strength	C2
	T_n	Nominal tensile strength	C2
	T_s	Strength of connection in tension	D1.1
||	t	Base steel thickness of any element or section	A1.2, A3.4, A5.2.1, B1.1, B1.1b, B1.2, B2.1, B4, B4.1, B4.2, B5, B6.1, C3.1.1., C3.1.3, C3.2, C3.4, C3.5.2, C4, C6.1, C6.2, D1.2, D4, E2.4, E2.5
	t	Total thickness of the two welded sheets	E2.2
	t	Thickness of thinnest connected part	E2.2, E3.1, E4
	t_s	Equivalent thickness of a multiple–stiffened element	B5, B6.1
	t_w	Effective throat of weld	E2.4, E2.5
	V	Actual shear force	C3.3
	V_a	Allowable shear force	B6.2, C3.2, C3.3

SYMBOLS AND DEFINITIONS

Symbol	Definition	Section
W	Total load supported by the purlin lines between adjacent supports, lbs.	D3.2.1.
w	Flat width of element exclusive of radii	A1.2, B1.1, B2.1,B2.2, B3.1, B4, B4.1, B4.2, B5, C3.1.1., C3.1.3, C4, D1.2
w	Flat width of the beam flange which contacts the bearing plate	C3.5
w_f	Width of flange projection beyond the web or half the distance between webs for box– or U–type sections	B1.1
w_f	Projection of flanges from inside face of web	D1.1
w_1, w_2	Leg on weld	E2.4
x	Distance from concentrated load to brace	D3.2
x_o	Distance from shear center to centroid along the principal x–axis	C3.1.1, C4.2, D4.1
Y	Yield point of web steel divided by yield point of stiffener steel	B6.2
$1/\alpha_x$ $1/\alpha_y$	Magnification factors	C5
β	Coefficient	C4.2, D4.1
γ	Actual shear strain in the sheathing	D4.1
$\bar{\gamma}$	Permissible shear strain of the sheathing	D4.1
θ	Angle between web and bearing surface >45° but not more than 90°	C3.4
θ	Angle between the vertical and the plane of the web of the Z–section, degrees	D3.2.1
σ	Stress related to shear strain in sheathing	D4.1
σ_{CR}	Theoretical elastic buckling stress	D4.1
σ_t	Torsional buckling stress	C3.1.1, C4.2, D4.1
ρ	Reduction factor	B2.1
λ, λ_c	Slenderness factors	B2.1, C3.5
ψ	f_2/f_1	B2.3
Ω_b	Factor of safety for bearing	E3.3
Ω_c	Factor of safety for axial compression	B6.1, C4, C5, C6.2, D4.1

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Symbol	Definition	Section
Ω_e	Factor of safety for sheet tearing	E2.2, E3.1
Ω_f	Factor of safety for flexure	C3.1, C6.1
Ω_s	Factor of safety for diaphragm shear	D5
Ω_{st}	Factor of safety for end crushing of transverse stiffener	B6.1
Ω_t	Factor of safety for tension on net section	C2, E3.2
Ω_v	Factor of safety for shear rupture	E4
Ω_w	Factor of safety for welded connections	E2

SPECIFICATION FOR THE DESIGN OF COLD–FORMED STEEL STRUCTURAL MEMBERS AUGUST 19, 1986 EDITION WITH DECEMBER 11, 1989 ADDENDUM

A. GENERAL PROVISIONS

A1 Limits of Applicability and Terms

A1.1 Scope and Limits of Applicability

This Specification shall apply to the design of structural members cold–formed to shape from carbon or low–alloy steel sheet, strip, plate or bar not more than one inch in thickness and used for load–carrying purposes in buildings. It may also be used for structures other than buildings provided appropriate allowances are made for dynamic effects. Appendices to this Specification shall be considered as integral parts of the Specification.

A1.2 Terms

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

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

(b) *Unstiffened Compression Elements.* An unstiffened compression element is a flat compression element which is stiffened at only one edge parallel to the direction of stress.

(c) *Multiple–Stiffened Elements.* A multiple–stiffened element is an element that is stiffened between webs, or between a web and a stiffened edge, by means of intermediate stiffeners which are parallel to the direction of stress. A *sub–element* is the portion between adjacent stiffeners or between web and intermediate stiffener or between edge and intermediate stiffener.

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

(e) *Effective Design Width.* Where the flat width of an element is reduced for design purposes, the reduced design width is termed the effective width or effective design width.

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

(g) *Torsional–Flexural Buckling.* Torsional–flexural buckling is a mode of buckling in which compression members can bend and twist simultaneously.

(h) *Point–Symmetric Section.* A point–symmetric section is a section symmetrical about a point (centroid) such as a Z–section having equal flanges.

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

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

(k) *Confirmatory Test.* A confirmatory test is a test made, when desired, on members, connections, and assemblies designed according to the provisions of Sections A through E of this Specification or its specific references, in order to compare actual versus calculated performance.

(l) *Performance Test.* A performance test is a test made on structural members, connections, and assemblies whose performance cannot be determined by the provisions of Sections A through E of this Specification or its specific references.

(m) *Virgin Steel.* Virgin steel refers to steel as received from the steel producer or warehouse before being cold worked as a result of fabricating operations.

(n) *Virgin Steel Properties.* Virgin steel properties refer to mechanical properties of virgin steel such as yield point, tensile strength, and elongation.

(o) *Specified Minimum Yield Point.* The specified minimum yield point is the lower limit of yield point which must be equaled or exceeded in a specification test to qualify a lot of steel for use in a cold–formed steel structural member designed at that yield point.

(p) *Cold–Formed Steel Structural Members.* Cold–formed steel structural members are shapes which are manufactured by press–braking blanks sheared from sheets, cut lengths of coils or plates, or by roll forming cold– or hot–rolled coils or sheets; both forming operations being performed at ambient room temperature, that is, without manifest addition of heat such as would be required for hot forming.

A1.3 Units of Symbols and Terms

The Specification is written so that any compatible system of units may be used except where explicitly stated otherwise in the text of these provisions.

A2 Non–Conforming Shapes and Construction

The provisions of the Specification are not intended to prevent the use of alternate shapes or constructions not specifically prescribed herein. Such alternates shall meet the provisions of Section F of the Specification and be approved by the appropriate building code authority.

A3 Material

A3.1 Applicable Steels

This Specification requires the use of steel of structural quality as defined in general by the provisions of the following specifications of the American Society for Testing and Materials:

ASTM A36/A36M, Structural Steel

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

ASTM A441M, High–Strength Low–Alloy Structural Manganese Vanadium Steel

ASTM A446/A446M (Grades A, B, C, D, & F) Steel, Sheet, Zinc–Coated (Galvanized) by the Hot–Dip Process, Structural (Physical) Quality

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

ASTM A529/A529M, Structural Steel with 42 ksi Minimum Yield Point (1/2 in. Maximum Thickness)

ASTM A570/A570M Steel, Sheet and Strip, Carbon, Hot-Rolled, Structural Quality

ASTM A572/A572M, High-Strength Low-Alloy Columbium-Vanadium Steels of Structural Quality

ASTM A588/A588M, High-Strength Low-Alloy Structural Steel with 50 ksi Minimum Yield Point to 4 in. Thick

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

ASTM A607 Steel Sheet and Strip, High Strength, Low Alloy, Columbium or Vanadium, or both, Hot-Rolled and Cold-Rolled

ASTM A611 (Grades A, B, C, & D) Steel, Sheet, Carbon, Cold-Rolled, Structural Quality

ASTM A715 (Grades 50 and 60) Sheet Steel and Strip, High-Strength, Low-Alloy, Hot-Rolled, With Improved Formability

ASTM A792 (Grades 33, 37, 40 & 50) Steel Sheet, Aluminum-Zinc Alloy-Coated by the Hot-Dip Process, General Requirements

A3.2 Other Steels

The listing in Section A3.1 does not exclude the use of steel up to and including one inch in thickness ordered or produced to other than the listed specifications provided such steel conforms to the chemical and mechanical requirements of one of the listed specifications or other published specification which establishes its properties and suitability, and provided it is subjected by either the producer or the purchaser to analyses, tests and other controls to the extent and in the manner prescribed by one of the listed specifications and Section A3.3.

A3.3 Ductility

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

A3.3.1 The ratio of tensile strength to yield point shall not be less than 1.08, and the total elongation shall not be less than 10 percent for a two-inch gage length or 7 percent for an eight-inch gage length standard specimen tested in accordance with ASTM A370-77$^{\varepsilon}$. If these requirements cannot be met, the following criteria shall be satisfied: (1) local elongation in a 1/2 inch gage length across the fracture shall not be less than 20%, (2) uniform elongation outside the fracture shall not be less than 3%*. When material ductility is determined on the basis of the local and uniform elongation criteria, the use of such material is restricted to the design of purlins and girts** in accordance with Sections C3.1.1(a), C3.1.2, and C3.1.3. For purlins and girts subject to combined axial load and bending moment (Section C5), P/P_a shall not exceed 0.15. The provisions of Chapters B through E of this Specification are limited to steels conforming to these requirements.

A3.3.2 Steels conforming to ASTM A446 Grade E and A611 Grade E and other steels which do not meet the provisions of Section A3.3.1 may be used for particular configurations provided (1) the yield strength, F_y, used for design in Chapters B, C and D

* Further information on the test procedures should be obtained from the Commentary.

** Horizontal structural members which support roof deck or panel covering and applied loads principally by bending.

is taken as 75 percent of the specified minimum yield point or 60 ksi, whichever is less and (2) the tensile strength, F_u, used for design in Chapter E is taken as 75 percent of the specified minimum tensile stress or 62 ksi, whichever is less. Alternatively, the suitability of such steels for the configuration shall be demonstrated by load tests in accordance with Section F1. Allowable loads based on these tests shall not exceed the loads calculated according to Chapters B through E, using the specified minimum yield point, F_{sy}, for F_y and the specified minimum tensile strength, F_u.

Allowable loads based on existing use shall not exceed the loads calculated according to Chapters B through E, using the specified minimum yield point, F_{sy}, for F_y and the specified minimum tensile strength, F_u.

A3.4 Delivered Minimum Thickness

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

A4 Loads

A4.1 Dead Load

The dead load to be assumed in design shall consist of the weight of steelwork and all material permanently fastened thereto or supported thereby.

A4.2 Live Load

The live load shall be that stipulated by the applicable code or specification under which the structure is being designed or that dictated by the conditions involved.

A4.3 Impact Load

For structures carrying live loads which induce impact, the assumed live load shall be increased sufficiently to provide for impact.

A4.4 Wind or Earthquake Loads

Where load combinations specified by the applicable building code include wind or earthquake loads, the resulting forces may be multiplied by 0.75 for strength determination.

A4.5 Ponding

Unless a roof surface is provided with sufficient slope toward points of free drainage or adequate individual drains to prevent the accumulation of rainwater, the roof system shall be investigated by rational analysis to assure stability under ponding conditions.

A5 Structural Analysis and Design

A5.1 Design Basis

This Specification is based upon the allowable stress concept presented in terms of allowable moments and loads. The allowable moments and loads are determined by dividing the corresponding nominal capacities by an accepted factor of safety.

A5.2 Yield Point and Strength Increase from Cold Work of Forming

A5.2.1 Yield Point

The yield point used in design, F_y, shall not exceed the specified minimum yield point of steels as listed in Section A3.1 or A3.2, as established in accordance with Chapter F, or as increased for cold work of forming in Section A5.2.2, or as reduced for low ductility steels in Section A3.3.2.

A5.2.2 Strength Increase from Cold Work of Forming

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

(a) For axially loaded compression members and flexural members whose proportions are such that the quantity ρ is unity as determined according to Section B2 for each of the component elements of the section, the design yield stress, F_{ya}, of the steel shall be determined on the basis of one of the following methods:

(1) full section tensile tests [see paragraph (a) of Section F3.1]

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

(3) computed as follows:

$$F_{ya} = CF_{yc} + (1 - C)\, F_{yf} \qquad (Eq.\ A5.2.2\text{–}1)$$

where

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

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

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

F_{yc} = $B_cF_{yv}/(R/t)^m$, tensile yield point of corners $\qquad (Eq.\ A5.2.2\text{–}2)$

when

$F_{uv}/F_{yv} > 1.2$, $R/t < 7$, minimum included angle $< 120°$

$B_c = 3.69\,(F_{uv}/F_{yv}) - 0.819\,(F_{uv}/F_{yv})^2 - 1.79 \qquad (Eq.\ A5.2.2\text{–}3)$

$m = 0.192\,(F_{uv}/F_{yv}) - 0.068 \qquad (Eq.\ A5.2.2\text{–}4)$

R = Inside bend radius.

F_{yv} = Tensile yield point of virgin steel* specified by Section A3 or established in accordance with Section F3.3

F_{uv} = Ultimate tensile strength of virgin steel* specified by Section A3 or established in accordance with Section F3.3

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

* Virgin steel refers to the condition (i.e., coiled or straight) of the steel prior to the cold–forming operation.

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

A5.3 Serviceability and Durability

A structure shall be designed to perform its required functions during its expected life, including serviceability and durability considerations.

A6 Reference Documents

This Specification recognizes other published and latest approved specifications and manuals for designs contemplated herein, as follows:

- American National Standards Institute, ANSI A58.1–1982, "Minimum Design Loads in Buildings and Other Structures,"* American National Standards Institute, Inc. (ANSI), 1430 Broadway, New York, New York 10018
- American Institute of Steel Construction, "Specification for the Design, Fabrication and Erection of Structural Steel for Buildings," American Institute of Steel Construction (AISC), One East Wacker Drive, Chicago, Illinois 60601, November 1, 1978
- American Welding Society, AWS D1.3–81, "Structural Welding Code – Sheet Steel," American Welding Society (AWS), 550 N.W. LeJeune Road, Miami, Florida 33126
- Research Council on Structural Connections, Allowable Stress Design, "Specification for Structural Joints Using ASTM A325 or A490 Bolts," Research Council on Structural Connections (RCSC), American Institute of Steel Construction (AISC), One East Wacker Drive, Chicago, Illinois 60601, November 13, 1985.
- Metal Building Manufacturers Association, *Low Rise Building Systems Manual*, Metal Building Manufacturers Association (MBMA), 1230 Keith Building, Cleveland, Ohio 44115
- Steel Deck Institute, "Design Manual for Composite Decks, Formed Decks, and Roof Decks," Steel Deck Institute (SDI), Inc., P.O. Box 9506, Canton, Ohio 44711, 1984
- Steel Joist Institute, "Standard Specifications Load Tables and Weight Tables for Steel Joists and Joist Girders," Steel Joist Institute (SJI), Suite A, 1205 48th Avenue North, Myrtle Beach, South Carolina 29577, 1986
- Rack Manufacturers Institute, " Specification for the Design, Testing and Utilization of Industrial Steel Storage Racks," Rack Manufacturers Institute (RMI), 8720 Red Oak Boulevard, Suite 201, Charlotte, North Carolina 28210, 1985
- American Iron and Steel Institute, "Stainless Steel Cold–Formed Structural Design Manual," 1974 Edition, American Iron and Steel Institute (AISI), 1133 15th Street, N.W., Washington, D.C. 20005
- American Society of Civil Engineers, "ASCE Standard, Specification for the Design and Construction of Composite Slabs, "American Society of Civil Engineers (ASCE), 345 East 47th Street, New York, New York 10017, October, 1984 .

* For further information contact ASCE, 345 East 47th Street, New York, New York 10017.

- American Iron and Steel Institute, "Tentative Criteria for Structural Applications of Steel Tubing and Pipe," American Iron and Steel Institute (AISI), 1133 15th Street, N.W., Washington, D.C. 20005, August, 1976

In addition to the above references this Specification recognizes the following standards from the American Society for Testing and Materials (ASTM), 1916 Race Street, Philadelphia, Pennsylvania 19013:

ASTM A36/A36M–84a, Structural Steel

ASTM A194–88, Carbon and Alloy Steel Nuts for Bolts for High–Pressure and High–Temperature Service

ASTM A242/A242M–85, High–Strength Low–Alloy Structural Steel

ASTM A307–84 (Type A), Carbon Steel Externally and Internally Threaded Standard Fasteners

ASTM A325–84, High Strength Bolts for Structural Steel Joints

ASTM A354–84 (Grade BD), Quenched and Tempered Alloy Steel Bolts, Studs, and Other Externally Threaded Fasteners (for diameter of bolt smaller than 1/2 inch)

ASTM A370–77$^{\epsilon}$ Mechanical Testing of Steel Products

ASTM A441M–85, High–Strength Low–Alloy Structural Manganese Vanadium Steel

ASTM A446/A446M–85 (Grades A, B, C, D, & F) Steel, Sheet, Zinc–Coated (Galvanized) by the Hot–Dip Process, Structural (Physical) Quality

ASTM A449–84a, Quenched and Tempered Steel Bolts and Studs (for diameter of bolt smaller than 1/2 inch)

ASTM A490–84, Quenched and Tempered Alloy Steel Bolts for Structural Steel Joints.

ASTM A500–84, Cold–Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes

ASTM A529/A529M–85, Structural Steel with 42 ksi Minimum Yield Point (1/2 in. Maximum Thickness)

ASTM A563–88a, Carbon and Alloy Steel Nuts

ASTM A570/A570M–85 Steel, Sheet and Strip, Carbon, Hot–Rolled, Structural Quality

ASTM A572/A572M–85, High–Strength Low–Alloy Columbium–Vanadium Steels of Structural Quality

ASTM A588/A588M–85, High–Strength Low–Alloy Structural Steel with 50 ksi Minimum Yield Point to 4 in. Thick

ASTM A606–85 Steel, Sheet and Strip, High Strength, Low Alloy, Hot–Rolled and Cold–Rolled, with Improved Atmospheric Corrosion Resistance

ASTM A607–85 Steel Sheet and Strip, High Strength, Low Alloy, Columbium or Vanadium, or both, Hot–Rolled and Cold–Rolled

ASTM A611–85 (Grades A, B, C, & D) Steel, Sheet, Carbon, Cold–Rolled, Structural Quality

ASTM A715–85 (Grades 50 & 60) Sheet Steel and Strip, High–Strength, Low–Alloy, Hot–Rolled, With Improved Formability

ASTM A792–85a (Grades 33, 37, 40 & 50) Steel Sheet, Aluminum–Zinc Alloy–Coated by the Hot–Dip Process, General Requirements

ASTM F436–86, Hardened Steel Washers

ASTM F844–83(1988), Washers, Steel, Plain (Flat), Unhardened for General Use

ASTM F959–85, Compressible Washer–Type Direct Tension Indicators for Use with Structural Fasteners

B. ELEMENTS

B1 Dimensional Limits and Considerations

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

(a) *Maximum Flat–Width–to–Thickness Ratios*
Maximum allowable overall flat–width–to–thickness ratios, w/t, disregarding intermediate stiffeners and taking as t the actual thickness of the element, shall be as follows:

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

Simple lip	60
Any other kind of stiffener having $I_s > I_a$ and $D/w < 0.8$ according to Section B4.2	90

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

(3) Unstiffened compression element and elements with an edge stiffener having $I_s < I_a$ and $D/w \leq 0.8$ according to Section B4.2 — 60

Note: Unstiffened compression elements that have w/t ratios exceeding approximately 30 and stiffened compression elements that have w/t ratios exceeding approximately 250 are likely to develop noticeable deformation at the full allowable load, without affecting the ability of the member to carry design loads.

Stiffened elements having w/t ratios larger than 500 may be used with safety to support loads, but substantial deformation of such elements under load may occur and may render inapplicable the design formulas of this Specification.

(b) *Flange Curling*
Where the flange of a flexural member is unusually wide and it is desired to limit the maximum amount of curling or movement of the flange toward the neutral axis, the following formula applies to compression and tension flanges, either stiffened or unstiffened:

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

where

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

t = Flange thickness

d = Depth of beam

c_f = Amount of curling

f_{av} = Average stress in the full, unreduced flange width. (Where members are designed by the effective design width procedure, the average stress equals

the maximum stress multiplied by the ratio of the effective design width to the actual width.)

(c) *Shear Lag Effects – Unusually Short Spans Supporting Concentrated Loads*
Where the span of the beam is less than $30w_f$ (w_f as defined below) and it carries one concentrated load, or several loads spaced farther apart than $2w_f$, the effective design width of any flange, whether in tension or compression, shall be limited to the following:

TABLE B1.1
SHORT, WIDE FLANGES
MAXIMUM ALLOWABLE RATIO OF EFFECTIVE DESIGN WIDTH TO ACTUAL WIDTH

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

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

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

B1.2 Maximum Web Depth–to–Thickness Ratio

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

(a) For unreinforced webs: $(h/t)_{max} = 200$
(b) For webs which are provided with transverse stiffeners satisfying the requirements of Section B6.1:
  (1) When using bearing stiffeners only, $(h/t)_{max} = 260$
  (2) When using bearing stiffeners and intermediate stiffeners, $(h/t)_{max} = 300$
  In the above,
  h = Depth of flat portion of web measured along the plane of web
  t = Web thickness
  Where a web consists of two or more sheets, the h/t ratio shall be computed for the individual sheets.

B2 Effective Widths of Stiffened Elements

B2.1 Uniformly Compressed Stiffened Elements

(a) *Load Capacity Determination*

The effective widths, b, of uniformly compressed elements shall be determined from the following formulas:

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

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

where

w = Flat width as shown in Figure B2.1

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

λ is a slenderness factor determined as follows:

$$\lambda = \frac{1.052}{\sqrt{k}}\left(\frac{w}{t}\right)\sqrt{\frac{f}{E}} \quad (Eq.\ B2.1\text{–}4)$$

where

f for load capacity determination is as follows:

For flexural members:

(1) If Procedure I of Section C3.1.1 is used, $f = F_y$ if the initial yielding is in compression in the element considered.

If the initial yielding is in tension, the compressive stress, f, in the element considered shall be determined on the basis of the effective section at M_y (moment causing initial yield).

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

(3) If Section C3.1.2 is used, then $f = \dfrac{M_c}{S_f}$ as described in that Section in determining S_c

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

E = Modulus of elasticity

k = Plate buckling coefficient

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

(b) *Deflection Determination*

The effective widths, b_d, used in computing deflections shall be determined from the following formulas:

$b_d = w$ when $\lambda \leq 0.673$ (*Eq.* B2.1–5)

$b_d = \rho w$ when $\lambda > 0.673$ (*Eq.* B2.1–6)

where

w = Flat width

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

(1) Procedure I.

A low estimate of the effective width may be obtained from Eqs. B2.1–3

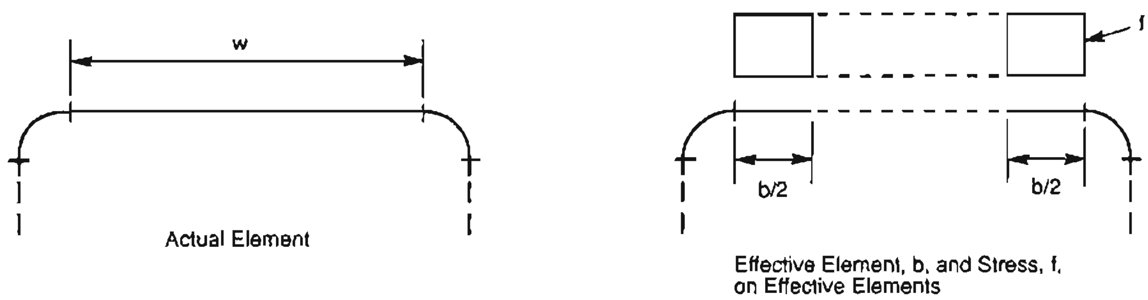


Figure B2.1–1 Stiffened Elements

and B2.1–4 where f_d is substituted for f where f_d is the computed compressive stress in the element being considered.

(2) Procedure II.
For stiffened elements supported by a web on each longitudinal edge an improved estimate of the effective width can be obtained by calculating ρ as follows:

$\rho = 1$ when $\lambda \leq 0.673$ (*Eq.* B2.1–7)

$\rho = (1.358 - 0.461/\lambda)/\lambda$ when $0.673 < \lambda < \lambda_c$ (*Eq.* B2.1–8)

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

ρ shall not exceed 1.0 for all cases.

where

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

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

B2.2 Uniformly Compressed Stiffened Elements with Circular Holes

(a) *Load Capacity Determination*

The effective width, b, of stiffened elements with uniform compression having circular holes shall be determined as follows:

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

center–to–center spacing of holes > 0.50w, and $3d_h$,

$b = w - d_h$ when $\lambda \leq 0.673$ (*Eq.* B2.2–1)

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

b shall not exceed $w - d_h$

where

w = Flat width

d_h = Diameter of holes

λ is as defined in Section B2.1.

(b) *Deflection Determination*

The effective width, b_d, used in deflection calculations shall be equal to b deter-

mined in accordance with Procedure I of Section B2.2a except that f_d is substituted for f, where f_d is the computed compressive stress in the element being considered.

B2.3 Effective Width of Webs and Stiffened Elements with Stress Gradient

(a) *Load Capacity Determination*

The effective widths, b_1 and b_2, as shown in Figure B2.3–1 shall be determined from the following formulas:

$$b_1 = b_e/(3 - \psi) \qquad (Eq.\ B2.3\text{–}1)$$

For $\psi \leq -0.236$

$$b_2 = b_e/2 \qquad (Eq.\ B2.3\text{–}2)$$

$b_1 + b_2$ shall not exceed the compression portion of the web calculated on the basis of effective section

For $\psi > -0.236$

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

where

b_e = Effective width b determined in accordance with Section B2.1 with f_1 substituted for f and with k determined as follows:

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

$\psi = f_2/f_1$

f_1, f_2 = Stresses shown in Figure B2.3–1 calculated on the basis of effective section. f_1 is compression (+) and f_2 can be either tension (–) or compression. In case f_1 and f_2 are both compression, $f_1 \geq f_2$

(b) *Deflection Determination*

The effective widths in computing deflections at a given load shall be determined in accordance with Section B2.3a except that f_{d1} and f_{d2} are substituted for f_1 and f_2, where f_{d1}, f_{d2} = Computed stresses f_1 and f_2 as shown in Figure B2.3–1. Calculations are based on the effective section at the load for which deflections are determined.

B3 Effective Widths of Unstiffened Elements

B3.1 Uniformly Compressed Unstiffened Elements

(a) *Load Capacity Determination*

Effective widths, b, of unstiffened compression elements with uniform compression shall be determined in accordance with Section B2.1a with the exception that k shall be taken as 0.43 and w as defined in Figure B3.1–1.

(b) *Deflection Determination*

The effective widths used in computing deflections shall be determined in accordance with Procedure I of Section B2.1b except that f_d is substituted for f and k = 0.43.

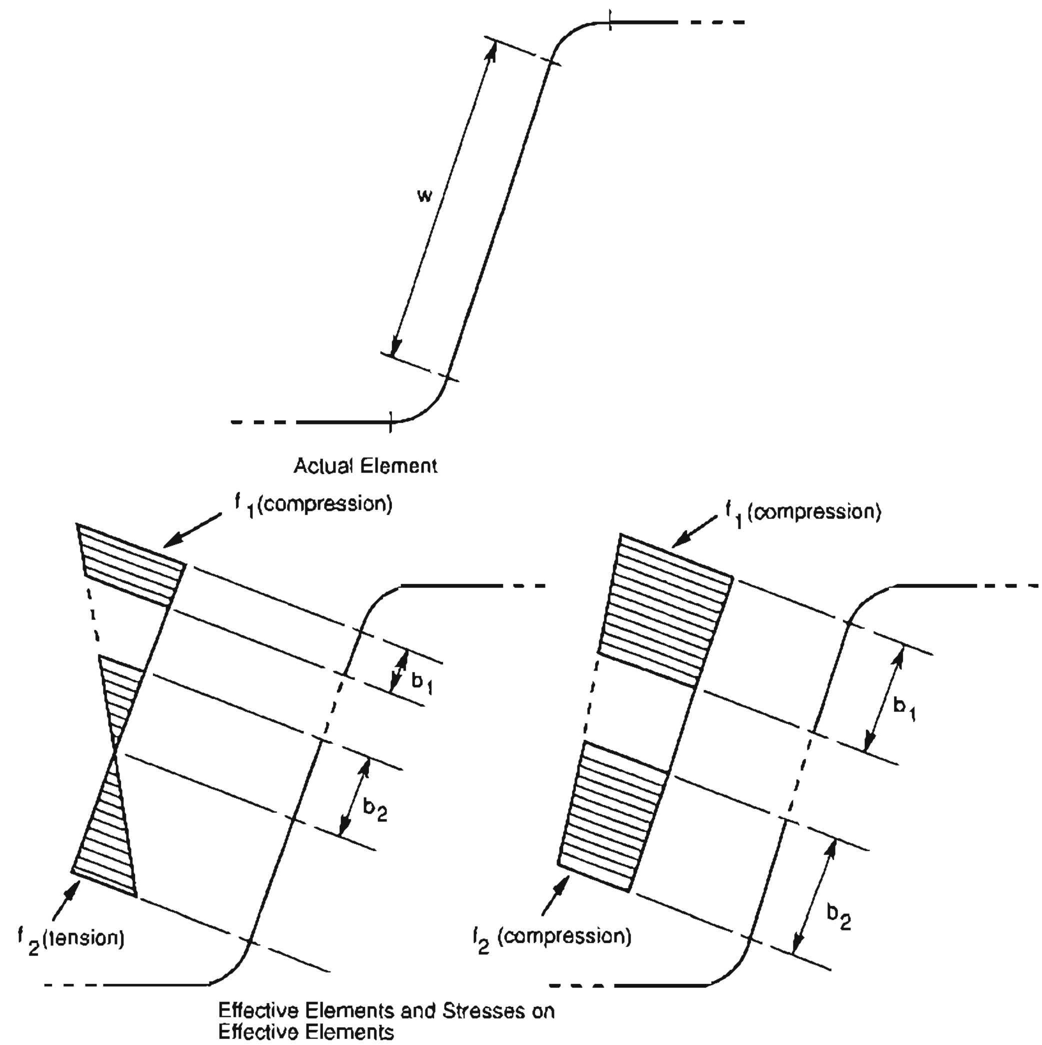


Figure B2.3–1 Stiffened Elements with Stress Gradient and Webs

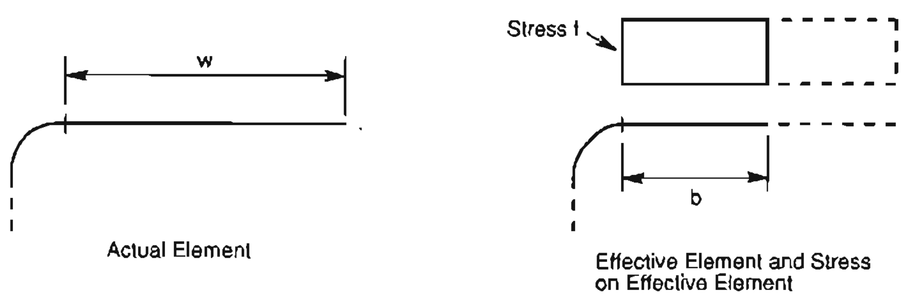


Figure B3.1–1 Unstiffened Element with Uniform Compression

B3.2 Unstiffened Elements and Edge Stiffeners with Stress Gradient

(a) *Load Capacity Determination*
Effective widths, b, of unstiffened compression elements and edge stiffeners with stress gradient shall be determined in accordance with Section B2.1a with $f = f_3$ as in Figure B4–2 in the element and $k = 0.43$.

(b) *Deflection Determination*
Effective widths, b, of unstiffened compression elements and edge stiffeners with

stress gradient shall be determined in accordance with Procedure I Section B2.1b except that f_{d3} is substituted for f and k = 0.43.

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

The following notation is used in this section.

S	= $1.28\sqrt{E/f}$	(*Eq.* B4–1)
k	= Buckling coefficient	
b_o	= Dimension defined in Figure B4–1	
d, w, D	= Dimensions defined in Figure B4–2	
d_s	= Reduced effective width of the stiffener as specified in this section. d_s, calculated according to Section B4.2, is to be used in computing the overall effective section properties (see Figure B4–2)	
d'_s	= Effective width of the stiffener calculated according to Section B3.1 (see Figure B4–2)	
C_1, C_2	= Coefficients defined in Figure B4–2	
A_s	= Reduced area of the stiffener as specified in this section. A_s is to be used in computing the overall effective section properties. The centroid of the stiffener is to be considered located at the centroid of the full area of the stiffener, and the moment of inertia of the stiffener about its own centroidal axis shall be that of the full section of the stiffener.	
I_a	= Adequate moment of inertia of stiffener, so that each component element will behave as a stiffened element.	
I_s, A'_s	= Moment of inertia of the full stiffener about its own centroidal axis parallel to the element to be stiffened and the effective area of the stiffener, respectively. For edge stiffeners the round corner between the stiffener and the element to be stiffened shall not be considered as a part of the stiffener.	

For the stiffener shown in Figure B4–2:

I_s	= $(d^3 t \sin^2\theta)/12$	(*Eq.* B4–2)
A'_s	= $d'_s t$	(*Eq.* B4–3)

B4.1 Uniformly Compressed Elements with an Intermediate Stiffener

(a) *Load Capacity Determination*

Case I: $b_o/t \leq S$ (*Eq.* B4.1–1)

$I_a = 0$ (no intermediate stiffener needed) (*Eq.* B4.1–2)

$b = w$ (*Eq.* B4.1–3)

$A_s = A'_s$ (*Eq.* B4.1–4)

Case II: $S < b_o/t < 3S$ (*Eq.* B4.1–5)

$I_a/t^4 = [50(b_o/t)/S] - 50$ (*Eq.* B4.1–6)

b and A_s shall be calculated according to Section B2.1a where

$k = 3(I_s/I_a)^{1/2} + 1 \leq 4$ (*Eq.* B4.1–7)

$A_s = A'_s(I_s/I_a) \leq A'_s$ (*Eq.* B4.1–8)

Case III: $b_o/t \geq 3S$

$I_a/t^4 = [128(b_o/t)/S] - 285$ (*Eq.* B4.1–9)

b and A_s are calculated according to Section B2.1a where

$k = 3(I_s/I_a)^{1/3} + 1 \leq 4$ (*Eq.* B4.1–10)

$A_s = A'_s (I_s/I_a) \leq A'_s$ (*Eq.* B4.1–11)

(b) *Deflection Determination*

Effective widths shall be determined as in Section B4.1a except that f_d is substituted for f.

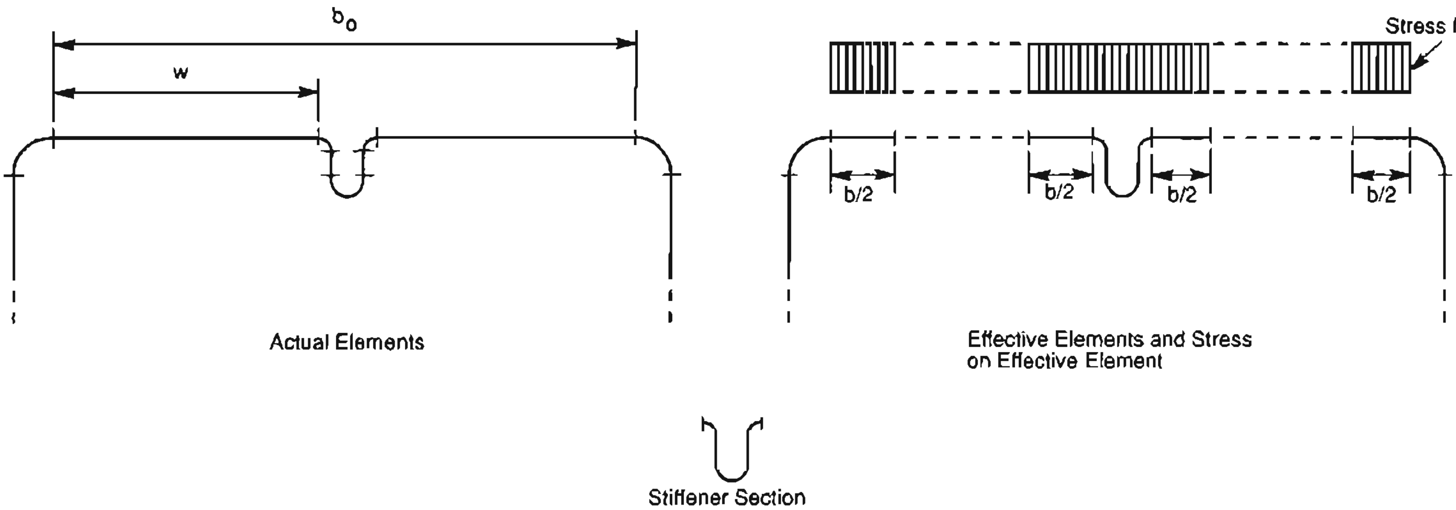


Figure B4–1 Elements with Intermediate Stiffener

B4.2 Uniformly Compressed Elements with an Edge Stiffener

(a) *Load Capacity Determination*

Case I: $w/t \leq S/3$ (*Eq.* B4.2–1)

$I_a = 0$ (no edge stiffener needed) (*Eq.* B4.2–2)

$b = w$ (*Eq.* B4.2–3)

$d_s = d'_s$ for simple lip stiffener (*Eq.* B4.2–4)

$A_s = A'_s$ for other stiffener shapes (*Eq.* B4.2–5)

Case II: $S/3 < w/t < S$

$I_a/t^4 = 399\{[(w/t)/S] - 0.33\}^3$ (*Eq.* B4.2–6)

$n = 1/2$

$C_2 = I_s/I_a \leq 1$ (*Eq.* B4.2–7)

$C_1 = 2 - C_2$ (*Eq.* B4.2–8)

b shall be calculated according to Section B2.1 where

$k = [4.82 - 5(D/w)](I_s/I_a)^n + 0.43 \leq 5.25 - 5(D/w)$ (*Eq.* B4.2–9)

for $0.8 \geq D/w > 0.25$

$k = 3.57(I_s/I_a)^n + 0.43 \leq 4.0$ (*Eq.* B4.2–10)

for $(D/w) \leq 0.25$

$d_s = d'_s (I_s/I_a) \leq d'_s$ (*Eq.* B4.2–11)

for simple lip stiffener

$A_s = A'_s (I_s/I_a) \leq A'_s$ (*Eq.* B4.2–12)

for other stiffener shape

Case III: $w/t \geq S$

$I_a/t^4 = [115 (w/t)/S] + 5$ (*Eq.* B4.2–13)

C_1, C_2, b, k, d_s, A_s are calculated per Case II with $n = 1/3$.

(b) *Deflection Determination*

Effective widths shall be determined as in Section B4.2a except that f_d is substituted for f.

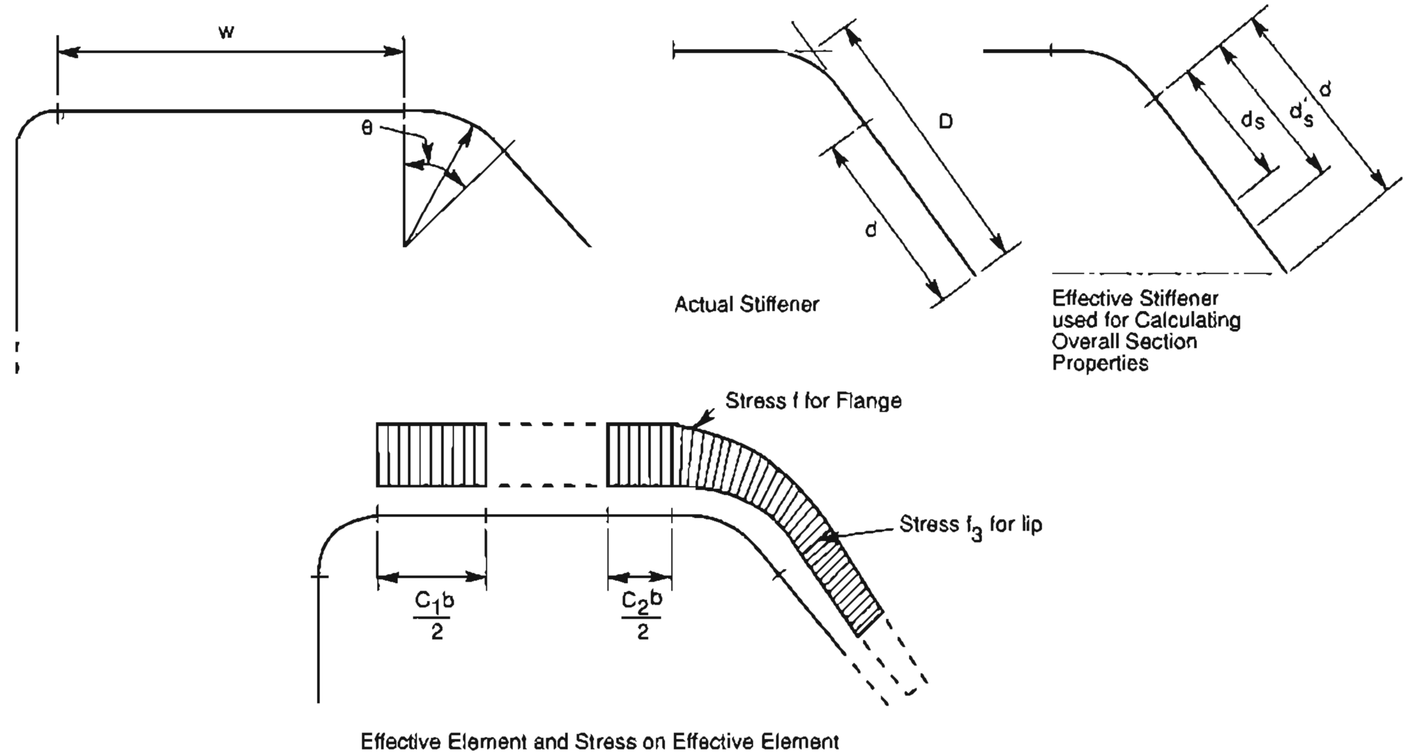


Figure B4–2 Elements with Edge Stiffener

B5 Effective Widths of Edge Stiffened Elements with Intermediate Stiffeners or Stiffened Elements with More Than One Intermediate Stiffener

For the determination of the effective width, the intermediate stiffener of an edge stiffened element or the stiffeners of a stiffened element with more than one stiffener shall be disregarded unless each intermediate stiffener has the minimum I_s as follows:

$$I_{min} = \left[3.66\sqrt{\left((w/t)^2 - (0.136E)/F_y\right)}\right]t^4 \quad (Eq.\ B5\text{–}1)$$

but not less than $18.4\ t^4$

where

w/t = Width–thickness ratio of the larger stiffened sub–element

I_s = Moment of inertia of the full stiffener about its own centroid axis parallel to the element to be stiffened

(a) If the spacing of intermediate stiffeners between two webs is such that for the sub–element between stiffeners $b < w$ as determined in Section B2.1, only two intermediate stiffeners (those nearest each web) shall be considered effective.

(b) If the spacing of intermediate stiffeners between a web and an edge stiffener is such that for the sub–element between stiffeners $b < w$ as determined in Section B2.1, only one intermediate stiffener, that nearest the web, shall be considered effective.

(c) If intermediate stiffeners are spaced so closely that for the elements between stiffeners $b = w$ as determined in Section B2.1, all the stiffeners may be considered effective. In computing the flat–width to thickness ratio of the entire multiple–stiffened element, such element shall be considered as replaced by an "equivalent element" without intermediate stiffeners whose width, b_o, is the full width between webs or from web to edge stiffener, and whose equivalent thickness, t_s, is determined as fol-

lows:

$$t_s = \sqrt[3]{12 I_{sf} / b_o} \qquad (Eq.\ B5\text{–}2)$$

where

I_{sf} = Moment of inertia of the full area of the multiple–stiffened element, including the intermediate stiffeners, about its own centroidal axis. The moment of inertia of the entire section shall be calculated assuming the "equivalent element" to be located at the centroidal axis of the multiple stiffened element, including the intermediate stiffener. The actual extreme fiber distance shall be used in computing the section modulus.

(d) If $w/t > 60$, the effective width, b_e, of the sub–element or element shall be determined from the following formula:

$$\frac{b_e}{t} = \frac{b}{t} - 0.10\left[\frac{w}{t} - 60\right] \qquad (Eq.\ B5\text{–}3)$$

where:

w/t = flat–width ratio of sub–element or element

b = effective design width determined in accordance with the provisions of Section B2.1, in.

b_e = effective design width of sub–element or element to be used in design computations, in.

For computing the effective structural properties of a member having compression sub–elements or element subject to the above reduction in effective width, the area of stiffeners (edge stiffener or intermediate stiffeners) shall be considered reduced to an effective area as follows:

For $60 < w/t < 90$:

$$A_{ef} = \alpha A_{st} \qquad (Eq.\ B5\text{–}4)$$

where

$$\alpha = (3 - 2b_e / w) - \frac{1}{30}\left[1 - \frac{b_e}{w}\right]\left[\frac{w}{t}\right] \qquad (Eq.\ B5\text{–}5)$$

For $w/t \geq 90$:

$$A_{ef} = (b_e/w)\, A_{st} \qquad (Eq.\ B5\text{–}6)$$

In the above expressions, A_{ef} and A_{st} refer only to the area of the stiffener section, exclusive of any portion of adjacent elements.

The centroid of the stiffener is to be considered located at the centroid of the full area of the stiffener, and the moment of inertia of the stiffener about its own centroidal axis shall be that of the full section of the stiffener.

B6 Stiffeners

B6.1 Transverse Stiffeners

Transverse stiffeners attached to beam webs at points of concentrated loads or reactions, shall be designed as compression members. Concentrated loads or reactions shall be applied directly into the stiffeners, or each stiffener shall be fitted accurately to the flat portion of the flange to provide direct load bearing into the end of the stiffener. Means for shear transfer between the stiffener and the web shall be provided according to Chapter E. The concentrated loads or reactions shall not exceed the smaller of the allowable loads, P_a, given by (a) and (b) as follows:

(a) $P_a = P_n/\Omega_{st}$ (*Eq.* B6.1–1)

where

$P_n = F_{wy}A_c$ (*Eq.*B6.1–2)

$\Omega_{st} = 2.00$

$A_c = 18t^2 + A_s$, for transverse stiffeners at interior support and under concentrated load (*Eq.* B6.1–3)

$A_c = 10t^2 + A_s$, for transverse stiffeners at end support (*Eq.* B6.1–4)

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

(b) $P_a = P_n/\Omega_c$ (*Eq.* B6.1–5)

where

P_n =Nominal axial load evaluated according to Section C4(a) with A_e replaced by A_b

Ω_c =Factor of safety for axial compression evaluated according to Section C4(a)

$A_b = b_1t + A_s$, for transverse stiffeners at interior support and under concentrated load (*Eq.* B6.1–6)

$A_b = b_2t + A_s$, for transverse stiffeners at end support (*Eq.* B6.1–7)

A_s =Cross sectional area of transverse stiffeners

$b_1 = 25t\ [0.0024(L_{st}/t) + 0.72] \leq 25t$ (*Eq.* B6.1–8)

$b_2 = 12t\ [0.0044(L_{st}/t) + 0.83] \leq 12t$ (*Eq.* B6.1–9)

L_{st} =Length of transverse stiffener

t =Base thickness of beam web

The w/t_s ratio for the stiffened and unstiffened elements of cold–formed steel transverse stiffeners shall not exceed $1.28\sqrt{(E/F_{ys})}$ and $0.37\sqrt{(E/F_{ys})}$ respectively, where F_{ys} is the yield stress, F_y, and t_s the thickness of the stiffener steel.

B6.2 Shear Stiffeners

Where shear stiffeners are required, the spacing shall be such that the web shear force shall not exceed the allowable shear force, V_a, permitted by Section C3.2, and the ratio a/h shall not exceed $[260/(h/t)]^2$ nor 3.0.

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

$$I_{smin} = 5ht^3[h/a - 0.7(a/h)] \geq (h/50)^4 \qquad (Eq.\ B6.2\text{–}1)$$

The gross area of shear stiffeners shall be not less than

$$A_{st} = \frac{1 - C_v}{2}\left[\frac{a}{h} - \frac{(a/h)^2}{(a/h) + \sqrt{1 + (a/h)^2}}\right]YDht \qquad (Eq.\ B6.2\text{–}2)$$

where

$$C_v = \frac{45{,}000k_v}{F_y(h/t)^2} \quad \text{when } C_v \leq 0.8 \qquad (Eq.\ B6.2\text{–}3)$$

$$C_v = \frac{190}{h/t}\left(\sqrt{\frac{k_v}{F_y}}\right) \quad \text{when } C_v > 0.8 \qquad (Eq.\ B6.2\text{–}4)$$

$$k_v = 4.00 + \frac{5.34}{(a/h)^2} \quad \text{when } a/h \leq 1.0 \qquad (Eq.\ B6.2\text{–}5)$$

$$k_v = 5.34 + \frac{4.00}{(a/h)^2} \quad \text{when } a/h > 1.0 \qquad (Eq.\ B6.2\text{–}6)$$

a = Distance between transverse stiffeners

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

D = 1.0 for stiffeners furnished in pairs

D = 1.8 for single–angle stiffeners

D = 2.4 for single–plate stiffeners

t and h are as defined in Section B1.2

B6.3 Non–Conforming Stiffeners

The allowable load carrying capacity of members with transverse stiffeners that do not meet the requirements of Sections B6.1 or B6.2, such as stamped or rolled–in transverse stiffeners shall be determined by tests in accordance with Chapter F of this Specification.

C. MEMBERS

C1 Properties of Sections

Properties of sections (cross-sectional area, moment of inertia, section modulus, radius of gyration, etc.) shall be determined in accordance with conventional methods of structural design. Properties shall be based on the full cross section of the members (or net sections where the use of net section is applicable) except where the use of a reduced cross section, or effective design width, is required.

C2 Tension Members

For axially loaded tension members, the applied tensile force shall not exceed T_a determined as follows:

$$T_a = T_n/\Omega_t \qquad (Eq.\ C2\text{-}1)$$

where

T_n = Strength of member when loaded in tension

$= A_nF_y \qquad (Eq.\ C2\text{-}2)$

Ω_t = Factor of safety for tension

= 1.67

A_n = Net area of the cross section

F_y = Design yield stress as determined in Section A5.2.1

C3 Flexural Members

C3.1 Strength for Bending Only

In flexural members, the applied moment uncoupled from axial load, shear, and local concentrated forces or reactions shall not exceed the allowable M_a calculated as follows:

$$M_a = M_n/\Omega_f \qquad (Eq.\ C3.1\text{-}1)$$

where

M_n = Smaller of the nominal moment strengths calculated according to Sections C3.1.1, C3.1.2, and C3.1.3

Ω_f = Factor of safety for bending

= 1.67

C3.1.1 Nominal Section Strength

Section strength shall be calculated either on the basis of initiation of yielding in the effective section (Procedure I) or on the basis of the inelastic reserve capacity (Procedure II) as applicable.

(a) Procedure I – Based on Initiation of Yielding
Effective yield moment based on section strength, M_n, shall be determined as follows:

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

where

F_y = Design yield stress as determined in Section A5.2.1

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

(b) Procedure II – Based on Inelastic Reserve Capacity
The inelastic flexural reserve capacity may be used when the following conditions are met:

(1) The member is not subject to twisting or to lateral, torsional, or torsional–flexural buckling.
(2) The effect of cold forming is not included in determining the yield point F_y.
(3) The ratio of the depth of the compressed portion of the web to its thickness does not exceed λ_1.
(4) The shear force does not exceed $0.35F_y$ times the web area, $h \times t$.
(5) The angle between any web and the vertical does not exceed 30 degrees.

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

where

e_y = Yield strain = F_y/E

E = Modulus of elasticity

C_y = Compression strain factor determined as follows:

(a) Stiffened compression elements without intermediate stiffeners

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

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

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

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

$C_y = 1$

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

$C_y = 1$

When applicable, effective design widths defined in Section B3.1 shall be used in calculating section properties. M_n shall be calculated considering equilibrium of stresses, assuming an ideally elastic–plastic stress–strain curve which is the same in tension as in compression, assuming small deformation and assuming that plane sections remain plane during bending. Combined bending and web crippling shall be checked by provisions of Section C3.5.

C3.1.2 Lateral Buckling Strength

|| For the laterally unbraced segments of singly–, doubly–, and point–symmetric sections* subject to lateral buckling, M_n shall be determined as follows:

$$M_n = S_c \frac{M_c}{S_f} \qquad (Eq.\ C3.1.2\text{–}1)$$

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

where

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

S_c = Elastic section modulus of the effective section calculated at a stress M_c / S_f in the extreme compression fiber

M_c = Critical moment calculated according to (a) or (b) below:

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

For $M_e > 0.5M_y$

$$M_c = M_y\left(1 - \frac{M_y}{4M_e}\right) \quad (Eq.\ C3.1.2\text{–}2)$$

For $M_e \leq 0.5M_y$

$$M_c = M_e \quad (Eq.\ C3.1.2\text{–}3)$$

where

M_y = Moment causing initial yield at the extreme compression fiber of the full section

= $S_f F_y$ (*Eq.* C3.1.2–4)

M_e = Elastic critical moment computed by the following equations:

= $C_b r_o A\sqrt{\sigma_{ey}\sigma_t}$ for bending about the symmetry axis. For singly–symmetric sections, x–axis is the axis of symmetry oriented such that the shear center has a negative x–coordinate. For point–symmetric sections, use 0.5 M_e. (*Eq.* C3.1.2–5)

Alternatively, M_e can be calculated using the formula for doubly–symmetric I–sections or point–symmetric sections given in (b) below

= $C_s A\sigma_{ex}\left[j + C_s\sqrt{j^2 + r_o^2(\sigma_t / \sigma_{ex})}\right] / C_{TF}$ for bending about the centroidal axis perpendicular to the symmetry axis for singly –symmetric sections only (*Eq.* C3.1.2–6)

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

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

$$\sigma_{ex} = \frac{\pi^2 E}{(K_x L_x / r_x)^2} \quad (Eq.\ C3.1.2\text{–}7)$$

$$\sigma_{ey} = \frac{\pi^2 E}{(K_y L_y / r_y)^2} \quad (Eq.\ C3.1.2\text{–}8)$$

$$\sigma_t = \frac{1}{Ar_o^2}\left[GJ + \frac{\pi^2 EC_w}{(K_t L_t)^2}\right] \quad (Eq.\ C3.1.2\text{–}9)$$

A = Full cross–sectional area

C_b = Bending coefficient which can conservatively be taken as unity, or calculated from

C_b = $1.75 + 1.05[(M_1/M_2)] + 0.3\,[(M_1/M_2)]^2 \leq 2.3$

where

M_1 is the smaller and M_2 the larger bending moment at the ends of the unbraced length, taken about the strong axis of the mem-

ber, and where M_1/M_2, the ratio of end moments, is positive when M_1 and M_2 have the same sign (reverse curvature bending) and negative when they are of opposite sign (single curvature bending). When the bending moment at any point within an unbraced length is larger than that at both ends of this length, and for members subject to combined axial load and bending moment (Section C5), C_b shall be taken as unity.

C_{TF} $= 0.6 - 0.4\,(M_1/M_2)$

where

M_1 is the smaller and M_2 the larger bending moment at the ends of the unbraced length, and where M_1/M_2, the ratio of end moments, is positive when M_1 and M_2 have the same sign (reverse curvature bending) and negative when they are of opposite sign (single curvature bending). When the bending moment at any point within an unbraced length is larger than that at both ends of this length, and for members subject to combined axial load and bending moment (Section C5), C_{TF} shall be taken as unity.

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

$$= \sqrt{r_x^2 + r_y^2 + x_o^2} \qquad (Eq.\ C3.1.2\text{–}10)$$

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

E = Modulus of elasticity

G = Shear modulus

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

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

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

J = St. Venant torsion constant of the cross section

C_w = Torsional warping constant of the cross section

$$j = \frac{1}{2I_y}\left[\int_A x^3 dA + \int_A xy^2 dA\right] - x_o \qquad (Eq.\ C3.1.2\text{–}11)$$

(b) For I– or Z–sections bent about the centroidal axis perpendicular to the web (x–axis):

In lieu of (a), the following equations may be used to evaluate M_c:

For $M_e > 2.78M_y$

$$M_c = M_y \qquad (Eq. C3.1.2\text{–}12)$$

For $2.78M_y > M_e > 0.56M_y$

$$M_c = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_e}\right) \qquad (Eq.\ C3.1.2\text{–}13)$$

For $M_e \le 0.56M_y$

$$M_c = M_e \qquad (Eq. C3.1.2\text{–}14)$$

where

M_e = Elastic critical moment determined either as defined in (a) above or as follows:

$$= \pi^2 E C_b \frac{d I_{yc}}{L^2} \quad \text{for doubly–symmetric I–sections} \qquad (Eq.\ C3.1.2\text{–}15)$$

$$= \frac{\pi^2 E C_b d I_{yc}}{2L^2} \quad \text{for point–symmetric Z–sections} \qquad (Eq.\ C3.1.2\text{–}16)$$

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

Other terms are defined in (a).

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

This section does not apply to a continuous beam for the region between inflection points adjacent to a support, or to a cantilever beam.

The nominal moment strength of a Channel or Z–section loaded in a plane parallel to the web, with the tension flange attached to deck or sheathing and with the compression flange laterally unbraced shall be determined as follows:

$$M_n = R S_e F_y \qquad (Eq.\ C3.1.3\text{–}1)$$

where

R = 0.40 for simple span C sections
= 0.50 for simple span Z sections
= 0.60 for continuous span C sections
= 0.70 for continuous span Z sections

S_e and F_y are defined in Section C3.1.1

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

- Member depth less than 11.5 inches
- The flanges are edge stiffened compression elements
- $60 \leq$ depth/thickness ≤ 170
- $2.8 \leq$ depth/flange width ≤ 4.5
- $16 \leq$ flat width/thickness of flange ≤ 43
- For continuous span systems, the lap length at each interior support in each direction (distance from center of support to end of lap) shall not be less than:
  1.5d for zee sections
  3.0d for channel sections
- Member span length no greater than 33 feet
- For continuous span systems, the longest member span shall not be more than 20% greater than the shortest span
- Both flanges are prevented from moving laterally at the supports
- Roof or wall panels shall be steel sheets, minimum of 0.019 in. coated thickness, having a minimum rib depth of 1 in., spaced a maximum of 12 in. on centers and attached in a manner to effectively inhibit relative movement between the panel and purlin flange

- Insulation shall be glass fiber blanket 0 to 6 inches thick compressed between the member and panel in a manner consistent with the fastener being used
- Fastener type: minimum No. 12 self–drilling or self–tapping sheet metal screws or 3/16 – in. rivets, washers 1/2 in. diameter
- Fasteners shall not be standoff type screws
- Fasteners shall be spaced not greater than 12 in. on centers and placed near the center of the beam flange

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

C3.2 Strength for Shear Only

The shear force at any section shall not exceed the allowable shear, V_a, calculated as follows:

(a) For $h/t \le 1.38\sqrt{Ek_v/F_y}$

$$V_a = 0.38t^2\sqrt{k_vF_yE} \le 0.4F_yht \qquad (Eq.\ C3.2\text{–}1)$$

(b) For $h/t > 1.38\sqrt{Ek_v/F_y}$

$$V_a = 0.53Ek_vt^3/h \qquad (Eq.\ C3.2\text{–}2)$$

where

t = Web thickness

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

k_v = Shear buckling coefficient determined as follows:

1. For unreinforced webs, $k_v = 5.34$
2. For beam webs with transverse stiffeners satisfying the requirements of Section B6

when $a/h \le 1.0$

$$k_v = 4.00 + \frac{5.34}{(a/h)^2} \qquad (Eq.\ C3.2\text{–}3)$$

when $a/h > 1.0$

$$k_v = 5.34 + \frac{4.00}{(a/h)^2} \qquad (Eq.\ C3.2\text{–}4)$$

where

a = the shear panel length for unreinforced web element

= distance between transverse stiffeners for web elements.

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

C3.3 Strength for Combined Bending and Shear

For beams with unreinforced webs, the moment, M, and shear, V, shall satisfy the following interaction equation:

$$(M/M_{axo})^2 + (V/V_a)^2 \le 1.0$$

For beams with transverse web stiffeners, the moment, M, and shear, V, shall not exceed M_a and V_a, respectively. When $M/M_{axo} > 0.5$ and $V/V_a > 0.7$, then M and V shall satisfy the following interaction equation:

$$0.6\,(M/M_{axo}) + (V/V_a) \le 1.3$$

In the above:

M_a = Allowable moment when bending alone exists

M_{axo} = Allowable moment about the centroidal axes determined in accordance with Section C3.1 excluding the provisions of Section C3.1.2,

V_a = Allowable shear force when shear alone exists

C3.4 Web Crippling Strength

These provisions are applicable to webs of flexural members subject to concentrated loads or reactions, or the components thereof, acting perpendicular to the longitudinal axis of the member, acting in the plane of the web under consideration, and causing compressive stresses in the web.

To avoid crippling of unreinforced flat webs of flexural members having a flat width ratio, h/t, equal to or less than 200, concentrated loads and reactions shall not exceed the values of P_a given in Table C3.4–1. Webs of flexural members for which h/t is greater than 200 shall be provided with adequate means of transmitting concentrated loads and/or reactions directly into the webs.

The formulas in Table C3.4–1 apply to beams when $R/t \le 6$ and to deck when $R/t \le 7$, $N/t \le 210$ and $N/h \le 3.5$.

P_a represents the concentrated load or reaction for one solid web connecting top and bottom flanges. For two or more webs, P_a shall be computed for each individual web and the results added to obtain the allowable load or reaction for the multiple web.

For built–up I–sections, or similar sections, the distance between the web connector and beam flange shall be kept as small as practical.

TABLE C3.4–1

P_a

		Shapes Having Single Webs		Shapes Having Multiple Webs(1)
		Stiffened Flanges	Unstiffened Flanges	Stiffened and Un–stiffened Flanges
Opposing Loads Spaced > 1.5h(2)	End Reaction(3)	*Eq*. C3.4–1	*Eq*. C3.4–2	*Eq*. C3.4–3
	Interior Reaction(4)	*Eq*. C3.4–4	*Eq*. C3.4–4	*Eq*. C3.4–5
Opposing Loads Spaced ≤ 1.5h(5)	End Reaction(3)	*Eq*. C3.4–6	*Eq*. C3.4–6	*Eq*. C3.4–7
	Interior Reaction(4)	*Eq*. C3.4–8	*Eq*. C3.4–8	*Eq*. C3.4–9

Footnotes and Equation References to Table C3.4–1:

(1) I–sections made of two channels 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 channel).

(2) At locations of one concentrated load or reaction acting either on the top or bottom flange, when the clear distance between the bearing edges of this and adjacent opposite concentrated loads or reactions is greater than 1.5h.

(3) For end reactions of beams or concentrated loads on the end of cantilevers when the distance from the edge of the bearing to the end of the beam is less than 1.5h.

(4) For reactions and concentrated loads when the distance from the edge of bearing to the end of the beam is equal to or greater than 1.5h.

(5) At locations of two opposite concentrated loads or of a concentrated load and an opposite reaction acting simultaneously on the top and bottom flanges, when the clear distance between their adjacent bearing edges is equal to or less than 1.5h.

Equations for Table C3.4–1:

$t^2 kC_3C_4C_\theta[179 - 0.33(h/t)]\,[1 + 0.01(N/t)]$ (*Eq.* C3.4–1)

$t^2 kC_3C_4C_\theta[117 - 0.15(h/t)]\,[1 + 0.01(N/t)]$ (*Eq.* C3.4–2)

When N/t > 60, the factor $[1 + 0.01(N/t)]$ may be increased to $[0.71 + 0.015(N/t)]$

$t^2F_yC_6\left(5.0 + 0.63\sqrt{N/t}\right)$ (*Eq.* C3.4–3)

$t^2 kC_1C_2C_\theta[291 - 0.40(h/t)]\,[1 + 0.007(N/t)]$ (*Eq.* C3.4–4)

When N/t > 60, the factor $[1 + 0.007(N/t)]$ may be increased to $[0.75 + 0.011(N/t)$

$t_2F_yC_5(0.88+0.12m)\left(7.50 + 1.63\sqrt{N/t}\right)$ (*Eq.* C3.4–5)

$t^2 kC_3C_4C_\theta[132 - 0.31(h/t)]\,[1 + 0.01(N/t)]$ (*Eq.* C3.4–6)

$t^2F_yC_8(0.64+0.31m)\left(5.0 + 0.63\sqrt{N/t}\right)$ (*Eq.* C3.4–7)

$t^2 kC_1C_2C_\theta[417 - 1.22(h/t)]\,[1 + 0.0013(N/t)]$ (*Eq.* C3.4–8)

$t^2F_yC_7(0.82+0.15m)\left(7.50 + 1.63\sqrt{N/t}\right)$ (*Eq.* C3.4–9)

In the above–referenced formulas:

P_a = Allowable concentrated load or reaction per web, kips

$C_1 = (1.22 - 0.22k)$ (*Eq.* C3.4–10)

$C_2 = (1.06 - 0.06R/t) \le 1.0$ (*Eq.* C3.4–11)

$C_3 = (1.33 - 0.33k)$ (*Eq.* C3.4–12)

$C_4 = 0.50 < (1.15 - 0.15R/t) \le 1.0$ (*Eq.* C3.4–13)

$C_5 = (1.49 - 0.53k) \ge 0.6$ (*Eq.* C3.4–14)

$C_6 = 1 + \left(\frac{h/t}{750}\right)$ when $h/t \le 150$ (*Eq.* C3.4–15)

$= 1.20$, when $h/t > 150$ (*Eq.* C3.4–16)

$C_7 = 1/k$, when $h/t \le 66.5$ (*Eq.* C3.4–17)

$= \left[1.10 - \frac{h/t}{665}\right]\frac{1}{k}$, when $h/t > 66.5$ (*Eq.* C3.4–18)

$C_8 = \left[0.98 - \frac{h/t}{865}\right]\frac{1}{k}$ (*Eq.* C3.4–19)

$C_\theta = 0.7 + 0.3\,(\theta/90)^2$ (*Eq.* C3.4–20)

|| F_y = Design yield stress of the web, see Section A5.2.1

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

$k = F_y/33$ (*Eq.* C3.4–21)

$m = t/0.075$ (*Eq.* C3.4–22)

t = Web thickness, inches

N = Actual length of bearing, inches. For the case of two equal and opposite concentrated loads distributed over unequal bearing lengths, the smaller value of N shall be taken

R = Inside bend radius

θ = Angle between the plane of the web and the plane of the bearing surface $\geq 45°$, but not more than 90°

C3.5 Combined Bending and Web Crippling Strength

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

$$1.2\,(P/P_a) + (M/M_{axo}) < 1.5 \qquad (Eq.\ C3.5\text{–}1)$$

Exception: At the interior supports of continuous spans, the above formula 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 inches.

For shapes having multiple unreinforced webs such as I–sections made of two channels 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 channel);

$$1.1\,(P/P_a) + (M/M_{axo}) < 1.5 \qquad (Eq.\ C3.5\text{–}2)$$

Exception: When $h/t \leq 2.33/\sqrt{(F_y / E)}$ and $\lambda \leq 0.673$, the allowable concentrated load or reaction may be determined by Section C3.4.

In the above formulas:

P = Concentrated load or reaction in the presence of bending moment
P_a = Allowable concentrated load or reaction in the absence of bending moment determined in accordance with Section C3.4
M = Applied bending moment at, or immediately adjacent to, the point of application of the concentrated load or reaction
M_{axo} = Allowable moment about the centroidal axes determined in accordance with Section C3.1 excluding the provisions of Section C3.1.2,
w = Flat width of the beam flange which contacts the bearing plate
t = Thickness of the web or flange
λ = Slenderness factor given by Section B2.1

C4 Concentrically Loaded Compression Members

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

(a) The axial load shall not exceed P_a calculated as follows:

$$P_a = P_n/\Omega_c \qquad (Eq.\ C4\text{–}1)$$

where

$$P_n = A_eF_n \qquad (Eq.\ C4\text{–}2)$$

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

F_n is determined as follows:

For $F_e > F_y/2$ $\quad F_n = F_y(1 - F_y/4F_e)$ (*Eq.* C4–3)

For $F_e \leq F_y/2$ $\quad F_n = F_e$ (*Eq.* C4–4)

F_e is the least of the elastic flexural, torsional and torsional–flexural buckling stress determined according to Sections C4.1 through C4.3.

Ω_c = Factor of safety for axial compression

= 1.92, except when F_e is determined according to Section C4.1 for fully effective sections having wall thicknesses greater than or equal to 0.09 inches and $F_e > F_y/2$. In this case,

$$\Omega_c = \frac{5}{3} + \frac{3}{8}R - \frac{1}{8}R^3$$

where

$$R = \sqrt{(F_y / 2F_e)}$$

(b) For C– and Z–shapes, and single–angle sections with unstiffened flanges, P_n shall be taken as the smaller of P_n calculated above and P_n calculated as follows:

$$P_n = \frac{A\pi^2 E}{25.7(w/t)^2} \quad (Eq.\ C4\text{–}5)$$

where

A = Area of the full, unreduced cross section

w = Flat width of the unstiffened element

t = Thickness of the unstiffened element

(c) Angle sections shall be designed for the applied axial load, P, acting simultaneously with a moment equal to PL/1000 applied about the minor principal axis causing compression in the tips of the angle legs.

(d) The slenderness ratio, KL/r, of all compression members preferably should not exceed 200, except that during construction only, KL/r preferably should not exceed 300.

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

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

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

where

E = Modulus of elasticity

K = Effective length factor*

L = Unbraced length of member

r = Radius of gyration of the full, unreduced cross section

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

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

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

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

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

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

where σ_t and σ_{ex} are as defined in C3.1.2(b)

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

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

C4.3 Nonsymmetric Sections

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

C5 Combined Axial Load and Bending

The axial force and bending moments shall satisfy the following interaction equations:

$$\frac{P}{P_a} + \frac{C_{mx}M_x}{M_{ax}\alpha_x} + \frac{C_{my}M_y}{M_{ay}\alpha_y} \le 1.0 \qquad (Eq.\ C5\text{–}1)$$

$$\frac{P}{P_{ao}} + \frac{M_x}{M_{ax}} + \frac{M_y}{M_{ay}} \le 1.0 \qquad (Eq.\ C5\text{–}2)$$

When $P/P_a \le 0.15$, the following formula may be used in lieu of the above two formulas:

$$\frac{P}{P_a} + \frac{M_x}{M_{ax}} + \frac{M_y}{M_{ay}} \le 1.0 \qquad (Eq.\ C5\text{–}3)$$

where

P = Applied axial load

M_x and M_y = Applied moments with respect to the centroidal axes of the effective section determined for the axial load alone. For angle sections, M_y shall be taken either as the applied moment or the applied moment plus PL/1000, whichever results in a lower value of P_a.

P_a = Allowable axial load determined in accordance with Section C4

P_{ao} = Allowable axial load determined in accordance with Section C4, with $F_n = F_y$

M_{ax} and M_{ay} = Allowable moments about the centroidal axes determined in accordance with Section C3

$1/\alpha_x$, $1/\alpha_y$ = Magnification factors

$= 1/[1 - (\Omega_c P/P_{cr})]$ (Eq. C5–4)

Ω_c = Factor of safety used in determining P_a

I_b = Moment of inertia of the full, unreduced cross section about the axis of bending

P_{cr} $= \dfrac{\pi^2 E I_b}{(K_b L_b)^2}$ (*Eq.* C5-5)

L_b = Actual unbraced length in the plane of bending

K_b = Effective length factor in the plane of bending

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

1. For compression members in frames subject to joint translation (sidesway)

   $C_m = 0.85$

2. For restrained compression members in frames braced against joint translation and not subject to transverse loading between their supports in the plane of bending

   $C_m = 0.6 - 0.4\,(M_1/M_2)$ (*Eq.* C5-6)

   where

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

3. For compression members in frames braced against joint translation in the plane of loading and subject to transverse loading between their supports, the value of C_m may be determined by rational analysis. However, in lieu of such analysis, the following values may be used:

   (a) for members whose ends are restrained, $C_m = 0.85$,

   (b) for members whose ends are unrestrained, $C_m = 1.0$.

C6 Cylindrical Tubular Members

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

C6.1 Bending

For flexural members, the actual moment uncoupled from axial load, shear, and local concentrated forces or reactions shall not exceed M_a calculated as follows:

$M_a = M_n/\Omega_f$ (*Eq.* C6.1-1)

where

M_n = Nominal moment

Ω_f = Factor of safety for bending

= 1.67

For $D/t \le 0.070\,E/F_y$

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

For $0.070\,E/Fy < D/t \le 0.319\,E/F_y$

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

For $0.319\,E/F_y < D/t \le 0.441\,E/F_y$

$M_n = [0.328E/(D/t)]S_f$ (*Eq.* C6.1-4)

where

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

C6.2 Compression

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

The axial load shall not exceed P_a calculated as follows:

$$P_a = P_n/\Omega_c \qquad (Eq.\ C6.2\text{–}1)$$

where

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

For F_e greater than $F_y/2$

F_n = Flexural buckling stress

$$= F_y [1 - F_y/4F_e] \qquad (Eq.\ C6.2\text{–}3)$$

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

Ω_c = Factor of safety for axial compression

$$= \frac{5}{3} + \frac{3}{8}R - \frac{1}{8}R^3 \qquad (Eq.\ C6.2\text{–}4)$$

$$R = \sqrt{F_y / 2F_e} \qquad (Eq.\ C6.2\text{–}5)$$

$$A_e = [1 - (1 - R^2)(1 - A_o/A)]A \qquad (Eq.\ C6.2\text{–}6)$$

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

A = Area of the unreduced cross section

For $F_e \le F_y/2$

$F_n = F_e$

Ω_c = Factor of safety for axial compression

= 1.92

$A_e = A$

C6.3 Combined Bending and Compression

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

D. STRUCTURAL ASSEMBLIES

D1 Built–Up Sections

D1.1 I – Sections Composed of Two Channels

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

(a) For compression members:

$$s_{max} = \frac{Lr_{cy}}{2r_I} \qquad (Eq.\ D1.1\text{–}1)$$

where

L = Unbraced length of compression member

r_I = Radius of gyration of the I–section about the axis perpendicular to the direction in which buckling would occur for the given conditions of end support and intermediate bracing

r_{cy} = Radius of gyration of one channel about its centroidal axis parallel to the web

(b) For flexural members:

$$s_{max} = L/6 \qquad (Eq.\ D1.1\text{–}2)$$

In no case shall the spacing exceed the value

$$s_{max} = \frac{2gT_s}{mq} \qquad (Eq.\ D1.1\text{–}3)$$

where

L = Span of beam

T_s = Strength of connection in tension

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

q = Intensity of load on the beam (For methods of determination, see below).

m = Distance from the shear center of one channel to the mid–plane of its web. For simple channels without stiffening lips at the outer edges,

$$m = \frac{w_f^2}{2w_f + d/3} \qquad (Eq.\ D1.1\text{–}4)$$

For channels with stiffening lips at the outer edges,

$$m = \frac{w_f dt}{4I_x}\left[w_f d + 2D\left(d - \frac{4D^2}{3d}\right)\right] \qquad (Eq.\ D1.1\text{–}5)$$

w_f = Projection of flanges from the inside face of the web (For channels with flanges of unequal width, w_f shall be taken as the width of the wider flange)

d = Depth of channel or beam

D = Overall depth of lip

I_x = Moment of inertia of one channel about its centroidal axis normal to the web.

The intensity of load, q, is obtained by dividing the magnitude of concentrated loads or reactions by the length of bearing. For beams designed for a uniformly distributed load, q shall be taken equal to three times the intensity of the uniformly distributed design load. If the length of bearing of a concentrated load or reaction is smaller than the weld spacing, s, the required strength of the welds or connections closest to the load or reaction is

$$T_s = Pm/2g \qquad (Eq.\ D1.1\text{–}6)$$

The required maximum spacing of connections, s_{max}, depends upon the intensity of the load directly at the connection. Therefore, if uniform spacing of connections is

used over the whole length of the beam, it shall be determined at the point of maximum local load intensity. In cases where this procedure would result in uneconomically close spacing, either one of the following methods may be adopted: (a) the connection spacing may be varied along the beam according to the variation of the load intensity; or (b) reinforcing cover plates may be welded to the flanges at points where concentrated loads occur. The shear strength of the connections joining these plates to the flanges shall then be used for T_s, and g shall be taken as the depth of the beam.

D1.2 Spacing of Connections in Compression Elements

The spacing, s, in the line of stress, of welds, rivets, or bolts connecting a cover plate, sheet, or a non–integral stiffener in compression to another element shall not exceed

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

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

(c) three times the flat width, w, of the narrowest unstiffened compression element tributary to the connections, but need not be less than $1.11t\sqrt{(E/F_y)}$ if $w/t < 0.50\sqrt{(E/F_y)}$, or $1.33t\sqrt{(E/F_y)}$ if $w/t \geq 0.50\sqrt{(E/F_y)}$, unless closer spacing is required by (a) or (b) above.

In the case of intermittent fillet welds parallel to the direction of stress, the spacing shall be taken as the clear distance between welds, plus one–half inch. In all other cases, the spacing shall be taken as the center–to–center distance between connections.

Exception: The requirements of this Section do not apply to cover sheets which act only as sheathing material and are not considered as load–carrying elements.

D2 Mixed Systems

The design of members in mixed systems using cold–formed steel components in conjunction with other materials shall conform to this Specification and the applicable specification of the other material.

D3 Lateral Bracing

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

D3.1 Symmetrical Beams and Columns

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

D3.2 Channel–Section and Z–Section Beams

The following provisions for bracing to restrain twisting of channels and Z–sections used as beams loaded in the plane of the web, apply only when (a) the top flange is connected to deck or sheathing material in such a manner as to effectively restrain lateral deflection of the connected flange*, or (b) neither flange is so connected. When both flanges are so connected, no further bracing is required.

* Where the Specification does not provide an explicit method for design, further information should be obtained from the Commentary.

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

For channels and Z-sections designed according to Section C3.1.1, and having deck or sheathing fastened directly to the top flanges in such a manner shown to effectively inhibit relative movement between the deck or sheathing and the purlin flange, provisions shall be made to restrain the flanges so that the maximum top flange lateral displacements with respect to the purlin reaction points do not exceed the span length divided by 360. If the top flanges of all purlins face in the same direction, anchorage of the restraint system must be capable of satisfying the requirements of Sections D3.2.1(a) and D3.2.1(b). If the top flanges of adjacent lines of purlins face in opposite directions, the provisions of Section D3.2.1(a) and D3.2.1(b) do not apply.

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

For bracing arrangements other than those covered in Sections D3.2.1(a) and D3.2.1(b), tests in accordance with Chapter F shall be performed so that the type and/or spacing of braces selected are such that the test strength of the braced Z-section assembly is equal to or greater than 5/3 times its flexural design strength, instead of that required by Chapter F.

(a) Channel Sections

For roof systems using channel sections for purlins with all compression flanges facing in the same direction, a restraint system capable of resisting 0.05W, in addition to other loading, shall be provided where W is the load supported by all purlin lines being restrained. Where more than one brace is used at a purlin line, the restraint force 0.05W shall be divided equally between all braces.

(b) Z-Sections

For roof systems having a diaphragm stiffness of at least 2,000 lb/in., having four to twenty Z-purlin lines with all top flanges facing in the direction of the upward roof slope, and with restraint braces at the purlin supports, midspan or one-third points, each brace shall be designed to resist a force determined as follows:

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

$$P_L = 0.5\left[\frac{0.220b^{1.50}}{n_p^{0.72}d^{0.90}t^{0.60}} - \sin\theta\right]W \qquad (Eq.\ D3.2.1\text{-}1)$$

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

$$P_L = 0.5\left[\frac{0.474b^{1.22}}{n_p^{0.57}d^{0.89}t^{0.33}} - \sin\theta\right]W \qquad (Eq.\ D3.2.1\text{-}2)$$

(3) Single-Span System with Midspan Restraint:

$$P_L = \left[\frac{0.224b^{1.32}}{n_p^{0.65}d^{0.83}t^{0.50}} - \sin\theta\right]W \qquad (Eq.\ D3.2.1\text{-}3)$$

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

with

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

$$P_L = C_{tr}\left[\frac{0.053b^{1.88}L^{0.13}}{n_p^{0.95}d^{1.07}t^{0.94}} - \sin\theta\right]W \qquad (Eq.\ D3.2.1\text{–}4)$$

C_{tr} = 0.87 for braces at the first interior supports
C_{tr} = 0.81 for all other braces

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

$$P_L = C_{th}\left[\frac{0.181b^{1.15}L^{0.25}}{n_p^{0.54}d^{1.11}t^{0.29}} - \sin\theta\right]W \qquad (Eq.\ D3.2.1\text{–}5)$$

with

C_{th} = 0.57 for outer braces in exterior spans
C_{th} = 0.48 for all other braces

(6) Multiple–Span System with Midspan Restraints:

$$P_L = C_{ms}\left[\frac{0.116b^{1.32}L^{0.18}}{n_p^{0.70}d^{1.00}t^{0.50}} - \sin\theta\right]W \qquad (Eq.\ D3.2.1\text{–}6)$$

with

C_{ms} = 1.05 for braces in exterior spans
C_{ms} = 0.90 for all other braces

where

b = Flange width, in.
d = Depth of section, in.
t = Thickness, in.
L = Span length, in.
θ = Angle between the vertical and the plane of the web of the Z–section, degrees
n_p = Number of parallel purlin lines
W = Total load supported by the purlin lines between adjacent supports, pounds

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

For systems having less than four purlin lines, the brace force can be determined by taking 1.1 times the force found from Equations D3.2.1–1 through D3.2.1–6, with $n_p = 4$. For systems having more than twenty purlin lines, the brace force can be determined from Equations D3.2.1–1 through D3.2.1–6, with $n_p = 20$.

D3.2.2 Neither Flange Connected to Sheathing

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

(a) For uniform loads, $P_L = 1.5K'$ times the load within a distance 0.5a each side of the brace.

(b) For concentrated loads, $P_L = 1.0K'$ times each concentrated load within a distance 0.3a each side of the brace, plus $1.4K'[1-(x/a)]$ times each concentrated load located farther than 0.3a but not farther than 1.0a from the brace.

In the above formulas:
For channels and Z–sections:

x = Distance from the concentrated load to the brace

a = Distance between center line of braces

For channels:

$K' = m/d$ (*Eq.* D3.2.2-1)

where

m = Distance from the shear center to the mid-plane of the web, as specified in Section D1.1

d = Depth of channel

For Z-sections:

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

where

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

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

Braces shall be designed to avoid local crippling at the points of attachment to the member.

Braces shall be attached both to the top and bottom flanges of the sections, at the ends and at intervals not greater than one-quarter of the span length, in such a manner as to prevent tipping at the ends and lateral deflection of either flange in either direction at intermediate braces. If one-third or more of the total load on the beam is concentrated over a length of one-twelfth or less of the span of the beam, an additional brace shall be placed at or near the center of this loaded length.

Exception: 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 rotation and lateral displacement, no other braces will be required.

D3.3 Laterally Unbraced Box Beams

For closed box-type sections used as beams subject to bending about the major axis, the ratio of the laterally unsupported length to the distance between the webs of the section shall not exceed $0.086\ E/F_y$.

D4 Wall Studs and Wall Stud Assemblies

The safe load-carrying capacity of a stud may be computed on the basis of Section C (neglecting sheathing and using steel only) or on the basis that sheathing (attached to one or both sides of the stud) furnishes adequate lateral and rotational support to the stud in the plane of the wall, provided that the stud, sheathing, and attachments comply with the following requirements:

Both ends of the stud shall be braced to restrain rotation about the longitudinal stud axis and horizontal displacement perpendicular to the stud axis; however, the ends may or may not be free to rotate about both axes perpendicular to the stud axis. The sheathing shall be connected to the top and bottom members of the wall assembly to enhance the restraint provided to the stud and stabilize the overall assembly.

When sheathing is utilized for stability of the wall studs, the sheathing shall retain adequate strength and stiffness for the expected service life of the wall and additional bracing shall be provided as required for adequate structural integrity during construction and in the completed structure.

The equations given are based on solid–web steel studs and are applicable within the following limits:

Yield point, $F_y \leq$ 50 ksi
Section depth, $d \leq 6.0$ in.
Thickness, $t \leq 0.075$ in.
Overall length, $L \leq 16$ ft.
Stud spacing, B, not less than 12 in. nor greater than 24 in.
Studs with perforations shall be designed using the results of stub column tests and/or rational analysis.

D4.1 Wall Studs in Compression

For studs having identical sheathing attached to both flanges, and neglecting any rotational restraint provided by the sheathing*, the applied axial load, P, shall not exceed P_a calculated as follows:

$$P_a = A_e F_n / \Omega_c \qquad (Eq.\ D4.1\text{–}1)$$

where

A_e = Effective area determined at F_n

Ω_c = Factor of safety for axial compression, i.e., in accordance with Section C4(a) when either Sections D4.1(a) or D4.1(b) govern or 1.92 when Section D4.1(c) governs.

F_n = The lowest value determined by the following three conditions:

(a) To prevent column buckling between fasteners in the plane of the wall, F_n shall be calculated according to Section C4 with KL equal to two times the distance between fasteners.

(b) To prevent flexural and/or torsional overall column buckling, F_n shall be calculated in accordance with Section C4 with F_e taken as the smaller of the two σ_{CR} values specified for the following section types, where σ_{CR} is the theoretical elastic buckling stress under concentric loading.

(1) Singly–symmetric channels and C–Sections

$$\sigma_{CR} = \sigma_{ey} + \overline{Q}_a \qquad (Eq.\ D4.1\text{–}2)$$

$$\sigma_{CR} = \frac{1}{2\beta}\left[(\sigma_{ex} + \sigma_{tQ}) - \sqrt{(\sigma_{ex} + \sigma_{tQ})^2 - (4\beta\sigma_{ex}\sigma_{tQ})}\right] \qquad (Eq.\ D4.1\text{–}3)$$

(2) Z–Sections

$$\sigma_{CR} = \sigma_t + \overline{Q}_t \qquad (Eq.\ D4.1\text{–}4)$$

$$\sigma_{CR} = \frac{1}{2}\left\{(\sigma_{ex} + \sigma_{ey} + \overline{Q}_a) - \sqrt{\left[(\sigma_{ex} + \sigma_{ey} + \overline{Q}_a)^2 - 4\left(\sigma_{ex}\sigma_{ey} + \sigma_{ex}\overline{Q}_a - \sigma^2_{exy}\right)\right]}\right\} \qquad (Eq.\ D4.1\text{–}5)$$

(3) I–Sections (doubly–symmetric)

$$\sigma_{CR} = \sigma_{ey} + \overline{Q}_a \qquad (Eq.\ D4.1\text{–}6)$$

$$\sigma_{CR} = \sigma_{ex} \qquad (Eq.\ D4.1\text{–}7)$$

In the above formulas:

$$\sigma_{ex} = \frac{\pi^2 E}{(L / r_x)^2} \qquad (Eq.\ D4.1\text{–}8)$$

*Studs with sheathing on one flange only, or with unidentical sheathing on both flanges, or having rotational restraint that is not neglected, or having any combination of the above, shall be designed in accordance with the same basic analysis principles used in deriving the provisions of this Section.

$\sigma_{exy} = (\pi^2 E I_{xy}) / (AL^2)$ (*Eq.* D4.1–9)

$$\sigma_{cy} = \frac{\pi^2 E}{(L/r_y)^2}$$ (*Eq.* D4.1–10)

$$\sigma_t = \frac{1}{Ar_o^2}\left[GJ + \frac{\pi^2 EC_w}{(L)^2}\right]$$ (*Eq.* D4.1–11)

$\sigma_{tQ} = \sigma_t + \bar{Q}_t$ (*Eq.* D4.1–12)

$\bar{Q}$ = $\bar{q}$ B = Design shear rigidity for sheathing on both sides of wall assembly

$\bar{q}$ = Design shear rigidity for sheathing per inch of stud spacing (see Table D4)

B = Stud spacing

$\bar{Q}_a = \bar{Q} / A$ (*Eq.* D4.1–13)

A = Area of full unreduced cross section

L = Length of stud

$\bar{Q}_t = (\bar{Q}d^2)/(4Ar_o^2)$ (*Eq.* D4.1–14)

d = Depth of section

I_{xy} = Product of inertia

(c) To prevent shear failure of the sheathing, a value of F_n shall be used in the following equations so that the shear strain of the sheathing, γ, does not exceed the permissible shear strain, $\bar{\gamma}$. The shear strain, γ, shall be determined as follows:

$\gamma = (\pi/L)\,[C_1 + (E_1\, d/2)]$ (*Eq.* D4.1–15)

where

C_1 and E_1 are the absolute values of C_1 and E_1 specified below for each section type:

(1) Singly–Symmetric Channels and C–Sections

$C_1 = (F_n C_o)/(\sigma_{cy} - F_n + \bar{Q}_a)$ (*Eq.* D4.1–16)

$$E_1 = \frac{F_n\left[(\sigma_{ex} - F_n)(r_o^2 E_o - x_o D_o) - F_n x_o (D_o - x_o E_o)\right]}{(\sigma_{ex} - F_n) r_o^2 (\sigma_{tQ} - F_n) - (F_n x_o)^2}$$ (*Eq.* D4.1–17)

(2) Z–Sections

$$C_1 = \frac{F_n\left[C_o(\sigma_{ex} - F_n) - D_o \sigma_{exy}\right]}{(\sigma_{cy} - F_n + \bar{Q}_a)(\sigma_{ex} - F_n) - \sigma_{exy}^2}$$ (*Eq.* D4.1–18)

$E_1 = (F_n E_o) / (\sigma_{tQ} - F_n)$ (*Eq.*D4.1–19)

(3) I–Sections

$C_1 = (F_n C_o)/(\sigma_{cy} - F_n + \bar{Q}_a)$ (*Eq.* D4.1–20)

$E_1 = 0$

where

x_o = distance from shear center to centroid along principal x–axis, in. (absolute value)

C_o, E_o, and D_o are initial column imperfections which shall be assumed to be at least

C_o = L/350 in a direction parallel to the wall (*Eq.* D4.1–21)

D_o = L/700 in a direction perpendicular to the wall (*Eq.* D4.1–22)

E_o = L/(d × 10,000), rad., a measure of the initial twist of the stud from the initial, ideal, unbuckled shape (*Eq.*D4.1–23)

If $F_n > 0.5\, F_y$, then in the definitions for σ_{ey}, σ_{ex}, σ_{exy} and σ_{tQ}, the parameters E and G shall be replaced by E′ and G′, respectively, as defined below

$E' = 4EF_n(F_y - F_n)/F_y^2$ (*Eq.* D4.1-24)

$G' = G\,(E'/E)$ (*Eq.* D4.1-25)

Sheathing parameters $\bar{q}_o$ and $\bar{\gamma}$ may be determined from representative full-scale tests, conducted and evaluated as described by published documented methods (see Commentary), or from the small-scale-test values given in Table D4.

D4.2 Wall Studs in Bending

For studs having identical sheathing attached to both flanges, and neglecting any rotational restraint provided by the sheathing,* the allowable moments are M_{axo} and M_{ayo}

where

M_{axo} and M_{ayo} = Allowable moments about the centroidal axes determined in accordance with Section C3.1, excluding the provisions of Section C3.1.2 (lateral buckling)

D4.3 Wall Studs with Combined Axial Load and Bending

The axial load and bending moments shall satisfy the interaction equations of Section C5 with the following redefined terms:

P_a = Allowable axial load determined according to Section D4.1

M_{ax} and M_{ay} in Equations C5-1 and C5-3 shall be replaced by allowable moments, M_{axo} and M_{ayo}, respectively.

TABLE D4
Sheathing Parameters(1)

Sheathing(2)	$\bar{q}_o$(3) k/in.	$\bar{\gamma}$ in./in.
3/8 to 5/8 in. thick gypsum	2.0	0.008
Lignocellulosic board	1.0	0.009
Fiberboard (regular or impregnated)	0.6	0.007
Fiberboard (heavy impregnated)	1.2	0.010

(1) The values given are subject to the following limitations:
All values are for sheathing on both sides of the wall assembly.
All fasteners are No. 6, type S-12, self-drilling drywall screws with pan or bugle head, or equivalent, at 6- to 12-inch spacing.

(2) All sheathing is 1/2-inch thick except as noted.

(3) $\bar{q} = \bar{q}_o\,(2 - s/12)$ (*Eq.* D4.1-26)

where s = fastener spacing, in.
For other types of sheathing, $\bar{q}_o$ and $\bar{\gamma}$ may be determined conservatively from rep-

* Studs with sheathing on one flange only, or with unidentical sheathing on both flanges, or having rotational restraint that is not neglected, or having any combination of the above, shall be designed in accordance with the same basic analysis principles used in deriving the provisions of this Section.

resentative small–specimen tests as described by published documented methods (see Commentary).

D5 Floor, Roof or Wall Steel Diaphragm Construction

The in–plane structural performance of floor, roof or wall steel diaphragm construction shall be established by calculation or test. The allowable in–plane load carrying capacity S_a shall be:

$$S_a = S_n/\Omega_s \qquad (Eq.\ D5\text{–}1)$$

where

S_n = the nominal diaphragm shear strength
Ω_s = the factor of safety for diaphragm shear as specified below:
= 3.0 for welded connections or for combinations of welds and mechanical connections subjected to earthquake loads, or subjected to load combinations which include earthquake loads.
= 2.5 for mechanical connections subjected to earthquake loads, or subjected to load combinations which include earthquake loads. For backed–up fasteners (bolts, rivets, spreading back fasteners or the like) Ω_s may be taken as 2.3.
= 2.75 for welded connections or for combinations of welds and mechanical connections subjected to wind loads or subjected to load combinations which include wind loads.
= 2.35 for mechanical connections subjected to wind loads or subjected to load combinations which include wind loads. For backed–up fasteners (bolts, rivets, spreading back fasteners or the like) Ω_s may be taken as 2.1.

Although Section A4.4 of the Specification allows forces to be multiplied by 0.75 when the loading consists of wind or seismic loads acting either alone or in combination with other loads, this decrease in loads is not permitted for diaphragms.

For load combinations not involving wind or seismic loads:

$\Omega_s = \dfrac{2.75}{0.75}$ for welded connections or for combinations of welds and mechanical connections.

$\Omega_s = \dfrac{2.35}{0.75}$ for mechanical connections.

E. CONNECTIONS AND JOINTS

E1 General Provisions

Connections shall be designed to transmit the maximum load in the connected member. Proper regard shall be given to eccentricity.

E2 Welded Connections

Arc welds on steel where each connected part is over 0.18 inch in thickness shall be made in accordance with AISC Specification (Section A6).

Except as modified herein, arc welds on steel where at least one of the connected parts is 0.18 inch or less in thickness shall be made in accordance with the AWS D–1.3 (Section A6) and its Commentary. Welders and welding procedures shall be qualified as specified in AWS D1.3. These provisions are intended to cover the welding positions as shown in Table E2.

Resistance welds shall be made in conformance with the procedures given in AWS C1.1–66, "Recommended Practices for Resistance Welding" or AWS C1.3–70, "Recommended Practice for Resistance Welding Coated Low Carbon Steels."

TABLE E2

Connection	Welding Position					
	Square Groove Butt Weld	Arc Spot Weld	Arc Seam Weld	Fillet Weld, Lap or T	Flare–Bevel Groove	Flare–V Groove Weld
Sheet to Sheet	F H V OH	— — — —	F H — —	F H V OH	F H V OH	F H V OH
Sheet to Supporting Member	— — — —	F — — —	F — — —	F H V OH	F H V OH	— — — —

(F = flat, H = horizontal, V = vertical, OH = overhead)

The load on each weld shall not exceed P_a, calculated as follows:

$$P_a = P_n/\Omega_w \qquad (Eq.\ E2\text{–}1)$$

where

Ω_w = Factor of safety for arc welded connections
= 2.50

P_n = Nominal strength of welds determined according to Sections E2.1 through E2.5.

E2.1 Groove Welds in Butt Joints

The maximum load for a groove weld in a butt joint, welded from one or both sides, shall be determined on the basis of the lower strength base steel in the connection, provided that an effective throat equal to or greater than the thickness of the material is consistently obtained.

E2.2 Arc Spot Welds

Arc spot welds permitted by this Specification are for welding sheet steel to thicker supporting members in the flat position. Arc spot welds (puddle welds) shall not

be made on steel where the thinnest connected part is over 0.15 inch thick, nor through a combination of steel sheets having a total thickness over 0.15 inch.

Weld washers, Figures E2.2(A) and E2.2(B), shall be used when the thickness of the sheet is less than 0.028 inch. Weld washers shall have a thickness between 0.05 and 0.08 inch with a minimum prepunched hole of 3/8– inch diameter.

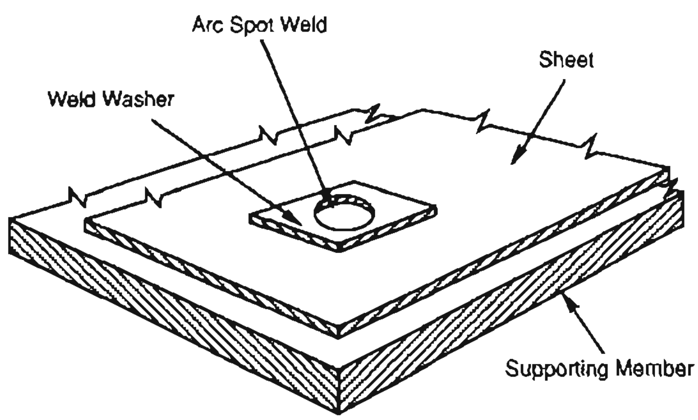


Figure E2.2A Typical Weld Washer

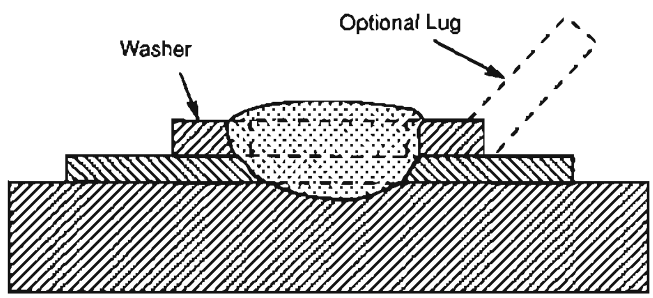


Figure E2.2B Arc Spot Weld Using Washer

Arc spot welds shall be specified by minimum effective diameter of fused area, d_e. Minimum allowable effective diameter is 3/8 inch.

The nominal shear load, P_n, on each arc spot weld between sheet or sheets and supporting member shall not exceed the smaller of either

$$P_n = 0.625\, d_e^2 F_{xx}\ ;\ \text{or} \qquad (Eq.\ E2.2\text{–}1)$$

For $(d_a/t) \le 0.815\sqrt{(E/F_u)}$:

$$P_n = 2.20\, t\, d_a F_u\ ; \qquad (Eq.\ E2.2\text{–}2)$$

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

$$P_n = 0.280\left[1+\frac{5.59t\sqrt{E}}{d_a\sqrt{F_u}}\right]td_aF_u; \qquad (Eq.\ E2.2\text{–}3)$$

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

$$P_n = 1.40\ t\ d_a\ F_u \qquad (Eq.\ E2.2\text{–}4)$$

where

d = Visible diameter of outer surface of arc spot weld

d_a = Average diameter of the arc spot weld at mid–thickness of t [where $d_a = (d - t)$ for a single sheet, and $(d - 2t)$ for multiple sheets (not more than four lapped sheets over a supporting member)}

d_e = Effective diameter of fused area

$d_e = 0.7d - 1.5t$ but $\leq 0.55d$ (*Eq.* E2.2–5)

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

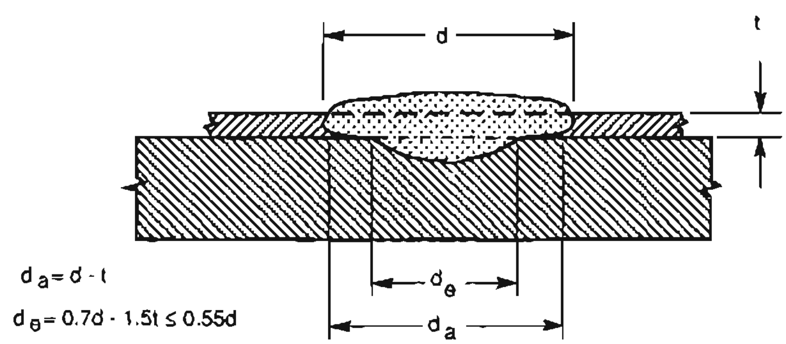
F_{xx} = Stress level designation in AWS electrode classification

F_{sy} = Yield point as specified in Section A3.1 or A3.2.

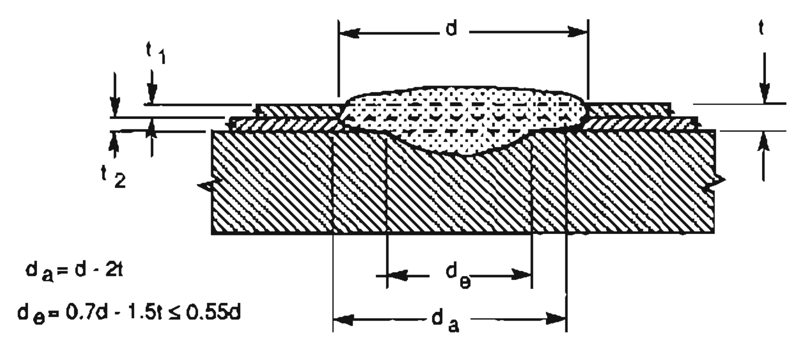
F_u = Tensile strength as specified in Section A3.1 or A3.2 or as reduced for low ductility steel.

Note: See Figures E2.2(C) and E2.2(D) for diameter definitions

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



(C) Arc Spot Weld-Single Thickness of Sheet



(D) Arc Spot Weld-Double Thickness of Sheet

Figure E2.2 C, D Arc Spot Welds

$$e_{min} = e\,\Omega_e \quad (Eq.\ E2.2\text{–}6)$$

where

$$e = \frac{P}{F_u t} \quad (Eq.\ E2.2\text{–}7)$$

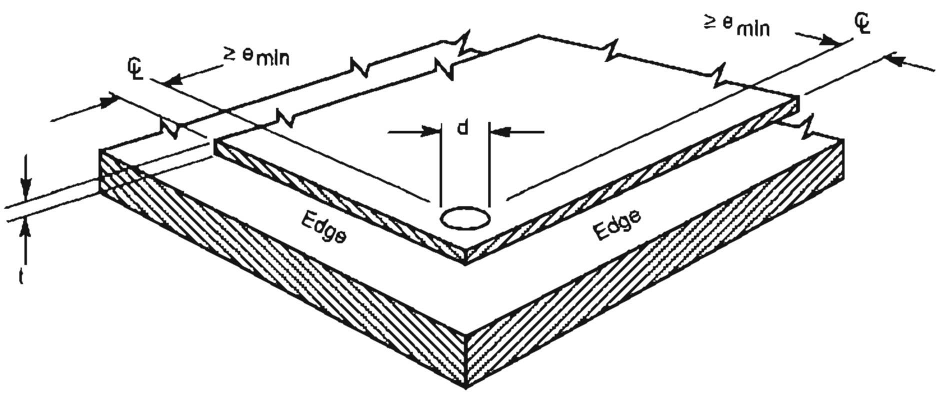
Ω_e = Factor of safety for sheet tearing
= 2.0 when $F_u/F_{sy} \geq 1.15$
= 2.22 when $F_u/F_{sy} < 1.15$

P = Force transmitted by weld

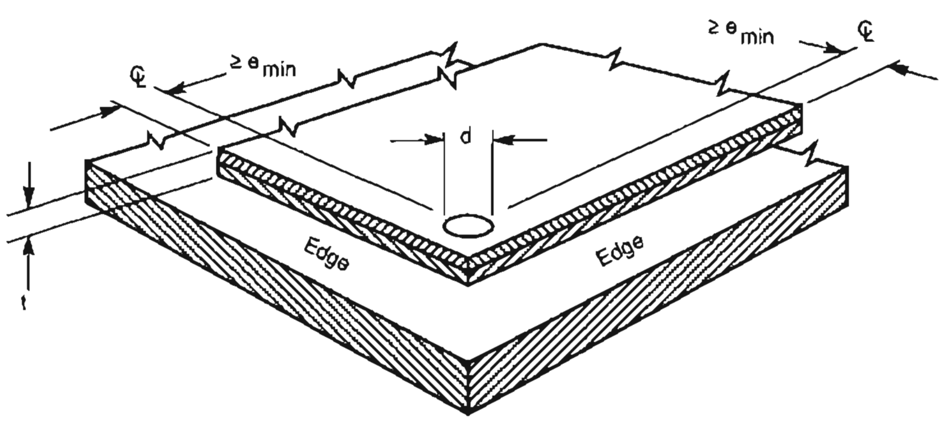
t = Thickness of thinnest connected sheet

Note: See Figures E2.2(E) and E2.2(F) for edge distances of arc welds.

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



(E) Single Sheet



(F) Double Sheet

Figure E2.2 E, F Edge Distances for Arc Spot Welds

The nominal tension load, P_n, on each arc spot weld between sheet and supporting member, shall not exceed:

$$P_n = 0.7\, t\, d_a\, F_u \qquad (Eq.\ E2.2\text{–}8)$$

The following additional limitations for use in Eq. 2.2–8 shall apply:

$e_{min} \geq d$
$F_{xx} \geq 60$ ksi
$F_u \leq 60$ ksi
$t \geq 0.028$ in.

If it can be shown by measurement that a given weld procedure will consistently give a larger effective diameter, d_e, or average diameter, d_a, as applicable, this larger diameter may be used providing the particular welding procedure used for making those welds is followed.

E2.3 Arc Seam Welds

Arc seam welds [Figure E2.3(A)] covered by this Specification apply only to the following joints:
(a) Sheet to thicker supporting member in the flat position.
(b) Sheet to sheet in the horizontal or flat position.
The shear load, P_n, on each arc seam weld shall not exceed either

$$P_n = \left[\frac{d_e^2}{4} + \frac{LD_e}{3}\right] 2.5 F_{xx};\ \text{or} \qquad (Eq.\ E2.3\text{–}1)$$

$$P_n = 2.5\, tF_u(0.25L + 0.96\, d_a) \qquad (Eq.\ E2.3\text{–}2)$$

where
d = width of arc seam weld
L = Length of seam weld not including the circular ends
(For computation purposes, L shall not exceed $3d$.)
d_a = Average width of seam weld
where
d_a = $(d - t)$ for a single sheet, and (*Eq.* E2.3–3)
$(d - 2t)$ for a double sheet (*Eq.* E2.3–4)
d_e = Effective width of arc seam weld at fused surfaces
d_e = $0.7d - 1.5t$ (*Eq.* E2.3–5)
and F_u and F_{xx} are defined in Section E2.2. The minimum edge distance shall be as determined for the arc spot weld, Section E2.2 [see Figure E2.3(B)].

If it can be shown by measurement that a given weld procedure will consistently give a larger effective width, d_e or d_a as applicable, this value may be used providing the particular welding procedure used for making the welds that are measured is followed.

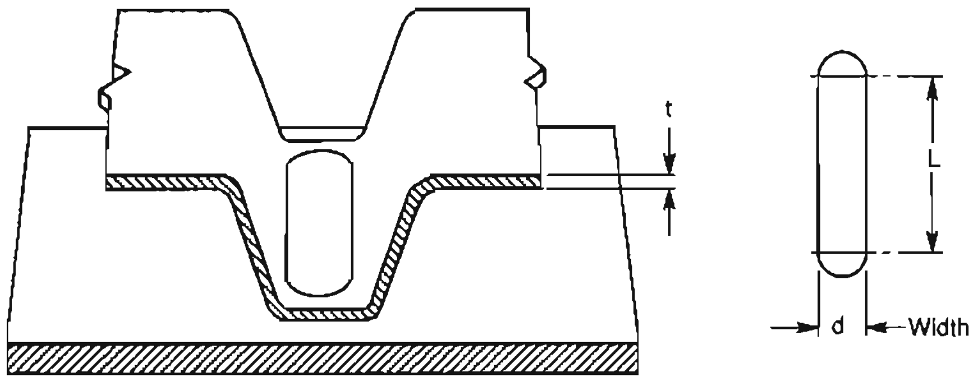


Figure E2.3A Arc Seam Welds – Sheet to Supporting Member in Flat Position

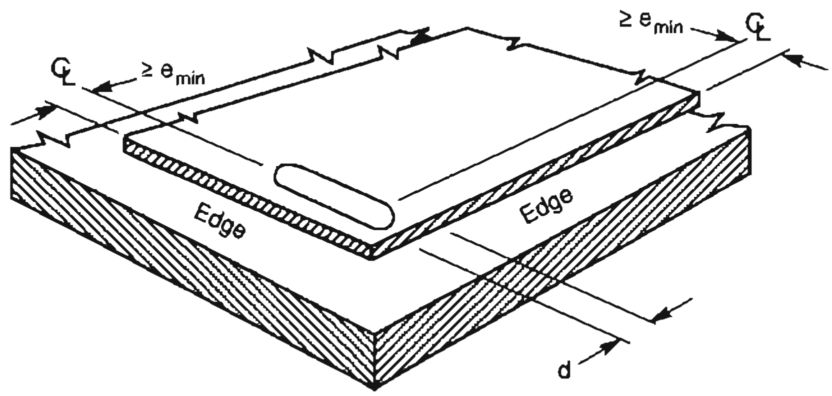


Figure E2.3B Edge Distances for Arc Seam Welds

E2.4 Fillet Welds

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

(a) Sheet to sheet, or
(b) Sheet to thicker steel member.

The shear load, P_n, on a fillet weld in lap and T–joints shall not exceed the following:

For longitudinal loading:

For $L/t < 25$:

$$P_n = \left(1 - \frac{0.01L}{t}\right) tLF_u \qquad (Eq.\ E2.4\text{–}1)$$

For $L/t \geq 25$:

$$P_n = 0.75\, tLF_u \qquad (Eq.\ E2.4\text{–}2)$$

For transverse loading:

$$P_n = tLF_u \qquad (Eq.\ E2.4\text{–}3)$$

where

t = Least value of t_1 or t_2, Figure E2.4

In addition, for $t > 0.150$ inch the allowable load for a fillet weld in lap and T–joints shall not exceed:

$$P_n = 0.75\, t_w LF_{xx} \qquad (Eq.\ E2.4\text{–}4)$$

where

L = Length of fillet weld

t_w = Effective throat = 0.707 w_1 or 0.707 w_2, whichever is smaller. A larger effective throat may be taken if it can be shown by measurement that a given welding procedure will consistently give a larger value providing the particular welding procedure used for making the welds that are measured is followed.

w_1 and w_2 = leg on weld (see Figure E2.4).

F_u and F_{xx} are defined in Section E2.2.

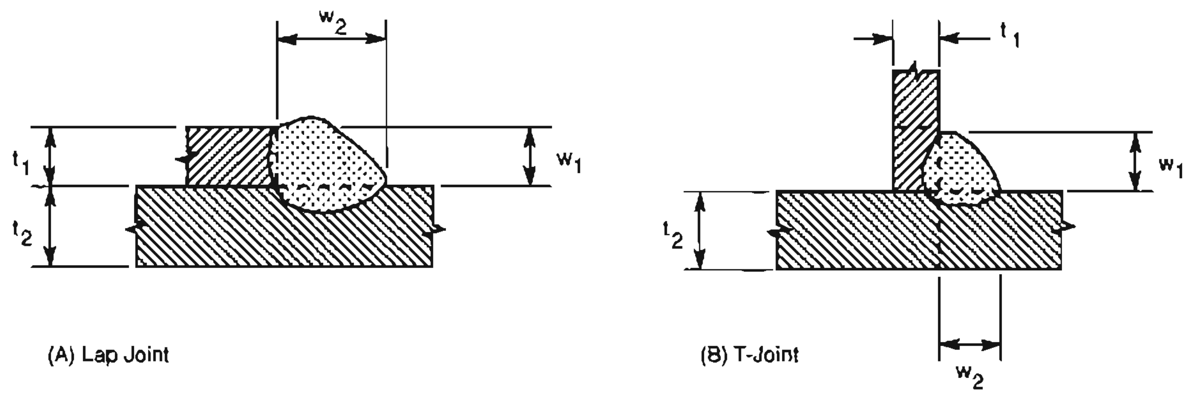


Figure E2.4 Fillet Welds

E2.5 Flare Groove Welds

Flare groove welds covered by this Specification apply to welding of joints in any position, either:

(a) Sheet to sheet for flare–V groove welds, or
(b) Sheet to sheet for flare–bevel groove welds, or
(c) Sheet to thicker steel member for flare–bevel groove welds.

The shear load, P_n, on a weld shall be governed by the thickness, t, of the sheet steel adjacent to the weld. The load shall not exceed:

For flare–bevel groove welds, transverse loading [see Figure E2.5(A)]:

$$P_n = 0.833tLF_u \qquad (Eq.\ E2.5\text{–}1)$$

For flare groove welds, longitudinal loading [see Figures E2.5(B), E2.5(C), and E2.5(D)]:

If the effective throat, t_w, is equal to or greater than t but less than 2t or if the lip height is less than weld length, L, then:

$$P_n = 0.75tLF_u \qquad (Eq.\ E2.5\text{–}2)$$

If t_w is equal to or greater than 2t and the lip height is equal to or greater than L, then:

$$P_n = 1.50tLF_u \qquad (Eq.\ E2.5\text{–}3)$$

In addition, if t > 0.15 inch, then:

$$P_n = 0.75t_wLF_{xx} \qquad (Eq.\ E2.5\text{–}4)$$

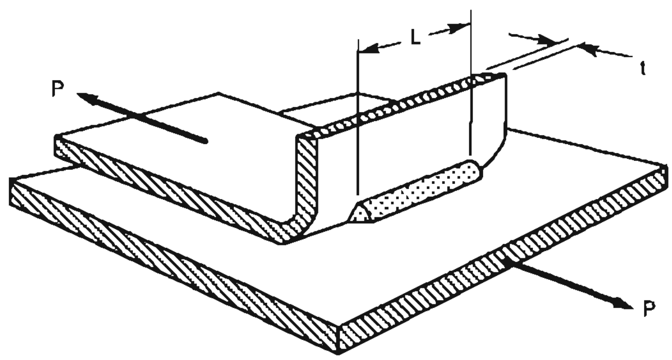
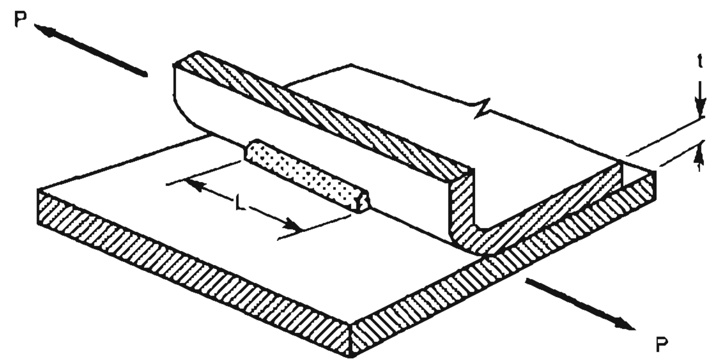
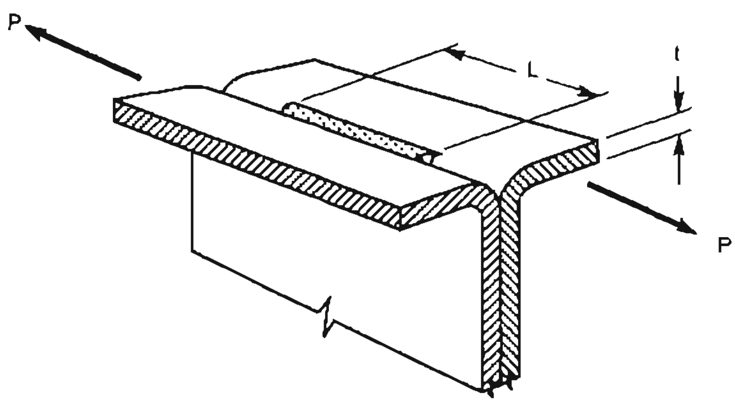


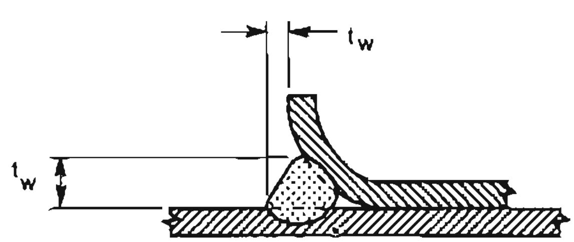
Figure E2.5A Flare–Bevel Groove Weld



(B) Flare Bevel Groove



(C) Flare V-Groove



(D) Throat

Figure E2.5 B, C, D Shear In Flare Groove Welds

E2.6 Resistance Welds

In sheets joined by spot welding the nominal shear strength per spot, P_n, is as follows:

TABLE E2.6

Thickness of Thinnest Outside Sheet, in.	Nominal Shear Strength per Spot, kips	Thickness of Thinnest Outside Sheet, in.	Nominal Shear Stength per Spot, kips
0.010	0.13	0.080	3.33
0.020	0.48	0.090	4.00
0.030	1.00	0.100	4.99
0.040	1.42	0.110	6.07
0.050	1.65	0.125	7.29
0.060	2.28	0.190	10.16
0.070	2.83	0.250	15.00

E3 Bolted Connections

The following requirements govern bolted connections of cold–formed steel structural members in which the thickness of the thinnest connected part is less than $^3/_{16}$ inch and there are no gaps between connected parts. For bolted connections in which the thinnest connected part is equal to or greater than $^3/_{16}$ inch, refer to AISC Specification (Section A6).

Bolts, nuts, and washers shall generally conform to one of the following specifications:

ASTM A194 Carbon and Alloy Steel Nuts for Bolts for High–Pressure and High–Temperature Service
ASTM A307(Type A), Carbon Steel Externally and Internally Threaded Standard Fasteners
ASTM A325 High Strength Bolts for Structural Steel Joints
ASTM A354 (Grade BD), Quenched and Tempered Alloy Steel Bolts, Studs,and Other Externally Threaded Fasteners (for diameter of bolt smaller than $^1/_2$ inch)
ASTM A449 Quenched and Tempered Steel Bolts and Studs (for diameter of bolt smaller than $^1/_2$ inch)
ASTM A490 Quenched and Tempered Alloy Steel Bolts for Structural Steel Joints.
ASTM A563 Carbon and Alloy Steel Nuts
ASTM F436 Hardened Steel Washers
ASTM F844 Washers, Steel, Plain (Flat), Unhardened for General Use
ASTM F959 Compressible Washer–Type Direct Tension Indicators for Use with Structural Fasteners

When other than the above are used, drawings shall indicate clearly the type and size of fasteners to be employed and the allowable force assumed in design.

Bolts shall be installed and tightened to achieve satisfactory performance of the connections involved under usual service conditions.

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

TABLE E3
Maximum Size of Bolt Holes, Inches

Nominal Bolt Diameter, d in.	Standard Hole Diameter, d in.	Oversized Hole Diameter, d in.	Short-Slotted Hole Dimensions in.	Long-Slotted Hole Dimensions in.
$< 1/2$	$d + 1/32$	$d + 1/16$	$(d + 1/32)$ by $(d + 1/4)$	$(d + 1/32)$ by $(2\ 1/2\ d)$
$\geq 1/2$	$d + 1/16$	$d + 1/8$	$(d + 1/16)$ by $(d + 1/4)$	$(d + 1/16)$ by $(2\ 1/2\ d)$

Standard holes shall be used in bolted connections, except that oversized and slotted holes may be used as approved by the designer. The length of slotted holes shall be normal to the direction of the shear load. Washers or backup plates shall be installed over oversized or short-slotted holes in an outer ply unless suitable performance is demonstrated by load tests in accordance with Section F.

E3.1 Spacing and Edge Distance

The distance, e, measured in the line of force from the center of a standard hole to the nearest edge of an adjacent hole or to the end of the connected part toward which the force is directed shall not be less than the value of e_{min} determined as follows:

$$e_{min} = e\,\Omega_e \qquad (Eq.\ E3.1-1)$$

where

$$e = \frac{P}{F_u t} \qquad (Eq.\ E3.1-2)$$

(a) When $F_u/F_{sy} \geq 1.15$:

Ω_e = Factor of safety for sheet tearing
= 2.0

(b) When $F_u/F_{sy} < 1.15$:

Ω_e = Factor of safety for sheet tearing
= 2.22

where

P = Force transmitted by bolt
t = Thickness of thinnest connected part
F_u = Tensile strength of the connected part as specified in Sections A3.1 or A3.2
F_{sy} = Yield point of the connected part as specified in Sections A3.1 or A3.2

In addition, the minimum distance between centers of bolt holes shall provide sufficient clearance for bolt heads, nuts, washers and the wrench but shall not be less than 3 times the nominal bolt diameter, d. Also, the distance from the center of any standard hole to the end or other boundary of the connecting member shall not be less than $1\ 1/2$ d.

For oversized and slotted holes, the distance between edges of two adjacent holes and the distance measured from the edge of the hole to the end or other boundary of the connecting member in the line of stress shall not be less than the value of $[e_{min} - (d_h/2)]$, in which e_{min} is the required distance computed from the applicable equation given above, and d_h is the diameter of a standard hole defined in Table E3. In no case shall the clear

distance between edges of two adjacent holes be less than 2d and the distance between the edge of the hole and the end of the member be less than d.

E3.2 Tension in Connected Part

The tension force on the net section of a bolted connection shall not exceed T_a from Section C2 or P_a calculated as follows:

$P_a = P_n/\Omega_t$ (*Eq.* E3.2–1)

where

$P_n = A_nF_t$

A_n = Net section area

F_t and Ω_t are determined as follows:

(a) When $t \geq {}^3/_{16}$ in.:
See AISC Specification (Reference 3 of Section A6)

(b) When $t < {}^3/_{16}$ inch and washers are provided under both the bolt head and the nut

$F_t = (1.0 - 0.9r + 3rd/s)\, F_u \leq F_u$ (*Eq.* E3.2–2)

Ω_t = Factor of safety for tension on the net section
= 2.0 for double shear
= 2.22 for single shear

(c) When $t < {}^3/_{16}$ inch and either washers are not provided under the bolt head and nut, or only one washer is provided under either the bolt head or nut

$F_t = (1.0 - r + 2.5rd/s)\, F_u \leq F_u$ (*Eq.* E3.2–3)

Ω_t = Factor of safety for tension on the net section
= 2.22

where

r = Force transmitted by the bolt or bolts at the section considered, divided by the tension force in the member at that section. If r is less than 0.2, it may be taken equal to zero.

s = Spacing of bolts perpendicular to line of stress.
In the case of a single bolt, s = Width of sheet

F_t = Nominal tension stress limit on net section

F_u = Tensile strength of the connected part as specified in Sections A3.1 or A3.2

d and t are defined in Section E3.1

E3.3 Bearing

The bearing force shall not exceed P_a calculated as follows:

$P_a = P_n/\Omega_b$ (*Eq.* E3.3–1)

where

$P_n = F_pdt$ (*Eq.* E3.3–2)

Ω_b = Safety factor for bearing
= 2.22

F_p = Nominal bearing stress as given in Tables E3.3–1 and E3.3–2.

For conditions not shown, forces shall be determined on the basis of test data using a factor of safety of 2.22

TABLE E3.3–1
Nominal Bearing Stress for Bolted Connections with Washers under Both Bolt Head and Nut

Thickness of connected part in.	Type of joint	F_u/F_{sy} ratio of connected part	Nominal bearing stress, F_p
≥0.024 but < $^3/_{16}$	Inside sheet of double shear connection	≥ 1.15	3.33 F_u
		< 1.15	3.00 F_u
	Single shear and outside sheets of double shear connection	No limit	3.00 F_u
≥ $^3/_{16}$	See AISC Specification (Reference 3 of Section A6)		

TABLE E3.3–2
Nominal Bearing Stress for Bolted Connections Without Washers Under Both Bolt Head and Nut, or With Only One Washer

Thickness of connected part in.	Type of joint	F_u/F_{sy} ratio of connected part	Nominal bearing stress, Fp
≥ 0.036 but < $^3/_{16}$	Inside sheet of double shear connection	≥ 1.15	3.00 F_u
	Single shear and outside sheets of double shear connection	≥ 1.15	2.22 F_u
≥ $^3/_{16}$	See AISC Specification (Reference 3 of Section A6)		

E3.4 Shear and Tension in Bolts

The bolt force resulting from shear, tension or combination of shear and tension shall not exceed allowable bolt force, P_a, calculated as follows (The factor of safety is included in Tables E3.4–1 and E3.4–2):

$$P_a = A_b F \qquad (Eq.\ E3.4\text{–}1)$$

where

A_b = Gross cross–sectional area of bolt

F is given by F_v, F_t or F'_t in Tables E3.4–1 and E3.4–2.

TABLE E3.4–1

Description of Bolts	Allowable Shear Stress*, F_v, ksi — Threads not Excluded from Shear Plane	Allowable Shear Stress*, F_v, ksi — Threads Ex– cluded from Shear Plane	Allowable Tension Stress, F_t, ksi
A325 Bolts	21	30	44
A354 Grade B Bolts ($^1/_4$ in. $\leq$ d < $^1/_2$ in.)	24	40	49
A449 Bolts ($^1/_4$ in. $\leq$ d < $^1/_2$ in.)	18	30	40
A490 Bolts	28	40	54
A307 Bolts, Grade A ($^1/_4$ in. $\leq$ d < $^1/_2$ in.)	9		18
A307 Bolts, Grade A (d $\geq$ $^1/_2$ in.)	10		20

* Applies to bolts in holes as limited by Table E3. Washers or back–up plates shall be installed over long–slotted holes and the capacity of connections using long–slotted holes shall be determined by load tests in accordance with Section F.

The pullover strength of the connected sheet at the bolt head, nut or washer should be considered where bolt tension is involved, See Section E5.2.

When bolts are subject to a combination of shear and tension, the tension force shall not exceed the allowable force, P_a, based on F'_t, given in Table E3.4–2, where f_v, the shear stress produced by the same forces, shall not exceed the allowable value F_v given above.

TABLE E3.4–2
Allowable Tension Stress, F'_t for Bolts Subject to the Combination of Shear and Tension

Description of Bolts	Threads Not Excluded from Shear Planes	Threads Excluded from Shear Planes
A325 Bolts	$55 - 1.8f_v \leq 44$	$55 - 1.4f_v \leq 44$
A354 Grade BD Bolts	$61 - 1.8f_v \leq 49$	$61 - 1.4f_v \leq 49$
A449 Bolts	$50 - 1.8f_v \leq 40$	$50 - 1.4f_v \leq 40$
A490 Bolts	$68 - 1.8f_v \leq 54$	$68 - 1.4f_v \leq 54$
A307 Bolts, Grade A		
When $^1/_4$ in. $\leq$ d < $^1/_2$ in.	$23 - 1.8f_v \leq 18$	
When d $\geq$ $^1/_2$ in.	$26 - 1.8f_v \leq 20$	

E4 Shear Rupture

At beam–end connections, where one or more flanges are coped and failure might occur along a plane through the fasteners, the shear force shall not exceed the allowable shear force V_a, calculated as follows:

$V_a = V_n/\Omega_v$ (*Eq.* E4–1)

where

$V_n = 0.6 F_u A_{wn}$ (*Eq.* E4–2)

$A_{wn} = (d_{wc} - nd_h)t$ (*Eq.* E4–3)

d_{wc} = coped web depth
n = number of holes in the critical plane
d_h = hole diameter
F_u = Tensile strength as specified in Sections A3.1 or A3.2
t = Thickness of coped web
Ω_v = Factor of safety for shear rupture
= 2.00

E5 Connections to Other Materials

E5.1 Bearing

Proper provisions shall be made to transfer bearing forces resulting from axial loads and moments from steel components covered by the Specification to adjacent structural components made of other materials. The bearing force in the contact area shall not exceed the allowable bearing force P_a calculated as follows:

$P_a = F_p A$

where

A = Contact area
F_p = Allowable bearing stress. (The factor of safety is included in values for F_p.)

In the absence of code regulations for other materials, the following allowable stresses may be used:

F_p = 0.40 ksi on sandstone and limestone
F_p = 0.25 ksi on brick in cement mortar
$F_p = 0.35 f'_c$ on the full area of a concrete support
$F_p = 0.35 f'_c \sqrt{(A_2/A_1)} \le 0.7 f'_c$ on less than the full area of a concrete support

where

f'_c = Specified compression strength of concrete
A_1 = Bearing area
A_2 = Full cross–sectional area of concrete support

E5.2 Tension

The pull–over shear/tension forces in the steel sheet around the head of the fastener should be considered as well as the pull–out force resulting from axial loads and bending moments transmitted onto the fastener from various adjacent structural components in the assembly.

The allowable tensile strength of the fastener and the allowable imbedment strength of the adjacent structural component shall be determined by applicable product code approvals, or product specifications and/or product literature.

E5.3 Shear

Proper provisions shall be made to transfer shearing forces from steel components covered by this Specification to adjacent structural components made of other materials. The allowable shear and/or bearing forces on the steel components shall not exceed that allowed by this Specification. The allowable shear force on the fasteners and other material shall not be exceeded. Imbedment requirements are to be met. Proper provision shall also be made for shearing forces in combination with other forces.

F. TESTS FOR SPECIAL CASES

(a) Tests shall be made by an independent testing laboratory or by a testing laboratory of a manufacturer.
(b) The provisions of Chapter F do not apply to cold–formed steel diaphragms.

F1 Tests for Determining Structural Performance

Where the composition or configuration of elements, assemblies, connections, or details of cold–formed steel structural members are such that calculation of their safe load–carrying capacity or deflection cannot be made in accordance with the provisions of this Specification, their structural performance shall be established from tests and evaluated in accordance with the following procedure.

(a) Where practicable, evaluation of tests results shall be made on the basis of the mean values resulting from tests of not fewer than three identical specimens, provided the deviation of any individual test result from the mean value obtained from all tests does not exceed ±10 percent. If such deviation from the mean exceeds 10 percent, at least three more tests of the same kind shall be made. The average of the three lowest values of all tests made shall then be regarded as the result of the series of tests.

(b) The required load–carrying capacity shall be:

$$R = D\,F_D + L\,F_L \qquad (Eq.\ F1\text{–}1)$$

where D and L are the dead and live loads, respectively, D shall include the weight of the test specimen. F_D and F_L are the dead and live load factors specified below. R shall be taken as the largest applicable value determined as follows:

(1) The minimum load–carrying capacity, R, shall be calculated from the formula

$$R \geq 1.5D + 2L \qquad (Eq.\ F1\text{–}2)$$

R shall be multiplied by 1.25 for steels not listed in Section A3.1

R may be divided by $1\,{}^1/_3$ when the loading consists of wind or earthquake loads alone, or in combination with dead, live, or snow loads, but shall not be less than R calculated for the combination of dead and live loads only, without wind or earthquake loads.

(2) The load at which distortions interfere with the proper functioning of the specimen in actual use shall not be less than:

$$R \geq D + 1.5L \qquad (Eq.\ F1\text{–}3)$$

(3) The load carrying capacity when limited by connection failure shall not be less than:

$$R \geq 2.5D + 2.5L \qquad (Eq.\ F1\text{–}4)$$

(c) If the yield point of the steel from which the tested sections are formed is larger than the specified value, the test results shall be adjusted down to the specified minimum yield point of the steel which the manufacturer intends to use. The test results shall not be adjusted upward if the yield point of the test specimen is less than the minimum specified yield point. Similar adjustments shall be made on the basis of tensile strength instead of yield point where tensile strength is the critical factor.

Consideration must also be given to any variation or differences which may exist between the design thickness and the thickness of the specimens used in the tests.

F2 Tests for Confirming Structural Performance

The procedures and formulas specified in Section F1 are not applicable to confirmation tests on specimens whose capacities can be computed according to this Specification or its

specific references. A successful confirmatory test shall demonstrate a safety factor not less than that implied in the Specification for the type of behavior involved.

F3 Tests for Determining Mechanical Properties

F3.1 Full Section

Tests for determination of mechanical properties of full sections to be used in Section A5.2.2 shall be made as specified below:

(a) Tensile testing procedures shall agree with Standard Methods and Definitions for Mechanical Testing of Steel Products, ASTM A370.
Compressive yield point determinations shall be made by means of compression tests of short specimens of the section.

(b) The comprehensive yield stress shall be taken as the smaller value of either the maximum compressive strength of the sections divided by the cross section area or the stress defined by one of the following methods:

(1) For sharp yielding steel, the yield point shall be determined by the autographic diagram method or by the total strain under load method.

(2) For gradual yielding steel, the yield point shall be determined by the strain under load method or by the 0.2 percent offset method.
When the total strain under load method is used, there shall be evidence that the yield point so determined agrees within 5 percent with the yield point which would be determined by the 0.2 percent offset method

(c) Where the principal effect of the loading to which the member will be subjected in service will be to produce bending stresses, the yield point shall be determined for the flanges only. In determining such yield points, each specimen shall consist of one complete flange plus a portion of the web of such flat width ratio that the value of ρ for the specimen is unity.

(d) For acceptance and control purposes, two full section tests shall be made from each lot of not more than 50 tons nor less than 30 tons of each section, or one test from each lot of less than 30 tons of each section. For this purpose a lot may be defined as that tonnage of one section that is formed in a single production run of material from one heat.

(e) At the option of the manufacturer, either tension or compression tests may be used for routine acceptance and control purposes, provided the manufacturer demonstrates that such tests reliably indicate the yield point of the section when subjected to the kind of stress under which the member is to be used.

F3.2 Flat Elements of Formed Sections

Tests for determining mechanical properties of flat elements of formed sections and representative mechanical properties of virgin steel to be used in Section A5.2.2 shall be made in accordance with the following provisions:

The yield point of flats, F_{yf}, shall be established by means of a weighted average of the yield points of standard tensile coupons taken longitudinally from the flat portions of a representative cold-formed member. The weighted average shall be the sum of the products of the average yield point for each flat portion times its cross sectional area, divided by the total area of flats in the cross section. The exact number of such coupons will depend on the shape of the member, i.e.,on the number of flats in the cross section. At least one tensile coupon shall be taken from the middle of each flat. If the actual virgin yield point exceeds the specified minimum yield point, the yield point of the flats, F_{yf},

shall be adjusted by multiplying the test values by the ratio of the specified minimum yield point to the actual virgin yield point.

F3.3 Virgin Steel

The following provisions apply to steel produced to other than the ASTM Specifications listed in Section A3.1 when used in sections for which the increased yield point of the steel after cold forming shall be computed from the virgin steel properties according to Section A5.2.2. For acceptance and control purposes, at least four tensile specimens shall be taken from each lot as defined in Section F3.1(d) for the establishment of the representative values of the virgin tensile yield point and ultimate strength. Specimens shall be taken longitudinally from the quarter points of the width near the outer end of the coil.

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1133 15th Street, NW
Washington, DC 20005

AISI
SG-671 0291-7.5M-BPG
Printed in U.S.A.

SPECIFICATION FOR THE DESIGN OF COLD-FORMED STEEL STRUCTURAL MEMBERS

AUGUST 19, 1986, EDITION

Cold-Formed Steel Design Manual - Part I

AMERICAN IRON AND STEEL INSTITUTE
1000 16th STREET, NW
WASHINGTON, DC 20036

1st Printing—March 1987

Produced by W. P. Reyman Associates, Inc. New York

PREFACE

In memory of George Winter, in
recognition of his many contributions
and achievements to the enhancements
of cold-formed steel design.

The newly published Edition of AISI's Specification for the Design of Cold-Formed Steel Structural Members represents a major revision, with many changes made to keep the Specification responsive to the needs of users. It reflects the results of research projects and improvements in design techniques. Moreover, it embodies the results of efforts to simplify the use of the Specification by changes in its format, organization, and content. To accomplish this simplification, relevant sections needed to design a particular member, such as a beam or a column, have been collected together as much as possible.

AISI acknowledges the devoted efforts of the members of the Advisory Group on the Specification of the Design of Cold-Formed Steel Structural Members. This group, comprised of consulting engineers, researchers, designers from companies manufacturing cold-formed steel members, components, assemblies, and complete structures, and specialists from the steel producing industry, has met two to three times per year since its establishment in 1973. Its current members, who have made extensive contributions of time and effort in developing and reaching consensus on the changes which have been described above, are:

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The activities of the Advisory Group are sponsored by AISI's Committee of Sheet Steel Producers. The Specification is issued under the auspices of AISI's Committee on Construction Codes and Standards.

Users of the Specification are invited to continue to offer their valuable comments and suggestions. The cooperation of all involved, the users as well as the writers, is needed to continue to keep the Specification up to date and a useful tool for the designer.

American Iron and Steel Institute
August 19, 1986

*Past Chairman

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Symbol	Definition	Section
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D	Shear stiffener coefficient	B6.2
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SYMBOLS AND DEFINITIONS

Symbol	Definition	Section
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e_{min}	The distance e measured in the line of force from the center of a standard hole to the nearest edge of an adjacent hole or to the end of the connected part toward which the force is directed	E3.1
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F_{ya}	Average yield point of section	A5.2.2
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SYMBOLS AND DEFINITIONS

Symbol	Definition	Section
F_{yv}	Tensile yield point of virgin steel specified by Section A3 or established in accordance with Section F3.3	A5.2.2
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f'_c	Specified compression stress of concrete	E5.1
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f_v	Computed shear stress on a bolt	E4
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I_s	Actual moment of inertia of the full stiffener about its own centroidal axis parallel to the element to be stiffened	B1.1, B4, B4.1, B4.2, B5
I_{sf}	Moment of inertia of the full area of the multiple stiffened element, including the intermediate stiffeners, about its own centroidal axis parallel to the element to be stiffened	B5
I_{yc}	Moment of inertia of the compression portion of a section about the gravity axis of the entire section about the y-axis	D3.1.1
I_x, I_y	Moment of inertia of full section about principal axis	D3.2.2, D1.1
I_{xy}	Product of inertia of full section about major and minor centroidal axes	D3.2.2, D4.1
J	St. Venant torsion constant	C3.1.1

SYMBOLS AND DEFINITIONS

Symbol	Definition	Section
j	Section property for torsional-flexural buckling	C3.1.1
K	Effective length factor	C3.1.3, C4, C4.1, C5
K'	A constant	D3.2.2
K_b	Effective length factor in the plane of bending	C5
K_t	Effective length factor for torsion	C3.1.2
K_x	Effective length factor for bending about x-axis	C3.1.2
K_y	Effective length factor for bending about y-axis	C3.1.2
k	Plate buckling coefficient	B2.1, B2.3, B3.1, B3.2, B4, B4.1, B4.2
k_v	Shear buckling coefficient	B6.2, C3.2
L	Full span for simple beams, distance between inflection points for continuous beams, twice the length of cantilever beams	B1.1c, D3.2.1
L	Length of seam weld not including the circular ends	E2.3
L	Length of fillet weld	E2.4, E2.5
L	Unbraced length of member	C3.1.2, C3.1.3, C4.1, D1.1
L	Live load	F1
L_a	Length of the portion of the span between supports where the flange that is not connected to the sheathing is in compression	C3.1.3
L_{st}	Length of transverse stiffener	B6.1
L_t	Unbraced length of compression member for torsion	C3.1.1
L_x	Unbraced length of compression member for bending about x-axis	C3.1.1
L_y	Unbraced length of compression member for bending about y-axis	C3.1.1
M	Applied bending moment	C3.3, C3.5.1, C3.5.2
M_a	Allowable bending moment permitted if bending stress only exists	C3.1, C3.3, C3.5.1, C3.5.2, C6.1
M_{ax} M_{ay}	Allowable moments about the centroidal axes determined in accordance with Section C3	C5
M_{axo} M_{ayo}	Allowable moments about the centroidal axes determined in accordance with Section C3.1 excluding the provisions of Section C3.1.2	C5, D4.2
M_c	Critical moment	C3.1.2
M_e	Elastic critical moment	C3.1.2
M_n	Nominal moment strength	C3.1, C3.1.1, C3.1.2, C6.1
M_x, M_y	Applied moments about the centroidal axes determined in accordance with Section C3	C5
M_y	Moment causing a maximum strain of e_y	B2.1, C3.1
M_1	Smaller end moment	C3.1.1, C5
M_2	Larger end moment	C3.1.1, C5
m	Distance from the shear center of one channel to the mid-plane of its web	D3.2.2, D1.1
m	$0.192(F_{uv}/F_{yv})-0.068$	A5.2.2

SYMBOLS AND DEFINITIONS

Symbol	Definition	Section
N	Actual length of bearing	D3.6
n	Number of holes	E4
n_p	Number of parallel purlin lines	D3.2.1
P	Concentrated load or reaction	C3.5
P	Applied axial load	C5, D4.1
P	Force transmitted by bolt	E3, E3.1
P	Force transmitted by weld	E2, E2.2
P_s	Allowable concentrated load or reaction for one transverse stiffener	B6.1
P_{ao}	Allowable axial load determined in accordance with Section C4 for $L = 0$	C5
P_L	Force to be resisted by intermediate beam brace	B3.2.2
P_n	Nominal axial strength of member	C4, C6.2
P_n	Nominal strength of connection component	E2, E2.2, E2.3, E2.4, E2.5
$\overline{Q}$	Design shear rigidity for sheathing on both sides of the wall assembly	D4.1
q	Uniformly distributed load in the plane of the web	C3.1.3, D1.1
q_w	Allowable uniform load	C3.1.3
$\overline{q}$	Design shear rigidity for sheathing per inch of stud spacing	D4.1
$\overline{q}_o$	Factor used to determine design shear rigidity	D4.1
q_u	Maximum uniformly distributed load in the plane of the web	C3.1.3
R	Required load carrying capacity	F1
R	Coefficient	C4, C6.2
R	Inside bend radius	A5.2.2 C3.4
r	Radius of gyration of full unreduced cross section	C3.1.1, C4, C4.1
r	Force transmitted by the bolt or bolts at the section considered, divided by the tension force in member at that section	E3.2
r_{cy}	Radius of gyration of one channel about its centroidal axis parallel to web	D1.1
r_o	Polar radius of gyration of cross section about the shear center	C3.1.1, C4.2, D4.1
r_x, r_y	Radius of gyration of cross section about centroidal principal axes	C3.1.1
r_I	Radius of gyration of I-section about the axis perpendicular to the direction in which buckling would occur for the given conditions of end support and intermediate bracing	D1.1
S	$1.28 \sqrt{E/f}$	B4, B4.1
S_c	Elastic section modulus of the effective section calculated at a stress M_c/S_f in the extreme compression fiber	C3.1.1, C3.1.2, C4
S_e	Elastic section modulus of the effective section calculated with extreme compression or tension fiber at F_y	C3.1.1
S_f	Elastic section modulus of full unreduced section for the extreme compression fiber	C3.1.1, C3.1.2, C6.1

SYMBOLS AND DEFINITIONS

Symbol	Definition	Section
s_{max}	Maximum permissible longitudinal spacing of welds or other connectors joining two channels to form an I-section	D1.1
s	Fastener spacing	D1.2, D4.1
s	Spacing in line of stress of welds, rivets, or bolts connecting a compression coverplate or sheet to a non-integral stiffener or other element	E3.2
s	Weld spacing	D1.1
T_a	Allowable tensile strength	C2
T_n	Nominal tensile strength	C2
T_s	Strength of connection in tension	D1.1
t	Base steel thickness of any element or section	A1.2, A3.4, A5.2.1, B1.1, B1.1b, B1.2, B2.1, B4, B4.1, B4.2, B5, B6.1, C3.1.1, C3.1.3, C3.2, C3.4, C3.5.2, C4, C6.1, C6.2, D1.2, E2.4, E2.5
t	Total thickness of the two welded sheets	E2.2
t	Thickness of thinnest connected part	E2.2, E3.1, E4
t_s	Equivalent thickness of a multiple-stiffened element	B5, B6.1
t_w	Effective throat of weld	E2.4, E2.5
V	Actual shear force	C3.3
V_a	Allowable shear force	B6.2, C3.2, C3.3
W	Total load supported by the purlin lines between adjacent supports, lbs.	D3.2.1
w	Flat width of element exclusive of radii	A1.2, B1.1, B2.1, B2.2, B3.1, B4, B4.1, B4.2, B5, C3.1.1, C3.1.3, C4, D1.2
w	Flat width of the beam flange which contacts the bearing plate	C3.5
w_f	Width of flange projection beyond the web or half the distance between webs for box- or U-type sections	B1.1
w_f	Projection of flanges from inside face of web	D1.1
w_1	Leg on weld	E2.4
w_2	Leg on weld	E2.4
x	Distance from concentrated load to brace	D3.2
x_o	Distance from shear center to centroid along the principal x-axis	C3.1.1, C4.2, D4.1

SYMBOLS AND DEFINITIONS

Symbol	Definition	Section
Y	Yield point of web steel divided by yield point of stiffener steel	B6.2
$1/\alpha_x$ $1/\alpha_y$	Magnification factors	C5
β	Coefficient	C4.2, D4.1
γ	Actual shear strain in the sheathing	D4.1
$\bar{\gamma}$	Permissible shear strain of the sheathing	D4.1
θ	Angle between web and bearing surface > 45° but no more than 90°	C3.4
θ	Angle between the vertical and the plane of the web of the Z-section, degrees	D3.2.1
σ	Stress related to shear strain in sheathing	D4.1
σ_{CR}	Theoretical elastic buckling stress	D4.1
σ_t	Torsional buckling stress	C3.1.1, C4.2, D4.1
ρ	Reduction factor	B2.1
λ, λ_c	Slenderness factors	B2.1, C3.5.2
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Ω_b	Factor of safety for bearing	E3.3
Ω_c	Factor of safety for axial compression	B6.1, C4, C5, C6.2, D4.1
Ω_f	Factor of safety for flexure	C3.1, C6.1
Ω_e	Factor of safety for sheet tearing	E2.2, E3.1
Ω_{st}	Factor of safety for end crushing of transverse stiffener	B6.1
Ω_t	Factor of safety for tension on net section	C2, E3.2
Ω_w	Factor of safety for welded connections	E2

SPECIFICATION FOR THE DESIGN OF COLD-FORMED STEEL STRUCTURAL MEMBERS AUGUST 19,1986

A. GENERAL PROVISIONS

A1 Limits of Applicability and Terms

A1.1 Scope and Limits of Applicability

This specification shall apply to the design of structural members cold-formed to shape from carbon or low-alloy steel sheet, strip, plate or bar not more than one inch in thickness and used for load-carrying purposes in buildings. It may also be used for structures other than buildings provided appropriate allowances are made for dynamic effects. Appendices to this Specification shall be considered as integral parts of the Specification.

A1.2 Terms

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

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

(b) *Unstiffened Compression Elements.* An unstiffened compression element is a flat compression element which is stiffened at only one edge parallel to the direction of stress.

(c) *Multiple-Stiffened Elements.* A multiple-stiffened element is an element that is stiffened between webs, or between a web and a stiffened edge, by means of intermediate stiffeners which are parallel to the direction of stress. A *sub-element* is the portion between adjacent stiffeners or between web and intermediate stiffener or between edge and intermediate stiffener.

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

(e) *Effective Design Width.* Where the flat width of an element is reduced for design purposes, the reduced design width is termed the effective width or effective design width.

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

(g) *Torsional-Flexural Buckling.* Torsional-flexural buckling is a mode of buckling in which compression members can bend and twist simultaneously.

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

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

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

(k) *Confirmatory Test.* A confirmatory test is a test made, when desired, on members, connections, and assemblies designed according to the provisions of Sections A through E of this Specification or its specific references, in order to compare actual versus calculated performance.

(l) *Performance Test.* A performance test is a test made on structural members, connections, and assemblies whose performance cannot be determined by the provisions of Sections A through E of this Specification or its specific references.
(m) *Virgin Steel.* Virgin steel refers to steel as received from the steel producer or warehouse before being cold worked as a result of fabricating operations.
(n) *Virgin Steel Properties.* Virgin steel properties refer to mechanical properties of virgin steel such as yield point, tensile strength, and elongation.
(o) *Specified Minimum Yield Point.* The specified minimum yield point is the lower limit of yield point which must be equalled or exceeded in a specification test to qualify a lot of steel for use in a cold-formed steel structural member designed at that yield point.
(p) *Cold-Formed Steel Structural Members.* Cold-formed steel structural members are shapes which are manufactured by press-braking blanks sheared from sheets, cut lengths of coils or plates, or by roll forming cold- or hot-rolled coils or sheets; both forming operations being performed at ambient room temperature, that is, without manifest addition of heat such as would be required for hot forming.

A1.3 Units of Symbols and Terms

The Specification is written so that any compatible system of units may be used except where explicitly stated otherwise in the text of these provisions.

A2 Non-Conforming Shapes and Construction

The provisions of the Specification are not intended to prevent the use of alternate shapes or constructions not specifically prescribed herein. Such alternates shall meet the provisions of Section F of the Specification and be approved by the appropriate building code authority.

A3 Material

A3.1 Applicable Steels

This Specification requires the use of steel of structural quality as defined in general by the provisions of the following specifications of the American Society for Testing and Materials:

ASTM A36/A36M-84a, Structural Steel
ASTM A242/A242M-85, High-Strength Low-Alloy Structural Steel
ASTM A441M-85, High-Strength Low-Alloy Structural Manganese Vanadium Steel
ASTM A446/A446M-85 (Grades A, B, C, D, & F) Steel, Sheet, Zinc-Coated (Galvanized) by the Hot-Dip Process, Structural (Physical) Quality
ASTM A500-84, Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes
ASTM A529/A529M-85, Structural Steel with 42 ksi Minimum Yield Point (½ in. Maximum Thickness)
ASTM A570/A570M-85 Steel, Sheet and Strip, Carbon, Hot-Rolled, Structural Quality
ASTM A572/A572M-85, High-Strength Low-Alloy Columbium-Vanadium Steels of Structural Quality
ASTM A588/A588M-85, High-Strength Low-Alloy Structural Steel with 50 ksi Minimum Yield Point to 4 in. Thick
ASTM A606-85 Steel, Sheet and Strip, High Strength, Low Alloy, Hot-Rolled and Cold-Rolled, with Improved Atmospheric Corrosion Resistance
ASTM A607-85 Steel Sheet and Strip, High Strength, Low Alloy, Columbium or Vanadium, or both, Hot-Rolled and Cold-Rolled
ASTM A611-85 (Grades A, B, C, & D) Steel, Sheet, Carbon, Cold-Rolled, Structural Quality
ASTM A715-85 (Grades 50 and 60) Sheet Steel and Strip, High-Strength, Low-Alloy, Hot-Rolled, With Improved Formability
ASTM A792-85a (Grades 33, 37, 40 & 50) Steel Sheet, Aluminum-Zinc Alloy-Coated by the Hot-Dip Process, General Requirements

A3.2 Other Steels

The listing in Section A3.1 does not exclude the use of steel up to and including one inch in thickness ordered or produced to other than the listed specifications provided such steel conforms to the chemical and mechanical requirements of one of the listed specifications or other published specification which establishes its properties and suitability, and provided it is subjected by either the producer or the purchaser to analyses, tests and other controls to the extent and in the manner prescribed by one of the listed specifications and Section A3.3.

A3.3 Ductility

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

A3.3.1 The ratio of tensile strength to yield point shall not be less than 1.08, and the total elongation shall not be less than 10 percent for a two-inch gage length or 7 percent for an eight-inch gage length standard specimen tested in accordance with ASTM A370-77[ε]. The provisions of Chapters B through E of this Specification are limited to steels conforming to these requirements.

A3.3.2 Steels conforming to ASTM A446 Grade E and A611 Grade E and other steels which do not meet the provisions of Section A3.3.1 may be used for particular configurations provided (1) the yield strength, F_y, used for design in Chapters B, C and D is taken as 75 percent of the specified minimum yield point or 60 ksi, whichever is less and (2) the tensile strength, F_u, used for design in Chapter E is taken as 75 percent of the specified minimum tensile stress or 62 ksi, whichever is less. Alternatively, the suitability of such steels for the configuration shall be demonstrated by load tests in accordance with Section F1. Allowable loads based on these tests shall not exceed the loads calculated according to Chapters B through E, using the specified minimum yield point, F_{sy}, for F_y and the specified minimum tensile strength, F_u.

Allowable loads based on existing use shall not exceed the loads calculated according to Chapters B through E, using the specified minimum yield point, F_{sy}, for F_y and the specified minimum tensile strength, F_u.

A3.4 Delivered Minimum Thickness

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

A4 Loads

A4.1 Dead Load

The dead load to be assumed in design shall consist of the weight of steelwork and all material permanently fastened thereto or supported thereby.

A4.2 Live Load

The live load shall be that stipulated by the applicable code or specification under which the structure is being designed or that dictated by the conditions involved.

A4.3 Impact Load

For structures carrying live loads which induce impact, the assumed live load shall be increased sufficiently to provide for impact.

A4.4 Wind or Earthquake Loads

Where load combinations specified by the applicable building code include wind or earthquake loads, the resulting forces may be multiplied by 0.75.

A4.5 Ponding

Unless a roof surface is provided with sufficient slope toward points of free drainage or adequate individual drains to prevent the accumulation of rainwater, the roof system shall be investigated by rational analysis to assure stability under ponding conditions.

A5 Structural Analysis and Design

A5.1 Design Basis

This Specification is based upon the allowable stress concept presented in terms of allowable moments and loads. The allowable moments and loads are determined by dividing the corresponding nominal capacities by an accepted factor of safety.

A5.2 Yield Point and Strength Increase from Cold Work of Forming

A5.2.1 Yield Point

The yield point used in design, F_y, shall not exceed the specified minimum yield point, or as established in accordance with Chapter F, or as increased for cold work of forming in Section A5.2.2, or as reduced for low ductility steels in Section A3.3.2.

A5.2.2 Strength Increase from Cold Work of Forming

Provisions for the strength increase from cold work of forming are given in Appendix A5.2.2.

A5.3 Serviceability and Durability

A structure shall be designed to perform its required functions during its expected life, including serviceability and durability considerations.

A6 Reference Documents

This Specification recognizes other published and latest approved specifications and manuals for designs contemplated herein, as follows:

1. American National Standards Institute, ANSI A58.1-1982, "Minimum Design Loads in Buildings and Other Structures,"* American National Standards Institute, Inc., (ANSI), 1430 Broadway, New York, New York 10018
2. Applicable standards of the American Society for Testing and Materials, (ASTM), 1916 Race Street, Philadelphia, Pennsylvania 19013
3. American Institute of Steel Construction, "Specification for the Design, Fabrication and Erection of Structural Steel for Buildings," American Institute of Steel Construction, (AISC), 400 North Michigan Avenue, Chicago, Illinois 60611, November 1, 1978
4. American Welding Society, AWS D1.3-81, "Structural Welding Code—Sheet Steel," American Welding Society, (AWS), 550 N.W. LeJeune Road, Miami, Florida 33126
5. Research Council on Structural Connections, Allowable Stress Design, "Specification for Structural Joints Using ASTM A325 or A490 Bolts," Research Council on Structural Connections, (RCSC), American Institute of Steel Construction (AISC) 400 North Michigan Avenue, Chicago, Illinois 60611, November 13, 1985.
6. Metal Building Manufacturers Association, *Low Rise Building Systems Manual*, Metal Building Manufacturers Association (MBMA), 1230 Keith Building, Cleveland, Ohio 44115
7. Steel Deck Institute, "Design Manual for Composite Decks, Formed Decks, and Roof Decks," Steel Deck Institute, Inc., P.O. Box 9506, Canton, Ohio, 44711, 1984
8. Steel Joist Institute, "Standard Specifications Load Tables and Weight Tables for Steel

*For further information contact ASCE, New York, New York.

Joists and Joist Girders," Steel Joist Institute, (SJI), Suite A, 1205 48th Avenue North, Myrtle Beach, South Carolina 29577, 1986

9. Rack Manufacturers Institute, "Specification for the Design, Testing and Utilization of Industrial Steel Storage Racks," Rack Manufacturers Institute, (RMI) 8720 Red Oak Boulevard, Suite 201, Charlotte, North Carolina 28210, 1985
10. American Iron ahd Steel Institute, "Stainless Steel Cold-Formed Structural Design Manual," 1974 Edition, American Iron and Steel Institute, (AISI), 1000 16th Street, N.W., Washington, D.C. 20036
11. American Society of Civil Engineers, "ASCE Standard, Specification for the Design and Construction of Composite Slabs," American Society of Civil Engineers, (ASCE), 345 East 47th Street, New York, New York 10017, October, 1984
12. American Iron and Steel Institute, "Tentative Criteria for Structural Applications of Steel Tubing and Pipe," American Iron and Steel Institute, (AISI), 1000 16th Street, N.W., Washington, D.C. 20036, August, 1976

B. ELEMENTS

B1 Dimensional Limits and Considerations

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

(a) *Maximum Flat-Width-to-Thickness Ratios*
Maximum allowable overall flat-width-to-thickness ratios, w/t, disregarding intermediate stiffeners and taking as t the actual thickness of the element, shall be as follows:

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

Simple lip	60
Any other kind of stiffener having $I_s > I_a$ and $D/w < 0.8$ according to Section B4.2	90

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

(3) Unstiffened compression element and elements with an edge stiffener having $I_s < I_a$ and $D/w \leq 0.8$ according to Section B4.2 — 60

Note: Unstiffened compression elements that have w/t ratios exceeding approximately 30 and stiffened compression elements that have w/t ratios exceeding approximately 250 are likely to develop noticeable deformation at the full allowable load, without affecting the ability of the member to carry design loads.

Stiffened elements having w/t ratios larger than 500 may be used with safety to support loads, but substantial deformation of such elements under load may occur and may render inapplicable the design formulas of this Specification.

(b) *Flange Curling*
Provisions for limiting the amount of curling or the deflection of a tension or compression flange are given in Appendix B1.1(b).

(c) *Shear Lag Effects—Unusually Short Spans Supporting Concentrated Loads*
The effective design width of tension and compression flanges of beams supporting concentrated loads and having short spans shall be limited in accordance with Appendix B1.1(c).

B1.2 Maximum Web Depth-to-Thickness Ratio

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

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

(b) For webs which are provided with transverse stiffeners satisfying the requirements of Section B6.1:

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

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

In the above,

h = Depth of flat portion of web measured along the plane of web
t = Web thickness

Where a web consists of two or more sheets, the h/t ratio shall be computed for the individual sheets.

B2 Effective Widths of Stiffened Elements

B2.1 Uniformly Compressed Stiffened Elements

(a) *Load Capacity Determination*

The effective widths, b, of uniformly compressed elements shall be determined from the following formulas:

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

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

where

w = Flat width as shown in Figure B2.1

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

λ is a slenderness factor determined as follows:

$$\lambda = \frac{1.052}{\sqrt{k}}\left(\frac{w}{t}\right)\sqrt{\frac{f}{E}} \quad (Eq.\ B2.1\text{-}4)$$

where

f for load capacity determination is as follows:

For flexural members:

(1) If Procedure I of Section C3.1.1 is used, $f = F_y$ if the initial yielding is in compression in the element considered.

If the initial yielding is not in compression in the element considered, then the stress f shall be determined for the element considered on the basis of the effective section at M_y (moment causing initial yield).

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

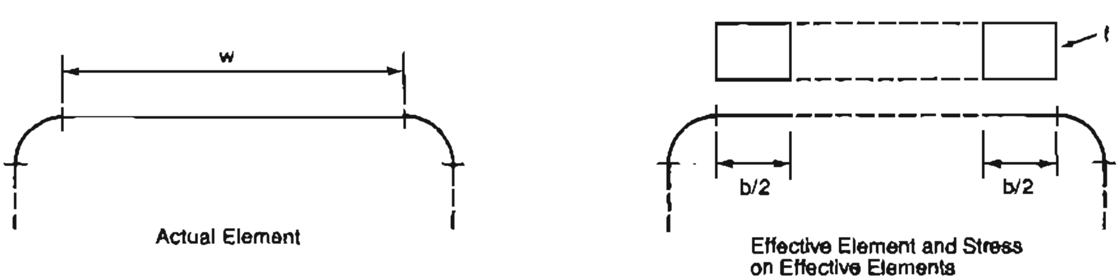


Figure B2.1-1 Stiffened Elements

(3) If Section C3.1.2 is used, then the $f = \frac{M_c}{S_f}$ as described in that Section in determining S_c.

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

E = Modulus of elasticity
k = Plate buckling coefficient
= 4 for stiffened elements supported by a web on each longitudinal edge. Values for different types of elements are given in the applicable sections.

(b) *Deflection Determination*
The effective widths, b_d, used in computing deflections shall be determined from the following formulas:

$b_d = w$ when $\lambda \leq 0.673$ (*Eq.* B2.1-5)
$b_d = \rho w$ when $\lambda > 0.673$ (*Eq.* B2.1-6)

where
w = Flat width
ρ = Reduction factor determined by either of the following two procedures:

(1) Procedure I.
A low estimate of the effective width may be obtained from Eqs. B2.1-3 and B2.1-4 where f_d is substituted for f where f_d is the computed compressive stress in the element being considered.

(2) Procedure II.
For stiffened elements supported by a web on each longitudinal edge an improved estimate of the effective width can be obtained by calculating ρ as follows:

$\rho = 1$ when $\lambda \leq 0.673$ (*Eq.* B2.1-7)
$\rho = (1.358 - 0.461/\lambda)/\lambda$ when $0.673 < \lambda < \lambda_c$ (*Eq.* B2.1-8)
$\rho = (0.41 + 0.59\sqrt{F_y/f_d} - 0.22/\lambda)/\lambda$ when $\lambda \geq \lambda_c$ (*Eq.* B2.1-9)

where
$\lambda_c = 0.256 + 0.328\,(w/t)\sqrt{F_y/E}$ (*Eq.* B2.1-10)
and λ is as defined by Eq. B2.1-4 except that f_d is substituted for f.

B2.2 Uniformly Compressed Stiffened Elements with Circular Holes

(a) *Load Capacity Determination*
The effective width, b, of stiffened elements with uniform compression having circular holes shall be determined as follows:

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

center-to-center spacing of holes $> 0.50w$, and $3d_h$,

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

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

where
w = Flat width
d_h = Diameter of holes
λ is as defined in Section B2.1.

(b) *Deflection Determination*
The effective width, b_d, used in deflection calculations shall be equal to b determined in accordance with Procedure I of Section B2.2a except that f_d is substituted for f, where f_d is the computed compressive stress in the element being considered.

B2.3 Effective Width of Webs and Stiffened Elements with Stress Gradient

(a) *Load Capacity Determination*
The effective widths, b_1 and b_2, as shown in Figure B2.3-1 shall be determined from the following formulas:

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

For $\psi \le -0.236$

$b_2 = b_e/2$ (*Eq.* B2.3-2)

$b_1 + b_2$ shall not exceed the compression portion of the web calculated on the basis of effective section

For $\psi > -0.236$

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

where

b_e = Effective width b determined in accordance with Section B2.1 with f_1 substituted for f and with k determined as follows:

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

$\psi = f_2/f_1$

f_1, f_2 = Stresses shown in Figure B2.3-1 calculated on the basis of effective section. f_1 is compression (+) and f_2 can be either tension (−) or compression. In case f_1 and f_2 are both compression, $f_1 \ge f_2$

(b) *Deflection Determination*
The effective widths in computing deflections at a given load shall be determined in accordance with Section B2.3a except that f_{d1} and f_{d2} are substituted for f_1 and f_2, where f_{d1}, f_{d2} = Computed stresses f_1 and f_2 as shown in Figure B2.3-1. Calculations are based on the effective section at the load for which deflections are determined.

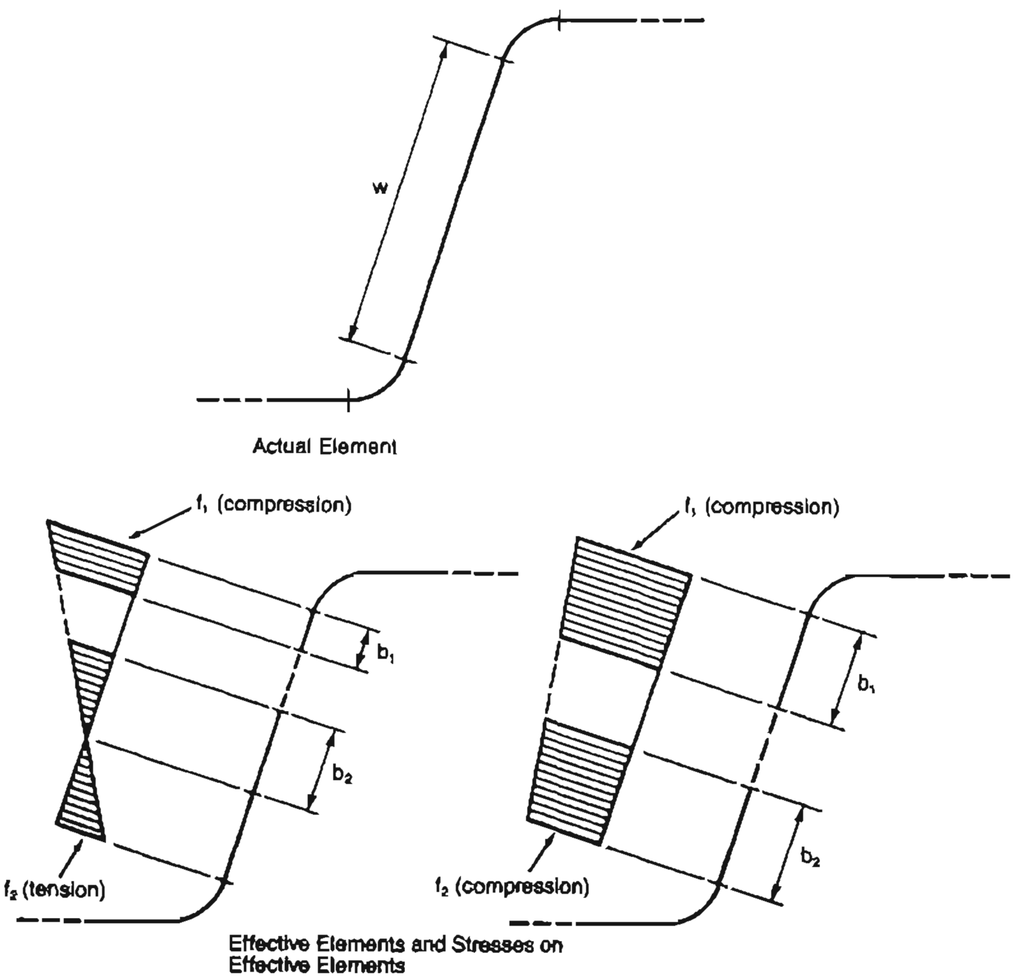


Figure B2.3-1 Stiffened Elements with Stress Gradient and Webs

B3 Effective Widths of Unstiffened Elements

B3.1 Uniformly Compressed Unstiffened Elements

(a) *Load Capacity Determination*
Effective widths, b, of unstiffened compression elements with uniform compression shall be determined in accordance with Section B2.1a with the exception that k shall be taken as 0.43 and w as defined in Figure B3.1-1.

(b) *Deflection Determination*
The effective widths used in computing deflections shall be determined in accordance with Procedure I of Section B2.1b except that f_d is substituted for f and k = 0.43.

B3.2 Unstiffened Elements and Edge Stiffeners with Stress Gradient

(a) *Load Capacity Determination*
Effective widths, b, of unstiffened compression elements and edge stiffeners with stress gradient shall be determined in accordance with Section B2.1a with $f = f_3$ as in Figure B4-2 in the element and k = 0.43.

(b) *Deflection Determination*
Effective widths, b, of unstiffened compression elements and edge stiffeners with stress gradient shall be determined in accordance with Procedure I Section B2.1b except that f_{d3} is substituted for f and k = 0.43.

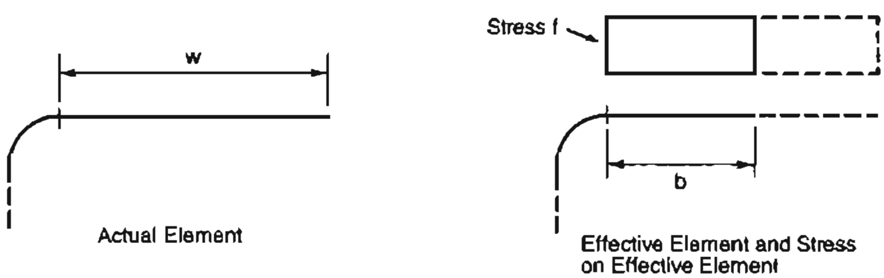


Figure B3.1-1 Unstiffened Element with Uniform Compression

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

The following notation is used in this section.

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

k = Buckling coefficient

b_o = Dimension defined in Figure B4-1

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

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

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

C_1, C_2 = Coefficients defined in Figures B4-1 and B4-2

A_s = Reduced area of the stiffener as specified in this section. A_s is to be used in computing the overall effective section properties. The centroid of the stiffener is to be considered located at the centroid of the full area of the stiffener, and the moment of inertia of the stiffener about its own centroidal axis shall be that of the full section of the stiffener.

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

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

For the stiffener shown in Figure B4-2,

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

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

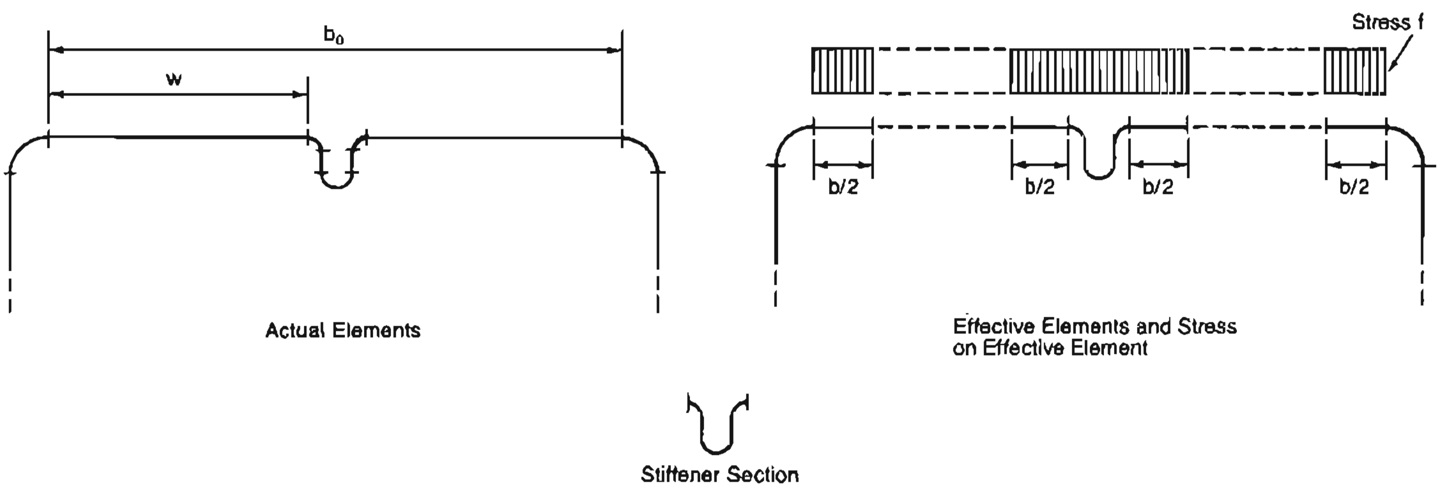


Figure B4-1 Elements with Intermediate Stiffener

B4.1 Uniformly Compressed Elements with an Intermediate Stiffener

(a) *Load Capacity Determination*

Case I: $b_o/t \le S$ (*Eq.* B4.1-1)

$I_a = 0$ (no intermediate stiffener needed) (*Eq.* B4.1-2)

$b = w$ (*Eq.* B4.1-3)

$A_s = A'_s$ (*Eq.* B4.1-4)

Case II: $S < b_o/t < 3S$ (*Eq.* B4.1-5)

$I_a/t^4 = [50(b_o/t)/S] - 50$ (*Eq.* B4.1-6)

b and A_s shall be calculated according to Section B2.1a where

$k = 3(I_s/I_a)^{1/2} + 1 \le 4$ (*Eq.* B4.1-7)

$A_s = A'_s(I_s/I_a) \le A'_s$ (*Eq.* B4.1-8)

Case III: $b_o/t \ge 3S$

$I_a/t^4 = [128(b_o/t)/S] - 285$ (*Eq.* B4.1-9)

b and A_s are calculated according to Section B2.1a where

$k = 3(I_s/I_a)^{1/3} + 1 \le 4$ (*Eq.* B4.1-10)

$A_s = A'_s(I_s/I_a) \le A'_s$ (*Eq.* B4.1-11)

(b) *Deflection Determination*
Effective widths shall be determined as in Section B4.1a except that f_d is substituted for f.

B4.2 Uniformly Compressed Elements with an Edge Stiffener

(a) *Load Capacity Determination*

Case I: $w/t \le S/3$ (*Eq.* B4.2-1)

$I_a = 0$ (no edge stiffener needed) (*Eq.* B4.2-2)

$b = w$ (*Eq.* B4.2-3)

$d_s = d'_s$ for simple lip stiffener (*Eq.* B4.2-4)

$A_s = A'_s$ for other stiffener shapes (*Eq.* B4.2-5)

Case II: $S/3 < w/t < S$

$I_a/t^4 = 399\{[(w/t)/S] - 0.33\}^3$ (*Eq.* B4.2-6)

$n = 1/2$

$C_2 = I_s/I_a \leq 1$ (*Eq.* B4.2-7)

$C_1 = 2 - C_2$ (*Eq.* B4.2-8)

b shall be calculated according to Section B2.1 where

$k = [4.82 - 5(D/w)](I_s/I_a)^n + 0.43 \leq 5.25 - 5(D/w)$ (*Eq.* B4.2-9)

for $0.8 \geq D/w > 0.25$

$k = 3.57(I_s/I_a)^n + 0.43 \leq 4.0$ (*Eq.* B4.2-10)

for $(D/w) \leq 0.25$

$d_s = d'_s(I_s/I_a) \leq d'_s$ (*Eq.* B4.2-11)

for simple lip stiffener

$A_s = A'_s(I_s/I_a) \leq A'_s$ (*Eq.* B4.2-12)

for other stiffener shape

Case III: $w/t \geq S$

$I_a/t^4 = [115(w/t)/S] + 5$ (*Eq.* B4.2-13)

C_1, C_2, b, k, d_s, A_s are calculated per Case II with $n = 1/3$.

(b) *Deflection Determination*

Effective widths shall be determined as in Section B4.2a except that f_d is substituted for f.

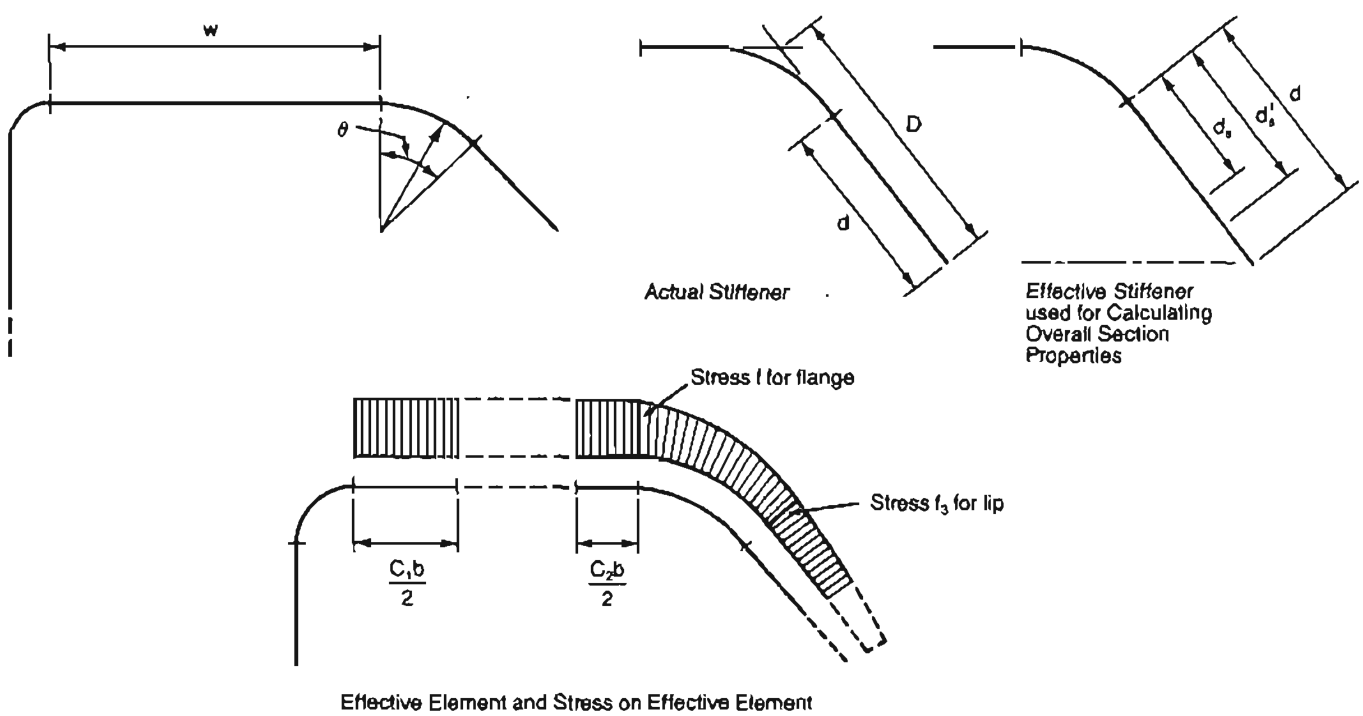


Figure B4-2 Elements with Edge Stiffener

B5 Effective Widths of Edge Stiffened Elements with Intermediate Stiffeners or Stiffened Elements with More Than One Intermediate Stiffener

For the determination of the effective width, the intermediate stiffener of an edge stiffened element or the stiffeners of a stiffened element with more than one stiffener shall be disregarded unless each intermediate stiffener has the minimum I_s as follows:

$$I_{min} = [3.66\sqrt{(w/t)^2 - (0.136E)/F_y}]t^4 \quad (Eq.\ B5\text{-}1)$$

but not less than $18.4t^4$

where

w/t = Width-thickness ratio of the larger stiffened sub-element

I_s = Moment of inertia of the full stiffener about its own centroid axis parallel to the element to be stiffened

(a) If the spacing of intermediate stiffeners between two webs is such that for the sub-element between stiffeners $b < w$ as determined in Section B2.1, only two intermediate stiffeners (those nearest each web) shall be considered effective.

(b) If the spacing of intermediate stiffeners between a web and an edge stiffener is such that for the sub-element between stiffeners $b < w$ as determined in Section B2.1, only one intermediate stiffener, that nearest the web, shall be considered effective.

(c) If intermediate stiffeners are spaced so closely that for the elements between stiffeners $b = w$ as determined in Section B2.1, all the stiffeners may be considered effective. In computing the flat-width to thickness ratio of the entire multiple-stiffened element, such element shall be considered as replaced by an "equivalent element" without intermediate stiffeners whose width, b_o, is the full width between webs or from web to edge stiffener, and whose equivalent thickness, t_s, is determined as follows:

$$t_s = \sqrt[3]{12 I_{sf}/b_o} \qquad (Eq.\ B5\text{-}2)$$

where

I_{sf} = Moment of inertia of the full area of the multiple-stiffened element, including the intermediate stiffeners, about its own centroidal axis. The moment of inertia of the entire section shall be calculated assuming the "equivalent element" to be located at the centroidal axis of the multiple stiffened element, including the intermediate stiffener. The actual extreme fiber distance shall be used in computing the section modulus.

(d) If $w/t > 60$, the effective width, b_e, of the sub-element or element shall be determined from the following formula:

$$\frac{b_e}{t} = \frac{b}{t} - 0.10\left[\frac{w}{t} - 60\right] \qquad (Eq.\ B5\text{-}3)$$

where:

w/t = flat-width ratio of sub-element or element

b = effective design width determined in accordance with the provisions of Section B2.1, in.

b_e = effective design width of sub-element or element to be used in design computations, in.

For computing the effective structural properties of a member having compression sub-elements or element subject to the above reduction in effective width, the area of stiffeners (edge stiffener or intermediate stiffeners) shall be considered reduced to an effective area as follows:

For $60 < w/t < 90$:

$$A_{ef} = \alpha A_{st} \qquad (Eq.\ B5\text{-}4)$$

where

$$\alpha = (3 - 2b_e/w) - \frac{1}{30}\left[1 - \frac{b_e}{w}\right]\left[\frac{w}{t}\right] \qquad (Eq.\ B5\text{-}5)$$

For $w/t \geq 90$:

$$A_{ef} = (b_e/w) A_{st} \qquad (Eq.\ B5\text{-}6)$$

In the above expressions, A_{ef} and A_{st} refer only to the area of the stiffener section, exclusive of any portion of adjacent elements.

The centroid of the stiffener is to be considered located at the centroid of the full area of the stiffener, and the moment of inertia of the stiffener about its own centroidal axis shall be that of the full section of the stiffener.

B6 Stiffeners

B6.1 Transverse Stiffeners

Transverse stiffeners attached to beam webs at points of concentrated loads or reactions, shall be designed as compression members. Concentrated loads or reactions shall be applied directly into the stiffeners, or each stiffener shall be fitted accurately to the flat portion of the flange to provide direct load bearing into the end of the stiffener. Means for shear transfer between the stiffener and the web shall be provided according to Chapter E. The concentrated loads or reactions shall not exceed the smaller of the allowable loads, P_a, given by (a) and (b) as follows:

(a) $P_a = P_n/\Omega_{st}$ (*Eq.* B6.1-1)

where

$P_n = F_{wy}A_c$ (*Eq.* B6.1-2)

$\Omega_{st} = 2.00$

$A_c = 18t^2 + A_s$, for transverse stiffeners at interior support and under concentrated load (*Eq.* B6.1-3)

$A_c = 10t^2 + A_s$, for transverse stiffeners at end support (*Eq.* B6.1-4)

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

(b) $P_a = P_n/\Omega_c$ (*Eq.* B6.1-5)

where

P_n = Nominal axial load evaluated according to Section C4(a) with A_e replaced by A_b

Ω_c = Factor of safety for axial compression evaluated according to Section C4(a)

$A_b = b_1t + A_s$, for transverse stiffeners at interior support and under concentrated load (*Eq.* B6.1-6)

$A_b = b_2t + A_s$, for transverse stiffeners at end support (*Eq.* B6.1-7)

A_s = Cross sectional area of transverse stiffeners

$b_1 = 25t[0.0024(L_{st}/t) + 0.72] \le 25t$ (*Eq.* B6.1-8)

$b_2 = 12t[0.0044(L_{st}/t) + 0.83] \le 12t$ (*Eq.* B6.1-9)

L_{st} = Length of transverse stiffener

t = Base thickness of beam web.

The w/t_s ratio for the stiffened and unstiffened elements of cold-formed steel transverse stiffeners shall not exceed $1.28\sqrt{(E/F_{ys})}$ and $0.37\sqrt{(E/F_{ys})}$ respectively, where F_{ys} is the yield stress, f_y, and t_s the thickness of the stiffener steel.

B6.2 Shear Stiffeners

Where shear stiffeners are required, the spacing shall be such that the web shear force shall not exceed the allowable shear force, V_a, permitted by Section C3.2, and the ratio a/h shall not exceed $[260/(h/t)]^2$ nor 3.0.

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

$$I_{smin} = 5ht^3[h/a - 0.7(a/h)] \ge (h/50)^4 \quad (Eq.\ B6.2\text{-}1)$$

The gross area of shear stiffeners shall be not less than

$$A_{st} = \frac{1-C_v}{2}\left[\frac{a}{h} - \frac{(a/h)^2}{(a/h)+\sqrt{1+(a/h)^2}}\right]YDht \quad (Eq.\ B6.2\text{-}2)$$

where

$$C_v = \frac{45{,}000k_v}{F_y(h/t)^2} \quad \text{when } C_v \le 0.8 \quad (Eq.\ B6.2\text{-}3)$$

$$C_v = \frac{190}{h/t}\left(\frac{k_v}{\sqrt{F_y}}\right) \text{ when } C_v > 0.8 \qquad (Eq.\ B6.2\text{-}4)$$

$$k_v = 4.00 + \frac{5.34}{(a/h)^2} \text{ when } a/h \le 1.0 \qquad (Eq.\ B6.2\text{-}5)$$

$$k_v = 5.34 + \frac{4.00}{(a/h)^2} \text{ when } a/h > 1.0 \qquad (Eq.\ B6.2\text{-}6)$$

a = Distance between transverse stiffeners

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

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

B6.3 Non-Conforming Stiffeners

The allowable load carrying capacity of members with transverse stiffeners that do not meet the requirements of Sections B6.1 or B6.2, such as stamped or rolled-in transverse stiffeners shall be determined by tests in accordance with Chapter F of this Specification.

C. MEMBERS

C1 Properties of Sections

Properties of sections (cross-sectional area, moment of inertia, section modulus, radius of gyration, etc.) shall be determined in accordance with conventional methods of structural design. Properties shall be based on the full cross section of the members (or net sections where the use of net section is applicable) except where the use of a reduced cross section, or effective design width, is required.

C2 Tension Members

For axially loaded tension members, the applied tensile force shall not exceed T_a determined as follows:

$$T_a = T_n/\Omega_t \qquad (Eq.\ C2\text{-}1)$$

where
T_n = Strength of member when loaded in tension
$= A_nF_y$ (*Eq.* C2-2)
Ω_t = Factor of safety for tension
= 1.67
A_n = Net area of the cross section
F_y = Design yield stress

C3 Flexural Members

C3.1 Strength for Bending Only

In flexural members, the applied moment uncoupled from axial load, shear, and local concentrated forces or reactions shall not exceed the allowable M_a calculated as follows:

$$M_a = M_n/\Omega_f \qquad (Eq.\ C3.1\text{-}1)$$

where

M_n = Smaller of the nominal moment strengths calculated according to Sections C3.1.1 and C3.1.2

Ω_f = Factor of safety for bending
= 1.67

C3.1.1 Nominal Section Strength

Section strength shall be calculated either on the basis of initiation of yielding in the effective section (Procedure I) or on the basis of the inelastic reserve capacity (Procedure II) as applicable.

(a) Procedure I—Based on Initiation of Yielding
Effective yield moment based on section strength, M_n, shall be determined as follows:

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

where

F_y = Design yield stress as determined in Section A5.2.1

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

(b) Procedure II—Based on Inelastic Reserve Capacity
The inelastic flexural reserve capacity may be used when the following conditions are met:

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

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

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

(4) The shear force does not exceed $0.35F_y$ times the web area, $h \times t$.

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

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

where

e_y = Yield strain = F_y/E

E = Modulus of elasticity

C_y = Compression strain factor determined as follows:

(a) Stiffened compression elements without intermediate stiffeners

$$C_y = 3 \text{ for } w/t \le \lambda_1$$

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

$$C_y = 1 \text{ for } w/t \ge \lambda_2$$

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

$$C_y = 1$$

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

$$C_y = 1$$

When applicable, effective design widths defined in Section B3.1 shall be used in calculating section properties. M_n shall be calculated considering equilibrium of stresses, assuming an ideally elastic-plastic stress-strain curve which is the same in tension as in compression, assuming small deformation and assuming that plane sections remain plane during bending. Combined bending and web crippling shall be checked by provisions of Section C3.4.

C3.1.2 Lateral Buckling Strength

For the laterally unbraced segments of doubly- or singly-symmetric sections* subject to lateral buckling, M_n shall be determined as follows:

$$M_n = S_c \frac{M_c}{S_f} \qquad (Eq.\ C3.1.2\text{-}1)$$

where

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

S_c = Elastic section modulus of the effective section calculated at a stress M_c/S_f in the extreme compression fiber

M_c = Critical moment calculated according to (a) or (b) below:

(a) For I- or Z-sections bent about the centroidal axis perpendicular to the web (x-axis):

For $M_e \geq 2.78M_y$

$$M_c = M_y \qquad (Eq.\ C3.1.2\text{-}2)$$

For $2.78M_y > M_e > 0.56M_y$

$$M_c = \frac{10}{9}M_y\left(1-\frac{10M_y}{36M_e}\right) \qquad (Eq.\ C3.1.2\text{-}3)$$

For $M_e \leq 0.56M_y$

$$M_c = M_e \qquad (Eq.\ C3.1.2\text{-}4)$$

where

M_y = Moment causing initial yield at the extreme compression fiber of the full section

$= S_f F_y \qquad (Eq.\ C3.1.2\text{-}5)$

M_e = Elastic critical moment determined either as defined in (b) below or as follows:

$$= \pi^2 EC_b \frac{dI_{yc}}{L^2} \text{ for doubly-symmetric I-sections} \qquad (Eq.\ C3.1.2\text{-}6)$$

$$= \frac{\pi^2 EC_b dI_{yc}}{2L^2} \text{ for point-symmetric Z-sections} \qquad (Eq.\ C3.1.2\text{-}7)$$

L = Unbraced length of the member

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

Other terms are defined in (b) below.

(b) For singly-symmetric sections (x-axis is assumed to be the axis of symmetry):

For $M_e > 0.5M_y$

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

$$M_c = M_y\left(1 - \frac{M_y}{4M_e}\right) \qquad (Eq.\ C3.1.2\text{-}8)$$

For $M_e \le 0.5M_y$

$$M_c = M_e \qquad (Eq.\ C3.1.2\text{-}9)$$

where

M_y is as defined in (a) above

M_e = Elastic critical moment

= $C_b r_o A\sqrt{\sigma_{ey}\sigma_t}$ for bending about the symmetry axis (x-axis is the axis of symmetry oriented such that the shear center has a negative x-coordinate.)

Alternatively, M_e can be calculated using the formula for doubly-symmetric I-sections given in (a) above (*Eq.* C3.1.2-10)

= $C_s A\sigma_{ex}[j + C_s\sqrt{j^2 + r_o^2(\sigma_t/\sigma_{ex})}\,]/C_{TF}$ for bending centroidal axis perpendicular to the symmetry axis (*Eq.* C3.1.2-11)

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

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

$$\sigma_{ex} = \frac{\pi^2 E}{(K_x L_x/r_x)^2} \qquad (Eq.\ C3.1.2\text{-}12)$$

$$\sigma_{ey} = \frac{\pi^2 E}{(K_y L_y/r_y)^2} \qquad (Eq.\ C3.1.2\text{-}13)$$

$$\sigma_t = \frac{1}{Ar_o^2}\left[GJ + \frac{\pi^2 EC_w}{(K_t L_t)^2}\right] \qquad (Eq.\ C3.1.2\text{-}14)$$

A = Full cross-sectional area

C_b = Bending coefficient which can conservatively be taken as unity, or calculated from

C_b = $1.75 + 1.05[\,(M_1/M_2)\,] + 0.3[\,(M_1/M_2)\,]^2 \le 2.3$

where

M_1 is the smaller and M_2 the larger bending moment at the ends of the unbraced length, taken about the strong axis of the member, and where M_1/M_2, the ratio of end moments, is positive when M_1 and M_2 have the same sign (reverse curvature bending) and negative when they are of opposite sign (single curvature bending). When the bending moment at any point within an unbraced length is larger than that at both ends of this length, and for members subject to combined axial load and bending moment (Section C5), C_b shall be taken as unity.

E = Modulus of elasticity

d = Depth of section

C_{TF} = $0.6 - 0.4(M_1/M_2)$

where

M_1 is the smaller and M_2 the larger bending moment at the ends of the unbraced length, and where M_1/M_2, the ratio of end moments, is positive when M_1 and M_2 have the same sign (reverse curvature bending) and negative when they are of opposite sign

(single curvature bending). When the bending moment at any point within an unbraced length is larger than that at both ends of this length, and for members subject to combined axial load and bending moment (Section C5), C_b shall be taken as unity.

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

$$= \sqrt{r_x^2 + r_y^2 + x_o^2} \qquad (Eq.\ C3.1.2\text{-}15)$$

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

E = Modulus of elasticity

G = Shear modulus

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

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

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

J = St. Venant torsion constant of the cross section

C_w = Torsional warping constant of the cross section

$$j = \frac{1}{2I_y}\left[\int_A x^3 dA + \int_A xy^2 dA\right] - x_o \qquad (Eq.\ C3.1.2\text{-}16)$$

C3.1.3 Beams With One Flange Attached to Deck or Sheathing

C- and Z-sections with the tension flange attached to deck or sheathing and with the compression flange laterally unbraced shall be designed using a rational method of analysis; alternatively, full scale testing in accordance with Section F1 may be used.

C3.2 Strength for Shear Only

The shear force at any section shall not exceed the allowable shear, V_a, calculated as follows:

(a) For $h/t \le 1.38\sqrt{Ek_v/F_y}$

$$V_a = 0.38t^2\sqrt{k_v F_y E} \le 0.4F_y ht \qquad (Eq.\ C3.2\text{-}1)$$

(b) For $h/t > 1.38\sqrt{Ek_v/F_y}$

$$V_a = 0.53\ Ek_v t^3/h \qquad (Eq.\ C3.2\text{-}2)$$

where

t = Web thickness

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

k_v = Shear buckling coefficient determined as follows:

1. For unreinforced webs, $k_v = 5.34$
2. For beam webs with transverse stiffeners satisfying the requirements of Section B6

when $a/h \le 1.0$

$$k_v = 4.00 + \frac{5.34}{(a/h)^2} \qquad (Eq.\ C3.2\text{-}3)$$

when $a/h > 1.0$

$$k_v = 5.34 + \frac{4.00}{(a/h)^2} \qquad (Eq.\ C3.2\text{-}4)$$

where

a = the shear panel length for unreinforced web element
= distance between transverse stiffeners for web elements.

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

C3.3 Strength for Combined Bending and Shear

For beams with unreinforced webs, the moment, M, and shear, V, shall satisfy the following interaction equation:

$$(M/M_a)^2 + (V/V_a)^2 \leq 1.0 \qquad (Eq.\ C3.3\text{-}1)$$

For beams with transverse web stiffeners, the moment, M, and shear, V, shall not exceed M_a and V_a, respectively. When $M/M_a > 0.5$ and $V/V_a > 0.7$, then M and V shall satisfy the following interaction equation:

$$0.6(M/M_a) + (V/V_a) \leq 1.3 \qquad (Eq.\ C3.3\text{-}2)$$

In the above equations:
M_a = Allowable moment when bending alone exists
V_a = Allowable shear force when shear alone exists

C3.4 Web Crippling Strength

These provisions are applicable to webs of flexural members subject to concentrated loads or reactions, or the components thereof, acting perpendicular to the longitudinal axis of the member, acting in the plane of the web under consideration, and causing compressive stresses in the web.

To avoid crippling of unreinforced flat webs of flexural members having a flat width ratio, h/t, equal to or less than 200, concentrated loads and reactions shall not exceed the values of P_a given in Table C3.4-1. Webs of flexural members for which h/t is greater than 200 shall be provided with adequate means of transmitting concentrated loads and/or reactions directly into the webs.

The formulas in Table C3.4-1 apply to beams when $R/t \leq 6$ and to deck when $R/t \leq 7$, $N/t \leq 210$ and $N/h \leq 3.5$.

P_a represents the concentrated load or reaction for one solid web connecting top and bottom flanges. For two or more webs, P_a shall be computed for each individual web and the results added to obtain the allowable load or reaction for the multiple web.

For built-up I-sections, or similar sections, the distance between the web connector and beam flange shall be kept as small as practical.

TABLE C3.4-1
P_a

		Shapes Having Single Webs		Shapes Having Multiple Webs(1)
		Stiffened Flanges	Unstiffened Flanges	Stiffened and Unstiffened Flanges
Opposing Loads Spaced $> 1.5h_{(2)}$	End Reaction(3)	*Eq.* C3.4-1	*Eq.* C3.4-2	*Eq.* C3.4-3
	Interior Reaction(4)	*Eq.* C3.4-4	*Eq.* C3.4-4	*Eq.* C3.4-5
Opposing Loads Spaced $\leq 1.5h_{(5)}$	End Reaction(3)	*Eq.* C3.4-6	*Eq.* C3.4-6	*Eq.* C3.4-7
	Interior Reaction(4)	*Eq.* C3.4-8	*Eq.* C3.4-8	*Eq.* C3.4-9

Footnotes and Equation References to Table C3.4-1:

(1) I-sections made of two channels 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 channel).

Footnotes to Table C3.4-1 continued:

(2) At locations of one concentrated load or reaction acting either on the top or bottom flange, when the clear distance between the bearing edges of this and adjacent opposite concentrated loads or reactions is greater than 1.5h.

(3) For end reactions of beams or concentrated loads on the end of cantilevers when the distance from the edge of the bearing to the end of the beam is less than 1.5h.

(4) For reactions and concentrated loads when the distance from the edge of bearing to the end of the beam is equal to or greater than 1.5h.

(5) At locations of two opposite concentrated loads or of a concentrated load and an opposite reaction acting simultaneously on the top and bottom flanges, when the clear distance between their adjacent bearing edges is equal to or less than 1.5h.

Equations for Table C3.4-1:

$$t^2kC_3C_4C_\theta[179-0.33(h/t)][1+0.01(N/t)] \quad (Eq.\ C3.4\text{-}1)$$

$$t^2kC_3C_4C_\theta[117-0.15(h/t)][1+0.01(N/t)] \quad (Eq.\ C3.4\text{-}2)$$

When $N/t > 60$, the factor $[1+0.01(N/t)]$ may be increased to $[0.71+0.015(N/t)]$

$$t^2F_yC_6(5.0+0.63\sqrt{N/t}) \quad (Eq.\ C3.4\text{-}3)$$

$$t^2kC_1C_2C_\theta[291-0.40(h/t)][1+0.007(N/t)] \quad (Eq.\ C3.4\text{-}4)$$

When $N/t > 60$, the factor $[1+0.007(N/t)]$ may be increased to $[0.75+0.011(N/t)]$

$$t^2F_yC_5(0.88+0.12m)(7.50+1.63\sqrt{N/t}) \quad (Eq.\ C3.4\text{-}5)$$

$$t^2kC_3C_4C_\theta[132-0.31(h/t)][1+0.01(N/t)] \quad (Eq.\ C3.4\text{-}6)$$

$$t^2F_yC_8(0.64+0.31m)(5.0+0.63\sqrt{N/t}) \quad (Eq.\ C3.4\text{-}7)$$

$$t^2kC_1C_2C_\theta[417-1.22(h/t)][1+0.0013(N/t)] \quad (Eq.\ C3.4\text{-}8)$$

$$t^2F_yC_7(0.82+0.15m)(7.50+1.63\sqrt{N/t}) \quad (Eq.\ C3.4\text{-}9)$$

In the above-referenced formulas,

P_a = Allowable concentrated load or reaction per web, kips

$C_1 = (1.22-0.22k)$ (*Eq.* C3.4-10)

$C_2 = (1.06-0.06R/t) \leq 1.0$ (*Eq.* C3.4-11)

$C_3 = (1.33-0.33k)$ (*Eq.* C3.4-12)

$C_4 = 0.50 < (1.15-0.15R/t) \leq 1.0$ (*Eq.* C3.4-13)

$C_5 = (1.49-0.53k) \geq 0.6$ (*Eq.* C3.4-14)

$$C_6 = 1+\left(\frac{h/t}{750}\right) \text{ when } h/t \leq 150 \quad (Eq.\ C3.4\text{-}15)$$

$= 1.20$, when $h/t > 150$ (*Eq.* C3.4-16)

$C_7 = 1/k$, when $h/t \leq 66.5$ (*Eq.* C3.4-17)

$$= \left[1.10-\frac{h/t}{665}\right]\frac{1}{k}, \text{ when } h/t > 66.5 \quad (Eq.\ C3.4\text{-}18)$$

$$C_8 = \left[0.98-\frac{h/t}{865}\right]\frac{1}{k} \quad (Eq.\ C3.4\text{-}19)$$

$C_\theta = 0.7+0.3(\theta/90)^2$ (*Eq.* C3.4-20)

F_y = Design yield stress of the web, ksi

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

$k = F_y/33$ (*Eq.* C3.4-21)

$m = t/0.075$ (*Eq.* C3.4-22)

t = Web thickness, inches

N = Actual length of bearing, inches. For the case of two equal and opposite concentrated loads distributed over unequal bearing lengths, the smaller value of N shall be taken

R = Inside bend radius

θ = Angle between the plane of the web and the plane of the bearing surface $\geq 45°$, but not more than 90°

C3.5 Combined Bending and Web Crippling Strength

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

For shapes having single unreinforced webs:

$$1.2(P/P_a) + (M/M_a) \leq 1.5 \qquad (Eq.\ C3.5\text{-}1)$$

Exception: At the interior supports of continuous spans, the above formula 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 inches.

For shapes having multiple unreinforced webs such as I-sections made of two channels 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 channel);

$$1.1(P/P_a) + (M/M_a) \leq 1.5 \qquad (Eq.\ C3.5\text{-}2)$$

Exception: When $h/t \leq 2.33/\sqrt{(F_y/E)}$ and $\lambda \leq 0.673$, the allowable concentrated load or reaction may be determined by Section C3.4.

In the above formulas,

P = Concentrated load or reaction in the presence of bending moment
P_a = Allowable concentrated load or reaction in the absence of bending moment determined in accordance with Section C3.4
M = Applied bending moment at, or immediately adjacent to, the point of application of the concentrated load or reaction
M_a = Allowable bending moment if bending alone exists
w = Flat width of the beam flange which contacts the bearing plate
t = Thickness of the web or flange
λ = Slenderness factor given by Section B2.1

C4 Concentrically Loaded Compression Members

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

(a) The axial load shall not exceed P_a calculated as follows:

$$P_a = P_n/\Omega_c \qquad (Eq.\ C4\text{-}1)$$

where

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

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

F_n is determined as follows:

For $F_e > F_y/2$ $\qquad F_n = F_y(1 - F_y/4F_e)$ $\qquad (Eq.\ C4\text{-}3)$

For $F_e \leq F_y/2$ $\qquad F_n = F_e$ $\qquad (Eq.\ C4\text{-}4)$

F_e is the least of the elastic flexural, torsional and torsional-flexural buckling stress determined according to Sections C4.1 through C4.3.

Ω_c = Factor of safety for axial compression
= 1.92, except when F_e is determined according to Section C4.1 for fully effective sections having wall thicknesses greater than 0.09 inches and $F_e > F_y/2$. In this case,

$$\Omega_c = \frac{5}{3} + \frac{3}{8}R - \frac{1}{8}R^3$$

where
$R = \sqrt{(F_y/2F_e)}$

(b) For C- and Z-shapes, and single-angle sections with unstiffened flanges, P_n shall be taken as the smaller of P_n calculated above and P_n calculated as follows:

$$P_n = \frac{A\pi^2 E}{25.7(w/t)^2} \qquad (Eq.\ C4\text{-}5)$$

where
A = Area of the full, unreduced cross section
w = Flat width of the unstiffened element
t = Thickness of the unstiffened element

(c) Angle sections shall be designed for the applied axial load, P, acting simultaneously with a moment equal to PL/1000 applied about the minor principal axis causing compression in the tips of the angle legs.

(d) The slenderness ratio, KL/r, of all compression members preferably should not exceed 200, except that during construction only, KL/r preferably should not exceed 300.

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

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

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

where
E = Modulus of elasticity
K = Effective length factor*
L = Unbraced length of member
r = Radius of gyration of the full, unreduced cross section

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

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

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

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

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

where σ_t and σ_{ex} are as defined in C3.1.2(b)

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

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

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

C4.3 Nonsymmetric Sections

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

C5 Combined Axial Load and Bending

The axial force and bending moments shall satisfy the following interaction equations:

$$\frac{P}{P_a} + \frac{C_{mx} M_x}{M_{ax} \alpha_x} + \frac{C_{my} M_y}{M_{ay} \alpha_y} \leq 1.0 \qquad (Eq.\ C5\text{-}1)$$

$$\frac{P}{P_{ao}} + \frac{M_x}{M_{axo}} + \frac{M_y}{M_{ayo}} \leq 1.0 \qquad (Eq.\ C5\text{-}2)$$

When $P/P_a \leq 0.15$, the following formula may be used in lieu of the above two formulas:

$$\frac{P}{P_a} + \frac{M_x}{M_{ax}} + \frac{M_y}{M_{ay}} \leq 1.0 \qquad (Eq.\ C5\text{-}3)$$

where

P = Applied axial load

M_x and M_y = Applied moments with respect to the centroidal axes of the effective section determined for the axial load alone. For angle sections, M_y shall be taken either as the applied moment or the applied moment plus PL/1000, whichever results in a lower value of P_a.

P_a = Allowable axial load determined in accordance with Section C4

P_{ao} = Allowable axial load determined in accordance with Section C4, with $F_n = F_y$

M_{ax} and M_{ay} = Allowable moments about the centroidal axes determined in accordance with Section C3

M_{axo} and M_{ayo} = Allowable moments about the centroidal axes determined in accordance with Section C3.1, excluding the provisions of Section C3.12 (lateral buckling)

$1/\alpha_x$, $1/\alpha_y$ = Magnification factors

$= 1/[1 - (\Omega_c P/P_{cr})]$ (Eq. C5-4)

Ω_c = Factor of safety used in determining P_a

$$P_{cr} = \frac{\pi^2 E I_b}{(K_b L_b)^2} \qquad (Eq.\ C5\text{-}5)$$

I_b = Moment of inertia of the full, unreduced cross section about the axis of bending

L_b = Actual unbraced length in the plane of bending

K_b = Effective length factor in the plane of bending

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

1. For compression members in frames subject to joint translation (sidesway)

   $C_m = 0.85$

2. For restrained compression members in frames braced against joint translation and not subject to transverse loading between their supports in the plane of bending

   $C_m = 0.6 - 0.4(M_1/M_2)$ (Eq. C5-6)

   where

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

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

C6 Cylindrical Tubular Members

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

C6.1 Bending

For flexural members, the actual moment uncoupled from axial load, shear, and local concentrated forces or reactions shall not exceed M_a calculated as follows:

$$M_a = M_n/\Omega_f \qquad (Eq.\ C6.1\text{-}1)$$

where
M_n = Nominal moment
Ω_f = Factor of safety for bending
= 1.67

For $D/t \le 0.070\,E/F_y$

$$M_n = 1.25\,F_yS_f \qquad (Eq.\ C6.1\text{-}2)$$

For $0.070\,E/F_y < D/t \le 0.319\,E/F_y$

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

For $0.319\,E/F_y < D/t \le 0.441\,E/F_y$

$$M_n = [0.328E/(D/t)]S_f \qquad (Eq.\ C6.1\text{-}4)$$

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

C6.2 Compression

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

The axial load shall not exceed P_a calculated as follows:

$$P_a = P_n/\Omega_c \qquad (Eq.\ C6.2\text{-}1)$$

where

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

For F_e greater than $F_y/2$
F_n = Flexural buckling stress
$= F_y[1 - F_y/4F_e]$ (Eq. C6.2-3)
F_e = The elastic flexural buckling stress determined according to Section C4.1
Ω_c = Factor of safety for axial compression

$$= \frac{5}{3} + \frac{3}{8}R - \frac{1}{8}R^3 \qquad (Eq.\ C6.2\text{-}4)$$

$$R = \sqrt{F_y/2F_e} \qquad (Eq.\ C6.2\text{-}5)$$

$$A_e = [1 - (1 - R^2)(1 - A_o/A)]\,A \qquad (Eq.\ C6.2\text{-}6)$$

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

A = Area of the unreduced cross section

For $F_e \leq F_y/2$

$F_n = F_e$

Ω_c = Factor of safety for axial compression

= 1.92

$A_e = A$

C6.3 Combined Bending and Compression

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

D. STRUCTURAL ASSEMBLIES

D1 Built-Up Sections

D1.1 I-Sections Composed of Two Channels

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

(a) For compression members:

$$s_{max} = \frac{L\, r_{cy}}{2\, r_I} \quad (Eq.\ D1.1\text{-}1)$$

where

L = Unbraced length of compression member

r_I = Radius of gyration of the I-section about the axis perpendicular to the direction in which buckling would occur for the given conditions of end support and intermediate bracing

r_{cy} = Radius of gyration of one channel about its centroidal axis parallel to the web

(b) For flexural members:

$$s_{max} = L/6 \quad (Eq.\ D1.1\text{-}2)$$

In no case shall the spacing exceed the value

$$s_{max} = \frac{2g\, T_s}{mq} \quad (Eq.\ D1.1\text{-}3)$$

where

L = Span of beam

T_s = Strength of connection in tension

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

q = Intensity of load on the beam (For methods of determination, see below)

m = Distance from the shear center of one channel to the mid-plane of its web. For simple channels without stiffening lips at the outer edges,

$$m = \frac{w_f^2}{2w_f + d/3} \quad (Eq.\ D1.1\text{-}4)$$

For channels with stiffening lips at the outer edges,

$$m_f = \frac{w_f dt}{4\, I_x}\left[w_f d + 2D\left(d - \frac{4D^2}{3d} \right) \right] \quad (Eq.\ D1.1\text{-}5)$$

w_f = Projection of flanges from the inside face of the web (For channels with flanges of unequal width, w_f shall be taken as the width of the wider flange)
d = Depth of channel or beam
D = Overall depth of lip
I_x = Moment of inertia of one channel about its centroidal axis normal to the web.

The intensity of load, q, is obtained by dividing the magnitude of concentrated loads or reactions by the length of bearing. For beams designed for a uniformly distributed load, q shall be taken equal to three times the intensity of the uniformly distributed design load. If the length of bearing of a concentrated load or reaction is smaller than the weld spacing, s, the required strength of the welds or connections closest to the load or reaction is

$$T_s = Pm/2g \qquad (Eq.\ D1.1\text{-}6)$$

The required maximum spacing of connections, s_{max}, depends upon the intensity of the load directly at the connection. Therefore, if uniform spacing of connections is used over the whole length of the beam, it shall be determined at the point of maximum local load intensity. In cases where this procedure would result in uneconomically close spacing, either one of the following methods may be adopted: (a) the connection spacing may be varied along the beam according to the variation of the load intensity; or (b) reinforcing cover plates may be welded to the flanges at points where concentrated loads occur. The shear strength of the connections joining these plates to the flanges shall then be used for T_s, and g shall be taken as the depth of the beam.

D1.2 Spacing of Connections in Compression Elements

The spacing, s, in the line of stress, of welds, rivets, or bolts connecting a compression cover plate or sheet to a non-integral stiffener or other element shall not exceed

(a) that which is required to transmit the shear between the connected parts on the basis of the design strength per connection specified elsewhere herein; nor
(b) $1.16t\sqrt{(E/f_c)}$, where t is the thickness of the cover plate or sheet, and f_c is the stress at design load in the cover plate or sheet; nor
(c) three times the flat width, w, of the narrowest unstiffened compression element tributary to the connections, but need not be less than $1.11t\sqrt{(E/F_y)}$ if $w/t < 0.50\sqrt{(E/F_y)}$, or $1.33t\sqrt{(E/f_c)}$ if $w/t \geq 0.50\sqrt{(E/F_y)}$, unless closer spacing is required by (a) or (b) above.

In the case of intermittent fillet welds parallel to the direction of stress, the spacing shall be taken as the clear distance between welds, plus one-half inch. In all other cases, the spacing shall be taken as the center-to-center distance between connections.

Exception: The requirements of this Section do not apply to cover sheets which act only as sheathing material and are not considered as load-carrying elements.

D2 Mixed Systems

The design of members in mixed systems using cold-formed steel components in conjunction with other materials shall conform to this Specification and the applicable Specification of the other material.

D3 Lateral Bracing

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

D3.1 Symmetrical Beams and Columns

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

D3.2 Channel-Section and Z-Section Beams

The following provisions for bracing to restrain twisting of channels and Z-sections used as beams loaded in the plane of the web, apply only when (a) the top flange is connected

to deck or sheathing material in such a manner as to effectively restrain lateral deflection of the connected flange* or (b) neither flange is so connected. When both flanges are so connected, no further bracing is required.

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

For channels and Z-sections designed according to Section C3.1.1, and having deck or sheathing fastened directly to the top flanges in such a manner shown to effectively inhibit relative movement between the deck or sheathing and the purlin flange, provisions shall be made to restrain the flanges so that the maximum top flange lateral displacements with respect to the purlin reaction points do not exceed the span length divided by 360. If the top flanges of all purlins face in the same direction, anchorage of the restraint system must be capable of satisfying the requirements of Sections D3.2.1(a) and D3.2.1(b). If the top flanges of adjacent lines of purlins face in opposite directions, the provisions of Section D3.2.1(a) and D3.2.1(b) do not apply.

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

For bracing arrangements other than those covered in Sections D3.2.1(a) and D3.2.1(b), tests in accordance with Chapter F shall be performed so that the type and/or spacing of braces selected are such that the test strength of the braced Z-section assembly is equal to or greater than 5/3 times its flexural design strength, instead of that required by Chapter F.

(a) Channel Sections
For roof systems using channel sections for purlins with all compression flanges facing in the same direction, a restraint system capable of resisting 0.05W, in addition to other loading, shall be provided where W is the load supported by all purlin lines being restrained. Where more than one brace is used at a purlin line, the restraint force 0.05W shall be divided equally between all braces.

(b) Z-Sections
For roof systems having a diaphragm stiffness of at least 2,000 lb/in., having four to twenty Z-purlin lines with all top flanges facing in the direction of the upward roof slope, and with restraint braces at the purlin supports, midspan or one-third points, each brace shall be designed to resist a force determined as follows:

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

$$P_L = 0.5\left[\frac{0.220\ b^{1.50}}{n_p^{0.72} d^{0.90} t^{0.60}} - \tan\theta\right]W \quad (Eq.\ D3.2.1\text{-}1)$$

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

$$P_L = 0.5\left[\frac{0.474\ b^{1.22}}{n_p^{0.57} d^{0.89} t^{0.33}} - \tan\theta\right]W \quad (Eq.\ D3.2.1\text{-}2)$$

(3) Single-Span System with Midspan Restraint:

$$P_L = \left[\frac{0.224\ b^{1.32}}{n_p^{0.65} d^{0.83} t^{0.50}} - \tan\theta\right]W \quad (Eq.\ D3.2.1\text{-}3)$$

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

$$P_L = C_{tr}\left[\frac{0.053\ b^{1.88} L^{0.13}}{n_p^{0.95} d^{1.07} t^{0.94}} - \tan\theta\right]W \quad (Eq.\ D3.2.1\text{-}4)$$

*Where the Specification does not provide an explicit method for design, further information should be obtained from the Commentary.

with
C_{tr} = 0.63 for braces at end supports of multiple-span systems
C_{tr} = 0.87 for braces at the first interior supports
C_{tr} = 0.81 for all other braces

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

$$P_L = C_{th}\left[\frac{0.181\ b^{1.15}L^{0.25}}{n_p^{0.54}d^{1.11}t^{0.29}} - \tan\theta\right]W \qquad (Eq.\ D3.2.1\text{-}5)$$

with
C_{th} = 0.57 for outer braces in exterior spans
C_{th} = 0.48 for all other braces

(6) Multiple-Span System with Midspan Restraints:

$$P_L = C_{ms}\left[\frac{0.116\ b^{1.32}L^{0.18}}{n_p^{0.70}d^{1.00}t^{0.50}} - \tan\theta\right]W \qquad (Eq.\ D3.2.1\text{-}6)$$

with
C_{ms} = 1.05 for braces in exterior spans
C_{ms} = 0.90 for all other braces

where
b = Flange width, in.
d = Depth of section, in.
t = Thickness, in.
L = Span length, in.
θ = Angle between the vertical and the plane of the web of the Z-section, degrees
n_p = Number of parallel purlin lines
W = Total load supported by the purlin lines between adjacent supports, pounds

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

For systems having less than four purlin lines, the brace force can be determined by taking 1.1 times the force found from Equations D3.2.1-1 through D3.2.1-6, with $n_p = 4$. For systems having more than twenty purlin lines, the brace force can be determined from Equations D3.2.1-1 through D3.2.1-6, with $n_p = 20$.

D3.2.2 Neither Flange Connected to Sheathing

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

(a) For uniform loads, $P_L = 1.5K'$ times the load within a distance 0.5a each side of the brace.

(b) For concentrated loads, $P_L = 1.0K'$ times each concentrated load within a distance 0.3a each side of the brace, plus $1.4K'[1-(x/a)]$ times each concentrated load located farther than 0.3a but not farther than 1.0a from the brace.

In the above formulas:
For channels and Z-sections:
x = Distance from the concentrated load to the brace
a = Distance between center line of braces

For channels:

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

where
m = Distance from the shear center to the mid-plane of the web, as specified in Section D1.1
d = Depth of channel

For Z-sections:

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

where

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

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

Braces shall be designed to avoid local crippling at the points of attachment to the member.

Braces shall be attached both to the top and bottom flanges of the sections, at the ends and at intervals not greater than one-quarter of the span length, in such a manner as to prevent tipping at the ends and lateral deflection of either flange in either direction at intermediate braces. If one-third or more of the total load on the beam is concentrated over a length of one-twelfth or less of the span of the beam, an additional brace shall be placed at or near the center of this loaded length.

Exception: 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 rotation and lateral displacement, no other braces will be required.

D3.3 Laterally Unbraced Box Beams

For closed box-type sections used as beams subject to bending about the major axis, the ratio of the laterally unsupported length to the distance between the webs of the section shall not exceed $0.086E/F_y$.

D4 Wall Studs and Wall Stud Assemblies

The safe load-carrying capacity of a stud may be computed on the basis that sheathing (attached to one or both sides of the stud) furnishes adequate lateral and rotational support to the stud in the plane of the wall, provided that the stud, sheathing, and attachments comply with the following requirements:

Both ends of the stud shall be braced to restrain rotation about the stud axis and horizontal displacement perpendicular to the stud axis; however, the ends may or may not be free to rotate about both axes perpendicular to the stud axis. The sheathing shall be connected to the top and bottom members of the wall assembly to enhance the restraint provided to the stud and stabilize the overall assembly.

D4.1 Wall Studs in Compression

For studs having identical sheathing attached to both flanges, and neglecting any rotational restraint provided by the sheathing,* the applied axial load, P, shall not exceed P_a calculated as follows:

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

where

A_e = Effective area determined at F_n

Ω_c = Factor of safety for axial compression, i.e., in accordance with Section C4(a) when either Sections D4.1(a) or D4.1(b) govern or 1.92 when Section D4.1(c) governs.

F_n = The lowest value determined by the following three conditions:

(a) To prevent column buckling between fasteners in the plane of the wall, F_n shall be calculated according to Section C4 with KL equal to two times the distance between fasteners.

(b) To prevent flexural and/or torsional overall column buckling, F_n shall be calculated in accordance with Section C4 with F_e taken as the smaller of the two σ_{CR} values specified for the following section types, where σ_{CR} is the theoretical elastic buckling stress under concentric loading.

*Studs with sheathing on one flange only, or with unidentical sheathing on both flanges, or having rotational restraint that is not neglected, or having any combination of the above, shall be designed in accordance with the same basic analysis principles used in deriving the provisions of this Section.

(1) Singly-symmetric channels and C-Sections

$$\sigma_{CR} = \sigma_{ey} + \overline{Q}_a \qquad (Eq.\ D4.1\text{-}2)$$

$$\sigma_{CR} = \frac{1}{2\beta}\left[(\sigma_{ex} + \sigma_{tQ}) - \sqrt{(\sigma_{ex} + \sigma_{tQ})^2 - (4\beta\sigma_{ex}\sigma_{tQ})}\right] \qquad (Eq.\ D4.1\text{-}3)$$

(2) Z-Sections

$$\sigma_{CR} = \sigma_t + \overline{Q}_t \qquad (Eq.\ D4.1\text{-}4)$$

$$\sigma_{CR} = \frac{1}{2}\{(\sigma_{ex} + \sigma_{ey} + \overline{Q}_a) - [(\sigma_{ex} + \sigma_{ey} + \overline{Q}_a)^2 - 4(\sigma_{ex}\sigma_{ey} + \sigma_{ex}\overline{Q}_a - \sigma_{exy}^2)]^{1/2}\} \qquad (Eq.\ D4.1\text{-}5)$$

(3) I-Sections (doubly-symmetric)

$$\sigma_{CR} = \sigma_{ey} + \overline{Q}_a \qquad (Eq.\ D4.1\text{-}6)$$

$$\sigma_{CR} = \sigma_{ex} \qquad (Eq.\ D4.1\text{-}7)$$

In the above formulas

$$\sigma_{ex} = \frac{\pi^2 E}{(K_x L_x / r_x)^2} \qquad (Eq.\ D4.1\text{-}8)$$

$$\sigma_{exy} = (\pi^2 E I_{xy})/(AL^2) \qquad (Eq.\ D4.1\text{-}9)$$

$$\sigma_{ey} = \frac{\pi^2 E}{(K_y L_y / r_y)^2} \qquad (Eq.\ D4.1\text{-}10)$$

$$\sigma_t = \frac{1}{Ar_o^2}\left[GJ + \frac{\pi^2 E C_w}{(K_t L_t)^2}\right] \qquad (Eq.\ D4.1\text{-}11)$$

$\sigma_{tQ} = \sigma_t + \overline{Q}_t$ (*Eq.* D4.1-12)
$\overline{Q}$ = $\overline{q}B$ = Design shear rigidity for sheathing on both sides of the wall assembly (*Eq.* D4.1-12)
$\overline{q}$ = Design shear rigidity for sheathing per inch of stud spacing (see Table D4)
B = Stud spacing
$\overline{Q}_a$ = $\overline{Q}/A$ (*Eq.* D4.1-13)
A = Area of full unreduced cross section
L = Length of stud
$\overline{Q}_t$ = $(\overline{Q}d^2)/(4Ar_o^2)$ (*Eq.* D4.1-14)
d = Depth of section
I_{xy} = Product of inertia

(c) To prevent shear failure of the sheathing, a value of F_n shall be used in the following equations so that the shear strain of the sheathing, γ, does not exceed the permissible shear strain, $\overline{\gamma}$. The shear strain, γ, shall be determined as follows:

$$\gamma = (\pi/L)[C_1 + (E_1 d/2)] \qquad (Eq.\ D4.1\text{-}15)$$

where

C_1 and E_1 are the absolute values of C_1 and E_1 specified below for each section type:

(1) Singly-Symmetric Channels

$$C_1 = (F_n C_o)/(\sigma_{ey} - F_n + \overline{Q}_a) \qquad (Eq.\ D4.1\text{-}16)$$

$$E_1 = \frac{F_n[(\sigma_{ex} - F_n)(r_o^2 E_o - x_o D_o) - F_n x_o (D_o - x_o E_o)]}{(\sigma_{ex} - F_n) r_o^2 (\sigma_{tQ} - F_n) - (F_n x_o)^2} \qquad (Eq.\ D4.1\text{-}17)$$

(2) Z-Sections

$$C_1 = \frac{F_n[C_o(\sigma_{ex} - F_n) - D_o\sigma_{exy}]}{(\sigma_{ey} - F_n + \overline{Q}_a)(\sigma_{ex} - F_n) - \sigma_{exy}^2} \qquad (Eq.\ D4.1\text{-}18)$$

$E_1 = (F_n E_o)/(\sigma_{tQ} - F_n)$ (*Eq*. D4.1-19)

(3) I-Sections

$C_1 = (F_n C_o)/(\sigma_{ey} - F_n + \overline{Q}_a)$ (*Eq*. D4.1-20)
$E_1 = 0$

where
x_o = distance from shear center to centroid along principal x-axis, in. (absolute value)
C_o, E_o, and D_o are initial column imperfections which shall be assumed to be at least
C_o = L/350 in a direction parallel to the wall (*Eq*. D4.1-21)
D_o = L/700 in a direction perpendicular to the wall (*Eq*. D4.1-22)
E_o = L/(d × 10,000), rad., a measure of the initial twist of the stud from the initial, ideal, unbuckled shape. (*Eq*. D4.1-23)

If $F_n > 0.5F_y$, then in the definitions for σ_{ey}, σ_{ex}, σ_{exy} and σ_{tQ}, the parameters E and G shall be replaced by E′ and G′, respectively, as defined below

$E' = 4EF_n(F_y - F_n)/F_y^2$ (*Eq*. D4.1-24)
$G' = G(E'/E)$ (*Eq*. D4.1-25)

Sheathing parameters $\overline{q}_o$ and $\overline{\gamma}$ may be determined from representative full-scale tests, conducted and evaluated as described by published documented methods (see Commentary), or from the small-scale-test values given in Table D4.

D4.2 Wall Studs in Bending

For studs having identical sheathing attached to both flanges, and neglecting any rotational restraint provided by the sheathing,* the allowable moments are M_{axo} and M_{ayo}

where
M_{axo} and M_{ayo} = Allowable moments about the centroidal axes determined in accordance with Section C3.1, excluding the provisions of Section C3.1.2 (lateral buckling)

D4.3 Wall Studs with Combined Axial Load and Bending

The axial load and bending moments shall satisfy the interaction equations of Section C5 with the following redefined terms:
P_a = Allowable axial load determined according to Section D4.1
M_{ax} and M_{ay} in Equations C5-1 and C5-3 shall be replaced by allowable moments, M_{axo} and M_{ayo}, respectively.

TABLE D4
Sheathing Parameters(1)

Sheathing(2)	$\overline{q}_o$(3) k/in.	$\overline{\gamma}$ in./in.
3/8 to 5/8 in. thick gypsum	2.0	0.008
Lignocellulosic board	1.0	0.009
Fiberboard (regular or impregnated)	0.6	0.007
Fiberboard (heavy impregnated)	1.2	0.010

(1) The values given are subject to the following limitations:
All values are for sheathing on both sides of the wall assembly.

*Studs with sheathing on one flange only, or with unidentical sheathing on both flanges, or having rotational restraint that is not neglected, or having any combination of the above, shall be designed in accordance with the same basic analysis principles used in deriving the provisions of this Section.

All fasteners are No.6, type S-12, self-drilling drywall screws with pan or bugle head, or equivalent, at 6- to 12-inch spacing.

(2) All sheating is 1/2-inch thick except as noted.

(3) $\bar{q} = \bar{q}_o (2 - s/12)$ (*Eq.* D4.1-26)

where s = fastener spacing, in.

For other types of sheathing, $\bar{q}_o$ and $\bar{\gamma}$ may be determined conservatively from representative small-specimen tests as described by published documented methods (see Commentary).

E. CONNECTIONS AND JOINTS

E1 General Provisions

Connections shall be designed to transmit the maximum load in the connected member. Proper regard shall be given to eccentricity.

E2 Welded Connections

Arc welds on steel where each connected part is over 0.18 inch in thickness shall be made in accordance with AISC Specification (Reference 3 of Section A6).

Except as modified herein, arc welds on steel where at least one of the connected parts is 0.18 inch or less in thickness shall be made in accordance with the AWS D-1.3 (Reference 4 of Section A6) and its Commentary. Welders and welding procedures shall be qualified as specified in AWS D1.3. These provisions are intended to cover the welding positions as shown in Table E2.

The load on each weld shall not exceed P_a, calculated as follows:

$$P_a = P_n/\Omega_w \quad (Eq.\ E2\text{-}1)$$

where

Ω_w = Factor of safety for arc welded connections
= 2.50

P_n = Nominal strength of welds determined according to Sections E2.1 through E2.5.

Allowable loads, P_a, for resistance welds made in conformance with the procedures given in AWS C1.1-66, "Recommended Practices for Resistance Welding" or AWS C1.3-70, "Recommended Practice for Resistance Welding Coated Low Carbon Steels" are given in Section E2.6.

TABLE E2

Connection	Welding Position					
	Square Groove Butt Weld	Arc Spot Weld	Arc Seam Weld	Fillet Weld, Lap or T	Flare-Bevel Groove	Flare-V Groove Weld
Sheet to Sheet	F H V OH	— — — —	F H — —	F H V OH	F H V OH	F H V OH
Sheet to Supporting Member	— — — —	F — — —	F — — —	F H V OH	F H V OH	— — — —

(F = flat, H = horizontal, V = vertical, OH = overhead)

E2.1 Groove Welds in Butt Joints

The maximum load for a groove weld in a butt joint, welded from one or both sides, shall be determined on the basis of the lower strength base steel in the connection, provided that an effective throat equal to or greater than the thickness of the material is consistently obtained.

E2.2 Arc Spot Welds

Arc spot welds permitted by this Specification are for welding sheet steel to thicker supporting members in the flat position. Arc spot welds (puddle welds) shall not be made on steel where the thinnest connected part is over 0.15 inch thick, nor through a combination of steel sheets having a total thickness over 0.15 inch.

Weld washers, Figures E2.2(A) and E2.2(B), shall be used when the thickness of the sheet is less than 0.028 inch. Weld washers shall have a thickness between 0.05 and 0.08 inch with a minimum prepunched hole of ⅜-inch diameter.

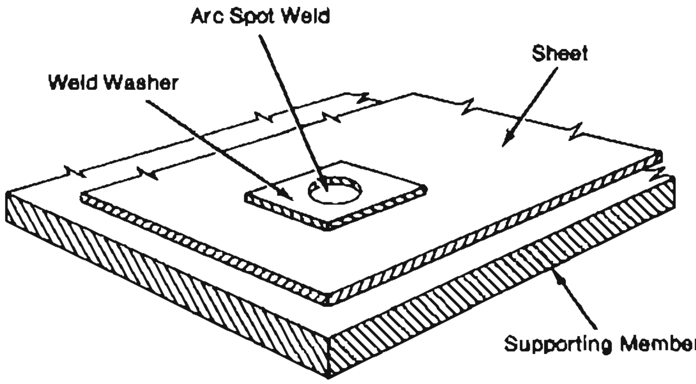


Figure E2.2A Typical Weld Washer

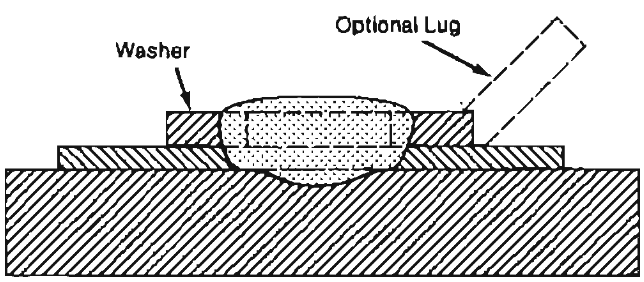


Figure E2.2B Arc Spot Weld Using Washer

Arc spot welds shall be specified by minimum effective diameter of fused area, d_e. Minimum allowable effective diameter is ⅜ inch.

The nominal shear load, P_n, on each arc spot weld between sheet or sheets and supporting member shall not exceed the smaller of either

$$P_n = 0.625\, d_e^2 F_{xx}; \text{ or} \qquad (Eq.\ E2.2\text{-}1)$$

For $(d_a/t) \le 0.815\sqrt{(E/F_u)}$:

$$P_n = 2.20\, t\, d_a F_u; \qquad (Eq.\ E2.2\text{-}2)$$

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

$$P_n = 0.280\left[1 + \frac{5.59t\sqrt{E}}{d_a\sqrt{F_u}}\right] t\, d_a F_u; \qquad (Eq.\ E2.2\text{-}3)$$

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

$$P_n = 1.40\, t\, d_a\, F_u \qquad (Eq.\ E2.2\text{-}4)$$

where

d = Visible diameter of outer surface of arc spot weld

d_a = Average diameter of the arc spot weld at mid-thickness of t [where $d_a = (d - t)$ for a single sheet, and $(d - 2t)$ for multiple sheets (not more than four lapped sheets over a supporting member)]

d_e = Effective diameter of fused area

$d_e = 0.7d - 1.5t$ but $\leq 0.55d$ (*Eq.* E2.2-5)

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

F_{xx} = Stress level designation in AWS electrode classification

F_{sy} = Yield point as specified in Sections A3.1 or A3.2

F_u = Tensile strength as specified in Sections A3.1 or A3.2 or as reduced for low-ductility steel.

Note: See Figures E2.2(C) and E2.2(D) for diameter definitions

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

$$e_{min} = e\Omega_e \qquad (Eq.\ E2.2\text{-}6)$$

where

$$e = \frac{P}{F_u t} \qquad (Eq.\ E2.2\text{-}7)$$

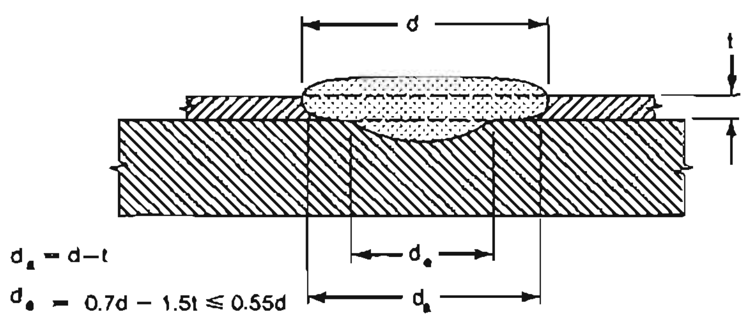
Ω_e = Factor of safety for sheet tearing

= 2.0 when $F_u/F_{sy} \geq 1.15$

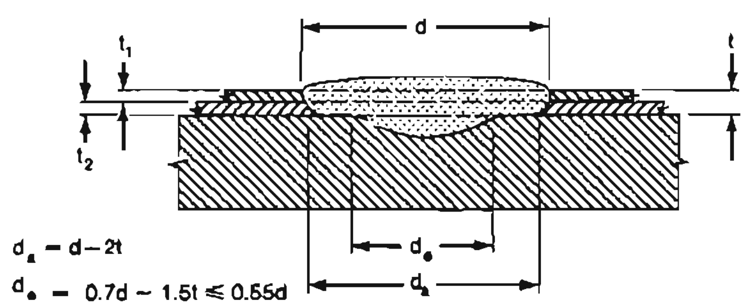
= 2.22 when $F_u/F_{sy} < 1.15$

P = Force transmitted by weld

t = Thickness of thinnest connected sheet



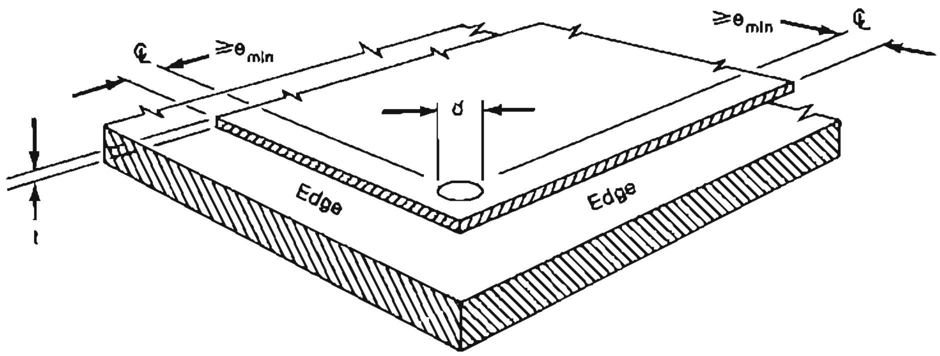
(C) Arc Spot Weld—Single Thickness of Sheet



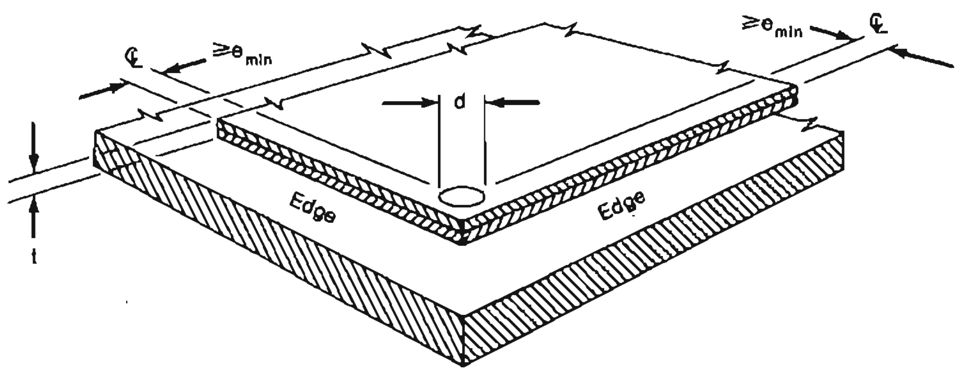
(D) Arc Spot Weld—Double Thickness of Sheet

Figure E2.2 C, D Arc Spot Welds

Note: See Figures E2.2(E) and E2.2(F) for edge distances of arc welds.
In addition, the distance from the centerline of any weld to the end or boundary of the connected member shall not be less than 1.5d. In no case shall the clear distance between welds and the end of member be less than 1.0d.



(E) Single Sheet



(F) Double Sheet

Figure E2.2 E, F Edge Distances for Arc Spot Welds

If it can be shown by measurement that a given weld procedure will consistently give a larger effective diameter, d_e, or average diameter, d_a, as applicable, this larger diameter may be used providing the particular welding procedure used for making those welds is followed.

E2.3 Arc Seam Welds

Arc seam welds [Figure E2.3(A)] covered by this Specification apply only to the following joints:

(a) Sheet to thicker supporting member in the flat position.

(b) Sheet to sheet in the horizontal or flat position.

The shear load, P_n, on each arc seam weld shall not exceed either

$$P_n = \left[\frac{d_e^2}{4} + \frac{Ld_e}{3}\right] 2.5\,F_{xx}\text{ ; or} \qquad (Eq.\ E2.3\text{-}1)$$

$$P_n = 2.5tF_u(0.25L + 0.96d_a) \qquad (Eq.\ E2.3\text{-}2)$$

where

d = width of arc seam weld

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

d_a = Average width of seam weld
where
$d_a = (d - t)$ for a single sheet, and (*Eq.* E2.3-3)
$(d - 2t)$ for a double sheet (*Eq.* E2.3-4)
d_e = Effective width of arc seam weld at fused surfaces
$d_e = 0.7d - 1.5t$ (*Eq.* E2.3-5)

and F_u and F_{xx} are defined in Section E2.2. The minimum edge distance shall be as determined for the arc spot weld, Section E2.2 [see Figure E2.3(B)].

If it can be shown by measurement that a given weld procedure will consistently give a larger effective width, d_e or d_a as applicable, this value may be used providing the particular welding procedure used for making the welds that are measured is followed.

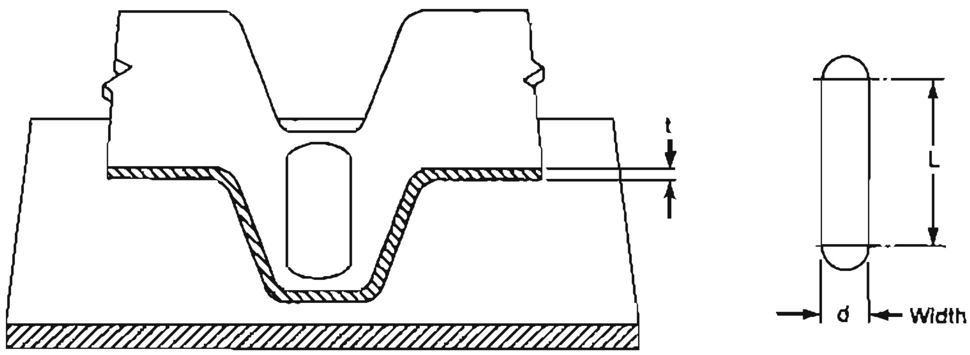


Figure E2.3A Arc Seam Welds—Sheet to Supporting Member in Flat Position

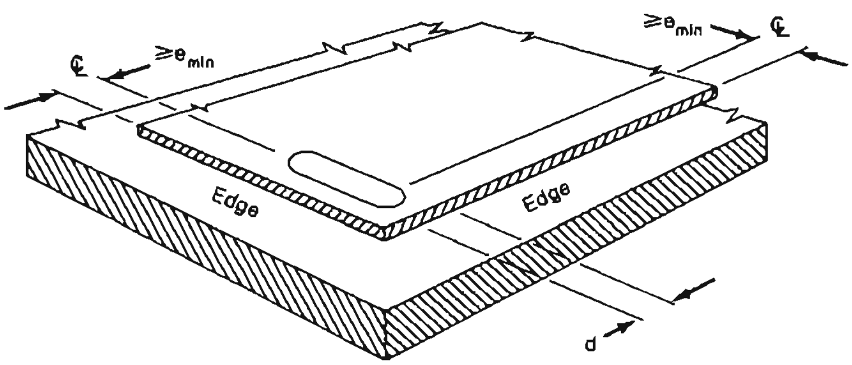


Figure E2.3B Edge Distances for Arc Seam Welds

E2.4 Fillet Welds

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

(a) Sheet to sheet, or
(b) Sheet to thicker steel member.

The shear load, P_n, on a fillet weld in lap and T-joints shall not exceed the following:

For longitudinal loading:
For $L/t < 25$:

$$P_n = \left(1 - \frac{0.01L}{t}\right) tLF_u \qquad (Eq.\ E2.4\text{-}1)$$

For $L/t \geq 25$:

$$P_n = 0.75tLF_u \quad (Eq.\ E2.4\text{-}2)$$

For transverse loading:

$$P_n = tLF_u \quad (Eq.\ E2.4\text{-}3)$$

where

t = Least value of t_1 or t_2, Figure E2.4

In addition, for $t > 0.150$ inch the allowable load for a fillet weld in lap and T-joints shall not exceed:

$$P_n = 0.75t_wLF_{xx} \quad (Eq.\ E2.4\text{-}4)$$

where

L = Length of fillet weld

t_w = Effective throat $= 0.707w_1$ or $0.707w_2$, whichever is smaller. A larger effective throat may be taken if it can be shown by measurement that a given welding procedure will consistently give a larger value providing the particular welding procedure used for making the welds that are measured is followed.

w_1 and w_2 = leg on weld (see Figure E2.4).

F_u and F_{xx} are defined in Section E2.2.

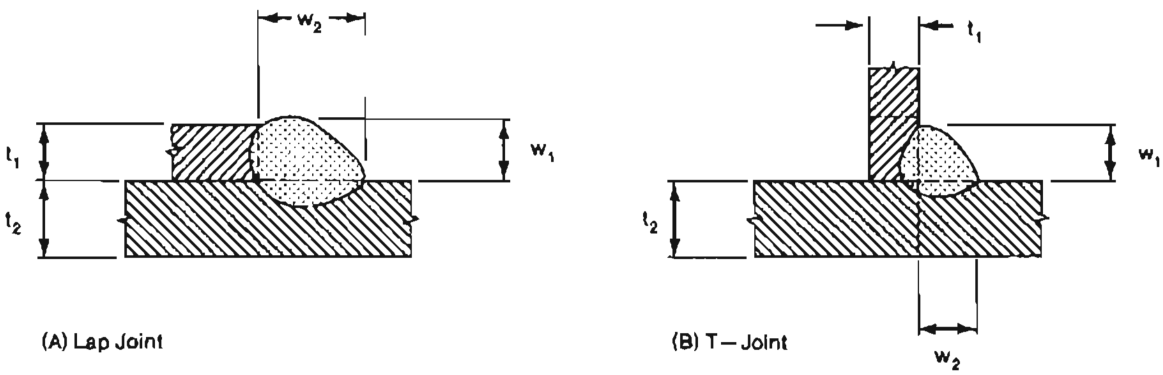


Figure E2.4 Fillet Welds

E2.5 Flare Groove Welds

Flare groove welds covered by this Specification apply to welding of joints in any position, either:

(a) Sheet to sheet for flare-V groove welds, or
(b) Sheet to sheet for flare-bevel groove welds, or
(c) Sheet to thicker steel member for flare-bevel groove welds.

The shear load, P_n, on a weld shall be governed by the thickness, t, of the sheet steel adjacent to the weld. The load shall not exceed:

For flare-bevel groove welds, transverse loading [see Figure E2.5(A)]:

$$P_n = 0.833tLF_u \quad (Eq.\ E2.5\text{-}1)$$

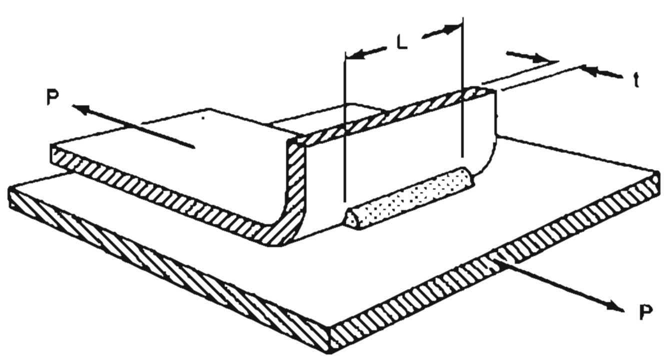


Figure E2.5A Flare-Bevel Groove Weld

For flare groove welds, longitudinal loading [see Figures E2.5(B), E2.5(C), and E2.5(D)]:

If the effective throat, t_w, is equal to or greater than t but less than 2t or if the lip height is less than weld length, L, then:

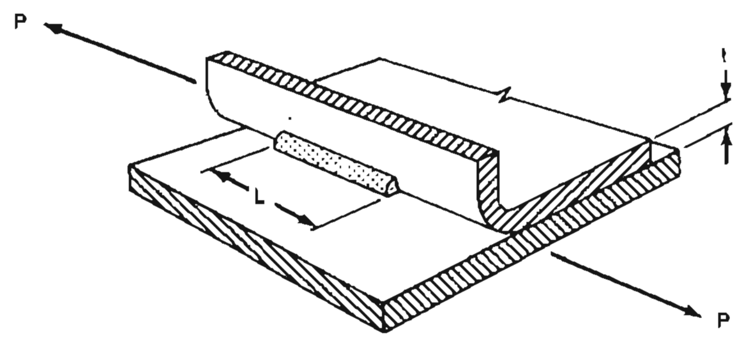
$$P_n = 0.75tLF_u \qquad (Eq.\ E2.5\text{-}2)$$

If t_w is equal to or greater than 2t and the lip height is equal to or greater than L, then:

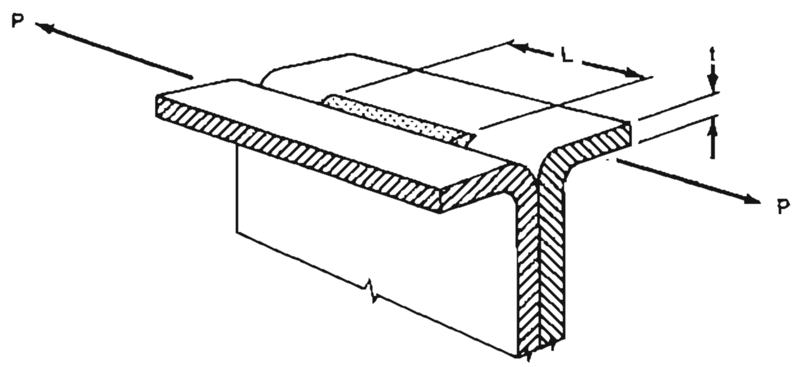
$$P_n = 1.50tLF_u \qquad (Eq.\ E2.5\text{-}3)$$

In addition, if $t > 0.15$ inch, then:

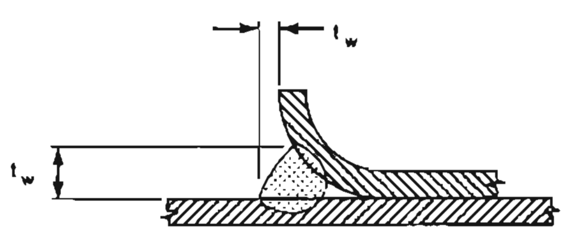
$$P_n = 0.75t_wLF_{xx} \qquad (Eq.\ E2.5\text{-}4)$$



(B) Flare Bevel Groove



(C) Flare V-Groove



(D) Throat

Figure E2.5 B, C, D Shear in Flare Groove Welds

E2.6 Resistance Welds

In sheets joined by spot welding the allowable shear per spot, P_a, shall be as follows (the safety factor is included in Table E2.6):

TABLE E2.6

Thickness of Thinnest Outside Sheet, in.	Allowable Shear Strength per Spot, kips	Thickness of Thinnest Outside Sheet, in.	Allowable Shear Strength per Spot, kips
0.010	0.050	0.080	1.330
0.020	0.175	0.094	1.725
0.030	0.400	0.109	2.395
0.040	0.570	0.125	2.88
0.050	0.660	0.188	4.00
0.060	0.910	0.250	6.00

E3 Bolted Connections

The following requirements govern bolted connections of cold-formed steel structural members in which the thickness of the thinnest connected part is less than 3/16 inch and there are no gaps between connected parts. For bolted connections in which the thinnest connected part is equal to or greater than 3/16 inch, refer to AISC Specification (Reference 3 of Section A6).

Bolts, nuts, and washers shall generally conform to one of the following specifications:

ASTM A307-84 (Type A), Carbon Steel Externally and Internally Threaded Standard Fasteners

ASTM A325-84, High Strength Bolts for Structural Steel Joints

ASTM A354-84 (Grade BD), Quenched and Tempered Alloy Steel Bolts, Studs, and Other Externally Threaded Fasteners (for diameter of bolt smaller than ½ inch)

ASTM A449-84a, Quenched and Tempered Steel Bolts and Studs (for diameter of bolt smaller than ½ inch)

ASTM A490-84, Quenched and Tempered Alloy Steel Bolts for Structural Steel Joints.

When other than the above are used, drawings shall indicate clearly the type and size of fasteners to be employed and the allowable force assumed in design.

Bolts shall be installed and tightened to achieve satisfactory performance of the connections involved under usual service conditions.

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

TABLE E3
Maximum Size of Bolt Holes, Inches

Nominal Bolt Diameter, d in.	Standard Hole Diameter, d in.	Oversized Hole Diameter, d in.	Short-Slotted Hole Dimensions in.	Long-Slotted Hole Dimensions in.
1/2	d + 1/32	d + 1/16	(d + 1/32)by(d + 1/4)	(d + 1/32)by(2-1/2 d)
≥ 1/2	d + 1/16	d + 1/8	(d + 1/16)by(d + 1/4)	(d + 1/16)by(2-1/2 d)

Standard holes shall be used in bolted connections, except that oversized and slotted holes may be used as approved by the designer. The length of slotted holes shall be normal to the direction of the shear load. Washers or backup plates shall be installed over oversized or short-slotted holes in an outer ply unless suitable performance is demonstrated by load tests in accordance with Section F.

E3.1 Spacing and Edge Distance

The distance, e, measured in the line of force from the center of a standard hole to the nearest edge of an adjacent hole or to the end of the connected part toward which the force is directed shall not be less than the value of e_{min} determined as follows:

$$e_{min} = e\Omega_e \qquad (Eq.\ E3.1\text{-}1)$$

where

$$e = \frac{P}{F_u t} \qquad (Eq.\ E3.1\text{-}2)$$

(a) When $F_u/F_{sy} \geq 1.15$:
Ω_e = Factor of safety for sheet tearing
= 2.0

(b) When $F_u/F_{sy} < 1.15$:
Ω_e = Factor of safety for sheet tearing
= 2.22

where
P = Force transmitted by bolt
t = Thickness of thinnest connected part
F_u = Tensile strength of the connected part as specified in Sections A3.1 or A3.2.
F_{sy} = Yield point of the connected part as specified in Sections A3.1 or A3.2.

In addition, the minimum distance between centers of bolt holes shall provide sufficient clearance for bolt heads, nuts, washers and the wrench but shall not be less than 3 times the nominal bolt diameter, d. Also, the distance from the center of any standard hole to the end or other boundary of the connecting member shall not be less than 1½ d.

For oversized and slotted holes, the distance between edges of two adjacent holes and the distance measured from the edge of the hole to the end or other boundary of the connecting member in the line of stress shall not be less than the value of $[e_{min} - (d_h/2)]$, in which e_{min} is the required distance computed from the applicable equation given above, and d_h is the diameter of a standard hole defined in Table E3. In no case shall the clear distance between edges of two adjacent holes be less than 2d and the distance between the edge of the hole and the end of the member be less than d.

E3.2 Tension in Connected Part

The tension force on the net section of a bolted connection shall not exceed T_a from Section C2 or P_a calculated as follows:

$$P_a = P_n/\Omega_t \qquad (Eq.\ E3.2\text{-}1)$$

where
$P_n = A_n F_t$
A_n = Net section area
F_t and Ω_t are determined as follows:

(a) When $t \geq 3/16$ in.:
See AISC Specification (Reference 3 of Section A6)

(b) When $t < 3/16$ inch and washers are provided under both the bolt head and the nut

$$F_t = (1.0 - 0.9r + 3rd/s)F_u \leq F_u \qquad (Eq.\ E3.2\text{-}2)$$

Ω_t = Factor of safety for tension on the net section
= 2.0 for double shear
= 2.22 for single shear

(c) When $t < 3/16$ inch and either washers are not provided under the bolt head and nut, or only one washer is provided under either the bolt head or nut

$$F_t = (1.0 - r + 2.5rd/s)F_u \leq F_u \qquad (Eq.\ E3.2\text{-}3)$$

Ω_t = Factor of safety for tension on the net section
= 2.22

where
r = Force transmitted by the bolt or bolts at the section considered, divided by the tension force in the member at that section. If r is less than 0.2, it may be taken equal to zero.

s = Spacing of bolts perpendicular to line of stress.
In the case of a single bolt, s = Width of sheet
F_t = Nominal tension stress limit on net section
F_u = Tensile strength of the connected part as specified in Sections A3.1 or A3.2
d and t are defined in Section E3.1.

E3.3 Bearing

The bearing force shall not exceed P_a calculated as follows:

$$P_a = P_n/\Omega_b \qquad (Eq.\ E3.3\text{-}1)$$

where

$$P_n = F_p dt \qquad (Eq.\ E3.3\text{-}2)$$

Ω_b = Safety factor for bearing
= 2.22
F_p = Nominal bearing stress as given in Tables E3.3-1 and E3.3-2.

For conditions not shown, forces shall be determined on the basis of test data using a factor of safety of 2.22.

TABLE E3.3-1
Nominal Bearing Stress for Bolted Connections with Washers under Both Bolt Head and Nut

Thickness of connected part in.	Type of joint	F_u/F_{sy} ratio of connected part	Nominal bearing stress, F_p
≥ 0.024 but < 3/16	Inside sheet of double shear connection	≥ 1.15	3.33 F_u
		< 1.15	3.00 F_u
	Single shear and outside sheets of double shear connection	No limit	3.00 F_u
≥ 3/16	See AISC Specification (Reference 3 of Section A6)		

TABLE E3.3-2
Nominal Bearing Stress for Bolted Connections Without Washers Under Both Bolt Head and Nut, or With Only One Washer

Thickness of connected part in.	Type of joint	F_u/F_{sy} ratio of connected part	Nominal bearing stress, F_p
≥ 0.036 but < 3/16	Inside sheet of double shear connection	≥ 1.15	3.00 F_u
	Single shear and outside sheets of double shear connection	≥ 1.15	2.22 F_u
≥ 3/16	See AISC Specification (Reference 3 of Section A6)		

E3.4 Shear and Tension in Bolts

The bolt force resulting from shear, tension or combination of shear and tension shall not exceed allowable bolt force, P_a, calculated as follows (The factor of safety is included in Tables E3.4-1 and E3.4-2):

$$P_a = A_b F \qquad (Eq.\ E3.4\text{-}1)$$

where

A_b = Gross cross-sectional area of bolt

F is given by F_v, F_t or F'_t in Tables E3.4-1 and E3.4-2.

TABLE E3.4-1

Description of Bolts	Allowable Shear Stress*, F_v, ksi: Threads not Excluded from Shear Plane	Allowable Shear Stress*, F_v, ksi: Threads Excluded from Shear Plane	Allowable Tension Stress, F_t, ksi
A325 Bolts	21	30	44
A354 Grade B Bolts (1/4 in. $\leq$ d < 1/2 in.)	24	40	49
A449 Bolts (1/4 in. $\leq$ d < 1/2 in.)	18	30	40
A 490 Bolts	28	40	54
A307 Bolts, Grade A (1/4 in. $\leq$ d < 1/2 in.)	9		18
A307 Bolts, Grade A (d $\geq$ 1/2 in.)	10		20

*Applies to bolts in holes as limited by Table E3. Washers or back-up plates shall be installed over long-slotted holes and the capacity of connections using long-slotted holes shall be determined by load tests in accordance with Section F.

The pullover strength of the connected sheet at the bolt head, nut or washer should be considered where bolt tension is involved, See Section E5.2.

When bolts are subject to a combination of shear and tension, the tension force shall not exceed to a combination of shear and tension, the tension force shall not exceed the allowable force, P_a, based on F'_t, given in Table E3.4-2, where f_v, the shear stress produced by the same forces, shall not exceed the allowable value F_v given above.

TABLE E3.4-2

Allowable Tension Stress, F'_t for Bolts Subject to the Combination of Shear and Tension

Description of Bolts	Threads Not Excluded from Shear Planes	Threads Excluded from Shear Planes
A325 Bolts	$55 - 1.8f_v \leq 44$	$55 - 1.4f_v \leq 44$
A354 Grade BD Bolts	$61 - 1.8f_v \leq 49$	$61 - 1.4f_v \leq 49$
A449 Bolts	$50 - 1.8f_v \leq 40$	$50 - 1.4f_v \leq 40$
A490 Bolts	$68 - 1.8f_v \leq 54$	$68 - 1.4f_v \leq 54$
A307 Bolts, Grade A		
When 1/4 in. $\leq$ d < 1/2 in.	$23 - 1.8f_v \leq 18$	
When d $\geq$ 1/2 in.	$26 - 1.8f_v \leq 20$	

E4 Shear Rupture

At beam-end connections, where one or more flanges are coped and failure might occur along a plane through the fasteners, the shear force shall not exceed the allowable shear force V_a, calculated as follows:

$$V_a = V_n/\Omega_v \qquad (Eq.\ E4\text{-}1)$$

where

V_n = $0.6\, F_u A_{wn}$ (*Eq.* E4-2)
A_{wn} = $(d_{wc} - nd_h)t$ (*Eq.* E4-3)
d_{wc} = coped web depth
n = number of holes in the critical plane
d_h = hole diameter
F_u = Tensile strength as specified in Sections A3.1 or A3.2
t = Thickness of coped web
Ω_v = Factor of safety for shear rupture
= 2.00

E5 Connections to Other Materials

E5.1 Bearing

Proper provisions shall be made to transfer bearing forces resulting from axial loads and moments from steel components covered by the Specification to adjacent structural components made of other materials. The bearing force in the contact area shall not exceed the allowable bearing force P_a calculated as follows:

$$P_a = F_p A$$

where
A = Contact area
F_p = Allowable bearing stress. (The factor of safety is included in values for F_p.)

In the absence of code regulations for other materials, the following allowable stresses may be used:

F_p = 0.40 ksi on sandstone and limestone
F_p = 0.25 ksi on brick in cement mortar
F_p = $0.35f'_c$ on the full area of a concrete support
F_p = $0.35f'_c\sqrt{(A_2/A_1)} \le 0.7f'_c$ on less than the full area of a concrete support

where
f'_c = Specified compression strength of concrete
A_1 = Bearing area
A_2 = Full cross-sectional area of concrete support

E5.2 Tension

The pull-over shear/tension forces in the steel sheet around the head of the fastener should be considered as well as the pull-out force resulting from axial loads and bending moments transmitted onto the fastener from various adjacent structural components in the assembly.

The allowable tensile strength of the fastener and the allowable imbedment strength of the adjacent structural component shall be determined by applicable product code approvals, or product specifications and/or product literature.

E5.3 Shear

Proper provisions shall be made to transfer shearing forces from steel components covered by this Specification to adjacent structural components made of other materials. The allowable shear and/or bearing forces on the steel components shall not exceed that

allowed by this Specification. The allowable shear force on the fasteners and other material shall not be exceeded. Imbedment requirements are to be met. Proper provision shall also be made for shearing forces in combination with other forces.

F. TESTS FOR SPECIAL CASES

(a) Tests shall be made by an independent testing laboratory or by a testing laboratory of a manufacturer.
(b) The provisions of Chapter F do not apply to cold-formed steel diaphragms.

F1 Tests for Determining Structural Performance

Where the composition or configuration of elements, assemblies, connections, or details of cold-formed steel structural members are such that calculation of their safe load-carrying capacity or deflection cannot be made in accordance with the provisions of this Specification, their structural performance shall be established from tests and evaluated in accordance with the following procedure.

(a) Where practicable, evaluation of tests results shall be made on the basis of the mean values resulting from tests of not fewer than three identical specimens, provided the deviation of any individual test result from the mean value obtained from all tests does not exceed ± 10 percent. If such deviation from the mean exceeds 10 percent, at least three more tests of the same kind shall be made. The average of the three lowest values of all tests made shall then be regarded as the result of the series of tests.
(b) The required load-carrying capacity shall be:

$$R = DF_D + LF_L \quad (Eq.\ F1\text{-}1)$$

where D and L are the dead and live loads, respectively, D shall include the weight of the test specimen. F_D and F_L are the dead and live load factors specified below. R shall be taken as the largest applicable value determined as follows:

(1) The minimum load-carrying capacity, R, shall be calculated from the formula

$$R \geq 1.5D + 2L \quad (Eq.\ F1\text{-}2)$$

R shall be multiplied by 1.25 for steels not listed in Section A3.1
R may be divided by 1⅓ when the loading consists of wind or earthquake loads alone, or in combination with dead, live, or snow loads, but shall not be less than R calculated for the combination of dead and live loads only, without wind or earthquake loads.

(2) The load at which distortions interfere with the proper functioning of the specimen in actual use shall not be less than:

$$R \geq D + 1.5L \quad (Eq.\ F1\text{-}3)$$

(3) The load carrying capacity when limited by connection failure shall not be less than:

$$R = 2.5D + 2.5L \quad (Eq.\ F1\text{-}4)$$

(c) If the yield point of the steel from which the tested sections are formed is larger than the specified value, the test results shall be adjusted down to the specified minimum yield point of the steel which the manufacturer intends to use. The test results shall not be adjusted upward if the yield point of the test specimen is less than the minimum specified yield point. Similar adjustments shall be made on the basis of tensile strength instead of yield point where tensile strength is the critical factor.
Consideration must also be given to any variation or differences which may exist between the design thickness and the thickness of the specimens used in the tests.

F2 Tests for Confirming Structural Performance

The procedures and formulas specified in Section F1 are not applicable to confirmation tests on specimens whose capacities can be computed according to this Specification or its specific references. A successful confirmatory test shall demonstrate a safety factor not less than that implied in the Specification for the type of behavior involved.

F3 Tests for Determining Mechanical Properties

F3.1 Full Section

Tests for determination of mechanical properties of full sections to be used in Section A5.2.2 shall be made as specified below:

(a) Tensile testing procedures shall agree with Standard Methods and Definitions for Mechanical Testing of Steel Products, ASTM A370-77ε.
Compressive yield point determinations shall be made by means of compression tests of short specimens of the section.

(b) The comprehensive yield stress shall be taken as the smaller value of either the maximum compressive strength of the sections divided by the cross section area or the stress defined by one or the following methods:
   (1) For sharp yielding steel, the yield point shall be determined by the autographic diagram method or by the total strain under load method.
   (2) For gradual yielding steel, the yield point shall be determined by the strain under load method or by the 0.2 percent offset method.
   When the total strain under load method is used, there shall be evidence that the yield point so determined agrees within 5 percent with the yield point which would be determined by the 0.2 percent offset method.

(c) Where the principal effect of the loading to which the member will be subjected in service will be to produce bending stresses, the yield point shall be determined for the flanges only. In determining such yield points, each specimen shall consist of one complete flange plus a portion of the web of such flat width ratio that the value of ρ for the specimen is unity.

(d) For acceptance and control purposes, two full section tests shall be made from each lot of not more than 50 tons nor less than 30 tons of each section, or one test from each lot of less than 30 tons of each section. For this purpose a lot may be defined as that tonnage of one section that is formed in a single production run of material from one heat.

(e) At the option of the manufacturer, either tension or compression tests may be used for routine acceptance and control purposes, provided the manufacturer demonstrates that such tests reliably indicate the yield point of the section when subjected to the kind of stress under which the member is to be used.

F3.2 Flat Elements of Formed Sections

Tests for determining mechanical properties of flat elements of formed sections and representative mechanical properties of virgin steel to be used in Section A5.2.2 shall be made in accordance with the following provisions:

The yield point of flats, F_{yf}, shall be established by means of a weighted average of the yield points of standard tensile coupons taken longitudinally from the flat portions of a representative cold-formed member. The weighted average shall be the sum of the products of the average yield point for each flat portion times its cross-sectional area, divided by the total area of flats in the cross section. The exact number of such coupons will depend on the shape of the member, i.e., on the number of flats in the cross section. At least one tensile coupon shall be taken from the middle of each flat. If the actual virgin yield point exceeds the specified minimum yield point, the yield point of the flats, F_{yf}, shall be adjusted by multiplying the test values by the ratio of the specified minimum yield point to the actual virgin yield point.

F3.3 Virgin Steel

The following provisions apply to steel produced to other than the ASTM Specifications listed in Section A3.1 when used in sections for which the increased yield point of the steel after cold forming shall be computed from the virgin steel properties according to Section A5.2.2. For acceptance and control purposes, at least four tensile specimens shall be taken from each lot as defined in Section F3.1(d) for the establishment of the representative values of the virgin tensile yield point and ultimate strength. Specimens shall be taken longitudinally from the quarter points of the width near the outer end of the coil.

APPENDICIES

Appendix A5.2.2 Strength Increase from Cold Work of Forming

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

(a) For axially loaded compression members and flexural members whose proportions are such that the quantity ρ is unity as determined according to Section B2 for each of the component elements of the section, the design yield stress, F_{ya}, of the steel shall be determined on the basis of one of the following methods:
   (1) full section tensile tests [see paragraph (a) of Section F3.1].
   (2) stub column tests [see paragraph (b) of Section F3.1]
   (3) computed as follows:

$$F_{ya} = CF_{yc} + (1 - C)F_{yf} \qquad (Eq.\ A5.2.1\text{-}1)$$

where

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

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

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

$$F_{yc} = B_c F_{yv}/(R/t)^m,\ \text{tensile yield point of corners} \qquad (Eq.\ A5.2.1\text{-}2)$$

when

$F_{uv}/F_{yv} \geq 1.2$, $R/t \leq 7$, minimum included angle $\leq 120°$

$$B_c = 3.69(F_{uv}/F_{yv}) - 0.819(F_{uv}/F_{yv})^2 - 1.79 \qquad (Eq.\ A5.2.1\text{-}3)$$

$$m = 0.192(F_{uv}/F_{yv}) - 0.068 \qquad (Eq.\ A5.2.1\text{-}4)$$

R = Inside bend radius.

F_{yv} = Tensile yield point of virgin steel* specified by Section A3 or established in accordance with Section F3.3.

F_{uv} = Ultimate tensile strength of virgin steel* specified by Section A3 or established in accordance with Section F3.3

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

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

*Virgin steel refers to the condition (i.e., coiled or straight) of the steel prior to the cold-forming operation.

Appendix B1.1(b) Flange Curling*

Where the flange of a flexural member is unusually wide and it is desired to limit the maximum amount of curling or movement of the flange toward the neutral axis, the following formula applies to compression and tension flanges, either stiffened or unstiffened:

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

where

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

t = Flange thickness

d = Depth of beam

c_f = Amount of curling

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

Appendix B1.1(c) Shear Lag Effects

Where the span of the beam is less than $30w_f$ (w_f as defined below) and it carries one concentrated load, or several loads spaced farther apart than $2w_f$, the effective design width of any flange, whether in tension or compression, shall be limited to the following:

APPENDIX TABLE B1.1(c)
SHORT, WIDE FLANGES
MAXIMUM ALLOWABLE RATIO OF EFFECTIVE DESIGN WIDTH TO ACTUAL WIDTH

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

where

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

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

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

*The amount of curling that can be tolerated will vary with different kinds of sections and must be established by the designer. Amount of curling in the order of 5 percent of the depth of the section is usually not considered excessive.

COMMENTARY

COMMENTARY ON THE SPECIFICATION FOR THE DESIGN OF COLD–FORMED STEEL STRUCTURAL MEMBERS

AUGUST 19, 1986 EDITION

WITH

DECEMBER 11, 1989 ADDENDUM

Cold–Formed Steel Design Manual – Part II

AMERICAN IRON AND STEEL INSTITUTE
1133 15th STREET, NW
WASHINGTON, DC 20005–2701

1st Printing – January 1991

Produced by Computerized Structural Design, Inc.
Milwaukee, Wisconsin

PREFACE
TO 1989 ADDENDUM

The Commentary on the Specification has been updated to describe the background for the changes in and additions to the *Specification* contained in the 1989 Addendum.

American Iron and Steel Institute
December 11, 1989

PREFACE

This document, *Part II* of the *Cold-Formed Steel Design Manual*, provides a commentary on the background for the 1986 Edition of the *Specification for the Design of Cold-Formed Steel Structural Members*.

The *Commentary* should be used in conjunction with the other parts of the *Design Manual* which include *Supplementary Information (Part III)*, *Illustrative Examples (Part IV)*, *Charts and Tables (Part V)*, *Computer Aids (Flow Charts) (Part VI)*, and *Test Procedures (Part VII)* in addition to the *Specification (Part I)*.

The careful assistance and close cooperation of the Advisory Group on the *Specification* is gratefully acknowledged. Special thanks are extended to the Subcommittee which drafted and edited this *Commentary*; D. S. Ellifritt (Chairman), R. E. Albrecht, R. A. LaBoube, T. B. Pekoz, D. S. Wolford, W. W. Yu, S. J. Errera and A. L. Johnson.

American Iron and Steel Institute
August 19, 1986

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COMMENTARY ON THE SPECIFICATION FOR THE DESIGN OF COLD–FORMED STEEL STRUCTURAL MEMBERS AUGUST 19, 1986 EDITION WITH DECEMBER 11, 1989 ADDENDUM

INTRODUCTION

This *Commentary* is intended to facilitate the use, and provide an understanding of the background, of the *Specification for the Design of Cold-Formed Steel Structural Members* – 1986 Edition with 1989 Addendum. It illustrates the substance and limitations of the various provisions, with particular emphasis on changes made since the 1980 edition. Further, it provides information on the background for the various *Specification* provisions. Mainly, this is accomplished through an extensive integrated bibliography, as well as brief, substantive discussions.

Research on cold-formed steel construction started under the sponsorship of the American Iron and Steel Institute at Cornell University in 1939. Since that time, research has continued at Cornell, as well as other institutions. In 1946 the first edition of the *Specification* was published (Reference 1). This and subsequent editions are listed as References 1 through 9. As Reference 3 indicates, since 1956, the *Specification* has been Part I of the AISI's *Cold-Formed Steel Design Manual*. The other sections of the *Manual* included are: Part II: *Commentary*; Part III: *Supplementary Information*; Part IV: *Illustrative Examples*, and Part V: *Charts and Tables*. Parts II through V illuminate and facilitate the use of the *Specification* for the designer, building official, teacher, and student. The 1986 edition of the *Manual* contains two new sections, Part VI: *Computer Aids (Flow Charts)*, and Part VII: *Test Procedures*.

The *Specification* is coordinated with AISC's *Specification for the Design, Fabrication and Erection of Structural Steel for Buildings* (Reference 10). The latter applies to hot-rolled shapes and built-up members, while the AISI *Specification* deals with members cold formed to shape from flat steel, usually of relatively small thickness. The range of shapes which can be produced by cold-forming processes is practically unlimited.

In the *Commentary*, the individual sections, equations, figures and tables are identified by the same notation as in the *Specification* and the material is presented in the same sequence.

A. GENERAL PROVISIONS

A1 Limits of Applicability and Terms

A1.1 Scope and Limits of Applicability

The *Specification* is limited to steel structural members cold formed from carbon or low-alloy sheet, strip, plate or bar. The forming process is carried out at, or near, room temperature by the use of bending brakes, press brakes or roll-forming machines.

Some of the significant differences between cold-formed sections and hot-rolled shapes are (1) absence of residual cooling stresses, (2) lack of corner fillets, (3) presence of increased yield strength and decreased proportional limit resulting from cold-forming, (4) presence of cold-reducing stresses when cold-rolled steel stock has not been final-annealed, (5) prevalence of elements having large width-thickness ratios, (6) rounded corners, and (7) for some steels different stress-strain curves, some without sharp yield points.

Research conducted at the University of Missouri-Rolla (References 11 and 12) verified the applicability of the *Specification's* provisions for members cold formed of flat steels up to and including one inch in thickness.

A1.2 Terms

Many of the definitions in *Specification* Section A1.2 are self-explanatory. Only those which are not self-explanatory, or not listed, are briefly discussed below.

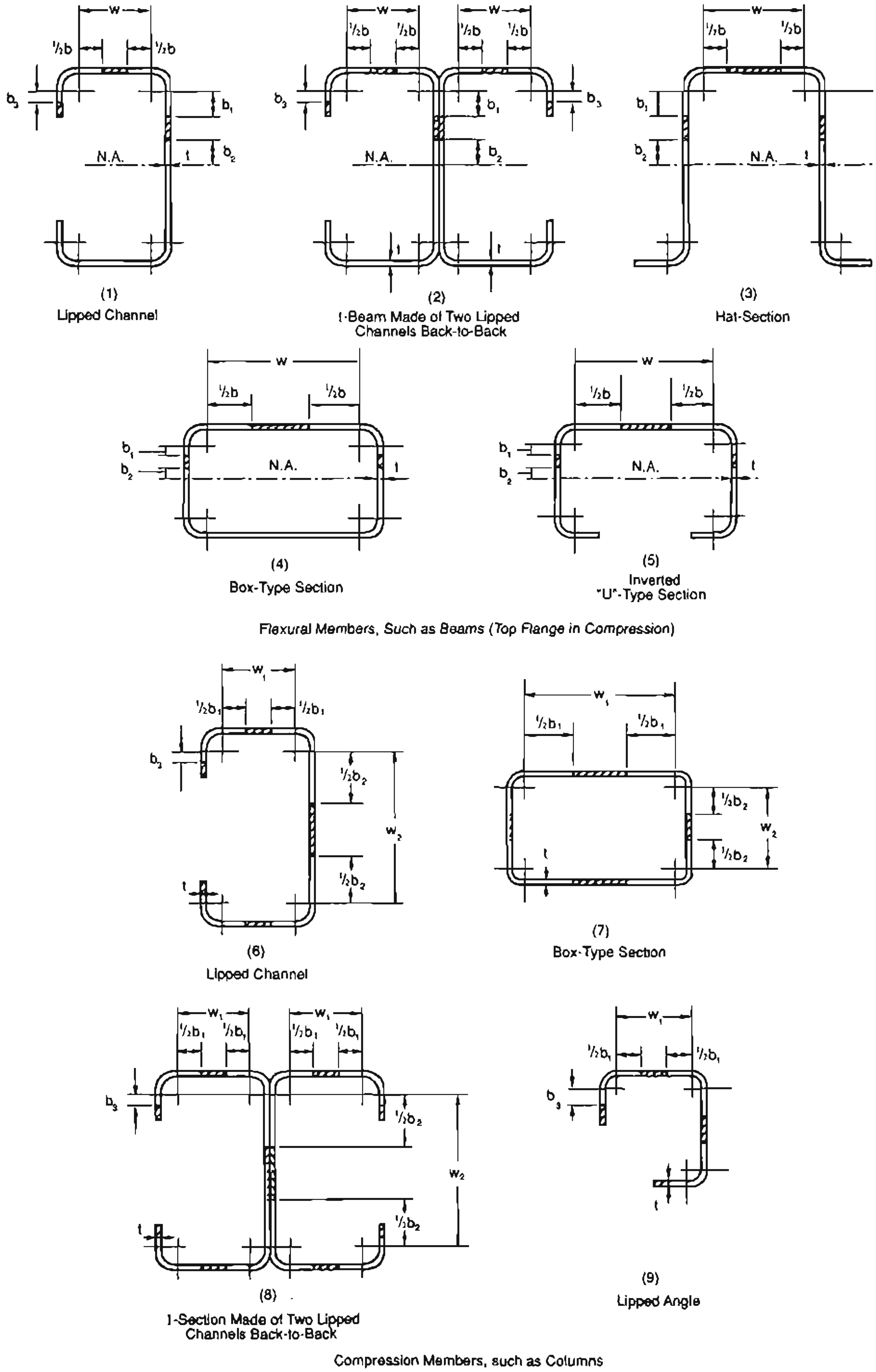


Figure A1.2(a) Stiffened Compression Elements Used in Cold–Formed Steel Structural Members

(a) *Stiffened or Partially Stiffened Compression Elements*

Stiffened compression elements of various sections are shown in Figure A1.2(a). Sections (1) and (2) each have a web and a lip to stiffen the compression element, i.e., the compression flange, the ineffective portion of which is shown shaded. For the explanation of these ineffective portions, see definition (e) Effective Design Width, below, and Chapter B. Sections (3), (4) and (5) show compression elements stiffened by two webs. Sections (6) and (8) show edge stiffened flange elements that have a vertical element (web) and an edge stiffener (lip) to stiffen the elements while the web itself is stiffened by the flanges, (7) has four compression elements stiffening each other, and (9) has each stiffened element stiffened by a lip and the other stiffened element.

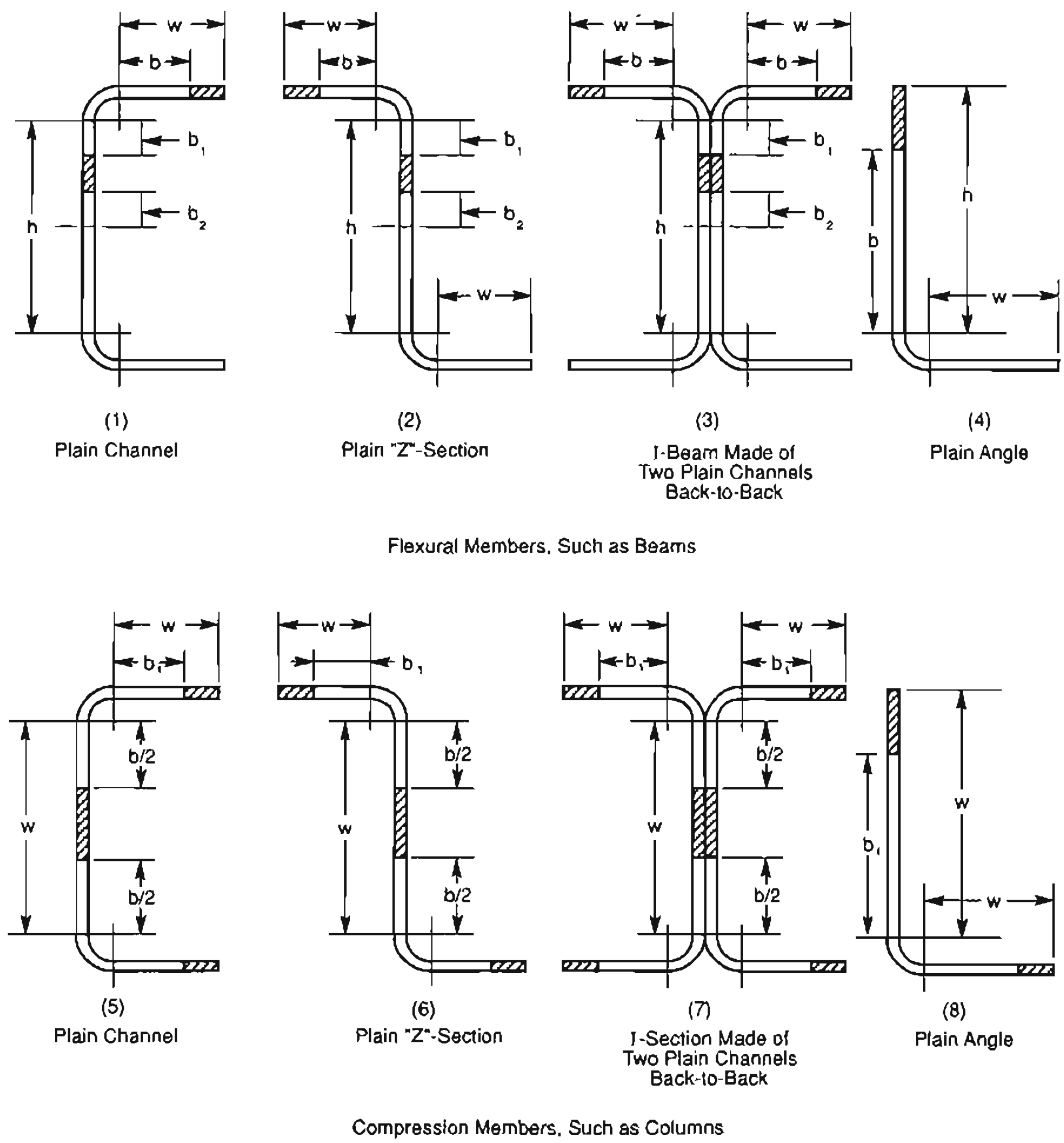


Figure A1.2(b) Unstiffened Compression Elements Used In Cold-Formed Steel Structural Members

(b) *Unstiffened Compression Elements*

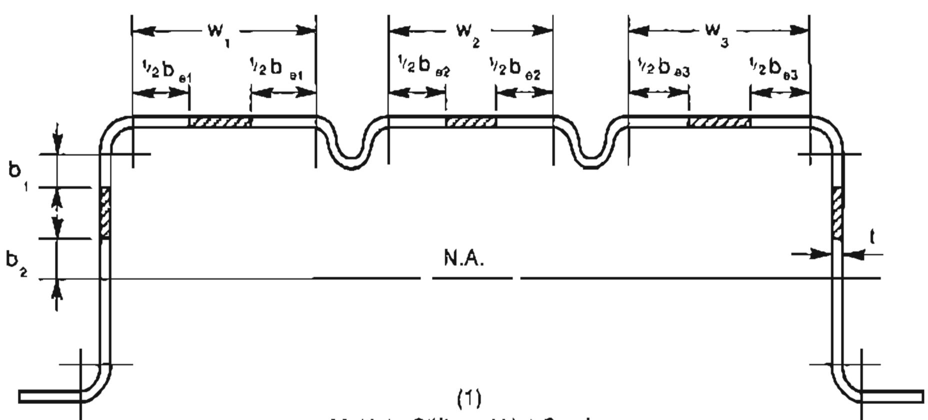
Unstiffened elements of various sections are shown in Figure A1.2(b). Sections (1), (2), and (3) have only a web to stiffen the compression flange element. The legs of (4) provide mutual stiffening action to each other along their common edges. Sections (5), (6), and (7) acting as columns have vertical stiffened elements (webs) which provide support for one edge of the unstiffened flange elements. The legs of (8) provide mutual stiffening action to each other.

(c) *Multiple–Stiffened Elements*

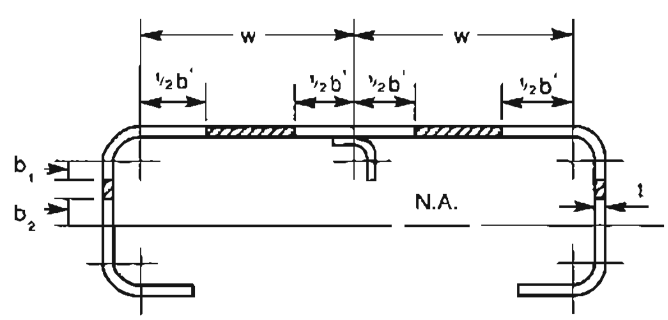
Multiple–stiffened elements of two sections are shown in Figure A1.2(c). Each of the two outer sub–elements of (1) are stiffened by a web and an intermediate stiffener while the middle sub–element is stiffened by two intermediate stiffeners. The two sub–elements of (2) are stiffened by a web and the attached intermediate middle stiffener.

(e) *Effective Design Width*

The effective design width is a concept which facilitates taking account of local buckling and post–buckling strength for compression elements. The effect of shear lag on short, wide flanges is also handled by using an effective design width. These matters are treated in *Specification* Chapter B, and the corresponding effective widths are discussed in the *Commentary* on that chapter.



(1)
Multiple-Stiffened Hat-Section



(2)
Multiple-Stiffened Inverted "U"-Type Section

Flexural Members, such as Beams

Figure A1.2(c) Multiple–Stiffened Compression Elements Used in Cold–Formed Steel Structural Members

(f) *Thickness*
In calculating section properties, the reduction in thickness that occurs at corner bends is ignored, and the thickness of the flat steel stock, exclusive of coatings, is used in all structural calculations.

(g) *Torsional–Flexural Buckling*
The 1968 *Specification* pioneered methods for computing column loads of cold–formed steel sections prone to buckle by simultaneous twisting and bending. This complex behavior results in lower column loads than would result from primary buckling by flexure alone.

A1.3 Units of Symbols and Terms

The non–dimensional character of the majority of the *Specification* provisions is intended to facilitate design in any compatible system of units (U.S. customary, SI, or metric). It will also simplify conversion to a load and resistance factor design format.

The official having jurisdiction (authority) may approve any alternate shape of constructions provided the proposed alternate is satisfactory and complies with the provisions of Section F of the *Specification* and the particular building code.

A2 Non–Conforming Shapes and Construction

If there is insufficient evidence of compliance with the requirements of the particular building code, the authority administering the code may require tests, at the applicant's expense, as proof of compliance. Test procedures shall be as stipulated by Section F of the *Specification*. If there is no recognized or accepted test method, the authority may prescribe appropriate test procedures.

A3 Material

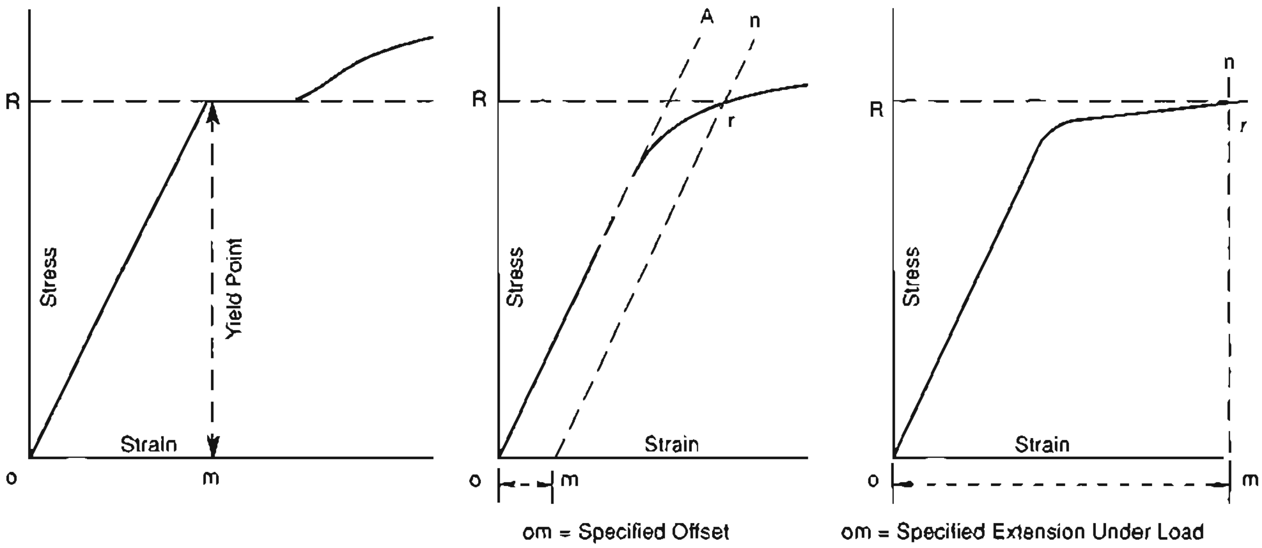
A3.1 Applicable Steels

The American Society for Testing and Materials (ASTM) is the basic source of steel designations for use with the *Specification*. Section A3.1 contains the complete list of ASTM Standards for steels that are accepted in connection with the *Specification*. Dates of issue are included in Section A6. The important structural properties necessary for cold forming are: yield point, tensile strength and elongation in 2 inches.

Ductility is the ability of a steel to undergo sizable plastic or permanent strains before fracturing and is important both for structural safety and for cold forming. It is measured by the elongation in a 2 inch–length. The ratio of the tensile strength to the yield point is also an important material property; this is an indication of the ability of the material to redistribute stress. For the listed ASTM Standards, yield points of steels with adequate ductility range from 25 to 70 ksi, though steels with yield points below 33 ksi are rarely used. The tensile–to–yield ratios are no less than 1.17, and the elongations no less than 12 percent. Exceptions are ASTM A446 and ASTM A611 which feature Grade E steels with a yield point of 80 ksi, a tensile strength of 82 ksi, and with no stipulated minimum elongation in 2 inches. These low–ductility steels permit only limited amounts of cold forming, require fairly large corner radii, and have other limits on their applicability. Nevertheless, they have been used successfully for specific applications, such as decks and panels with large corner radii and little, if any, stress concentrations. The conditions for use of these Grade E steels are outlined in *Specification* Section A3.3.2.

Standard methods and definitions for mechanical testing of steels and steel products are given in ASTM A370 (Reference 13). The methods for determining yield points for sharp and for gradual yielding steels are schematically shown on Figure A3.1.

The *Specification* formulas involving buckling are based on the tacit assumption that proportional limits of suitable steels shall be at least 70 percent of yield based on tensile tests. Determination of proportional limits for informational purposes can be done simply by using the offset method shown in Figure A3.1(b) with *om* equal to 0.0001 in./in. (0.01 percent offset) and calling the stress *R* where *mn* intersects the stress–strain curve at *r*, the proportional limit (0.01 percent offset).



(a) Showing Yield Point Corresponding with Top of Knee.

(b) Showing Yield Point of Yield Strength by the Offset Method. (Also Used for Proportional Limit)

(c) Determination of Yield Strength by Extension Under Load Method.

Figure A3.1 Stress–Strain Diagrams Showing Methods of Yield Point and Yield Strength Determination

A3.2 Other Steels

Although the use of ASTM–designated steels listed in *Specification* Section A3.1 is encouraged, other steels may also be used in cold–formed steel structures, provided they satisfy the requirements stipulated in this provision.

A3.3 Ductility

The nature and importance of ductility and the ways in which this property is measured were briefly discussed in *Commentary* Section A3.1.

Low–carbon sheet and strip steels with specified minimum yield points from 25 to 50 ksi need to meet ASTM specified minimum elongations in a 2–inch gage length of 12 to 27 percent. In order to meet ductility requirements, steels with yield points higher than 50 ksi are often low–alloy steels. However, Grades E of ASTM A446 and A611 are carbon steels, for which specified minimum yield strength is 80 ksi and no elongation requirement is specified. These differ from the array of steels listed under *Specification* Section A3.1.

In 1968, because new steels of higher strengths were being developed, sometimes with lower elongations, the question of how much elongation is really needed in a structure was the focus of a study initiated at Cornell University. Steels were studied that had yield strengths ranging from 45 to 100 ksi, elongations in 2 inches ranging from 50 to 1.3 percent, and tensile–to–yield strength ratios ranging from 1.51 to 1.00 (References

14–16). The investigators developed elongation requirements for ductile steels. These measurements are more accurate but cumbersome to make; therefore, the investigators recommended the following determination for adequately ductile steels: The tensile-to-yield strength ratio shall not be less than 1.08 and the total elongation in a 2-inch gage length shall not be less than 10 percent, or not less than 7 percent in an 8-inch gage length. Also, the *Specification* limits the use of Chapters B through E to adequately ductile steels. In lieu of the tensile-to-yield strength limit of 1.08, the *Specification* permits the use of elongation requirements using the measurement technique as given in Reference 15, and Part VII of the *Manual*. Because of limited experimental verification of the structural performance of members using material having a tensile-to-yield strength ratio less than 1.08 (68), the *Specification* limits the use of this material to purlins and girts meeting the elastic design requirements of Section C3.1.1(a), C3.1.2, and C3.1.3. Thus, the use of such steel in other applications (compression members, tension members, other flexural members including those whose strength is based on inelastic reserve capacity, etc.) is prohibited. However, in purlins and girts, concurrent axial loads of relatively small magnitude are acceptable providing the requirements of Section C5 are met and P/P_a does not exceed 0.15.

Grade E steels of ASTM A446 and A611 do not have adequate ductility as defined by *Specification* Section A3.3.1. Their use has been limited to particular configurations like roofing, siding, and floor decking.

Limiting the design yield strength to 75 percent of the specified minimum yield point, or 60 ksi, and the design tensile strength to 75 percent of the specified minimum tensile strength, or 62 ksi, whichever is lower, introduces a higher safety factor, but still makes low-ductility steels, such as Grade E, useful for the named applications. Load tests are permitted, but not for the purpose of using higher loads than can be calculated under *Specification* Sections B through E.

A3.4 Delivered Minimum Thickness

Sheet and strip steels, both coated and uncoated, may be ordered to nominal or minimum thickness. If the steel is ordered to minimum thickness, all thickness tolerances are over (+) and nothing under (–). If the steel is ordered to nominal thickness, the thickness tolerances are divided equally between over and under. Therefore, in order to provide equity between the two methods of ordering sheet and strip steel, it was decided to require that the delivered thickness of a cold-formed product be at least 95 percent of the design thickness. Thus, it is apparent that a portion of the factor of safety may be considered to cover minor negative thickness tolerances.

Generally, thickness measurements should be made in the center of flanges. For decking and siding, measurements should be made as close as practical to the center of the first full flat of the section. Thickness measurements should not be made closer to edges than the minimum distances specified in ASTM A568.

The responsibility of meeting this requirement for a cold-formed product is clearly that of the manufacturer of the product, not the steel producer.

A4 Loads

The *Specification* does not establish the dead, live or snow loading requirements for which a structure should be designed. In most cases, these loads are adequately covered by the applicable building code or design standard. See *Specification* Section A6.

Recognized engineering procedures should be employed to reflect the effect of impact loads on a structure.

Ellifritt investigated the basis of the one–third stress increase in wind and earthquake stresses (Reference 17) and concluded that the historical justification for the increase in wind stresses was "The action of wind on a structure is highly localized and of very short duration. Therefore it is not necessary to have as high a safety factor when designing for wind loads." The logic for the one–third increase in allowable design stresses for earthquake loads is similar to that for the wind provision.

The *Specification* recognizes the generally accepted practice of increasing the allowable stress by 33 percent for wind and earthquake. In this *Specification* this is accomplished by permitting a 25 percent reduction in design loads.

When gravity and lateral loads produce forces of opposite sign in members, consideration should be given to the minimum gravity loads acting in combination with wind or earthquake loads.

The final deflected shape of the member should be considered when calculating the load on a relatively flat roof resulting from ponding of rainwater or snowmelt. Guidance can be obtained from Section 1.13.3 of the AISC Specification (Reference 10).

A5 Structural Analysis and Design

A5.1 Design Basis

A safe load is determined by applying a factor of safety to the maximum strength, buckling or yielding, calculated for the member or connection in question. The fundamental nature of the factor of safety is to compensate for uncertainties inherent in the design, fabrication and erection of building components as well as in the applied load (Reference 18). Table A5.1 summarizes the factors of safety adopted in the *Specification*.

A5.2 Yield Point and Strength Increase from Cold Work of Forming

The mechanical properties of the flat sheet, such as yield strength, tensile strength and elongation, may differ from the properties exhibited by the formed section. This difference can be attributed to cold working of the steel sheet during the forming process.

A combination of strain hardening, resulting from stretching of the sheet during the forming process, and strain aging causes an increase in the yield strength and tensile strength and a decrease in ductility. This phenomenon was studied by Winter and others (References 19–22), and this work serves as the basis for the provisions of Section A5.2.2. Yu (Reference 23) provides a more thorough discussion of the cold–work phenomenon.

In some cases, when evaluating the effective area of the web, ρ may be less than unity but the sum of b_1 and b_2 may be such that the web is fully effective, and cold work of forming may be used.

A5.3 Serviceability and Durability

Further information may be found in the applicable building code and contract documents.

A6 Reference Documents

The tabulated references pertain to various aspects of cold–formed steel design. Some of the documents are referenced by the *Specification* while others provide information to aid the design engineer.

TABLE A5.1

Safety Factors by Subjects and Sections of the 1986 AISI Specification for the Design of Cold-Formed Steel Structural Members

Subject	Section	Safety Factor, Ω
Tension Member	C2	1.67 against yielding
Flexural Members		
Flexure only	C3.1	1.67 against yielding and buckling
Shear	C3.2	1.44 against shear yielding 1.67 against inelastic shear buckling 1.71 against elastic shear buckling
Web Crippling	C3.4	1.85 for single unreinforced webs 2.0 for I–beams
Concentrically Loaded Compression Members	C4	1.92 except when F_e is determined according to *Specification* Section C4.1 for fully effective sections having wall thickness greater than 0.09in. and $F_e > F_y/2$. In this case Ω_c varies from 1.92 to 1.67
Wall Studs	D4	1.92 against column buckling 1.92 for computing the permissible shear strain in sheathing material
Fusion Welds	E2	2.50 against ultimate test value
Resistance Welds	E2.6	2.50 against ultimate test value
Bolted Connections	E3	
Spacings and Edge Distance	E3.1	2.0 when $F_u/F_y \geq 1.15$ 2.22 when $F_u/F_y < 1.15$
Tension on Net Section	E3.2	2.0 when washers are provided under bolt head and nut 2.22 when only one washer is used, or no washers
Bearing	E3.3	2.22 against bearing failure
Bolt Shear	E3.4	2.25–2.52 against shear failure of bolts
Wind or Earthquake Loads	A4.4	A twenty five percent reduction of nominal safety factor is permissible provided that the section thus designed is not less than that required for combination of dead and live load

B. ELEMENTS

This edition of the *Specification* differs from previous editions because a unified design approach has been adopted for treating compression elements. The effective width approach, previously applied only to stiffened compression elements, is being universally applied to all compression elements. *Commentary* Section B2 discusses the effective width approach.

B1 Dimensional Limits and Considerations

Because of the relatively large flat–width–to–thickness ratios (w/t) that are possible in cold–formed steel construction, dimensional limits are established in the *Specification*. Also, phenomena not germane to hot–rolled steel construction, e.g., flange curling and shear lag effects, are given consideration in the *Specification*.

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

(a) *Maximum w/t Ratios*

The limits imposed in *Specification* Section B1.1a are unchanged from previous editions of the *Specification*. Field experience has indicated that these limits are reasonable and achievable for typical cold–formed construction. In such cases where the limits are exceeded, tests in accordance with *Specification* Chapter F are required.

The note regarding noticeable deformations for larger flat–width–to–thickness ratios is a caution and is not intended to prevent the use of such compression elements.

(b) *Flange Curling*

Unusually wide, thin, but stable flanges tend to curl, i.e., deflect toward the neutral axis, during loading. An approximate, analytical treatment of the problem is given in Reference 24. Equation B1.1–1 enables the evaluation of the maximum admissible flange width, w_f, for a given amount of curling, c_f. The *Specification* does not stipulate the amount of curling; this is subjective because the distortions may or may not be offensive depending upon the application. The suggested curling of 5 percent of the member depth is not considered excessive under usual conditions.

(c) *Shear Lag Effects*

For beams having relatively small span–to–flange–width ratios, L/w_f, shear deformations may significantly alter the stress distribution in both the compression and tension flanges. Table B1.1 is based on analytical and experimental data summarized in Reference 25. It should be noted that the flange width in this case is the projection beyond the web, not the flat portion of the flange, as is the case in subsequent sections of Chapter B.

B1.2 Maximum Web–Depth–to–Thickness Ratio

The limits prescribed in Section B1.2 are unchanged from the 1980 AISI *Specification*. These limitations are based on the studies reported in References 26–28. However, because the definition for h, the depth of the flat portion of the web measured along the plane of the web, differs from previous editions of the *Specification*, the prescribed limits may appear to be more liberal. An unpublished study by LaBoube concluded that the new definition for h had negligible influence on the web strength.

B2 Effective Widths of Stiffened Elements

The use of effective widths for stiffened compression elements is not new to the *Specification*. Previous editions of the *Specification* have treated only uniformly compressed stiffened elements by using an effective width approach. The scope of application for effective widths has been expanded to include (1) uniformly compressed stiffened elements, (2) uniformly compressed stiffened elements with circular holes and (3) webs and stiffened elements with stress gradients.

B2.1 Uniformly Compressed Stiffened Elements

(a) *Load Capacity Determination*

A non-dimensional format for presentation of *Specification* provisions has been adopted. See Reference 29 for a thorough discussion of the basis for this format as well as the research basis for the minor differences in numerical results. Therefore, the equations of Section B2.1a, although in outward appearance different from the 1980 AISI *Specification*, are, in fact, nearly identical to equations as developed by Winter (References 24, 30-32).

In addition to the non-dimensional format, Equation B2.1-4 is given with k, the plate buckling coefficient, as a variable. The factor k depends upon the boundary conditions of the plate element and the manner of loading. Presenting the Equation in this form, enables its use for all compression elements in the *Specification*. The equations of Section B2.1a are a common thread that runs through Sections B2 through B4 of the *Specification*.

(b) *Deflection Determination*

The design engineer has the option of using two procedures for determining the effective width to be used for deflection calculations. Procedure I uses the effective width Equation of Section B2.1a evaluated at the actual stress level. This is consistent with the practice of the 1980 AISI *Specification*. Procedure II is based upon recently completed research (Reference 29) and yields a more accurate estimate of the effective width for deflection analysis.

B2.2 Uniformly Compressed Stiffened Elements with Circular Holes

These are new provisions and have a limited application as stated in the *Specification*. Reference 29 summarizes the background studies relative to this section.

B2.3 Effective Width of Webs and Stiffened Elements with Stress Gradient

The use of effective widths for web elements subjected to a stress gradient is a deviation from the past practice of using a full area in conjunction with a reduced stress to account for local buckling and post-buckling strength (Reference 23). The effective widths are based upon Winter's effective width equation distributed as shown by Figure B2.3-1 of the *Specification*. Background regarding the development of the provisions of Section B2.3 is provided in Reference 29.

B3 Effective Widths of Unstiffened Elements

The 1986 edition of the *Specification* marks the first time that unstiffened compression elements are treated using an effective width approach. The provisions of this section are based upon analytical and experimental data reported in Reference 29. The research demonstrated that Winter's effective width equation was an adequate predictor of section capacity if the appropriate buckling coefficient, k, is employed.

B3.1 Uniformly Compressed Unstiffened Elements

The theoretical buckling coefficient, as given by Reference 33, is 0.425. This value has been rounded to two significant figures in *Specification* Section B3.1.

B3.2 Unstiffened Elements and Edge Stiffeners with Stress Gradient

There is a very limited amount of information on the behavior of unstiffened plate elements with a stress gradient. Cornell research (Reference 29), into the behavior of edge stiffeners for flexural members, has demonstrated that by using Winter's effective width equation in conjunction with a $k = 0.43$ (uniform compression), good correlation was achieved between test and calculated capacity. This same trend was also true for deflection determination.

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

The provisions of *Specification* Section B4 represent the latest research findings in regard to stiffeners (Reference 29). Previous specifications treated stiffeners as fully effective plate elements. By using effective areas for stiffeners, all compression elements are analyzed on the basis of effective widths or areas.

The design provisions, which are based on both critical local buckling and ultimate strength criteria, recognize the interaction of the plate elements (i.e., flange and stiffener). Also, for the first time, provisions for analyzing partially stiffened, as well as adequately stiffened, compression elements are contained in the *Specification*.

B4.1 Uniformly Compressed Elements with an Intermediate Stiffener

The 1980 AISI *Specification* contained provisions for the minimum required moment of inertia, which was based on the assumption that an intermediate stiffener needed to be twice as rigid as an edge stiffener. Subsequent research (Reference 34) has developed expressions for evaluating the required stiffener rigidity based upon the geometry of the contiguous flat elements. Using the ratio of actual stiffener moment of inertia, I_s, to adequate stiffener moment of inertia, I_a, (i.e., I_s/I_a) to evaluate the buckling coefficient and the stiffener area, a partially stiffened compression flange can also be evaluated.

B4.2 Uniformly Compressed Elements with an Edge Stiffener

Specification Section B4.2 recognizes that the necessary stiffener rigidity depends upon the slenderness of the plate element being stiffened. Thus, Cases I, II and III each contain different definitions for an adequate stiffener moment of inertia.

The interaction of the plate elements, as well as the degree of edge support, full or partial, is compensated for in the expressions for k, d_s and A_s. For more in–depth coverage of this topic, see Reference 29.

Test data to verify the accuracy of the simple lip stiffener design was collected from a number of sources, both university and industry. These tests showed good correlation with the equations in Section B4.2. However, proprietary testing conducted in 1989 revealed that lip lengths with a d/t ratio of greater than 14 gave unconservative results.

A review of the original research data showed a lack of data for simple stiffening lips with d/t ratios greater than 14. Therefore, an upper limit of 14 is recommended pending further research.

B5 Effective Widths of Edge Stiffened Elements with Intermediate Stiffeners or Stiffened Elements with More Than One Intermediate Stiffener

There has been no research to further our understanding of the behavior of multiple stiffened elements. Thus the *Specification* has retained Equation B5–1 from the 1980 *Specification* for evaluating the minimum required rigidity, I_{min}, of an intermediate stiffener for a multiple–stiffened element. This value of I_{min} is used when evaluating the effective widths in *Specification* Section B4.

In addition, *Specification* Section B5(a) stipulates that only intermediate stiffeners adjacent to webs elements (see Figure A1.2–1(c)(1)) shall be counted as effective. Additional stiffeners would have two or more sub–elements between themselves and the nearest shear–transmitting element (i.e., web) and hence, could be ineffective. *Specification* Section B5(b) applies the same reasoning to intermediate stiffeners between a web and an edge stiffener.

If intermediate stiffeners are spaced so closely that the sub–elements are fully effective, i.e., b < w, no plate buckling of the sub–elements will occur. Therefore, the entire assembly of sub–elements and intermediate stiffeners between webs behaves like a single compression element whose rigidity is given by the moment of inertia, I_s, of the full, multiple–stiffened element, including stiffeners. Although the effective width calculations are based upon an equivalent element having width, w_s, and thickness, t_s, the actual thickness must be used when calculating section properties.

B6 Transverse Stiffeners

Design requirements for attached transverse stiffeners and for intermediate stiffeners were added in the 1980 AISI *Specification* and are unchanged in the 1986 *Specification*. Equation B6.1–1 serves to prevent end crushing of the transverse stiffeners, while Equation B6.1–5 is to prevent column–type buckling of the web–stiffeners. The equations for computing the effective areas (A_b and A_c) and the effective widths (b_1 and b_2) were adopted with minor modifications from Reference 28.

The requirements for intermediate stiffeners included in *Specification* Section B6.2 were adapted from Section 1.10.5.3 of the AISC Specification (Reference 10). The equations for determining the minimum required moment of inertia (*Eq.* B6.2–1) and the minimum required gross area (*Eq.* B6.2–2) of attached intermediate stiffeners are based on the studies summarized in Reference 28. In Equation B6.2–1, the minimum value of $(h/50)^4$ was selected from the AISC Specification.

Tests on rolled–in transverse stiffeners covered in *Specification* Section B6.3 were not made in the experimental program reported in Reference 28. Lacking reliable information, the required dimensions and the allowable loads should be determined by special tests.

C. MEMBERS

To simplify the use of the *Specification*, all design provisions relative to a specific member type, e.g., beam, column or beam–column, have been assembled in one location within the *Specification*. Also, design provisions are given in terms of allowable load or moment, instead of allowable stress, as was the approach in previous specifications. To enable a clearer understanding of the phenomenon being evaluated, the nominal capacity and required safety factor are explicitly stated.

C1 Properties of Sections

The geometric properties of a member, i.e., area, moment of inertia, section modulus and radius of gyration, shall be evaluated using conventional methods of structural design. These properties are based upon either full cross–section dimensions, effective widths or net section, as applicable.

For flexural members and axially loaded members both the full and effective dimensions are used. The full dimensions are used when calculating the critical load or moment, while the effective dimensions, evaluated at the stress corresponding to the critical load or moment, are used to calculate the nominal capacity. References 29 and 35 discuss this concept in more detail.

The net section is employed when computing the capacity of a tension member.

C2 Tension Members

There is a very limited amount of data regarding the capacity of cold–formed steel tension members. Because the provisions of the 1980 AISI *Specification* have been field tested with no known deficiency, they have been carried forward to the 1986 *Specification*.

C3 Flexural Members

The provisions contained in Section C3 of the *Specification* address the various design aspects related to cross–section capacity, that is, flexure strength, shear strength, combined bending and shear strength, web crippling strength and combined bending and web crippling strength. For brace design, see *Specification* Section D3.

C3.1 Strength for Bending Only

Flexural strength is a function of the geometry of the member in question, as well as the degree of lateral restraint, i.e., bracing, provided. The provisions of *Specification* Section C3.1 encompass these considerations.

C3.1.1 Nominal Section Strength

Beams not subject to lateral (flexural), torsional or torsional–flexural buckling, are designed on the basis of the yield strength being achieved in the extreme fibers. Depending upon the cross–section geometry, initial yielding may occur in either the extreme tension or compression fibers. This is the premise in Section C3.1.1a, which is consistent with previous editions of the *Specification*.

For certain beam members, depending upon geometry, the potential exists for partial yielding of the cross section. The 1980 AISI *Specification* introduced guidelines which enabled design engineers to take advantage of this increased moment capacity. The provisions of the 1986 *Specification* have been modified by removing the requirement that the depth–to–thickness ratio of the entire web does not exceed

$640/\sqrt{F_y}$. This restriction had been adopted from the AISC Specification (Reference 10) and was based on research on the plastic rotation capacity of hot-rolled, doubly symmetric wide-flange beams. For typical cold-formed sections, this limitation is redundant because the requirement of *Specification* Section C3.1.1b(3), the ratio of the depth of the compression portion of the web to its thickness not exceed $\lambda_1 = 1.11\sqrt{F_y / E}$ is the governing restriction. A detailed discussion of the inelastic reserve capacity of flexural members is given in Reference 36. The development of this procedure is described in Reference 29.

C3.1.2 Lateral Buckling Strength

The design expressions for laterally unbraced segments of flexural members in *Specification* Section C3.1.2 represent an improvement over previous specifications. The approach that has been adopted enables direct consideration of the interaction between local and overall buckling by a reduction of the critical moment. This reduction is equal to the ratio of the effective section modulus to the full section modulus.

Unlike previous specifications, the 1986 *Specification* is expressed in terms of moment instead of stress. The critical moment for I-and Z-sections employ the same critical buckling stress equations that formed the basis for the 1980 AISI *Specification* (References 23 and 37–40).

For singly symmetric sections for which the x-axis is the axis of symmetry, new, more theoretically based equations are given. The background for these equations is given in Reference 41.

The case of laterally unbraced compression flanges is treated in Part III, Section 3, of the *Manual*, based on Reference 42. A slightly different approach is given in Reference 43.

The 1989 *Specification* Addendum format has been revised to first present the general lateral buckling equations, followed by the simplified, more restrictive equations. Application of the equations for singly-, doubly-, and point-symmetric sections has also been clarified.

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

For beams having the tension flange attached to deck or sheathing and the compression flange unbraced, e.g., a roof purlin or wall girt subjected to wind suction, the bending capacity is less than a fully braced member, but greater than an unbraced member. This partial restraint is a function of the rotational stiffness provided by the panel-to-purlin connection. The *Specification* contains factors that represent the reduction in capacity from a fully braced condition. These factors are based on experimental results obtained for both simple and continuous span purlins (44, 45, 57, 69, 70).

As indicated in Reference 71, the rotational stiffness of the panel-to-purlin connection is primarily a function of the member thickness, sheet thickness, fastener type and fastener location. For compressed glass fiber blanket insulation of initial thicknesses of zero to six inches, the rotational stiffness was not measurably affected (71). To ensure adequate rotational stiffness of the roof and wall systems designed using the *Specification* provision, Section C3.1.3 explicitly states the acceptable panel and fastener types.

Continuous beam tests were made on three equal spans and the R values were calculated from the failure loads, using as a maximum positive moment, $M = 0.08\ WL^2$.

The provisions of Section C3.1.3 apply to beams on which the tension flange is attached to deck or sheathing and the compression flange is completely unbraced. Beams with discrete point braces on the compression flange may have a bending capacity greater than those completely unbraced. Available data from simple span tests (References 44, 69, 80, 81, 82) indicate that the capacity of certain members with discrete braces may be increased over those without discrete braces.

C3.2 Strength for Shear Only

The *Specification* provisions are applicable for slender webs of beams and decks either with or without stiffeners. These provisions are identical to Section 1.10.5.2 for the design of plate girders and rolled beams in the 1978 AISC Specification (Reference 10). The acceptance of the AISC equations for cold–formed sections is based upon the study summarized in Reference 46.

C3.3 Strength for Combined Bending and Shear

For cantilever beams and continuous beams, high bending stresses often combine with high shear stresses at the supports. In the design of such members, it has been the practice to use *Specification* Equation C3.3–1 to safeguard against buckling of flat webs due to the combination of bending and shear stresses (References 7 and 33). In addition, a new interaction equation (*Eq.* C3.2–2) was included in the 1980 *Specification* for beam webs with adequate transverse stiffeners. The correlations between the test data and both formulas are given in Reference 47. Limitations remain imposed on M_a and V_a as loads, in contrast to removal of limits when stress interaction equations were used.

C3.4 Web Crippling Strength

Section C3.4 of the *Specification* provides design equations to prevent web crippling of flexural members having flat single webs (channels, Z–sections, hat sections, tubular members, roof deck, floor deck, etc.) and I–beams (made of two channels connected back to back, by welding two angles to a channel, or by connecting three channels). Different design equations are used for different loading conditions. As shown in Figure C3.4–1, Equations C3.4–1, C3.4–2, and C3.4–3 are used for end one–flange loading; Equations C3.4–4 and C3.4–5 for interior one–flange loading; Equations C3.4–6 and C3.4–7 for end two–flange loading; and Equations C3.4–8 and C3.4–9 for interior two–flange loading. These design equations are based on experimental evidence (Reference 27) and the distribution of loads or reactions into the web as shown in Figure C3.4–2.

The distribution of loads or reactions into the web as shown in Figure C3.4–2 are independent of the flexural response of the beam. Due to flexure, the point of bearing will vary relative to the plane of bearing resulting in non–uniform bearing load distribution into the web. The value of P_a will vary because of a transition from the interior one–flange loading (Figure C3.4–2(b)) to the end one–flange loading (Figure C3.4–2) condition. These discrete conditions represent the experimental basis on which the design provisions were founded (Reference 27).

C3.5 Combined Bending and Web Crippling Strength

This *Specification* section contains two interaction formulas for the combination of bending and web crippling. These formulas are based on studies at the University of

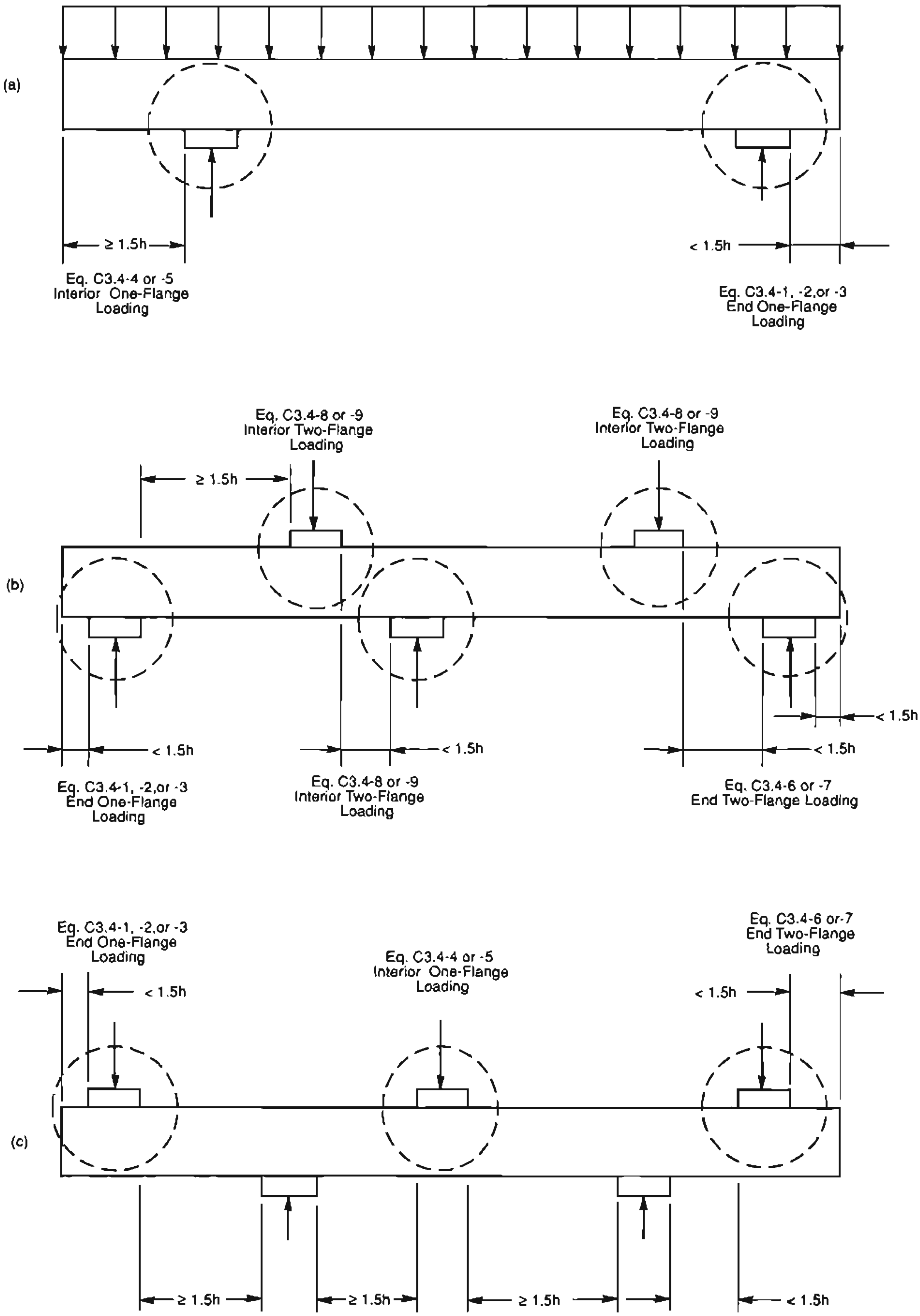


Figure C3.4–1 Application of Design Equations Listed in Table C3.4–1

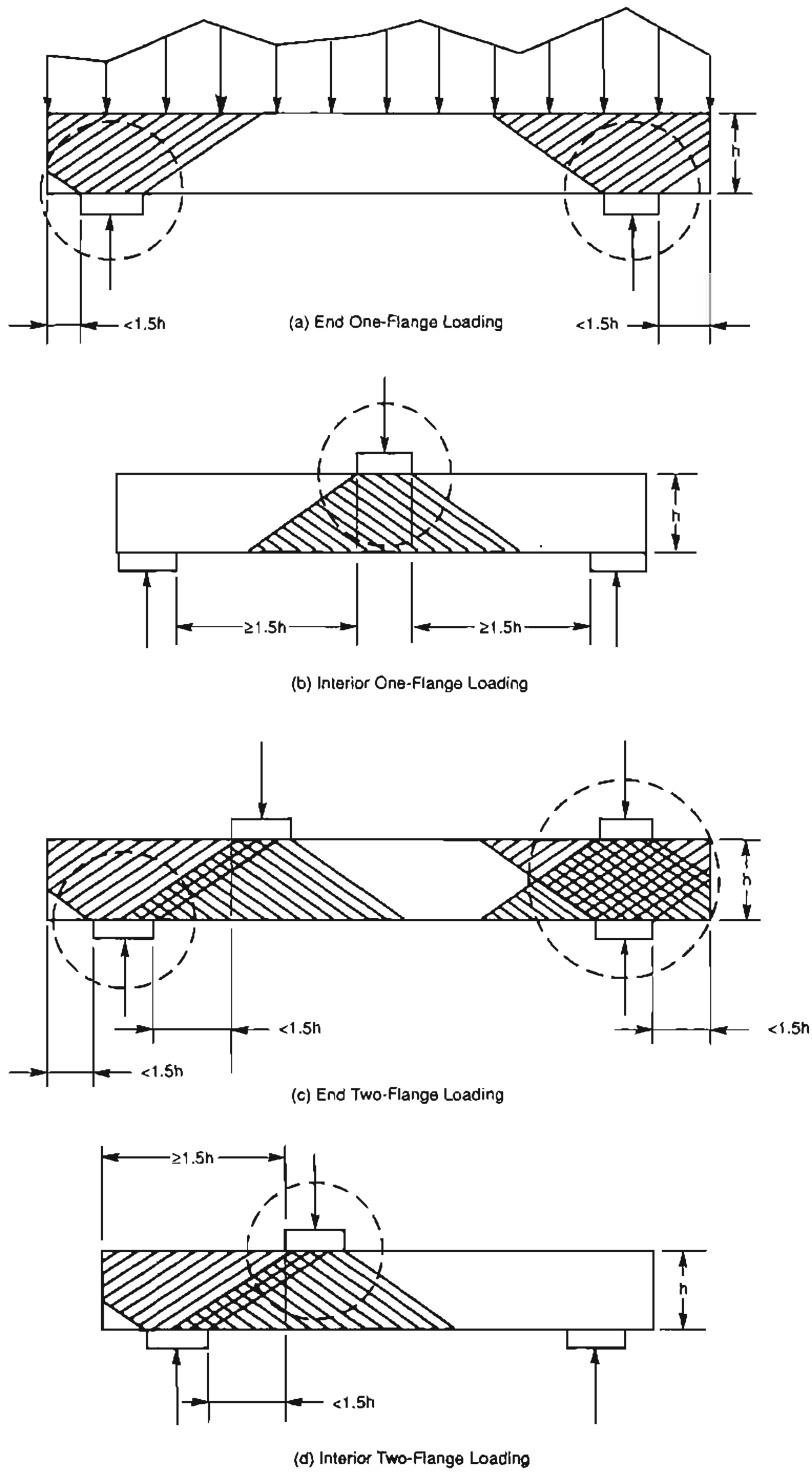


Figure C3.4–2 Assumed Distribution of Reaction Or Load

Missouri–Rolla on the effect of bending on the reduction of web crippling loads (References 23, 27, 48 and 49). For embossed webs, crippling strength should be determined by tests according to *Specification* Chapter F.

The exception clause in *Specification* Section C3.5.1 applies to the interior supports of continuous spans using decks and beams, as shown in Figure C3.5–1. Results of continuous beam tests of steel decks (Reference 49) and several independent studies by manufacturers indicate that, for these types of members, the post–buckling behavior of webs at interior supports differs from the type of failure mode occurring under concentrated loads on single span beams. This post–buckling strength enables the member to redistribute the moments in continuous spans. For this reason, Equation C3.5–1 is not applicable to the interaction between bending and the reaction at interior supports of continuous spans. This exception applies only for the members shown in Figure C3.5–1 and similar situations explicitly described in *Specification* Section C3.5.

The exception clause should be interpreted to mean that the effects of combined bending and web crippling need not be checked for determining load carrying capacity. Furthermore the positive bending resistance of the beam should be at least 90 percent of the negative bending resistance in order to insure the factor of safety implied by the *Specification*.

Using this procedure the allowable loads may (1) produce slight deformations in the beam over the support, (2) increase the actual compressive bending stresses over the support to as high as 0.8 F_y, and (3) result in additional bending deflection of up to 22 percent due to elastic moment redistribution.

If load carrying capacity is not the primary design concern because of the above behaviors, the designer is urged to use Equation C3.5–1.

With regard to Equation C3.5–2, previous tests indicate that when the h/t ratio of an I–beam web does not exceed $2.33/\sqrt{F_y / E}$ and when $\lambda \leq 0.673$, the bending moment has little or no effect on the web crippling load. For this reason, the allowable reaction or concentrated load can be determined by the formulas given in *Specification* Section C3.4 without reduction for the presence of bending.

C4 Concentrically Loaded Compression Members

The provisions of this *Specification* section represent a significant improvement over previous provisions for concentrically loaded compression members subject to either flexural, torsional or torsional–flexural buckling. A unified approach is presented wherein Equations C4–1 through C4–4 are general equations for the three buckling modes. The variation in the three modes is recognized by the appropriate elastic buckling equation as given in *Specification* Sections C4.1 and C4.2.

A simple approach to account for the interaction of local and overall buckling is also included in the provisions of *Specification* Section C4. The critical stress, evaluated for the full section, is the stress level for which the effective area is calculated. Nominal axial strength is then the product of the critical stress and the effective area (Equation C4–2) except that for members with unstiffened flanges the axial strength shall be limited by Equation C4–5. For shapes for which the effective area, A_e, cannot be calculated according to this *Specification*, the stub column test procedure given in Part VII may be used. For a more in–depth discussion of the background for these provisions, see Reference 29.

Specification Section C4(c) is a new provision and accounts for the possibility of a reduction in capacity due to initial sweep of the member.

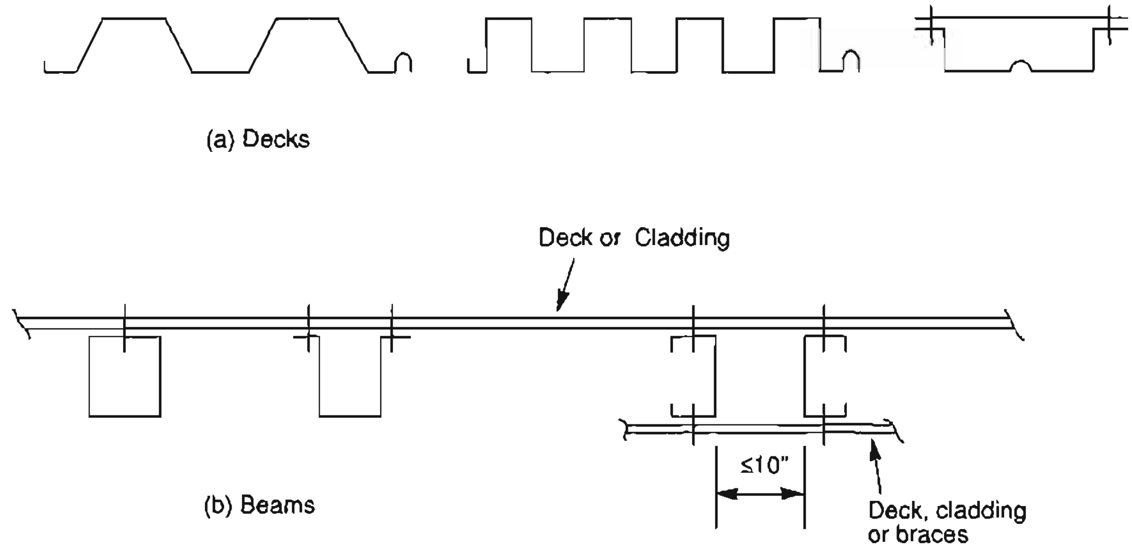


Figure C3.5 Sections used for one Exception Clause of Section C3.5

C5 Combined Axial Load and Bending

The interaction equations contained in Section 3.7.1 of the 1980 AISI *Specification* were for locally stable sections and are identical to the AISC Specification requirements (Reference 10). Cornell University researchers have extended the use of these interaction equations to locally unstable sections (Reference 29). This enables the 1986 *Specification* to present a consistent set of interaction equations applicable for all cold–formed steel shapes.

The limitation that $C_m \geq 0.4$ has been deleted to correct an oversight identified in development of AISC's LRFD *Specification for Steel Buildings*. See the *Commentary* on that specification for the detailed reasoning for this change (Reference 50).

For angle sections, the applied moment M_y accounts for the possibility that when the load is applied between the centroid of the full section and the shear center, a lower axial load capacity may occur.

Equation C5–2 has been modified from the 1986 *Specification* since lateral–torsional buckling must be considered in all cases to represent a proper strength envelope for a beam–column member. Otherwise, it is possible that for low compression loads, the simple beam bending capacity can be exceeded.

C6 Cylindrical Tubular Members

The design provisions for cylindrical tubular members has been expanded to include explicit guidelines for members subject to either axial compression or flexure.

C6.1 Bending

For thick cylinders in bending, the initiation of yielding does not represent a failure condition as is generally assumed for axial loading. Failure is at the plastic moment capacity which is at least 1.29 times the moment at first yielding. In addition, the conditions for inelastic local buckling are not as severe as in axial compression due to the stress gradient.

Equations C6.1–2, –3, and –4 are based upon the work reported in Reference 51 and an assumed minimum shape factor of 1.29. A slight further reduction in the inelastic

range has been made to limit the maximum bending stress to 0.75 F_y, a value typically used for solid sections in bending. The reduction also brings the criteria closer to a lower bound for inelastic local buckling. A small range of elastic local buckling has been included so that the upper D/t limit of 0.441 E/F_y is the same as for axial compression.

C6.2 Compression

Previous editions of the *Specification* contained no provisions for the interaction of local and column buckling. The provisions of *Specification* Section C6.2 include this interaction. The basis for the interaction is the approach recommended in the third edition of the SSRC guide (52), where the critical stress for local buckling is substituted for the yield stress in the column equation. The critical stress computed by Equation C6.2–3 is the same equation as used for overall buckling in previous editions of the *Specification*, except that the factor of safety has been removed. The effective area defined by Equation C6.2–6 converts the results into a design approach consistent with that for concentrically loaded compression members (*Specification* Section C4). This equation provides results that are within a few percent of the SSRC approach.

C6.3 Combined Bending and Compression

Sherman (Reference 53) has confirmed the applicability of linear interaction equations for cylindrical tubular members, including situations where local buckling may occur.

D. STRUCTURAL ASSEMBLIES

D1 Built-Up Sections

D1.1 I-Sections Composed of Two Channels

I-sections made by connecting two channels back to back often are used as either compression or flexural members. For the I-sections to be used as compression members, the longitudinal spacing of connectors must not exceed the value of s_{max}, computed by using Equation D1.1-1 of the *Specification*. This prevents flexural buckling of the individual channels about the axis parallel to the web at a load smaller than that at which the entire I-section would buckle. This provision is based on the requirement that the slenderness ratio of an individual channel between connectors, s_{max}/r_{cy}, not be greater than one-half of the pertinent slenderness ratio, L/r_I, of the entire I-section (References 7 and 23). This accounts for one of the connectors becoming loose or ineffective.

Even though Section D1 of the *Specification* refers only to I-sections, Equation D1.1 also can be used for determining the maximum spacing of welds for box-shaped compression members made by connecting two channels tip to tip. In this case, r_I is the smaller of the two radii of gyration of the box-shaped section.

For the I-sections to be used as flexural members, the longitudinal spacing of connectors is limited by Equations D1.1-2 and D1.1-3 of the *Specification*. The first requirement (Equation D1.1-2) is an arbitrarily selected limit to prevent any possible excessive distortion of the top flange between connectors. The second (Equation D1.1-3) is based on the strength and arrangement of connectors and the intensity of the load acting on the beam. Reference 23 provides further discussion on this topic.

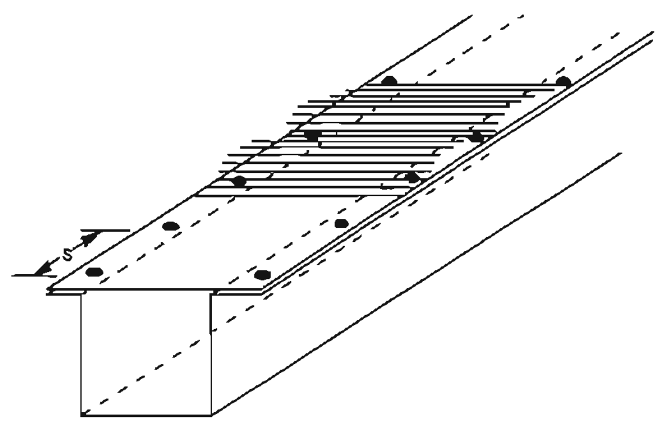


Figure D1.2-1 Spacing of Connections in Compression Elements

D1.2 Spacing of Connections in Compression Elements

When compression elements are joined to other parts of built-up members by intermittent connections, these connections must be closely spaced to develop the required strength of the connected element. Figure D1.2-1 shows a box-shaped beam made by connecting a flat sheet to an inverted hat section. If the connectors are appropriately placed, this flat sheet will act as a stiffened compression element with a width, w, equal to the distance between rows of connectors, and the sectional properties can be calculated accordingly. This is the intent of the provisions in Section D1.2.

Section D1.2(a) of the *Specification* requires that the necessary shear strength be provided by the same standard structural design procedure that is used in calculating flange connections in bolted or welded plate girders or similar structures.

Section D1.2(b) of the *Specification* ensures that the part of the flat sheet between two adjacent connectors will not buckle as a column (see Figure D1.2–1) at a stress less than 1.67f, where f is the design stress of the connected compression element (Reference 23).

Section D1.2(c) ensures satisfactory spacing to make a row of connections act as a continuous line of stiffening for the flat sheet under most conditions (References 7 and 23).

D2 Mixed Systems

When cold–formed steel members are used in conjunction with other construction materials, the design requirements of the other material specifications also must be satisfied.

D3 Lateral Bracing

Bracing design requirements have been expanded in the 1986 *Specification* to include a general statement regarding bracing for symmetrical beams and columns and specific requirements for the design of roof systems subjected to gravity load.

D3.1 Symmetrical Beams and Columns

There are no simple, generally accepted techniques for determining the required strength and stiffness for discrete braces in steel construction. Winter (Reference 54) offered a partial solution and others (References 43, 55–57) have extended this knowledge. The design engineer is encouraged to seek out the stated references to obtain guidance for design of a brace or brace system.

D3.2 Channel–Section and Z–Section Beams

Channel and Z–sections used as beams to support transverse loads applied in the plane of the web may twist and deflect laterally unless adequate lateral supports are provided. Section D3.2 of the *Specification* includes two subsections. The first (Section D3.2.1) is a new provision that deals with the bracing requirement when one flange of the beam is connected to deck or sheathing material. The second subsection (Section D3.2.2) covers the requirements for spacing and design of braces, when neither flange of the beam is braced by deck or sheathing material.

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

In conventional metal building roof systems, the roof panels are attached to the top flange of each purlin throughout its length using self–drilling or self–tapping, through–the–sheet, fasteners spaced at approximately 1 foot on center. This panel usually provides sufficient stiffness to prevent the relative movement of the purlins with respect to each other, however, unless external restraint is provided, the system as a whole will tend to move laterally. This restraint or anchorage may consist of members attached to the purlin at discrete locations along the span and designed to carry forces necessary to restrain the system against lateral movement. The design rules for Z–purlin supported roof systems are based on a first order, elastic stiffness model (Reference 58).

D3.2.2 Neither Flange Connected to Sheathing

When neither flange is braced by deck or sheathing material, discrete bracing must be provided. Section D3.2.2 specifies the spacing of such braces, applicable to both channel and Z–beams, and the forces for which these braces must be designed. A detailed discussion of the background for these provisions is contained in Reference 23.

An exception in the *Specification* permits omission of discrete braces when all loads and reactions on a beam are transmitted through members that frame into the section in such a manner as to effectively restrain the member rotation and lateral displacement. Frequently this occurs in the end walls of metal buildings.

D3.3 Laterally Unbraced Box Beams

Box beams are more laterally stable than single–web sections having the same depth/width ratio. *Specification* Section D3.3 specifies that when the ratio of the laterally unsupported length to the distance between the webs of the section (L/b) does not exceed 0.086 E/F_y, the box sections can be used without any strength reduction to account for lateral buckling. This design requirement is based on Part II of Reference 8 which indicates that the failure stress in box beams is practically unaffected by lateral buckling up to L/b as high as 100 for a steel having a yield point of 33 ksi. On the basis of this AISI design criterion, when the box beams are bent about the major axis, the laterally unsupported length shall not exceed 75 times the distance between webs for a steel with $F_y = 33$ ksi. This conservative design approach is identical with the previous edition of the AISI *Specification.*

D4 Wall Studs and Wall Stud Assemblies

References 59 and 60 which are summarized in Reference 23, contain procedures for computing the strength of C– and Z–section wall studs that are braced by sheathing materials. The bracing action is due to both the shear rigidity and the rotational restraint supplied by the sheathing material. The treatment in References 59 and 60 is quite general and includes the case of studs braced on one as well as on both flanges. However, the provisions of Section D4 deal with the simplest case of identical sheathing material on both sides of the stud. For simplicity, only the restraint due to the shear rigidity of the sheathing material is considered. Other cases, such as studs with sheathing on one flange only, dissimilar sheathing materials on each flange, or the evaluation of the effect of rotational restraint, are covered in footnotes to *Specification* Sections D4.1 and D4.2.

The 1989 Addendum moves the design limitations from the *Commentary* to the *Specification*, and introduces stub column tests and/or rational analysis for design of studs with perforations (References 72 and 73).

D4.1 Wall Studs in Compression

The provisions in this *Specification* section are given to prevent three possible modes of failure. Provision (a) is for column buckling between fasteners, even if one fastener is missing or otherwise ineffective. Provision (b) contains formulas for allowable axial stresses for overall column buckling. Essential to these provisions is the magnitude of the shear rigidity of the sheathing material. A Table of values and an Equation for determining the shear rigidity is provided in the *Specification*. These values are based on the small scale tests described in References 59 and 60. For other types of materials, the sheathing parameters can be determined using the procedures described in these references.

Provision (c) is to insure that the sheathing has sufficient distortion capacity. The procedure involves assuming a value of the ultimate stress and checking whether the shear strain at the load corresponding to the ultimate stress exceeds the allowable design shear strain of the sheathing material. In principle, the procedure is one of successive approximations. However, if the smaller of F_e (provision a) or σ_{cr} (provision b) is tried and shown to be satisfactory, then the need for iteration is eliminated.

In the 1986 *Specification*, the Q method for treating the local buckling effects is eliminated. The approach recommended is to find the overall buckling stress on the basis of the full unreduced section. The ultimate loads are determined by multiplying the buckling stress by the effective area determined at the buckling stress.

In the 1989 Addendum, the effective length factors K_x, K_y, and K_t have been eliminated from Equations D4.1-8, D4.1-10, D4.1-11, respectively. This is consistent with the 1980 Edition of the *Specification*. Inclusion of the effective length factors could lead to unconservative designs where both sheathing and strap or channel bracing are present. The equations are based on tests with only sheathing as bracing.

D4.2 Wall Studs in Bending

This provision modifies the 1980 *Specification*, in that a limitation on the bending moment has been deleted. Tests of long, deep studs have confirmed the excess conservatism of the 1980 provision.

D4.3 Wall Studs with Combined Axial Load and Bending

The general interaction equations of *Specification* Section C5 are also applicable to wall studs with the exception that the allowable moments be evaluated excluding lateral buckling considerations.

D5 Floor, Roof or Wall Steel Diaphragm Construction

The structural performance of a diaphragm construction can be evaluated by either calculation or test. Several analytical procedures exist, and are summarized in References 74, 75 and 76. Tested performance is measured by the procedures of the Standard Method for Static Load Testing of Framed Floor, Roof and Wall Diaphragm Construction for Buildings, ASTM E455. A general discussion of structural diaphragm behavior is given in Reference 23.

The factors of safety required in the *Specification* are based on statistical studies of the nominal and mean resistances from full scale tests (77). The study concluded that the quality of mechanical connectors is easier to control than welded connections. The variation in the strength of mechanical connectors is smaller than for welded connections, and their performance is more predictable. Therefore, a smaller factor of safety is justified for mechanical connections. The factors of safety for earthquake loading are slightly larger than those for wind due to the ductility demands required by seismic loading. Factors of safety for load combinations not involving wind or seismic load should be greater than those involving wind and seismic loads, thus the *Specification* provides for appropriate factors of safety.

E. CONNECTIONS AND JOINTS

E1 General Provisions

The *Specification* contains provisions only for welded and bolted connections. Other means of connection find application in cold–formed construction and some of these are listed below:

(a) *Rivets*

While hot rivets have little application in cold–formed construction, cold rivets find considerable use, particularly in special forms, such as blind rivets (for application from one side only), tubular rivets (to increase bearing area), high shear rivets, and explosive rivets.

(b) *Screws*

These are usually self–drilling and self–tapping, and come in a wide variety of shapes, dimensions, and details.

(c) *Special devices*

These include: (1) metal stitching, achieved by tools that are special developments of the common office stapler, and (2) connecting by means of special clinching tools that draw the sheets into interlocking projections.

Most of these connections are proprietary devices for which information on strength of connections must be obtained from manufacturers or from tests carried out by or for the user. Guidelines provided in *Specification* Chapter F are to be used in these tests.

The plans and/or specifications are to contain adequate information and design requirement data for the adequate detailing of each connection if each connection is not detailed on the engineering design drawings.

E2 Welded Connections

The provisions contained in this *Specification* section have been primarily based on experimental evidence obtained in an extensive test program at Cornell University. The results of this program are summarized in References 23 and 61. All possible failure modes are covered in the present provisions, whereas the earlier provisions mainly dealt with shear failure.

For most of the connection tests reported in Reference 61, the onset of yielding was either poorly defined or followed closely by ultimate failure. Therefore, in the provisions of this section, rupture rather than yielding is used as a more reliable criterion of failure.

In addition, the Cornell research has provided the experimental basis for Reference 62. In most cases, the provisions of Reference 62 are in agreement with this *Specification* section.

The tests, which served as the basis of the provisions given in *Specification* Section E2.1 through E2.5, were conducted on sections with single and double sheets (*Specification* Figure E2.2). The largest total sheet thickness of the cover plates was approximately 0.15 inch. Within the *Specification*, the validity of the equations was extended to connections with cover plates of 0.18 inch combined thickness. For arc spot welds the maximum combined thickness is set at 0.15 inch.

The terms used in this *Specification* section agree with the standard nomenclature given in Reference 62.

E2.2 Arc Spot Weld

The thickness limitation is due to the range of the test program that served as the basis of these provisions.

On sheets below 0.028 inch thick, weld washers are required to avoid excessive burning of the sheets and, therefore, inferior quality welds.

The Cornell tests identified four modes of failure for arc spot welds, which are addressed in this *Specification* section. It should be noted that many failures, particularly those of the plate tearing type, may be preceded or accompanied by considerable inelastic out–of–plane deformation of the type indicated in Figure E2.1. This form of behavior is similar to that observed in wide, pin–connected plates. Such behavior should be avoided by closer spacing of welds.

For tension loads on arc spot welds, tests were performed which included variables of steel strengths from 50 to 68.5 ksi, steel sheet thickness ranging from 0.031 to 0.072 inches, galvanized steel sheet, prime painted and galvanized steel plate, and visible diameter of welds ranging from 0.47 to 0.94 inches. Thus, the additional limitations only applicable to arc spot welds in tension were included in the *Specification*. For use in calculating the tension load on arc spot welds, the tensile strength of the connected sheet, F_u, is limited to a maximum of 60 ksi, although sheet with greater tensile strengths may be used. The development of the equation is contained in Reference 78. The tests were reported in Reference 79.

E2.3 Arc Seam Welds

The general behavior of arc seam welds is similar to that of arc spot welds. No simple shear failures of arc seam welds were observed in the Cornell tests. Therefore, Equation E2.3–1 which accounts for shear failure, is adopted from Reference 62.

Equation E2.3–2 is intended to prevent failure by a combination of tensile tearing plus shearing of the cover plates.

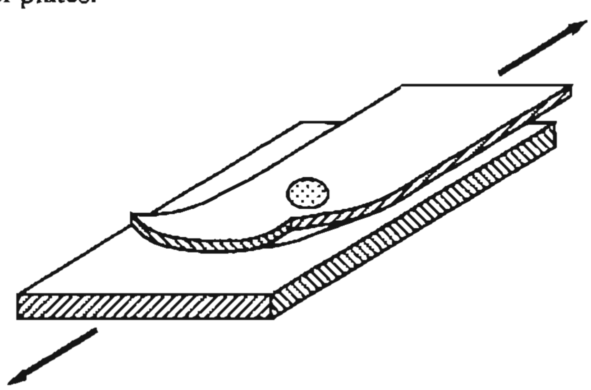


Figure E2.1 Out–of–Plane Distortion

E2.4 Fillet Welds

For fillet welds on the lap–joint specimens tested in the Cornell research, the dimension, w_1, of the leg on the sheet edge generally was equal to the sheet thickness; the other leg, w_2, often was two or three times longer than w_1. In connections of this type, the fillet weld throat commonly is larger than the throat of a conventional fillet weld of the same size (see *Specification* Figure E2.4a). Usually ultimate failure of fillet welded joints is found to occur by the tearing of the plate adjacent to the weld.

In most cases, the higher strength of the weld material prevents weld shear failure, therefore the provisions of this *Specification* section are based on sheet tearing. Because specimens up to 0.15 inch thickness were tested in the Cornell research, the last provision in this section is to cover the possibility that for sections thicker than 0.15 inch, the throat dimension may be less than the thickness of the cover plate and the tear may occur in the weld rather than in the plate material.

E2.5 Flare Groove Welds

The chief mode of failure in cold-formed channels welded by flare groove welds, loaded transversely or longitudinally, also was sheet tearing along the contour of the weld.

Except for Equation E2.5-4, the provisions of this *Specification* section are intended to prevent shear tear failure. This equation covers the possibility that thicker channels may have effective throats less than the thickness of the channel and weld failure may become critical.

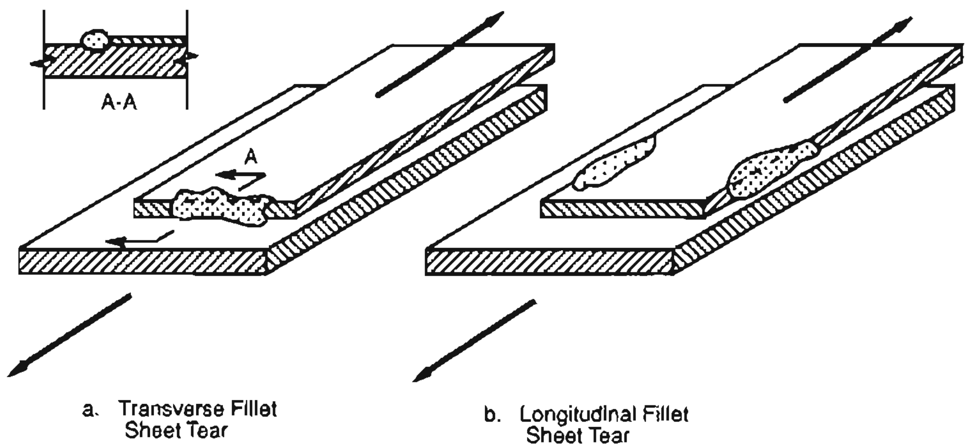


Figure E2.4 Fillet Weld Failure Modules

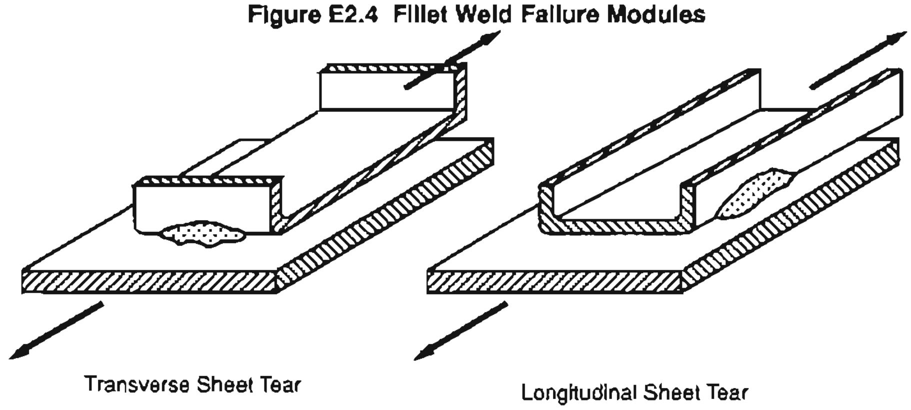


Figure E2.5 Flare Groove Weld

E2.6 Resistance Welds

The shear values listed for outside sheets of 0.125 inches or less in thickness are based on "Recommended Practice for Resistance Welding Coated Low-Carbon Steels," AWS C1.3-70, (Table 2.1 – Spot Welding Galvanized Low-Carbon Steel). Shear values for outside sheets thicker than 0.125 are based upon "Recommended Practices for Resis-

tance Welding," AWS C1.1–66, (Table 1.3 – Pulsation Welding Low–Carbon Steel) and apply to pulsation welding as well as spot welding. They are applicable for all structural grades of low–carbon steel, uncoated or galvanized with 0.90 oz/ft^2 of sheet, or less, and are based on values selected from AWS C1.3–70 Table 2.1 and AWS C1.1–66 Table 1.3. Values for intermediate thicknesses may be obtained by straight–line interpolation. The above values may also be applied to medium–carbon and low–alloy steels. Spot welds in such steels give somewhat higher shear strengths than those upon which the above values are based; however, they may require special welding conditions. In all cases, welding shall be performed in accordance with AWS C1.3–70 and AWS C1.1–66. (References 63 and 64)

E3 Bolted Connections

The AISI design provisions for bolted connections were revised in the *Specification* to reflect the results of continuing research (References 23 and 65) and general practice as well as to provide a better coordination with the specifications of the Research Council on Structural Connections (RCSC) (Reference 66) and AISC (Reference 10). In the revised design provisions, several additions were made concerning the maximum size of holes (Table E3) and the allowable tension stress for bolts.

(a) *Scope*

Previous studies and practical experiences have indicated that the structural behavior of bolted connections used for joining relatively thick cold–formed steel members is similar to that for connecting hot–rolled shapes and built–up members. The *Specification* criteria are applicable only to cold–formed steel members or elements less than $^3/_{16}$ inch in thickness. For materials not less than $^3/_{16}$ inch, reference is made to the AISC Specification (Reference 10).

Because of a lack of appropriate test data and the use of numerous surface conditions, this *Specification* does not provide design criteria for slip–critical (also called friction–type) connections. When such connections are used with cold–formed members where the thickness of the thinnest connected part is less than $^3/_{16}$ inch, it is recommended that tests be conducted to confirm their design capacity. The test data should verify that the specified design capacity for the connection provides a sufficient safety factor against initial slip at least equal to that implied by the AISC provisions. In addition, the safety factor against ultimate capacity should be at least equal to that implied by this *Specification* for bearing–type connections.

The *Specification* provisions apply only when there are no gaps between plies. The designer should recognize that the connection of a rectangular tubular member by means of bolt(s) through such members may have less strength than if no gap existed. Structural performance of connections containing unavoidable gaps between plies would require tests in accordance with *Specification* Section F1.

(b) *Materials*

This section lists five different types of fasteners which are normally used for cold–formed steel construction. In view of the fact that A325 and A490 bolts are available only for diameters of $^1/_2$ inch and larger, A449 and A354 Grade BD bolts should be used as an equivalent of A325 and A490 bolts, respectively, whenever smaller bolts (less than $^1/_2$ inch in diameter) are required.

During recent years, other types of fasteners, with or without special washers, have been widely used in steel structures using cold–formed members. The allowable stresses for design of these fasteners should be determined by tests in accordance with Section F of this *Specification*.

(c) *Bolt Installation*

Bolted connections in cold–formed steel structures use either mild or high–strength steel bolts and are designed on the basis of bearing strength. Bolt pretensioning, as is customary in hot–rolled steel construction, is not required because the ultimate strength of a bolted connection is independent of the level of bolt preload. Installation is therefore essentially limited to ensuring that the bolted assembly will not come apart during service. Experience has shown that bolts installed to a snug tight condition do not loosen or "back–off" under normal building conditions.

Bolts in slip–critical connections must be tightened in a manner which assures the development of the fastener tension forces required by AISC for the particular size and type of bolts. Turn–of–nut rotations specified by AISC may not be applicable. This is because such rotations are based on larger grip lengths than are encountered in usual cold–formed construction. Use of load indicator washers is also a possibility for tightening slip–critical connections.

(d) *Hole Sizes*

In Table E3, the maximum size of holes for bolts having diameters not less than 1/2 inch is based on Table 1.23.4 of the AISC Specification, except that for the oversized hole diameter, the AISC Table permits a slightly larger diameter.

For bolts having diameters less than 1/2 inch, the maximum size of bolt holes is based on previous editions of the AISI *Specification*.

The inclusion of the design information in Table E3 for oversized and slotted holes is because such holes are often used in practice to meet dimensional tolerances during erection. However, when using oversized holes care must be exercised by the designer to ensure that excessive deformation due to slip will not occur at working loads.

Short–slotted holes are usually treated in the same manner as oversized holes. Washers or backup plates should be used over oversized or short–slotted holes in an outer ply unless suitable performance is demonstrated by tests. For connections using long–slotted holes, *Specification* Section E3.4 required the use of washers or back–up plates and that the shear capacity of bolts be determined by tests because a reduction in strength may be encountered.

E3.1 Spacing and Edge Distance

The AISI provisions for minimum spacing and edge distance are the same as that included in the 1980 *Specification*. The minimum edge distance of each individual connected part, e_{min}, is determined by using the tensile strength of steel (F_u) of the connected part and the F_u/F_y ratio (References 23 and 65).

E3.2 Tension in Connected Part

In the revised criteria, the formulas used for computing the allowable tension force on the net section of connected parts are based on the yield point of steel, F_y, and the tension stress, F_t, depending on the type of joint and the use of washers. The appropriate factor of safety should be used for determining the allowable load.

E3.3 Bearing

The allowable bearing forces determined on the basis of the nominal bearing stresses given in Tables E3.3–1 and –2 are the same as that permitted by the 1980 *Specification*. For the thicknesses of materials not covered in these two tables, the allowable bearing force must be determined by tests in accordance with Section F of the *Specification*, using a safety factor of 2.22.

E3.4 Shear and Tension in Bolts

In the 1986 *Specification*, allowable tension stresses were added in Section E3.4 for bolts subject to tension. The allowable stresses specified for A307 ($d \geq 1/2$ inch), A325, and A490 bolts are based on Section 1.5.2.1 of the AISC Specification (Reference 10). For A307, A449, and A354 bolts with diameters less than $1/2$-inch, the allowable tension stresses are reduced by 10 percent, as compared with A307, A325 and A490 bolts with diameters not less than $1/2$-inch, because the average ratio of tensile-stress area/gross-area for $1/4$-inch and $3/8$-inch diameter bolts is 0.68, which is about 10 percent less than the average area ratio of 0.75 for $1/2$-inch and 1-inch diameter bolts.

The possibility of pullover of the connected sheet at the bolt head, nut, or washer should be considered, especially in the case of sheathing. For unsymmetrical sections, such as C- and Z-sections used as purlins or girts, the problem is more severe because of the prying action resulting from rotation of the member which occurs as a consequence of loading normal to the sheathing. The designer should refer to applicable product code approvals, product specifications or literature, or tests.

In addition, design provisions for bolts subjected to a combination of shear and tension were added in *Specification* Section E3.4. The design equations given in Table E3.4 are based on Section 1.6.3 of the AISC Specification for the design of bolts used in bearing-type connections. The design equations used for A354, A449, and A307 bolts with $d > 1/2$ inch were derived from the following Equation:

$$F'_t = 1.25\,F_t - Af_v \leq F_t$$

in which

F'_t = Reduced allowable tension stress for bolts subjected to a combination of shear and tension

F_t = Allowable tension stress for bolts subject only to tension

A = 1.8 for threads not excluded from shear planes

A = 1.4 for threads excluded from shear planes

f_v = Shear stress in bolt

E4 Shear Rupture

The provisions for allowable shear rupture have been adopted from the AISC *Specification for the Design, Fabrication and Erection of Structural Steel for Buildings* (Nov. 1, 1978). These provisions are considered to yield a conservative estimate of the shear rupture capacity at the coped end of a beam.

E5 Connections to Other Materials

E5.1 Bearing

The allowable stresses for bearing on masonry have been adopted from the November 1, 1978, edition of the AISC Specification.

E5.2 Tension

This *Specification* section has been added to raise the awareness of the design engineer regarding tension on fasteners and the connected parts.

E5.3 Shear

This *Specification* section has been added to raise the awareness of the design engineer.

F. TESTS FOR SPECIAL CASES

Information on tests for cold–formed steel diaphragms can be found in Reference 67. A general discussion of structural diaphragms is given in Reference 23.

F1 Tests for Determining Structural Performance

This *Specification* section makes provision for proof of structural adequacy by load tests. This section is restricted to cases where calculation of safe load–carrying capacity of deflection cannot be made in accordance with the provision of this *Specification*.

There are in cold–formed steel (as in other kinds of structures) perfectly acceptable and safe types of construction whose composition or configuration are not covered by provisions of the *Specification*. Their performance and adequacy, therefore, cannot be demonstrated by reference to the *Specification*. For example, apart from those methods of connection covered in the *Specification*, a number of other means of connecting are in use. The fact that these are not specifically covered in the *Specification* is not intended to exclude their use. However, since structures so connected cannot be calculated according to the *Specification* (at least as to strength of connections), tests according to Chapter F are the only means of supplying proof of structural adequacy. Other similar examples could be cited.

Provision (b) prescribes that the structure, when tested, shall support without failure or harmful distortions, design loads increased by load factors. This is to provide reasonable safety factors against failure or harmful distortions. The safety factor thus obtained is somewhat larger than the basic factor of safety of 1.67 on which the body of the *Specification* is based. This takes into account the uncertainties in translating test results into reliable carrying capacities as mentioned above. Additionally, in some instances, the factors of safety in the *Specification* are larger than 1.67 as can be seen in Table A5–1. For connections, the factors of safety implied in the *Specification* are significantly larger than 2. Therefore, provision (b) is also aimed at ensuring factors of safety consistent with those implied in the *Specification* for connections.

Provision (c) applies when the strength of the test specimen is greater than the minimum specified steel strength. This requires that the test results must be reduced in the ratio of the actual strength to the specified minimum strength to obtain the load–carrying capacity. The provision is self–explanatory regarding similar corrections for tensile strength or sheet thickness.

F2 Tests for Confirming Structural Performance

Members, connections and assemblies which can be designed according to the provisions of Chapter A through E of the *Specification* need no confirmation of calculated results by test. However, special situations may arise where it is desirable to confirm by test the results of calculations. Tests may be called for by the manufacturer, the engineer, or a third party.

Since design was in accordance with the *Specification*, the higher load factors of *Specification* Section F1 do not apply. All that is needed is that the tested specimen or assembly perform according to the safety factor implied in the applicable specification provision. The appropriate value of the applicable safety factor can generally be obtained from the *Specification* or *Commentary* Table A5.1.

F3 Tests for Determining Mechanical Properties

Explicit methods for utilizing the effects of cold work are incorporated in Section A5.2.2 of the *Specification*. In that section, it is specified that as–formed mechanical properties, in

particular the yield strength, can be determined either by full–section tests or by calculating the strength of the corners and computing the weighted average for the strength of corners and flats. The strength of flats can be taken as the virgin strength of the steel before forming, or can be determined by special tension tests on specimens cut from flat portions of the formed section. This *Specification* section spells out in considerable detail the types and methods of these tests, and their number as required for use in connection with *Specification* Section A5.2.2. For details of testing procedures which have been used for such purposes, but which in no way should be regarded as mandatory, see References 7, 19, and 30. A *Stub–Column Test Method for Effective Area of Cold–Formed Steel Columns* is included in Part VII of American Iron and Steel Institute's *Cold–Formed Steel Design Manual*, 1986.

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Washington, DC 20005

AISI
SG-672 0291-7.5M-BPG
Printed in U.S.A.

COMMENTARY

ON THE AUGUST 19, 1986, EDITION OF THE

SPECIFICATION FOR THE DESIGN OF COLD-FORMED STEEL STRUCTURAL MEMBERS

Cold-Formed Steel Design Manual - Part II

AMERICAN IRON AND STEEL INSTITUTE
1000 16th STREET, NW
WASHINGTON, DC 20036

1st Printing—March 1987

Produced by W. P. Reyman Associates, Inc. New York

PREFACE

This document, *Part II* of the *Cold-Formed Steel Design Manual*, provides a commentary on the background for the 1986 Edition of the *Specification for the Design of Cold-Formed Steel Structural Members*

The *Commentary* should be used in conjunction with the other parts of the *Design Manual* which include *Supplementary Information (Part III), Illustrative Examples (Part IV), Charts and Tables (Part V), Computer Aids (Flow Charts) (Part VI)*, and *Test Procedures (Part VII)* in addition to the *Specification (Part I)*.

The careful assistance and close cooperation of the Advisory Group on the *Specification* is gratefully acknowledged. Special thanks are extended to the Subcommittee which drafted and edited this *Commentary;* D. S. Ellifritt (Chairman), R. E. Albrecht, R. A. LaBoube, T. B. Pekoz, D. S. Wolford, W. W. Yu, S. J. Errera and A. L. Johnson.

American Iron and Steel Institute
August 1986

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COMMENTARY OF THE AUGUST 19, 1986, EDITION OF THE SPECIFICATION FOR THE DESIGN OF COLD-FORMED STEEL STRUCTURAL MEMBERS

INTRODUCTION

This *Commentary* is intended to facilitate the use, and provide an understanding of the background, of the 1986 edition of the *Specification for the Design of Cold-Formed Steel Structural Members*. It illustrates the substance and limitations of the various provisions, with particular emphasis on changes made since the 1980 edition. Further, it provides information on the background for the various *Specification* provisions. Mainly, this is accomplished through an extensive integrated bibliography, as well as brief, substantive discussions.

Research on cold-formed steel construction started under the sponsorship of the American Iron and Steel Institute at Cornell University in 1939. Since that time, research has continued at Cornell, as well as other institutions. In 1946 the first edition of the *Specification* was published (Reference 1). This and subsequent editions are listed as References 1 through 9. As Reference 3 indicates, since 1956, the *Specification* has been Part I of the AISI's *Cold-Formed Steel Design Manual*. The other sections of the *Manual* included are: Part II: *Commentary*; Part III: *Supplementary Information*; Part IV: *Illustrative Examples*, and Part V: *Charts and Tables*. Parts II through V illuminate and facilitate the use of the *Specification* for the designer, building official, teacher, and student. The 1986 edition of the *Manual* contains two new sections, Part VI: *Computer Aids (Flow Charts)*, and Part VII: *Test Procedures*.

The *Specification* is coordinated with AISC's *Specification for the Design, Fabrication and Erection of Structural Steel for Buildings* (Reference 10). The latter applies to hot-rolled shapes and built-up members, while the AISI *Specification* deals with members cold formed to shape from flat steel, usually of relatively small thickness. The range of shapes which can be produced by cold-forming processes is practically unlimited.

In the *Commentary*, the individual sections, equations, figures and tables are identified by the same notation as in the *Specification* and the material is presented in the same sequence.

A. GENERAL PROVISIONS

A1 Limits of Applicability and Terms

A1.1 Scope and Limits of Applicability

The *Specification* is limited to steel structural members cold formed from carbon or low-alloy sheet, strip, plate or bar. The forming process is carried out at, or near, room temperature by the use of bending brakes, press brakes or roll-forming machines.

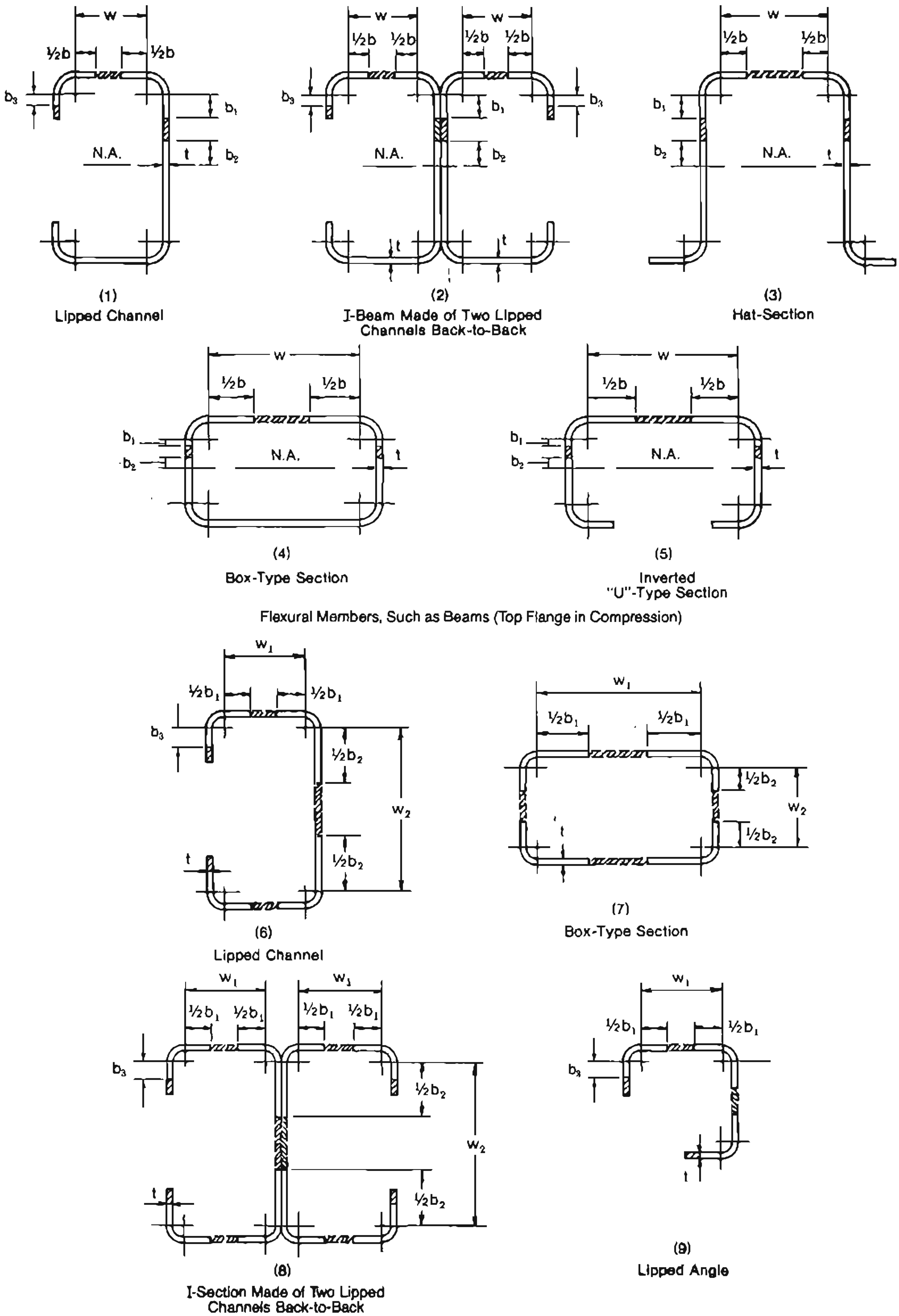
Some of the significant differences between cold-formed sections and hot-rolled shapes are (1) absence of residual cooling stresses, (2) lack of corner fillets, (3) presence of increased yield strength and decreased proportional limit resulting from cold-forming, (4) presence of cold-reducing stresses when cold-rolled steel stock has not been final-annealed, (5) prevalence of elements having large width-thickness ratios, (6) rounded corners, and (7) for some steels different stress-strain curves, some without sharp yield points.

Research conducted at the University of Missouri–Rolla (References 11 and 12) verified the applicability of the *Specification's* provisions for members cold formed of flat steels up to and including one inch in thickness.

A1.2 Terms

Many of the definitions in *Specification* Section A1.2 are self-explanatory. Only those which are not self-explanatory, or not listed, are briefly discussed below.

(a) *Stiffened or Partially Stiffened Compression Elements*
Stiffened compression elements of various sections are shown in Figure A1.2(a). Sections (1) and (2) each have a web and a lip to stiffen the compression element, i.e., the compression flange, the ineffective portion of which is shown shaded. For the explanation of these ineffective portions, see definition (e) Effective Design Width, below, and Chapter B. Sections (3), (4), and (5) show compression elements stiffened by two webs. Sections (6) and (8) show edge stiffened flange elements that have a vertical element (web) and an edge stiffener (lip) to stiffen the elements while the web itself is stiffened by the flanges, (7) has four compression elements stiffening each other, and (9) has each stiffened element stiffened by a lip and the other stiffened element.



Compression Members, such as Columns

Figure A1.2(a) Stiffened Compression Elements Used in Cold-Formed Steel Structural Members

(b) *Unstiffened Compression Elements*

Unstiffened elements of various sections are shown in Figure A1.2(b). Sections (1), (2), and (3) have only a web to stiffen the compression flange element. The legs of (4) provide mutual stiffening action to each other along their common edges. Sections (5), (6), and (7) acting as columns have vertical stiffened elements (webs) which provide support for one edge of the unstiffened flange elements. The legs of (8) provide mutual stiffening action to each other.

(c) *Multiple-Stiffened Elements*

Multiple-stiffened elements of two sections are shown in Figure A1.2(c). Each of the two outer sub-elements of (1) are stiffened by a web and intermediate stiffener while the middle sub-element is stiffened by two intermediate stiffeners. The two sub-elements of (2) are stiffened by a web and the attached intermediate middle stiffener.

(e) *Effective Design Width*

The effective design width is a concept which facilitates taking account of local buckling and post-buckling strength for compression elements. The effect of shear lag on short, wide flanges is also handled by using an effective design width. These matters are treated in *Specification* Chapter B, and the corresponding effective widths are discussed in the *Commentary* on that chapter.

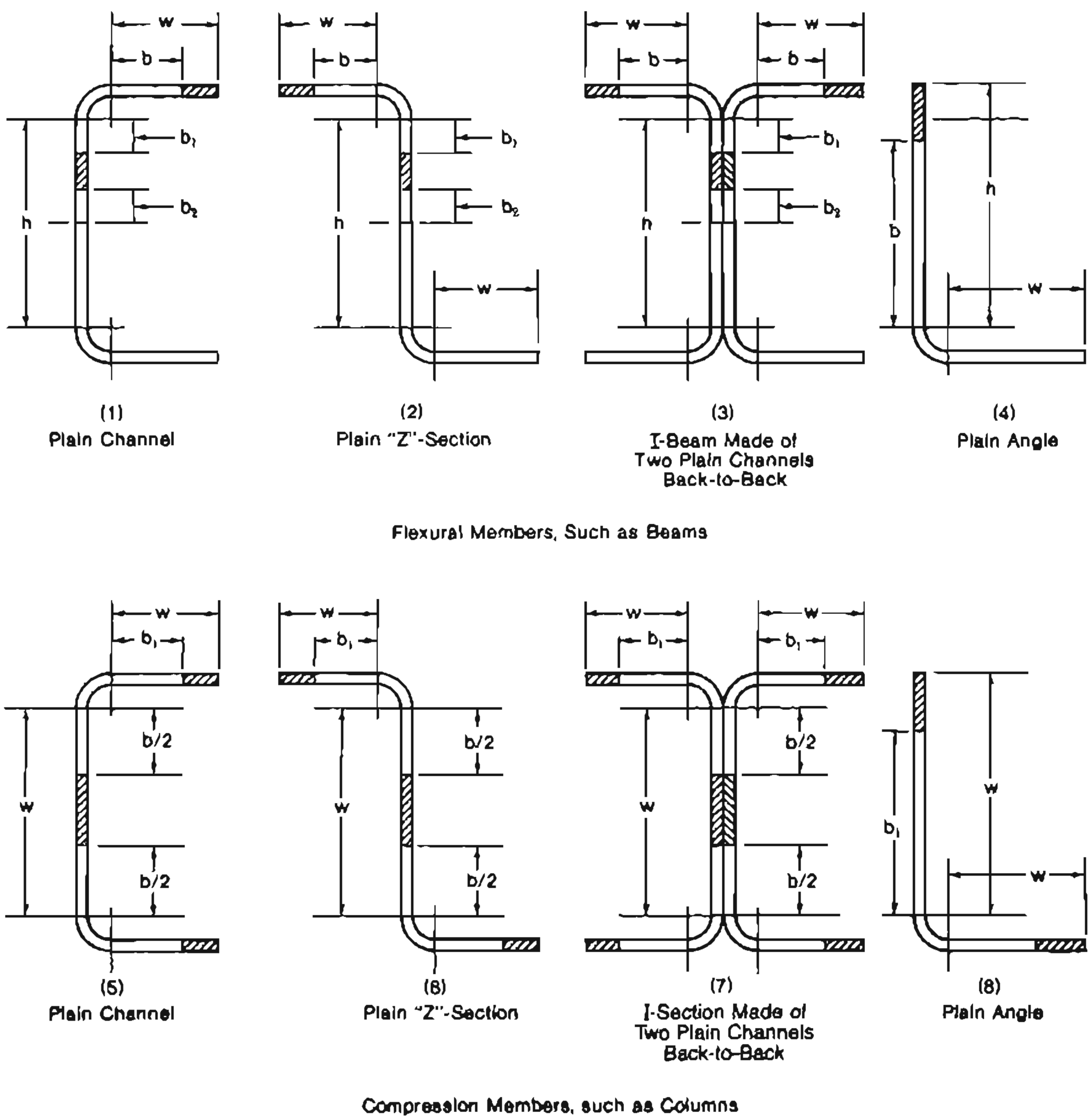


Figure A1.2(b) Unstiffened Compression Elements Used in Cold-Formed Steel Structural Members

(f) *Thickness*
In calculating section properties, the reduction in thickness that occurs at corner bends is ignored, and the thickness of the flat steel stock, exclusive of coatings, is used in all structural calculations.

(g) *Torsional-Flexural Buckling*
The 1968 *Specification* pioneered methods for computing column loads of cold-formed steel sections prone to buckle by simultaneous twisting and bending. This complex behavior results in lower column loads than would result from primary buckling by flexure alone.

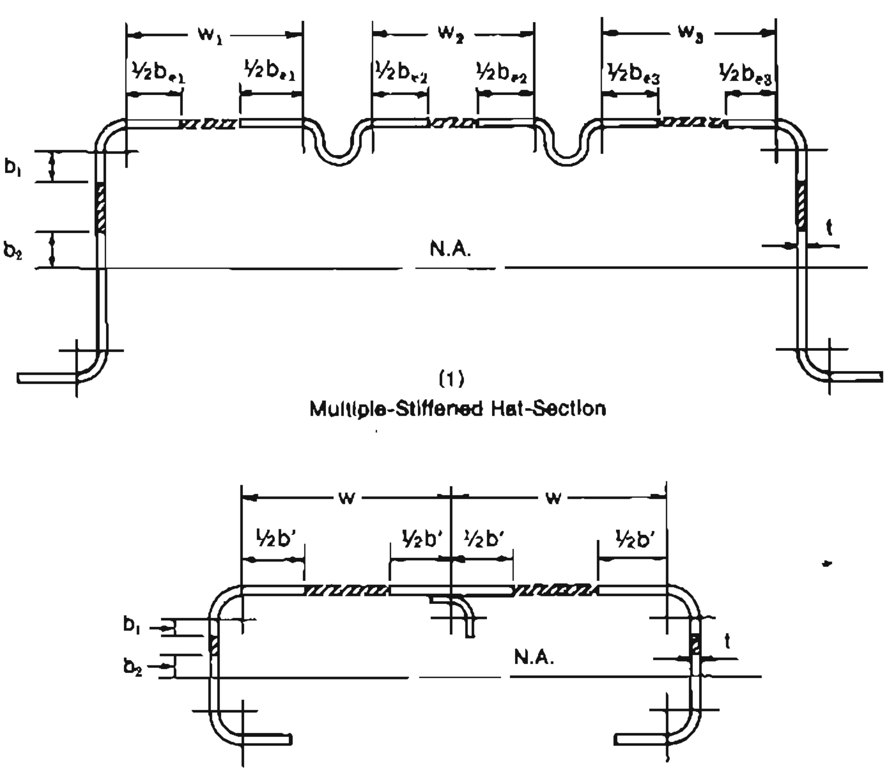
A1.3 Units of Symbols and Terms

The non-dimensional character of the majority of the *Specification* provisions is intended to facilitate design in any compatable system of units (U.S. customary, SI, or metric). It will also simplify conversion to a load and resistance factor design format.

A2 Non-Conforming Shapes and Constructions

The official having jurisdiction (authority) may approve any alternate shape of constructions provided the proposed alternate is satisfactory and complies with the provisions of Section F of the *Specification* and the particular building code.

If there is insufficient evidence of compliance with the requirements of the particular building code, the authority administering the code may require tests, at the applicant's expense, as proof of compliance. Test procedures shall be as stipulated by Section F of the *Specification*. If there is no recognized or accepted test method, the authority may prescribe appropriate test procedures.



(2)
Multiple-Stiffened Inverted "U"-Type Section

Flexural Members, such as Beams

Figure A1.2(c) Multiple-Stiffened Compression Elements Used in Cold-Formed Steel Structural Members

A3 Material

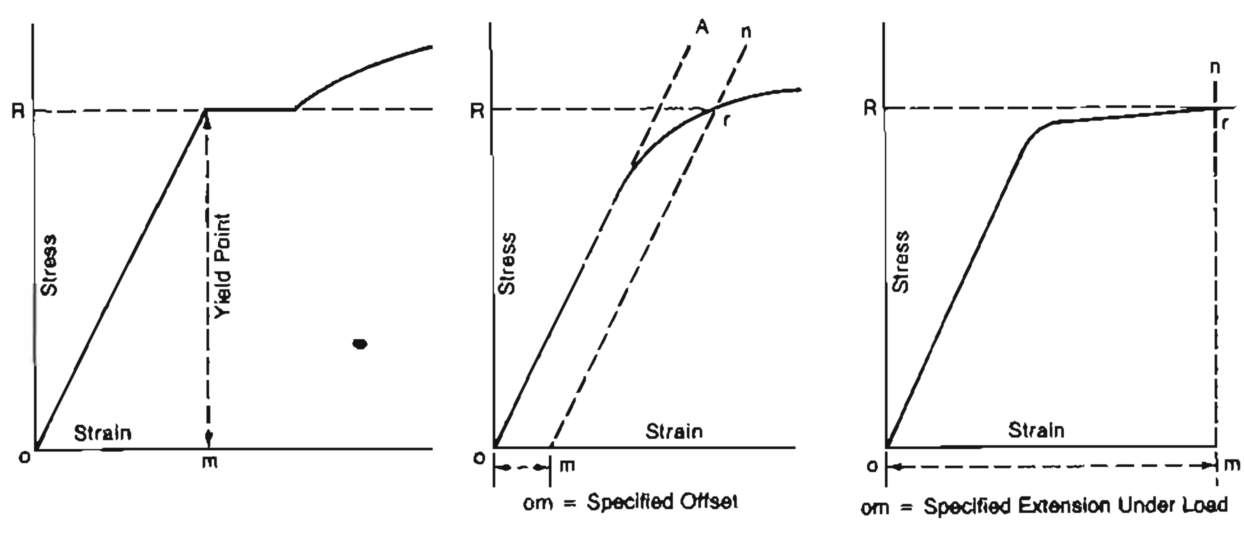
A3.1 Applicable Steels

The American Society for Testing and Materials (ASTM) is the basic source of steel designations for use with the *Specification*. Section A3.1 contains the complete list of ASTM Standards for steels that are accepted in connection with the *Specification*, including their dates of issue. The important structural properties necessary for cold forming are: yield point, tensile strength and elongation in 2 inches.

Ductility is the ability of a steel to undergo sizeable plastic or permanent strains before fracturing and is important both for structural safety and for cold forming. It is measured by the elongation in a 2-inch length. The ratio of the tensile strength to the yield point is also an important material property; this is an indication of the ability of the material to redistribute stress. For the listed ASTM Standards, yield points of steels with adequate ductility range from 25 to 70 ksi, though steels with yield points below 33 ksi are rarely used. The tensile-to-yield ratios are no less than 1.17, and the elongations no less than 12 percent. Exceptions are ASTM A446 and ASTM A611 which feature Grade E steels with a yield point of 80 ksi, a tensile strength of 82 ksi, and with no stipulated minimum elongation in 2 inches. These low-ductility steels permit only limited amounts of cold forming, require fairly large corner radii, and have other limits on their applicability. Nevertheless, they have been used successfully for specific applications, such as decks and panels with large corner radii and little, if any, stress concentrations. The conditions for use of these Grade E steels are outlined in *Specification* Section A3.3.2.

Standard methods and definitions for mechanical testing of steels and steel products are given in ASTM A370 (Reference 13). The methods for determining yield points for sharp and for gradual yielding steels are schematically shown on Figure A3.1.

The *Specification* formulas involving buckling are based on the tacit assumption that proportional limits of suitable steels shall be at least 70 percent of yield based on tensile tests. Determination of proportional limits for informational purposes can be done simply by using the offset method shown in Figure A3.1(b) with om equal to 0.0001 in./in. (0.01 percent offset) and calling the stress R where mn intersects the stress-strain curve at r, the proportional limit (0.01 percent offset).



(a) Showing Yield Point Corresponding with Top of Knee.

(b) Showing Yield Point of Yield Strength by the Offset Method. (Also Used for Proportional Limit)

(c) Determination of Yield Strength by Extension Under Load Method.

Figure A3.1 Stress-Strain Diagrams Showing Methods of Yield Point and Yield Strength Determination

A3.2 Other Steels

Although the use of ASTM-designated steels listed in *Specification* Section A3.1 is encouraged, other steels may also be used in cold-formed steel structures, provided they satisfy the requirements stipulated in this provision.

A3.3 Ductility

The nature and importance of ductility and the ways in which this property is measured were briefly discussed in *Commentary* Section A3.1.

Low-carbon sheet and strip steels with specified minimum yield points from 25 to 50 ksi need to meet ASTM specified minimum elongations in a 2-inch gage length of 12 to 27 percent. In order to meet ductility requirements, steels with yield points higher than 50 ksi are often low-alloy steels. However, Grades E of ASTM A446 and A611 are carbon steels, for which specified minimum yield strength is 80 ksi and no elongation requirement is specified. These differ from the array of steels listed under *Specification* Section A3.1.

In 1968, because new steels of higher strengths were being developed, sometimes with lower elongations, the question of how much elongation is really needed in a structure was the focus of a study initiated at Cornell University. Steels were studied that had yield strengths ranging from 45 to 100 ksi, elongations in 2 inches ranging from 50 to 1.3 percent, and tensile-to-yield strength ratios ranging from 1.51 to 1.00 (References 14-16). The investigators developed elongation requirements for ductile steels. These measurements were more accurate but cumbersome and impractical to make; therefore, the investigators recommended the following determination for adequately ductile steels: The tensile-to-yield strength ratio shall not be less than 1.08 and the total elongation in a 2-inch gage length shall not be less than 10 percent, or not less than 7 percent in an 8-gage length. Also, the *Specification* limits the use of Chapters B through E to adequately ductile steels.

The low tensile-to-yield strength ratio of 1.02 for Grade E steels of ASTM A446 and A611, and the fact that no elongation requirement is specified, means that these steels do not have adequate ductility as defined by *Specification* Section A3.3.1. Their use has been limited to particular configurations like roofing, siding, and floor decking.

Limiting the design yield strength to 75 percent of the specified minimum yield point, or 60 ksi, and the design tensile strength to 75 percent of the specified minimum tensile strength, or 62 ksi, whichever is lower, introduces a higher safety factor, but still makes low-ductility steels, such as Grade E, useful for the named applications. Load tests are permitted, but not for the purpose of using higher loads than can be calculated under *Specification* Sections B through E.

A3.4 Delivered Minimum Thickness

Sheet and strip steels, both coated and uncoated, may be ordered to nominal or minimum thickness. If the steel is ordered to minimum thickness, all thickness tolerances are over (+) and nothing under (−). If the steel is ordered to nominal thickness, the thickness tolerances are divided equally between over and under. Therefore, in order to provide equity between the two methods of ordering sheet and strip steel, it was decided to require that the delivered thickness of a cold-formed product be at least 95 percent of the design thickness. Thus, it is apparent that a portion of the factor of safety may be considered to cover minor negative thickness tolerances.

Generally, thickness measurements should be made in the center of flanges. For decking and siding, measurements should be made as close as practical to the center of the first full flat of the section. Thickness measurements should not be made closer to edges than the minimum distances specified in ASTM A568.

The responsibility of meeting this requirement for a cold-formed product is clearly that of the manufacturer of the product, not the steel producer.

A4 Loads

The *Specification* does not establish the dead, live or snow loading requirements for which a structure should be designed. In most cases, these loads are adequately covered by the applicable building code or design standard. See *Specification* Section A6.

Recognized engineering procedures should be employed to reflect the effect of impact loads on a structure.

Ellifritt investigated the basis of the one-third stress increase in wind and earthquake stresses (Reference 17) and concluded that the historical justification for the increase in wind stresses was "The action of wind on a structure is highly localized and of very short duration. Therefore it is not necessary to have as high a safety factor when designing for wind loads." The logic for the one-third increase in allowable design stresses for earthquake loads is similar to that for the wind provision.

The *Specification* recognizes the generally accepted practice of increasing the allowable stress by 33 percent for wind and earthquake. In this *Specification* this is accomplished by permitting a 25 percent reduction in design loads.

When gravity and lateral loads produce forces of opposite sign in members, consideration should be given to the minimum gravity loads acting in combination with wind or earthquake loads.

The final deflected shape of the member should be considered when calculating the load on a relatively flat roof resulting from ponding of rainwater or snowmelt. Guidance can be obtained from Section 1.13.3 of the AISC *Specification* (Reference 10).

A5 Structural Analysis and Design

A5.1 Design Basis

A safe load is determined by applying a factor of safety to the maximum strength, buckling or yielding, calculated for the member or connection in question. The fundamental nature of the factor of safety is to compensate for uncertainties inherent in the design, fabrication, and erection of building components as well as in the applied load (Reference 18). Table A5.1 summarizes the factors of safety adopted in the *Specification*.

A5.2 Yield Point and Strength Increase from Cold Work of Forming

The mechanical properties of the flat sheet, such as yield strength, tensile strength and elongation, may differ from the properties exhibited by the formed section. This difference can be attributed to cold working of the steel sheet during the forming process.

A combination of strain hardening, resulting from stretching of the sheet during the forming process, and strain aging causes an increase in the yield strength and tensile strength and a decrease in ductility. This phenomenon was studied by Winter and others (References 19-22), and this work serves as the basis for the provisions of Section A5.2.2. Yu (Reference 23) provides a more thorough discussion of the cold-work phenomenon.

In some cases, when evaluating the effective area of the web, ρ may be less than unity, but the sum of b_1 and b_2 may be such that the web is fully effective, and cold work of forming may be used.

A5.3 Serviceability and Durability

Further information may be found in the applicable building code and contract documents.

A6 Reference Documents

The tabulated references pertain to various aspects of cold-formed steel design. Some of the documents are referenced by the *Specification* while others provide information to aid the design engineer.

TABLE A5.1
Safety Factors by Subjects and Sections of the 1986 AISI Specification for the Design of Cold-Formed Steel Structural Members

Subject	Section	Safety Factor, Ω
Tension Member	C2	1.67 against yielding
Flexural Members		
Flexure only	C3.1	1.67 against yielding and buckling
Shear	C3.2	1.44 against shear yielding 1.67 against inelastic shear buckling 1.71 against elastic shear buckling
Web Crippling	C3.4	1.85 for single unreinforced webs 2.0 for I-beams
Concentrically Loaded Compression Members	C4	1.92 except when F_e is determined according to *Specification* Section C4.1 for fully effective sections having wall thickness greater than 0.09in. and $F_e > F_y/2$. In this case Ω_c varies from 1.92 to 1.67
Wall Studs	D4	1.92 against column buckling 1.92 for computing the permissable shear strain in sheathing material.
Fusion Welds	E2	2.50 against ultimate test value
Resistance Welds	E2.6	2.50 against ultimate test value
Bolted Connections	E3	
Spacing and Edge Distance	E3.1	2.0 when $F_u/F_y \geq 1.15$ 2.22 when $F_u/F_y < 1.15$
Tension on Net Section	E3.2	2.0 when washers are provided under bolt head and nut 2.22 when only one washer is used, or no washers
Bearing	E3.3	2.22 against bearing failure
Bolt shear	E3.4	2.25-2.52 against shear failure of bolts
Wind or Earthquake Loads	A4.4	A twenty five percent reduction of nominal safety factor is permissable provided that the section thus designed is not less than that required for combination of dead and live load.

B. ELEMENTS

This edition of the *Specification* differs from previous editions because a unified design approach has been adopted for treating compression elements. The effective width approach, previously applied only to stiffened compression elements, is being universally applied to all compression elements. *Commentary* Section B2 discusses the effective width approach.

B1 Dimensional Limits and Considerations

Because of the relatively large flat-width-to-thickness ratios (w/t) that are possible in cold-formed steel construction, dimensional limits are established in the *Specification*. Also, phenomena not germane to hot-rolled steel construction, e.g., flange curling and shear lag effects, are given consideration in the *Specification*.

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

(a) *Maximum* w/t *Ratios*
The limits imposed in *Specification* Section B1.1a are unchanged from previous editions of the *Specification*. Field experience has indicated that these limits are reasonable and achievable for typical cold-formed construction. In such cases where the limits are exceeded, tests in accordance with *Specification* Chapter F are required.
The note regarding noticeable deformations for larger flat-width-to-thickness ratios is a caution and is not intended to prevent the use of such compression elements.

(b) *Flange Curling*
Unusually wide, thin, but stable flanges tend to curl, i.e., deflect toward the neutral axis, during loading. An approximate, analytical treatment of the problem is given in Reference 24. Equation B1.1b-1 (Appendix B) enables the evaluation of the maximum admissible flange width, w_f, for a given amount of curling, c_f. The *Specification* does not stipulate the amount of curling; this is subjective because the distortions may or may not be offensive depending upon the application. The suggested curling of 5 percent of the member depth is not considered excessive under usual conditions.

(c) *Shear Lag Effects*
For beams having relatively small span-to-flange-width ratios, L/w_f, shear deformations may significantly alter the stress distribution in both the compression and tension flanges. Table B1.1c (Appendix B) is based on analytical and experimental data summarized in Reference 25. It should be noted that the flange width in this case is the projection beyond the web, not the flat portion of the flange, as in the case in subsequent sections of Chapter B.

B1.2 Maximum Web-Depth-to-Thickness Ratio

The limits prescribed in Section B1.2 are unchanged from the 1980 AISI *Specification*. These limitations are based on the studies reported in References 26-28. However, because the definition for h, the depth of the flat portion of the web measured along the plane of the web, differs from previous editions of the *Specification*, the prescribed limits may appear to be more liberal. An unpublished study by LaBoube concluded that the new definition for h had negligible influence on the web strength.

B2 Effective Widths of Stiffened Elements

The use of effective widths for stiffened compression elements is not new to the *Specification*. Previous editions of the *Specification* have treated only uniformly compressed stiffened elements by using an effective width approach. The scope of application for effective widths has been expanded to include (1) uniformly compressed stiffened elements, (2) uniformly compressed stiffened elements with circular holes and (3) webs and stiffened elements with stress gradients.

B2.1 Uniformly Compressed Stiffened Elements

(a) *Load Capacity Determination*
A non-dimensional format for presentation of *Specification* provisions has been adopted. See Reference 29 for a thorough discussion of the basis for this format as well as the research basis for the minor differences in numerical results. Therefore, the equations of Section B2.1a, although in outward appearance different from the 1980 AISI *Specification*, are, in fact, nearly identical to equations as developed by Winter (References 24, 30-32).

In addition to the non-dimensional format, Equation B2.1-4 is given with k, the plate buckling coefficient, as a variable. The factor k depends upon the boundary conditions of the plate element and the manner of loading. Presenting the Equation in this form, enables its use for all compression elements in the *Specification*. The equations of Section B2.1a are a common thread that runs through Sections B2 through B4 of the *Specification*.

(b) *Deflection Determination*
The design engineer has the option of using two procedures for determining the effective width to be used for deflection calculations. Procedure I uses the effective width Equation of Section B2.1a evaluated at the actual stress level. This is consistent with the practice of the 1980 AISI *Specification*. Procedure II is based upon recently completed research (Reference 29) and yields a more accurate estimate of the effective width for deflection analysis.

B2.2 Uniformly Compressed Stiffened Elements with Circular Holes

These are new provisions and have a limited application as stated in the *Specification*. Reference 29 summarizes the background studies relative to this section.

B2.3 Effective Width of Webs and Stiffened Elements with Stress Gradient

The use of effective widths for web elements subjected to a stress gradient is a deviation from the past practice of using a full area in conjunction with a reduced stress to account for local buckling and post-buckling strength (Reference 23). The effective widths are based upon Winter's effective width equation distributed as shown by Figure B2.3-1 of the *Specification*. Background regarding the development of the provisions of Section B2.3 is provided in Reference 29.

B3 Effective Widths of Unstiffened Elements

The 1986 edition of the *Specification* marks the first time that unstiffened compression elements are treated using an effective width approach. The provisions of this section are based upon analytical and experimental data reported in Reference 29. The research demonstrated that Winter's effective width equation was an adequate predictor of section capacity if the appropriate buckling coefficient, k, is employed.

B3.1 Uniformly Compressed Unstiffened Elements

The theoretical buckling coefficient, as given by Reference 33, is 0.425. This value has been rounded to two significant figures in *Specification* Section B3.1

B3.2 Unstiffened Elements and Edge Stiffeners with Stress Gradient

There is a very limited amount of information on the behavior of unstiffened plate elements with a stress gradient. Cornell research (Reference 29), into the behavior of edge stiffeners for flexural members, has demonstrated that by using Winter's effective width equation in conjunction with a $k = 0.43$ (uniform compression), good correlation was achieved between test and calculated capacity. This same trend was also true for deflection determination.

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

The provisions of *Specification* Section B4 represent the latest research findings in regard to stiffeners (Reference 29). Previous specifications treated stiffeners as fully effective plate elements. By using effective areas for stiffeners, all compression elements are analyzed on the basis of effective widths or areas.

The design provisions, which are based on both critical local buckling and ultimate strength criteria, recognize the interaction of the plate elements (i.e., flange and stiffener). Also, for the first time, provisions for analyzing partially stiffened, as well as adequately stiffened, compression elements are contained in the *Specification*.

B4.1 Uniformly Compressed Elements with an Intermediate Stiffener

The 1980 AISI *Specification* contained provisions for the minimum required moment of inertia, which was based on the assumption that an intermediate stiffener needed to be twice as rigid as an edge stiffener. Subsequent research (Reference 34) has developed expressions for evaluating the required stiffener rigidity based upon the geometry of the contiguous flat elements. Using the ratio of actual stiffener moment of inertia, I_s, to adequate stiffener moment of inertia, I_a, (i.e., I_s/I_a) to evaluate the buckling coefficient and the stiffener area, a partially stiffened compression flange can also be evaluated.

B4.2 Uniformly Compressed Elements with an Edge Stiffener

Specification Section B4.2 recognizes that the necessary stiffener rigidity depends upon the slenderness of the plate element being stiffened. Thus, Cases, I, II and III, each contain different definitions for an adequate stiffener moment of inertia.

The interaction of the plate elements, as well as the degree of edge support, full or partial, is compensated for in the expressions for k, d_s and A_s. For more in-depth coverage of this topic see Reference 29.

B5 Effective Widths of Edge Stiffened Elements with Intermediate Stiffeners or Stiffened Elements with More Than One Intermediate Stiffener

There has been no research to further our understanding of the behavior of multiple stiffened elements. Thus the *Specification* has retained Equation B5-1 from the 1980 *Specification* for evaluating the minimum required rigidity, I_{min}, of an intermediate stiffener for a multiple-stiffened element. This value of I_{min} is used when evaluating the effective widths in *Specification* Section B4.

In addition, *Specification* Section B5(a) stipulates that only intermediate stiffeners adjacent to web elements (see Figure A1.2-1(c)(1)) shall be counted as effective. Additional stiffeners would have two or more sub-elements between themselves and the nearest shear-transmitting element (i.e., web) and hence, could be ineffective. *Specification* Section B5(b) applies the same reasoning to intermediate stiffeners between a web and an edge stiffener.

If intermediate stiffeners are spaced so closely that the sub-elements are fully effective, i.e., $b < w$, no plate buckling of the sub-elements will occur. Therefore, the entire assembly of sub-elements and intermediate stiffeners between webs behaves like a single compression element whose rigidity is given by the moment of inertia, I_s, of the full, multiple-stiffened element, including stiffeners. Although the effective width calculations are based upon an equivalent element having width, w_s, and thickness, t_s, the actual thickness must be used when calculating section properties.

B6 Transverse Stiffeners

Design requirements for attached transverse stiffeners and for intermediate stiffeners were added in the 1980 AISI *Specification* and are unchanged in the 1986 *Specification*. Equation B6.1-1 serves to prevent end crushing of the transverse stiffeners, while Equation B6.1-5 is to prevent column-type buckling of the web-stiffeners. The equations for computing the effective areas (A_b and A_c) and the effective widths (b_1 and b_2) were adopted with minor modifications from Reference 28.

The requirements for intermediate stiffeners included in *Specification* Section B6.2 were adapted from Section 1.10.5.3 of the AISC *Specification* (Reference 10). The equations for determining the minimum required moment of inertia (*Eq.* B6.2-1) and the minimum required gross area (*Eq.* B6.2-2) of attached intermediate stiffeners are based on the studies summarized in Reference 28. In Equation B6.2-1, the minimum value of $(h/50)^4$ was selected from the AISC *Specification*.

Tests on rolled-in transverse stiffeners covered in *Specification* Section B6.3 were not made in the experimental program reported in Reference 28. Lacking reliable information, the required dimensions and the allowable loads should be determined by special tests.

C. MEMBERS

To simplify the use of the *Specification*, all design provisions relative to a specific member type, e.g., beam, column or beam-column, have been assembled in one location within the *Specification*. Also, design provisions are given in terms of allowable load or moment, instead of allowable stress, as was the approach in previous specifications. To enable a clearer understanding of the phenomenon being evaluated, the nominal capacity and required safety factor are explicitly stated.

C1 Properties of Sections

The geometric properties of a member, i.e., area, moment of inertia, section modulus and radius of gyration, shall be evaluated using conventional methods of structural design. These properties are based upon either full cross-section dimensions, effective widths or net section, as applicable.

For flexural members and axially loaded members both the full and effective dimensions are used. The full dimensions are used when calculating the critical load or moment, while the effective dimensions, evaluated at the stress corresponding to the critical load or moment, are used to calculate the nominal capacity. References 29 and 35 discuss this concept in more detail.

The net section is employed when computing the capacity of a tension member.

C2 Tension Members

There is a very limited amount of data regarding the capacity of cold-formed steel tension members. Because the provisions of the 1980 AISI *Specification* have been field tested with no known deficiency, they have been carried forward to the 1986 *Specification*.

C3 Flexural Members

The provisions contained in Section C3 of the *Specification* address the various design aspects related to cross-section capacity, that is, flexure strength, shear strength, combined bending and shear strength, web crippling strength and combined bending and web crippling strength. For brace design, see *Specification* Section D3.

C3.1 Strength for Bending Only

Flexural strength is a function of the geometry of the member in question, as well as the degree of lateral restraint, i.e., bracing, provided. The provisions of *Specification* Section C3.1 encompass these considerations.

C3.1.1 Nominal Section Strength

Beams not subject to lateral (flexural), torsional or torsional-flexural buckling, are designed on the basis of the yield strength being achieved in the extreme fibers. Depending upon the cross-section geometry, initial yielding may occur in either the extreme tension or compression fibers. This is the premise in Section C3.1.1a, which is consistent with previous editions of the *Specification*.

For certain beam members, depending upon geometry, the potential exists for partial yielding of the cross section. The 1980 AISI *Specification* introduced guidelines which enabled design engineers to take advantage of this increased moment capacity. The

provisions of the 1986 *Specification* have been modified by removing the requirement that the depth-to-thickness ratio of the entire web does not exceed $640/\sqrt{F_y}$. This restriction had been adopted from the AISC *Specification* (Reference 10) and was based on research on the plastic rotation capacity of hot-rolled, doubly symmetric wide-flange beams. For typical cold-formed sections, this limitation is redundant because the requirement of *Specification* Section C3.1.1b(3), the ratio of the depth of the compression portion of the web to its thickness not exceed $\lambda_1 = 1.11/\sqrt{Fy/E}$ is the governing restriction. A detailed discussion of the inelastic reserve capacity of flexural members is given in Reference 36. The development of this procedure is described in Reference 29.

C3.1.2 Lateral Buckling Strength

The design expressions for laterally unbraced segments of flexural members in *Specification* Section C3.1.2 represent an improvement over previous specifications. The approach that has been adopted enables direct consideration of the interaction between local and overall buckling by a reduction of the critical moment. This reduction is equal to the ratio of the effective section modulus to the full section modulus.

Unlike previous specifications, the 1986 *Specification* is expressed in terms of moment instead of stress. The critical moment for I- and Z-sections employ the same critical buckling stress equations that formed the basis for the 1980 AISI *Specification* (References 23 and 37-40).

For singly symmetric sections for which the x-axis is the axis of symmetry, new, more theoretically based equations are given. The background for these equations is given in Reference 41.

The case of laterally unbraced compression flanges is treated in Part III, Section 3, of the *Manual*, based on Reference 42. A slightly different approach is given in Reference 43.

C3.1.3 Beams Having One Flange Attached to Deck or Sheathing

Studies (References 44, 45) of simple span C- and Z-purlin roof systems under uplift load have demonstrated that the roof panel, although attached to the tension flange, does provide some degree of restraint. This restraint is a function of the rotational stiffness of the panel-to-purlin connection which can be evaluated by the test procedure given in Part VII of the Manual.

For continuous purlin systems, the analytical procedure described in Reference 44 is a rational analysis provided limitations in specimen dimension and construction details in supporting test evidence are recognized, and proper consideration is given for the behavior of continuous-span systems.

In lieu of a rational analysis, tests may be conducted in accordance with Chapter F.

C3.2 Strength for Shear Only

The *Specification* provisions are applicable for slender webs of beams and decks either with or without stiffeners. These provisions are identical to Section 1.10.5.2 for the design of plate girders and rolled beams in the 1978 AISC *Specification* (Reference 10). The acceptance of the AISC equations for cold-formed sections is based upon the study summarized in Reference 46.

C3.3 Strength for Combined Bending and Shear

For cantilever beams and continuous beams, high bending stresses often combine with high shear stresses at the supports. In the design of such members, it has been the practice to use *Specification* Equation C3.3-1 to safeguard against buckling of flat webs due to the combination of bending and shear stresses (References 7 and 33). In addition, a new interaction equation (*Eq.* C3.2-2) was included in the 1980 *Specification* for beam webs with adequate transverse stiffeners. The correlations between the test data and both formulas are given in Reference 47. Limitations remain imposed on M_a and V_a as loads, in contrast to removal of limits when stress interaction equations were used.

C3.4 Web Crippling Strength

Section C3.4 of the *Specification* provides design equations to prevent web crippling of flexural members having flat single webs (channels, Z-sections, hat sections, tubular members, roof deck, floor deck, etc.) and I-beams (made of two channels connected back to back, by welding two angles to a channel, or by connecting three channels). Different design equations are used for different loading conditions. As shown in Figure C3.4-1, Equations C3.4-1, C3.4-2, and C3.4-3 are used for end one-flange loading; Equations C3.4-4 and C3.4-5 for interior one-flange loading; Equations C3.4-6 and C3.4-7 for end two-flange loading; and Equations C3.4-8 and C3.4-9 for interior two-flange loading. These design equations are based on experimental evidence (Reference 27) and the distribution of loads or reactions into the web as shown in Figure C3.4-2.

The distribution of loads or reactions into the web as shown in Figure C3.4-2 are independent of the flexural response of the beam. Due to flexure, the point of bearing will vary relative to the plane of bearing resulting in non-uniform bearing load distribution into the web. The value of P_a will vary because of a transition from the interior one-flange loading (Figure C3.4-2(b)) to the end one-flange loading (Figure C3.4-2) condition. These discrete conditions represent the experimental basis on which the design provisions were founded (Reference 27).

C3.5 Combined Bending and Web Crippling Strength

This *Specification* section contains two interaction formulas for the combination of bending and web crippling. These formulas are based on studies at the University of Missouri–Rolla on the effect of bending on the reduction of web crippling loads (Reference 23, 27, 48 and 49). For embossed webs, crippling strength should be determined by tests according to *Specification* Chapter F.

The exception clause in *Specification* Section C3.5.1 applies to the interior supports of continuous spans using decks and beams, as shown in Figure C3.5-1. Results of continuous beam tests of steel decks (References 49) and several independent studies by manufacturers indicate that, for these types of members, the post-buckling behavior of webs at interior supports differs from the type of failure mode occurring under concentrated loads on single span beams. This post-buckling strength enables the member to redistribute the moments in continuous spans. For this reason, Equation C3.5-1 is not applicable to the interaction between bending and the reaction at interior supports of continuous spans. This exception applies only for the members shown in Figure C3.5-1 and similar situations explicitly described in *Specification* Section C3.5.

The exception clause should be interpreted to mean that the effects of combined bending and web crippling need not be checked for determining load carrying capacity. Furthermore the positive bending resistance of the beam should be at least 90 percent of the negative bending resistance in order to insure the factor of safety implied by the *Specification.*

Using this procedure the allowable loads may (1) produce slight deformations in the beam over the support, (2) increase the actual compressive bending stresses over the support to as high as $0.8\,F_y$, and (3) result in additional bending deflection of up to 22 percent due to elastic moment redistribution.

If load carrying capacity is not the primary design concern because of the above behaviors, the designer is urged to use Equation C3.5-1.

With regard to Equation C3.5-2, previous tests indicate that when the h/t ratio of an I-beam web does not exceed $2.33/\sqrt{F_y/E}$ and when $\lambda \le 0.673$, the bending moment has little or no effect on the web crippling load. For this reason, the allowable reaction or concentrated load can be determined by the formulas given in *Specification* Section C3.4 without reduction for the presence of bending.

C4 Concentrically Loaded Compression Members

The provisions of this *Specification* section represent a significant improvement over previous provisions for concentrically loaded compression members subject to either flexural, torsional or torsional-flexural buckling. A unified approach is presented wherein Equations C4-1 through C4-4 are general equations for the three buckling modes. The variation in the

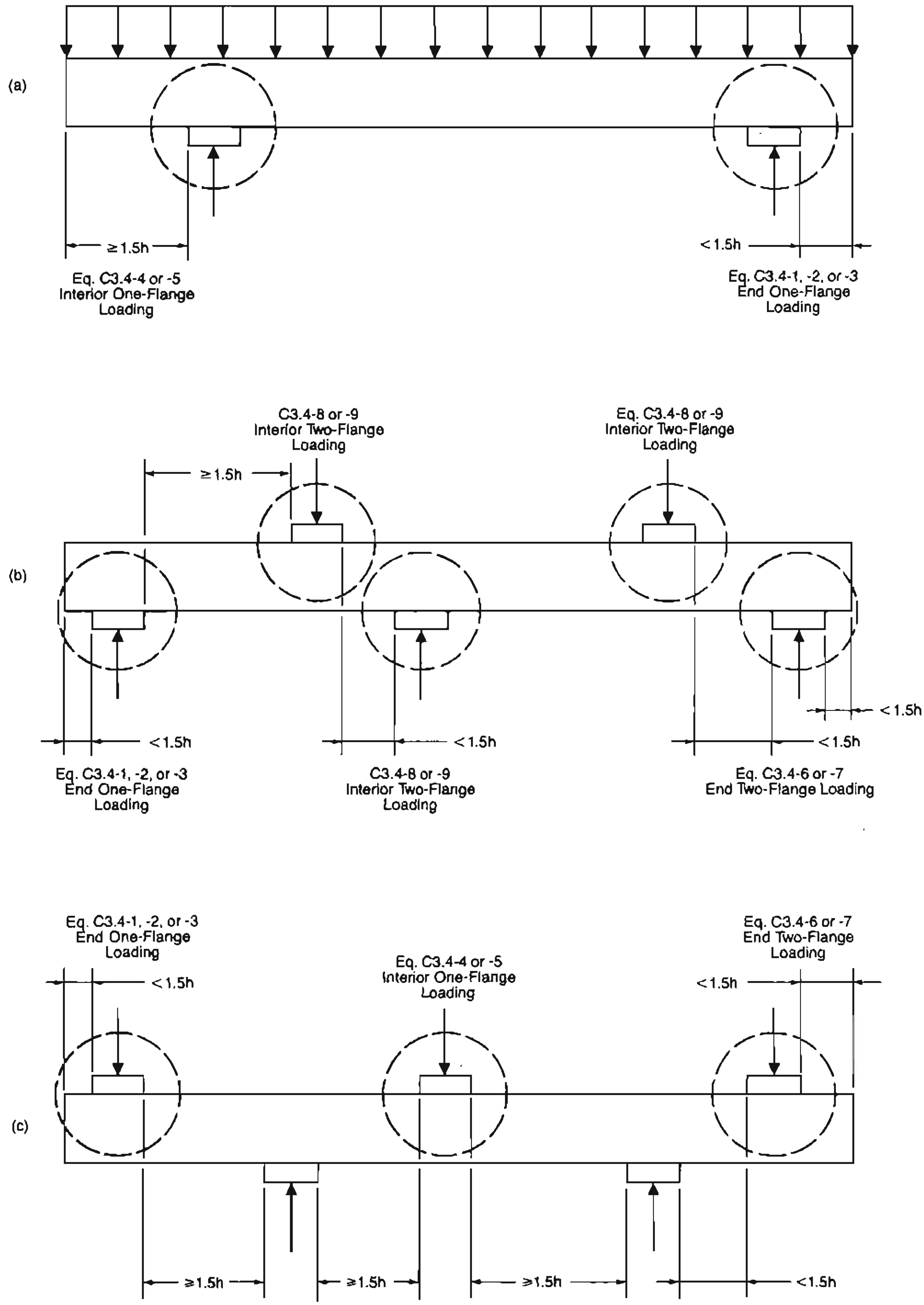


Figure C3.4-1 Application of Design Equations Listed in Table C3.4-1

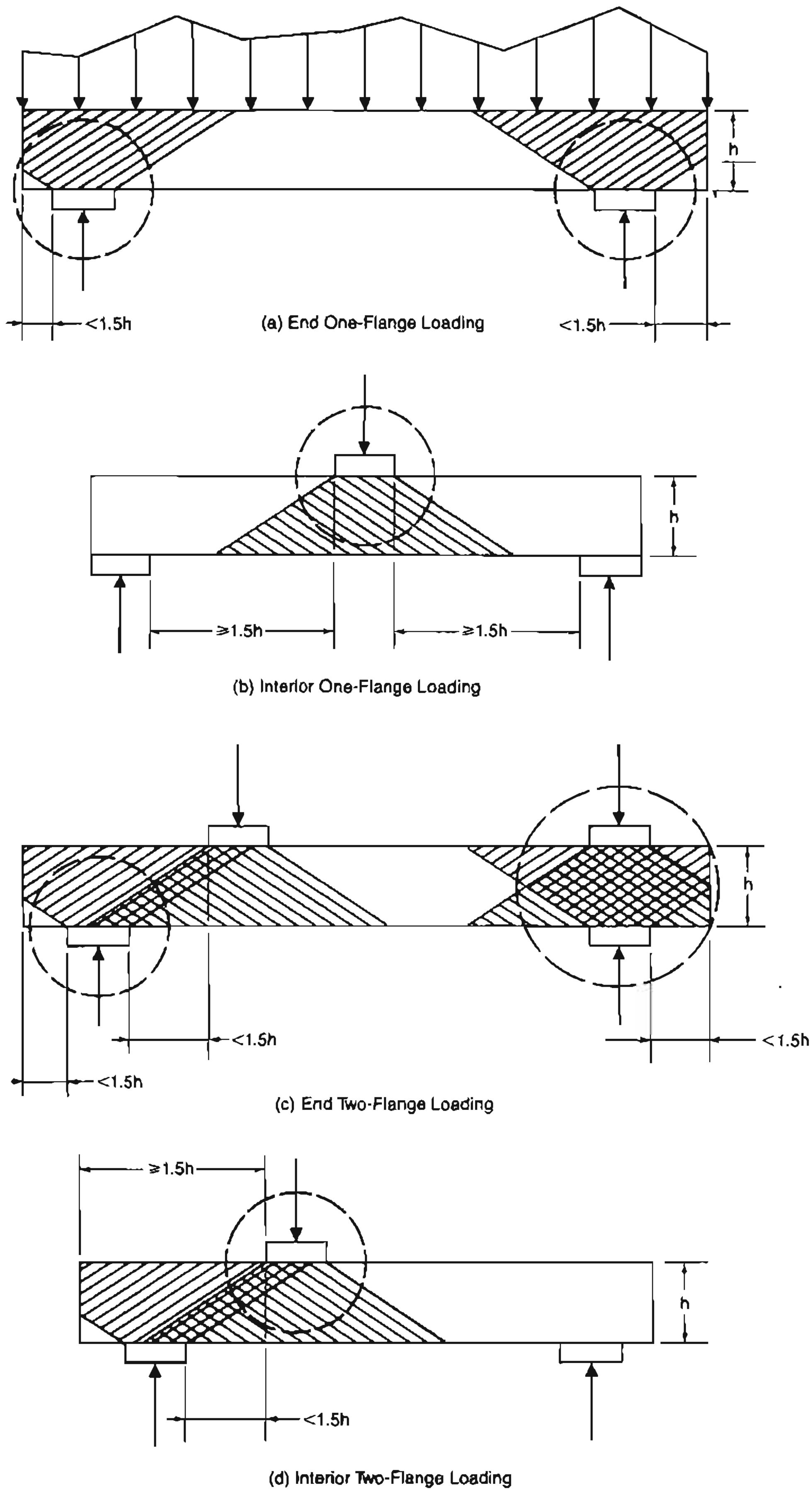


Figure C3.4-2 Assumed Distribution of Reaction Or Load

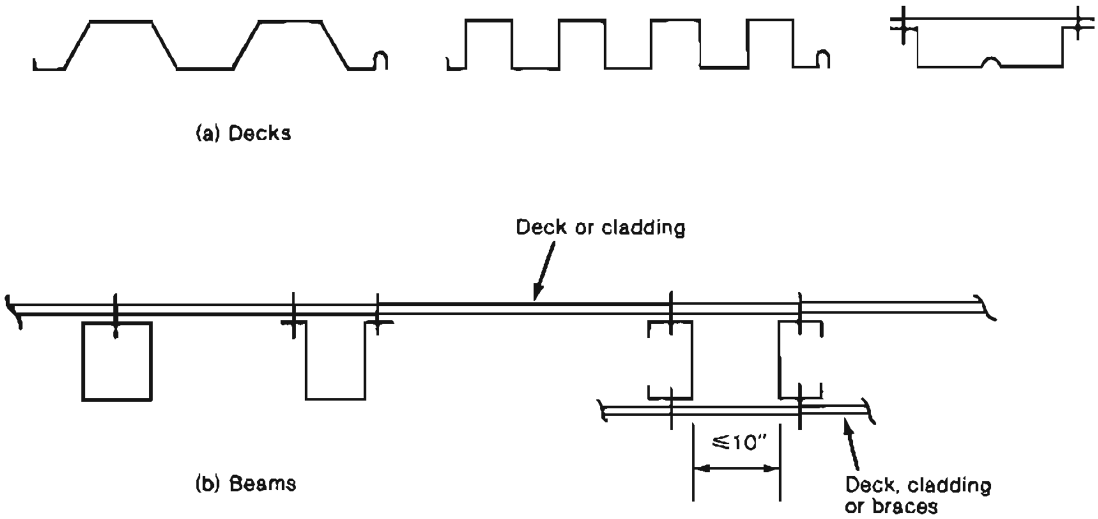


Figure C3.5 Sections used for one Exception Clause of Section C3.5

three modes is recognized by the appropriate elastic buckling equation as given in *Specification* Sections C4.1 and C4.2.

A simple approach to account for the interaction of local and overall buckling is also included in the provisions of *Specification* Section C4. The critical stress, evaluated for the full section, is the stress level for which the effective area is calculated. Nominal axial strength is then the product of the critical stress and the effective area (Equation C4-2) except that for members with unstiffened flanges the axial strength shall be limited by Equation C4-5. For shapes for which the effective area, A_e, cannot be calculated according to this Specification, the stub column test procedure given in Part VII may be used. For a more in-depth discussion of the background for these provisions, see Reference 29.

Specification Section C4(c) is a new provision and accounts for the possibility of a reduction in capacity due to initial sweep of the member.

C5 Combined Axial Load and Bending

The interaction equations contained in Section 3.7.1 of the 1980 AISI *Specification* were for locally stable sections and are identical to the AISC *Specification* requirements (Reference 10). Cornell University researchers have extended the use of these interaction equations to locally unstable sections (Reference 29). This enables the 1986 *Specification* to present a consistent set of interaction equations applicable for all cold-formed steel shapes.

The limitation that $C_m \geq 0.4$ has been deleted to correct an oversight identified in development of AISC's LRFD *Specification for Steel Buildings*. See the *Commentary* on that specification for the detailed reasoning for this change (Reference 50).

For angle sections, the applied moment M_y accounts for the possibility that when the load is applied between the centroid of the full section and the shear center, a lower axial load capacity may occur.

C6 Cylindrical Tubular Members

The design provisions for cylindrical tubular members have been expanded to include explicit guidelines for members subject to either axial compression or flexure.

C6.1 Bending

For thick cylinders in bending, the initiation of yielding does not represent a failure condition as is generally assumed for axial loading. Failure is at the plastic moment capacity which is at least 1.29 times the moment at first yielding. In addition, the conditions for inelastic local buckling are not as severe as in axial compression due to the stress gradient.

Equations C6.1-2, -3, and -4 are based upon the work reported in Reference 51 and an assumed minimum shape factor of 1.29. A slight further reduction in the inelastic range has been made to limit the maximum bending stress to 0.75 F_y, a value typically used for solid sections in bending. The reduction also brings the criteria closer to a lower bound for inelastic local buckling. A small range of elastic local buckling has been included so that the upper D/t limit of 0.441 E/F_y is the same as for axial compression.

C6.2 Compression

Previous editions of the *Specification* contained no provisions for the interaction of local and column buckling. The provisions of *Specification* Section C6.2 include this interaction. The basis for the interaction is the approach recommended in the third edition of the SSRC guide (52), where the critical stress for local buckling is substituted for the yield stress in the column equation. The critical stress computed by Equation C6.2-3 is the same equation as used for overall buckling in previous editions of the *Specification*, except that the factor of safety has been removed. The effective area defined by Equation C6.2-6 converts the results into a design approach consistent with that for concentrically loaded compression members (*Specification* Section C4). This equation provides results that are within a few percent of the SSRC approach.

C6.3 Combined Bending and Compression

Sherman (Reference 53) has confirmed the applicability of linear interaction equations for cylindrical tubular members, including situations where local buckling may occur.

D. STRUCTURAL ASSEMBLIES

D1 Built-Up Sections

D1.1 I-Sections Composed of Two Channels

I-sections made by connecting two channels back to back often are used as either compression or flexural members. For the I-sections to be used as compression members, the longitudinal spacing of connectors must not exceed the value of s_{max}, computed by using Equation D1.1-1 of the *Specification*. This prevents flexural buckling of the individual channels about the axis parallel to the web at a load smaller than that at which the entire I-section would buckle. This provision is based on the requirement that the slenderness ratio of an individual channel between connectors, s_{max}/r_{cy}, not be greater than one-half of the pertinent slenderness ratio, L/r_I, of the entire I-section (References 7 and 23). This accounts for one of the connectors becoming loose or ineffective.

Even though Section D1 of the *Specification* refers only to I-sections, Equation D1.1 also can be used for determining the maximum spacing of welds for box-shaped compression members made by connecting two channels tip to tip. In this case, r_I is the smaller of the two radii of gyration of the box-shaped section.

For the I-sections to be used as flexural members, the longitudinal spacing of connectors is limited by Equations D1.1-2 and D1.1-3 of the *Specification*. The first requirement (Equation D1.1-2) is an arbitrarily selected limit to prevent any possible excessive distortion of the top flange between connectors. The second (Equation D1.1-3) is based on the strength and arrangement of connectors and the intensity of the load acting on the beam. Reference 23 provides further discussion on this topic.

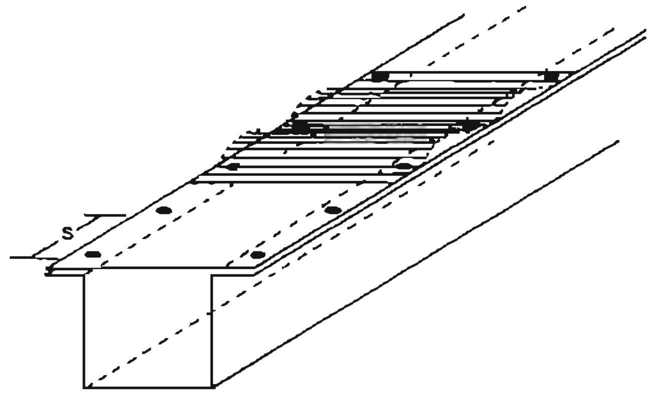


Figure D1.2-1 Spacing of Connections in Compression Elements

D1.2 Spacing of Connections in Compression Elements

When compression elements are joined to other parts of built-up members by intermittent connections, these connections must be closely spaced to develop the required strength of the connected element. Figure D1.2-1 shows a box-shaped beam made by connecting a flat sheet to an inverted hat section. If the connectors are appropriately placed, this flat sheet will act as a stiffened compression element with a width, w, equal to the distance between rows of connectors, and the sectional properties can be calculated accordingly. This is the intent of the provisions in Section D1.2.

Section D1.2(a) of the *Specification* requires that the necessary shear strength be provided by the same standard structural design procedure that is used in calculating flange connections in bolted or welded plate girders or similar structures.

Section D1.2(b) of the *Specification* ensures that the part of the flat sheet between two adjacent connectors will not buckle as a column (see Figure D1.2-1) at a stress less than 1.67f, where f is the design stress of the connected compression element (Reference 23).

Section D1.2(c) ensures satisfactory spacing to make a row of connections act as a continuous line of stiffening for the flat sheet under most conditions (References 7 and 23).

D2 Mixed Systems

When cold-formed steel members are used in conjunction with other construction materials, the design requirements of the other material specifications also must be satisfied.

D3 Lateral Bracing

Bracing design requirements have been expanded in the 1986 *Specification* to include a general statement regarding bracing for symmetrical beams and columns and specific requirements for the design of roof systems subjected to gravity load.

D3.1 Symmetrical Beams and Columns

There are no simple, generally accepted techniques for determining the required strength and stiffness for discrete braces in steel construction. Winter (Reference 54) offered a partial solution and others (References 43, 55-57) have extended this knowledge. The design engineer is encouraged to seek out the stated references to obtain guidance for design of a brace or brace system.

D3.2 Channel-Section and Z-Section Beams

Channel and Z-sections used as beams to support transverse loads applied in the plane of the web may twist and deflect laterally unless adequate lateral supports are provided. Section D3.2 of the *Specification* includes two subsections. The first (Section

D3.2.1) is a new provision that deals with the bracing requirement when one flange of the beam is connected to deck or sheathing material. The second subsection (Section D3.2.2) covers the requirements for spacing and design of braces, when neither flange of the beam is braced by deck or sheathing material.

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

In conventional metal building roof systems, the roof panels are attached to the top flange of each purlin throughout its length using self-drilling or self-tapping, through-the-sheet, fasteners spaced at approximately 1 foot on center. This panel usually provides sufficient stiffness to prevent the relative movement of the purlins with respect to each other, however, unless external restraint is provided, the system as a whole will tend to move laterally. This restraint or anchorage may consist of members attached to the purlin at discrete locations along the span and designed to carry forces necessary to restrain the system against lateral movement. The design rules for Z-purlin supported roof systems are based on a first order, elastic stiffness model (Reference 58).

D3.2.2 Neither Flange Connected to Sheathing

When neither flange is braced by deck or sheathing material, discrete bracing must be provided. Section D3.2.2 specifies the spacing of such braces, applicable to both channel and Z-beams, and the forces for which these braces must be designed. A detailed discussion of the background for these provisions is contained in Reference 23.

An exception in the *Specification* permits omission of discrete braces when all loads and reactions on a beam are transmitted through members that frame into the section in such a manner as to effectively restrain the member rotation and lateral displacement. Frequently this occurs in the end walls of metal buildings.

D3.3 Laterally Unbraced Box Beams

Box beams are more laterally stable than single-web sections having the same depth/width ratio. *Specification* Section D3.3 specifies that when the ratio of the laterally unsupported length to the distance between the webs of the section (L/b) does not exceed 0.086 E/F_y, the box sections can be used without any strength reduction to account for lateral buckling. This design requirement is based on Part II of Reference 8 which indicates that the failure stress in box beams is practically unaffected by lateral buckling up to L/b as high as 100 for a steel having a yield point of 33 ksi. On the basis of this AISI design criterion, when the box beams are bent about the major axis, the laterally unsupported length shall not exceed 75 times the distance between webs for a steel with $F_y = 33$ ksi. This conservative design approach is identical with the previous edition of the AISI *Specification*.

D4 Wall Studs and Wall Stud Assemblies

The provisions of this section are primarily based on References 59 and 60 which are summarized in Reference 23. These equations are based on tests of solid-web cold-formed steel studs with the following maximum values: yield point $F_y = 50$ ksi, depth $d = 6$ inches, thickness $t = 0.075$ inches, and overall length $L = 16$ feet.

These references contain procedures for computing the strength of C- and Z-section wall studs that are braced by sheathing materials. The bracing action is due to both the shear rigidity and the rotational restraint supplied by the sheathing material. The treatment in References 59 and 60 is quite general and includes the case of studs braced on one as well as on both flanges. However, the provisions of Section D4 deal with the simplest case of identical sheathing material on both sides of the stud. For simplicity, only the restraint due to the shear rigidity of the sheathing material is considered. Other cases, such as studs with sheathing on one flange only, dissimilar sheathing materials on each flange, or the evaluation of the effect of rotational restraint, are covered in footnotes to *Specification* Sections D4.1 and D4.2.

D4.1 Wall Studs in Compression

The provisions in this *Specification* section are given to prevent three possible modes of failure. Provision (a) is for column buckling between fasteners, even if one fastener is missing or otherwise ineffective. Provision (b) contains formulas for allowable axial stresses for overall column buckling. Essential to these provisions is the magnitude of the shear rigidity of the sheathing material. A Table of values and an Equation for determining the shear rigidity is provided in the *Specification*. These values are based on the small scale tests described in References 59 and 60. For other types of materials, the sheathing parameters can be determined using the procedures described in these references.

Provision (c) is to insure that the sheathing has sufficient distortion capacity. The procedure involves assuming a value of the ultimate stress and checking whether the shear strain at the load corresponding to the ultimate stress exceeds the allowable design shear strain of the sheathing material. In principle, the procedure is one of successive approximations. However, if the smaller of F_e (provision a) or σ_{cr} (provision b) is tried and shown to be satisfactory, then the need for iteration is eliminated.

In the 1986 *Specification*, the Q method for treating the local buckling effects is eliminated. The approach recommended is to find the overall buckling stress on the basis of the full unreduced section. The ultimate loads are determined by multiplying the buckling stress by the effective area determined at the buckling stress.

D4.2 Wall Studs in Bending

This provision modifies the 1980 *Specification*, in that a limitation on the bending moment has been deleted. Tests of long, deep studs have confirmed the excess conservatism of the 1980 provision.

D4.3 Wall Studs with Combined Axial Load and Bending

The general interaction equations of *Specification* Section C5 are also applicable to wall studs with the exception that the allowable moments be evaluated excluding lateral buckling considerations.

E. CONNECTIONS AND JOINTS

E1 General Provisions

The *Specification* contains provisions only for welded and bolted connections. Other means of connection find application in cold-formed construction and some of these are listed below:

(a) *Rivets*
While hot rivets have little application in cold-formed construction, cold rivets find considerable use, particularly in special forms, such as blind rivets (for application from one side only), tubular rivets (to increase bearing area), high shear rivets, and explosive rivets.

(b) *Screws*
These are usually self-drilling and self-tapping, and come in a wide variety of shapes, dimensions, and details.

(c) *Special devices*
These include: (1) metal stitching, achieved by tools that are special developments of the common office stapler, and (2) connecting by means of special clinching tools that draw the sheets into interlocking projections.

Most of these connections are proprietary devices for which information on strength of connections must be obtained from manufacturers or from tests carried out by or for the user. Guidelines provided in *Specification* Chapter F are to be used in these tests.

The plans and/or specifications are to contain adequate information and design requirement data for the adequate detailing of each connection if each connection is not detailed on the engineering design drawings.

E2 Welded Connections

The provisions contained in this *Specification* section have been primarily based on experimental evidence obtained in an extensive test program at Cornell University. The results of this program are summarized in References 23 and 61. All possible failure modes are covered in the present provisions, whereas the earlier provisions mainly dealt with shear failure.

For most of the connection tests reported in Reference 61, the onset of yielding was either poorly defined or followed closely by ultimate failure. Therefore, in the provisions of this section, rupture rather than yielding is used as a more reliable criterion of failure.

In addition, the Cornell research has provided the experimental basis for Reference 62. In most cases, the provisions of Reference 62 are in agreement with this *Specification* section.

The tests, which served as the basis of the provisions given in *Specification* Section E2.1 through E2.5, were conducted on sections with single and double sheets (*Specification* Figure E2.2). The largest total sheet thickness of the cover plates was approximately 0.15 inch. Within the *Specification*, the validity of the equations was extended to connections with cover plates of 0.18 inch combined thickness. For arc spot welds the maximum combined thickness is set at 0.15 inch.

The terms used in this *Specification* section agree with the standard nomenclature given in Reference 62.

E2.2 Arc Spot Weld

The thickness limitation is due to the range of the test program that served as the basis of these provisions.

On sheets below 0.028 inch thick, weld washers are required to avoid excessive burning of the sheets and, therefore, inferior quality welds.

The Cornell tests identified four modes of failure for arc spot welds, which are addressed in this *Specification* section. It should be noted that many failures, particularly those of the plate tearing type, may be preceded or accompanied by considerable inelastic out-of-plane deformation of the type indicated in Figure E2.1. This form of behavior is similar to that observed in wide, pin-connected plates. Such behavior should be avoided by closer spacing of welds.

E2.3 Arc Seam Welds

The general behavior of arc seam welds is similar to that of arc spot welds. No simple shear failures of arc seam welds were observed in the Cornell tests. Therefore, Equation E2.3-1 which accounts for shear failure, is adopted from Reference 62.

Equation E2.3-2 is intended to prevent failure by a combination of tensile tearing plus shearing of the cover plates.

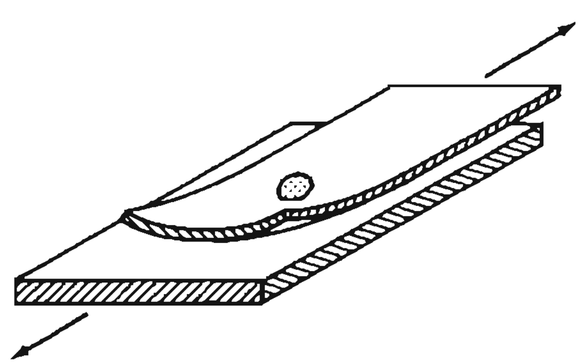


Figure E2.1 Out-of-Plane Distortion

E2.4 Fillet Welds

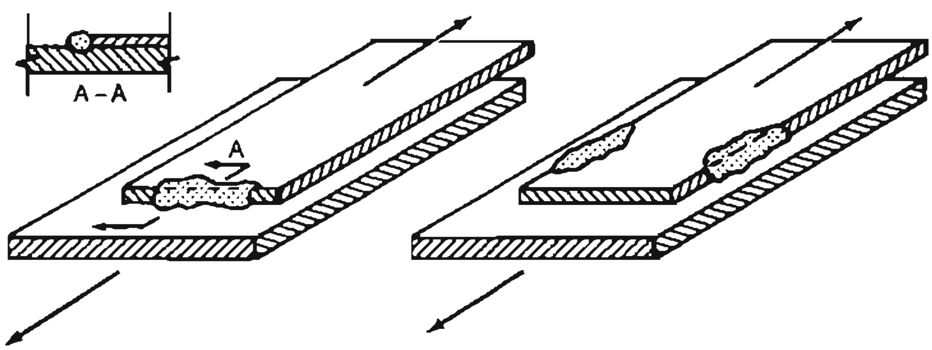
For fillet welds on the lap-joint specimens tested in the Cornell research, the dimension, w_1, of the leg on the sheet edge generally was equal to the sheet thickness; the other leg, w_2, often was two or three times longer than w_1. In connections of this type, the fillet weld throat commonly is larger than the throat of a conventional fillet weld of the same size (see *Specification* Figure E2.4a). Usually ultimate failure of fillet welded joints is found to occur by the tearing of the plate adjacent to the weld.

In most cases, the higher strength of the weld material prevents weld shear failure, therefore the provisions of this *Specification* section are based on sheet tearing. Because specimens up to 0.15 inch thickness were tested in the Cornell research, the last provision is this section is to cover the possibility that for sections thicker than 0.15 inch, the throat dimension may be less than the thickness of the cover plate and the tear may occur in the weld rather than in the plate material.

E2.5 Flare Groove Welds

The chief mode of failure in cold-formed channels welded by flare groove welds, loaded transversely or longitudinally, also was sheet tearing along the contour of the weld.

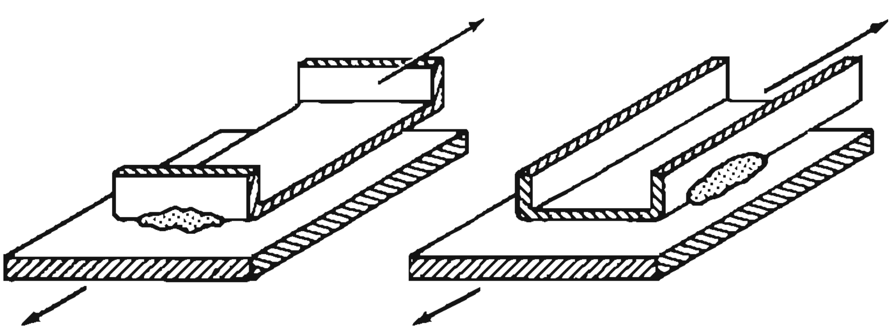
Except for Equation E2.5-4, the provisions of this *Specification* section are intended to prevent shear tear failure. This equation covers the possibility that thicker channels may have effective throats less than the thickness of the channel and weld failure may become critical.



a. Transverse Fillet Sheet Tear

b. Longitudinal Fillet Sheet Tear

Figure E2.4 Fillet Weld Failure Modules



Transverse Sheet Tear

Longitudinal Sheet Tear

Figure E2.5 Flare Groove Weld

E2.6 Resistance Welds

The shear values listed for outside sheets of 0.125 inches or less in thickness are based on "Recommended Practice for Resistance Welding Coated Low-Carbon Steels," AWS C1.3-70, (Table 2.1—Spot Welding Galvanized Low-Carbon Steel). Shear values for outside sheets thicker than 0.125 are based upon "Recommended Practices for Resistance Welding," AWS C1.1-66, (Table 1.3—Pulsation Welding Low-Carbon Steel) and apply to pulsation welding as well as spot welding. They are applicable for all structural grades of low-carbon steel, uncoated or galvanized with 0.90 oz/ft^2 of sheet, or less, and are based on a factor of safety of approximately 2½ applied to selected values from AWS C1.3-70 Table 2.1 and AWS C1.1-66 Table 1.3. Values for intermediate thicknesses may be obtained by straight-line interpolation. The above values may also be applied to medium-carbon and low-alloy steels. Spot welds in such steels give somewhat higher shear strengths than those upon which the above values are based; however, they may require special welding conditions. In all cases welding shall be performed in accordance with AWS C1.3-70 and AWS C1.1-66. (References 63 and 64)

E3 Bolted Connections

The AISI design provisions for bolted connections were revised in the *Specification* to reflect the results of continuing research (References 23 and 65) and general practice as well as to provide a better coordination with the specifications of the Research Council on Structural Connections (RCSC) (Reference 66) and AISC (Reference 10). In the revised design provisions, several additions were made concerning the maximum size of holes (Table E3) and the allowable tension stress for bolts.

(a) *Scope*

Previous studies and practical experiences have indicated that the structural behavior of bolted connections used for joining relatively thick cold-formed steel members is similar to that for connecting hot-rolled shapes and build-up members. The *Specification* criteria are applicable only to cold-formed steel members or elements less than 3/16 inch in thickness. For materials not less than 3/16 inch, reference is made to the AISC *Specification* (Reference 10).

Because of a lack of appropriate test data and the use of numerous surface conditions, this *Specification* does not provide design criteria for slip-critical (also called friction-type) connections. When such connections are used with cold-formed members where the thickness of the thinnest connected part is less than 3/16 inch, it is recommended that tests be conducted to confirm their design capacity. The test data should verify that the specified design capacity for the connection provides a sufficient safety factor against initial slip at least equal to that implied by the AISC provisions. In addition, the safety factor against ultimate capacity should be at least equal to that implied by this *Specification* for bearing-type connections.

The Specification provisions apply only when there are no gaps between plies. The designer should recognize that the connection of a rectangular tubular member by means of bolt(s) through such members may have less strength than if no gap existed. Structural performance of connections containing unavoidable gaps between plies would require tests in accordance with *Specification* Section F1.

(b) *Materials*

This section lists five different types of fasteners which are normally used for cold-formed steel construction. In view of the fact that A325 and A490 bolts are available only for diameters of ½ inch and larger, A449 and A354 Grade BD bolts should be used as an equivalent of A325 and A490 bolts, respectively, whenever smaller bolts (less than ½ inch in diameter) are required.

During recent years, other types of fasteners, with or without special washers, have been widely used in steel structures using cold-formed members. The allowable stresses for design of these fasteners should be determined by tests in accordance with Section F of this *Specification*.

(c) *Bolt Installation*

Bolted connections in cold-formed steel structures use either mild or high-strength steel bolts and are designed on the basis of bearing strength. Bolt pretensioning, as is

customary in hot-rolled steel construction, is not required because the ultimate strength of a bolted connection is independent of the level of bolt preload. Installation is therefore essentially limited to ensuring that the bolted assembly will not come apart during service. Experience has shown that bolts installed to a snug tight condition do not loosen or "back-off" under normal building conditions.

Bolts in slip-critical connections must be tightened in a manner which assures the development of the fastener tension forces required by AISC for the particular size and type of bolts. Turn-of-nut rotations specified by AISC may not be applicable. This is because such rotations are based on larger grip lengths than are encountered in usual cold-formed construction. Use of load indicator washers is also a possibility for tightening slip-critical connections.

(d) *Hole Sizes*

In Table E3, the maximum size of holes for bolts having diameters not less than ½ inch is based on Table 1.23.4 of the AISC *Specification*, except that for the oversized hole diameter, the AISC Table permits a slightly larger diameter.

For bolts having diameters less than ½ inch, the maximum size of bolt holes is based on previous editions of the AISI *Specification.*

The inclusion of the design information in Table E3 for oversized and slotted holes is because such holes are often used in practice to meet dimensional tolerances during erection. However, when using oversized holes care must be exercised by the designer to ensure that excessive deformation due to slip will not occur at working loads.

Short-slotted holes are usually treated in the same manner as oversized holes. Washers or backup plates should be used over oversized or short-slotted holes in an outer ply unless suitable performance is demonstrated by tests. For connections using long-slotted holes, *Specification* Section E3.4 required the use of washers or back-up plates and that the shear capacity of bolts be determined by tests because a reduction in strength may be encountered.

E3.1 Spacing and Edge Distance

The AISI provisions for minimum spacing and edge distance are the same as that included in the 1980 *Specification.* The minimum edge distance of each individual connected part, e_{min}, is determined by using the tensile strength of steel (F_u) of the connected part and the F_u/F_y ratio (References 23 and 65).

E3.2 Tension in Connected Part

In the revised criteria, the formulas used for computing the allowable tension force on the net section of connected parts are based on the yield point of steel, F_y, and the tension stress, F_t, depending on the type of joint and the use of washers. The appropriate factor of safety should be used for determining the allowable load.

E3.3 Bearing

The allowable bearing forces determined on the basis of the nominal bearing stresses given in Tables E3.3-1 and -2 are the same as that permitted by the 1980 *Specification.* For the thicknesses of materials not covered in these two tables, the allowable bearing force must be determined by tests in accordance with Section F of the *Specification*, using a safety factor of 2.22.

E3.4 Shear and Tension in Bolts

In the 1986 *Specification*, allowable tension stresses were added in Section E3.4 for bolts subject to tension. The allowable stresses specified for A307 ($d \geq$ ½ inch), A325, and A490 bolts are based on Section 1.5.2.1 of the AISC *Specification* (Reference 10). For A307, A449, and A354 bolts with diameters less than ½ inch, the allowable tension stresses are reduced by 10 percent, as compared with A307, A325 and A490 bolts with diameters not less than ½ inch, because the average ratio of tensile-stress area/gross-area for ¼-inch and ⅜-inch diameter bolts is 0.68, which is about 10 percent less than the average area ratio of 0.75 for ½-inch and 1-inch diameter bolts.

The possibility of pullover of the connected sheet at the bolt head, nut, or washer should be considered, especially in the case of sheathing. For unsymmetrical sections, such as C- and Z-sections used as purlins or girts, the problem is more severe because of the prying action resulting from rotation of the member which occurs as a consequence of loading normal to the sheathing. The designer should refer to applicable product code approvals, product specifications or literature, or tests.

In addition, design provisions for bolts subjected to a combination of shear and tension were added in *Specification* Section E3.4. The design equations given in Table E3.4 are based on Section 1.6.3 of the AISC *Specification* for the design of bolts used in bearing-type connections. The design equations used for A354, A449, and A307 bolts with d > ½ inch were derived from the following Equation:

$$F'_t = 1.25F_t - Af_v \leq F_t$$

in which

F'_t = Reduced allowable tension stress for bolts subjected to a combination of shear and tension
F_t = Allowable tension stress for bolts subject only to tension
A = 1.8 for threads not excluded from shear planes
A = 1.4 for threads excluded from shear planes
f_v = Shear stress in bolt

E4 Shear Rupture

The provisions for allowable shear rupture have been adopted from the AISC *Specification for the Design, Fabrication and Erection of Structural Steel for Buildings* (Nov. 1, 1978). These provisions are considered to yield a conservative estimate of the shear-rupture capacity at the coped end of a beam.

E5 Connections to Other Materials

E5.1 Bearing

The allowable stresses for bearing on masonry have been adopted from the November 1, 1978, edition of the AISC *Specification*.

E5.2 Tension

This *Specification* section has been added to raise the awareness of the design engineer regarding tension on fasteners and the connected parts.

E5.3 Shear

This *Specification* section has been added to raise the awareness of the design engineer.

F. TESTS FOR SPECIAL CASES

Information on test for cold-formed steel diaphragms can be found in Reference 67. A general discussion of structural diaphragms is given in Reference 23.

F1 Tests for Determining Structural Performance

This *Specification* section makes provision for proof of structural adequacy by load tests. This section is restricted to cases where calculation of safe load-carrying capacity of deflection cannot be made in accordance with the provision of this *Specification*.

There are in cold-formed steel (as in other kinds of structures) perfectly acceptable and safe types of construction whose composition or configuration are not covered by provisions of the *Specification*. Their performance and adequacy, therefore, cannot be demonstrated by reference to the *Specification*. For example, apart from those methods of connection covered in the *Specification*, a number of other means of connecting are in use. The fact that these are

not specifically covered in the *Specification* is not intended to exclude their use. However, since structures so connected cannot be calculated according to the *Specification* (at least as to strength of connections), tests according to Chapter F are the only means of supplying proof of structural adequacy. Other similar examples could be cited.

Provision (b) prescribes that the structure, when tested, shall support without failure or harmful distortions, design loads increased by load factors. This is to provide reasonable safety factors against failure or harmful distortions. The safety factor thus obtained is somewhat larger than the basic factor of safety of 1.67 on which the body of the *Specification* is based. This takes into account the uncertainties in translating test results into reliable carrying capacities as mentioned above. Additionally, in some instances, the factors of safety in the *Specification* are larger than 1.67 as can be seen in Table A5-1. For connections, the factors of safety implied in the *Specification* are significantly larger than 2. Therefore, provision (b) is also aimed at ensuring factors of safety consistent with those implied in the *Specification* for connections.

Provision (c) applies when the strength of the test specimen is greater than the minimum specified steel strength. This requires that the test results must be reduced in the ratio of the actual strength to the specified minimum strength to obtain the load-carrying capacity. The provision is self-explanatory regarding similar corrections for tensile strength or sheet thickness.

F2 Tests for Confirming Structural Performance

Members, connections and assemblies which can be designed according to the provisions of Chapter A through E of the *Specification*, need no confirmation of calculated results by test. However, special situations may arise where it is desirable to confirm by test the results of calculations. Tests may be called for by the manufacturer, the engineer, or a third party.

Since design was in accordance with the *Specification*, the higher load factors of *Specification* Section F1 do not apply. All that is needed is that the tested specimen or assembly perform according to the safety factor implied in the applicable specification provision. The appropriate value of the applicable safety factor can generally be obtained from the *Specification* or *Commentary* Table A5.1.

F3 Tests for Determining Mechanical Properties

Explicit methods for utilizing the effects of cold work are incorporated in Section A5.2.2 of the *Specification*. In that section, it is specified that as-formed mechanical properties, in particular the yield strength, can be determined either by full-section tests or by calculating the strength of the corners and computing the weighted average for the strength of corners and flats. The strength of flats can be taken as the virgin strength of the steel before forming, or can be determined by special tension tests on specimens cut from flat portions of the formed section. This *Specification* section spells out in considerable detail the types and methods of these tests, and their number as required for use in connection with *Specification* Section A5.2.2. For details of testing procedures which have been used for such purposes, but which in no way should be regarded as mandatory, see References 7, 19, and 30. A *Stub-Column Test Method for Effective Area of Cold-Formed Steel Columns* is included in Part VII of American Iron and Steel Institute's *Cold-Formed Steel Design Manual*, 1986.

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SUPPLEMENTARY INFORMATION

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TO THE AUGUST 19, 1986, EDITION OF THE

SPECIFICATION FOR THE DESIGN OF COLD-FORMED STEEL STRUCTURAL MEMBERS

Cold-Formed Steel Design Manual - Part III

AMERICAN IRON AND STEEL INSTITUTE
1000 16th STREET, NW
WASHINGTON, DC 20036

1st Printing—March 1987

Produced by W. P. Reyman Associates, Inc. New York

PREFACE

This document, *Part III of the Cold-Formed Steel Design Manual* supplements the *Specification for the Design of Cold-Formed Steel Structural Members*. It contains two different types of information: (a) design procedures of specification nature which are not included in the *Specification* itself, either because they are infrequently used or are regarded as too complex for routine design, and (b) other information intended to assist users of cold-formed steel.

This *Supplementary Information* should be used in conjunction with the other parts of the *Design Manual*, which include *Commentary (Part II), Illustrative Examples (Part IV), Charts and Tables (Part V), Computer Aids (Part VI), and Test Procedures (Part VII)*, in addition to the *Specification (Part I)*.

American Iron and Steel Institute
August 1986

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PART III
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SECTION 1— LINEAR METHOD FOR COMPUTING PROPERTIES OF FORMED SECTIONS

Computation of properties of formed sections may be simplified by using a so-called linear method, in which the material of the section is considered concentrated along the centerline of the steel sheet and the area elements replaced by straight or curved "line elements." The thickness dimension, t, is introduced after the linear computations have been completed.

The total area of the section is found from the relation: Area = $L \times t$, where L is the total length of all line elements.

The moment of inertia of the section, I, is found from the relation: $I = I' \times t$, where I' is the moment of inertia of the centerline of the steel sheet. The section modulus is computed as usual by dividing I or $I' \times t$ by the distance from the neutral axis to *the extreme fiber*, not to the centerline of the extreme element.

First power dimensions, such as x, y, and r (radius of gyration) are obtained directly by the linear method and do not involve the thickness dimension.

When the flat width, w, of a stiffened compression element is reduced for design purposes, the effective design width, b, is used directly to compute the total effective length L_{eff} of the line elements.

The elements into which most sections may be divided for application of the linear method consist of straight lines and circular arcs. For convenient reference, the moments of inertia and location of centroid of such elements are identified in the sketches and formulas in Section 1.1.

The formulas for line elements are exact, since the line as such has no thickness dimension; but in computing the properties of an actual section, where the line element represents an actual element with a thickness dimension, the results will be approximate for the following reasons:

(1) The moment of inertia of a straight actual element about its longitudinal axis is considered negligible.

(2) The moment of inertia of a straight actual element inclined to the axes of reference is slightly larger than that of the corresponding line element, but for elements of like length the error involved is even less than the error involved in neglecting the moment of inertia of the element about its longitudinal axis. Obviously, the error disappears when the element is normal to the axis.

(3) Small errors are involved in using the properties of a linear arc to find those of an actual corner, but with the usual small corner radii the error in the location of the centroid of the corner is of little importance, and the moment of inertia generally negligible. When the mean radius of a circular element is over four times its thickness, as for tubular sections and for sheets with circular corrugations, the errors in using linear arc properties practically disappear.

Using the computed values of I_x, I_y, and I_{xy} the moment of inertia about principal axes of the section can be calculated by the following equation:

$$I_{\substack{Max\\Min}} = \frac{I_x + I_y}{2} \pm \sqrt{\left(\frac{I_y - I_x}{2}\right)^2 + I_{xy}^2}$$

where I_x and I_y are the moment of inertia of the section about x- and y-axis, respectively, and I_{xy} is the product of inertia.

The angle between the x-axis, and the minor axis is

$$\theta = \frac{1}{2}\tan^{-1}\left[\frac{2I_{xy}}{I_y - I_x}\right]$$

Examples of Part IV illustrate the application of the linear method.

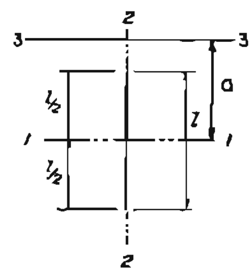
1.1 Properties of Line Elements

1.1.1 Straight Line Elements

Moments of inertia of straight line elements can be calculated using the equations given below:

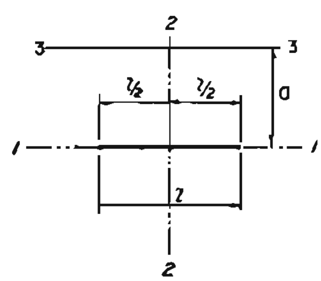
$$I_1 = \frac{l^3}{12}, \qquad I_2 = 0$$

$$I_3 = la^2 + \frac{l^3}{12} = l\left(a^2 + \frac{l^2}{12}\right)$$



$$I_1 = 0, \qquad I_2 = \frac{l^3}{12}$$

$$I_3 = la^2$$

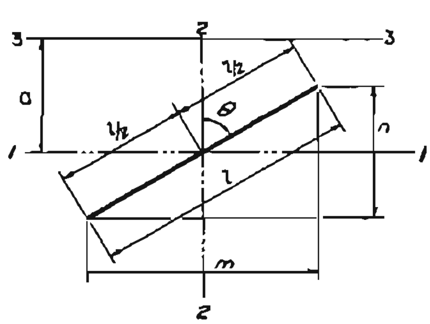


$$I_1 = \left[\frac{\cos^2\theta}{12}\right]l^3 = \frac{ln^2}{12}$$

$$I_2 = \left[\frac{\sin^2\theta}{12}\right]l^3 = \frac{lm^2}{12}$$

$$I_{12} = \left[\frac{\sin\theta\cos\theta}{12}\right]l^3 = \frac{lmn}{12}$$

$$I_3 = la^2 + \frac{ln^2}{12} = l\left(a^2 + \frac{n^2}{12}\right)$$



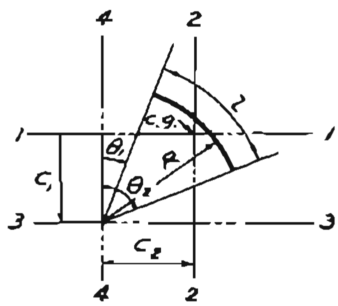
1.1.2 Circular Line Elements

Moments of inertia of circular line elements can be calculated using the equations given below:

θ (expressed in radians) = 0.01745 θ (expressed in degrees and decimals thereof)

$$l = (\theta_2 - \theta_1)\,R$$

$$c_1 = \frac{\sin\theta_2 - \sin\theta_1}{\theta_2 - \theta_1}R, \qquad c_2 = \frac{\cos\theta_1 - \cos\theta_2}{\theta_2 - \theta_1}R$$



$$I_1 = \left[\frac{\theta_2 - \theta_1 + \sin\theta_2\cos\theta_2 - \sin\theta_1\cos\theta_1}{2} - \frac{(\sin\theta_2 - \sin\theta_1)^2}{\theta_2 - \theta_1}\right]R^3$$

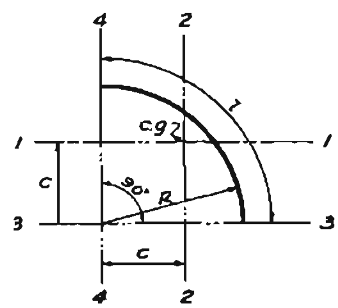
$$I_2 = \left[\frac{\theta_2 - \theta_1 - \sin\theta_2\cos\theta_2 + \sin\theta_1\cos\theta_1}{2} - \frac{(\cos\theta_1 - \cos\theta_2)^2}{\theta_2 - \theta_1}\right]R^3$$

$$I_{12} = \left[\frac{\sin^2\theta_2 - \sin^2\theta_1}{2} + \frac{(\sin\theta_2 - \sin\theta_1)(\cos\theta_2 - \cos\theta_1)}{\theta_2 - \theta_1}\right]R^3$$

$$I_3 = \left[\frac{\theta_2 - \theta_1 + \sin\theta_2\cos\theta_2 - \sin\theta_1\cos\theta_1}{2}\right]R^3,$$

$$I_4 = \left[\frac{\theta_2 - \theta_1 - \sin\theta_2\cos\theta_2 + \sin\theta_1\cos\theta_1}{2}\right]R^3, \quad I_{34} = \left[\frac{\sin^2\theta_2 - \sin^2\theta_1}{2}\right]R^3$$

CASE I: $\theta_1 = 0,\ \theta_2 = 90°$



$l = 1.57\,R,\ c = 0.637\,R$

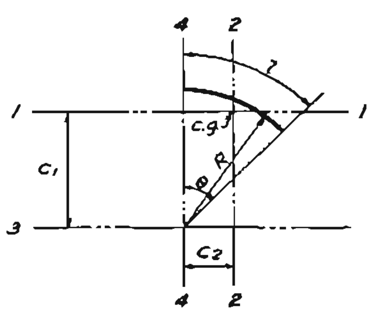
$I_1 = I_2 = 0.149\,R^3$

$I_{12} = -0.137\,R^3$

$I_3 = I_4 = 0.785\,R^3$

$I_{34} = 0.5\,R^3$

CASE II: $\theta_1 = 0,\ \theta_2 = \theta$



$l = \theta R$

$$c_1 = \frac{R\sin\theta}{\theta}$$

$$c_2 = \frac{R(1-\cos\theta)}{\theta}$$

$$I_1 = \left[\frac{\theta + \sin\theta\cos\theta}{2} - \frac{\sin^2\theta}{\theta}\right]R^3, \quad I_2 = \left[\frac{\theta - \sin\theta\cos\theta}{2} - \frac{(1-\cos\theta)^2}{\theta}\right]R^3$$

$$I_{12} = \left[-\frac{\sin^2\theta}{2} + \frac{\sin\theta(\cos\theta - 1)}{\theta}\right]R^3$$

$$I_3 = \left[\frac{\theta + \sin\theta\cos\theta}{2}\right]R^3, \qquad I_4 = \left[\frac{\theta - \sin\theta\cos\theta}{2}\right]R^3$$

$$I_{34} = \left[\frac{\sin^2\theta}{2}\right]R^3$$

1.2 Properties of Sections

Section properties of some sections can be calculated using the equations given below. The following are to be noted:

(1) Three different types of dimensions are employed: capital letters (A) for outside dimensions, lower case barred letters ($\bar{a}$) for centerline dimensions, lower case letters (a) for flat dimensions. The flat dimensions are required to obtain properties such as I where corners are assumed to be round. The centerline dimensions are needed for torsional properties such as C_w where corners are assumed to be square. The outside dimensions are shown because they are the dimensions usually given in tables.

(2) All expressions consider the sections to contain round corners with the exception of those for some torsional properties (m, j and C_w). These expressions are based on a square corner approximation with the exception that round corner values are used for quantities such as moment of inertia which appear in the torsional property expressions. However, allowable stresses calculated by this procedure are sufficiently accurate for routine engineering design of sections with small ratios of corner radius to thickness.

(3) In the moment of inertia calculations, all quantities are accounted for except the moment of inertia of a flat element about its own axis when this is the weak axis. Moments of inertia of corners about their own axis are included to provide for the case of sections with large corner radii.

(4) All expressions are given for the full, unreduced sections.

1.2.1 Equal Angles (Singly-Symmetric) With and Without Lips

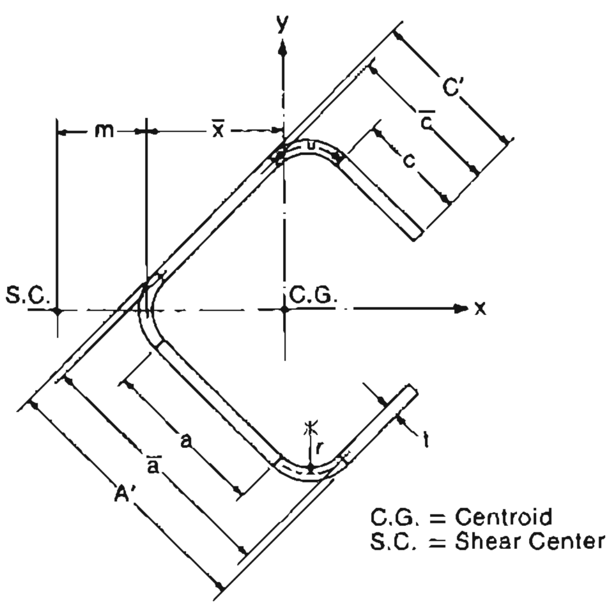


Figure 1.2.1-1
Equal Angle (Singly-Symmetric) With Lips

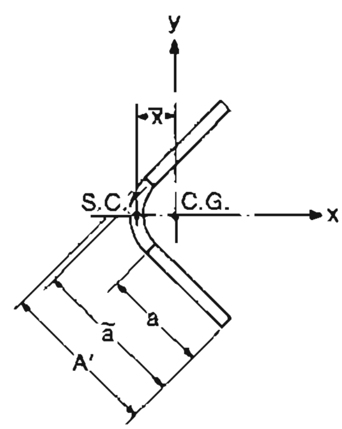


Figure 1.2.1-2
Equal Angle (Singly-Symmetric) Without Lips

NOTE: The x- and y- axes defined in these figures are referred to as the x_2- and y_2- axes in the *Tables of Section Properties, Part V of the Design Manual.*

1. Basic parameters

$a = A' - [r + t/2 + \alpha(r + t/2)]$*

$\bar{a} = A' - [t/2 + \alpha t/2]$

$c = \alpha[C' - (r + t/2)]$

$\bar{c} = \alpha[C' - t\ 2]$

$u = 1.57r$

*For sections with lips, $\alpha = 1.0$; for sections without lips, $\alpha = 0$

2. Cross-sectional area

$$A = t[2a + u + \alpha(2c + 2u)]$$

3. Moment of inertia about x-axis

$$I_x = 2t\{a[\tfrac{1}{2}(a/2 + r)^2 + 0.0417a^2] + 0.143r^3 + \alpha[c(\tfrac{1}{2}(r + a - c/2)^2 + 0.0417c^2)] + u(0.707a + 0.898r)^2 + 0.014r^3\}$$

4. Distance between centroid and centerline of corner

$$\bar{x} = \frac{2t}{A}\left\{a(0.353a + 0.293r) + [(u/2)0.102r] + \alpha[c(0.707a + 0.353c + 1.707r) + u(0.707a + r)]\right\}$$

5. Moment of inertia about y-axis

$$I_y = 2t\{a(0.353a + 0.293r)^2 + 0.0417a^3 + 0.015r^3 + \alpha[u(0.707a + r)^2 + c(0.707a + 0.353c + 1.707r)^2 + 0.417c^3 + 0.285r^3]\} - A(\bar{x})^2$$

6. Distance between shear center and centerline of corner

$$m = \frac{t\bar{a}(\bar{c})^2}{3\sqrt{2}I_x}(3\bar{a} - 2\bar{c})$$

7. St. Venant torsion constant

$$J = \frac{t^3}{3}[2a + u + \alpha(2c + 2u)]$$

8. Warping constant $C_w = \dfrac{t^2(\bar{a})^4(\bar{c})^3}{18\, I_x}(4\bar{a} + 3\bar{c})$

9. Distance from centroid to shear center $x_o = -(\bar{x} + m)$*

10. Parameter used to determine elastic critical moment

$$j = \frac{\sqrt{2}t}{48\, I_y}[(\bar{a})^4 + 4(\bar{a})^3(\bar{c}) - 6(\bar{a})^2(\bar{c})^2 + (\bar{c})^4] - x_o$$

1.2.2 Channels (Singly-Symmetric) With and Without Lips and Hat Sections (Singly-Symmetric)

1. Basic parameters

$a = A' - (2r + t)$
$\bar{a} = A' - t$
$b = B' - [r + t/2 + \alpha(r + t/2)]$
$\bar{b} = B' - (t/2 + \alpha t/2)$
$c = \alpha[C' - (r + t/2)]$
$\bar{c} = \alpha(C' - t/2)$
$u = 1.57r$

2. Cross-sectional area $A = t[a + 2b + 2u + \alpha(2c + 2u)]$

3. Moment of inertia about x-axis

Channel: $$I_x = 2t\{0.0417a^3 + b(a/2 + r)^2 + u(a/2 + 0.637r)^2 + 0.149 r^3 + \alpha[0.0833c^3 + \frac{c}{4}(a - c)^2 + u(a/2 + 0.637r)^2 + 0.149r^3]\}$$

Hat Section: $$I_x = 2t\{0.0417a^3 + b(a/2 + r)^2 + u(a/2 + 0.637r)^2 + 0.149 r^3 + \alpha[0.0833c^3 + \frac{c}{4}(a + c + 4r)^2 + u(a/2 + 1.363r)^2 + 0.149r^3]\}$$

*Negative sign indicates x_o is measured in negative x direction.

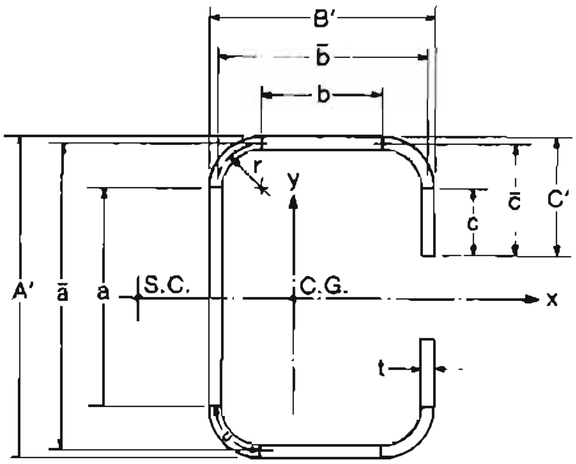


Figure 1.2.2-1
Channel (Singly-Symmetric) With Lips

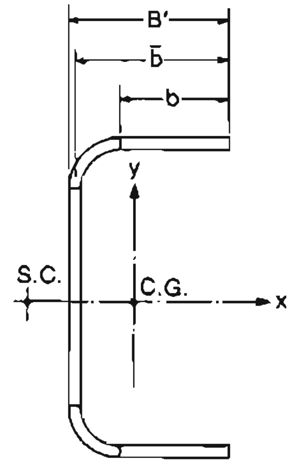
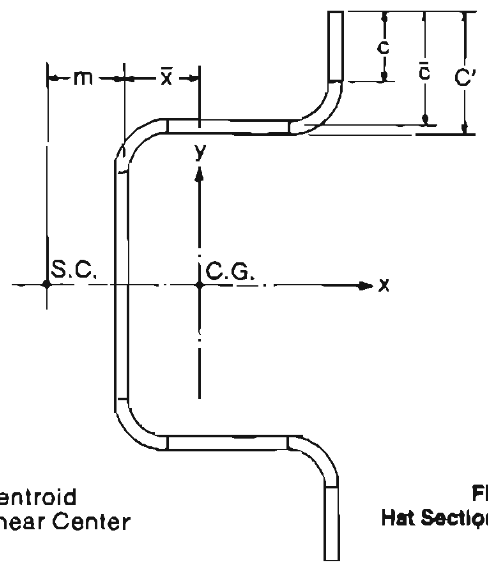


Figure 1.2.2-2
Channel (Singly-Symmetric) Without Lips



C.G. = Centroid
S.C. = Shear Center

Figure 1.2.2-3
Hat Section (Singly-Symmetric)

NOTE: The x- and y- axes defined in these figures are referred to as the x_2- and y_2- axes in the *Tables of Section Properties, Part V of the Design Manual.*

4. Distance between centroid and web centerline

$$\bar{x} = \frac{2t}{A}\{b(b/2 + r) + u(0.363r) + \alpha[u(b + 1.637r) + c(b + 2r)]\}$$

5. Moment of inertia about y-axis

$$I_y = 2t\{b(b/2 + r)^2 + 0.0833b^3 + 0.356r^3 + \alpha[c(b + 2r)^2 + u(b + 1.637r)^2 + 0.149r^3]\} - A(\bar{x})^2$$

6. Distance between shear center and web centerline

$$m = \frac{\bar{b}t}{12I_x}[6\bar{c}(\bar{a})^2 + 3\bar{b}(\bar{a})^2 - 8(\bar{c})^3]$$

7. Distance between centroid and shear center

$$x_o = -(\bar{x} + m)^*$$

8. St. Venant torsion constant

$$J = \frac{t^3}{3}[a + 2b + 2u + \alpha(2c + 2u)]$$

*Negative sign indicates x_o is measured in negative x direction.

9. Warping constant
   a) Channel with lips:

$$C_w = \frac{t^2}{A}\left\{\frac{\bar{x}A(\bar{a})^2}{t}\left[\frac{(\bar{b})^2}{3} + m^2 - m\bar{b}\right]\right.$$

$$+\frac{A}{3t}\left[(m)^2(\bar{a})^3 + (\bar{b})^2(\bar{c})^2(2\bar{c} + 3\bar{a})\right] - \frac{I_x m^2}{t}(2\bar{a} + 4\bar{c})$$

$$+\frac{m(\bar{c})^2}{3}\left[8(\bar{b})^2(\bar{c}) + 2m\Big(2\bar{c}(\bar{c} - \bar{a}) + \bar{b}(2\bar{c} - 3\bar{a})\Big)\right]$$

$$\left. +\frac{(\bar{b})^2(\bar{a})^2}{6}\left[(3\bar{c} + \bar{b})\,(4\bar{c} + \bar{a}) - 6(\bar{c})^2\right] - \frac{m^2(\bar{a})^4}{4}\right\}$$

   Channel without lips:

$$C_w = \frac{t\bar{a}^2\bar{b}^3}{12}\;\frac{(3\bar{b} + 2\bar{a})}{6\bar{b} + \bar{a}}$$

   b) Hat section:

$$C_w = \frac{(\bar{a})^2}{4}\left[I_y + (\bar{x})^2 A\left(1 - \frac{(\bar{a})^2 A}{4I_x}\right)\right]$$

$$+\left[\frac{2(\bar{b})^2 t(\bar{c})^3}{3} - \bar{a}(\bar{b})^2(\bar{c})^2 t + \frac{(\bar{a})^2\bar{b}t(\bar{c})^3\bar{x}A}{3I_x} - \frac{4(\bar{b})^2 t^2(\bar{c})^6}{9I_x}\right]$$

10. Parameter β_w

$$\beta_w = -[0.0833\,(t\bar{x}(\bar{a})^3) + t\,(\bar{x})^3\bar{a}]$$

11. Parameter β_f

$$\beta_f = \frac{t}{2}\left[(\bar{b} - \bar{x})^4 - (\bar{x})^4\right] + \frac{t(\bar{a})^2}{4}\left[(\bar{b} - \bar{x})^2 - (\bar{x})^2\right]$$

12. Parameter β_l

   a) Channel: $\beta_l = 2\bar{c}t(\bar{b} - \bar{x})^3 + \frac{2}{3}t(\bar{b} - \bar{x})\,[(\bar{a}/2)^3 - (\bar{a}/2 - \bar{c})^3]$

   b) Hat section: $\beta_l = 2\bar{c}t\,(\bar{b} - \bar{x})^3 + \frac{2}{3}t(\bar{b} - \bar{x})\,[(\bar{a}/2 + \bar{c})^3 - (\bar{a}/2)^3]$

13. Parameter used in determination of elastic critical moment

$$j = \frac{1}{2I_y}(\beta_w + \beta_f + \beta_l) - x_o$$

1.2.3 I-Sections With Unequal Flanges (Singly-Symmetric) and T-Sections (Singly-Symmetric)

1. Basic parameters
   $a = A' - [r + t/2 + \alpha(r + t/2)]$*
   $\bar{a} = A' - (t/2 + \alpha t/2)$
   $b = B' - (r + t/2)$
   $\bar{b} = B' - t/2$
   $c = \alpha[C' - (r + t/2)]$
   $\bar{c} = \alpha(C' - t/2)$
   $u = 1.57r$

*For I-sections $\alpha = 1.0$; for T-sections $\alpha = 0$

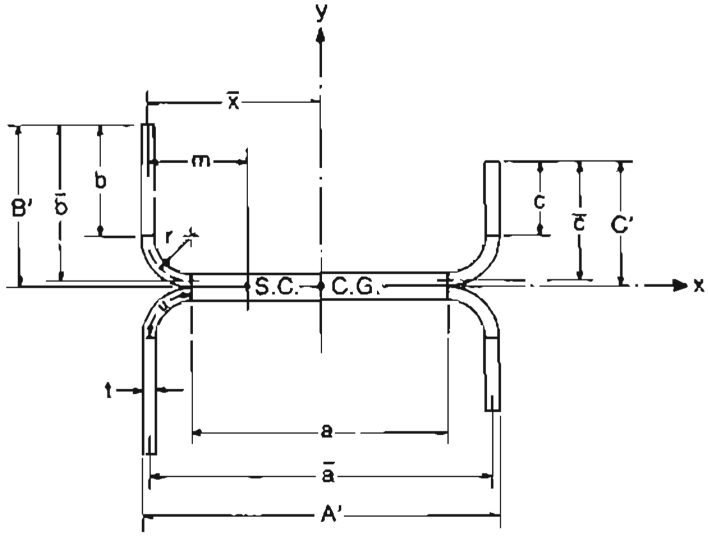


Figure 1.2.3-1
I-Section With Unequal Flanges (Singly-Symmetric)

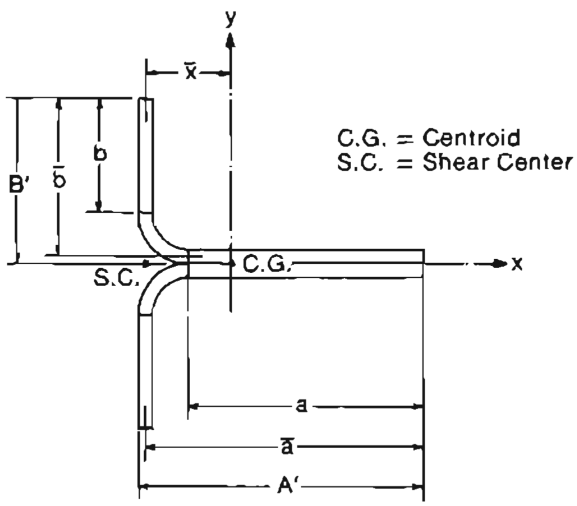


Figure 1.2.3-2
T-Section (Singly-Symmetric)

2. Cross-sectional area

$$A = t[2a + 2b + 2u + \alpha(2c + 2u)]$$

3. Moment of inertia about x-axis

$$I_x = 2t\{b(b/2 + r + t/2)^2 + 0.0833b^3 + u(0.363r + t/2)^2 + 0.149r^3 + \alpha[c(c/2 + r + t/2)^2 + 0.0833b^3 + u(0.363r + t/2)^2 + 0.149r^3]\}$$

4. Distance between centroid and longer flange centerline

$$\bar{x} = \frac{2t}{A}\left\{u(0.363r) + a(a/2 + r) + \alpha[u(a + 1.637r) + c(a + 2r)]\right\}$$

5. Moment of inertia about y-axis

$$I_y = 2t\{0.358r^3 + a(a/2 + r)^2 + 0.0833a^3 + \alpha[u(a + 1.637r)^2 + 0.149r^3 + c(a + 2r)^2]\} - A(\bar{x})^2$$

6. Distance between shear center and longer flange centerline

$$m = \bar{a}\left(1 - \frac{(\bar{b})^3}{(\bar{b})^3 + (\bar{c})^3}\right)$$

7. Distance between shear center and centroid

$$x_o = -(\bar{x} - m)^*$$

8. St. Venant torsion constant

$$J = \frac{2t^3}{3}[a + b + u + \alpha(u + c)]$$

9. Warping constant
For I-Sections the value of C_w is twice the value of each channel if fastened at the middle of the webs; however, if the two channels are continuously welded at both edges of the web to form the I-Section, the warping constants are as follows:

Unlipped I-Sections and T-Sections

$$C_w = \frac{t\bar{a}^2}{12}\left(\frac{8(\bar{b})^3(\bar{c})^3}{(\bar{b})^3 + (\bar{c})^3}\right)$$

For double symmetric, lipped I-Sections

$\bar{c}$ = length of lip, see Figure 1.2.3-1

$$C_w = \frac{t(\bar{b})^2}{3}\left((\bar{a})^2\bar{b} + 3(\bar{a})^2\bar{c} + 6\bar{a}(\bar{c})^2 + 4(\bar{c})^3\right)$$

10. Parameter used in determination of elastic critical moment

$$j = \frac{t}{2I_y}\left\{-2\bar{x}\bar{b}\left((\bar{x})^2 + (\bar{b})^2/3\right) + 2\bar{c}(\bar{a} - \bar{x})\left[(\bar{a} - \bar{x})^2 + (\bar{c})^2/3\right] + \frac{1}{4}\left[(\bar{a} - \bar{x})^4 - (\bar{x})^4\right]\right\} - x_o$$

1.2.4 Z-Sections (Point-Symmetric) With and Without Lips

1. Basic parameters
$a = A' - (2r + t)$
$\bar{a} = A' - t$
$b = B' - [r + t/2 + \alpha(r + t/2)]^{**}$
$\bar{b} = B' - (t/2 + \alpha t/2)$
$c = \alpha[C' - (r + t/2)]$
$\bar{c} = \alpha(C' - t/2)$
$u = 1.57r$

2. Cross-sectional area

$$A = t[a + 2b + 2u + \alpha(2c + 2u)]$$

3. Moment of inertia about x-axis

$$I_x = 2t\{0.0417a^3 + b(a/2 + r)^2 + u(a/2 + 0.637r)^2 + 0.149r^3 + \alpha[0.149r^3 + u(a/2 + 0.637r)^2 + 0.0833c^3 + \frac{c}{4}(a - c)^2]\}$$

4. Moment of inertia about y-axis

$$I_y = 2t\{b(b/2 + r)^2 + 0.0833b^3 + 0.356r^3 + \alpha[c(b + 2r)^2 + u(b + 1.637r)^2 + 0.149r^3]\}$$

*Negative sign indicates x_o is measured in negative x direction.
**For sections with lips $\alpha = 1.0$, for sections without lips, $\alpha = 0$

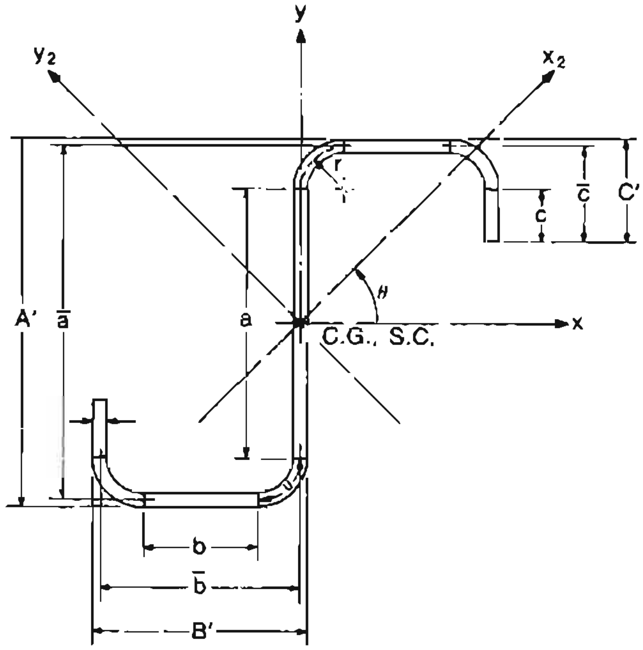


Figure 1.2.4-1
Z-Section (Point-Symmetric) With Lips

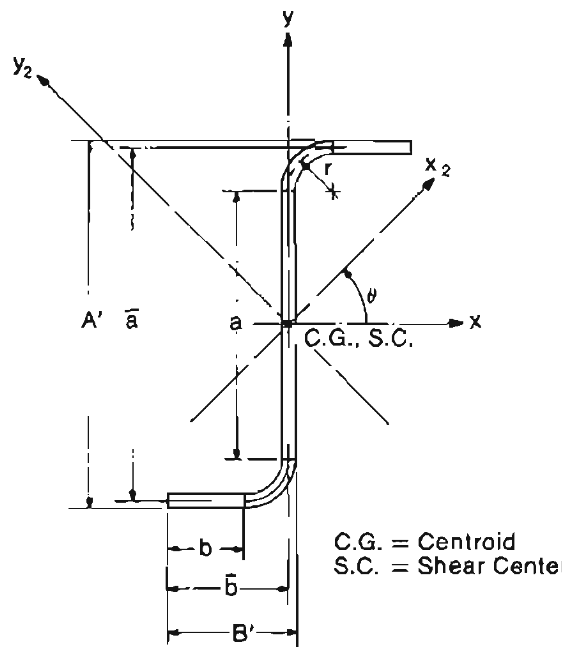


Figure 1.2.4-2
Z-Section (Point Symmetric) Without Lips

5. Product of inertia (See note below)

$$I_{xy} = 2t\{b(a/2 + r)(b/2 + r) + 0.5r^3 + 0.285ar^2 + \alpha[c(2r + b)(a/2 - c/2) - 0.137r^3 + u(b + 1.637r)(0.5a + 0.637r)]\}$$

6. Location of principal axis (See note below)

$$2\theta = \arctan\frac{2I_{xy}}{I_y - I_x}$$

7. Moment of inertia about x_2 axis (See note below)

$$I_{x2} = I_x\cos^2\theta + I_y\sin^2\theta - 2I_{xy}\sin\theta\cos\theta$$

8. Moment of inertia about y_2 axis (See note below)

$$I_{y2} = I_x\sin^2\theta + I_y\cos^2\theta + 2I_{xy}\sin\theta\cos\theta$$

Note: The algebraic signs in Formulas 5, 6, 7 and 8 are correct for the cross-section oriented with respect to the coordinate axes as shown in Figure 1.2.4-1 and Figure 1.2.4-2.

9. Radius of gyration about any axis

$$r = \sqrt{I/A}$$

10. Minimum radius of gyration, about x_2 axis

$$r_{min} = \sqrt{I_{x2}/A}$$

11. St. Venant torsion constant

$$J = \frac{t^3}{3}[a + 2b + 2u + \alpha(2c + 2u)\}$$

12. Warping constant

$$C_w = \frac{t(\bar{b})^2}{12}[(\bar{a})^2((\bar{b})^2 + 2\bar{a}\bar{b} + 4\alpha\bar{b}\bar{c} + 6\bar{a}\bar{c}) + 4\alpha(\bar{c})^2(3(\bar{a})^2 + 3\bar{a}\bar{b} + 4\bar{b}\bar{c} + 2\bar{a}\bar{c} + (\bar{c})^2)]/(\bar{a} + 2\bar{b} + 2\bar{c})$$

SECTION 2—GRAPHICAL DESIGN AIDS

The use of the charts given in Part V are explained in this section. The charts are given for angle, lipped angle, channel, lipped channel and hat sections.

2.1 Design Aids for Specification Section C3.1.2—Lateral Buckling Strength

Buckling parameters σ_{ex} and σ_t used in *Specification* Sections C3.1.2b, C4.2 and D4.1 can be determined using the Charts given in Part V.

Using the value of C_x given in Charts V1.2, V2.2 and V3.2, parameter σ_{ex} can be determined as follows:

$$\sigma_{ex} = \left(\frac{C_x}{\bar{a}^2}\right)\left(\frac{\bar{a}}{K_x L}\right)^2$$

Using the values of C_T and σ_{to} given in Charts V1.3, V1.4, V2.3, V2.4, V3.3 and V3.4, parameter σ_t can be determined as follows:

$$\sigma_t = \frac{\sigma_{to}\bar{a}^2}{t^2}\left(\frac{t}{\bar{a}}\right)^2 + \frac{C_T}{\bar{a}^2}\left(\frac{\bar{a}}{K_t L}\right)^2$$

The lateral buckling moment about the centroidal axis perpendicular to the symmetry axis causing tension on the shear center side of the centroid, M_e can be calculated using the value of W and G_1 determined from Charts V1.6, V1.7, V2.6, V2.7, V3.6 and V3.7 as follows:

$$M_e = A\,\sigma_{ex}\,aWG_1$$

when A is the full cross-sectional area.

2.2 Design Aids for Specification Section C4—Concentrically Loaded Compression Members

2.2.1 Buckling Mode

In *Specifiction* Section C4, F_e is the least of the elastic flexural, torsional or torsional-flexural buckling stress determined according to Sections C4.1 through C4.3. Charts V1.1, V2.1 and V3.1 provide an easy means of determining which buckling mode governs. Based on the cross-sectional dimensions, the governing mode of buckling is determined as explained in these charts. These charts apply when K_y and K_t are equal to 1.0.

2.2.2 Determination of Buckling Parameters

Parameters σ_{ex} and σ_t are determined as described in Section 2.1 above. The torsional-flexural buckling stress F_e needed in *Specification* Section C4.2 can be determined using the value of F given in Charts V1.5, V2.5 and V3.5 as follows:

$$F_e = F\sigma_{ex}$$

2.3 Design Aids for Specification Section D4.1—Wall Studs in Compression

Parameters σ_x and σ_t are determined as described in Section 2.1 above.

SECTION 3—LATERALLY UNBRACED COMPRESSION FLANGES

There are many situations in cold-formed steel structures where a flexural member is so shaped or connected that it will not buckle laterally as a unit, but where the compression flange or flanges themselves are laterally unbraced and can buckle separately by a deflection of the compression flange relative to the tension flange, accompanied by out-of-plane bending of the web and the rest of the section. An example of such a situation is the use of a hat section as a flexural member in such a manner that the "brims" are in compression.

An accurate analysis of such situations is extremely complex and beyond the scope of routine design procedures. The method outlined below is based on considerable simplifications of an exact analysis. Its results have been checked against more than a hundred tests. It has been found that discrepancies rarely exceed 30 percent on the conservative to 20 percent on the

unconservative side. Thus, this method allows a reasonable estimate of allowable design strength to be made which, if desired, can be further improved by test.

The following design procedure was developed based upon tests on individual roof panels or hat-shaped beams. These members were tested as simply supported members with two concentrated loads thus creating a region of uniform moment. Therefore, the design procedure is applicable only to an individual hat-shape type section having its free flange subjected to compression resulting from flexure. It does not apply to the following:

(1) The compression elements of roof panels interconnected by welds, mechanical fasteners or mechanical seams.

(2) A system comprised of flexural members and panels.

For ease of explanation, the design procedure is presented in the following nine steps:

(1) Determine the location of the neutral axis and define as the "equivalent column" the portion of the beam from the extreme compression fiber to a level which is

$\left(\frac{3c_c - c_t}{12c_c}\right)$ d distance from the extreme compression fiber.

In this expression c_c and c_t are the distances from the neutral axis to the extreme compression and tension fibers respectively; d is the depth of the section.

(2) Determine the distance, y_o, measured parallel to the web, from the centroid of the equivalent column to its shear center. (If the cross section of the equivalent column is of angle or T-shape, its shear center is at the intersection of web and flange; if of channel shape, the location of the shear center is obtained from Section D1.1 of the *Specification*. If the flanges of the channel are of unequal width, for an approximation take w as the mean of the two flange widths, or compute the location of the shear center by rigorous methods.)

(3) To determine the spring constant β, isolate a portion of the member one inch long, apply a force of 0.001 kip perpendicular to the web at the level of the column centroid, and compute the corresponding lateral deflection D of that centroid. Then the spring constant $\beta = 0.001/D$

(4) Calculate $T_o = h/(h + 3.4y_o)$ where h is the distance from the tension flange to the centroid of the equivalent column in inches.

(5) If the flange is laterally braced at two or more points calculate

$P_e = 290{,}000\ I/L^2$, $C = \beta L^2/P_e$, and $L' = 3.7\sqrt[4]{I(h/t)^3}$

where I = moment of inertia of equivalent column about its gravity parallel to web, in.^{4},
L = unbraced length of equivalent column, in.
If C is smaller than or equal to 30, compute
$P_{cr} = TP_e[1 + \beta L^2/(\pi^2 P_e)]$
If C is larger than 30, compute
$P_{cr} = TP_e(0.60 + 0.635\sqrt{\beta L^2/P_e})$
In both cases, $T = T_o$ if L is equal to or greater than L'
$T = LT_o/L'$ if L is less than L'

(6) If the flange is braced at less than two points, compute

$P_{er} = T_o\sqrt{4\beta EI}$

(7) Determine the slenderness ratio of the equivalent column

$(KL/r)_{eq} = 490/\sqrt{P_{er}/A_c}$

where A_c = cross-sectional area of equivalent column.

(8) From paragraph (a) of Section C4.1 of the *Specification*, compute the stress, F_e, corresponding to (KL/r).

(9) The allowable compression bending stress is

$F_{b2} = 1.15F_e(c_c/y_c)$ with a maximum of F

where

c_c = distance from neutral axis of beam to extreme compression fiber, in.

y_c = distance from neutral axis of beam to centroid of equivalent column, in.

SECTION 4—TORSIONAL-FLEXURAL BUCKLING OF NON-SYMMETRICAL SHAPES

Torsional-flexural buckling of non-symmetrical sections is not covered by the *Specification*. These sections can be designed by taking F_e in Section C4 equal to σ_{TFO}.

The elastic torsional-flexural buckling stress, σ_{TFO}, is less than the smallest of the Euler buckling stresses about the x- and y-axes and the torsional buckling stress. The value of σ_{TFO} can be obtained from the following equation by trial and error:

$$\left(\frac{\sigma_{TFO}{}^3}{\sigma_{ex}\sigma_{ey}\sigma_t}\right)\rho-\left(\frac{\sigma_{TFO}{}^2}{\sigma_{ey}\sigma_t}\right)\gamma-\left(\frac{\sigma_{TFO}{}^2}{\sigma_{ex}\sigma_t}\right)\beta-\left(\frac{\sigma_{TFO}{}^2}{\sigma_{ex}\sigma_{ey}}\right)+\frac{\sigma_{TFO}}{\sigma_{ex}}+\frac{\sigma_{TFO}}{\sigma_{ey}}+\frac{\sigma_{TFO}}{\sigma_t}=1$$

The following equation may be used for a first approximation:

$$\sigma_{TFO}=\frac{(\sigma_{ex}\sigma_{ey}+\sigma_{ex}\sigma_t+\sigma_{ey}\sigma_t)}{2\,(\sigma_{ex}\gamma+\sigma_{ey}\beta+\sigma_t)}-\frac{\sqrt{(\sigma_{ex}\sigma_{ey}+\sigma_{ex}\sigma_t+\sigma_{ey}\sigma_t)^2-4(\sigma_{ex}\sigma_{ey}\sigma_t)\,(\sigma_{ex}\gamma+\sigma_{ey}\beta+\sigma_t)}}{2(\sigma_{ex}\gamma+\sigma_{ey}\beta+\sigma_t)}$$

where

$$\sigma_{ex}=\frac{\pi^2E}{(KL/r_x)^2},\ \text{ksi}$$

$$\sigma_{ey}=\frac{\pi^2E}{(KL/r_y)^2},\ \text{ksi}$$

$$\sigma_t=\frac{1}{I_p}\left[GJ+\frac{\pi^2EC_w}{(KL)^2}\right],\ \text{ksi}$$

ρ $= 1-(x_o/r_o)^2-(y_o/r_o)^2$

γ $= 1-(y_o/r_o)^2$

β $= 1-(x_o/r_o)^2$

E = modulus of elasticity = 29,500 ksi

L = unbraced length of compression member, in.

r_x = radius of gyration of cross section about the x-axis, in.

r_y = radius of gyration of cross section about the y-axis, in.

r_o = polar radius of gyration of cross section about the shear center, in.

I_p = polar moment of inertia about shear center, in.^{4} $= Ar_o^2 = I_x + I_y + Ax_o^2 + Ay_o^2$

G = shear modulus = 11,300 ksi

J = St. Venant torsion constant of the cross section, in.^{4} For open sections composed of n segments of uniform thickness $= (1/3)\,(l_1t_1^3 + l_2t_2^3 + \ldots + l_nt_n^3)$

C_w = warping constant of torsion of the cross section, in.^{6}

l_i = length of cross section middle line of segment i, in.

t_i = wall thickness of segment i, in.

x_o = distance from shear center to centroid along the principal x-axis, in.

y_o = distance from shear center to centroid along the principal y-axis, in.

For any section, the values of x_o, y_o and C_w can be computed from the following relationships (terms are defined in Figure 4-1):

$$x_o=\frac{1}{I_x}\int_0^l w_o y t\, ds,\ \text{in.}$$

$y_o \quad = -\frac{1}{I_y} \int_o^l w_c x t \, ds$, in.

$C_w \quad = \int_o^l (w_o)^2 t ds - \frac{1}{A}\left[\int_o^l w_o t ds\right]^2$, in.[6]

where

I_x and I_y = centroidal moments of inertia of the cross section about the principal x- and y-axes, in.[4]

A = total area of the cross section, in.[2]

t = wall thickness, in.

$w_c \quad = \int_o^s R_c \, ds$, in.[2]

$w_o \quad = \int_o^s R_o \, ds$, in.[2]

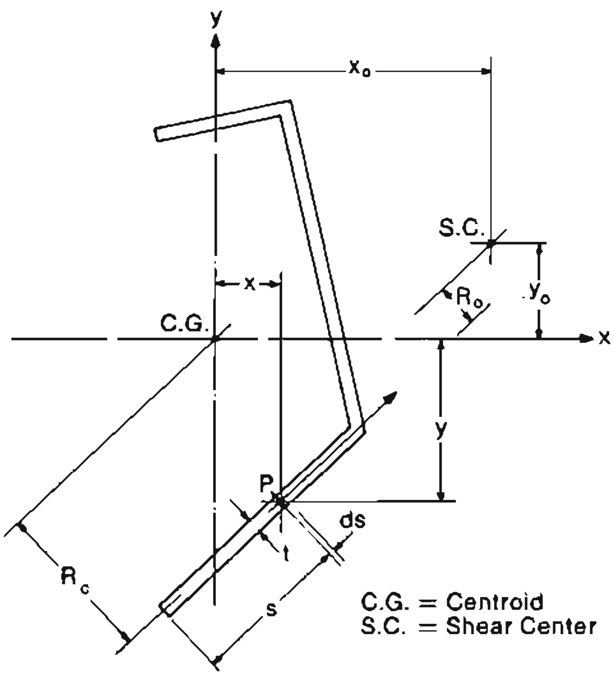


Figure 4-1 Non-Symmetric Cross-Section

x and y = the coordinates measured from the centroid to any point P along the middle line of the cross section, in.

s = distance measured along middle line of cross section from one end to the point P, in.

l = total length of the middle line of the cross section, in.

R_c and R_o = perpendicular distances from the centroid (C.G.) and shear center (S.C.), respectively, to the middle line at P. R_c or R_o is positive if a vector tangent to the middle line at P in the direction of increasing s has a counter-clockwise moment about C.G. or S.C. as shown in Figure 4-1, in.

SECTION 5—SUMMARY OF SCOPE AND PRINCIPLE TENSILE PROPERTIES, ASTM SPECIFICATIONS

ASTM Designation SCOPE (after ASTM)	PRODUCT	GRADE	F_y, ksi (min)	F_u, ksi (min)	Percent elongation in 2 inches (min)	$\frac{F_u}{F_y}$
A36/A36M-84a This specification covers carbon steel shapes, plates, and bars of structural quality for use in riveted, bolted, or welded construction of bridges and buildings, and for general structural purposes. Supplemental requirements are provided where improved notch toughness is important. These shall apply only when specified by the purchaser in the order. When the steel is to be welded, it is presupposed that a welding procedure suitable for the grade of steel and intended use or service will be utilized.	Plates and Bars	—	36	58-80 (range)	23	1.61-2.22
A242/A242M-85 This specification covers high-strength low-alloy structural steel shapes, plates and bars for welded, riveted, or bolted construction intended primarily for use as structural members where savings in weight or added durability are important. These steels have enhanced atmospheric corrosion resistance of approximately two times that of carbon structural steels with copper (Note). This specification is limited to material up to 4 in. [100 mm], inclusive, in thickness. *Note:* Two times carbon structural steel with copper is equivalent to four times carbon structural steel without copper (copper 0.02 max). When the steel is to be welded, it is presupposed that a welding procedure suitable for the grade of steel and intended use or service will be utilized.	Plates and Bars $t \leq \frac{3}{4}$in.	—	50	70	18 (in 8 inches)	1.40
A441/A441M-85 This specification covers high-strength low-alloy structural steel shapes, plates, and bars for welded, riveted, or bolted construction but intended primarily for use in welded bridges and buildings where saving in weight or added durability are important. The atmospheric corrosion resistance of this steel is approximately twice that of structural carbon steel. This specification is limited to material up to 8 in. [200 mm] incl. in thickness. When the steel is to be welded, it is presupposed that a welding procedure suitable for the grade of steel and intended use or service will be utilized.	Plates and Bars	—	50	70	18 (in 8 inches)	1.40
A446/A446M-85 This specification covers steel sheet of structural (physical) quality in coils and cut lengths, zinc-coated (galvanized). Material of this quality is intended primarily where mechanical or structural properties of the base metal are specified or required. Such properties or values include those indicated by tension, hardness, or other commonly accepted mechanical tests. Material of this quality can be produced in six grades, A through F, according to the base metal mechanical requirements prescribed in the adjacent table. Structural (physical) quality galvanized sheet is produced with any of the types of coating and coating designations listed in the latest revision of Specification A525 or A525M.	Sheet	A B C D E F	33 37 40 50 80 50	45 52 55 65 82 70	20 18 16 12 — 12	1.36 1.41 1.38 1.30 1.02 1.40
A500-84 This specification covers cold-formed welded and seamless carbon steel round, square, rectangular, or special shape structural tubing for welded, riveted, or bolted construction of bridges and buildings, and for general structural purposes. This tubing is produced in both welded and seamless sizes with a maximum periphery of 64 in. (1626 mm) and a maximum wall of 0.625 in. (15.88 mm). *Note:* Products manufactured to this specification may not be suitable for those applications such as dynamically loaded elements in welded structures, etc., where low-temperature notch-toughness properties may be important.	Round Tubing Shaped Tubing	A B C A B C	33 42 46 39 46 50	45 58 62 45 58 62	25 23 21 25 23 21	1.36 1.38 1.35 1.15 1.26 1.24

ASTM Designation SCOPE (after ASTM)	PRODUCT	GRADE	F_y, ksi (min)	F_u, ksi (min)	Percent elongation in 2 inches (min)	$\frac{F_u}{F_y}$
A529/A529M-85 This specification covers carbon steel plates and bars ½ in. [13 mm] and under in thickness or diameter and Group 1 shapes shown in Table A of Specification A6/A6M of structural quality for use in metal building system frames, trusses, and related riveted, bolted, or welded construction. When used in welded construction, welding procedures shall be suitable for the steel and the intended service. When the steel is to be welded, it is presupposed that a welding procedure suitable for the grade of steel and intended use or service will be utilized.	Plates and Bars	—	42	60-85 (range)	19 (in 8 inches)	1.43-2.02 (range)
A570-85 This specification covers hot-rolled carbon steel sheet and strip of structural quality in cut lengths or coils. This material is intended for structural purposes where mechanical test values are required, and is available in a maximum thickness of 0.229 in. (6.0 mm) except as limited by Specification A568, A568M, A749, or A749M.	Sheet and Strip	30 33 36 40 45 50	30 33 36 40 45 50	49 52 53 55 60 65	21 to 25 18 to 23 17 to 22 15 to 21 13 to 19 11 to 17	1.63 1.58 1.47 1.38 1.33 1.30
A572/A572M-85 This specification covers four grades of high-strength low-alloy structural steel shapes, plates, sheet piling, and bars. Grades 42 [290] and 50 [345] are intended for riveted, bolted, or welded construction of bridges, buildings, and other structures. Grades 60 [415] and 65 [450] are intended for riveted or bolted construction of bridges, or for riveted, bolted, or welded construction in other applications. For welded bridge construction notch toughness is an important requirement. For this or other applications where notch-toughness requirements are indicated, they shall be negotiated between the purchaser and the producer. The use of columbium, vanadium, and nitrogen, or combinations thereof, within the limitations noted in Section 5, shall be at the option of the producer unless otherwise specified. Where designation of one of these elements or combination of elements is desired, reference is made to Supplementary Requirement S1 in which these elements and their common combinations are listed as to type. When such a designation is desired, both the grade and type must be specified. The maximum thicknesses available in the grades and products covered by this specification are shown in Table 1. When the steel is to be welded, it is presupposed that a welding procedure suitable for the grade of steel and intended use or service will be utilized.	Plates and Bars	42 50 60 65	60 65 75 80		24 21 18 17	1.43 1.30 1.25 1.23
A588/A588M-85 This specification covers high-strength low-alloy structural steel shapes, plates, and bars for welded, riveted, or bolted construction but intended primarily for use in welded bridges and buildings where savings in weight or added durability are important. The atmospheric corrosion resistance of this steel is approximately two times that of carbon structural steel with copper. This specification is limited to material up to 8 in. [200 mm] inclusive in thickness. *Note:* Two times carbon structural steel with copper is equivalent to four times carbon structural steel without copper (Cu 0.02 max). When the steel is to be welded, it is presupposed that a welding procedure suitable for the grade of steel and intended use or service will be utilized.	Plates and Bars, $t \leq 4$ inches	—	50	70	21	1.40
A606-85 This specification covers high-strength, low-alloy, hot-and-cold rolled sheet and strip in cut lengths or coils, intended for use in structural and miscellaneous purposes, where savings in weight or added durability are important. These steels have enhanced atmospheric corrosion resistance and are supplied in two types: Type 2 having corrosion resistance at least two times that of plain carbon steel and Type 4 having corrosion resistance at least four times that of plain carbon steel. The degree of corrosion resistance is based on data acceptable to the consumer. *Note:* Type 2 cold-rolled material is intended to replace ASTM Specification A374, for High-Strength Low-Alloy Cold-Rolled Steel Sheets and Strip, and Type 2 hot-rolled material is intended to replace ASTM Specification A375, for High-Strength Low Alloy Hot-Rolled Steel Sheets and Strip, which appear in the 1971 Annual Book of ASTM Standards, Part 3.	Sheet and Strip	Hot Rolled —As Rolled Cut Lengths Hot Rolled —As Rolled Coils Hot Rolled —Annealed or Normalized Cold Rolled	50 45 45 45	70 65 65 65	22 22 22 22	1.40 1.44 1.44 1.44

ASTM Designation SCOPE (after ASTM)	PRODUCT	GRADE	F_y ksi (min)	F_u ksi (min)	Percent elongation in 2 inches (min)	$\frac{F_u}{F_y}$
A607-85		Class 1				
This specification covers high-strength, low-alloy columbium, or vanadium hot-rolled sheet and strip, or cold-rolled sheet, or combinations thereof, in either cut lengths or coils, intended for applications where greater strength and savings in weight are important. The material is available as two classes. They are similar in strength level except that Class 2 offers improved weldability and more formability than Class 1. Atmospheric corrosion resistance of these steels is equivalent to plain carbon steels. With copper specified, the atmospheric corrosion resistance is twice that of plain carbon steel. Class 1 material was previously A607 without a class designation.	Sheet and Strip	45	45	60	Hot-Rolled 23-25 Cold-Rolled 22	1.33
		50	50	65	Hot-Rolled 20-22 Cold-Rolled 20	1.30
		55	55	70	Hot-Rolled 18-20 Cold-Rolled 18	1.27
		60	60	75	Hot-Rolled 16-18 Cold-Rolled 16	1.25
		65	65	80	Hot-Rolled 14-16 Cold-Rolled 15	1.23
		70	70	85	Hot-Rolled 12-14 Cold-Rolled 14	1.21
		Class 2				
		45	45	55	Hot-Rolled 23-25 Cold-Rolled 22	1.22
		50	50	60	Hot-Rolled 20-22 Cold-Rolled 20	1.20
		55	55	65	Hot-Rolled 18-20 Cold-Rolled 18	1.18
		60	60	70	Hot-Rolled 16-18 Cold-Rolled 16	1.17
		65	65	75	Hot-Rolled 14-16 Cold-Rolled 15	1.15
		70	70	80	Hot-Rolled 12-14 Cold-Rolled 14	1.14
A611-85						
This specification covers cold-rolled carbon steel sheet, in cut lengths or coils. It includes five strength levels designated as Grade A with yield point 25 ksi (170 MPa) minimum; Grade B with 30 ksi (205 MPa) minimum; Grade C with 33 ksi (230 MPa) minimum; Grade D types 1 and 2 with 40 ksi (275 Mpa) minimum; and Grade E with 80 ksi (550 Mpa) minimum.	Sheet	A	25	42	26	1.68
		B	30	45	24	1.50
		C	33	48	22	1.45
		D, Types 1 and 2	40	52	20	1.30
		E	80	82	—	1.02
A715-85						
This specification covers high-strength low-alloy, hot-rolled steel sheet and strip having improved formability when compared with steels covered by Specifications A606 and A607. The product is furnished as either cut lengths or coils and is available in four strength levels, Grades 50, 60, 70, and 80 (corresponding to minimum yield point), an in eight types (according to chemical composition). Not all grades are available in all types. The steel is killed, made to a fine grain practice, and includes microalloying elements such as columbium, titanium, vanadium, zirconium, etc. The product is intended for structural and miscellaneous applications where higher strength, savings in weight, improved formability, and weldability are important.	Sheet and Strip	50	50	60	22 to 24	1.20
		60	60	70	20 to 22	1.17
A792-85a						
This specification covers aluminum-zinc alloy-coated steel sheet in coils and cut lengths coated by the hot-dip process. The aluminum-zinc alloy composition by weight is nominally 55% aluminum, 1.6% silicon, and the balance zinc. The product is intended for applications requiring corrosion resistance or heat resistance or both. Aluminum-zinc alloy-coated sheet is available as Commercial Quality, Lock-Forming Quality, and Structural (Physical) Quality. The available grades of Structural Quality are shown in This Table.	Sheet	33	33	45	20	1.36
		37	37	52	18	1.41
		40	40	55	16	1.38
		50	50	65	12	1.30
		80	80	82	—	1.03

SECTION 6—SUGGESTED COLD-FORMED STEEL STRUCTURAL FRAMING ENGINEERING, FABRICATION, AND ERECTION PROCEDURES FOR QUALITY CONSTRUCTION

General

Those taking advantage of the economies in building construction afforded by cold-formed steel structural framing are warned to observe the procedures outlined herein to obtain quality construction.

Designing

Building design involving cold-formed steel structural framing for floors, roofs, load-bearing walls, or curtain walls should be performed by, or under the supervision of, registered professional structural engineers.

Detailing

Framing drawings should show size, thickness, type, and spacing of all structural members including bridging and bracing. Large-scale details should be included for all connections either welded or screwed. Details should show the method of anchorage of walls to the foundation.

Fabrication

Manufacturers of cold-formed steel structural members maintain in-house quality control programs. Assembly of components into walls, etc., may be done on the job by the "stick built" method or off the the job by the "panelized" method utilizing assembly jigs. Assemblers must follow details shown on fabrication and/or erection drawings.

Erection

Erection should be performed by experienced mechanics who follow the plans and specifications under the supervision of an experienced foreman or superintendent.

Inspection of Construction

Periodic inspections during the construction phase should be made by a professional structural engineer who either is or represents the design engineer of record.

Sign Off

At the conclusion of the cold-formed steel construction process a final inspection should be made by the engineer of record who should certify that the cold-formed steel framing has been constructed in accordance with the plans and specifications and in accordance with all applicable building codes and regulatory requirements.

ILLUSTRATIVE EXAMPLES

ILLUSTRATIVE EXAMPLES

BASED ON THE AUGUST 19, 1986, EDITION OF THE

SPECIFICATION FOR THE DESIGN OF COLD-FORMED STEEL STRUCTURAL MEMBERS

Cold-Formed Steel Design Manual - Part IV

AMERICAN IRON AND STEEL INSTITUTE
1000 16th STREET, NW
WASHINGTON, DC 20036

1st Printing—March 1987

Produced by W. P. Reyman Associates, Inc. New York

PREFACE

This document, *Part IV of the Cold-Formed Steel Design Manual*, contains examples intended to illustrate the application of various provisions of the *Specification.*

These *Illustrative Examples* should be used in conjuction with the other parts of the *Design Manual*, which includes *Commentary (Part II)*, *Supplementary Information (Part III)*, and *Design Aids (Part V)*, in addition to the *Specification (Part I).*

As a general rule, section properties are computed to three significant figures, while dimensions are given to three decimal places. However, in some cases it was impractical to adhere strictly to this guideline.

The weight of the sections is calculated based on steel weighing 40.80 pounds per square foot per inch thickness.

Slight discrepancies should be expected between the calculated section properties given in the examples and the tabulated values given in Part V of the Manual which were calculated by computer.

For the design of compression members, results obtained by utilizing either the graphical or the analytical procedure as outlined in *Part III of the Manual* will differ somewhat. The reason for this is that in the graphical procedure, properties are computed assuming square corners for the section, while the analytical procedure is based on round corners (except for the torsional properties given by C_w j and m which are based on square corners). In general, this will cause only small differences in the results.

The exception occurs when dealing with angle sections. The parameter x which is the distance between the centroid of the section and the centerline of the corner is sensitive to the type of corner utilized. This causes discrepancies in the order of ten percent between the two procedures.

The linear method outlined in *Part III of the Manual* is used for computing the properties of formed sections.

These Illustrative Examples were prepared at Cornell University by graduate students V. Sagan, M. Bou-Shahri, and T. Miller, except for the purlin examples which were prepared by graduate students at the University of Florida.

American Iron and Steel Institute
August 1986

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BASED ON THE
AUGUST 19, 1986, EDITION OF THE
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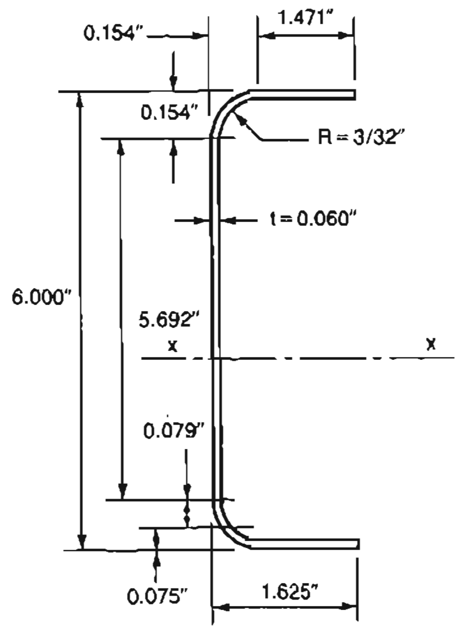
STEEL STRUCTURAL MEMBERS

EXAMPLE NO. 1

CHANNEL SECTION



Given: 1. Steel: $F_y = 50$ ksi.
2. Section: $6 \times 1.625 \times 0.060$ channel with unstiffened flanges.
3. Compression flange braced against lateral buckling.

Required: 1. Allowable moment based on initiation of yielding.
2. Effective moment of inertia based on procedure I for deflection determination at the allowable moment.

Solution:

1. Calculation of the allowable moment:

Properties of 90° corners:
$r = R + t/2 = 3/32 + 0.060/2 = 0.124$ in.
length of arc, $u = 1.57r = 1.57 \times 0.124 = 0.195$ in.
Distance of c.g. from center of radius, $c = 0.637r = 0.637 \times 0.124 = 0.079$ in.

Computation of I_x:
For the first approximation, assume a compression stress of $f = F_y = 50$ ksi in the top fibers of the section and that the web is fully effective.

Compression flange: $k = 0.43$ (unstiffened compression element)

$w/t = 1.471/0.060 = 24.52 < 60$ *OK* (Section B1.1-(a)-(3))

$\lambda = (1.052/\sqrt{k})(w/t)\sqrt{f/E}$ (*Eq.* B2.1-4)

$= (1.052/\sqrt{0.43})(24.52)\sqrt{50/29500} = 1.619 > 0.673$

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

$= [1-(0.22/1.619)]/1.619 = 0.534$

$b = \rho w$ (*Eq.* B2.1-2)

$= 0.534 \times 1.471$

$= 0.786$ in.

Effective section properties about x-axis:

Element	L Effective Length (in.)	y Distance from Top Fiber (in.)	Ly (in.^{2})	Ly^2 (in.^{3})	I'_1 About Own Axis (in.^{3})
Web	5.692	3.000	17.076	51.228	15.368
Upper Corner	0.195	0.075	0.015	0.001	—
Lower Corner	0.195	5.925	1.155	6.846	—
Compression Flange	0.786	0.030	0.024	0.001	—
Tension Flange	1.471	5.970	8.782	52.428	—
Sum	8.339		27.052	110.504	15.368

Distance from top fiber to x-axis is $y_{cg} = 27.052/8.339 = 3.244$ in.

Since distance of top compression fiber from neutral axis is greater than one half the beam depth, a compression stress of 50 ksi will govern as assumed (i.e., initial yield is in compression).

To check if web is fully effective: (Section B2.3)

$$
\begin{aligned}
f_1 &= [(3.244-0.154)/3.244] \times 50 = 47.63 \text{ ksi (compression).}\\
f_2 &= -[(2.756-0.154)/3.244] \times 50 = -40.11 \text{ ksi (tension).}\\
\psi &= f_2/f_1 = -40.11/47.63\\
&= -0.842\\
k &= 4+2(1-\psi)^3+2(1-\psi)\\
&= 4+2[1-(-0.842)]^3+2[1-(-0.842)]\\
&= 20.184\\
h &= w = 5.692 \text{ in.}, \ h/t = w/t = 5.692/0.060 = 94.87\\
&\quad h/t = 94.87 < 200 \ OK \text{ (Section B1.2-(a))}\\
\lambda &= (1.052/\sqrt{20.184})(94.87)\sqrt{47.63/29500} = 0.893 > 0.673\\
\rho &= [1-(0.22/0.893)]/0.893 = 0.844\\
b_e &= 0.844 \times 5.692\\
&= 4.804 \text{ in.}\\
b_2 &= b_e/2 && (Eq.\ B2.3\text{-}2)\\
&= 4.804/2 = 2.402 \text{ in.}\\
b_1 &= b_e/(3-\psi) && (Eq.\ B2.3\text{-}1)\\
&= 4.804/[3-(-0.842)] = 1.250 \text{ in.}\\
b_1+b_2 &= 1.250+2.402\\
&= 3.652 \text{ in.}
\end{aligned}
$$

compression portion of the web calculated on the basis of the effective section $= y_{cg} - 0.154 = 3.244 - 0.154 = 3.090$ in.

Since $b_1 + b_2 = 3.652$ in. > 3.090 in., $b_1 + b_2$ shall be taken as 3.090 in.

This verifies the assumption that the web is fully effective.

$$
\begin{aligned}
I'_x &= Ly^2 + I'_1 - Ly_{cg}^2\\
&= 110.504 + 15.368 - 8.339 \times (3.244)^2\\
&= 38.116 \text{ in.}^3\\
\text{Actual } I_x &= I'_x t\\
&= 38.116 \times 0.060\\
&= 2.287 \text{ in.}^4\\
S_e &= I_x/y_{cg}\\
&= 2.287/3.244\\
&= 0.705 \text{ in.}^3\\
M_n &= S_e F_y && (Eq.\ C3.1.1\text{-}1)\\
&= 0.705 \times 50 = 35.25 \text{ kip-in.}
\end{aligned}
$$

Ω_f = 1.67
M_a = M_n/Ω_f (*Eq.* C3.1-1)
= 35.25/1.67
= 21.11 kip-in. (positive bending)

2. Calculation of Effective moment of inertia based on procedure I for deflection determination at the allowable moment:

The procedure is iterative: one assumes the actual compressive stress f under this allowable moment. Knowing f, proceed as usual to obtain S_e and check to see if $(f \times S_e)$ is equal to M_a as it should. If not reiterate until one obtains the desired level of accuracy.

a. For the first iteration, assume a compression stress of $f = F_y/2 = 25$ ksi in the top fibers of the section and that the web is fully effective.

Compression flange: $\lambda = (1.052/\sqrt{0.43})\,(24.52)\sqrt{25/29500} = 1.145 > 0.673$
ρ = $[1-(0.22/1.145)]/1.145 = 0.706$
b_d = ρw (*Eq.* B2.1-6)
= $0.706 \times 1.471 = 1.039$ in.

Effective section properties about x-axis:
L = $8.339 - 0.786 + 1.039 = 8.592$ in.
Ly = $27.052 - 0.024 + 1.039 \times 0.030 = 27.059$ in.^{2}
Ly^2 = $110.504 - 0.001 + 1.039 \times (0.030)^2 = 110.504$ in.^{3}
I'_x = 15.368 in.^{3}

Y_{cg} = $27.059/8.592 = 3.149$ in. greater than one half beam depth. Thus top compression fiber controls in determination of S_e.

To check if web is fully effective: (Section B2.3-(a),(b))
f_1 = $[(3.149 - 0.154)/3.149] \times 25 = 23.78$ ksi.
f_2 = $-[(2.851 - 0.154)/3.149] \times 25 = -21.41$ ksi.
ψ = $-21.41/23.78 = -0.900$
k = $4 + 2[1-(-0.900)]^3 + 2[1-(-0.900)] = 21.518$
λ = $(1.052/\sqrt{21.518})\,(94.87)\sqrt{(23.78/29500)} = 0.611 < 0.673$
b_e = w (*Eq.* B2.1-1)
= 5.692 in.
b_2 = $5.692/2 = 2.846$ in.
b_1 = $5.692/[3-(-0.900)] = 1.459$ in.

compression portion of the web calculated on the basis of the effective section = $3.149 - 0.154 = 2.995$ in.

Since $b_1 + b_2 = 4.305$ in. > 2.995 in., $b_1 + b_2$ shall be taken as 2.995 in.

This verifies the assumption that the web is fully effective.
I'_x = $110.504 + 15.368 - 8.592 \times (3.149)^2$
= 40.672 in.^{3}

Actual I_x = 40.672×0.060
= 2.440 in.^{4}
S_e = $2.440/3.149 = 0.775$ in.^{3}
M = $f \times S_e = 25 \times 0.775$
= 19.38 k-in. $< M_a = 21.11$ kip-in.

Need to do another iteration and also to increase f.

b. For the second iteration, assume f = 27.65 ksi in the top fibers of the section and that the web is fully effective.

Compression flange: $\lambda = (1.052/\sqrt{0.43})\,(24.52)\sqrt{27.65/29500} = 1.204 > 0.673$

ρ = $[1-(0.22/1.204)]/1.204 = 0.679$
b_d = $0.679 \times 1.471 = 0.999$ in.

Effective section properties about x-axis:

$L = 8.339 - 0.786 + 0.999 = 8.552$ in.

$Ly = 27.052 - 0.024 + 0.999 \times 0.030 = 27.058$ in.^{2}

$Ly^2 = 110.504 - 0.001 + 0.999 \times (0.030)^2 = 110.504$ in.^{3}

$I_1' = 15.368$ in.^{3}

$Y_{cg} = 27.058/8.552 = 3.164$ in. greater than one half beam depth, thus top compression fiber controls in determination of S_e.

To check if web is fully effective:

$f_1 = [(3.164 - 0.154)/3.164] \times 27.65 = 26.30$ ksi.

$f_2 = -[(2.836 - 0.154)/3.164] \times 27.65 = -23.44$ ksi.

$\psi = -23.44/26.30 = -0.891$

$k = 4 + 2[1 - (-0.891)]^3 + 2[1 - (-0.891)] = 21.306$

$\lambda = (1.052/\sqrt{21.306})\,(94.87)\sqrt{26.30/29500} = 0.646 < 0.673$

$b_e = 5.692$ in.

$b_2 = 5.692/2 = 2.846$ in.

$b_1 = 5.692/[3 - (-0.891)] = 1.463$ in.

Compression portion of the web calculated on the basis of the effective section
$= 3.164 - 0.154 = 3.010$ in.

Since $b_1 + b_2 = 4.309$ in. > 3.010 in., $b_1 + b_2$ shall be taken as 3.010 in.

This verifies the assumption that the web is fully effective.

$I_x' = 110.505 + 15.368 - 8.552 \times (3.164)^2$

$= 40.259$ in.^{3}

Actual $I_x = 40.259 \times 0.060 = 2.416$ in.^{4}

$S_e = 2.416/3.164 = 0.764$ in.^{3}

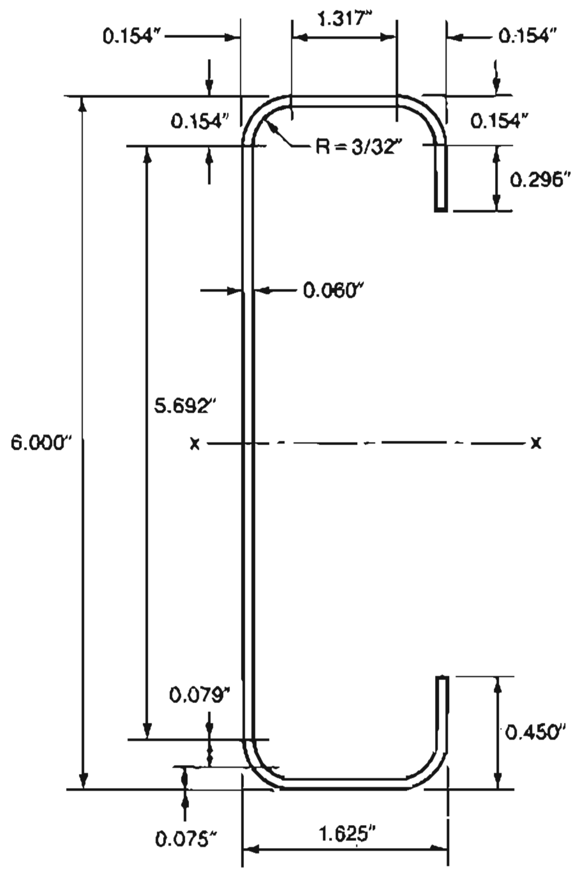
$M = f \times S_e = 27.65 \times 0.764$

$= 21.12$ k-in. close enough to $M_n = 21.11$ kip-in.

Thus $I_x = 2.416$ in.^{4} using procedure I for deflection determination.

EXAMPLE NO. 2

C-SECTION



Given:
1. Steel: $F_y = 50$ ksi.
2. Section: $6 \times 1.625 \times 0.060$ channel with stiffened flanges.
3. Compression flange braced against lateral buckling.

Required:
1. Allowable moment based on initiation of yielding.
2. Effective moment of inertia based on procedure I for deflection determination at the allowable moment.

Solution:

1. Calculation of the allowable moment:

Properties of 90° corners:

$r = R + t/2 = 3/32 + 0.060/2 = 0.124$ in.

length of arc, $u = 1.57r = 1.57 \times 0.124 = 0.195$ in.

Distance of c.g. from center of radius, $c = 0.637r = 0.637 \times 0.124 = 0.079$ in.

Computation of I_x:

for the first approximation, assume a compression stress of $f = F_y = 50$ ksi in the top fibers of the section and that the web is fully effective.

Compression flange:

$w = 1.317$ in.

$w/t = 1.317/0.060 = 21.95$

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

$= 1.28\sqrt{29500/50} = 31.09$

$S/3 = 10.36 < (w/t) = 21.95 < S = 31.09$

$I_a = 399t^4 \{[(w/t)/S] - 0.33\}^3$ (*Eq.* B4.2-6)
$= 399(0.060)^4 [(21.95/31.09) - 0.33]^3$
$= 0.000275$ in.^{4}
$D = 0.450$ in.
$d = 0.296$ in., $d/t = 0.296/0.060 = 4.93 < 60$ *OK* [Section B1.1-(a)-(3)]
$I_s = d^3t/12 = (0.296)^3 (0.060)/12$
$= 0.000130$ in.^{4}
$D/w = 0.450/1.317 = 0.342$, $0.25 < D/w = 0.342 < 0.80$
$k = [4.82 - 5(D/w)](I_s/I_a)^n + 0.43 \leq 5.25 - 5(D/w)$ (*Eq.* B4.2-9)
$n = 1/2$

$[4.82 - 5(D/w)](I_s/I_a)^n + 0.43 = [4.82 - 5(0.342)](0.000130/0.000275)^{1/2} + 0.43$
$= 2.568$
$5.25 - 5(D/w) = 5.25 - 5(0.342)$
$= 3.540 > 2.568$

$k = 2.568$

Since $I_s < I_a$, the stiffener is considered a simple lip.
$w/t = 21.95 < 60$ *OK* [Section B1.1-(a)-(1)]
$\lambda = (1.052/\sqrt{k})(w/t)\sqrt{f/E}$ (*Eq.* B2.1-4)
$= (1.052/\sqrt{2.568})(21.95)\sqrt{50/29500} = 0.593 < 0.673$
$b = w$ (*Eq.* B2.1-1)
$= 1.317$ in. (i.e. compression flange fully effective)

Compression (upper) stiffener:
$k = 0.43$ (unstiffened compression element)
$d/t = 4.93$

Also conservatively assume $f = 50$ ksi as for top compression fiber.
$\lambda = (1.052/\sqrt{0.43})(4.93)\sqrt{50/29500} = 0.326 < 0.673$

therefore,
$d_s' = d = 0.296$ in.
$d_s = d_s'(I_s/I_a) \leq d_s'$ (*Eq.* B4.2-11)
$d_s'(I_s/I_a) = 0.296(0.000130/0.000275)$
$= 0.140$ in. < 0.296 in.
$d_s = 0.140$ in. (i.e. compression stiffener is not fully effective)

Effective section properties about x-axis:

Element	L Effective Length (in.)	y Distance from Top Fiber (in.)	Ly (in.^{2})	Ly^2 (in.^{3})	I_1' About Own Axis (in.^{3})
Web	5.692	3.000	17.076	51.228	15.368
Upper Corners	2×0.195 = 0.390	0.075	0.029	0.002	—
Lower Corners	2×0.195 = 0.390	5.925	2.311	13.691	—
Compression Flange	1.317	0.030	0.040	0.001	—
Upper Stiffener	0.140	0.224	0.031	0.007	—
Tension Flange	1.317	5.970	7.862	46.939	—
Lower Stiffener	0.296	5.698	1.687	9.610	0.002
Sum	9.542		29.036	121.478	15.370

Distance from top fiber to x-axis is $y_{cg} = 29.036/9.542 = 3.043$ in.

Since distance of top compression fiber from neutral axis is greater than one half the beam depth, a compression stress of 50 ksi will govern as assumed (i.e. initial yield is in compression).

To check if web is fully effective (Section B2.3):

$f_1 = [(3.043 - 0.154)/3.043] \times 50 = 47.47$ ksi (compression)

$f_2 = [(2.957 - 0.154)/3.043] \times 50 = -46.06$ ksi (tension)

$\psi = f_2/f_1 = -(46.06/47.47) = -0.970$

$k = 4 + 2(1-\psi)^3 + 2(1-\psi)$
$= 4 + 2[1-(-0.970)]^3 + 2[1-(-0.970)]$
$= 23.231$

$h = w = 5.692$ in., $h/t = w/t = 5.692/0.060 = 94.87$

$h/t = 94.87 < 200$ *OK* [Section B1.2-(a)]

$\lambda = (1.052/\sqrt{23.231})(94.87)\sqrt{47.47/29500} = 0.831 > 0.673$

$\rho = [1-(0.22/\lambda)]/\lambda$ (*Eq.* B2.1-3)
$= [1-(0.22/0.831)]/0.831 = 0.885$

$b_e = \rho w$ (*Eq.* B2.1-2)
$= 0.885 \times 5.692$
$= 5.037$ in.

$b_2 = b_e/2$ (*Eq.* B2.3-2)
$= 5.037/2$
$= 2.519$ in.

$b_1 = b_e/(3-\psi)$ (*Eq.* B2.3-1)
$= 5.037/[3-(-0.970)]$
$= 1.269$ in.

$b_1 + b_2 = 1.269 + 2.519$
$= 3.788$ in.

Compression portion of the web calculated on the basis of the effective section = $y_{cg} - 0.154 = 3.043 - 0.154 = 2.889$ in.

Since $b_1 + b_2 = 3.788$ in. > 2.889 in., $b_1 + b_2$ shall be taken as 2.889 in.

This verifies the assumption that the web is fully effective.

$I'_x = Ly^2 + I'_1 - Ly_{cg}^2$
$= 121.478 + 15.370 - 9.542(3.043)^2$
$= 48.491$ in.^{3}

Actual $I_x = I'_x t$
$= 48.491 \times 0.060$
$= 2.909$ in.^{4}

$S_e = I_x/y_{cg}$
$= 2.909/3.043$
$= 0.956$ in.^{3}

$Mn = S_e F_y$ (*Eq.* C3.1.1-1)
$= 0.956 \times 50$
$= 47.80$ kip-in.

$\Omega_f = 1.67$

$M_a = Mn/\Omega_f$ (*Eq.* C3.1-1)
$= 47.80/1.67$
$= 28.62$ kip-in.

2. Calculation of the effective moment of inertia based on procedure I for deflection determination at the allowable moment:

The procedure is iterative: one assumes the actual compressive stress f under this allowable moment. Knowing f, one proceeds as usual to obtain S_e and checks to see if $(f \times S_e)$ is equal to M_a as it should. If not, reiterate until one obtains the desired level of accuracy. [Section B2.1-(b)-(1)]

(a) For the first iteration, assume a compression stress of $f = Fy/2 = 25$ ksi in the top fibers of the section and that the web is fully effective.

Compression flange:

$S = 1.28\sqrt{29500/25} = 43.97$

$S/3 = 14.66 < (w/t) = 21.95 < S = 43.97$

$I_a = 399(0.060)^4[(21.95/43.97) - 0.33]^3$

$= 0.000025$ in.^{4}

$I_s/I_a = 0.000130/0.000025 = 5.20$

$k = [4.82 - 5(0.342)](5.20)^{1/2} + 0.43 = 7.522 > 3.540$

$k = 3.540$

$\lambda = (1.052/\sqrt{3.540})(21.95)\sqrt{25/29500} = 0.357 < 0.673$

$b = 1.317$ in. (i.e. compression flange fully effective)

Compression (upper) stiffener:
Again assume conservatively $f = 25$ ksi as in top compression fiber.

$\lambda = (1.052/\sqrt{0.43})(4.93)\sqrt{25/29500} = 0.230 < 0.673$

therefore $d_s' = 0.296$ in.

Since $I_s/I_a = 5.20 > 1.0$, it follows that $d_s = d_s' = 0.296$ in. (i.e. compression stiffener fully effective). Thus, one concludes that the section is fully effective.

$y_{cg} = 6/2 = 3.000$ in. (from symmetry)

Full section properties about x-axis:

Element	L (in.)	y Distance from Centerline of Section (in.)	Ly^2 (in.^{3})	I_1' About Own Axis (in.^{3})
Web	5.692	—	—	15.368
Stiffeners	$2 \times 0.296 = 0.592$	2.698	4.309	0.004
Corners	$4 \times 0.195 = 0.780$	2.925	6.673	—
Flanges	$2 \times 1.317 = 2.634$	2.970	23.234	—
Sum			34.216	15.372

Since section is singly symmetric about x-axis and fully effective, top compression fiber (and also bottom tension fiber) may be used in computing S_e.

To check if web is fully effective:

$f_1 = [(3.000 - 0.154)/3.000] \times 25 = 23.72$ ksi (compression)

$f_2 = -23.72$ ksi (tension)

$\psi = -23.72/23.72 = -1.000$

$k = 4 + 2[1-(-1)]^3 + 2[1-(-1)] = 24.000$

$\lambda = (1.052/\sqrt{24})(94.87)\sqrt{23.72/29500} = 0.578 < 0.673$

$b_e = w$ (*Eq.* B2.1-1)

$= 5.692$ in.

$b_2 = 5.692/2 = 2.846$ in.

$b_1 = 5.692/[3-(-1)] = 1.423$ in.

$b_1 + b_2 = 4.269$ in.

Compression portion of the web $= 3.000 - 0.154 = 2.846$ in.
Since $b_1 + b_2 = 4.269$ in. > 2.846 in., $b_1 + b_2$ shall be taken as 2.846 in.
This verifies the assumption that the web is fully effective.

I'_x $= 34.216 + 15.372 = 49.588$ in.^{3}
Actual $I_x = 49.588 \times 0.060$
$= 2.975$ in.^{4}
S_e $= 2.975/3.000 = 0.992$ in.^{3}
M $= f \times S_e = 25 \times 0.992$
$= 24.80$ kip-in. $< M_a = 28.62$ kip-in.
Need to do another iteration and also to increase f.

b. For the second iteration, assume a compression stress of $f = 28.85$ ksi in the top fibers of the section and that the web is fully effective.

Compression flange:
$S = 1.28\sqrt{29500/28.85} = 40.93$
$S/3 = 13.64 < w/t = 21.95 < S = 40.93$
$I_a = 399(0.060)^4\,[(21.95/40.93) - 0.33]^3$
$= 0.000045$ in.^{4}
$I_s/I_a = 0.000130/0.000045 = 2.89$

$k = [4.82 - 5(0.342)](2.89)^{1/2} + 0.43 = 5.717 > 3.540$

$k = 3.540$

$\lambda = (1.052/\sqrt{3.540})\,(21.95)\,\sqrt{28.85/29500} = 0.384 < 0.673$
$b = 1.317$ in. (i.e. compression flange fully effective)

Compression (upper) stiffener: f conservatively taken as for top compression fiber
$\lambda = (1.052/\sqrt{0.43})\,(4.93)\,\sqrt{28.85/29500} = 0.247 < 0.673$
$d'_s = 0.296$ in.

Since $I_s/I_a = 2.89 > 1.0$ it follows that $d_s = d'_s = 0.296$ in. (i.e. compression stiffener fully effective). Thus the section is fully effective.

$y_{cg} = 6/2 = 3.000$ in. (from symmetry)

Full section properties are the same as were found in the first iteration. Thus, as before, top compression fiber may be used in computing S_e.

To check if web is fully effective:
$f_1 = [(3.000 - 0.154)/3.000] \times 28.85 = 27.37$ ksi (compression)
$f_2 = -27.37$ ksi (tension)
$\psi = -27.37/27.37 = -1.000$
$k = 24.000$
$\lambda = (1.052/\sqrt{24})\,(94.87)\,\sqrt{27.37/29500} = 0.621 < 0.673$
$b_e = w = 5.692$ in.

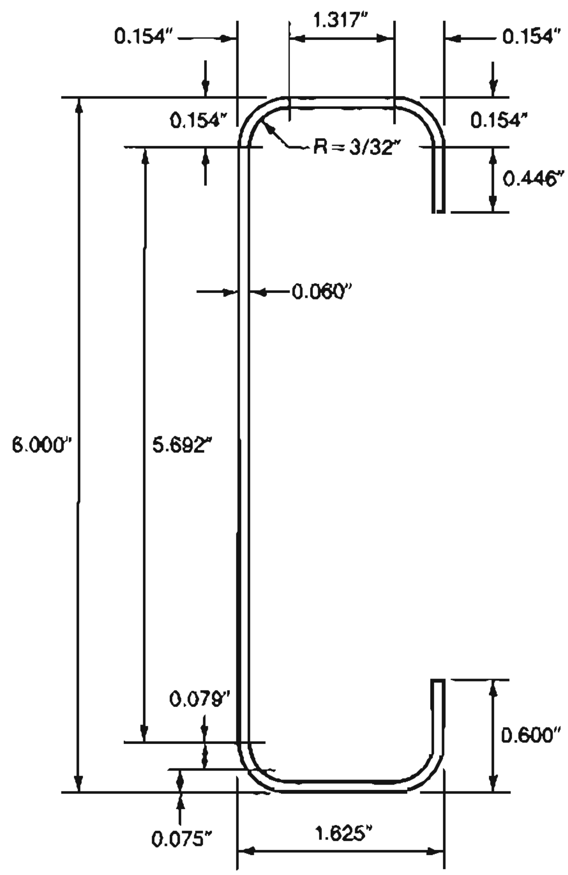
Hence, as in first iteration, $b_1 + b_2 = 2.846$ in. and thus the web is fully effective as assumed.

$I_x = 2.975$ in.^{4}
$S_e = 0.992$ in.^{3}
$M = f \times S_e = 28.85 \times 0.992$
$= 28.62$ kip-in. $= M_a$ *OK*

Thus $I_x = 2.975$ in.^{4} using procedure I for deflection determination.

EXAMPLE NO. 3

C-SECTION BRACED



Given: 1. Steel: $F_y = 50$ ksi.
2. Section: $6 \times 1.625 \times 0.060$ channel with stiffened flanges.
3. Compression flange braced against lateral buckling.

Required: 1. Allowable moment based on initiation of yielding.
2. Effective moment of inertia based on procedure I for deflection determination at the allowable moment.

Solution:

1. Calculation of the allowable moment:

Properties of 90° corners:
$r = R + t/2 = 3/32 + 0.060/2 = 0.124$ in.
length of arc, $u = 1.57r = 1.57 \times 0.124 = 0.195$ in.
Distance of c.g. from center of radius, $c = 0.637r = 0.637 \times 0.124 = 0.079$ in.

Computation of I_x:
for the first approximation, assume a compression stress of $f = F_y = 50$ ksi in the top fibers of the section and that the web is fully effective.

Compression flange:
$w = 1.317$ in.
$w/t = 1.317/0.060 = 21.95$
$S = 1.28\sqrt{(E/f)}$ (*Eq.* B4-1)
$= 1.28\sqrt{29500/50} = 31.09$
$S/3 = 10.36 < w/t = 21.95 < S = 31.09$
$I_a = 399\, t^4\{[(w/t)/S] - 0.33\}^3$ (*Eq.* B4.2-6)

$= 399(0.060)^4[(21.95/31.09) - 0.33]^3$
$= 0.000275 \text{in.}^4$
$D = 0.600$ in.
$d = 0.466$ in., $d/t = 0.446/0.060 = 7.43 < 60$ *OK* (Section B1.1-(a)-(3))
$I_s = d^3t/12 = (0.446)^3(0.060)/12$
$= 0.000444 \text{ in.}^4$
$D/w = 0.600/1.317 = 0.456,\ 0.25 < D/w = 0.456 < 0.80$
$k = [4.82 - 5(D/w)](I_s/I_a)^n + 0.43 \leq 5.25 - 5(D/w)$ (*Eq.* B4.2-9)
$n = 1/2$

$[4.82 - 5(D/w)]\ (I_s/I_a)^n + 0.43 = [4.82 - 5(0.456)]\ (0.000444/0.000275)^{1/2} + 0.43 = 3.657$

$5.25 - 5(D/w) = 5.25 - 5(0.456) = 2.970$

$k = 2.970$

Since $I_s > I_a$ and $D/w < 0.8$, the stiffener is not considered as a simple lip.
$w/t = 21.95 < 90$ *OK* (Section B1.1-(a)-(1))
$\lambda = (1.052/\sqrt{k})\ (w/t)\sqrt{f/E}$ (*Eq.* B2.1-4)
$= (1.052/\sqrt{2.970})\ (21.95)\sqrt{50/29500} = 0.552 < 0.673$
$b = w$ (*Eq.* B2.1-1)
$= 1.317$ in. (i.e., compression flange fully effective)

Compression (upper) stiffener:
$k = 0.43$ (unstiffened compression element)
$d/t = 7.43$

f conservatively taken equal to 50 ksi as in top compression fiber.

$\lambda = (1.052/\sqrt{0.43})\ (7.43)\sqrt{50/29500} = 0.491 < 0.673$

therefore, $d_s' = d = 0.446$ in.

$d_s = d_s'(I_s/I_a) \leq d_s'$ (*Eq.* B4.3-11)

$d_s'(I_s/I_a) = 0.446(0.000444/0.000275)$
$= 0.720 > 0.446$
$d_s = 0.446$ in. (i.e., compression stiffener is fully effective)

Thus, one concludes that the section is fully effective. $Y_{cg} = 6/2 = 3.000$ in. (from symmetry)

Full section properties about x-axis:

Element	L (in.)	y Distance from Centerline of Section (in.)	Ly^2 (in.^{3})	I_1' About Own Axis (in.^{3})
Web	5.692	—	—	15.368
Stiffeners	2 × 0.446 = 0.892	2.623	6.137	0.015
Corners	4 × 0.195 = 0.780	2.925	6.673	—
Flanges	2 × 1.317 = 2.634	2.970	23.234	—
Sum			36.044	15.383

Since section is singly symmetric about x-axis and fully effective, a compression stress of 50 ksi will govern as assumed. (At the bottom tension fibers a tensile stress of 50 ksi will develop simultaneously from symmetry).

To check if web is fully effective: (Section B2.3)

$f_1 = [(3.000 - 0.154)/3.000]\ (50) = 47.43$ ksi (compression)
$f_2 = -47.43$ ksi (tension)

$\psi = f_2/f_1 = -47.43/47.43$
$= -1.000$
$k = 4 + 2(1-\psi)^3 + 2(1-\psi)$
$= 4 + 2[1-(-1)]^3 + 2[1-(-1)]$
$= 24.000$
$h = w = 5.692$ in., $h/t = w/t = 5.692/0.060 = 94.87$
$h/t = 94.87 < 200$ *OK* (Section B1.2-(a))
$\lambda = (1.052/\sqrt{24})\,(94.87)\sqrt{47.43/29500} = 0.817 > 0.673$
$\rho = (1-0.22/\lambda)/\lambda$ (*Eq*. B2.1-3)
$= [1-(0.22/0.817)]/0.817 = 0.894$
$b_e = \rho w$ (*Eq*. B2.1-2)
$= 0.894 \times 5.692$
$= 5.089$ in.
$b_2 = b_e/2$ (*Eq*. B2.3-2)
$= 5.089/2$
$= 2.545$ in.
$b_1 = b_e/(3-\psi)$ (*Eq*. B2.3-1)
$= 5.089/[3-(-1)]$
$= 1.272$ in.
$b_1 + b_2 = 1.272 + 2.545$
$= 3.817$ in.

Compression portion of the web $= Y_{cg} - 0.154$
$= 3.000 - 0.154$
$= 2.846$ in.

Since $b_1 + b_2 = 3.817$ in. > 2.846 in., $b_1 + b_2$ shall be taken as 2.846 in.

This verifies the assumption that the web is fully effective.

$I_x' = Ly^2 + I_1'$
$= 36.044 + 15.383$
$= 51.427$ in.^{3}
Actual $I_x = I_x't$
$= 51.427 \times 0.060$
$= 3.086$ in.^{4}
$S_e = I_x/y_{cg}$
$= 3.086/3.000$
$= 1.029$ in.^{3}
$M_n = S_eF_y$ (*Eq*. C3.1.1-1)
$= 1.029 \times 50$
$= 51.45$ kip-in.
$\Omega_f = 1.67$
$M_a = M_n/\Omega_f$ (*Eq*. C3.1-1)
$= 51.45/1.67$
$= 30.81$ kip-in.

2. Calculation of the effective moment of inertia based on procedure I for deflection determination at the allowable moment:

   The procedure is iterative: one assumes the actual compressive stress f under this allowable moment. Knowing f, one proceed, as usual to obtain S_e and check to see if $(f \times S_e)$ is equal to M_a as it should. If not, reiterate until one obtains the desired level of accuracy. (Section B2.1-(b)-(1))

   a. For the first iteration, assume a compression stress of $f = 30$ ksi in the top fibers of the section and that the web is fully effective.

Compression flange:

$S = 1.28\sqrt{29500/30} = 40.14$
$S/3 = 13.38 < w/t = 21.95 < S = 40.14$
$I_a = 399(0.060)^4[(21.95/40.14) - 0.33]^3$
$= 0.000053 \text{ in.}^4$
$I_s/I_a = 0.000444/0.000053$
$= 8.377$
$k = [4.82 - 5(0.456)]\,(8.377)^{1/2} + 0.43 = 7.783 > 2.970$
$k = 2.970$
$\lambda = (1.052/\sqrt{2.970})\,(21.95)\sqrt{30/29500} = 0.427 < 0.673$
$b = 1.317$ in. (i.e., compression flange fully effective)

Compression (upper) stiffener: f conservatively taken equal to 30 ksi as in top compression fiber.

$\lambda = (1.052/\sqrt{0.43})\,(7.43)\sqrt{30/29500} = 0.380 < 0.673$

therefore, $d'_s = 0.446$ in.

Since $I_s/I_a = 8.377 > 1.0$, it follows that
$d_s = d'_s = 0.446$ in. (i.e., compression stiffener fully effective)

Thus the section is fully effective.
$y_{cg} = 6/2 = 3.000$ in. (from symmetry)

And since the section is singly symmetric about x-axis, top compression fiber (and also bottom tension fiber) may be used in computing S_e.

To check if web is fully effective:

$f_1 = [(3.000 - 0.154)/3.000]\,(30) = 28.46$ ksi (compression)
$f_2 = -28.46$ ksi (tension)
$\psi = f_2/f_1 = -28.46/28.46 = -1.000$
$k = 24.000$
$\lambda = (1.052/\sqrt{24})\,(94.87)\sqrt{28.46/29500} = 0.633 < 0.673$
$b_e = w$ (*Eq.* B2.1-1)
$= 5.692$
$b_2 = 5.692/2 = 2.846$ in.
$b_1 = 5.692/[3 - (-1.000)] = 1.423$ in.
$b_1 + b_2 = 4.269$ in. > compression portion of the web = 2.846 in.

thus $b_1 + b_2$ shall be taken as 2.846 in.

This verifies the assumption that the web is fully effective.

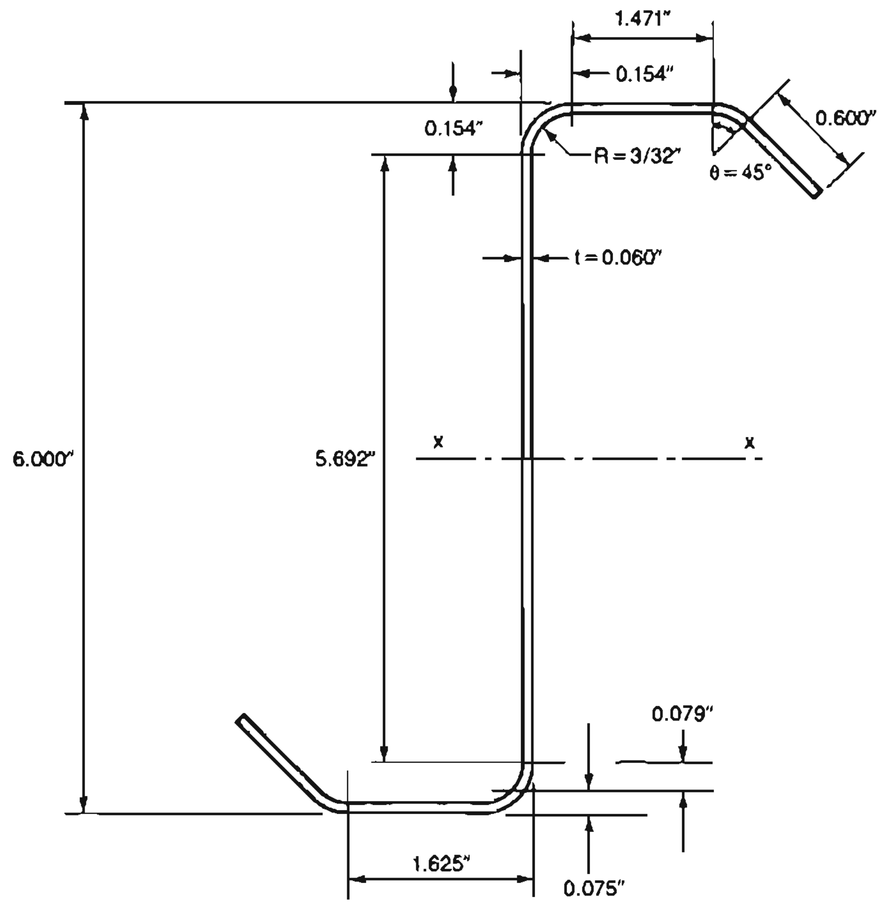
Full section properties are the same as were found in determination of M_a since the section is fully effective.

$I_x = 3.086 \text{ in.}^4$, $S_e = 1.029 \text{ in.}^3$
$M = f \times S_e = 30 \times 1.029$
$= 30.87$ k − in. close enough to $M_a = 30.81$ k − in. and no need for other iterations.

Thus $I_x = 3.086 \text{ in.}^4$ using procedure I for deflection determination.

EXAMPLE NO. 4

Z-SECTION BRACED



Given:
1. Steel: $F_y = 50$ ksi.
2. Section: $6 \times 1.625 \times 0.060$ Z-section with stiffened flanges.
3. Compression flange braced against lateral buckling.

Required:
1. Allowable moment based on initiation of yielding.
2. Effective moment of inertia based on procedure I for deflection determination at the allowable moment.

Solution:

1. Calculation of the allowable moment:

Properties of 90° corners:
$r = R + t/2 = 3/32 + 0.060/2 = 0.124$ in.
length of arc, $u = 1.57r = 1.57 \times 0.124 = 0.195$ in.
Distance of c.g. from center of radius, $c = 0.637r = 0.637 \times 0.124 = 0.079$ in.

Properties of 135° corners:
$r = R + t/2 = 3/32 + 0.060/2 = 0.124$ in.
length of arc, $u = (45°/180°)(3.14)r = 0.785r = 0.785 \times 0.124 = 0.097$ in.

Distance of c.g. from center of radius, $c_1 = r\sin\theta/\theta = [(0.124 \times \sin 45°)/0.785] = 0.112$ in.

Computation of I_x:
For the first approximation, assume a compression stress of $f = F_y = 50$ ksi in the top fibers of the section. For the second approximation, assume that the web is fully effective.

Compression flange:

$w = 1.471$ in.

$w/t = 1.471/0.060 = 24.52$

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

$= 1.28\sqrt{29500/50} = 31.09$

$S/3 = 10.36 < w/t = 24.52 < S = 31.09$

$I_a = 399t^4\{[(w/t)/S] - 0.33\}^3$ (*Eq.* B4.2-6)

$= 399(0.060)^4[(24.52/31.09) - 0.33]^3$

$= 0.000499$ in.^{4}

$d = (0.600$ in., $d/t = 0.600/0.060 = 10 < 60$ *OK* (Section B1.1-(a)-(3))

$D = d + 0.124\tan(\theta/2) = 0.600 + 0.124\tan(45°/2)$

$= 0.651$ in.

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

$= (0.600)^3(0.060)\sin^2(45°)/12$

$= 0.000540$ in.^{4}

$I_s/I_a = 0.000540/0.000499$

$= 1.082$

$D/w = 0.651/1.471 = 0.443,\ 0.25 < D/w = 0.443 < 0.8$

$k = [4.82 - 5(D/w)](I_s/I_a)^n + 0.43 \le 5.25 - 5(D/w)$ (*Eq.* B4.2-9)

$n = 1/2$

$k = [4.82 - 5(D/w)](I_s/I_a)^n + 0.43 = [4.82 - 5(0.443)](1.082)^{1/2} + 0.43 = 3.139$

$k \le 5.25 - 5(D/w) = 5.25 - 5(0.443) = 3.035$

$k = 3.035$

Since $I_s > I_a$ and $D/w < 0.8$, the stiffener is not considered as a simple lip.
$w/t = 24.52 < 90$ *OK* (Section B1.1-(a)-(1))

$\lambda = (1.052/\sqrt{k})(w/t)\sqrt{f/E}$ (*Eq.* B2.1-4)

$= (1.052/\sqrt{3.035})(24.52)\sqrt{50/29500} = 0.609 < 0.673$

$b = w$ (*Eq.* B2.1-1)

$= 1.471$ in. (i.e., compression flange fully effective)

Compression (upper) stiffener:

$k = 0.43$ (unstiffened compression element)

$d/t = 10.00$

f conservatively taken equal to 50 ksi as in top compression fiber.

$\lambda = (1.052/\sqrt{0.43})(10.00)\sqrt{50/29500} = 0.660 < 0.673$

therefore, $d_s' = d = 0.600$ in.

$d_s = d_s'(I_s/I_a) \le d_s'$ (*Eq.* B4.2-11)

$d_s'(I_s/I_a) = 0.600(1.082)$

$= 0.649$ in. > 0.600 in.

$d_s = 0.600$ in. (i.e., compression stiffener is fully effective)

Thus, one concludes that the section is fully effective.

$Y_{cg} = 6/2 = 3.000$ in. (from symmetry)

Full section properties about x-axis:

Element	L (in.)	y Distance from Centerline of Section (in.)	L_y^2 (in.^{3})	I_1' About Own Axis (in.^{3})
Web	5.692	—	—	15.368
Stiffeners	$2 \times 0.600 = 1.200$	2.743	9.029	0.018
90° corners	$2 \times 0.195 = 0.390$	2.925	3.337	—
135° corners	$2 \times 0.097 = 0.194$	2.958	1.697	—
Flanges	$2 \times 1.471 = 2.942$	2.970	25.951	—
Sum			40.014	15.386

Since section is singly symmetric about x-axis and fully effective, a compression stress of 50 ksi will govern as assumed. (At the bottom tension fibers a tensile stress of 50 ksi will develop simultaneously from geometry.)

To check if web is fully effective: (Section B2.3)

$f_1 = [(3.000 - 0.154)/3.000]\,(50) = 47.43$ ksi (compression)
$f_2 = -47.43$ ksi (tension)
$\psi = f_2/f_1 = -47.43/47.43 = -1.000$
$k = 4 + 2(1-\psi)^3 + 2(1-\psi)$
$= 4 + 2[1-(-1.000)]^3 + 2[1-(-1.000)]$
$= 24.000$
$h = w = 5.692$ in., $h/t = w/t = 5.692/0.060 = 94.87$
$h/t = 94.87 < 200$ *OK* (Section B1.2-(a))
$\lambda = (1.052/\sqrt{24})\,(94.87)\sqrt{47.43/29500} = 0.817 > 0.673$
$\rho = [1-(0.22/\lambda)]/\lambda$ (*Eq.* B2.1-3)
$= [1-(0.22/0.817)]/0.817 = 0.894$
$b_e = \rho w$ (*Eq.* B2.1-2)
$= 0.894 \times 5.692$
$= 5.089$ in.
$b_2 = b_e/2$ (*Eq.* B2.3-2)
$= 5.089/2 = 2.545$ in.
$b_1 = b_e/(3-\psi)$ (*Eq.* B2.3-1)
$= 5.089/[3-(-1.000)]$
$= 1.272$ in.
$b_1 + b_2 = 1.272 + 2.545$
$= 3.817$ in.

Compression portion of the web $= y_{cx} - 0.154$
$= 3.000 - 0.154$
$= 2.846$ in.

Since $b_1 + b_2 = 3.817$ in. > 2.846 in., $b_1 + b_2$ shall be taken as 2.846 in.

This verifies the assumption that the web is fully effective.

$I_x' = L_y^2 + I_1'$
$= 40.014 + 15.386$
$= 55.400$ in.^{3}
Actual $I_x = I_x' t$
$= 55.400 \times 0.060$
$= 3.324$ in.^{4}
$S_e = I_x/y_{cg}$
$= 3.324/3.000$
$= 1.108$ in.^{3}
$M_n = S_e F_y$ (*Eq.* C3.1.1-1)
$= 1.108 \times 50$
$= 55.40$ kip-in.

Ω_f = 1.67

M_a = M_n/Ω_f (*Eq.* C3.1-1)

= 55.40/1.67

= 33.17 k − in.

2. Calculation of the effective moment of inertia based on procedure I for deflection determination at the allowable moment:

   The procedure is iterative: one assumes the actual compressive stress f under this allowable moment. Knowing f, one proceeds as usual to obtain S_e and checks to see if $(f \times S_e)$ is equal to M_a as it should. If not, reiterate until one obtains the desired level of accuracy (Section B2.1-(b)-(1)).

   a. For the first iteration, assume a compressive stress of f = 30 ksi in the top fibers of the section and that the web is fully effective.

   Compression flange:

   S = $1.28\sqrt{29500/30} = 40.14$

   S/3 = $13.38 < w/t = 24.52 < S = 40.14$

   I_a = $399(0.060)^4[(24.52/40.14) - 0.33]^3$

   = 0.000115 in.^{4}

   I_s/I_a = 0.000540/0.000115

   = 4.696

   k = $[4.82 - 5(0.451)]\,(4.696)^{1/2} + 0.43 = 5.988 > 2.995$

   k = 2.995

   λ = $(1.052/\sqrt{2.995})\,(24.52)\sqrt{30/29500} = 0.475 < 0.673$

   b = 1.471 in. (i.e., compression flange fully effective)

   Compression (upper) stiffener:

   f conservatively taken equal to 30 ksi as in the top compression fiber.

   λ = $(1.052/\sqrt{0.43})\,(10.00)\sqrt{30/29500} = 0.512 < 0.673$

   therefore d_s' = 0.600 in.

   Since I_s/I_a = $4.696 > 1.0$ it follows that

   d_s = d_s' = 0.600 in. (i.e., compression stiffener is fully effective)

   Thus the section is fully effective.

   y_{cg} = 6/2 = 3.000 in. (from symmetry)

   And since the section is singly symmetric about x-axis, top compression fiber (and also bottom tension fiber) may be used in computing S_e.

   To check if web is fully effective:

   f_1 = $[(3.000 - 0.154)/3.000]\,(30) = 28.46$ ksi (compression)

   f_2 = −28.46 ksi (tension)

   ψ = $f_2/f_1 = -1.000$

   k = 24.000

   λ = $(1.052/\sqrt{24.00})\,(94.87)\sqrt{28.46/29500} = 0.633 < 0.673$

   b_e = w (*Eq.* B2.1-1)

   = 5.692 in.

   b_2 = 5.692/2 = 2.846 in.

   b_1 = 5.692/[3 − (−1.000)] = 1.423 in.

   $b_1 + b_2$ = 4.269 in. > compression portion of the web = 2.846 in.

   Thus $b_1 + b_2$ shall be taken as 2.846 in.

   This verifies the assumption that the web is fully effective.

   Full section properties are the same as were found in determination of M_a since the section is fully effective.

   I_x = 3.324 in.^{4}, S_e = 1.108 in.^{3}

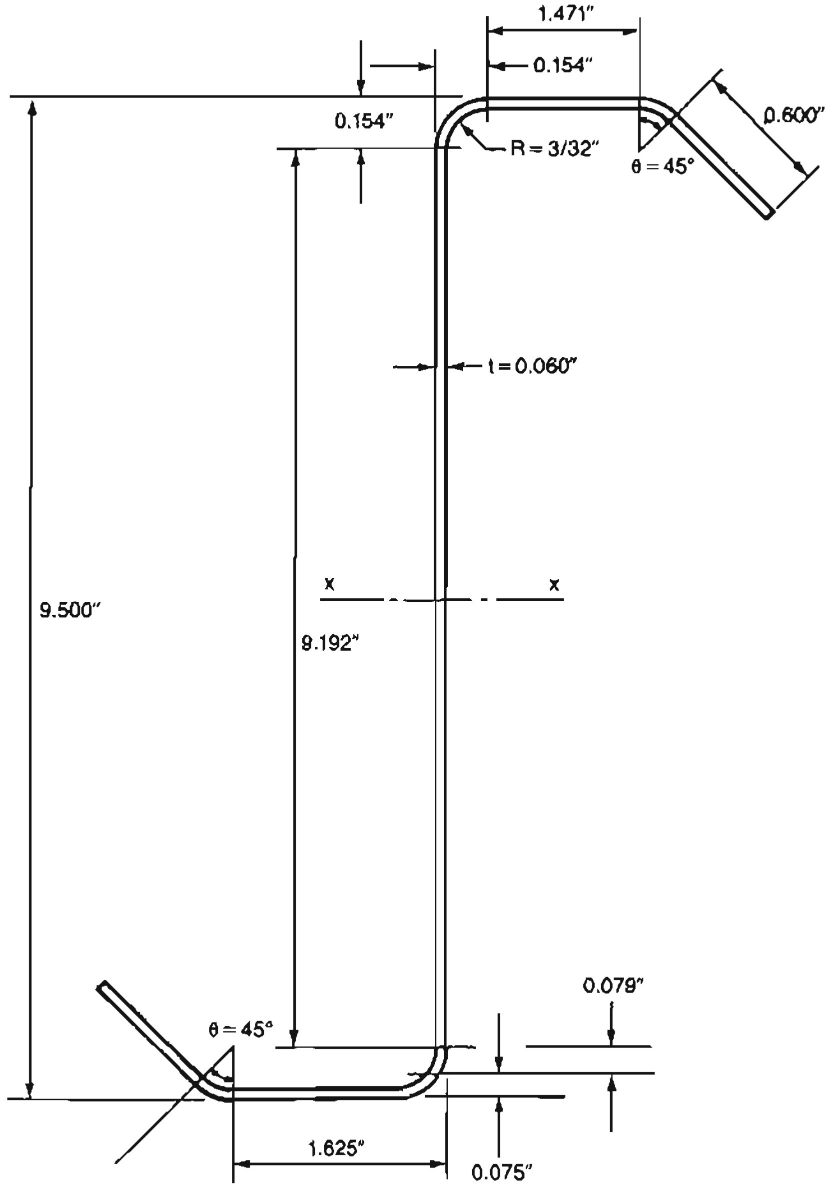
   M = $f \times S_e = 30 \times 1.108$

   = 33.24 k − in. close enough to M_a = 33.17 k − in. and no need for other iterations.

   Thus I_x = 3.324 in.^{4} using procedure I for deflection determination.

EXAMPLE NO. 4A

DEEP-Z SECTION w/STIFFENED FLANGE



Given:
1. Steel: $F_y = 50$ ksi.
2. Section: 9.5 × 1.625 × 0.060 Z-section with stiffened flanges.
3. Compression flange braced against lateral buckling.

Required:
1. Allowable moment based on initiation of yielding.
2. Effective moment of inertia based on procedure I for deflection determination at the allowable moment.

Solution:

1. Calculation of the allowable moment:

   Properties of 90° corners:

   $r = R + t/2 = 3/32 + 0.060/2 = 0.124$ in.

   length of arc, $u = 1.57r = 1.57 \times 0.124 = 0.195$ in.

   Distance of c.g. from center of radius, $c = 0.637r = 0.637 \times 0.124 = 0.079$ in.

Properties of 135° corners:

$r = R + t/2 = 3/32 + 0.060/2 = 0.124$ in.

length of arc, $u = (45°/180°)(3.14)r = 0.785r = 0.785 \times 0.124 = 0.097$ in.

Distance of c.g. from center of radius,

$c_1 = r \sin\theta/\theta = \{0.124 \sin 45°/[(45/180) \times 3.14\} = 0.112$ in.

Computation of I_x:

For the first approximation, assume a compression stress of $f = F_y = 50$ ksi in the top fibers of the section and also assume that the web is fully effective.

Compression flange:

$w = 1.471$ in.

$w/t = 1.471/0.060 = 24.52$

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

$= 1.28\sqrt{29500/50} = 31.09$

$S/3 = 10.36 < w/t = 24.52 < S = 31.09$

$I_a = 399t^4\{[(w/t)/S] - 0.33\}^3$ (*Eq.* B4.2-6)

$= 399(0.060)^4[(24.52/31.09) - 0.33]^3$

$= 0.000499$ in.^{4}

$d = 0.600$ in.

$d/t = 0.600/0.060 = 10 < 60$ *OK* [Section B1.1-(a)-(3)]

$D = d + 0.154 \tan(\theta/2) = 0.600 + 0.154 \tan(45°/2) = 0.6 + 0.064$

$= 0.664$ in.

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

$= (0.600)^3 (0.060) \sin^2(45°)/12$

$= 0.000540$ in.^{4}

$I_s/I_a = 0.000540/0.000499$

$= 1.082$

$D/w = 0.664/1.471 = 0.451,\ 0.25 < D/w = 0.451 < 0.8$

$k = [4.82 - 5(D/w)](I_s/I_a)^n + 0.43 \le 5.25 - 5(D/w)$ (*Eq.* B4.2-9)

$n = 1/2$

$[4.82 - 5(D/w)](I_s/I_a)^n + 0.43 = [4.82 - 5(0.451)](1.082)^{1/2} + 0.43$

$= 3.098$

$5.25 - 5(D/w) = 5.25 - 5(0.451) = 2.995$

Therefore, $k = 2.995$

Since $I_s > I_a$ and $D/w < 0.8$, the stiffener is not considered as a simple lip.

$w/t = 24.52 < 90$ *OK* [Section B1.1-(a)-(1)]

$\lambda = (1.052/\sqrt{k})(w/t)\sqrt{f/E}$ (*Eq.* B2.1-4)

$= (1.052/\sqrt{2.995})(24.52)\sqrt{50/29500} = 0.613 < 0.673$

$b = w$ (*Eq.* B2.1-1)

$= 1.471$ in. (i.e. compression flange fully effective)

Compression (upper) stiffener:

$k = 0.43$ (unstiffened compression element)

$d/t = 10.00$

f conservatively taken equal to 50 ksi as in top compression fiber.

$\lambda = (1.052/\sqrt{0.43})(10.00)\sqrt{50/29500} = 0.660 < 0.673$

therefore,

$d_s' = d = 0.600$ in.

$d_s = d_s'(I_s/I_a) \le d_s'$ (*Eq.* B4.2-11)

since $I_s/I_a = 1.082 > 1.000$

$d_s = d_s' = 0.600$ in. (i.e. compression stiffener is fully effective)

Thus, one concludes that the section is fully effective.

$y_{cg} = 9.5/2 = 4.750$ in. (from symmetry)

It follows that a compression stress of 50 ksi will govern as assumed.

To check if web is fully effective (Section B2.3):

$f_1 = [(4.750 - 0.154)/4.750](50) = 48.38$ ksi (compression)

$f_2 = -48.38$ ksi (tension)

$\psi = f_2/f_1 = -48.38/48.38 = -1.000$

$k = 4 + 2(1-\psi)^3 + 2(1-4)$
$= 4 + 2[i-(-1.000)]^3 + 2[i-(-1.000)]$
$= 24.000$

$h = w = 9.192$ in., $h/t = w/t = 9.192/0.060 = 153.20$

$h/t = 153.20 < 200$ *OK* [Section B1.2-(a)]

$\lambda = (1.052/\sqrt{24})(153.20)\sqrt{48.38/29500} = 1.332 > 0.673$

$\rho = [1-(0.22/\lambda)]/\lambda$ (*Eq.* B2.1-3)
$= [1-(0.22/1.332)]/1.332 = 0.627$

$b_e = \rho w$ (*Eq.* B2.1-2)
$= 0.627 \times 9.192$ in.
$= 5.763$ in.

$b_2 = b_e/2$ (*Eq.* B2.3-1)
$= 5.763/2$
$= 2.882$ in.

$b_1 = b_e/(3-\psi)$ (*Eq.* B2.3-2)
$= 5.763/[3-(-1.000)]$
$= 1.441$ in.

$b_1 + b_2 = 1.441 + 2.882$
$= 4.323$ in.

Compression portion of the web $= y_{cg} - 0.154$
$= 4.750 - 0.154$
$= 4.596$ in.

Since $b_1 + b_2 = 4.323$ in. < 4.596 in., it follows that the web is not fully effective. Hence $y_{cg} \neq 4.750$ as assumed.

The procedure to determine the location of the neutral axis (N.A.) based on partially effective web is iterative. We start with $y_{cg} = 4.750$ and from Figure B.2.3-1 scale b_1, b_2 already computed with respect to $y_{cg} = 4.750$ in. Then we proceed to compute a new N.A. and hence $b_1 + b_2$. If $b_1 + b_2$ same as before, the solution stabilizes and the location of N.A. is calculated according to this $(b_1 + b_2)$. If $(b_1 + b_2)$ differ than before, one reiterates in the same beforementioned manner until $b_1 + b_2$ stabilizes.

Thus for the first iteration, the web is divided into three segments: $b_1 = 1.441$ in., ineffective portion of web, and $b_2(= 2.882) + 4.750 - 0.154 = 7.478$ in. Thus the ineffective portion of web $= 9.192 - 1.441 - 7.478 = 0.273$ in.

The compression flange and stiffener remain fully effective since nothing is altered in their calculations.

Effective section properties about x-axis:

Element	L (in.)	y Distance from Top Fiber (in.)		Ly (in.²)
b_1	1.441	0.154 + (1.441/2)	= 0.875	1.261
$b_2 + (9.5 - y_{cg}) - 0.154$	7.478	9.5 − 0.154 − (7.478/2)	= 5.607	41.929
Compression flange	1.471		0.030	0.044
Compression stiffener	0.600	0.064 cos 45° + (0.600/2) cos 45°	= 0.257	0.154
Top 90° corner	0.195		0.075	0.015
Top 135° corner	0.097	0.154 − 0.112	= 0.042	0.004
Bottom 135° corner	0.097	9.5 − (0.154 − 0.112)	= 9.458	0.917
Bottom 90° corner	0.195	9.5 − 0.075	= 9.425	1.838
Bottom stiffener	0.600	9.5 − 0.257	= 9.243	5.546
Tension flange	1.471	9.5 − (0.060/2)	= 9.470	13.930
Sum	13.645			65.638

y_{cg} = Ly/L = 65.638/13.645
= 4.810 in. (measured from top compression fiber)

f_1 = [(4.810 − 0.154)/4.810](50) = 48.40 ksi (compression)
f_2 = −[(9.5 − 4.810 − 0.154)/4.810](50) = −47.15 ksi (tension)
ψ = −47.15/48.40
= −0.974
k = $4 + 2[1-(-0.974)]^3 + 2[1-(-0.974)]$
= 23.332
λ = $(1.052/\sqrt{23.332})(153.20)\sqrt{48.40/29500} = 1.351 > 0.673$
ρ = [1 − (0.22/1.351)]/1.351 = 0.620
b_e = 0.620 × 9.192
= 5.699 in.
b_2 = 5.699/2 = 2.850 in.
b_1 = 5.699/[3 − (−0.974)]
= 1.434 in.
$b_1 + b_2$ = 4.284 in. ≠ 4.323 in.
therefore need to reiterate

For the second iteration:
b_1 = 1.434 in.
$b_2 + (9.5 - y_{cg}) - 0.154 = 2.850 + 9.5 - 4.810 - 0.154 = 7.386$ in.

ineffective portion of web = 9.192 − 1.434 − 7.386 = 0.372 in.

Effective section properties about x-axis:

Element	L (in.)	y Distance from Top Fiber (in.)	Ly (in.^{2})
b_1	1.434	0.871	1.249
$b_2 + (9.5 - y_{cg}) - 0.154$	7.386	5.653	41.753
Compression flange	1.471	0.030	0.044
Compression stiffener	0.600	0.257	0.154
Top 90° corner	0.195	0.075	0.015
Top 135° corner	0.097	0.042	0.004
Bottom 135° corner	0.097	9.458	0.917
Bottom 90° corner	0.195	9.425	1.838
Bottom stiffener	0.600	9.243	5.546
Tension flange	1.471	9.470	13.930
Sum	13.546		65.450

y_{cg} = 65.450/13.546 = 4.832 in. (measured from top compression fiber)

f_1 = [(4.832 − 0.154)/4.832](50) = 48.41 ksi
f_2 = −[(9.5 − 4.832 − 0.154)/4.832](50) = −46.71 ksi
ψ = −46.71/48.41 = −0.965
k = $4 + 2[1-(-0.965)]^3 + 2[1-(-0.965)] = 23.105$
λ = $(1.052/\sqrt{23.105})(153.20)\sqrt{48.41/29500} = 1.358 > 0.673$
ρ = [1 − (0.22/1.358)]/1.358 = 0.617
b_e = 0.617 × 9.192 = 5.671 in.
b_2 = 5.671/2 = 2.836 in.
b_1 = 5.671/[3 − (−0.965)]
= 1.430 in.
$b_1 + b_2$ = 1.430 + 2.836 = 4.266 in. ≠ 4.284 in.
therefore need to reiterate

For the third iteration:

$b_1 = 1.430$ in.
$b_2 + (9.5 - y_{cg}) - 0.154 = 2.836 + 9.5 - 4.832 - 0.154 = 7.350$ in.

ineffective portion of web $= 9.192 - 1.430 - 7.350 = 0.412$ in.

Effective section properties about x-axis:

$L = 13.506$ in.
$Ly = 65.373$ in.^{2}
$y_{cg} = 65.373/13.506$
$= 4.840$ in.
$f_1 = [(4.840 - 0.154)/4.840](50) = 48.41$ ksi
$f_2 = -[(9.5 - 4.840 - 0.154)/4.840](50) = -46.55$ ksi
$\psi = -0.962$
$k = 23.029$
$\lambda = (1.052/\sqrt{23.029})(153.20)\sqrt{48.41/29500} = 1.360 > 0.673$
$\rho = [1 - (0.22/1.360)]/1.360 = 0.616$
$b_e = 0.616 \times 9.192 = 5.662$ in.
$b_2 = 5.662/2 = 2.831$ in.
$b_1 = 5.662/[3 - (-0.962)] = 1.429$ in.
$b_1 + b_2 = 4.260$ in. $\neq 4.266$ in.
therefore need to reiterate.

For the fourth iteration:

$b_1 = 1.429$ in.
$b_2 + (9.5 - y_{cg}) - 0.154 = 2.831 + 9.5 - 4.840 - 0.154 = 7.337$ in.

ineffective portion of the web $= 9.192 - 1.429 - 7.337 = 0.426$ in.

Effective section properties about x-axis:

$L = 13.492$ in.
$Ly = 65.349$ in.^{2}
$y_{cg} = 65.349/13.492 = 4.844$ in.
$f_1 = [(4.844 - 0.154)/4.844](50) = 48.41$ ksi
$f_2 = -[(9.5 - 4.844 - 0.154)/4.844](50) = -46.47$ ksi
$\psi = -0.960$
$k = 22.979$
$\lambda = (1.052/\sqrt{22.979})(153.20)\sqrt{48.41/29500} = 1.362 > 0.673$
$\rho = [1 - (0.22/1.362)]/1.362 = 0.616$
$b_e = 0.616 \times 9.192 = 5.662$ in.
$b_2 = 5.662/2 = 2.831$ in.
$b_1 = 5.662/[3 - (-0.960)] = 1.430$ in.
$b_1 + b_2 = 4.261$ in. close enough to 4.260 in.
thus the solution stabilizes.

Hence we now compute the location of N.A. and moment of inertia using $b_1 = 1.430$ in. and $b_2 = 2.831$ in.

Effective section properties about x-axis:

Element	L (in.)	y Distance from Top Fiber (in.)	Ly (in.^{2})	Ly^2 (in.^{3})	I'_1 About Own Axis (in.^{3})
b_1	1.430	0.869	1.243	1.080	0.244
$b_2 + (9.5 - y_{cg}) - 0.154$	7.333	5.680	41.651	236.578	32.860
Compression flange	1.471	0.030	0.044	0.001	—
Compression stiffener	0.600	0.257	0.154	0.040	0.009
Top 90° corner	0.195	0.075	0.015	0.001	—
Top 135° corner	0.097	0.042	0.004	—	—
Bottom 135° corner	0.097	9.458	0.917	8.673	—
Bottom 90° corner	0.195	9.425	1.838	17.323	—
Bottom stiffener	0.600	9.243	5.546	51.262	0.009
Tension flange	1.471	9.470	13.930	131.917	—
Sum	13.489		65.342	446.875	33.122

Distance of x-axis from top fiber is $y_{cg} = 65.342/13.489 = 4.844$ in.

Since distance of top compression fiber from neutral axis is greater than one half the beam depth (= 4.750 in.), a compression stress of 50 ksi will govern as assumed.

$$\begin{aligned} I'_x &= Ly^2 + I'_1 - Ly^2_{cg} \\ &= 446.875 + 33.122 - 13.489(4.844)^2 \\ &= 163.487 \text{ in.}^3 \end{aligned}$$

$$\begin{aligned} \text{Actual } I_x &= I'_x t \\ &= 163.487 \times 0.060 \\ &= 9.809 \text{ in.}^4 \end{aligned}$$

Section modulus

$$\begin{aligned} S_e &= I_x/y_{cg} \\ &= 9.809/4.844 \\ &= 2.025 \text{ in.}^3 \end{aligned}$$

$$\begin{aligned} M_n &= S_e F_y \\ &= 2.025 \times 50 \\ &= 101.25 \text{ kip-in.} \end{aligned}$$ (*Eq.* C3.1.1-1)

$$\Omega_f = 1.67$$

$$\begin{aligned} M_a &= M_n/\Omega_f \\ &= 101.25/1.67 \\ &= 60.63 \text{ kip-in.} \end{aligned}$$ (*Eq.* C3.1-1)

2. Calculation of the effective moment of inertia based on procedure I for deflection determination at the allowable moment:

   The procedure is iterative: one assumes the actual compressive stress f under this allowable moment. Knowing f, one proceeds as usual to obtain S_e and checks to see if $(f \times S_e)$ equals M_a as it should. If not, reiterate until one obtains the desired level of accuracy [Section B2.1-(b)-(1)].

   (a) For the first iteration, assume a compressive stress of $f = 30$ ksi in the top fibers of the section and that the web is fully effective.

Compression flange:

$S \quad = 1.28\sqrt{29500/30} = 40.14$

$S/3 \quad = 13.38 < w/t = 24.52 < S = 40.14$

$I_a \quad = 399(0.060)^4[(24.52/40.14) - 0.33]^3$

$\quad = 0.000115 \text{ in.}^4$

$I_s/I_a = 0.000540/0.000115$

$\quad = 4.696$

$k = [4.82 - 5(0.451)](4.696)^{1/2} + 0.43 = 5.988 > 2.995$

$k \quad = 2.995$

$\lambda \quad = (1.052/\sqrt{2.995})(24.52)\sqrt{30/29500} = 0.475 < 0.673$

$b \quad =$ **1.471** in. (i.e. compression flange fully effective)

Compression (upper) stiffener:
f conservatively taken equal to 30 ksi as in the top compression fiber

$\lambda \quad = (1.052/\sqrt{0.43})(10.00)\sqrt{30/29500} = 0.512 < 0.673$

therefore $d_s' = 0.600$ in.

Since $I_s/I_a = 4.696 > 1.0$, it follows that $d_s = d_s' = 0.600$ in. (i.e. compression stiffener fully effective). Thus section is fully effective (since web was assumed fully effective)

$y_{cg} \quad = 9.5/2 = 4.750$ in. (from symmetry)

To check if the web is fully effective:

$f_1 \quad = [(4.750 - 0.154)/4.750](30) = 29.03$ ksi

$f_2 \quad = -29.03$ ksi

$\psi \quad = -29.03/29.03 = -1.000$

$k \quad = 24.000$

$\lambda \quad = (1.052/\sqrt{24})(153.20)\sqrt{29.03/29500} = 1.032 > 0.673$

$\rho \quad = [1 - (0.22/1.032)]/1.032 = 0.762$

$b_e \quad = 0.762 \times 9.192$

$\quad = 7.004$ in.

$b_2 \quad = 7.004/2 = 3.502$ in.

$b_1 \quad = 7.004/[3 - (-1.000)]$

$\quad = 1.751$ in.

compression portion of the web $= y_{cg} - 0.154 = 4.750 - 0.154 = 4.596$ in.

$b_1 + b_2 = 5.253 \text{ in.} > 4.596 \text{ in.}$

Thus $b_1 + b_2$ shall be taken as 4.596 in. This verifies the assumption that the web is fully effective.

Full section properties about x-axis:

Element	L (in.)	y Distance from Centerline of Section (in.)	Ly^2 (in.^{3})	I_1' About Own Axis (in.^{3})
Web	9.192	—	—	64.722
Stiffeners	$2 \times 0.600 = 1.200$	4.493	24.224	0.018
90° corners	$2 \times 0.195 = 0.390$	4.675	8.524	—
135° corners	$2 \times 0.097 = 0.194$	4.708	4.300	—
Flanges	$2 \times 1.471 = 2.942$	4.720	65.543	—
Sum			102.591	64.740

$I'_x = Ly^2 + I'_l$
$= 102.591 + 64.740 = 167.331$ in.^{3}
$I_x = 167.331(0.060) = 10.040$ in.^{4}
$S_e = I_x/y_{cg} = 10.040/4.750$
$= 2.114$ in.^{3}

$M = f \times S_e = 30 \times 2.114 = 63.42$ kip-in.
not equal to $M_a = 60.63$ kip-in.
thus need to reiterate.

However, one sees that we need to assume a smaller stress than 30 ksi and hence since the section was fully effective for $f = 30$ ksi, it will be fully effective for $f < 30$ ksi.

Thus $S_e = 2.114$ in.^{3}

Therefore, the correct actual f at $M_a = M_a/S_e = 60.63/2.114 = 28.68$ ksi. And $I_x =$ 10.040 in.^{4} using procedure I for deflection determination.

Remark:

It was clearly seen that in the calculation of M_a the assumption of the web being fully effective was not true. However, it would be interesting to see the percentage of error if one neglected the partial effectiveness of the web and proceeded with the assumption of a fully effective web.

To demonstrate: neglect in the first approximation in the calculation of M_a the partial effectiveness of the web.

Thus the whole section is fully effective. Full section properties about x-axis (from part 2):

$I_x = 10.040$ in.^{4}
$S_e = 2.114$ in.^{3}
$M_a = (2.114 \times 50)/1.67$
$= 63.29$ kip-in.

$$\% \text{ error} = \frac{63.29 - 60.63}{60.63} \times 100\%$$

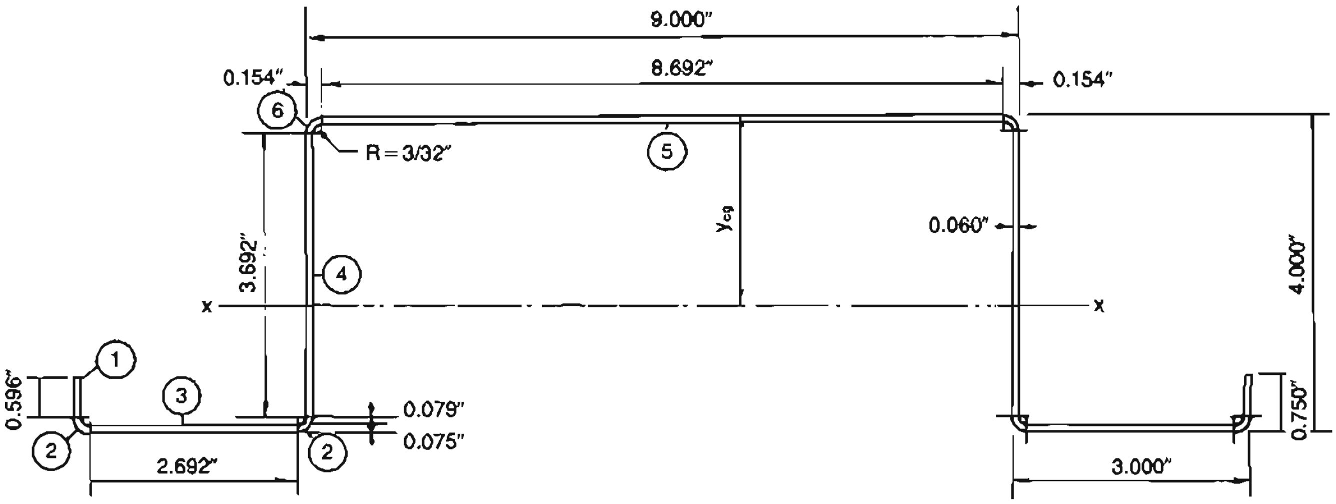
$$= 4.39\%$$

Since percentage of error is small, one could rationalize that in practical cases to get a first-hand quick answer one could assume web being fully effective.

EXAMPLE NO. 5

HAT SECTION

Complete Flexural Design,
Stiffened Compression Flange



Given:
1. Steel: $F_y = 50$ ksi.
2. Section: As shown in sketch.
3. Span: L = 8 ft., with simple supports, no overhang, and 6-in. support bearing lengths.
4. Loading: Live = 300 lb/ft.; Dead = 20 lb/ft.

Required: Check adequacy of section for:
1. Bending moment
2. Shear
3. Web Crippling
4. Deflection

Solution:

1. Properties of 90° Corners:

   Radius to centerline,
   $r = R + t/2 = 3/32 + 0.060/2 = 0.124$ in.
   Length of arc, $u = 1.57r = 1.57(0.124) = 0.195$ in.
   Distance of c.g. from center of radius, $c = 0.637r = 0.637(0.124) = 0.079$ in.
   I' of corner about its own centroidal axis is negligible
   Equations referenced from p. III-7 of Supplementary Information

2. Nominal Section Strength (Section C3.1.1)

   (a) Procedure I—Based on Initiation of Yielding
   Computation of I_x, first approximation:
   - Assume a compressive stress of $f = F_y = 50$ ksi in the top fibers of the section.
   - Also assume web is fully effective.

   Element ④:
   $h/t = 3.692/0.060 = 61.53 < (h/t)_{max} = 200$ *OK* [Section B1.2-(a)]

   Assumed fully effective

Element ⑤:

$w/t = 8.692/0.060 = 144.9 < 500$ *OK* [Section B1.1-(a)-(2)]

$k = 4$

$$\lambda = (1.052/\sqrt{k})\,(w/t)\,\sqrt{f/E} \qquad (Eq.\ B2.1\text{-}4)$$
$$= (1.052/\sqrt{4})\,(144.9)\,\sqrt{50/29500}$$
$$= 3.138$$

For $\lambda > 0.673$

$$\rho = (1 - 0.22/\lambda)/\lambda \qquad (Eq.\ B2.1\text{-}3)$$
$$= (1 - 0.22/3.138)/3.138$$
$$= 0.296$$

$$b = \rho w \qquad (Eq.\ B2.1\text{-}2)$$
$$= (0.296)\,(8.692)$$
$$= 2.573 \text{ in.}$$

Element	L Effective Length (in.)	y Distance from Top Fiber (in.)	Ly (in.^{2})	Ly^2 (in.^{3})	I'_1 About Own Axis (in.^{3})
1	2 × 0.596 = 1.192	3.548	4.229	15.005	0.035
2	4 × 0.195 = 0.780	3.925	3.062	12.016	—
3	2 × 2.692 = 5.384	3.970	21.375	84.857	—
4	2 × 3.692 = 7.384	2.000	14.768	29.536	8.388
5	2.573	0.030	0.077	0.002	—
6	2 × 0.195 = 0.390	0.075	0.029	0.002	—
Sum	17.703		43.540	141.418	8.423

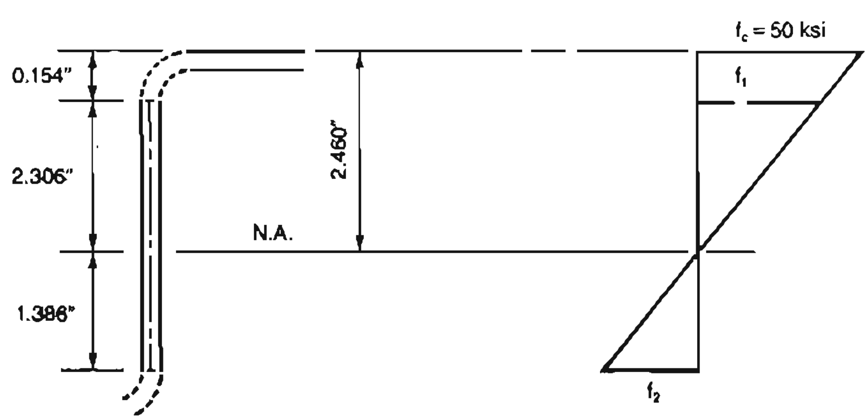
Distance of neutral axis from top fiber, $y_{cg} = Ly/L = 43.540/17.703 = 2.460$ in.

Since the distance of the top compression fiber from the neutral axis is greater than one half the beam depth, a compressive stress of F_y will govern as assumed.

$$I'_x = Ly^2 + I'_1 - Ly^2_{cg}$$
$$= 141.418 + 8.423 - 17.703(2.460)^2$$
$$= 42.71 \text{ in.}^3$$

$$\text{Actual } I_x = tI'_x$$
$$= (0.06)\,(42.71)$$
$$= 2.56 \text{ in.}^4$$

Check Web



$f_1 = (2.306/2.460)(50) = 46.87$ ksi (compression)
$f_2 = (1.386/2.460)(50) = 28.17$ ksi (tension)
$\psi = (f_2/f_1) = -28.17/46.87 = -0.601$

$k = 4 + 2(1-\psi)^3 + 2(1-\psi)$
$= 4 + 2[1-(-0.601)]^3 + 2[1-(-0.601)]$
$= 15.41$

$\lambda = (1.052/\sqrt{k})(w/t)\sqrt{f/E}, \quad f = f_1$ (*Eq.* B2.1-4)
$= (1.052/\sqrt{15.41})(61.53)\sqrt{46.87/29500}$
$= 0.657$

For $\lambda < 0.673$
$b = w$ (*Eq.* B2.1-1)
$b_e = 3.692$ in.
$b_2 = b_e/2 = 3.692/2 = 1.846$ in. (*Eq.* B2.3-1)
$b_1 = b_e/(3-\psi)$ (*Eq.* B2.3-1)
$= 3.692/[3-(-0.601)]$
$= 1.025$ in.
$b_1 + b_2 = 1.025 + 1.846$
$= 2.871 > 2.306$ (compression portion of web)

Therefore, web is fully effective.

$S_e = I_x/y_{cg} = 2.56/2.46 = 1.04 \text{ in.}^3$

$M_n = S_eF_y$ (*Eq.* C3.1.1-1)
$= (1.04)(50)$
$= 52.0$ kip-in.

(b) Procedure II—Based on Inelastic Reserve Capacity

$\lambda_1 = (1.11/\sqrt{F_y/E}) = (1.11/\sqrt{50/29500}) = 26.96$

$\lambda_2 = (1.28/\sqrt{F_y/E}) = (1.28/\sqrt{50/29500}) = 31.09$

$w/t = 8.692/0.06 = 144.9$

For $w/t > \lambda_2$, $C_y = 1$

Maximum compressive strain $= C_ye_y = e_y$

Therefore, the nominal ultimate moment, M_n, is the same as the M_n determined by procedure I because the compression flange will yield first.

3. Allowable Bending Moment (Section C3.1)

$M_a = M_n/\Omega_f$ (*Eq.* C3.1-1)
$= 52.0/1.67$
$= 31.1$ kip-in.

Maximum applied moment $= wL^2/8 = 0.32(8)^2/8 = 2.56 = 30.72$ kip-in.

$M = 30.72 < M_a = 31.1$ kip-in. *OK*

4. Strength for Shear Only (Section C3.2)

The shear force at any section shall not exceed the allowable shear V_a:

$$1.38\sqrt{Ek_v/F_y} = 1.38\sqrt{\frac{29500(5.34)}{50}} = 77.46$$

$h/t = 61.53$

For $h/t < 1.38\sqrt{Ek_v/F_y}$ (*Eq.* C3.2-1)

$V_a = 0.38t^2\sqrt{k_vF_yE}$
$= 0.38(0.06)^2\sqrt{5.34(50)(29500)}$
$= 3.84$ kips

$V_a = (11800F_y/E)ht$
$= 11800(50/29500)(3.692)(0.06)$
$= 4.43$ kips
$V_a = 3.84$ k controls (per web)

Total V_a for Section:
$V_a = 2(3.84)$
$= 7.68$ kips

Maximum Shear Force = Reaction
$V = wL/2$
$= 0.32(8)/2$
$= 1.28\text{ k} < V_a = 7.68\text{ k}$ *OK*

5. Web Crippling Strength (Section C3.4)

$R/t = [(3/32)/0.06] = 1.563 < 6$ *OK*
$h/t = (3.692/0.06) = 61.53 < 200$

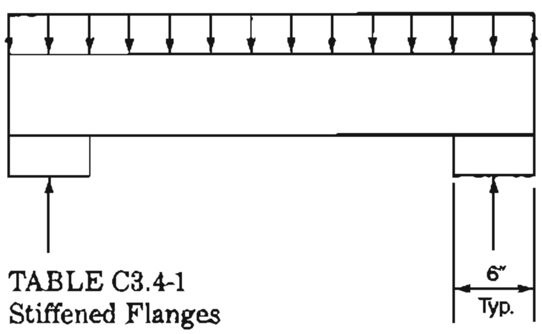


TABLE C3.4-1
Stiffened Flanges

$P_a = t^2kC_3C_4C_6[179 - 0.33(h/t)][1 + 0.01(N/t)]$ (*Eq*. C3.4-1)

$k = F_y/33 = 50/33 = 1.515$ (*Eq*. C3.4-22)

$C_3 = (1.33 - 0.33k)$ (*Eq*. C3.4-12)
$= [1.33 - 0.33(1.515)]$
$= 0.830$

$C_4 = (1.15 - 0.15\,R/t) \leq 1.0$ but not less than 0.50 (*Eq*. C3.4-13)

$(1.15 - 0.15\,R/t) = [1.15 - 0.15(1.563)]$
$= 0.916 \leq 1.0$ *OK*
> 0.50 *OK*

$C_4 = 0.916$

$C_6 = 0.7 + 0.3(\theta/90)^2$ (*Eq*. C3.4-20)
$= 0.7 + 0.3(90/90)^2$
$= 1.0$

$P_a = (0.06)^2(1.515)(0.830)(0.916)(1.0)[179 - 0.33(61.53)]$
$\times [1 + 0.01(6/0.06))$
$= 1.32$ k/web

$P_a = (2\text{ webs})(1.32\text{ k/web})$
$= 2.64$ k

Reaction $= 1.28\text{ k} < P_a = 2.64\text{ k}$ *OK*

6. Deflection Determination at Allowable Moment

Find I_{eff} at $M_a = 31.3$ kip-in.

(a) Procedure I

Computation of I_{eff}, first approximation

- Assume a compressive stress of $f = 0.6F_y = 30$ ksi in the top fiber of the section
- Also assume web is fully effective

Element ⑤:

$\lambda = (1.052/\sqrt{4})(144.9)\sqrt{30/29500}$ (*Eq.* B2.1-4)

$= 2.431$

For $\lambda > 0.673$

$\rho = (1-0.22/2.431)/2.431$ (*Eq.* B2.1-3)

$= 0.374$

$b_d = \rho w$ (*Eq.* B2.1-6)

$= 0.374(8.692)$

$= 3.251$ in.

Element	L Effective Length (in.)	y Distance from Top Fiber (in.)	Ly (in.²)	Ly² (in.³)	I_1' About Own Axis (in.³)
1	2 × 0.596 = 1.192	3.548	4.229	15.005	0.035
2	4 × 0.195 = 0.780	3.925	3.062	12.016	—
3	2 × 2.692 = 5.384	3.970	21.375	84.857	—
4	2 × 3.692 = 7.384	2.000	14.768	29.536	8.388
5	3.251	0.030	0.098	0.003	—
6	2 × 0.195 = 0.390	0.075	0.029	0.002	—
Sum	18.381		43.561	141.419	8.423

Distance of neutral axis from top fiber, $y_{cg} = Ly/L = (43.561/18.381) = 2.370$ in.

$I'_{eff} = Ly^2 + I_1' - Ly_{cg}^2$

$= 141.419 + 8.423 - 18.381(2.370)^2$

$= 46.60$ in.³

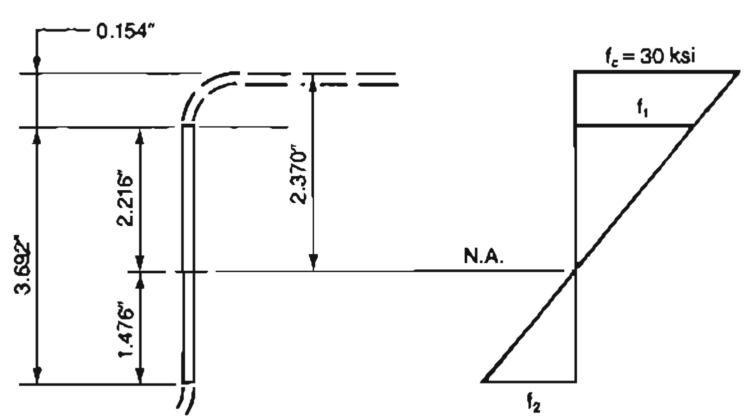
Actual $I_{eff} = tI'_{eff}$

$= (0.060)(46.60)$

$= 2.80$ in.⁴

Check Web

- Should be fully effective



$f_1 = (2.216/2.370)(30) = 28.05$ ksi (compression)

$f_2 = (1.476/2.370)(30) = 18.68$ ksi (tension)

$\psi = f_2/f_1 = -18.68/28.05 = -0.665$

$k = 4 + 2(1-\psi)^3 + 2(1-\psi)$
$= 4 + 2[1-(-0.665)]^3 + 2[1-(-0.665)]$
$= 16.56$

$\lambda = (1.052/\sqrt{k})(w/t)\sqrt{f/E}, \quad f = f_1$ (*Eq.* B2.1-4)
$= (1.052/\sqrt{16.56})(61.53)\sqrt{28.05/29500}$
$= 0.491$

For $\lambda < 0.673$

$b = w$ (*Eq.* B2.1-1)

$b_e = 3.692$ in.

$b_2 = b_e/2 = 3.692/2 = 1.846$ in. (*Eq.* B2.3-2)

$b_1 = b_e/(3-\psi)$ (*Eq.* B2.3-1)
$= 3.692/[3-(-0.665)]$
$= 1.007$

$b_1 + b_2 = 1.007 + 1.846$
$= 2.853 > 2.216$ (compressive portion of web)

Therefore, web is fully effective.

$S_{eff} = I_{eff}/y_{cg} = 2.80/2.370 = 1.18$ in.^{3}

$M_{f=0.6F_y} = S_{eff}(0.6F_y)$
$= (1.18)(30)$
$= 35.4$ kip-in.

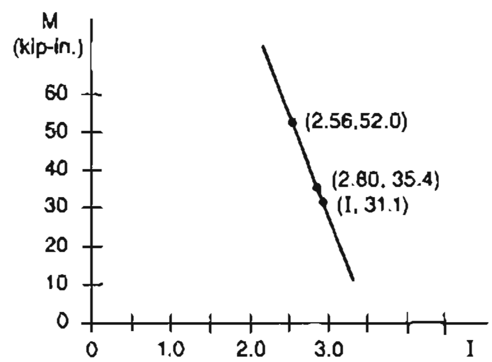
To determine I_{eff} at $M = 31.1$, extrapolate using

① $M = 52.0$, $I = 2.56$ in.^{4}

② $M = 35.4$, $I = 2.80$ in.^{4}

③ $M = 31.1$, $I = ?$

$$(31.1-35.4)/(I-2.80) = (35.4-52.0)/(2.80-2.56)$$
$$-4.30 = -69.17(I-2.80)$$
$$0.0622 = I - 2.80$$
$$I = 2.86 \text{ in.}^4$$



Use $I = 2.86$ in.^{4} in deflection calculations

Deflection $= 5wL^4/384EI$

(b) Procedure II

$\lambda_c = 0.256 + 0.328(w/t)\sqrt{F_y/E}$ (*Eq.* B2.1-10)
$= 0.256 + 0.328(144.9)\sqrt{50/29500}$
$= 2.213$

Computation of I_{eff}; Check case of $f = F_y$

$\lambda = 3.138$ (from pg. IV-31)

For $\lambda > \lambda_c$

$\rho = (0.41 + 0.59\sqrt{F_y/f} - 0.22/\lambda)/\lambda$ (*Eq.* B2.1-9)

$\rho = (1 - 0.22/\lambda)/\lambda$

$\rho = 0.296$, which is the same value for Load Capacity Determination

Computation of I_{eff}; Assume a compressive stress $f = 0.6F_y = 30$ ksi in top fiber of section

Note: Web is fully effective

$\lambda = 2.431$ (from pg. IV-34)

For $\lambda > \lambda_c$

$$\rho = [0.41 + 0.59\sqrt{F_y/f} - (0.22/\lambda)]/\lambda \qquad (Eq.\ B2.1\text{-}9)$$
$$= [0.41 + 0.59\sqrt{50/30} - (0.22/2.431)]/2.431$$
$$= 0.445$$

For $\lambda > 0.673$

$$b_d = \rho w \qquad (Eq.\ B2.1\text{-}6)$$
$$= 0.445(8.692)$$
$$= 3.868 \text{ in.}$$

Element	L Effective Length (in.)	y Distance from Top Fiber (in.)	Ly (in.^{2})	Ly^2 (in.^{3})	I_1' About Own Axis (in.^{3})
1	2 × 0.596 = 1.192	3.548	4.229	15.005	0.035
2	4 × 0.195 = 0.780	3.925	3.062	12.016	—
3	2 × 2.692 = 5.384	3.970	21.375	84.857	—
4	2 × 3.692 = 7.384	2.000	14.768	29.536	8.388
5	3.868	0.030	0.116	0.004	—
6	2 × 0.195 = 0.390	0.075	0.029	0.002	—
Sum	18.998		43.579	141.420	8.423

Distance of neutral axis from top fiber, $y_{cg} = Ly/L = 43.579/18.998 = 2.294$ in.

$$I'_{eff} = Ly^2 + I_1' - Ly_{cg}^2$$
$$= 141.420 + 8.423 - (18.998)(2.294)^2$$
$$= 49.87 \text{ in.}^3$$

$$\text{Actual } I_{eff} = tI'_{eff}$$
$$= (0.06)(49.87)$$
$$= 2.99 \text{ in.}^4$$

$$S_{eff} = I_{eff}/y_{cg} = 2.99/2.294 = 1.30 \text{ in.}^3$$

$$M_{f=0.6F_y} = S_{eff}(0.6F_y)$$
$$= (1.30)(30)$$
$$= 39.0 \text{ kip-in.}$$

To determine I_{eff} at $M = 31.1$, extrapolate using

① $M = 52.0$, $I = 2.56$ in.^{4}

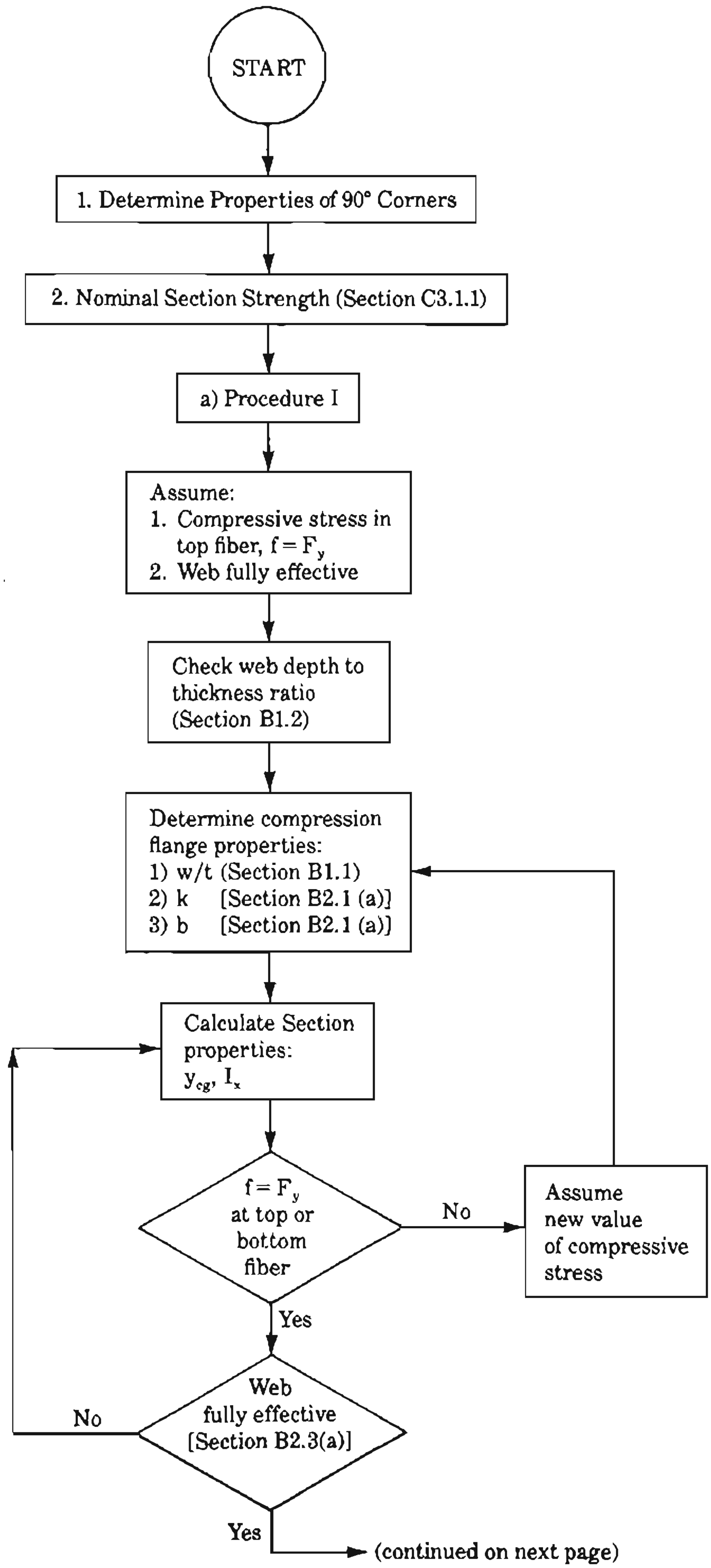
② $M = 39.0$, $I = 2.99$ in.^{4}

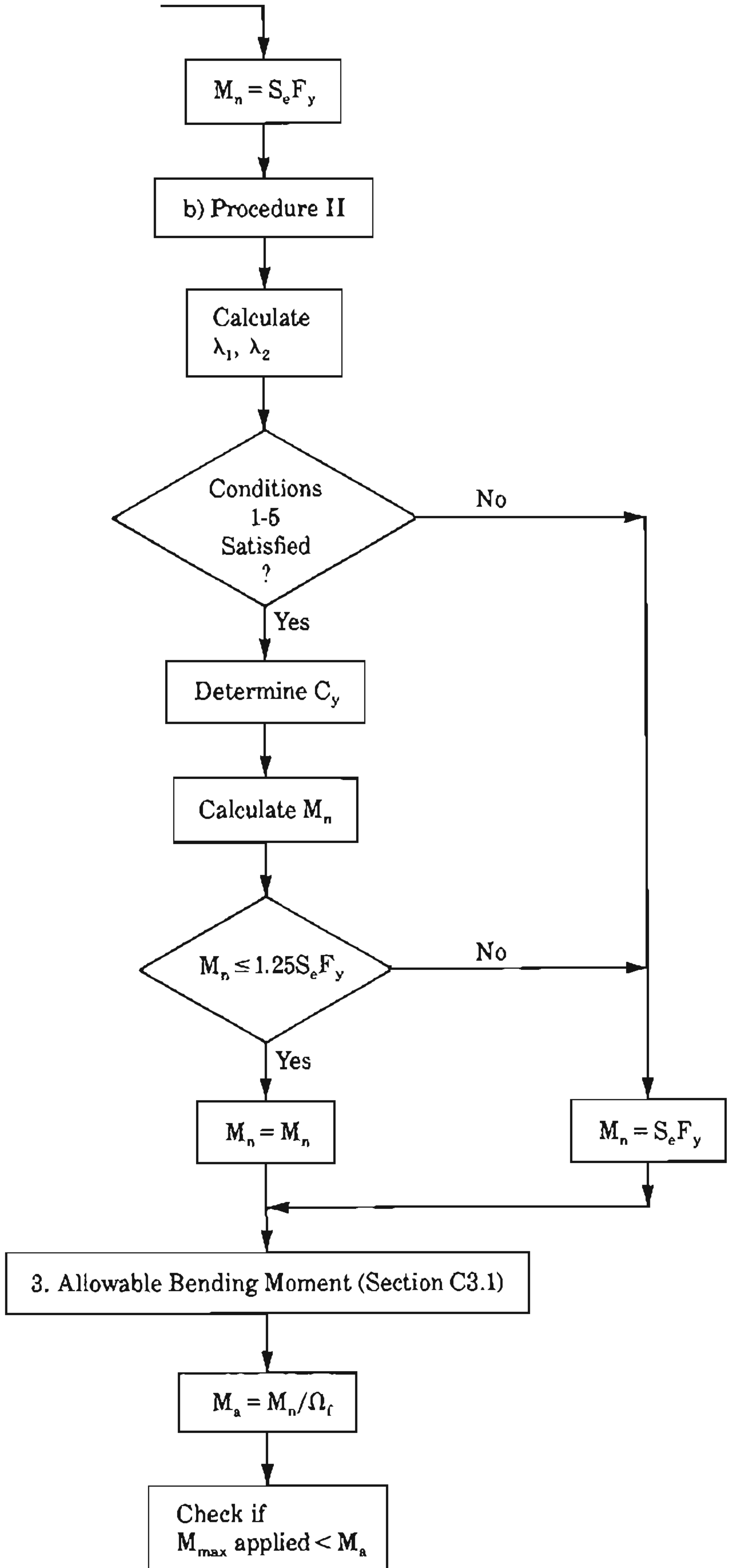
③ $M = 31.1$, $I = ?$

$$(31.1 - 39.0)/(I - 2.99) = (39.0 - 52.0)/(2.99 - 2.56)$$
$$-7.9 = -30.23(I - 2.99)$$
$$0.26 = I - 2.99$$
$$I = 3.25 \text{ in.}^4$$

Use $I = 3.25$ in.^{4} in deflection calculations

Deflection $= 5wL^4/384EI$


START
1. Determine Properties of 90° Corners
2. Nominal Section Strength (Section C3.1.1)
a) Procedure I
Assume:
1. Compressive stress in top fiber, $f = F_y$
2. Web fully effective
Check web depth to thickness ratio (Section B1.2)
Determine compression flange properties:
1) w/t (Section B1.1)
2) k [Section B2.1 (a)]
3) b [Section B2.1 (a)]
Calculate Section properties: y_{cg}, I_x
$f = F_y$ at top or bottom fiber
No
Assume new value of compressive stress
Yes
Web fully effective [Section B2.3(a)]
No
Yes
(continued on next page)



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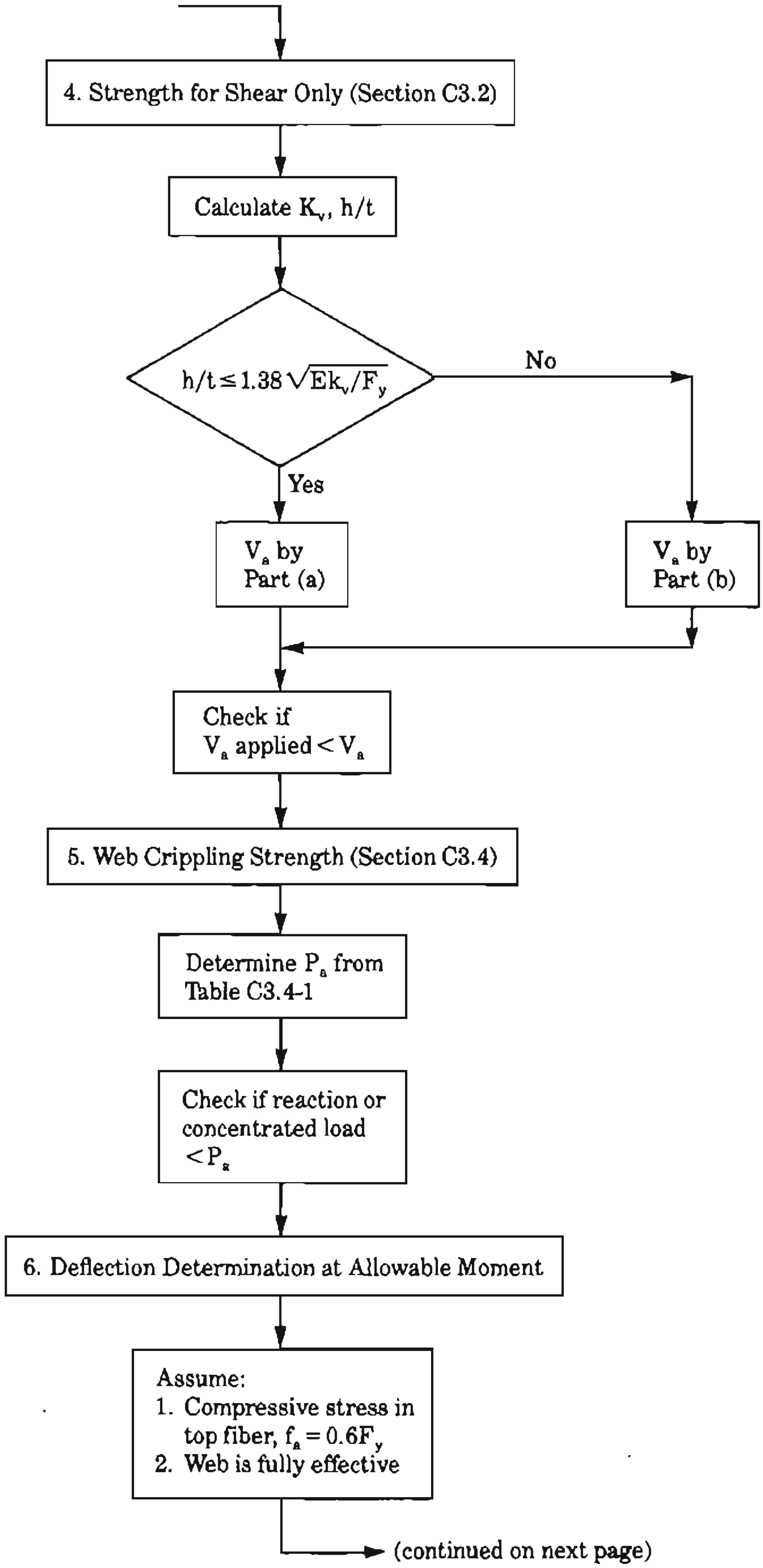

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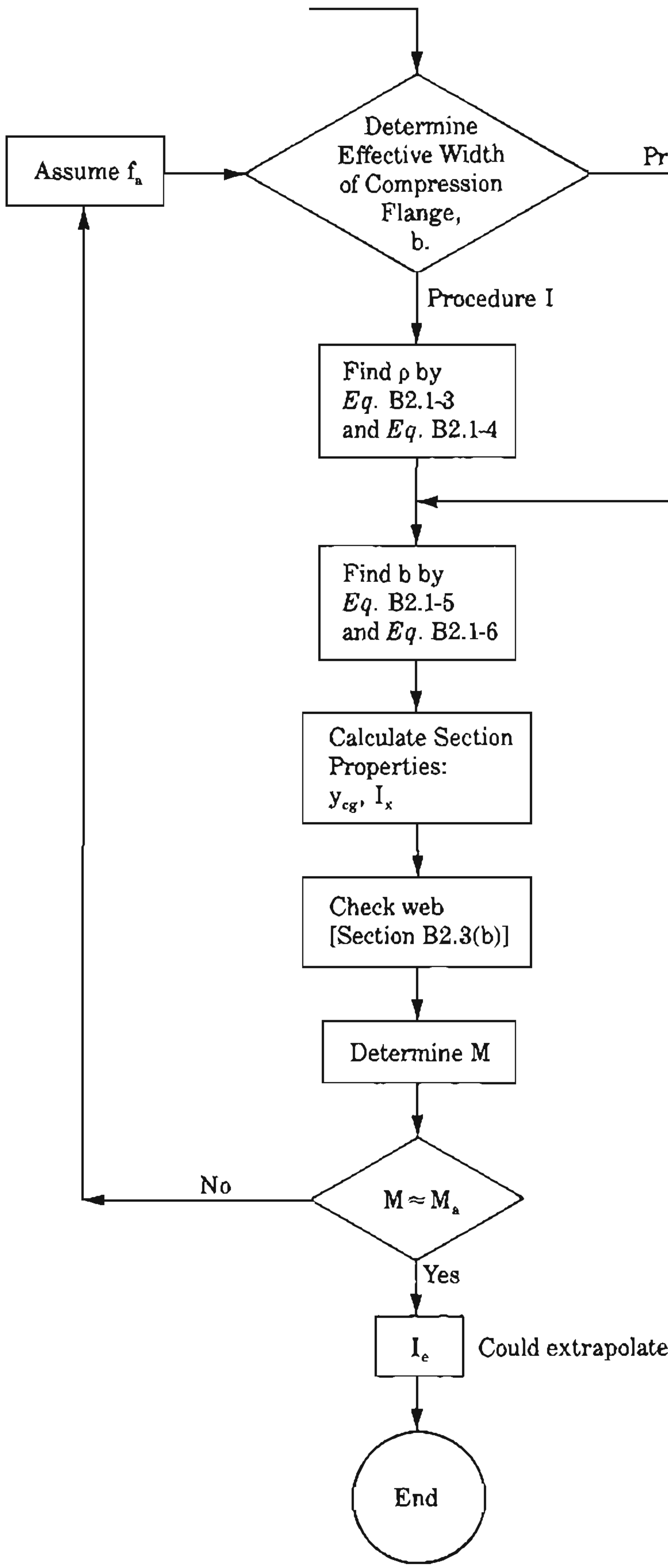
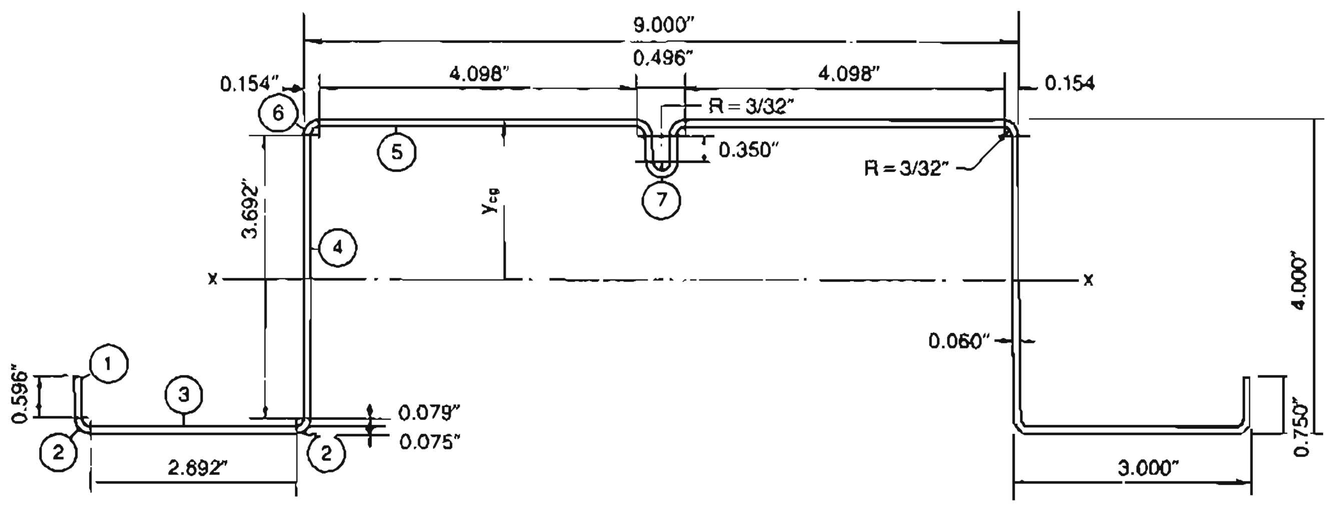

Assume f_a
Determine Effective Width of Compression Flange, b.
Procedure II
Procedure I
Find ρ by Eq. B2.1-3 and Eq. B2.1-4
Find ρ by Eq. B2.1-7 through Eq. B2.1-10
Find b by Eq. B2.1-5 and Eq. B2.1-6
Calculate Section Properties: y_{cg}, I_x
Check web [Section B2.3(b)]
Determine M
No
$M \approx M_a$
Yes
I_e
Could extrapolate
End

EXAMPLE NO. 6

HAT SECTION

Flexural Design, Compression Flange with Intermediate Stiffener



Given: 1. Steel: $F_y = 50$ ksi.
2. Section: As shown in the sketch.

Required: For the section, determine the flexural strength and the moment of inertia for deflection calculation.

Compare structural economy of this section with an almost identical section without an intermediate stiffener computed in previous example.

Solution:

1. Properties of 90° Corners:
   Radius to centerline,
   $r = R + t/2 = 3/32 + 0.060/2 = 0.124$ in.
   Length of arc, $u = 1.57r = 1.57(0.124) = 0.195$ in.
   Distance of c.g. from center of radius, $c = 0.637r = 0.637(0.124) = 0.079$ in.
   I' of corner about its own centroidal axis is negligible
   Equations referenced from p. III-7 of Supplementary Information

2. Nominal Section Strength (Section C3.1.1)

   (a) Procedure I—Based on Initiation of Yielding
   Computation of I_x, first approximation:
   - Assume a compressive stress of $f = F_y = 50$ ksi in the top fibers of the section.
   - Also assume web is fully effective.

   Element 4:
   $h/t = 3.692/0.060 = 61.53 < 200$ *OK* [Section B1.2-(a)]

   Assumed fully effective

   Element 5:
   $$S = 1.28\sqrt{E/f} \qquad (Eq.\ B4\text{-}1)$$
   $$= 1.28\sqrt{29500/50}$$
   $$= 31.09$$

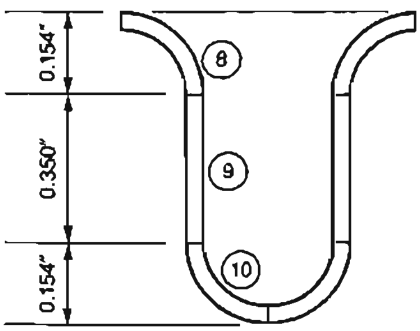
$b_o/t = 8.692/0.060 = 144.9 < 500$ *OK* [Section B1.1-(a)-(2)]

$3S = 3(31.09) = 93.27$

For $b_o/t > 3S$ (Case III)

$$I_a = t^4\{[128(b_o/t)/S] - 285\} \qquad (Eq.\ B4.1\text{-}9)$$
$$= (0.06)^4\{[128(144.9)/31.09] - 285\}$$
$$= 0.004038\ \text{in.}^4$$

Determine full section properties of stiffener ⑦:



All inner radii = 3/32

$r = R + 3/32 = 3/32 + 0.060 = 0.124$ in.

$u = 1.57r = 1.57(0.124) = 0.195$ in.

$c = 0.637r = 0.637(0.124) = 0.079$ in.

Element	L Length (in.)	y Distance from Top Fiber (in.)	Ly (in.^{2})	Ly^2 (in.^{3})	I_1' About Own Axis (in.^{3})
8	$2\times0.195 = 0.390$	0.075	0.0293	0.0022	—
9	$2\times0.350 = 0.700$	0.329	0.2303	0.0758	0.0071
10	$2\times0.195 = 0.390$	0.583	0.2274	0.1326	—
Sum	1.480		0.4870	0.2106	0.0071

Distance of neutral axis from top fiber, $y_{cg} = (Ly/L) = (0.4870/1.480) = 0.329$ in.

Total area of section, $Lt = (1.480)(0.060) = 0.0888$ in.^{2}

$$I_s' = Ly^2 + I_1' - Ly_{cg}^2$$
$$= 0.2106 + 0.0071 - 1.480(0.329)^2$$
$$= 0.0575\ \text{in.}^3$$

$$\text{Actual } I_s = tI_s'$$
$$= (0.060)(0.0575^3)$$
$$= 0.00345\ \text{in.}^4$$

Reduced Area of Stiffener

Element ⑨:

Stiffened element, $k = 4$

$f = F_y = 50$ ksi

$w/t = 0.350/0.060 = 5.83 < 500$ *OK* [Section B1.1-(a)-(2)]

$$\lambda = (1.052/\sqrt{k})(w/t)\sqrt{f/E} \qquad (Eq.\ B2.1\text{-}4)$$
$$= (1.052/\sqrt{4})(5.83)\sqrt{50/29500}$$
$$= 0.126$$

For $\lambda < 0.673$

$b = w \qquad (Eq.\ B2.1\text{-}1)$

$b = 0.350$ in. (fully effective)

$A'_s = Lt$
$A'_s = 0.0888 \text{ in.}^2$

$A_s = A'_s(I_s/I_a) \le A'_s$ (*Eq.* B4.1-11)
$= 0.0888(0.00345/0.004038)$
$= 0.0888(0.8544)$
$= 0.0759 < A'_s$ *OK*
$L_s = (A_s/t) = (0.0759/0.060) = 1.265 \text{ in.}$

Continuing with element ⑤:
$k = 3(I_s/I_a)^{1/3} + 1 \le 4$ (*Eq.* B4.1-10)
$= 3(0.8544)^{1/3} + 1$
$= 3.847 < 4$ *OK*

$w/t = (4.098/0.060) = 68.30$

$\lambda = (1.052/\sqrt{k})\,(w/t)\,\sqrt{f/E}$ (*Eq.* B2.1-4)
$= (1.052/\sqrt{3.847})\,(68.30)\,\sqrt{50/29500}$
$= 1.508$

For $\lambda > 0.673$
$\rho = (1 - 0.22/\lambda)/\lambda$ (*Eq.* B2.1-3)
$= (1 - 0.22/1.508)/1.508$
$= 0.566$

$b = \rho w$ (*Eq.* B2.1-2)
$= 0.566(4.098)$
$= 2.320 \text{ in.}$

Element	L Effective Length (in.)	y Distance from Top Fiber (in.)	Ly (in.^{2})	Ly^2 (in.^{3})	I'_1 About Own Axis (in.^{3})
1	2 × 0.596 = 1.192	3.548	4.229	15.005	0.035
2	4 × 0.195 = 0.780	3.925	3.062	12.016	—
3	2 × 2.692 = 5.384	3.970	21.375	84.857	—
4	2 × 3.692 = 7.384	2.000	14.768	29.536	8.388
5	2 × 2.320 = 4.640	0.030	0.139	0.004	—
6	2 × 0.195 = 0.390	0.075	0.029	0.002	—
7	Stiffener 1.265	0.329	0.416	0.137	0.058
Sum	21.035		44.018	141.557	8.481

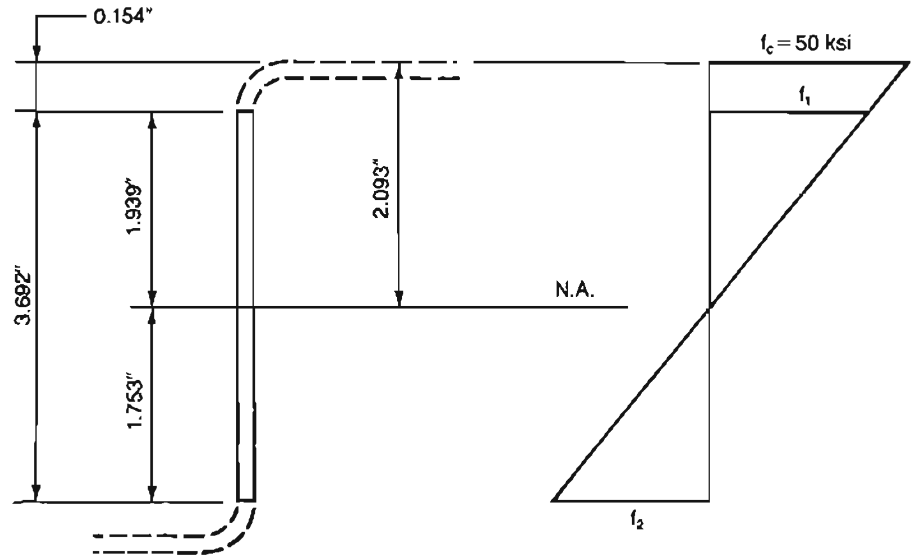
Distance of neutral axis from top fiber, $y_{cg} = (Ly/L) = (44.018/21.035) = 2.093$ in.

Since the distance of the top compression fiber from the neutral axis is greater than one half the beam depth, a compressive stress of F_y will govern as assumed.

$I'_x = Ly^2 + I'_1 - Ly_{cg}^2$
$= 141.557 + 8.481 - 21.035(2.093)^2$
$= 57.89 \text{ in.}^3$

Actual $I_x = tI'_x$
$= (0.060)\,(57.89)$
$= 3.47 \text{ in.}^4$

Check Web



$f_1 = (1.939/2.093)(50) = 46.32$ ksi (compression)
$f_2 = (1.753/2.093)(50) = 41.88$ ksi (tension)
$\psi = (f_2/f_1) = (-41.88/46.32) = -0.904$

$k = 4 + 2(1-\psi)^3 + 2(1-\psi)$
$= 4 + 2[1-(-0.904)]^3 + 2[1-(-0.904)]$
$= 21.61$

$\lambda = (1.052/\sqrt{k})(w/t)\sqrt{f/E}$, $\quad f = f_1$ (*q*. B2.1-4)
$= (1.052/\sqrt{21.61})(61.53)\sqrt{46.32/29500}$
$= 0.552$

For $\lambda < 0.673$
$b = w$ (*Eq*. B2.1-1)
$b_e = 3.692$ in.
$b_2 = b_e/2 = 3.692/2 = 1.846$ in. (*Eq*. B2.3-1)
$b_1 = b_e/(3-\psi)$ (*Eq*. B2.3-2)
$= 3.692/[3-(-0.904)]$
$= 0.946$
$b_1 + b_2 = 0.946 + 1.846$
$= 2.792$ in. > 1.939 in. (compressive portion of web)

Therefore, web is fully effective.

$S_e = (I_x/y_{cg}) = (3.47/2.093) = 1.66$ in.^{3}

$M_n = S_eF_y$
$= (1.66)(50)$
$= 83.0$ kip-in.

(b) Procedure II—Based on Inelastic Reserve Capacity

For multiple-stiffened compression elements

$C_y = 1$

Maximum compressive strain $= C_ye_y = e_y$

Therefore, the nominal ultimate moment, M_n, is the same as the M_n determined by Procedure I because the compression flange will yield first.

3. Allowable Bending Moment (Section C3.1)

$M_a = M_n/\Omega_f$ (*Eq.* C3.1-1)
$= 83.0/1.67$
$= 49.7$ kip-in.

4. For Complete Design, one should also check:
   a. Shear Strength
   b. Combined Bending and Shear, if applicable
   c. Web Crippling Strength

5. Deflection Determination at Allowable Moment

Find I_{eff} at $M_a = 49.7$ kip-in.

Note: Procedure II of Section B2.1-(b)-(2) does not apply.

Computation of I_{eff}; first approximation

- Assume a compressive stress of $f = 0.6F_y = 30$ ksi in top fiber of section
- Web is fully effective, because it was fully effective at a higher stress gradient.
- Element 9 of the stiffener, which was fully effective at $f = 50$ ksi will also be fully effective at $f = 30$ ksi.

Element 5:

$S = 1.28\sqrt{E/f}, \quad f = 30$ (*Eq.* B4-1)
$= 1.28\sqrt{29500/30}$
$= 40.14$

$b_o/t = 144.9$
$3S = 3(40.14) = 120.42$

For $b_o/t > 3S$ (Case III)

$I_a = t^4\{128(b_o/t)/S] - 285\}$ (*Eq.* B4.1-9)
$= (0.06)^4\{[128(144.9)/40.14] - 285\}$
$= 0.002295$ in.^{4}

$I_s = 0.00345$ in.^{4} (from pg. IV-42)

$k = 3(I_s/I_a)^{1/3} + 1 \leq 4$ (*Eq.* B4.1-10)

$3(I_s/I_a)^{1/3} + 1 = 3(0.00345/0.002295)^{1/3} + 1 = 4.437 > 4$

$k = 4$
$w/t = 68.30$

$\lambda = (1.052/\sqrt{k})(w/t)\sqrt{f/E}, \quad f = 30$ ksi (*Eq.* B2.1-4)
$= (1.052/\sqrt{4})(68.30)\sqrt{30/29500}$
$= 1.146$

For $\lambda > 0.673$

$\rho = (1 - 0.22/\lambda)/\lambda$ (*Eq.* B2.1-3)
$= (1 - 0.22/1.146)/1.146$
$= 0.705$

$b = \rho w$ (*Eq.* B2.1-2)
$= 0.705(4.098)$
$= 2.889$ in.

Stiffener, Element 7:

$A_s = A_s'(I_s/I_a) \leq A_s'$ (*Eq.* B4.1-11)
$= (0.0888 \text{ in.}^2)(0.00345/0.002295)$
$= 0.134 > A_s'$

$A_s = A_s'$
$= 0.0888$ in.^{2}
$L = A_s/t = 0.0888/0.060 = 1.480$ in.

Element	L Effective Length (in.)	y Distance from Top Fiber (in.)	Ly (in.^{2})	Ly^2 (in.^{3})	I'_1 About Own Axis (in.^{3})
1	$2 \times 0.596 = 1.192$	3.548	4.229	15.005	0.035
2	$4 \times 0.195 = 0.780$	3.925	3.062	12.016	—
3	$2 \times 2.692 = 5.384$	3.970	21.375	84.857	—
4	$2 \times 3.692 = 7.384$	2.000	14.768	29.536	8.388
5	$2 \times 2.889 = 5.778$	0.030	0.173	0.005	—
6	$2 \times 0.195 = 0.390$	0.075	0.029	0.002	—
7	Stiffener 1.480	0.329	0.487	0.160	0.058
Sum	22.388		44.123	141.581	8.481

Distance of neutral axis from top fiber, $y_{cg} = (Ly/L) = (44.123/22.388) = 1.971$ in.

$$
\begin{aligned}
I'_{eff} &= Ly^2 + I'_1 - Ly^2_{cg} \\
&= 141.581 + 8.481 - 22.388(1.971)^2 \\
&= 63.09 \text{ in.}^3
\end{aligned}
$$

$$
\begin{aligned}
\text{Actual } I_{eff} &= tI'_{eff} \\
&= (0.060)(63.09^3) \\
&= 3.79 \text{ in.}^4
\end{aligned}
$$

$$S_{eff} = I_{eff}/4 - y_{cg} = 3.79/(4 - 1.971) = 1.87 \text{ in.}^3$$

$$
\begin{aligned}
M_f &= 0.6F_y = S_{eff}(0.6F_y) \\
&= (1.87)(30) \\
&= 56.1 > M_a = 49.7 \text{ kip-in.}
\end{aligned}
$$

Computation of I_{eff}; second approximation
Extrapolate to obtain the stress value

① $M = 83.0, f = F_y = 50$

② $M = 56.1, f = 0.6F_y = 30$

③ $M = 49.7, f = ?$

$$
\begin{aligned}
f - 30/30 - 50 &= 49.7 - 56.1/56.1 - 83.0 \\
f &= 25.24 \text{ ksi}
\end{aligned}
$$

- Compressive stress of $f = 25.24$ in the top fiber of section
- Web is fully effective
- Element ⑨ of stiffener is fully effective

Element ⑤:

$$
\begin{aligned}
S &= 1.28\sqrt{E/f}, \quad f = 25.24 \text{ ksi} && (Eq.\ B4\text{-}1) \\
&= 1.28\sqrt{29500/25.24} \\
&= 43.76
\end{aligned}
$$

$b_o/t = 144.9$

$3S = 3(43.76) = 131.28$

For $b_o/t > 3S$ (Case III)

$$
\begin{aligned}
I_a &= t^4\{[128(b_o/t)/S] - 285\} && (Eq.\ B4.1\text{-}9) \\
&= 0.00180 \text{ in.}^4
\end{aligned}
$$

$I_s = 0.00345$ in.^{4} (from pg. IV-42)

$k = 3(I_s/I_a)^{1/3} + 1 \le 4$ *(Eq. B4.1-10)*

Since $I_s/I_a > 1$, $k = 4$

$w/t = 68.30$

$\lambda = (1.052/\sqrt{k})(w/t)\sqrt{f/E}, \quad f = 25.24$ ksi

$= (1.052/\sqrt{4})\,(68.30)\,\sqrt{25.24/29500}$
$= 1.051$

For $\lambda > 0.673$

$\rho = (1 - 0.22/\lambda)/\lambda$ (*Eq.* B2.1-3)
$= (1 - 0.22/1.051)/1.051$
$= 0.752$

$b = \rho w$ (*Eq.* B2.1-2)
$= 0.752(4.098)$
$= 3.082$ in.

Stiffener, Element ⑦:

$A_s = A_s'(I_s/I_a) \le A'_s$ (*Eq.* B4.1-11)

Since $I_s/I_a > 1$

$A_s = A_s'$
$= 0.0888$ in.^{2}

$L_s = A_s/t = 0.0888/0.060 = 1.480$ in.

Element	L Effective Length (in.)	y Distance from Top Fiber (in.)	Ly (in.^{2})	Ly^2 (in.^{3})	I_1' About Own Axis (in.^{3})
1	$2 \times 0.596 = 1.192$	3.548	4.229	15.005	0.035
2	$4 \times 0.195 = 0.780$	3.925	3.062	12.016	—
3	$2 \times 2.692 = 5.384$	3.970	21.375	84.857	—
4	$2 \times 3.692 = 7.384$	2.000	14.768	29.536	8.388
5	$2 \times 3.082 = 6.164$	0.030	0.185	0.006	—
6	$2 \times 0.195 = 0.390$	0.075	0.029	0.002	—
7	Stiffener 1.480	0.329	0.487	0.160	0.058
Sum	22.774		44.135	141.582	8.481

Distance of neutral axis from top fiber, $y_{cg} = (Ly/L) = (44.135/22.774) = 1.938$ in.

$I'_{eff} = Ly^2 + I_1' - Ly^2_{cg}$
$= 141.582 + 8.481 - 22.774(1.938)^2$
$= 64.53$ in.^{3}

Actual $I_{eff} = tI'_{eff}$
$= (0.060)\,(64.53)$
$= 3.87$ in.^{4}

$S_{eff} = [I_{eff}/(4 - y_{cg})] = [3.87/(4 - 1.938)] = 1.88$ in.^{3}

$M_{f=25.24\text{ ksi}} = (1.88)\,(25.24)$
$= 47.5$ kip-in. Close enough *OK*

Use I = 3.87 in.^{4} in deflection calculations

7. Comparison of sections with and without intermediate stiffeners

Section	Total Area (in.^{2})	Allowable Moment Capacity (kip-in.)
No Stiffener	1.43	31.1
Stiffener	1.49	49.7

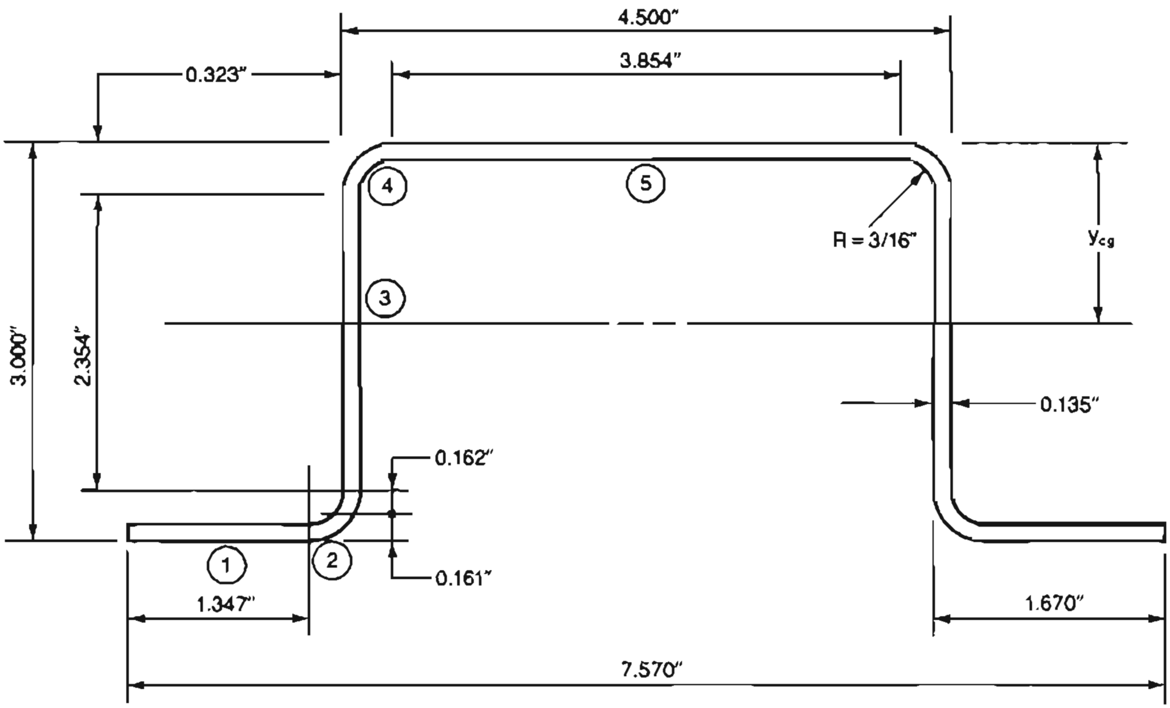
Increase in weight = [(1.49 − 1.43)/1.43] × 100% = 4.2%

Increase in moment capacity = [(49.7 − 31.1)/31.1] × 100% = 59.8%

EXAMPLE NO. 7

HAT SECTION

Inelastic Reserve



Given:
1. Steel: $F_y = 50$ ksi.
2. Section: Shown in sketch.
3. Top flange continuously supported.
4. Span = 8 ft. (simply supported).

Required: Determine flexural strength of section.

Solution:

1. Properties of 90° Corners:
   Radius to centerline,
   $r = R + t/2 = 3/16 + 0.135/2 = 0.255$ in.
   Length of arc, $u = 1.57r = 1.57(0.255) = 0.400$ in.
   Distance of c.g. from center of radius, $c = 0.637r = 0.637(0.255) = 0.162$ in.
   I' of corner about its own centroidal axis $= 0.149r^3 = 0.149(0.255)^3 = 0.003$ in.^{3}. *This is negligible.*
   Equations referenced from p. III-7 of Supplementary Information

2. Nominal Section Strength (Section C3.1.1)

   (a) Procedure I—Based on Initiation of Yielding
   Computation of I_x, first approximation:
   - Assume a compressive stress of $f = F_y = 50$ ksi in the top fibers of the section.
   - Assume web is fully effective.

   Element ③:
   $h/t = 2.354/0.135 = 17.44 < 200$ *OK* [Section B1.2-(a)]

   Assumed fully effective

Element ⑤:

$w/t = 3.854/0.135 = 28.55 < 500$ *OK* [Section B1.1-(a)-(2)]

$k = 4$

$\lambda = (1.052/\sqrt{k})\,(w/t)\,\sqrt{f/E}$ (*Eq.* B2.1-4)

$= (1.052/\sqrt{4})\,(28.55)\,\sqrt{50/29500}$

$= 0.618$

For $\lambda < 0.673$

$b = w$ (*Eq.* B2.1-2)

$b = 3.854$ in. (Fully effective)

Element	L Effective Length (in.)	y Distance from Top Fiber (in.)	Ly (in.²)	Ly² (in.³)	I'_1 About Own Axis (in.³)
1	2 × 1.347 = 2.694	2.933	7.902	23.175	—
2	2 × 0.400 = 0.800	2.839	2.271	6.448	—
3	2 × 2.354 = 4.708	1.500	7.062	10.593	2.174
4	2 × 0.400 = 0.800	0.161	0.129	0.021	—
5	3.854	0.068	0.262	0.018	—
Sum	12.856		17.626	40.255	2.174

Distance of neutral axis from top fiber, $y_{cg} = Ly/L = 17.626/12.856 = 1.371$ in.

Since the distance of the top compression fiber from the neutral axis is less than one half of the beam depth, a compressive stress of F_y will not govern as assumed. The actual compressive stress will be less than F_y and so the flange will still be fully effective. The tension flange will yield first. Section properties will not change.

Therefore,

$I'_x = Ly^2 + I'_1 - Ly^2_{cg}$

$= 40.255 + 2.174 - 12.856(1.371)^2$

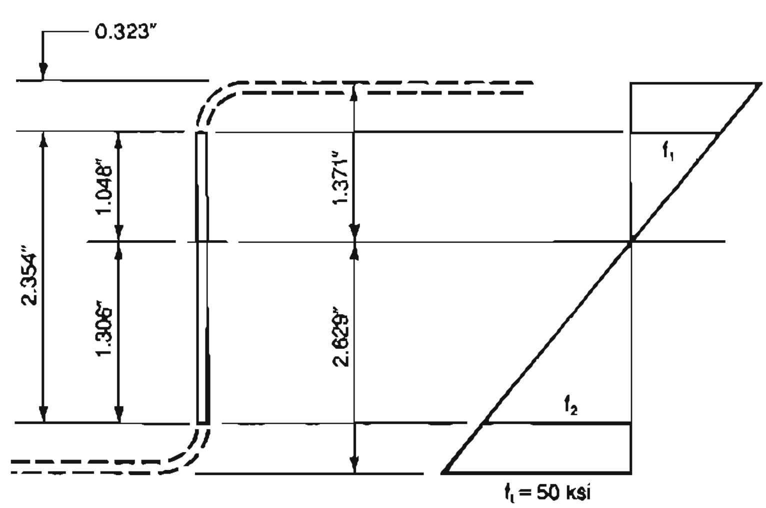
$= 18.26$ in.³

Actual $I_x = tI'_x$

$= (0.135)\,(18.26)$

$= 2.47$ in.⁴

Check Web



$f_1 = (1.048/2.629)(50) = 19.93$ ksi (compression)
$f_2 = (1.306/2.629)(50) = 24.84$ ksi (tension)
$\psi = f_2/f_1 = -24.84/19.93 = -1.246$

$k = 4 + 2(1-\psi)^3 + 2(1-\psi)$
$= 4 + 2[1-(-1.246)]^3 + 2[1-(-1.246)]$
$= 31.15$

$\lambda = (1.052/\sqrt{k})(w/t)\sqrt{f/E}$, $f = f_1$ (*Eq.* B2.1-4)
$= (1.052/\sqrt{31.15})(17.44)\sqrt{19.93/29500}$
$= 0.085$

For $\lambda < 0.673$
$b = w$ (*Eq.* B2.1-1)
$b_e = 2.354$ in.
$b_2 = b_e/2 = 2.354/2 = 1.177$ in. (*Eq.* B2.3-2)
$b_1 = b_e/(3-\psi)$
$= 2.354/[3-(-1.246)]$
$= 0.554$ in.
$b_1 + b_2 = 0.554 + 1.177$
$= 1.731 > 1.048$ (compression portion of web)

Therefore web is fully effective.

$S_e = I_x/(d - y_{cg}) = 2.47/(4-1.731) = 1.09$ in.^{3}

$M_n = S_eF_y$ (*Eq.* C3.1.1-1)
$= (1.09)(50)$
$= 54.5$ kip-in.

(b) Procedure II—Based on Inelastic Reserve Capacity

$\lambda_1 = (1.11/\sqrt{F_y/E}) = (1.11/\sqrt{50/29500}) = 26.96$

$\lambda_2 = (1.28/\sqrt{F_y/E}) = (1.28/\sqrt{50/29500}) = 31.09$

$w/t = 28.55$

For $26.96 = \lambda_1 < w/t < \lambda_2 = 31.09$

$C_y = 3 - 2[(w/t) - \lambda)/(\lambda_2 - \lambda_1)$
$= 3 - 2(28.55 - 26.96)/(31.09 - 26.96)$
$= 2.23$

Compute location of e_y on strain diagram, the summation of longitudinal forces should be zero.

Using equations from Reck, Peköz, and Winter, "Inelastic Strength of Cold-Formed Steel Beams," *Journal of the Structural Division*, November 1975, ASCE.

Distance from neutral axis to the outer compression fiber, y_c:
$t = 0.135$ in.
$b_t = 2(1.670) = 3.340$ in.
$b_c = 4.500$ in.
$d = 3.000$ in.

$y_c = (1/4)(b_t - b_c + 2d)$
$= (1/4)[3.340 - 4.500 + 2(3.000)]$
$= 1.210$ in.

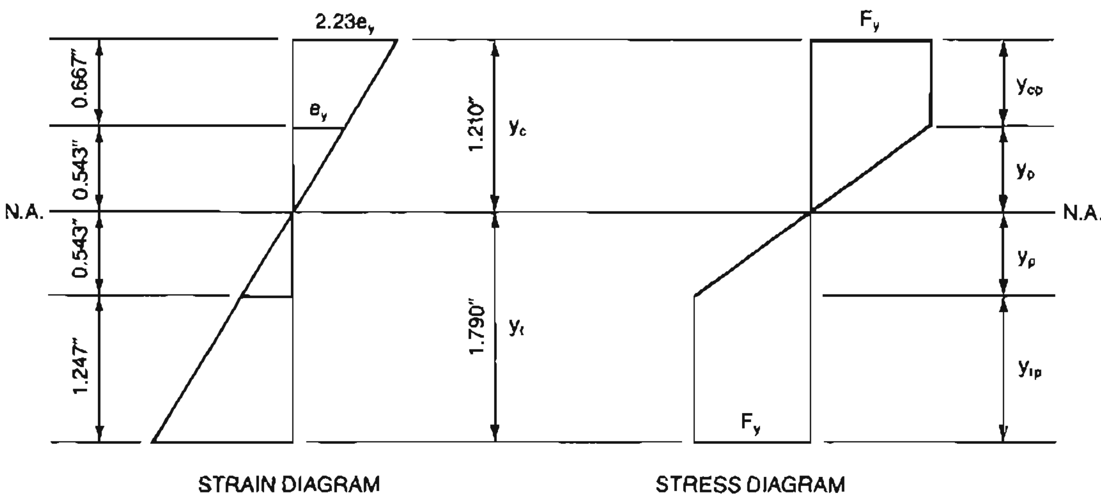
$y_p = y_c/C_y$
$= 1.21/2.23$
$= 0.543$

$y_t = d - y_c$
$= 3.000 - 1.210 = 1.790$ in.
$y_{cp} = y_c - y_p$
$= 1.210 - 0.543 = 0.667$ in.

$$y_{tp} = y_t - y_p$$
$$= 1.790 - 0.543 = 1.247 \text{ in.}$$

Summing moments of stresses in component plates:

$$M_n = F_y t\{b_c y_c + 2y_{cp}[y_p + (y_{cp}/2)] + (4/3)y_p^2 + 2y_{tp}[y_p + (y_{tp}/2)] + b_t y_t\}$$



$$M_n = 50(0.135)\{4.500(1.210) + 2(0.667)[0.543 + (0.667/2)] + (4/3)(0.543)^2 + 2(1.247)[0.543 + (1.247/2)] + 3.340(1.790)\}$$

$$M_n = 107.3 \text{ kip-in.}$$

M_n shall not exceed $1.25\, S_e F_y = 1.25(54.5) = 68.1$ kip-in.

Therefore
$$M_n = S_e F_y = 54.5 \text{ kip-in.}$$

The inelastic reserve capacity can be used because:

1) Member is not subject to twisting, lateral, torsional, or torsional-flexural buckling.
2) The effect of cold-forming is not included in determining the yield point, F_y.
3) The ratio of depth of the compressed portion of the web to its thickness does not exceed λ_1,

   $(1.210 - 0.323)/0.135 = 8.96 < \lambda_1 = 26.96$ *OK*

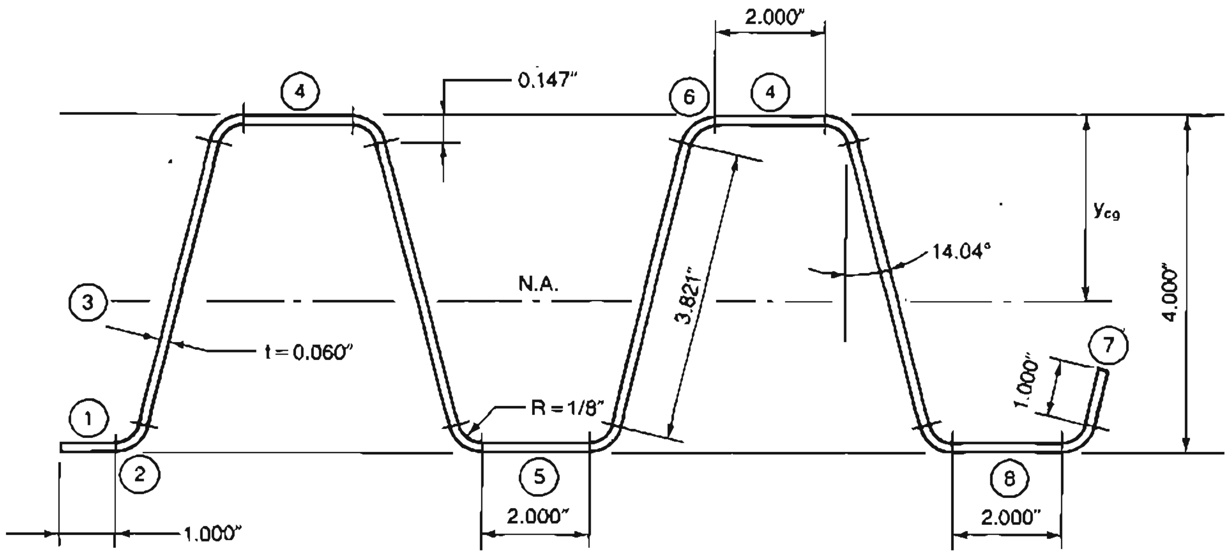
4) The shear force does not exceed $0.35F_y$ times the web area, $h \times t$.
   This still needs to be checked for a complete design.
5) The angle between any web and the vertical does not exceed 30°.

3. Allowable Bending Moment

$$M_a = M_n/\Omega_f \qquad (Eq.\ C3.1.1)$$
$$= 68.1/1.67$$
$$= 40.8 \text{ kip-in.}$$

EXAMPLE NO. 8

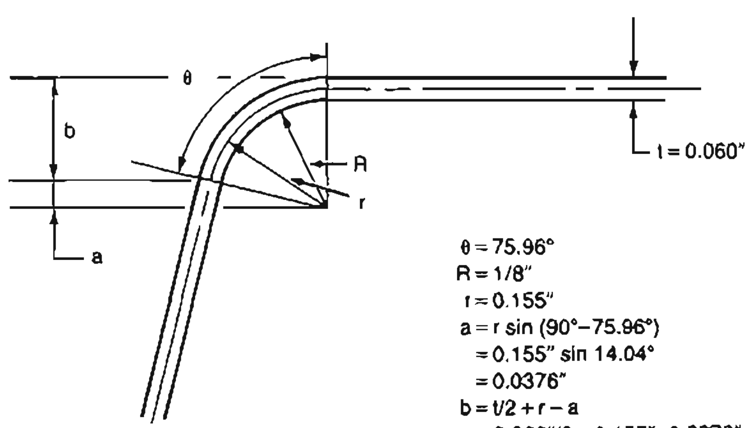
DECK SECTION



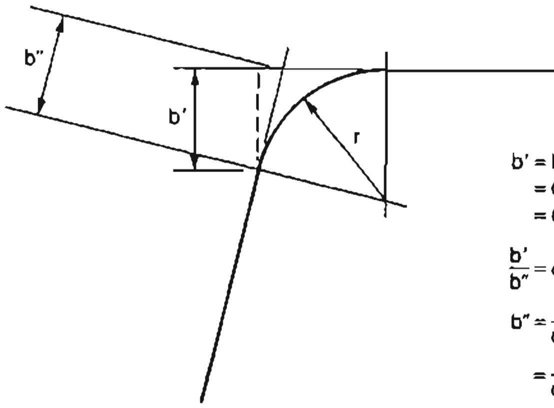
Given: 1. Steel: $F_y = 50$ ksi.
2. Section: Shown in sketch above.

Required: 1. Allowable bending moments for positive and negative bending.
2. Allowable uniform load as controlled either by bending or deflection when deck is continuous over three 10'-0" spans. Deflection due to live load is to be limited to 1/240 of the span.

Corner Properties:



$\theta = 75.96°$
$R = 1/8"$
$r = 0.155"$
$a = r \sin(90° - 75.96°)$
$= 0.155" \sin 14.04°$
$= 0.0376"$
$b = t/2 + r - a$
$= 0.060"/2 + 0.155" - 0.0376"$
$= 0.147"$

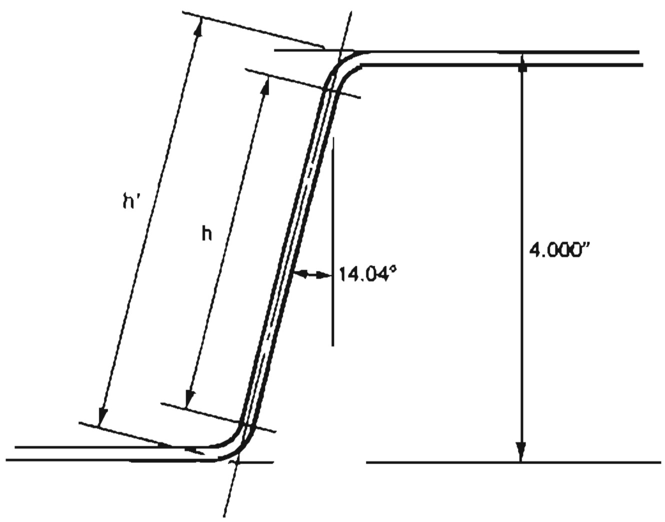


$$b' = b - t/2 = 0.147" - 0.060/2 = 0.117"$$

$$\frac{b'}{b"} = \cos(90^\circ - 75.96^\circ)$$

$$b" = \frac{b'}{\cos 14.04^\circ} = \frac{0.117"}{\cos 14.04^\circ} = 0.121$$

Flat portion of web



$$h' = \frac{4.000"}{\cos 14.04^\circ} = 4.123"$$

$$h = h' - 2b" = 4.123" - 2(0.121") = 3.881"$$

Solution:

1. Full Section Properties:

   Elements ② and ⑥:

   Radius to centerline,
   $r = R + t/2 = 1/8 + 0.060/2 = 0.155$ in.
   Angle, $\theta = 75.96^\circ = 1.326$ rad
   Length of arc, $u = \theta r = 1.326(0.155) = 0.206$ in.
   Distance of c.g. from center of radius, $c_1 = r \sin\theta/\theta = 0.155 \sin 1.326/1.326 = 0.113$ in.
   I_1' of arc element about its own centroidal axis is negligible.
   Equations referenced from p. III-7 of Supplementary Information

   Element ③:

   $l = 3.821$ in.
   $\theta = 14.04^\circ$
   $\cos\theta = 0.9701$
   †$I_1' = (\cos^2\theta\, l^3)/12 = [(0.9701)^2 (3.821)^3]/12 = 4.375$ in.^{3}
   †Equation referenced from p. III-6 of Supplementary Information

Element ⑦:

$l = 1.000$ in.
$\theta = 14.04°$
$\cos\theta = 0.9701$
$I_1' = (\cos^2\theta\, l^3)/12 = [(0.9701)^2 (1)^3]/12 = 0.0784$ in.^{3}
Distance of centroid of full section from top fiber,
$y = 4 - 0.147 - (1.000/2)\cos 14.04° = 3.368$ in.
Equation referenced from p. III-6 of Supplementary Information
Equivalent to *Eq.* B4-3 divided by t.

2. Section Modulus for Load Determination—Positive Bending (Based on Procedure I)

Since the effective design width of flat compressive elements is a function of stress, iteration is required.

Computation of I_x, first approximation:

- Assume a compressive stress of $f = F_y = 50$ ksi in the top fibers of the section.
- Assume web is fully effective.

Element ③:
$h/t = 3.821/0.060 = 63.68 < 200$ *OK* [Section B1.2-(a)]

Assumed fully effective

Element ④:
$w/t = 2.000/0.060 = 33.33 < 500$ *OK* [Section B1.1-(a)-(2)]
$K = 4$

$$\lambda = (1.052/\sqrt{k})(w/t)\sqrt{f/E}, \quad f = F_y \qquad (Eq.\ B2.1\text{-}4)$$
$$= (1.052/\sqrt{4})(33.33)\sqrt{50/29500}$$
$$= 0.722$$

For $\lambda < 0.673$

$$\rho = (1 - 0.22/\lambda)/\lambda \qquad (Eq.\ B2.1\text{-}3)$$
$$= (1 - 0.22/0.722)/0.722$$
$$= 0.963$$

$$b = \rho w \qquad (Eq.\ B2.1\text{-}2)$$
$$= 0.963(2.000)$$
$$= 1.926 \text{ in.}$$

Element	L Effective Length (in.)	y Distance from Top Fiber (in.)	Ly (in.^{2})	Ly^2 (in.^{3})	I_1' About Own Axis (in.^{3})
1	1.000	3.970	3.970	15.761	—
2	5 × 0.206 = 1.030	3.928	4.046	15.892	—
3	4 × 3.821 = 15.284	2.000	30.568	61.136	17.500
4	2 × 1.926 = 3.852	0.030	0.116	0.004	—
5 & 8	2 × 2.000 = 4.000	3.970	15.880	63.044	—
6	4 × 0.206 = 0.824	0.072	0.059	0.004	—
7	1.000	3.368	3.368	11.343	0.078
Sum	26.990		58.007	167.184	17.578

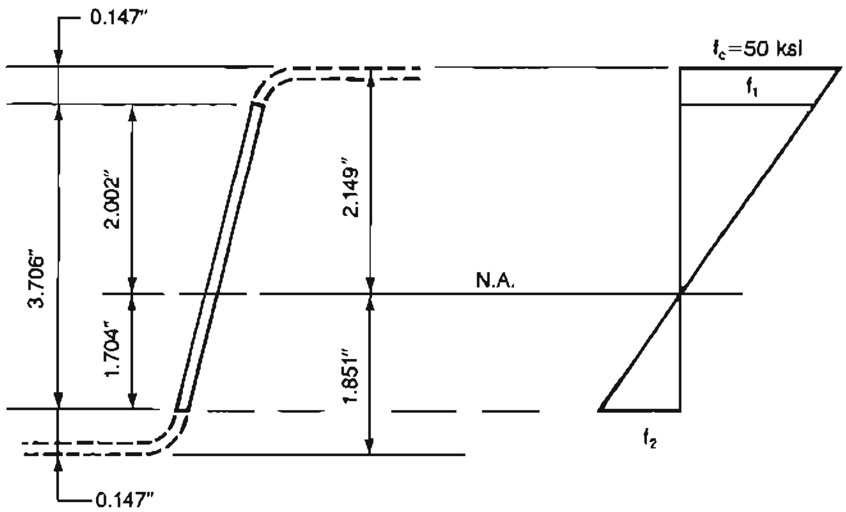
Distance of neutral axis from top fiber, $y_{cg} = Ly/L = 58.007/26.990 = 2.149$ in.

Since the distance of the top compression fiber from the neutral axis is greater than one half of the deck depth, a compressive stress of F_y will govern as assumed.

$I'_x = Ly^2 + I'_1 - Ly_{cg}^2$
$= 167.184 + 17.578 - 26.990(2.149)^2$
$= 60.12 \text{ in.}^3$

Actual $I_x = tI'_x$
$= (0.06)(60.12)$
$= 3.61 \text{ in.}^4$

Check Web



$f_1 = (2.002/2.149)(50) = 46.58$ ksi (compression)
$f_2 = (1.704/2.149)(50) = 39.65$ ksi (tension)
$\psi = f_2/f_1 = -39.65/46.58 = -0.851$

$k = 4 + 2(1-\psi)^3 + 2(1-\psi)$
$= 4 + 2[1-(-0.851)]^3 + 2[1-(-0.851)]$
$= 20.39$

$\lambda = (1.052/\sqrt{k})(w/t)\sqrt{f/E}, \quad f = f_1$ (*Eq.* B2.1-4)
$= (1.052/\sqrt{20.39})(63.68)\sqrt{46.58/29500}$
$= 0.590$

For $\lambda < 0.673$
$b = w$ (*Eq.* B2.1-1)
$b_e = 3.821$ in.
$b_2 = b_e/2 = 3.821/2 = 1.911$ in. (*Eq.* B2.3-2)
$b_1 = b_e/(3-\psi)$ (*Eq.* B2.3-1)
$= 3.821/[3-(-0.851)]$
$= 0.992$ in.
$b_1 + b_2 = 0.992 + 1.911$
$= 2.903 > 2.002$ (compressive portion of web)

Therefore web is fully effective.

$S_e = I_x/y_{cg} = 3.61/2.149 = 1.68 \text{ in.}^3$

$M_n = S_eF_y$ (*Eq.* C3.1.1-1)
$= (1.68)(50)$
$= 84.0$ kip-in.

$M_a = M_n/\Omega_f$ (*Eq.* C3.1-1)
$= 84.0/1.67$
$= 50.3$ kip-in.

3. Moment of Inertia for Deflection Determination—Positive Bending

Computation of I_{eff}, first approximation:

- Assume a compressive stress of $f = 0.6F_y = 30$ ksi in the top fibers of the section.
- Since the web was fully effective at a higher stress gradient, it will be fully effective at this stress level.

Element ④:

$w/t = 33.3$
$k = 4$

$\lambda = (1.052/\sqrt{k})\,(w/t)\,\sqrt{f/E}$ (*Eq.* B2.1-4)
$= (1.052/\sqrt{4})\,(33.33)\,\sqrt{30/29500}$
$= 0.559$

For $\lambda < 0.673$
$b_d = w$ (*Eq.* B2.1-5)
$b_d = 2.000$ in. (Fully effective)

Note: All elements are fully effective.

Element	L Effective Length (in.)	y Distance from Top Fiber (in.)	Ly (in.^{2})	Ly^2 (in.^{3})	I_1' About Own Axis (in.^{3})
1	1.000	3.970	3.970	15.761	—
2	5 × 0.206 = 1.030	3.928	4.046	15.892	—
3	4 × 3.821 = 15.284	2.000	30.568	61.136	17.500
4	2 × 2.000 = 4.000	0.030	0.120	0.004	—
5 & 8	2 × 2.000 = 4.000	3.970	15.880	63.044	—
6	4 × 0.206 = 0.824	0.072	0.059	0.004	—
7	1.000	3.368	3.368	11.343	0.078
Sum	27.138		58.011	167.184	17.578

Distance of neutral axis from top fiber, $y_{cg} = Ly/L = 58.011/27.138 = 2.138$ in.

Since the distance of the top compression fiber from the neutral axis is greater than one half the deck depth, the compressive stress of 30 ksi will govern as assumed.

$I'_{eff} = Ly^2 + I_1' - Ly_{cg}^2$
$= 167.184 + 17.578 - 27.138(2.138)^2$
$= 60.71$ in.^{3}

Actual $I_{eff} = tI'_{eff}$
$= (0.060)\,(60.71)$
$= 3.64$ in.^{4}

$S_{eff} = I_{eff}/y_{cg} = 3.64/2.138 = 1.70$ in.^{3}

$M_{f=30\text{ ksi}} = S_{eff}\,(30)$
$= 1.70(30)$
$= 51.0 > M_a = 50.3$ kip-in.

$I_{eff} = 3.64$ in.^{4} can be used for deflection calculations for the following reasons:

1. $M(I_{eff} = 3.64) \approx M_a$
2. In order to achieve $M(I_{eff}) = M_a$ a lower compressive stress would be required, but the section is already fully effective.

$I_{eff} = 3.64$ in.^{4}

4. Section Modulus for Load Determination—Negative Bending (Based on Procedure I)

Following a similar procedure as in positive bending.

Computation of I_x, first approximation:

- Assume a compressive stress of $f = F_y = 50$ ksi in the bottom fiber of the section.
- Assume web is fully effective.

Element ③:

$h/t = 3.821/0.060 = 63.68 < 200$ *OK* [Section B1.2-(a)]

Assumed fully effective

Element ①:

$w/t = 1.000/0.060 = 16.67 < 60$ *OK* [Section B1.1-(a)-(3)]

$k = 0.43$

$$\lambda = (1.052/\sqrt{k})(w/t)\sqrt{f/E} \quad (Eq.\ B2.1\text{-}4)$$
$$= (1.052/\sqrt{0.43})(16.67)\sqrt{50/29500}$$
$$= 1.101$$

For $\lambda > 0.673$

$$\rho = (1-0.22/\lambda)/\lambda \quad (Eq.\ B2.1\text{-}3)$$
$$= (1-0.22/0.22/1.101)/1.101$$
$$= 0.727$$

$$b = \rho w \quad (Eq.\ B2.1\text{-}2)$$
$$= 0.727(1.000)$$
$$= 0.727 \text{ in.}$$

Element ⑤:

Same as element ④ in positive bending case

$b = 1.926$ in.

Element ⑧:

$w/t = 2.000/0.060 = 33.33 < 60$ *OK* [Section B1.1-(a)-(3)] Conservative check.

$$S = 1.28\sqrt{E/f}$$
$$= 1.28\sqrt{29500/50}$$
$$= 31.09$$

For $w/t > S$

$$I_a = t^4\{[115(w/t)/S] + 5\}$$
$$= (0.060)^4\{[115(33.33)/31.09] + 5\}$$
$$= 0.00166 \text{ in.}^4$$

$$I_s = (d^3 t \sin^2\theta)/12$$
$$= (1.000)^3(0.060)(\sin 75.96°)^2/12$$
$$= 0.00471 \text{ in.}^4$$

$$D = 0.121 + 1.000$$
$$= 1.121 \text{ in.}$$

$D/w = 1.121/2.000 = 0.561$

For $0.25 < D/w < 0.8$

$$k = [4.82 - 5(D/w)](I_s/I_a)^{1/3} + 0.43 \le 5.25 - 5(D/w) \quad (Eq.\ B4.2\text{-}9)$$

$$[4.82 - 5(D/w)](I_s/I_a)^{1/3} + 0.43 = [4.82 - 5(0.561)](0.00471/0.00166)^{1/3}$$
$$= 2.856$$

$$5.25 - 5(D/w) = 5.25 - 5(0.561)$$
$$= 2.445$$

$$k = 2.445$$

$$\lambda = (1.052/\sqrt{k})(w/t)\sqrt{f/E} \quad (Eq.\ B2.1\text{-}4)$$
$$= (1.052/\sqrt{2.445})(33.33)\sqrt{50/29500}$$
$$= 0.923$$

For $\lambda > 0.673$

$\rho = (1-0.22/\lambda)/\lambda$ (*Eq.* B2.1-3)
$= (1-0.22/0.923)/0.923$
$= 0.825$

$b = \rho w$
$= 0.825(2.000)$
$= 1.650$ in.

Element ⑦:

$I_s = 0.00471$ in.^{4} (calculated previously)
$I_a = 0.00166$ in.^{4} (calculated previously)
$d = 1.000$ in.

Assume max stress in element, $f = F_y = 50$ ksi although it will be actually less.

$k = 0.43$
$w/t = 1.000/0.060 = 16.67 < 60$ *OK* [Section B1.1-(a)-(3)]

$\lambda = (1.052/\sqrt{k})(w/t)\sqrt{f/E}$ (*Eq.* B2.1-4)
$= (1.052/\sqrt{0.43})(16.67)\sqrt{50/29500}$
$= 1.101$

For $\lambda > 0.673$

$\rho = (1-0.22/\lambda)/\lambda$ (*Eq.* B2.1-3)
$= (1-0.22/1.101)/1.101$
$= 0.727$

$b = \rho w$ (*Eq.* B2.1-2)
$= 0.727(1.000)$
$= 0.727$

$d_s' = 0.727$ in.
$d_s = d_s'(I_s/I_a) \le d_s'$ (*Eq.* B4.3-11)

Since $I_s/I_a > 1$

$d_s = d_s' = 0.727$ in.
$I_1' = (d_s)^3 \sin^2\theta/12 = (0.727)^3 (\sin 75.96°)^2/12 = 0.030$ in.^{3}

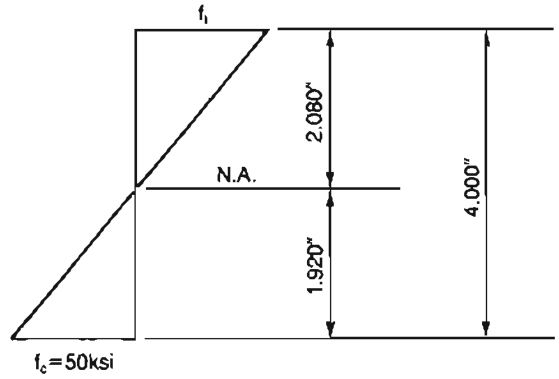
Distance of centroid of reduced section from top fiber,

$y = 4 - 0.147 - (0.727/2) \cos 14.04° = 3.500$ in.

Element	L Effective Length (in.)	y Distance from Top Fiber (in.)	Ly (in.^{2})	Ly^2 (in.^{3})	I_1' About Own Axis (in.^{3})
1	0.727	3.970	2.886	11.458	—
2	5 × 0.206 = 1.030	3.928	4.046	15.892	—
3	4 × 3.821 = 15.284	2.000	30.568	61.136	17.500
4	2 × 2.000 = 4.000	0.030	0.120	0.004	—
5	1.926	3.970	7.646	30.355	—
6	4 × 0.206 = 0.824	0.072	0.059	0.004	—
7	0.727	3.500	2.545	8.906	0.030
8	1.650	3.970	6.551	26.005	—
Sum	26.168		54.421	153.760	17.530

Distance of neutral axis from top fiber, $y_{cg} = (Ly/L) = (54.421/26.168) = 2.080$ in.

Since the distance of the top compression fiber from the neutral axis is greater than one-half the deck depth, a compressive stress of F_y will not govern as assumed. The compressive stress will be slightly less.



$$f_t = \frac{2.080}{1.920}(50\text{ ksi})$$
$$= 54.17\text{ ksi} > F_y$$

Computation of I_x, second approximation:

- Assume a compressive stress of $f = 45$ ksi
- Assume web is fully effective.

Element ①:

$w/t = 16.67$

$k = 0.43$

$\lambda = (1.052/\sqrt{k})(w/t)\sqrt{f/E}$ (*Eq.* B2.1-4)
$= (1.052/\sqrt{0.43})(16.67)\sqrt{45/29500}$
$= 1.045$

For $\lambda > 0.673$

$\rho = (1-0.22/\lambda)/\lambda$ (*Eq.* B2.1-3)
$= (1-0.22/1.045)/1.045$
$= 0.756$

$b = \rho w$ (*Eq.* B2.1-2)
$= 0.756(1.000)$
$= 0.756$ in.

Element ⑤:

$w/t = 33.33$

$k = 4$

$\lambda = (1.052/\sqrt{k})(w/t)\sqrt{f/E}$ (*Eq.* B2.1-4)
$= (1.052/\sqrt{4})(33.33)\sqrt{45/29500}$
$= 0.685$

For $\lambda > 0.673$

$\rho = (1-0.22/\lambda)/\lambda$ (*Eq.* B2.1-3)
$= (1-0.22/0.685)/0.685$
$= 0.991$

$b = \rho w$ (*Eq.* B2.1-2)
$= 0.991(2.000)$
$= 1.982$ in.

Element ⑧:

$w/t = 33.33$

$S = 1.28\sqrt{E/f}$ (*Eq.* B4-1)
$= 1.28\sqrt{29500/45}$
$= 32.77$

For $w/t > S$

$I_a = t^4[115(w/t)/S] + 5$ (*Eq.* B4.2-13)

$= (0.060)^4 \{[115(33.33)/32.77] + 5\}$
$= 0.00158$ in.^{4}

$I_s = 0.00471$ in.^{4} (calculated previously)
$D = 1.121$ in. (calculated previously)
$D/w = 0.561$ (calculated previously)

For $0.25 < D/w < 0.8$

$$k = [4.82 - 5(D/w)](I_s/I_a)^{1/3} + 0.43 \leq 5.25 - 5(D/w) \qquad (Eq.\ B4.2\text{-}9)$$

$$[4.82 - 5(D/w)](I_s/I_a)^{1/3} + 0.43 = [4.82 - 5(0.561)](0.00471/0.00158)^{1/3} + 0.43 = 3.330$$

$$5.25 - 5(D/w) = 5.25 - 5(0.561) = 2.445$$

$$k = 2.445$$

$$\lambda = (1.052/\sqrt{k})(w/t)\sqrt{f/E} \qquad (Eq.\ B2.1\text{-}4)$$
$$= (1.052/\sqrt{2.445})(33.33)\sqrt{45/29500}$$
$$= 0.876$$

For $\lambda > 0.673$

$$\rho = (1 - 0.22/\lambda)/\lambda \qquad (Eq.\ B2.1\text{-}3)$$
$$= (1 - 0.22/0.876)/0.876$$
$$= 0.855$$

$$b = \rho w \qquad (Eq.\ B2.1\text{-}2)$$
$$= 0.855(2.000)$$
$$= 1.710 \text{ in.}$$

Element ⑦:

$I_s = 0.00471$ in.^{4} (calculated previously)
$I_a = 0.00158$ in.^{4} (calculated previously)
$d = 1.000$ in.

Assume max stress in element, $f = 45$ ksi although it will be actually less.

$k = 0.43$
$w/t = 16.67$

$$\lambda = (1.052/\sqrt{k})(w/t)\sqrt{f/E} \qquad (Eq.\ B2.1\text{-}4)$$
$$= (1.052/\sqrt{0.43})(16.67)\sqrt{45/29500}$$
$$= 1.045$$

For $\lambda > 0.673$

$$\rho = (1 - 0.22/\lambda)/\lambda \qquad (Eq.\ B2.1\text{-}3)$$
$$= (1 - 0.22/1.045)/1.045$$
$$= 0.756$$

$$b = \rho w \qquad (Eq.\ B2.1\text{-}2)$$
$$= (0.756)(1.000)$$
$$= 0.756 \text{ in.}$$

$d_s' = 0.756$ in.

$$d_s = d_s'(I_s/I_a) \leq d_s' \qquad (Eq.\ B4.2\text{-}11)$$

Since $I_s/I_a > 1$

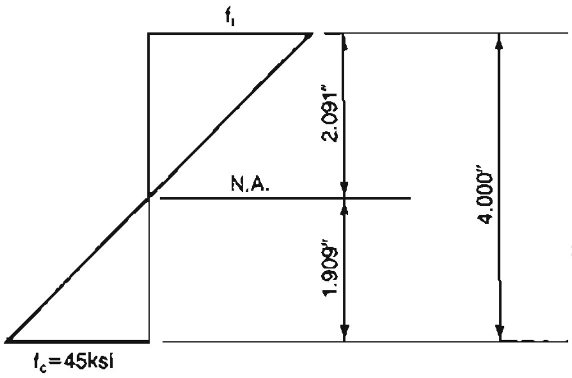
$d_s = d_s' = 0.756$ in.
$I_1' = (d_s)^3 \sin^2\theta/12 = (0.756)^3(\sin 75.96°)^2/12 = 0.034$ in.^{3}

Distance of centroid of reduced section from top fiber,

$y = 4 - 0.147 - (0.756/2)\cos 14.04 = 3.486$ in.

Element	L Effective Length (in.)	y Distance from Top Fiber (in.)	Ly (in.²)	Ly² (in.³)	I_1' About Own Axis (in.³)
1	0.756	3.970	3.001	11.915	—
2	5 × 0.206 = 1.030	3.928	4.046	15.892	—
3	4 × 3.821 = 15.284	2.000	30.568	61.136	17.500
4	2 × 2.000 = 4.000	0.030	0.120	0.004	—
5	1.982	3.970	7.869	31.238	—
6	4 × 0.206 = 0.824	0.072	0.059	0.004	—
7	0.756	3.486	2.635	9.187	0.034
8	1.710	3.970	6.789	26.951	—
Sum	26.342		55.087	156.327	17.534

Distance of neutral axis from top fiber, $y_{cg} = Ly/L = 55.087/26.342 = 2.091$ in.

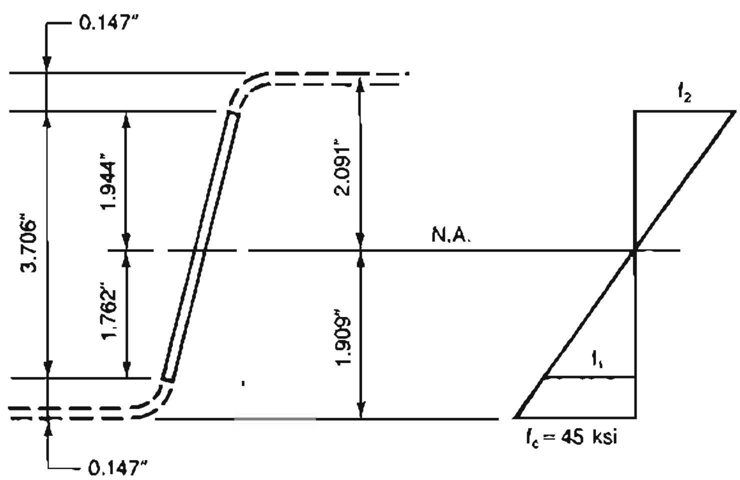


$f_1 = \frac{2.091}{1.909}$ (45 ksi)

$= 49.29$ ksi

Satisfactory

Check Web



$f_1 = (1.762/1.909)(45) = 41.53$ ksi (compression)

$f_2 = (1.944/1.909)(45) = 45.83$ ksi (tension)

$\psi = f_2/f_1 = -45.83/41.53 = -1.104$

$k = 4 + 2(1-\psi)^3 + 2(1-\psi)$

$= 4 + 2[1-(-1.104)]^3 + 2[1-(-1.104)]$

$= 26.84$

$\lambda = (1.052/\sqrt{k})(w/t)\sqrt{f/E}, \quad f = f_1$ (*Eq.* B2.1-4)
$= (1.052/\sqrt{26.84})(63.68)\sqrt{41.53/29500}$
$= 0.485$

For $\lambda < 0.673$

$b = w$ (*Eq.* B2.1-1)
$b_e = 3.821$ in.

$b_2 = b_e/2$ (*Eq.* B2.3-2)
$= 3.821/2$
$= 1.911$ in.

$b_1 = b_e/(3-\psi)$ (*Eq.* B2.3-1)
$= 3.821/[3-(-1.104)]$
$= 0.931$ in.

$b_1 + b_2 = 0.931 + 1.911$
$= 2.842 > 1.765$ (compression portion of web)

Therefore web is fully effective.

Check Element ⑦:

Maximum stress in element, $f = 41.54$ ksi

$k = 0.43$
$w/t = 16.67$

$\lambda = (1.052/\sqrt{k})(w/t)\sqrt{f/E}$ (*Eq.* B2.1-4)
$= (1.052/\sqrt{0.43})(16.67)\sqrt{41.53/29500}$
$= 1.003$

For $\lambda > 0.673$

$\rho = (1-0.22/\lambda)/\lambda$ (*Eq.* B2.1-3)
$= (1-0.22/1.003)/1.003$
$= 0.778$

$b = \rho w$ (*Eq.* B2.1-2)
$= (0.778)(1.000)$
$= 0.778$ in.

$d_s' = 0.778$ in.
$d_s = d_s'(I_s/I_a) \le d_s'$ (*Eq.* B4.2-11)

Since $I_s/I_a > 1$

$d_s = d_s'$
$= 0.778$ in.
$I_1' = (d_s)^3 \sin^2\theta/12 = (0.778)^3(\sin 75.96°)^2/12 = 0.037$ in.^{3}

Distance of centroid of reduced section from top fiber,

$y = 4 - 0.147 - (0.778/2)\cos 14.04° = 3.476$ in.

Determine section properties, but only the properties of element ⑦ have changed

$\Delta L = 0.778 - 0.756$
$= 0.022$ in.
$\Delta Ly = (0.778)(3.476) - 2.635$
$= 0.069$ in.^{2}
$\Delta Ly^2 = 0.778(3.476)^2 - 9.187$
$= 0.213$ in.^{3}
$\Delta I_1' = 0.037 - 0.034$
$= 0.003$ in.^{3}

Therefore,

$L = 26.342 + 0.022 = 26.364$ in.

$Ly = 55.087 + 0.069 = 55.156 \text{ in.}^2$
$Ly^2 = 156.327 + 0.213 = 156.540 \text{ in.}^3$
$I'_1 = 17.534 + 0.003 = 17.537 \text{ in.}^3$

Distance of neutral axis from top fiber, $y_{cg} = Ly/L = 55.156/26.364 = 2.092$ in.

$f_t = (2.092/1.908)(45) = 49.34 \text{ ksi} \approx 50 \text{ ksi}$ *OK*

$I'_x = Ly^2 + I'_1 - Ly_{cg}^2$
$= 156.540 + 17.537 - 26.364(2.092)^2$
$= 58.70 \text{ in.}^3$

Actual $I_x = tI'_x$
$= (0.060)(58.70)$
$= 3.52 \text{ in.}^4$

$S_e = I_x/y_{cg} = 3.52/2.092 = 1.68 \text{ in.}^3$

$M_n = S_eF_y$ (*Eq*. C3.1.1-1)
$= (1.68)(50)$
$= 84.0$ kip-in.

$M_a = M_n/\Omega_f$ (*Eq*. C3.1-1)
$= 84.0/1.67$
$= 50.3$ kip-in.

5. Moment of Inertia for Deflection Determination—Negative Bending

Computation of I_{eff}, first approximation:
- Assume a compressive stress of $f = 27$ ksi in the bottom fiber of the section.
- Since the web was fully effective at a higher stress gradient, it will be fully effective at this stress level.

Element ①:

$w/t = 16.67$

$k = 0.43$

$\lambda = (1.052/\sqrt{k})(w/t)\sqrt{f/E}$ (*Eq*. B2.1-4)
$= (1.052/\sqrt{0.43})(16.67)\sqrt{27/29500}$
$= 0.809$

For $\lambda > 0.673$
$\rho = (1 - 0.22/\lambda)/\lambda$ (*Eq*. B2.1-3)
$= (1 - 0.22/0.809)/0.809$
$= 0.900$

$b = \rho w$ (*Eq*. B2.1-2)
$= (0.900)(1.000)$
$= 0.900$ in.

Element ⑤:

$w/t = 33.33$
$k = 4$

$\lambda = (1.052/\sqrt{k})(w/t)\sqrt{f/E}$ (*Eq*. B2.1-4)
$= (1.052/\sqrt{4})(33.33)\sqrt{27/29500}$
$= 0.530$

For $\lambda < 0.673$
$b_d = w$ (*Eq*. B2.1-5)
$b_d = 2.000$ in. (Fully effective)

Element ⑧:

$w/t = 33.33$

$S = 1.28\sqrt{E/f}$ (*Eq.* B4-1)
$= 1.28\sqrt{29500/27}$
$= 42.31$

For $S/3 < w/t < S$,

$I_a = t^4\, 399\{[(w/t)/S] - 0.33\}^3$ (*Eq.* B4.2-6)
$= (0.060)^4 (399) [(33.33/42.31) - 0.33]^3$
$= 0.000496$ in.^{4}

$I_s = 0.00471$ in.^{4} (calculated previously)
$I_s/I_a = 0.00471/0.000496 = 9.5 > 1$
$D = 1.121$ (calculated previously)
$D/w = 0.561$ (calculated previously)

For $0.25 < D/w < 0.8$

$k = [4.82 - 5(D/w)](I_s/I_a)^{1/2} + 0.43 \le 5.25 - 5(D/w)$ (*Eq.* B4.2-9)

Since $I_s/I_a > 1$

$k = 5.25 - 5(D/w)$
$= 5.25 - 5(0.561)$
$= 2.445$

$\lambda = (1.052/\sqrt{k})\,(w/t)\,\sqrt{f/E}$ (*Eq.* B2.1-4)
$= (1.052/\sqrt{2.445})\,(33.33)\,\sqrt{27/29500}$
$= 0.678$

For $\lambda > 0.673$
$\rho = (1 - 0.22/\lambda)/\lambda$ (*Eq.* B2.1-3)
$= (1 - 0.22/0.678)/0.678$
$= 0.996$

$b = \rho w$ (*Eq.* B2.1-2)
$= (0.996)\,(2.000)$
$= 1.992$ in.

Element ⑦:

$I_s/I_a > 1$

$d = 1.000''$

Assume max stress in element, $f = 27$ ksi although it will be actually less.

$k = 0.43$
$w/t = 16.67$

$\lambda = (1.052/\sqrt{k})\,(w/t)\,\sqrt{f/E}$ (*Eq.* B2.1-4)
$= (1.052/\sqrt{0.43})\,(16.67)\,\sqrt{27/29500}$
$= 0.809$

For $\lambda > 0.673$
$\rho = (1 - 0.22/\lambda)/\lambda$ (*Eq.* B2.1-3)
$= (1 - 0.22/0.809)/0.809$
$= 0.900$

$b = \rho w$ (*Eq.* B2.1-2)
$= (0.900)\,(1.000)$
$= 0.900$ in.

$d_s' = 0.900$ in.
$d_s = d_s'(I_s/I_a) \le d_s'$ (*Eq.* B4.2-11)

Since $I_s/I_a > 1$
$d_s = d_s' = 0.900$ in.
$I_1' = (d_s)^3 \sin^2\theta/12 = (0.900)^3 (\sin 75.96°)^2/12 = 0.057$ in.^{3}

Distance of centroid of reduced section from top fiber,

$y = 4 - 0.147 - (0.900/2) \cos 14.04° = 3.416$ in.

Element	L Effective Length (in.)	y Distance from Top Fiber (in.)	Ly (in.²)	Ly² (in.³)	I′₁ About Own Axis (in.³)
1	0.900	3.970	3.573	14.185	—
2	5 × 0.206 = 1.030	3.928	4.046	15.892	—
3	4 × 3.821 = 15.284	2.000	30.568	61.136	17.500
4	2 × 2.000 = 4.000	0.030	0.120	0.004	—
5	2.000	3.970	7.940	31.522	—
6	4 × 0.206 = 0.824	0.072	0.059	0.004	—
7	0.900	3.416	3.074	10.502	0.057
8	1.992	3.970	7.908	31.396	—
Sum	26.930		57.288	164.641	17.557

Distance of neutral axis from top fiber, $y_{cg} = Ly/L = 57.288/26.930 = 2.127$ in.

$$\begin{aligned} I'_{eff} &= Ly^2 + I'_1 - Ly^2_{cg} \\ &= 164.641 + 17.557 - 26.930(2.127)^2 \\ &= 60.36 \text{ in.}^3 \end{aligned}$$

$$\begin{aligned} \text{Actual } I_{eff} &= tI'_{eff} \\ &= (0.060)(60.36) \\ &= 3.62 \text{ in.}^4 \end{aligned}$$

$S_{eff} = I_{eff}/(d - y_{cg}) = 3.62/(4 - 2.127) = 1.93$ in.³

$$\begin{aligned} M_{(f=27 \text{ ksi})} &= (1.93)(27) \\ &= 52.1 \text{ ksi} > M_a = 50.3 \text{ ksi} \quad \text{N.G.} \end{aligned}$$

Computation of I_{eff}, second approximation

- Assume a compressive stress in the bottom fiber of the section using extrapolation.

① $f = 45$ ksi, $M = 84.0$ kip-in.

② $f = 27$ ksi, $M = 52.1$ kip-in.

③ $f = ?$ $M = 50.3$ kip-in.

$$\begin{aligned} (f - 27)(27 - 45) &= (50.3 - 52.1)(52.1 - 84.0) \\ f &= 26.0 \text{ ksi} \end{aligned}$$

Element ①:

$w/t = 16.67$

$k = 0.43$

$$\begin{aligned} \lambda &= (1.052/\sqrt{k})(w/t)\sqrt{f/E} \qquad (Eq.\ B2.1\text{-}4) \\ &= (1.052/\sqrt{0.43})(16.67)\sqrt{26/29500} \\ &= 0.794 \end{aligned}$$

For $\lambda > 0.673$

$$\begin{aligned} \rho &= (1 - 0.22/\lambda)/\lambda \qquad (Eq.\ B2.1\text{-}3) \\ &= (1 - 0.22/0.794)/0.794 \\ &= 0.911 \end{aligned}$$

$b = \rho w$ (*Eq.* B2.1-2)
$= (0.911)(1.000)$
$= 0.911$ in.

Element ⑤:
Fully effective at $f = 27$ ksi
It will also be fully effective at $f = 26$ ksi

$b = 2.000$ in.

Element ⑧:

$w/t = 33.33$
$S = 1.28\sqrt{E/f}$ (*Eq.* B4-1)
$= 1.28\sqrt{29500/26}$
$= 43.12$

For $S/3 < w/t < S$

$I_s/I_a > 1$ by observation
$D/w = 0.561$

Since $I_s/I_a > 1$
$k = 2.445$

$\lambda = (1.052/\sqrt{k})(w/t)\sqrt{f/E}$ (*Eq.* B2.1-4)
$= (1.052/\sqrt{2.445})(33.33)\sqrt{26/29500}$
$= 0.666$

For $\lambda < 0.673$
$b = w$ (*Eq.* B2.1-2)
$= 2.000$ in. (Fully effective)

Element ⑦:

$I_s/I_a > 1$

$d = 1.000$ in.

Assume maximum stress in element, $f = 26$ ksi although it will be actually less.

$k = 0.43$
$w/t = 16.67$

$\lambda = (1.052/\sqrt{k})(w/t)\sqrt{f/E}$ (*Eq.* B2.1-4)
$= (1.052/\sqrt{0.43})(16.67)\sqrt{26/29500}$
$= 0.794$

For $\lambda > 0.673$
$\rho = (1 - 0.22/\lambda)/\lambda$ (*Eq.* B2.1-3)
$= (1 - 0.22/0.794)/0.794$
$= 0.911$

$b = \rho w$ (*Eq.* B2.1-2)
$= (0.911)(1.000)$
$= 0.911$ in.

$d_s' = 0.911$ in.

Since $I_s/I_a > 1$
$d_s = d_s' = 0.911$ in.
$I_s' = (d_s)^3 \sin^2\theta/12 = (0.911)^3(\sin 75.96°)^2/12 = 0.059$ in.^{3}

Distance of centroid of reduced section from top fiber,

$y = 4 - 0.147 - (0.911/2)\cos 14.04° = 3.411$ in.

Element	L Effective Length (in.)	y Distance from Top Fiber (in.)	Ly (in.^{2})	Ly^2 (in.^{3})	I_1' About Own Axis (in.^{3})
1	0.911	3.970	3.617	14.358	—
2	5 × 0.206 = 1.030	3.928	4.046	15.892	—
3	4 × 3.821 = 15.284	2.000	30.568	61.136	17.500
4	2 × 2.000 = 4.000	0.030	0.120	0.004	—
5	2.000	3.970	7.940	31.522	—
6	4 × 0.206 = 0.824	0.072	0.059	0.004	—
7	0.911	3.411	3.107	10.599	0.059
8	2.000	3.970	7.940	31.522	—
Sum	26.960		57.397	165.037	17.559

Distance of neutral axis from top fiber $y_{cg} = Ly/L = 57.397/26.960 = 2.129$ in.

$$
\begin{aligned}
I'_{eff} &= Ly^2 + I_1' - Ly_{cg}^2 \\
&= 165.037 + 17.559 - 26.960(2.129)^2 \\
&= 60.40 \text{ in.}^3
\end{aligned}
$$

$$
\begin{aligned}
\text{Actual } I_{eff} &= tI'_{eff} \\
&= (0.060)(60.40) \\
&= 3.62 \text{ in.}^4
\end{aligned}
$$

$$S_{eff} = I_{eff}/(d - y_{cg}) = 3.62/4 - 2.129) = 1.93 \text{ in.}^3$$

$$
\begin{aligned}
M_{fc=26\text{ ksi}} &= (1.93)(26) \\
&= 50.2 \text{ ksi} \approx M_a = 50.3 \text{ ksi} \quad \textit{OK}
\end{aligned}
$$

Note: A slight adjustment could be made for element ⑦ since the actual maximum stress is less than f = 26 ksi, but the net effect will be negligible.

6. Summary

Positive Bending: M_a = 50.3 kip-in.
I_{eff} = 3.64 in.^{4}
Negative Bending: M_a = 50.3 kip-in.
I_{eff} = 3.62 in.^{4}

7. Compute Allowable Uniform Load

For a continuous deck over three equal spans, the maximum bending moment is negative and occurs over the interior supports. It is given by:

$$M = 0.100wL^2$$

Therefore, the maximum uniform load is

$$w = M/0.100L^2 = 50.3/0.100(10' \times 12''/1)^2 = 0.0349 \text{ kip/in.}$$

$$w = 0.419 \text{ kip/ft.}$$

The maximum deflection occurs at a distance of 0.446L from the exterior supports. It is given by:

$$\Delta = 0.0069wL^4/EI$$

This deflection is limited to, $\Delta = L/240$ for live load. Therefore, the maximum live load which will satisfy the deflection requirements is

$$w_{LL} = EI/240(0.0069)L^3 = 29500(3.64)/240(0.0069)(10 \times 12)^3 = 0.0375 \text{ kip/in.}$$

$$w_{LL} = 0.450 \text{ kip/ft.}$$

$w < w_{LL}$, therefore allowable bending moment governs.

Allowable Uniform Load = 0.419 kip/ft.

8. Check Shear Strength (Section C3.2)

$k_v = 5.34$, unreinforced web

$1.38\sqrt{EK_v/F_y} = 1.38\sqrt{29500(5.34)/50}$
$= 77.46$

$h/t = 3.821/0.060 = 63.68$

For $h/t < 1.38\sqrt{EK_v/F_y}$ (*Eq.* C3.2-1)

$V_a = 0.38t^2\sqrt{K_vF_yE}$
$= (0.38)(0.060)^2\sqrt{5.34(50)(29500)}$
$= 3.84\text{k}$

$V_a = (11800\ F_y/E)ht$
$= (11800)(50/29500)(3.821)(0.060)$
$= 4.59\text{k}$

$V_a = 3.84\text{k}$ controls (per web)

Total V_a for section:

$V_a = 4(3.84\text{k})$
$= 15.36\text{k}$

The maximum shear force is given by

$V = 0.600wL$
$= (0.600)(0.419)(10)$
$= 2.51\text{k}$

$V = 2.51\text{k} < V_a = 15.36\text{k}$ *OK*

9. Check Strength for Combined Bending and Shear (Section C3.3)

At the interior supports there is a combination of web bending and web shear.

$M_a = 50.3$ kip-in. $\quad M = 0.100wL^2$
$V_a = 15.36$ k $\quad V = 0.600wL$

For unreinforced webs

$(M/M_a)^2 + (V/V_a)^2 \leq 1.0$ (*Eq.* C3.3-1)

Solve for w:

$$\{[0.100w(10 \times 12)^2]/50.3\}^2 + \{[0.600w(10 \times 12)]/15.36\}^2 = 1.0$$
$$819.58w^2 + 21.97w^2 = 1.0$$
$$841.55w^2 = 1.0$$
$$w = 0.0345 \text{ kip/in.}$$
$$= 0.414 \text{ kip/ft.}$$

Allowable Uniform Load = 0.414 kip/ft.

10. Check Web Crippling Strength (Section C3.4)

$h = 3.821$ in.
$t = 0.060$ in.
$h/t = 3.821/0.060 = 63.68 < 200$ *OK*

$R = 1/8$ in.
$R/t = 0.125/0.06 = 2.083 < 7$ *OK*

Let N = 6 in.
$N/t = 6/0.06 = 100 < 210$ *OK*
$N/h = 6/3.821 = 1.57 < 3.5$ *OK*

Table C3.4-1 applies
For end reactions: *Eq.* C3.4-2
For interior reaction: *Eq.* C3.4-3

$k = F_y/33 = 50/33 = 1.515$ (*Eq.* C3.4-21)

$C_1 = (1.22-0.22k) = (1.22-0.22)(1.515) = 0.887$ (*Eq.* C3.4-10)

$C_2 = (1.06-0.06R/t) = (1.06-0.06)(2.083) = 0.935 < 1.0$ *OK* (*Eq.* C3.4-11)

$C_3 = (1.33-0.33k) = (1.33-0.33)(1.515) = 0.830$ (*Eq.* C3.4-12)

$C_4 = (1.15-0.15R/t) \leq 1.0$ but not less than 0.50 (*Eq.* C3.4-13)

$$(1.15-0.15R/t) = (1.15-0.15)(2.083) = 0.838$$

$C_4 = 0.838$
$\theta = 75.96°$

$$C_\theta = 0.7+0.3(\theta/90)^2 = 0.7+0.3(75.96/90)^2 = 0.914$$ (*Eq.* C3.4-20)

For end reaction:

$$P_a = t^2kC_3C_4C_\theta[117-0.15(h/t)][1+0.01(N/t)]$$ (*Eq.* C3.4-2)

$$= (0.060)^2(1.515)(0.830)(0.838)(0.914)[117-0.15(63.68)][1+0.01(100)]$$

$$= 0.745k \text{ per web}$$

Total P_a for section:

$$P_a = 4(0.745k) = 2.98k$$

End reaction is given by

$$R = 0.400wL = (0.400)(0.414)(10) = 1.66k$$

$R = 1.66k < P_a = 2.98k$ *OK*

For interior reaction:

$$P_a = t^2kC_1C_2C_\theta[291-0.40(h/t)][1+0.007(N/t)]$$ (*Eq.* C3.4-4)

$$= (0.060)^2(1.515)(0.887)(0.935)(0.914)[291-0.40(63.68)][1+0.007(100)]$$

$$= 1.87k$$

Total P_a for Section:

$$P_a = 4(1.87k) = 7.48k$$

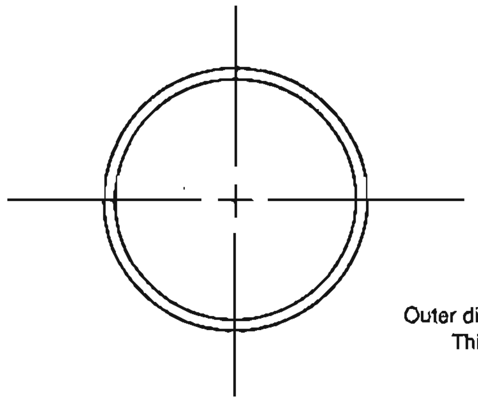
Interior reaction is given by

$$R = 1.01wL = (1.10)(0.414)(10) = 4.55k$$

$R = 4.55k < P_a = 7.48k$ *OK*

EXAMPLE NO. 9

CYLINDRICAL TUBULAR SECTION



Given: Steel: $F_y = 50$ ksi.
Section: Shown in sketch above.

Required: Allowable bending moment

Solution:

Ratio of outside diameter to wall thickness,

$D/t = 8.000/0.125 = 64.00$

$D/t < 0.441\ E/F_y = 0.441(29500/50) = 260.2$ *OK*

$0.070\ E/F_y = 0.070(29500/50) = 41.30$

$0.319\ E/F_y = 0.319(29500/50) = 188.2$

For $0.070\ E/F_y < D/t/0.319\ E/F_y$

$F_n = [0.97 + 0.02(E/F_y)/(D/t)]F_y$ (*Eq.* D5.1-4)

$= [0.97 + 0.02(29500/50)/64.00]\ (50)$

$= 57.22$ ksi

$S_f = \pi[(O.D.)^4 - (I.D.)^4]/32(O.D.)$

$= \pi[(8)^4 - (7.75)^4]/32(8)$

$= 5.995\ in.^3$

$M_n = F_n S_f$ (*Eq.* D5.1-1)

$= (57.22)\ (5.995)$

$= 343.0$ kip-in

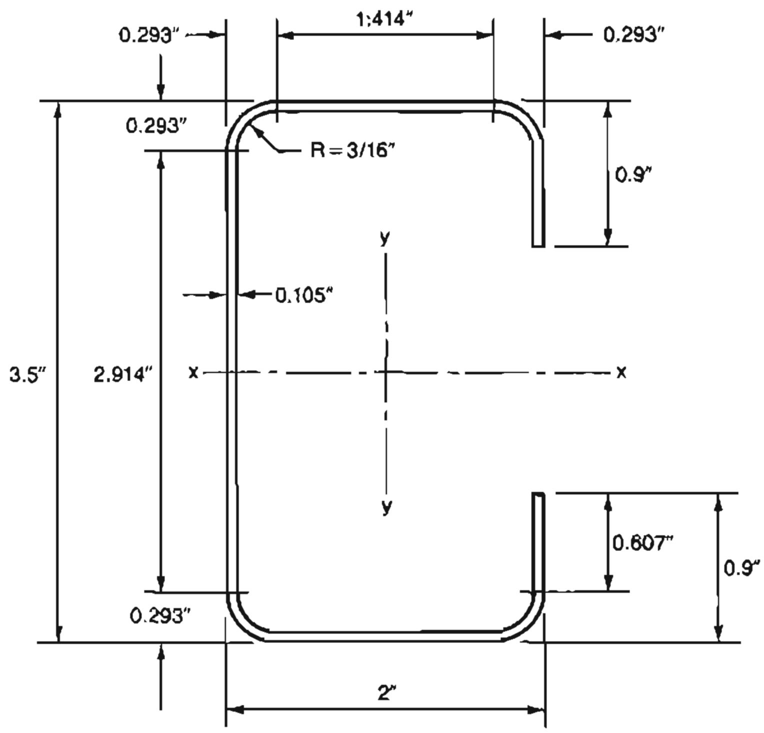
$M_a = M_n/\Omega_f$ (*Eq.* D5.1-1)

$= 343.0/1.67$

$= 205.4$ kip-in

EXAMPLE NO. 10

C-SECTION



Given:
1. Steel: $F_y = 50$ ksi.
2. Section: $3.5 \times 2 \times 0.105$ channel with stiffened flanges.
3. $K_xL_x = K_yL_y = K_tL_t = 6$ ft.

Required: Allowable axial load.

Solution:

1. Basic parameters:

   $r = R + t/2 = 3/16 + 0.105/2 = 0.240$ in.

   from the sketch $a = 2.914$ in., $b = 1.414$ in., $c = 0.607$ in., $\alpha = 1.00$

   (Since the section has lips)

   $\bar{a} \;=\; A' - t = 3.5 - 0.105 = 3.395$ in.

   $\bar{b} \;=\; B' - [t/2 + \alpha t/2] = B' - t = 2 - 0.105 = 1.895$ in.

   $\bar{c} \;=\; \alpha[C' - t/2] = C' - t/2 = 0.9 - 0.105/2 = 0.848$ in.

   $u \;=\; 1.57r = 1.57 \times 0.240 = 0.377$ in.

2. Area:

$$
\begin{aligned}
A &= t[a + 2b + 2u + \alpha(2c + 2u)] = t[a + 2b + 2c + 4u] \\
&= 0.105[2.914 + 2 \times 1.414 + 2 \times 0.607 + 4 \times 0.377] \\
&= 0.889 \text{ in.}^2
\end{aligned}
$$

3. Moment of inertia about x-axis:

$$
\begin{aligned}
I_x &= 2t\{0.0417a^3 + b(a/2 + r)^2 + u(a/2 + 0.637r)^2 + 0.149r^3 \\
&\quad + \alpha[0.0833c^3 + (c/4)(a - c)^2 + u(a/2 + 0.637r)^2 + 0.149r^3]\} \\
&= 2t[0.0417a^3 + b(a/2 + r)^2 + 2u(a/2 + 0.637r)^2 + 0.298r^3 + 0.0833c^3 \\
&\quad + (c/4)(a - c)^2]
\end{aligned}
$$

I_x = $2 \times 0.105[0.0417(2.914)^3 + 1.414(2.914/2 + 0.240)^2$
$+ 2 \times 0.377(2.914/2 + 0.637 \times 0.240)^2 + 0.298(0.240)^3 + 0.0833(0.607)^3$
$+ (0.607/4)(2.914 - 0.607)^2]$
$= 1.657 \text{ in.}^4$

4. Distance from centroid of section to centerline of web:
$\bar{x}$ = $(2t/A)\{b(b/2 + r) + u(0.363r) + \alpha[u(b + 1.637r) + c(b + 2r)]\}$
$= [(2 \times 0.105)/0.889]\{1.414(1.414/2 + 0.240) + 0.377(0.363 \times 0.240)$
$+ 0.3\,77(1.414 + 1.637 \times 0.240) + 0.607(1.414 + 2 \times 0.240)\}$
$= 0.757 \text{ in.}$

5. Moment of inertia about y-axis:
I_y = $2t\{b(b/2 + r)^2 + 0.0833b^3 + 0.356r^3 + \alpha[c(b + 2r)^2 + u(b + 1.637\,r)^2$
$+ 0.149r^3]\} - A(\bar{x})^2$
$= 2 \times 0.105\{1.414(1.414/2 + 0.240)^2 + 0.0833(1.414)^3$
$+ 0.356(0.240)^3 + 0.607(1.414 + 2 \times 0.240)^2$
$+ 0.377(1.414 + 1.637 \times 0.240)^2 + 0.149(0.240)^3\} - 0.889(0.757)^2$
$= 0.524 \text{ in.}^4$

6. Distance from shear center to centerline of web:
m = $(bt/12I_x)[6c(a)^2 + 3b(a)^2 - 8(c)^3]$
$= (1.895 \times 0.105/12 \times 1.657)[6 \times 0.848(3.395)^2$
$+ 3 \times 1.895(3.395)^2 - 8(0.848)^3]$
$= 1.194 \text{ in.}$

7. Distance from centroid to shear center:
x_o = $-(\bar{x} + m) = -(0.757 + 1.194)$
$= -1.951 \text{ in.}$

8. St. Venant torsion constant:
J = $(t^3/3)[a + 2b + 2u + \alpha(2c + 2u)]$
$= [(0.105)^3/3][2.914 + 2 \times 1.414 + 4 \times 0.377 + 2 \times 0.607]$
$= 0.003266 \text{ in.}^4$

9. Warping Constant:
C_w = $(t^2/A)\{[\bar{x}A(\bar{a})^2]/t[(\bar{b})^2/3 + m^2 - m\bar{b}] + (A/3t)[(m)^2(\bar{a})^3 + (\bar{b})^2(\bar{c})^2(2\bar{c} + 3\bar{a})]$
$-(I_x m^2/t)(2\bar{a} + 4\bar{c}) + [m(\bar{c})^2/3]\{8(\bar{b})^2(\bar{c}) + 2m(2\bar{c}(\bar{c} - \bar{a}) + \bar{b}(2\bar{c} - 3\bar{a}))]$
$+ [(\bar{b})^2(\bar{a})^2/6][(3\bar{c} + \bar{b})(4\bar{c} + \bar{a}) - 6(\bar{c})^2] - [m^2(\bar{a})^4]/4\}$
C_w = $[(0.105)^2/0.889]\{[0.757 \times 0.889 \times (3.395)^2]/0.105[(1.895)^2/3$
$+ (1.194)^2 - 1.194 \times 1.895] + 0.889/3 \times 0.105[(1.194)^2(3.395)^3$
$+ (1.895)^2(0.848)^2(2 \times 0.848 + 3 \times 3.395)]$
$-[1.657 \times (1.194)^2]/0.105(2 \times 3.395 + 4 \times 0.848)$
$+ [1.194 \times (0.848)^2]/3[8(1.895)^2(0.848)$
$+ 2 \times 1.194(2 \times 0.848(0.848 - 3.395) + 1.895(2 \times 0.848 - 3 \times 3.395))]$
$+ (1.895)^2(3.395)^2[3 \times 0.848 + 1.895)$
$(4 \times 0.848 + 3.395) - 6(0.848)^2] - [(1.194)^2(3.395)^4]/4\}$
$= 2.050 \text{ in.}^6$

10. Radii of gyration:
r_x = $\sqrt{(I_x/A)} = \sqrt{(1.657/0.889)} = 1.365 \text{ in.}$
r_y = $\sqrt{(I_y/A)} = \sqrt{(0.524/0.889)} = 0.768 \text{ in.}$
$(K_yL_y)/r_y = (6 \times 12)/0.768 = 93.75 < 200$
r_o^2 = $r_x^2 + r_y^2 + x_o^2 = (1.365)^2 + (0.768)^2 + (-1.951)^2 = 6.259 \text{ in.}^2$

11. Torsional-flexural constant:
β = $1 - (x_o/r_o)^2$ (*Eq.* C4.2-3)
$= 1 - (-1.951)^2/6.259$
$= 0.392$

12. Determination of F_e:
For this singly symmetric section (x-axis is the axis of symmetry), F_e shall be taken as the smaller of either (*Eq.* C4.1-1) or (*Eq.* C4.2-1):

$(F_e)_1 = (\pi^2 E)/(K_y L_y/r_y)^2$ (*Eq.* C4.1-1)
$= (\pi^2 \times 29500)/(6 \times 12/0.768)^2 = 33.13 \times$ ksi.
$\sigma_{ex} = (\pi^2 E)/(k_x L_x/r_x)^2$ (*Eq.* C3.1.2-12)
$= (\pi^2 \times 29500)/(6 \times 12/1.365)^2 = 104.65$ ksi.
$\sigma_t = 1/Ar_o^2[GJ + (\pi^2 EC_w)/(K_t L_t)^2]$ (*Eq.* C3.1.2-14)
$= 1/(0.889 \times 6.259)[11300 \times 0.003266 + (\pi^2 \times 29500 \times 2.050)/(6 \times 12)^2]$
$\sigma_t = 27.32$ ksi
$(F_e)_2 = (1/2\beta)[(\sigma_{ex} + \sigma_t) - \sqrt{(\sigma_{ex} + \sigma_t)^2 - 4\beta\sigma_{ex}\sigma_t}]$ (*Eq.* C4.2-1)
$= 1/2 \times 0.392\{(104.65 + 27.32)$
$- \sqrt{(104.65 + 27.32)^2 - 4 \times 0.392 \times 104.65 \times 27.32}\}$
$= 23.27$ ksi.
$F_e = 23.27$ ksi.

13. Determination of F_n:
$f_y/2 = 50/2 = 25.00$ ksi.
For $F_e < F_y/2$
$F_n = F_e$ (*Eq.* C4-4)
$= 23.27$ ksi.

14. Determination of A_e:
Flanges: $d = 0.607$ in.
$I_s = d^3t/12 = (0.607)^3(0.105)/12$
$= 0.001957$ in.^{4}
$D = 0.9$ in.
$w = 1.414$ in.
$D/w = 0.9/1.414 = 0.636 < 0.8$
$S = 1.28\sqrt{(E/f)}$, $f = F_n$ (*Eq.* B4-1)
$= 1.28\sqrt{(29500/23.27)} = 45.57$
$w/t = 1.414/0.105 = 13.47 < S/3 = 15.19$ (*Eq.* B4.2-1)
$I_a = 0$ (no edge stiffener needed) (*Eq.* B4.2-2)
$b = w$ (*Eq.* 4.2-3)
$= 1.414$ in. (flanges fully effective)
$w/t = 13.47 < 90$ (Section B1.1-(a)-(1))

Web: $w = 2.914$ in., $k = 4.00$
$\lambda = (1.052/\sqrt{k})\,(w/t)\sqrt{(f/E)}$, $f = F_n$ (*Eq.* B2.1-4)
$= (1.052/\sqrt{4})\,(2.914/0.105)\sqrt{(23.27/29500)}$
$= 0.410 < 0.673$
$b = w$ (*Eq.* B2.1-1)
$= 2.914$ in. (web fully effective)
$w/t = 2.914/0.105 = 27.75 < 500$ (Section B1.1-(a)-(2))

Lips: $d = 0.607$ in.
$k = 0.43$ (unstiffened compression element)
$d_s = d_s'$ (*Eq.* B4.3-4)
$\lambda = (1.052/\sqrt{0.43})\,(0.607/0.105)\sqrt{(23.27/29500)}$
$= 0.260 < 0.673$
$d_s' = d = 0.607$ in., $d_s = 0.607$ in.
$d/t = 5.78 < 60$ (Section B1.1-(a)-(3))

Since flanges, web, and lips are fully effective
$A_e = A = 0.889$ in.^{2}

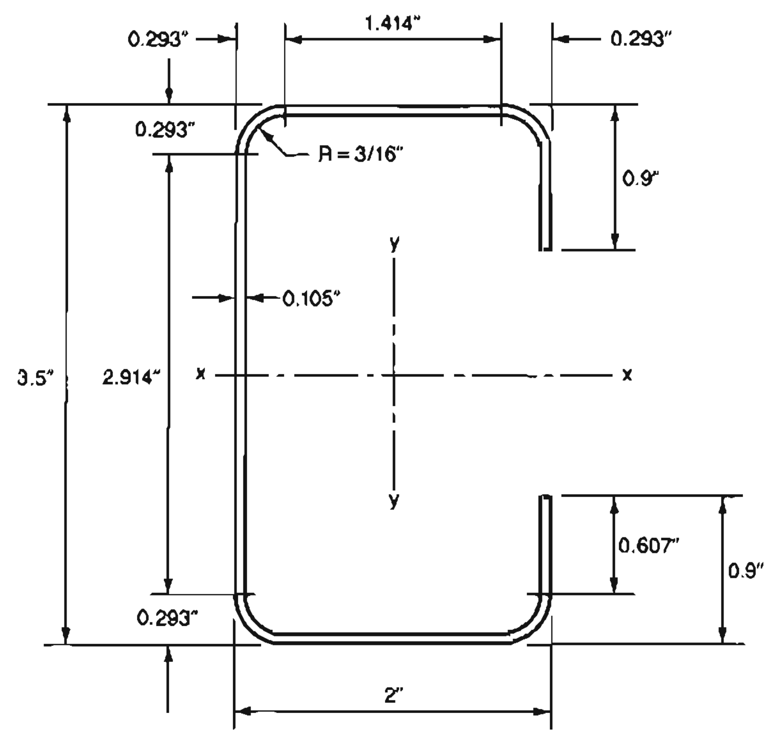
15. Determination of ρ_a:
$\rho_n = A_e F_n$ (*Eq.* C4-2)
$= 0.889 \times 23.27$
$= 20.69$ kips

$\Omega_c = 1.92$

$\rho_a = \rho_n/\Omega_c$ (*Eq.* C4-1)
$= 20.69/1.92$
$= 10.78$ kips

EXAMPLE NO. 11

C-SECTION w/HOLES



Given:
1. Steel: $F_y = 50$ ksi.
2. Section: $3.5 \times 2 \times 0.105$ channel with stiffened flanges.
3. $K_xL_x = K_yL_y = K_tL_t = 6$ ft.
4. Web is perforated with holes for bolts of ½-in. diameter (in standard hole) at 4 in. spacing along the height of the column.

Required: Allowable axial load.

Solution:

1. Basic parameters:

$r = R + t/2 = 3/16 + 0.105/2 = 0.240$ in. from the sketch $a = 2.914$ in., $b = 1.414$ in., $c = 0.607$ in., $\alpha = 1.00$ (since the section has lips)

$a = A' - t = 3.5 - 0.105 = 3.395$ in.

$b = B' - t = 2 - 0.105 = 1.895$ in.

$c = C' - t/2 = 0.9 - 0.105/2 = 0.848$ in.

$u = 1.57r = 1.57 \times 0.240 = 0.377$ in.

2. Area:

$$A = t[a + 2b + 2c + 4u]$$
$$= 0.105[2.914 + 2 \times 1.414 + 2 \times 0.607 + 4 \times 0.377]$$
$$= 0.889 \text{ in.}^2$$

3. Moment of inertia about x-axis:

$$I_x = 2t[0.0417a^3 + b(a/2 + r)^2 + 2u(a/2 + 0.637r)^2 + 0.298r^3 + 0.0833c^3 + (c/4)(a - c)^2]$$
$$I_x = 2 \times 0.105[0.0417(2.914)^3 + 1.414(2.914/2 + 0.240)^2$$
$$+ 2 \times 0.377(2.914/2 + 0.637 \times 0.240)^2 + 0.298(0.240)^3 + 0.0833(0.607)^3$$
$$+ 0.607/4(2.914 - 0.607)^2]$$
$$= 1.657 \text{ in.}^4$$

4. Distance from centroid of section to centerline of web:

$x = (2t/A)[b(b/2+r)+u(0.363r)+u(b+1/637r)+c(b+2r)]$

$= 2\times 0.105/0.889[1.414(1.414/2+0.240)+0.377(0.363\times 0.240)$
$+0.377$
$(1.414+1.637\times 0.240)+0.607(1.414+2\times 0.240)]$

$= 0.757$ in.

5. Moment of inertia about y-axis:

$I_y = 2t[b(b/2+r)^2+0.0833\,b^3+0.505r^3+c(b+2r)^2$
$+u(b+1.637r)^2]-A(\bar{x})^2$

$= 2\times 0.105[1.414(1.414/2+0.240)^2+0.0833(1.414)^3+0.505(0.240)^3+$
$0.607(1.414+2\times 0.240)^2+0.377(1.414+1.637\times 0.240)^2]-0.889(0.757)^2$

$= 0.524$ in.^{4}

6. Distance from shear center to centerline of web:

$m = (\bar{b}t/12I_x)[6\bar{c}(\bar{a})^2+3\bar{b}(\bar{a})^2-8(\bar{c})^3]$

$= (1.895\times 0.105)/(12\times 1.657)[6\times 0.848(3.395)^2+3\times 1.895(3.395)^2$
$-8(0.848)^3]$

$= 1.194$ in.

7. Distance from centroid to shear center:

$x_o = \leqslant(\bar{x}+m) = -(0.757+1.194)$

$= -1.951$ in.

8. St. Venant torsion constant:

$J = (t^3/3)[a+2b+2c+4u]$

$= [(0.105)^3/3][2.914+2\times 1.414+2\times 0.607+4\times 0.3077]$

$= 0.003266$ in.^{4}

9. Warping Constant:

$C_w = (t^2/A)\{[\bar{x}A(\bar{a})^2]/t[(\bar{b})^2/3+m^2-m\bar{b}]+(A/3t)[(m)^2(\bar{a})^3+(\bar{b})^2(\bar{c})^2(2\bar{c}+3\bar{a})]$
$-(I_xm^2/t)(2\bar{a}+4\bar{c})+[m(\bar{c})^2/3][8(\bar{b})^2(\bar{c})+2m[2\bar{c}(\bar{c}-\bar{a})+\bar{b}(2\bar{c}-3\bar{a})]$
$+[(\bar{b})^2(\bar{a})^2/6][(3\bar{c}+\bar{b})(4\bar{c}+\bar{a})-6(\bar{c})^2]-m^2(\bar{a})^4/4\}$

$C_w = [(0.105)^2/0.889]\{0.757\times 0.889\times(3.395)^2/0.105]$
$[(1.895)^2/3+(1.194)^2-1.194\times 1.895]$
$+0.889/3\times 0.105[(1.194)^2(3.395)^3+(1.895)^2(0.848)^2(2\times 0.848+3\times 3.395)]$
$-1.657(1.194)^2/0.105(2\times 3.395+4\times 0.848)$
$+2\times 1.194\{2\times 0.848(0.848-3.395)+1.895(2\times 0.848-3\times 3.395)]\}$
$+[(1.895)^2(3.395)^2]/6[(3\times 0.848+1.895)(4\times 0.848+3.395)$
$-6(0.848)^2]-[(1.194)^2(3.395)^4]/4\}$

$= 2.050$ in.^{6}

10. Radii of gyration:

$r_x = \sqrt{(I_x/A)} = \sqrt{(1.657/0.889)} = 1.365$ in.

$r_y = \sqrt{(I_y/A)} = \sqrt{(0.524/0.889)} = 0.768$ in.,

$(K_yL_y)/r_y = (6\times 12)/0.768 = 93.75 < 200$

$r_o^2 = r_x^2+r_y^2+x_o^2 = (1.365)^2+(0.768)^2+(-1.951)^2$

$= 6.259$ in.^{2}

11. Torsional-flexural constant:

$\beta = 1-(x_o/r_o)^2$ *(Eq. C.4.2-3)*

$= 1-(-1.951)^2/6.259$

$= 0.392$

12. Determination of F_e:

For this singly symmetric section (x-axis is the axis of symmetry), F_e shall be taken as the smaller of either (Eq. C4.1-1) or (Eq. C4.2-1):

$(F_e)_1 = (\pi^2E)/(K_yL_y/r_y)^2$ (Eq. C4.1-1)

$= (\pi^2\times 29500)/(93.75)^2 = 33.13$ ksi

$\sigma_{ex} = (\pi^2E)/(K_xL_x/r_x)^2$ *(Eq. C3.1.2-12)*

$= (\pi^2\times 29500)/(6\times 12/1.365)^2 = 104.65$ ksi

$\sigma_t = 1/Ar_o^2[GJ + (\pi^2 EC_w)/(K_t L_t)^2]$ (*Eq.* C3.1.2-14)

$= 1/(0.889 \times 6.259)[11300 \times 0.003266 + (\pi^2 \times 29500 \times 2.050)/(6 \times 12)^2]$

$\sigma_t = 27.32$ ksi

$(F_e)_2 = [1/2\beta]\{(\sigma_{ex} + \sigma_t) - \sqrt{[(\sigma_{ex} + \sigma_t)^2 - 4\beta\sigma_{ex}\sigma_t]}\}$ (*Eq.* C4.2-1)

$= (1/2 \times 0.392)\{(104.65 + 27.32)$
$- \sqrt{[(104.65 + 27.32)^2 - 4 \times 0.392 \times 104.65 \times 27.32]}\}$

$= 23.27$ ksi

$F_e = 23.27$ ksi

13. Determination of F_n:

$F_y/2 = 50/2 = 25.00$ ksi

$F_e < F_y/2$

$F_n = F_e$ (*Eq.* C4-4)

$= 23.27$ ksi

14. Determination of A_e:

Flanges: $d = 0.607$ in.

$I_s = d^3t/12 = (0.607)^4(0.105)/12$

$= 0.001957$ in.^{4}

$D = 0.9$ in.

$w = 1.414$ in.

$D/w = 0.9/1.414 = 0.636 < 0.8$

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

$= 1.28\sqrt{(29500/23.27)} = 45.57$

$w/t = 1.414/0.105 = 13.47 < (S/3) = 15.19$ (*Eq.* B4.2-1)

$I_a = 0$ (no edge stiffener needed) (*Eq.* B4.2-2)

$b = w$ (*Eq.* B4.2-3)

$= 1.414$ in. (flanges fully effective)

$w/t = 13.47 < 90$ (Section B1.1-(a)-(1))

Web: $w = 2.914$ in., $k = 4.00$

$\lambda = (1.052/\sqrt{k})\ (W/t)\sqrt{(f/E)},\ F = F_n$ (*Eq. B2.1-4)*

$= (1.052/\sqrt{4})\ (2.914/0.105)\sqrt{(23.27/29500)}$

$= 0.410$

$d_h = d + 1/16$ (Table E3)

$d =$ diameter of bolt $= 0.5$ in.

$d_h = 0.5 + 1/16$

$= 0.563$ in.

Number of holes in the effective length = 17

$(17 \times 0.563)/(6 \times 12) = 0.133 > 0.015$ then A_e must be determined with holes accounted for. (Section C4-(a))

d_h/w

$= 0.563/2.914 = 0.193 < 0.50$

$w/t = 2.914/0.105$

$= 27.75 < 70$

$0.5w = 0.5 \times 2.914 = 1.457$ in.

$3d_h = 3 \times 0.563 = 1.689$ in.

spacing of holes = 4 in. greater than both 0.5w and $3d_h$

Effective width, b, shall then be the smaller of (*Eq.* B2.2-1) and (*Eq.* B2.2-2).

$w - d_h$

$= 2.914 - 0.563$ (*Eq.* B2.2-1)

$= 2.351$ in.

$w[1 - (0.22/\lambda) - (0.8d_h/w)]/\lambda$

$= 2.914[1 - (0.22/0.410) - (0.8 \times 0.563/2.914)]/0.410 = 2.195$ in. (*Eq.* B2.2-2)

$b = 2.195$ in.

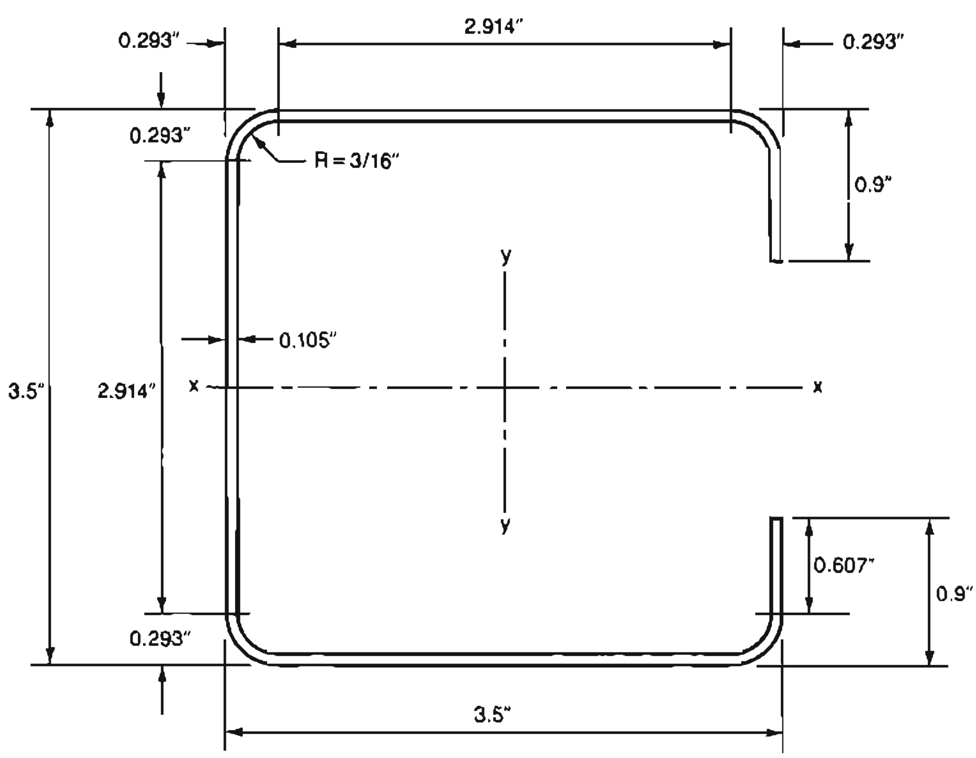
$w/t = 27.75 < 500$ (Section B1.1-(a)-(2))

Lips: $d = w = 0.607$ in.
$k = 0.43$ (unstiffened compression element)
$\lambda = (1.052/\sqrt{0.43})(0.607/0.150)\sqrt{(23.27/29500)} = 0.260 < 0.673$
$d_s' = d$
$= 0.607$ in.
$d_s = d_s'$ (*Eq.* B4.2-4)
$= 0.607$ in. (No reduction in lips area)
$w/t = 0.607/0.105 = 5.78 < 60$ (Section B1.1-(a)-(3))
$A_e = A - t(w-b)_{web}$
$= 0.889 - 0.105(2.914 - 2.195)$
$= 0.814$ in.^{2}

15. Determination of ρ_a:
$P = A_eF_n$ (*Eq.* C4-2)
$= 0.814 \times 23.27 = 18.94$ kips
$\Omega = 1.92$
$P_a = P_n/\Omega_c$ (*Eq.* C4-1)
$= 18.94/1.92$
$= 9.86$ kips

EXAMPLE NO. 12

C-SECTION w/WIDE FLANGE



Given: 1. Steel: $F_y = 50$ ksi.
2. Section: $4 \times 4 \times 0.065$ Square Tube.
3. $K_xL_x = K_yL_y = 6$ ft.

Required: Allowable Axial Load.

Solution:

1. Basic parameters:
$r = R + t/2 = 3/16 + 0.105/2 = 0.240$ in.
from the sketch $a = 2.914$ in., $b = 1.414$ in., $c = 0.607$ in., Equations will be simplified for $\alpha = 1.00$ (Since the section has lips)
$\bar{a} = A' - t = 3.5 - 0.105 = 3.395$ in.
$\bar{b} = B' - t = 3.5 - 0.105 = 3.395$ in.
$\bar{c} = C' - t/2 = 0.9 - 0.105/2 = 0.848$ in.
$u = 1.57r = 1.57 \times 0.240 = 0.377$ in.

2. Area:
$$A = t[a + 2b + 2u + 4c]$$
$$= 0.105[2.914 + 2 \times 2.914 + 2 \times 0.607 + 4 \times 0.377]$$
$$= 1.204 \text{ in.}^2$$

3. Moment of inertia about x-axis:
$$I_x = 2t[0.0417a^3 + b(a/2 + r)^2 + 2u(a/2 + 0.637r)^2 + 0.298r^3 + 0.0833c^3 + (c/4)(a - c)^2]$$
$$I_x = 2 \times 0.105[0.0417(2.914)^3 + 2.914(2.914/2 + 0.240)^2 + 2 \times 0.377(2.914/2 + 0.637 \times 0.240)^2 + 0.298(0.240)^3 + 0.0833(0.607)^3 + (0.607/4)(2.914 - 0.607)^2]$$
$$= 2.564 \text{ in.}^4$$

4. Distance from centroid of section to centerline of web:
$$\bar{x} = 2t/A[b(b/2 + r) + u(0.363r) + u(b + 1.637r) + c(b + 2r)]$$
$$= [(2 \times 0.105)/1.204][2.914(2.914/2 + 0.240) + 0.377(0.363 \times 0.240) + 0.377(2.914 + 1.637 \times 0.240) + 0.607(2.914 + 22 \times 0.240)]$$
$$= 1.445 \text{ in.}$$

5. Moment of inertia about y-axis:
$$I_y = 2t[b(b/2 + r)^2 + 0.0833b^3 + 0.505r^3 + c(b + 2r)^2 + u(b + 1.637r)^2] - A(\bar{x})^2$$
$$= 2 \times 0.105[2.914(2.914/2 + 0.240)^2 + 0.0833(2.914)^3 + 0.505(0.240)^3 + 0.607(2.914 + 2 \times 0.240)^2 + 0.377(2.914 + 1.637 \times 0.240)^2] - 1.204(1.445)^2$$
$$= 2.017 \text{ in.}^4$$

6. Distance from shear center to centerline of web:
$$m = (\bar{b}t/12I_x)[6\bar{c}(\bar{a})^2 + 3\bar{b}(\bar{a})^2 - 8(\bar{c})^3]$$
$$= [(3.395 \times 0.105)/(12 \times 2.564)][6 \times 0.848(3.395)^2 + 3 \times 3.395(3.395)^2 - 8(0.848)^3]$$
$$= 1.983 \text{ in.}$$

7. Distance from centroid to shear center:
$$x_o = -(\bar{x} + m) = -(1.445 + 1.983)$$
$$= -3.428 \text{ in.}$$

8. St. Venant torsion constant:
$$J = (t^3/3)[a + 2b + 2c + 4u]$$
$$= [(0.105)^3/3][2.914 + 2 \times 2.914 + 2 \times 0.607 + 4 \times 0.377]$$
$$= 0.004424 \text{ in.}^4$$

9. Warping constant:
$$C_w = (t^2/A)\{\bar{x}A(\bar{a})^2/t[(\bar{b})^2/3 + m^2 - m\bar{b}] + A/3t[(m)^2(\bar{a})^3 + (\bar{b})^2(\bar{c})^2(2\bar{c} + 3\bar{a})] - I_x m^2/t(2\bar{a} + 4\bar{c}) + m(\bar{c})^2/3\{8(\bar{b})^2(\bar{c}) + 2m[2\bar{c}(\bar{c} - \bar{a}) + \bar{b}(2\bar{c} - 3\bar{a})]\} + (\bar{b})^2(\bar{a})^2/6[(3\bar{c} + \bar{b})(4\bar{c} + \bar{a}) - 6(\bar{c})^2] - m^2(\bar{a})^4/4\}$$
$$C_w = [(0.105)^2/1.204]\{1.445 \times 1.204 \times (3.395)^2/0.105][(3.395)^2/3 + (1.983)^2 - 1.983 \times 3.395] + 1.204/3 \times 0.105[(1.983)^2(3.395)^3 + (3.395)^2(0.848)^2(2 \times 0.848 + 3 \times 3.395)] - 2.564 \times (1.983)^2/0.105(2 \times 3.395 + 4 \times 0.848) + 1.983(0.848)^2/3[8(3.395)^2(0.848) + 2 \times 1.983(2 \times 0.848(0.848 - 3.395) + 3.395(2 \times 0.848 - 3 \times 3.395))] + [(3.395)^2(3.395)^2]/6[(3 \times 0.848 + 3.395)(4 \times 0.848 + 3.395) - 6(0.848)^2] - [(1.193)^2(3.395)^4]/4\}$$
$$= 7.572 \text{ in.}^6$$

10. Radii of gyration:

$r_x = \sqrt{I_x/A} = \sqrt{2.564/1.204} = 1.459$ in.

$r_y = \sqrt{I_y/A} = \sqrt{2.017/1.204} = 1.294$ in., $K_yL_y/r_y = 6 \times 12/1.294 = 55.64 < 200$

$r_o = r_x^2 + r_y^2 + x_o^2 = (.459)^2 + (1.294)^2 + (-3.428)^2$

$= 15.554$ in.^{2}

11. Torsional-flexural constant:

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

$= 1-[(-3.428)^2/15.554]$

$= 0.244$

12. Determination of F_e:

For this singly symmetric section (x-axis is the axis of symmetry), F_e shall be taken as the smaller of either (*Eq.* C4.1-1) or (*Eq.* C4.2-1):

$F_{e1} = \pi^2E/(K_yL_y/r_y)^2$ (*Eq.* C4.1-1)

$= \pi^2 \times 29500/(55.64)^2$

$= 94.05$ ksi.

$\sigma_{ex} = \pi^2E/(K_xL_x/r_x)^2$ (*Eq.* C3.1.2-12)

$= \pi^2 \times 29500/(6 \times 12/1.459)^2$

$= 119.55$ ksi.

$\sigma_t = 1/Ar_o^2\{GJ + [\pi^2EC_w/(K_tL_t)^2]\}$ (*Eq.* C3.1.2-14)

$= 1/(1.204 \times 15.554)\{11300 \times 0.004424 + [\pi^2 29500 \times 7.572/(6 \times 12)^2]\}$

$= 25.38$ ksi

$F_{e2} = 1/2\beta\,[(\sigma_{ex} + \sigma_t) - \sqrt{(\sigma_{ex} + \sigma_t)^2 - 4\beta\sigma_{ex}\sigma_t}\,]$ (*Eq.* C4.2-1)

$= 1/(2 \times 0.244)\,[(119.55 + 25.38) - \sqrt{(119.55 + 25.38)^2 - 4 \times 0.244 \times 119.55 \times 25.38}\,]$

$= 21.73$ ksi.

$F_e = 21.73$ ksi.

13. Determination of F_n:

$F_y/2 = 50/2 = 25.00$ ksi

For $F_e < F_y/2$

$F_n = F_e$ (*Eq.* C4-4)

$= 21.73$ ksi

14. Determination of A_e:

Flanges: $d = 0.607$ in.

$I_s = d^3t/12 = (0.607)^3(0.105)/12$

$= 0.001957$ in.^{4}

$D = 0.9$ in.

$w = 2.914$ in. (for flange)

$D/w = 0.9/2.914 = 0.309 < 0.8$

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

$= 1.28\sqrt{29500/21.73} = 47.16$, $S/3 = 15.72$

$w/t = 2.914/0.105 = 27.75$

$S/3 < w/t < S$

$I_a = 399t^4\{[(w/t)/S] - 0.33\}^3$ (*Eq.* B4.2-6)

$= 399(0.105)^4\{[27.75/47.16] - 0.33\}^3$

$= 0.000837$ in.$^4 < I_s = 0.001957$ in.^{4}

$C_1 = 2-(I_s/I_a) \geq 1.0$ (*Eq.* B4.2-7)

$2-(I_s/I_a) = 2-(0.001957/0.000837) = -0.34 < 1.0$

$C_1 = 1.0$

$C_2 = I_s/I_a \leq 1.0$ (*Eq.* B4.2-8)

$I_s/I_a = (0.001957/0.000837) = 2.34 > 1.0$

$C_2 = 1.0$

$0.25 < D/w = 0.309 < 0.8$

$k = [4.82 - 5(D/w)](I_s/I_a)^n + 0.43 \leq 5.25 - 5(D/w)$ (*Eq.* B4.2-9)

$n = 1/2$

$[4.82 - 5(D/w)](I_s/I_a)^n + 0.43 = [4.82 - 5 \times 0.309](0.001957/0.000837)^{1/2} + 0.43$

$= 5.438$

$5.25 - 5(D/w) = 5.25 - 5(0.309)$

$= 3.705$

	k	$= 3.705$	
	λ	$= (1.052/\sqrt{k})\,(w/t)\,\sqrt{f/E}\,,\quad f = F_n$	(*Eq.* B2.1-4)
		$= (1.052/\sqrt{3.705})\,(27.75)\,\sqrt{21.73/29500} = 0.412 < 0.673$	
	b	$= w$	(*Eq.* B2.1-1)
		$= 2.914$ in. (flanges fully effective)	
	w/t	$= 27.75 < 90$ [Section B1.1-(a)-(1)]	
Web:	w	$= 2.914,\ k = 4.00$	
	λ	$= (1.052/\sqrt{4})\,(2.914/0.105)\,\sqrt{21.73/29500} = 0.396 < 0.673$	
	b	$= w = 2.914$ in. (web fully effective)	
	w/t	$= 2.914/0.105 = 27.75 < 500$ [Section B1.1-(a)-(2)]	
Lips:	d	$= 0.607$ in.	
	k	$= 0.43$ (unstiffened compression element)	
	λ	$= (1.052/\sqrt{0.43})\,(0.607/0.105)\,\sqrt{21.73/29500} = 0.252 < 0.673$	
	d'_s	$= d = 0.607$ in.	
	d_s	$= d'_s(I_s/I_a) \le d'_s$	(*Eq.* B4.2-11)
	$d'_s(I_s/I_a)$	$= 0.607(2.34) = 1.420 > d'_s = 0.607$ in.	
	d_s	$= 0.607$ in. (Lip fully effective in computing the overall effective area)	
	d/t	$= 5.78 < 60$ [Section B1.1-(a)-(3)]	

Since flanges, web, and lips are fully effective

$A_e = A = 1.204\ \text{in.}^2$

15. Determination of P_a:

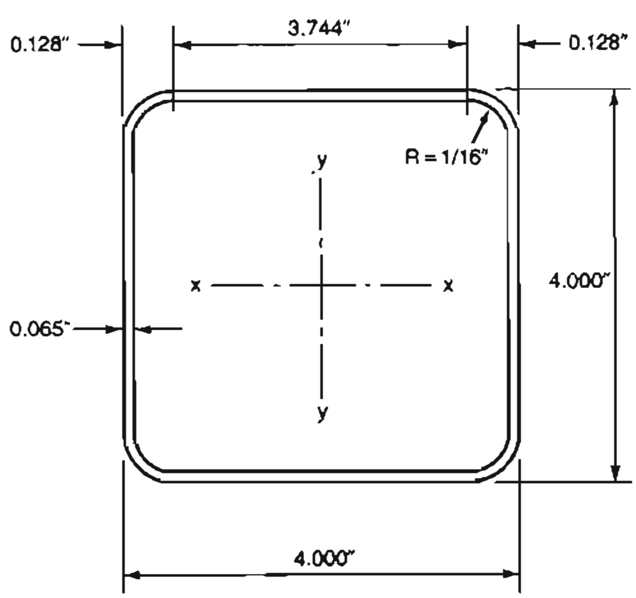
$P_n = A_eF_n$ (*Eq.* C4-2)
$= 1.204 \times 21.73$
$= 26.16$ kips

$\Omega_c = 1.92$

$P_a = P_n/\Omega_c$ (*Eq.* C4-1)
$= 26.16/1.92$
$= 13.63$ kips

EXAMPLE NO. 13

TUBULAR SECTION–SQUARE



Given:
1. Steel: $F_y = 50$ ksi.
2. Section: $4 \times 4 \times 0.065$ Square Tube.
3. $K_xL_x = K_yL_y = 10$ ft.

Required: Allowable Axial Load.

Solution:

1. Properties of 90° Corners:
   $r = R + t/2 = 1/16 + 0.065/2 = 0.095$ in.
   Length of arc, $u = 1.57r = 1.57 \times 0.095 = 0.149$ in.
   Distance of c.g. from center of radius, $c = 0.637r = 0.637 \times 0.095 = 0.061$ in.

   $I_x = I_y = I$ (doubly symmetric section)

Element	L (in.)	y Distance to Center of Section (in.)	Ly^2 (in.^{3})	I'_1 About Own Axis (in.^{3})
Flanges	$2 \times 3.744 = 7.488$	$2 - 0.065/2 = 1.968$	29.001	—
Corners	$4 \times 0.149 = 0.596$	$(3.744/2) + 0.061 = 1.933$	2.227	—
Web	$2 \times 3.744 = 7.488$	—	—	8.747
Sum	15.572		31.228	8.747

$w/t = 3.744/0.065 = 57.60 < 500$ [Section B1.1-(a)-(2)]
$A = Lt = 15.572 \times 0.065 = 1.012$ in.^{2}
$I' = Ly^2 + I'_1 = 31.228 + 8.747 = 39.975$ in.^{3}
$I = I'_t = 39.975 \times 0.065 = 2.598$ in.^{4}
$r = \sqrt{I/A} = \sqrt{2.598/1.012} = 1.602$ in.
$KL/r = 10 \times 12/1.602 = 74.91 < 200$ [Section C4-(d)]

2. Since the square tube is a doubly symmetric closed section, provisions of Section C4.1 apply, i.e., section is not subjected to torsional flexural buckling.

   $F_e = \pi^2 E/(KL/r)^2$ (*Eq.* C4.1-1)
   $= \pi^2 \times 29500/(74.91)^2 = 51.89$ ksi

   $F_y/2 = 50/2 = 25.00$ ksi

   For $F_e > F_y/2$:
   $F_n = F_y[1 - F_y/4F_e]$ (*Eq.* C4-3)
   $= 50[1 - 50/(4 \times 51.89)] = 37.96$ ksi

3. $\lambda = (1.052/\sqrt{k})\,(w/t)\,\sqrt{f/E}\,, \quad f = F_n$ (*Eq.* B2.1-4)
   $= (1.052/\sqrt{4})\,(3.744/0.065)\,\sqrt{37.96/29500} = 1.087 > 0.673$
   (Section not fully effective)

   $\rho = (1 - 0.22/\lambda)/\lambda$ (*Eq.* B2.1-3)
   $= (1 - 0.22/1.087)/1.087 = 0.734$

   $b = \rho w$ (*Eq.* B2.1-2)
   $= 0.734 \times 3.744 = 2.748$ in.

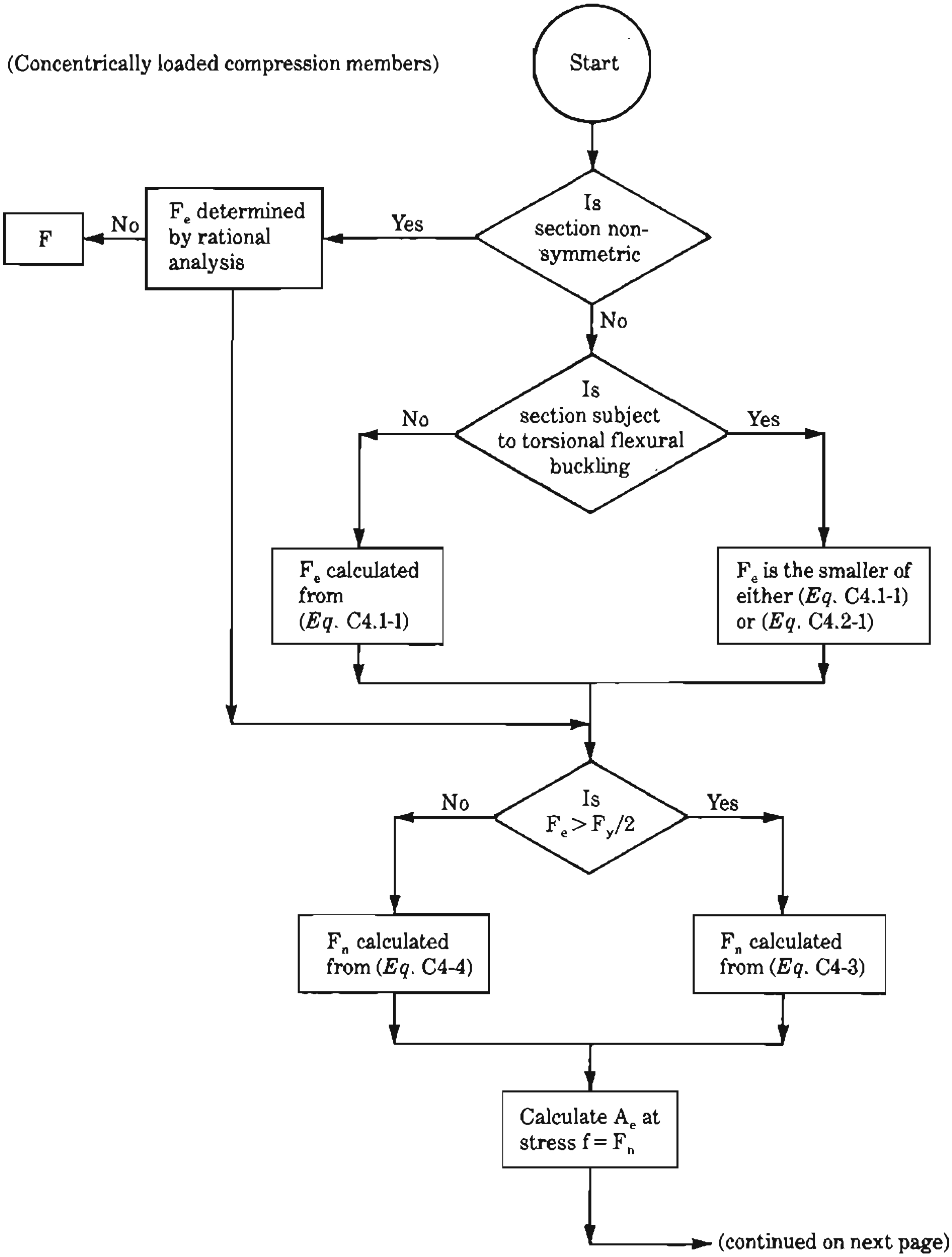
   $A_e = A - 4(w - b)t$
   $= 1.012 - 4(3.744 - 2.748) \times 0.065$
   $= 0.753$ in.^{2}

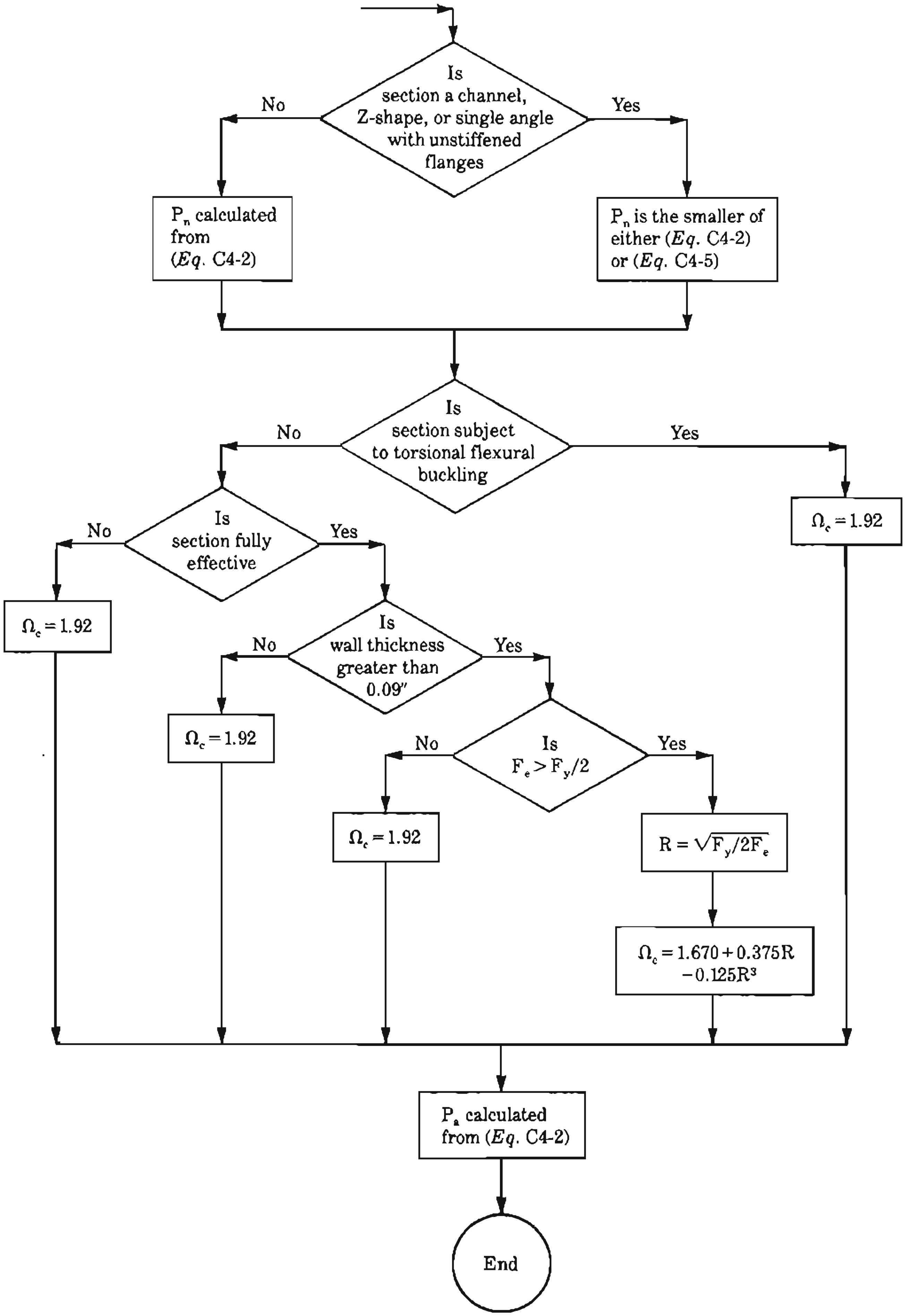
4. $P_n = A_e F_n$ (*Eq.* C4-2)
   $= 0.753 \times 37.96$
   $= 28.58$ kips

   $\Omega_c = 1.92$ [Section C4-(a)]

   $P_a = P_n/\Omega_c$ (*Eq.* C4-1)
   $= 28.58/1.92$
   $= 14.89$ kips

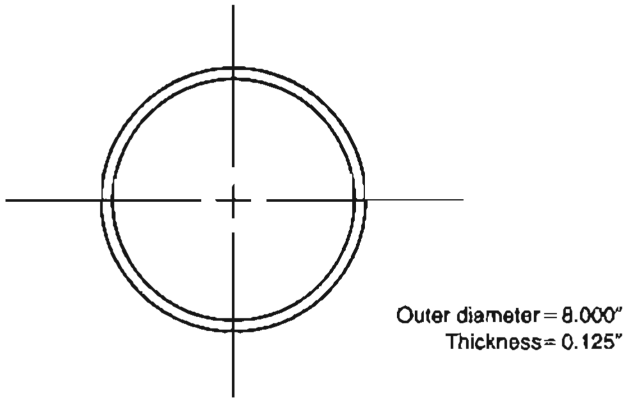
(Concentrically loaded compression members)




Is section a channel, Z-shape, or single angle with unstiffened flanges
No
Yes
P_n calculated from (*Eq.* C4-2)
P_n is the smaller of either (*Eq.* C4-2) or (*Eq.* C4-5)
Is section subject to torsional flexural buckling
No
Yes
$\Omega_c = 1.92$
Is section fully effective
No
Yes
$\Omega_c = 1.92$
Is wall thickness greater than 0.09″
No
Yes
$\Omega_c = 1.92$
Is $F_e > F_y/2$
No
Yes
$\Omega_c = 1.92$
$R = \sqrt{F_y/2F_e}$
$\Omega_c = 1.670 + 0.375R - 0.125R^3$
P_a calculated from (*Eq.* C4-2)
End

EXAMPLE NO. 14

TUBULAR SECTION–ROUND



Given:
1. Steel: $F_y = 50$ ksi.
2. Section: Shown in sketch above.
3. Height: L = 10'-0", simply supported at each end.

Required: Allowable axial load.

Solution:

Ratio of outside diameter to wall thickness,

$D/t = 8.000/0.125 = 64.00$

$D/t < 0.441\ E/F_y = 0.441(29500/50) = 260.2$ *OK*

$F_e = \pi^2 E/(KL/r)^2$ (*Eq.* C4.1-1)

$I = (1/4)\pi[(O.R.)^4 - (I.R.)^4]$
$= (1/4)\pi[(4)^4 - (3.875)^4]$
$= 23.98$ in.^{4}

$A = (1/4)\pi[(O.D.)^2 - (I.D.)^2]$
$= (1/4)\pi[(8)^2 - (7.75)^2]$
$= 3.093$ in.^{2}

$r = \sqrt{I/A}$
$= \sqrt{23.98/3.093}$
$= 2.784$ in.

$F_e = \pi^2(29500)/[10(12)/2.784]^2$
$= 156.71$ ksi

Since $F_e > F_y/2$

$F_n = F_y[1 - F_y/4F_e]$ (*Eq.* C6.2-3)
$= 50[1 - 50/(4 \times 156.71)]$
$= 46.01$ ksi

$A_o = \{0.037E/[(D/t)F_y] + 0.667\} A \leq A$ (*Eq.* C6.2-7)
$= [0.037 \times 29500/(64 \times 50) + 0.667](3.093)$
$= 3.118$

Therefore, $A_o = A = 3.093$ in.^{2}

$A_e = [1 - (1 - R^2)(1 - A_o/A)]A$ (*Eq.* C6.2-6)

Since $A_o/A = 1$, $A_e = A$

$A_e = 3.093$ in.^{2}

$$
\begin{aligned}
P_n &= F_n A_e && (Eq.\ C6.2\text{-}2)\\
&= (46.01)(3.093)\\
&= 142.31 \text{ kips}
\end{aligned}
$$

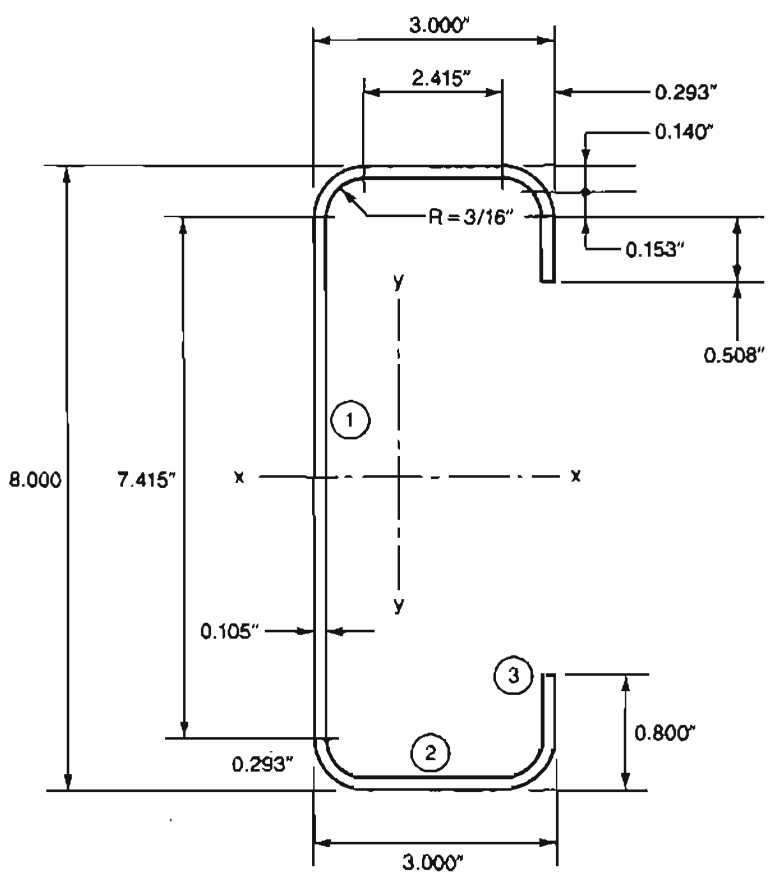
$$
\begin{aligned}
R &= \sqrt{F_y/2F_e} && (Eq.\ C6.2\text{-}5)\\
&= \sqrt{50/2(156.71)}\\
&= 0.399
\end{aligned}
$$

$$
\begin{aligned}
\Omega_c &= 1.670 + 0.375R - 0.125R^3 && (Eq.\ C6.2\text{-}4)\\
&= 1.670 + 0.375(0.399) - 0.125(0.399)^3\\
&= 1.812
\end{aligned}
$$

$$
\begin{aligned}
P_a &= P_n/\Omega_c && (Eq.\ C6.2\text{-}1)\\
&= 142.31/1.812\\
&= 78.5 \text{ kips}
\end{aligned}
$$

EXAMPLE NO. 15

C-SECTION



Given:

1. Steel: $F_y = 50$ ksi.
2. Section: Channel as shown.
3. Length of Section = 16 ft.
4. $L_x = L_y = L_t = 16$ ft.
5. $K_x = K_y = K_t = 1.0$
6. Axial load: P = 2.5 kips.
7. Eccentricity at both ends:
   (a) Axial load is applied 2 in. to the left of the c.g. of the full section
   (b) Axial load is applied at 2 in. to the left and 4 in. above the c.g. of the full section

Required: Check the adequacy of the given section for both cases.

Solution: *Part (a)*

1. Full section properties:
Using the equations given in the AISI *Manual, Part III*, Section 1.2, and noting that $\alpha = 1.00$ for section with Lips,one obtains:

$r = R + t/2 = 3/16 + 0.105/2 = 0.240$ in.
$a = A' - (2r + t) = 8.000 - (2 \times 0.240 + 0.105) = 7.415$ in.
$\bar{a} = A' - t = 8.000 - 0.105 = 7.895$ in.
$b = B' - (2r + t) = 3.000 - (2 \times 0.240 + 0.105) = 2.415$ in.
$\bar{b} = B' - t = 3.000 - 0.105 = 2.895$ in.
$c = C' - (r + t/2) = 0.800 - (0.240 + 0.105/2) = 0.508$ in.
$\bar{c} = C' - t/2 = 0.800 - (0.105/2) = 0.748$ in.
$u = 1.57r = 1.57 \times 0.240 = 0.377$ in.

Distance of corner's c.g. from center of radius $= 0.637r = 0.637(0.240) = 0.153$ in.

$$A = t[a + 2b + 2c + 4u] = 0.105[7.415 + 2 \times 2.415 + 2 \times 0.508 + 4 \times 0.377] = 1.551 \text{ in.}^2$$

$$\begin{aligned} I_x &= 2t[0.0417a^3 + b(a/2 + r)^2 + 2u(a/2 + 0.637r)^2 + 0.298r^3 \\ &\quad + 0.0833c^3 + (c/4)(a - c)^2] \\ &= 2(0.105)\{0.0417(7.415)^3 + 2.415[(7.415/2) + 0.240]^2 \\ &\quad + 2(0.377)[(7.415/2) + 0.637 \times 0.240]^2 \\ &\quad + 0.298(0.240)^3 + 0.0833(0.508)^3 + (0.508/4)(7.415 - 0.508)^2\} \\ &= 15.108 \text{ in.}^4 \end{aligned}$$

$$\begin{aligned} \bar{x} &= (2t/A)[b(b/2 + r) + u(0.363r) + u(b + 1.637r) + c(b + 2r)] \\ &= [2(0.105)/1.551]\{2.415[(2.415/2) + 0.240] + 0.377(0.363 \times 0.240) \\ &\quad + 0.377(2.415 + 1.637 \times 0.240) + 0.508(2.415 + 2 \times 0.240)\} \\ &= 0.820 \text{ in.} \end{aligned}$$

$$\begin{aligned} I_y &= 2t[b(b/2 + r)^2 + 0.0833b^3 + 0.505r^3 + c(b + 2r)^2 \\ &\quad + u(b + 1.637r)^2] - A(x)^2 \\ &= 2(0.105)\{2.415[(2.415/2) + 0.240]^2 + 0.0833(2.415)^3 + 0.505(0.240)^3 \\ &\quad + 0.508(2.415 + 2 \times 0.240)^2 + 0.377(2.415 + 1.637 \times 0.240)^2\} - 1.551(0.820)^2 \\ &= 1.786 \text{ in.}^4 \end{aligned}$$

$$\begin{aligned} m &= (\bar{b}t/12I_x)[6\bar{c}(\bar{a})^2 + 3\bar{b}(\bar{a})^2 - 8(\bar{c})^3] \\ &= [(2.895 \times 0.105)/(12 \times 15.108)][6 \times 0.748(7.895)^2 \\ &\quad + 3 \times 2.895(7.895)^2 - 8(0.748)^3] \\ &= 1.371 \text{ in.} \end{aligned}$$

$$\begin{aligned} x_o &= -(\bar{x} + m) = -(0.820 + 1.371) \\ &= -2.191 \text{ in.} \end{aligned}$$

$$\begin{aligned} J &= (t^3/3)[a + 2b + 2c + 4u] = [(0.105)^3/3][7.415 + 2 \times 2.415 + 2 \times 0.508 + 4 \times 0.377] \\ &= 0.005699 \text{ in.}^4 \end{aligned}$$

$$\begin{aligned} C_w = \frac{t^2}{A}\Bigg\{ &\frac{\bar{x}A(\bar{a})^2}{t}\left[\frac{(\bar{b})^2}{3} + m^2 - m\bar{b}\right] + \frac{A}{3t}\left[(m)^2(\bar{a})^3 + (\bar{b})^2(\bar{c})^2(2\bar{c} + 3\bar{a})\right] \\ &- \frac{I_x m^2}{t}(2\bar{a} + 4\bar{c}) + \frac{m(\bar{c})^2}{3}\{8(\bar{b})^2(\bar{c}) + 2m[2\bar{c}(\bar{c} - \bar{a}) \\ &+ \bar{b}(2\bar{c} - 3\bar{a})]\} + \frac{(\bar{b})^2(\bar{a})^2}{6}[(3\bar{c} + \bar{b})(4\bar{c} + \bar{a}) - 6(\bar{c})^2] \\ &- \frac{m^2(\bar{a})^4}{4}\Bigg\} \end{aligned}$$

$$\begin{aligned} C_w = \frac{(0.105)^2}{1.551}\Bigg\{ &\frac{0.820(1.551)(7.895)^2}{0.105}\left[\frac{(2.895)^2}{3} + (1.371)^2 - 1.371(2.895)\right] \\ &+ \frac{1.551}{3(0.105)}[(1.371)^2(7.895)^3 + (2.895)^2(0.748)^2(2 \times 0.748 + 3 \times 7.895)] \end{aligned}$$

$$- \frac{15.108(1.371)^2}{0.105}(2\times 7.895+4\times 0.748)+\frac{1.371(0.748)^2}{3}\{8(2.895)^2(0.748)$$

$$+\ 2\times 1.371[2\times 0.748(0.748-7.895)+2.895(2\times 0.748-3\times 7.895)]\}$$

$$+\ \frac{(2.895)^2(7.895)^2}{6}[(3\times 0.748+2.895)(4\times 0.748+7.895)-6(0.748)^2]$$

$$-\ \left.\frac{(1.371)^2(7.895)^4}{4}\right\}$$

$$= 23.468 \text{ in.}^6$$

$\beta_w = -[0.0833(t\bar{x}(\bar{a})^3)+t(\bar{x})^3\bar{a}]$
$= -\{0.0833[0.105\times 0.820(7.895)^3]+0.105(0.820)^3\times 7.895\}$
$= -3.987$

$\beta_f = (t/2)[(\bar{b}-\bar{x})^4-(\bar{x})^4]+[t(\bar{a})^2/4][(\bar{b}-\bar{x})^2-(\bar{x})^2]$
$= (0.105/2)[(2.895-0.820)^4-(0.820)^4]$
$+[0.105(7.895)^2/4][(2.895-0.820)^2-(0.820)^2]$
$= 6.895$

$\beta_l = 2\bar{c}t(\bar{b}-\bar{x})^3+(2/3)t(\bar{b}-\bar{x})\{(\bar{a}/2)^3-(\bar{a}/2-\bar{c})^3\}$
$= 2\times 0.748\times 0.105(2.895-0.820)^3+(2/3)\times 0.105(2.895-0.820)\{(7.895/2)^3$
$-[(7.895/2)-0.748]^3\}$
$= 5.581$

$j = (1/2I_y)(\beta_w+\beta_f+\beta_l)-x_o$ (*Eq.* C3.1.2-16)
$= [1/(2\times 1.786)](-3.987+6.895+5.581)-(-2.191)$
$= 4.568$

$r_x = \sqrt{I_x/A} = \sqrt{15.108/1.551} = 3.121$ in.
$K_xL_x/r_x = [1(16\times 12)]/3.121 = 61.52$

$r_y = \sqrt{I_y/A} = \sqrt{1.786/1.551} = 1.073$ in.
$K_yL_y/r_y = [1(16\times 12)]/1.073 = 178.94 < 200$ *OK* [Section C4-(d)]

$r_o = \sqrt{r_x^2+r_y^2+x_o^2} = \sqrt{(3.121)^2+(1.073)^2+(-2.191)^2} = 3.961$ in.

$\beta = 1-(x_o/r_o)^2$ (*Eq.* C4.2-3)
$= 1-(-2.191/3.961)^2 = 0.694$

2. Determination of P_a (Section C4):
Since the channel is singly symmetric, F_e shall be taken as the smaller of F_e calculated according to Section C4.1 or F_e calculated according to Section C4.2.

Section C4.1:
$F_e = (\pi^2E)/(K_yL_y/r_y)^2$ (*Eq.* C4.1-1)
$= (\pi^2\times 29500)/(178.94)^2 = 9.093$ ksi

Section C4.2:
$\sigma_{ex} = (\pi^2E)/(K_xL_x/r_x)^2$ (*Eq.* C3.1.2-12)
$= (\pi^2\times 29500)/(1\times 16\times 12/3.121)^2 = 76.93$ ksi

$\sigma_t = (1/Ar_o^2)\{GJ+[(\pi^2EC_w)/(K_tL_t)^2]\}$ (*Eq.* C3.1.2-14)
$= [1/1.551(3.961)^2]\{11300\times 0.005699+[(\pi^2\times 29500\times 23.468)/(1\times 16\times 12)^2]$
$= 10.26$ ksi

$F_e = (1/2\beta)[(\sigma_{ex}+\sigma_t)-\sqrt{(\sigma_{ex}+\sigma_t)^2-4\beta\sigma_{ex}\sigma_t}]$ (*Eq.* C4.2-1)
$= [1/2(0.694)][(76.93+10.26)-\sqrt{(76.93+10.26)^2-4(0.694)(76.93)(10.26)}]$
$= 9.820$ ksi

Therefore
$F_e = 9.093$ ksi
$F_y/2 = 50/2 = 25.00$ ksi

Since $F_e < F_y/2$ it follows that

$F_n = F_e$ (*Eq.* C4-4)
$= 9.093$ ksi

For element ①:

$w = 7.415$ in.
$w/t = 7.415/0.105 = 70.62 < 500$ *OK* [Section B1.1-(a)-(2)]
$k = 4.00$ (Since connected to two stiffened elements)

$\lambda = (1.052/\sqrt{k})\,(w/t)\,\sqrt{f/E}$, $f = F_n$ (*Eq.* B2.1-4)
$= (1.052/\sqrt{4.00})\,(70.62)\,\sqrt{9.093/29500} = 0.652 < 0.673$

$b = w$ (*Eq.* B2.1-1)
$= 7.415$ in. (Element ① fully effective)

For element ②:

$w = 2.415$ in.
$w/t = 2.415/0.105 = 23.00$
$S = 1.28\sqrt{E/f}$, $f = F_n$ (*Eq.* B4.1)
$= 1.28\sqrt{29500/9.093} = 72.91$
$S/3 = 24.30$
$w/t = 23.00 < S/3 = 24.30$
$b = w$ (*Eq.* B4.2-3)
$= 2.415$ in. (Element ② fully effective)

For element ③:

$d = 0.508$ in.
$d/t = 0.508/0.105 = 4.84 < 60$ *OK* [Section B1.1-(a)-(3)]
$k = 0.43$ (unstiffened compression element)
$\lambda = (1.052/\sqrt{0.43})\,(4.84)\,\sqrt{9.093/29500} = 0.136 < 0.673$
$d'_s = d = 0.508$ in.
$d_s = d'_s$ (*Eq.* B4.2-4)
$= 0.508$ in. (Element ③ fully effective)

Thus the whole section is fully effective.

$A_e = A$
$= 1.551$ in.^{2}

$P_n = A_e F_n$ (*Eq.* C4-2)
$= 1.551 \times 9.093$
$= 14.10$ kips

$\Omega_c = 1.92$ $(F_e < F_y/2)$

$P_a = P_n/\Omega_c$ (*Eq.* C4-1)
$= 14.10/1.92$
$= 7.344$ kips

3. $P/P_a = 2.50/7.344 = 0.340 > 0.15$
   Must check both interaction equations (*Eq.* C5-1), (*Eq.* C5-2)

4. Determination of P_{ao} (Section C4 for $L = 0$):

   for $L = 0$, $K_y L_y/r_y = K_x L_x/r_x = 0$, $K_t L_t = 0$

   Therefore $F_e = \infty$
   $F_e > F_y/2 = 25.00$ ksi

   $F_n = F_y(1 - F_y/4F_e)$ (*Eq.* C4-3)
   $= 50(1 - 0) = 50$ ksi

For element ①:

$\lambda = (1.052/\sqrt{4.00})(70.62)\sqrt{50/29500} = 1.529 > 0.673$

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

$= (1-0.22/1.529)/1.529 = 0.560$

$b = \rho w$ (*Eq.* B2.1-2)

$= 0.560 \times 7.415$

$= 4.152$ in.

For element ②:

$S = 1.28\sqrt{29500/50} = 31.09$

$S/3 = 10.36$

$S/3 = 10.36 < w/t = 23.00 < S = 31.09$

$I_a = 399t^4\{[(w/t)/S]-0.33\}^3$ (*Eq.* B4.2-6)

$= 399(0.105)^4[(23/31.09)-0.33]^3$

$= 0.003337$ in.^{4}

$I_s = d^3t/12 = (0.508)^3(0.105)/12$

$= 0.001147$ in.^{4}

$I_s/I_a = 0.001147/0.003337$

$= 0.344$

$D/w = 0.8/2.415 = 0.331$

$n = 1/2$

$k = [4.82-5(D/w)](I_s/I_a)^n + 0.43 \leq 5.25-5(D/w)$ (*Eq.* B4.2-9)

$[4.82-5(D/w)](I_s/I_a)^n + 0.43 = [4.82-5(0.331)](0.344)^{1/2} + 0.43$

$= 2.286$

$5.25-5(D/w) = 5.25-5(0.331)$

$= 3.595 > 2.286$

$k = 2.286$

$\lambda = (1.052/\sqrt{2.286})(23.00)\sqrt{50/29500} = 0.659 < 0.673$

$b = w = 2.415$ in. (Element ② fully effective)

For element ③:

$\lambda = (1.052/\sqrt{0.43})(4.84)\sqrt{50/29500} = 0.320 < 0.673$

$d'_s = d = 0.508$ in.

$d_s = d'_s(I_s/I_a) \leq d'_s$ (*Eq.* B4.2-11)

Since $I_s/I_a = 0.344 < 1.0$

$d_s = 0.508(0.344) = 0.175$ in.

$A_e = 1.551 - 0.105(7.415-4.152) - 0.105(0.508-0.175)$

$= 1.173$ in.^{2}

$P_{no} = 1.173 \times 50 = 58.65$ kips

$\Omega_c = (5/3) + (3/8)R - (R^3/8))]$ (*Eq.* C4-1)

$R = \sqrt{F_y/2F_e},\ F_e = \infty$

$R = 0$

$\Omega_c = 1.67$

$P_{ao} = 58.65/1.67$

$= 35.12$ kips

5. Determination of M_y (moment about y-axis): ($M_x = 0$ since $e_y = 0$)

M_y will be with respect to the centroidal axes of the effective section determined for the axial load alone.

$A_e = 1.551$ in.^{2} under axial load alone

Since $A_e = A$, the centroidal axes for the effective section are the same as those for the full section. Therefore, e_x did not change.

$M_y = 2.50(2.00)$

$= 5.00$ kip-in. (Applied Moment)

The interaction equations (*Eq.* C5-1) and (*Eq.* C5-2) reduce to the following:

$$P/P_a + C_{my}/M_{ay} \times M_y/\alpha_y \leq 1.0 \qquad (Eq.\ C5\text{-}1)$$

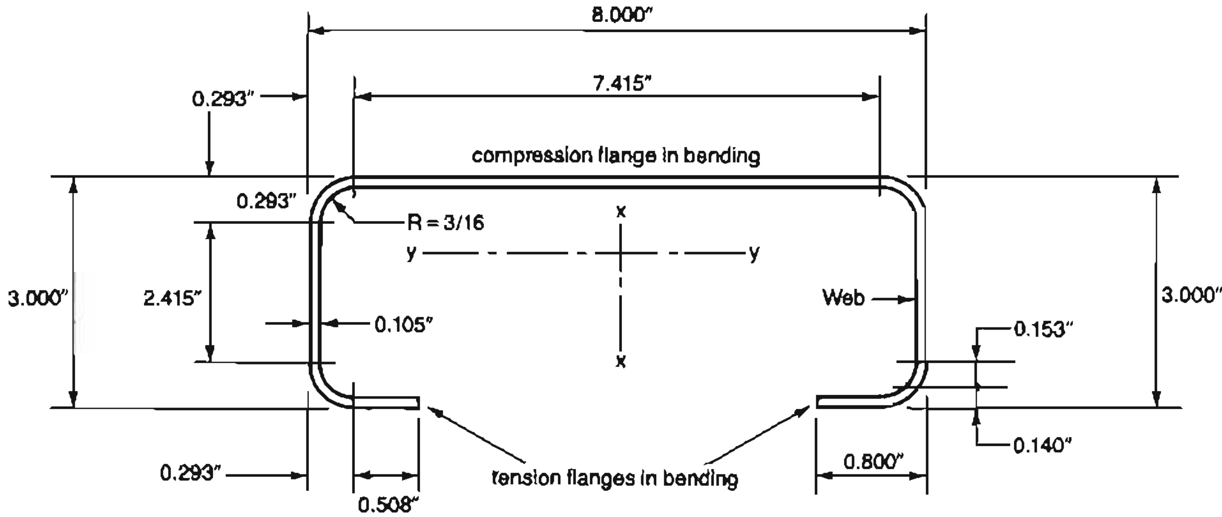
$$P/P_{ao} + M_y/M_{ayo} \leq 1.0 \qquad (Eq.\ C5\text{-}2)$$

6. Determination of M_{ay} (Section C3):

M_{ny} shall be taken as the smaller of the ultimate moments calculated according to sections C3.1.1 and C3.1.2:

(a) Section C3.1.1: M_{ny} will be calculated on the basis of initiation of yielding.

Here it is evident that the initial yielding will not be in the compression flange, rather it will be in the tension flange.



The procedure is iterative: one assumes the actual compressive stress f under M_{ny}. Knowing f one proceeds as usual to obtain x_{cg} (measured from top fiber) to neutral axis. Then one obtains $f = F_y[x_{cg}/(3 - x_{cg})]$ and checks if it equals the assumed value. If not, one reiterates by assuming another f until finally it checks. Then for this condition one obtains I_y and $M_{ny} = f(I_y/x_{cg}) = F_y[I_y/(3 - x_{cg})]$.

For the first iteration assume a compressive stress f = 20 ksi in the top compression fibers and that the webs are fully effective.

Compression flange:

$k = 4.00$
$w/t = 7.415/0.105 = 70.62$
$\lambda = (1.052/\sqrt{4.00})(70.62)\sqrt{20/29500} = 0.967 > 0.673$
$\rho = [1 - (0.22/0.967)]/0.967 = 0.799$
$b = 0.799 \times 7.415 = 5.925$ in.

To calculate effective section properties about y-axis:

Element	L Effective Length (in.)	x Distance from Top Fiber (in.)	Lx (in.^{2})	Lx^2 (in.^{3})	I'_1 About Own Axis (in.^{3})
Webs	2 × 2.415 = 4.830	1.500	7.245	10.868	2.347
Upper Corners	2 × 0.377 = 0.754	0.140	0.106	0.015	—
Lower Corners	2 × 0.377 = 0.754	2.860	2.156	6.167	—
Compression Flange	5.925	0.053	0.314	0.017	—
Tension Flanges	2 × 0.508 = 1.016	2.948	2.995	8.830	—
Sum	13.279		12.816	25..897	2.347

Distance from top fiber to y-axis is $x_{cg} = 12.816/13.279 = 0.965$ in.

$f = F_y[(x_{cg})/(3 - x_{cg})] = 50[0.965/(3.000 - 0.965)] = 23.71$ ksi > 20 ksi
need to do another iteration.

For the second iteration assume a compression stress $f = 24.95$ ksi in the top compression fibers, and that the web is fully effective.

Compression flange:
$\lambda = (1.052/\sqrt{4.00})\,(70.62)\,\sqrt{24.95/29500} = 1.080$
$\rho = [1 - (0.22/1.080)]/1.080 = 0.737$
$b = 0.737 \times 7.415 = 5.465$ in.

Effective section properties about y-axis:

Element	L Effective Length (in.)	x Distance from Top Fiber (in.)	Lx (in.^{2})	Lx^2 (in.^{3})	I'_1 About Own Axis (in.^{3})
Webs	$2 \times 2.415 = 4.830$	1.500	7.245	10.868	2.347
Upper Corners	$2 \times 0.377 = 0.754$	0.140	0.106	0.015	—
Lower Corners	$2 \times 0.377 = 0.754$	2.860	2.156	6.167	—
Compression Flange	5.465	0.053	0.290	0.015	—
Tension Flanges	$2 \times 0.508 = 1.016$	2.948	2.995	8.830	—
Sum	12.819		12.792	25..895	2.347

Distance from top fiber to y-axis is $x_{cg} = 12.792/12.819 = 0.998$ in.

$f = 50[0.998/(3.000 - 0.998)] = 24.93$ ksi (close enough)
Thus actual compressive stress $f = 24.95$ ksi

To check if the webs are fully effective (Section B2.3):

$f_1 = [(0.998 - 0.293)/2.002]\,(50) = 17.61$ ksi (compression)
$f_2 = -[(2.002 - 0.293)/2.002]\,(50) = -42.68$ ksi (tension)
$\psi = f_2/f_1 = -42.68/17.61 = -2.424$
$k = 4 + 2(1 - \psi)^3 + 2(1 - \psi)$
$= 4 + 2[1 - (-2.424)]^3 + 2[1 - (-2.424)]$
$= 91.132$
$h = w = 2.415$ in.
$w/t = 2.415/0.105 = 23.00 < 200$ *OK*[Section B1.2-(a)]
$\lambda = (1.052/\sqrt{91.132})\,(23.00)\,\sqrt{17.61/29500} = 0.062 < 0.673$
$b_e = 2.415$ in.
$b_2 = b_e/2$ (*Eq.* B2.3-2)
$= 2.415/2 = 1.208$ in.
$b_1 = b_e/(3 - \psi)$ (*Eq.* B2.3-1)
$= 2.415/[3 - (-2.424)]$
$= 0.445$ in.

Compression portion of each web calculated on the basis of the effective section = $x_{cg} - 0.293 = 0.998 - 0.293 = 0.705$ in.

Since $b_1 + b_2 = 1.653$ in. > 0.705 in., $b_1 + b_2$ shall be taken as 0.705 in. This verifies the assumption that the web is fully effective.

$I'_y = Lx^2 + I'_1 - Lx^2_{cg}$
$= 25.895 + 2.374 - 12.819(0.998)^2$
$= 15.501$ in.^{3}
Actual $I_y = I'_y t$
$= 15.501(0.105)$
$= 1.628$ in.^{4}

$S_e = I_y/(3.000 - x_{cg})$
$= 1.628/(3.000 - 0.998)$
$= 0.813$ in.^{3}

$M_{ny} = S_e F_y$ (*Eq.* C3.1.1-1)
$= 0.813(50)$
$= 40.65$ kip-in.

(b) Section C3.1.2:

M_{ny} will be calculated on the basis of the lateral buckling strength. (y-axis is the axis of bending)

For the full section:

$I_y = 1.786$ in.^{4}
$x_{cg} = \bar{x} + t/2 = 0.820 + 0.105/2$
$= 0.873$ in.
$S_f = I_y/x_{cg} = 1.786/0.873$
$= 2.046$ in.^{3}
$M_y = S_f F_y$ (Yield Moment) (*Eq.* C3.1.2-5)
$= 2.046(50)$
$= 102.30$ kip-in.

$C_s = +1.00$
$A = 1.551$ in.^{2}
$\sigma_{ex} = 76.93$ ksi
$\sigma_t = 10.26$ ksi
$M_1/M_2 = -1.00$ (single curvature)
$C_{TF} = 0.6 - 0.4(M_1/M_2)$
$= 0.6 - 0.4(-1.00) = 1.00$
$r_o = 3.961$ in.
$j = 4.568$

$M_e = C_s A \sigma_{ex}[j + C_s\sqrt{j^2 + r_o^2(\sigma_t/\sigma_{ex})}]/C_{TF}$ (*Eq.* C3.1.2-11)
$= 1.0(1.551)(76.93)[4.568 + 1.00\sqrt{(4.568)^2 + (3.961)^2(10.26/76.93)}]/1.00$
$= 1116.77$ kip-in.
$M_e = 1116.77$ kip-in. $> 0.5\,M_y = 51.15$ kip-in.
$M_c = M_y[1 - (M_y/4M_e)]$ (*Eq.* C3.1.2-8)
$= 102.30\{1 - [102.30/(4 \times 1116.77)]\}$
$= 99.96$ kip-in.
$M_c/S_f = 99.96/2.046 = 48.86$ ksi

To calculate effective section properties to obtain S_c at stress 48.86 ksi, we assume that the webs are fully effective.

Compression flange:
$\lambda = (1.052/\sqrt{4.00})(70.62)\sqrt{48.86/29500} = 1.512 > 0.673$
$\rho = [1 - (0.22/1.512)]/1.512 = 0.565$
$b = 0.565(7.415) = 4.189$ in.

Element	L Effective Length (in.)	x Distance from Top Fiber (in.)	Lx (in.^{2})	Lx^2 (in.^{3})	I'_1 About Own Axis (in.^{3})
Webs	$2 \times 2.415 = 4.830$	1.500	7.245	10.868	2.347
Upper Corners	$2 \times 0.377 = 0.754$	0.140	0.106	0.015	—
Lower Corners	$2 \times 0.377 = 0.754$	2.860	2.156	6.167	—
Compression Flange	4.189	0.053	0.222	0.012	—
Tension Flange	$2 \times 0.508 = 1.016$	2.948	2.995	8.830	—
Sum	11.543		12.724	25..892	2.347

Distance from top fiber to y-axis is $x_{cg} = 12.724/11.543 = 1.102$ in.

To check if the webs are fully effective (Section B2.3):

$f_1 = [(1.102 - 0.293)/1.102](48.91) = 35.91$ ksi (compression)
$f_2 = -[(1.898 - 0.293)/1.102](48.91) = -71.24$ ksi (tension)
$\psi = -71.24/35.91 = -1.984$
$k = 4 + 2[1-(-1.984)]^3 + 2[1-(-1.984)]$
$= 63.109$
$\lambda = (1.052/\sqrt{63.109})(23.00)\sqrt{35.91/29500} = 0.106 < 0.673$
$b_e = 2.415$ in.
$b_2 = 2.415/2 = 1.208$ in.
$b_1 = 2.415/[3-(-1.984)] = 0.485$ in.

Compression portion of each web calculated on the basis of the effective section $= 1.102 - 0.293 = 0.809$ in.

Since $b_1 + b_2 = 1.693$ in. > 0.809 in., $b_1 + b_2$ shall be taken as 0.809 in. This verifies the assumption that the web is fully effective.

$I'_y = 25.892 + 2.347 - 11.543(1.102)^2$
$= 14.221$ in.^{3}
Actual $I_y = 14.221(0.105)$
$= 1.493$ in.^{4}
$S_c = I_y/x_{cg} = 1.493/1.102$
$= 1.355$ in.^{3}
$M_{ny} = M_cS_c/S_f$ (*Eq.* C3.1.2-1)
$= 100.07(1.355)/2.046$
$= 66.27$ kip-in.

M_{ny} shall be the smaller of 40.65 kip-in. and 66.27 kip-in.

Thus
$M_{ny} = 40.65$ kip-in.
$\Omega_f = 1.67$
$M_{ay} = M_{ny}/\Omega_f$ (*Eq.* C3.1-1)
$= 40.65/1.67$
$= 24.34$ kip-in.

7. Determination of M_{ayo}:

M_{ayo} is the allowable moment about the centroidal axes determined in accordance with Section C3.1 excepting the provisions of Section C3.1.2 (excluding lateral buckling).

Therefore
$M_{nyo} = 40.65$ kip-in.
$\Omega_f = 1.67$
$M_{ayo} = 40.65/1.67$
$= 24.34$ kip-in.

8. $C_{my} = 0.6 - 0.4(M_1/M_2) \geq 0.4$

$M_1/M_2 = -1.00$ (single curvature)

$0.6 - 0.4(-1.00) = 1.00 > 0.4$

$C_{my} = 1.00$

9. Determination of $1/\alpha y$:

$\Omega_c = 1.92$
$P_{cr} = \pi^2EI_y/(K_yL_y)^2$ (*Eq.* C5-5)
$I_y = 1.786$ in.^{4}
$K_yL_y = 1.0(16 \times 12) = 192$ in.
$P_{cr} = [\pi^2(29500)(1.786)]/(192)^2 = 14.11$ kips
$1/\alpha_y = 1/[1-(\Omega_cP/P_{cr})]$ (*Eq.* C5-4)
$= 1/[1-(1.92 \times 2.5/14.11)] = 1.516$
$\alpha_y = 0.660$

10. Check interaction equations:

$$(P/P_a)+(C_{my}/M_{ay})\times(M_y/\alpha_y)=(2.500/7.344)+(1.00/24.34)\times(5.00/0.660) \quad (Eq.\ C5\text{-}1)$$
$$=0.340+0.311$$
$$=0.651<1.0 \quad OK$$

$$(P/P_{ao})+(M_y/M_{ayo})=(2.50/30.55)+(5.00/24.34) \quad (Eq.\ C5\text{-}2)$$
$$=0.082+0.205$$
$$=0.287<1.0 \quad OK$$

Therefore the section is adequate for the applied loads.

Solution: *Part (b)*

1. Full section properties are the same as previously calculated in part (a.1).
2. $P_a = 7.344$K (calculated in part (a)).
3. $P/P_a = 2.50/7.344 = 0.340 > 0.15$

   Therefore the following interaction equations must be satisfied.

   $$(P/P_a)+(C_{mx}M_x/M_{ax}\alpha_x)+(C_{my}M_y/M_{ay}\alpha_y)\le 1.0 \quad (Eq.\ C5\text{-}1)$$
   $$(P/P_{ao})+(M_x/M_{axo})+(M_y/M_{ayo})\le 1.0 \quad (Eq.\ C5\text{-}2)$$

4. $P_{ao} = 30.55$K (calculated in part (a) 4).
5. Determination of M_x (Section C5):

   The centroidal x-axis is the same for both the full and effective sections.

   $e_y = 4.000$ in.
   $M_x = Pe_y$
   $= (2.50)(4.000)$
   $= 10.00$ kip-in.

6. Determination of M_{ax} (Section C3):

   M_{nx} shall be taken as the smaller of the ultimate moments calculated according to Sections C3.1.1 and C3.1.2.

   (a) Section C3.1.1: M_{nx} will be calculated based on the initiation of yielding

   First approximation:
   - Assume a compressive stress of $f = F_y = 50$ ksi in the top fiber of the section.
   - Assume that the web is fully effective.

   Compression flange:

   $w = 2.415$ in.
   $w/t = 2.415/0.105 = 23.00$

   $$S = 1.28\sqrt{E/f} \quad (Eq.\ B4\text{-}1)$$
   $$= 1.28\sqrt{29500/50}$$
   $$= 31.09$$

   For $S/3 = 10.36 < w/t = 23.00 < S = 31.09$

   $$I_a = t^4 399\{[(w/t)/S]-0.33\}^3 \quad (Eq.\ B4.2\text{-}6)$$
   $$= (0.105)^4(399)[(23.00/31.09)-0.33]^3$$
   $$= 0.003337 \text{ in.}^4$$

   $$I_s = (0.508)^3(0.105)/12$$
   $$= 0.001147 \text{ in.}^4$$

   $$I_s/I_a = 0.001147/0.003337$$
   $$= 0.344$$

   $D = 0.800$ in.

$D/w = 0.800$ in./2.415 in.
$= 0.331$

$w/t = 23.00 < 60$ *OK* [Section B1.1-(a)-(3)]

For $0.25 > D/w = 0.331 < 0.8$

$k = [4.82 - 5(D/w)](I_s/I_a)^{1/2} + 0.43 \leq 5.25 - 5(D/w)$ (*Eq.* B4.2-9)

$[4.82 - 5(D/w)]\,(I_s/I_a)^{1/2} + 0.43 = [4.82 - 5(0.331)]\,(0.344)^{1/2} + 0.43$
$= 2.286$
$5.25 - 5(D/w) = 5.25 - 5(0.331)$
$= 3.595$

$k = 2.286$
$\lambda = (1.052/\sqrt{k})\,(w/t)\,\sqrt{f/E}$ (*Eq.* B2.1-4)
$= (1.052/\sqrt{2.286})\,(23.00)\,\sqrt{50/29500}$
$= 0.659$

For $\lambda < 0.673$
$b = w$ (*Eq.* B2.1-1)
$b = 2.415$ in. (fully effective)

Compression Stiffener:

$d = 0.508$ in.
$d/t = 0.508/0.105 = 4.84 < 60$ *OK* [Section B1.1-(c)-(3)]
$k = 0.43$

Assume max stress in element, $f = F_y = 50$ ksi although it will be actually less.

$\lambda = (1.052/\sqrt{k})\,(w/t)\,\sqrt{f/E}$ (*Eq.* B2.1-4)
$= (1.052/\sqrt{0.43})\,(4.84)\,\sqrt{50/29500}$
$= 0.320$

For $\lambda < 0.673$
$b = w$ (*Eq.* B2.1-1)
$d'_s = 0.508$ in.
$d_s = d'_s(I_s/I_a) \leq d'_s$ (*Eq.* B4.2-11)
$= 0.508(0.344)$
$= 0.175$ in.

Calculate section properties:

Element	L Effective Length (in.)	y Distance from Top Fiber (in.)	Ly (in.^{2})	Ly^2 (in.^{3})	I'_1 About Own Axis (in.^{3})
Compression Flange	2.415	0.053	0.128	0.007	—
Compression Stiffener	0.175	0.381	0.067	0.025	—
Compression Corners	2 × 0.377 = 0.754	0.140	0.106	0.015	—
Web	7.415	4.000	29.660	118.640	33.974
Tension Flange	2.415	7.948	19.194	152.557	—
Tension Stiffener	0.508	7.453	3.786	28.218	0.011
Tension Corners	2 × 0.377 = 0.754	7.860	5.926	46.582	—
Sum	14.436		58.867	346.044	33.985

Distance from neutral axis to top fiber, $y_{cg} = Ly/L = 58.867/14.436 = 4.078$ in.

Since the distance from the neutral axis to the top compression fiber is greater than half the depth of the section, a compressive stress of $F_y = 50$ ksi governs as assumed.

$I'_x = Ly^2 + I'_1 - Ly_{cg}^2$
$= 346.044 + 33.985 - 14.436(4.078)^2$
$= 140.0 \text{ in.}^3$

Actual $I_x = tI'_x$
$= (0.105)(140.0)$
$= 14.70 \text{ in.}^4$

Check Web ·

$w/t = 7.415/0.105 = 70.62 < 200$ *OK* [Section B1.2-(a)]

$f_1 = [(4.078-0.293)/4.078](50) = 46.41$ ksi (compression)
$f_2 = [(3.922-0.293)/4.078](50) = 44.50$ ksi (tension)
$\psi = f_2/f_1 = -44.50/46.41 = -0.959$

$k = 4 + 2(1-\psi)^3 + 2(1-\psi)$
$= 4 + 2[1-(-0.959)]^3 + 2[1-(-0.959)]$
$= 22.95$

$\lambda = (1.052/\sqrt{k})(w/t)\sqrt{f/E}$, $\quad f = f_1$ (*Eq.* B2.1-4)
$= (1.052/\sqrt{22.95})(70.62)\sqrt{46.41/29500}$
$= 0.615$

For $\lambda < 0.673$
$b = w$ (*Eq.* B2.1-1)
$b_e = 7.415$ in.
$b_2 = b_e/2$
$= 7.415/2$
$= 3.708$ in.
$b_1 = b_e/(3-\psi)$
$= 7.415/[3-(-0.959)]$
$= 1.873$ in.
$b_1 + b_2 = 3.708 +$
$= 5.581 > 3.785$ (compression portion of web)

Therefore web is fully effective as assumed.

Check Compression Stiffener

Actual maximum stress in stiffener = 46.41 ksi

$\lambda = (1.052/\sqrt{0.43})(4.84)\sqrt{46.41/29500}$
$= 0.308$

For $\lambda < 0.673$
$d'_s = 0.508$ in.

Since I_s/I_a is unchanged
$d_s = 0.175$ in.

Conservative assumption *OK*

$S_e = I_x/y_{cg} = 14.70/4.078 = 3.605 \text{ in.}^3$
$M_{nx} = S_eF_y$ (*Eq.* C3.1.1-1)
$= (3.605)(50)$
$= 180.3$ kip-in.

b. Section C3.1.2: M_{nx} will be calculated based on the lateral buckling strength

For the full section:
$I_x = 15.108 \text{ in.}^4$
$y_{cg} = 4.000$ in.
$S_f = I_x/y_{cg} = 15.108/4.000 = 3.777 \text{ in.}^3$

$M_y = S_f F_y$ (*Eq.* C3.1.2-5)
$= (3.777)(50)$
$= 188.9$ kip-in.

$C_b = 1.0$ (for members subject to combined axial load and bending moment)

$r_o = 3.961$ in.
$A = 1.551$ in.^{2}
$\sigma_{ey} = 9.093$ ksi
$\sigma_t = 10.26$ ksi

$M_e = C_b r_o A \sqrt{\sigma_{ey}\sigma_t}$ (*Eq.* C3.1.2-10)
$= (1.000)(3.961)(1.551)\sqrt{(9.093)(10.26)}$
$= 59.34$ kip-in.

$0.5M_y = 0.5(188.9)$
$= 94.45$ kip-in.

For $M_e < 0.5M_y$
$M_c = M_e$ (*Eq.* C3.1.2-9)
$= 59.34$ kip-in.

$M_c/S_f = 59.34/3.777$
$= 15.71$ ksi

Determine S_c, the elastic section modulus of the effective section calculated at a stress of M_c/S_f in the extreme compression fiber.

For compression flange:
$w = 2.415$ in.
$w/t = 2.415/0.105 = 23.00$
$S = 1.28\sqrt{E/f}$
$= 1.28\sqrt{29500/15.71} = 55.47$
$S/3 = 18.49 < w/t = 23.00 < S = 55.47$
$I_a = 399(0.105)^4[(23/55.47) - 0.33]^3$
$= 0.000029$ in.^{4}
$I_s = 0.001147$ in.^{4}
$I_s/I_a = 39.55$

$[4.82 - 5(0.331)](39.55)^{1/2} + 0.43 = 20.334 > 3.595$

$k = 3.595$
$\lambda = (1.052/\sqrt{3.595})(23)\sqrt{15.71/29500} = 0.294 < 0.673$
$b = w = 2.415$ in. (compression flange fully effective)

For compression stiffener:

f is taken conservatively 15.71 ksi as in top compression fiber.

$d/t = 4.84$ in.
$\lambda = (1.052/\sqrt{0.43})(4.84)\sqrt{15.71/29500} = 0.179 < 0.673$
$d'_s = d = 0.508$ in.

And since $I_s/I_a = 39.55 > 1.0$
$d_s = d'_s = 0.508$ in. (compression stiffener fully effective)

And since the web was fully effective at the stress $f = F_y = 50$ ksi, it will be fully effective for $f = 15.71$ ksi

Thus the whole section is fully effective at $M_c/S_f = 15.71$ ksi

Therefore
$S_c = S_e$
$= 3.777$ in.^{3}

$M_{nx} = M_c S_c/S_f$
$= 59.34(3.777)/3.777$
$= 59.34$ kip-in.

M_{nx} shall be the smaller of 180.3 kip-in. and 59.34 kip-in.

Therefore

$$\begin{aligned} M_{nx} &= 59.34 \text{ kip-in.} \\ M_{ax} &= M_n/\Omega_f \\ &= 59.34/1.67 \\ &= 35.53 \text{ kip-in.} \end{aligned}$$

7. Determination of M_{axo} (Section C3.1):

   M_{nxo} will be calculated based on the nominal section strength and on the initiation of yielding.

$$\begin{aligned} M_{nxo} &= 180.3 \text{ kip-in. [calculated in part (6)]} \\ M_a &= M_n/\Omega_f \qquad (Eq.\ C3.1.1) \\ M_{axo} &= M_{nx}/\Omega_f \\ &= 180.3/1.67 \\ &= 108.0 \text{ kip-in.} \end{aligned}$$

8. Determination of C_{mx} (Section C5):

   $M_1/M_2 = -1.0$ (single curvature)

   $C_{mx} = 0.6 - 0.4(-1.0) = 1.0 > 0.4$ *OK*

9. Determination of α_x (Section C5):

$$\begin{aligned} P &= 2.5\text{k} \\ P_{cr} &= \pi^2 EI_x/(K_xL_x)^2 \qquad (Eq.\ C5\text{-}5) \\ &= [\pi^2(29500)(15.108)]/[1(16)\times 12]^2 \\ &= 119.3\text{k} \\ \Omega_c &= 1.92 \\ 1/\alpha_x &= 1/[1-(\Omega_c P/P_{cr})] \qquad (Eq.\ C5\text{-}4) \\ &= 1/\{1-[1.92(2.5)/119.3]\} \\ &= 1.042 \\ d_x &= 0.960 \end{aligned}$$

10. $M_y = 5.0$ kip-in. [calculated in part (a) 5]
11. $M_{ay} = 24.34$ kip-in. [calculated in part (a) 6]
12. $M_{ayo} = 24.34$ kip-in. [calculated in part (a) 7]
13. $C_{my} = 1.0$ [calculated in part (a) 8]
14. $\alpha_y = 0.660$ [calculated in part (a) 9]
15. Interaction equations (Section C5):

$$(P/P_a) + (C_{mx}M_x/M_{ax}\alpha_x) + (C_{my}M_y/M_{ay}\alpha_y) \le 1.0 \qquad (Eq.\ C5\text{-}1)$$

$$(2.50/7.344) + [1.0(10.00)/35.53(0.960)] + [1.0(5.0)/24.34(0.660)]$$

$$0.340 + 0.293 + 0.311 = 0.944 < 1.0 \quad OK$$

$$(P/P_{ao}) + (M_x/M_{axo}) + (M_y/M_{ayo}) \le 1.0 \qquad (Eq.\ C5\text{-}2)$$

$$(2.50/30.55) + (10.00/108.0) + (5.0/24.34)$$

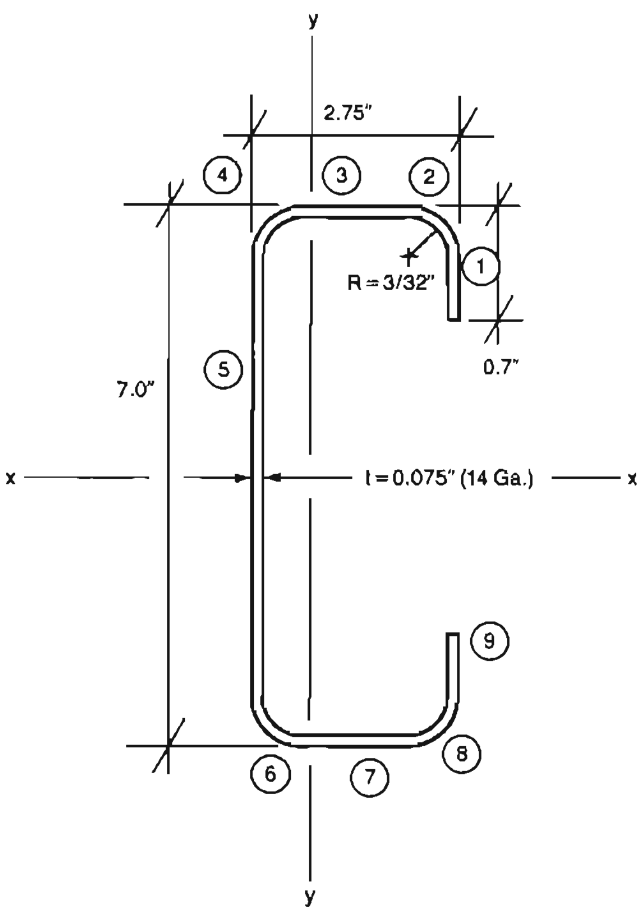
$$0.082 + 0.093 + 0.205 = 0.380 < 1.0 \quad OK$$

Therefore the section is adequate for the applied loads.

EXAMPLE NO. 16

C-SECTION-WALL STUD

Stud Cross Section/Properties.



$S_x = 2.17$ in.^{3}
$A = 1.003$ in.^{2}
$I_x = 7.66$ in.^{4}
$I_y = 1.001$ in.^{4}
$r_x = 2.76$ in.
$r_y = 0.999$ in.
$J = .00188$ in.^{4} (St. Venant Torsion Constant)
$C_w = 9.94$ in.^{6} (Warping Constant of Torsion)
$r_o = 3.57$ in. (Polar r About Shear Ctr.)
$X_o = 2.03$ in. (Distance From Shear Ctr. to x-y axis)
$G = 11{,}300$ ksi (Shear Modulus)

Note—y = y axis perpendicular to wall board

Given:
1. Steel: $F_y = 50$ ksi.
2. Section: As shown below, spacing 24 in. O.C.
3. Length: 15 ft.-0 in.
4. Cladding: On both sides, ½ in. gypsum board with No. 6 Type S-12 self-drilling screws @ 12 in. O.C. vertically.

Required:
1. The allowable axial load.
2. The allowable axial load in combination w/5 PSF lateral load.

Solution:

1. No Lateral Load (Section D4.1)

   (a) Check column buckling between fasteners

   $KL/r_y = (2)(12)/(0.999) = 24.0$

   For flexural buckling about the y-y axis

   $F_{e1} = \pi^2E/(KL/r_y)^2 = [\pi^2(29500)]/(24.0)^2 = 505$ ksi (*Eq.* C4.1-1)

   For torsional-flexural buckling

   $F_{e2} = (1/2\beta)[(\sigma_{ex} + \sigma_t) - \sqrt{(\sigma_{ex} + \sigma_t)^2 - 4\beta\sigma_{ex}\sigma_t}]$ (*Eq.* C4.2-1)

   $\sigma_{ex} = \pi^2E/(K_xL_x/r_x)^2 = \pi^2(29500)/[(2)(12)/2.76]^2 = 3850$ ksi (*Eq.* C3.1.2-12)

   $\sigma_t = (1/Ar_o^2)\{GJ + [\pi^2EC_w/(K_tL_t)^2]\}$ (*Eq.* C3.1.2-14)
   $\sigma_t = [1/1.003(3.57)^2]\{(11300 \times 0.00188) + [\pi^2(29500)(9.94)/(2 \times 12)^2]\}$
   $\sigma_t = 395$ ksi

   $\beta = 1 - (x_o/r_o)^2 = 1 - (2.03/3.57)^2 = 0.677$ (*Eq.* C4.2-3)

   $F_{e2} = [1/(2 \times 0.677)][(3850 + 395) - \sqrt{(3850 + 395)^2 - 4(0.677)(3850)(395)}]$
   $F_{e2} = 381$ ksi (<505 ksi)

   Since $F_{e2} > (F_y/2 = 25$ ksi$)$,
   $F_n = F_y(1 - F_y/4F_e)$ (*Eq.* C4-3)
   $F_{n1} = 50[1 - 50/(4 \times 381)] = 48.4$ ksi

   (b) Check flexural and/or torsional overall column buckling

   $\sigma_{CR} = \sigma_{ey} + \bar{Q}_a$ (*Eq.* D4.1-2)
   OR
   $\sigma_{CR} = (1/2\beta)[(\sigma_{ex} + \sigma_{tQ}) - \sqrt{(\sigma_{ex} + \sigma_{tQ})^2 - 4\beta\sigma_{ex}t_Q}]$ (*Eq.* D4.1-3)

   $\sigma_{ex} = \pi^2E/(L/r_x)^2 = [\pi^2(29500)]/[(15)(12)/2.76]^2 = 68.5$ ksi (*Eq.* C3.1.2-12)

   $\sigma_{tQ} = \sigma_t + \bar{Q}_t$ (*Eq.* D4.1-12)

   $\sigma_t = (1/Ar_o^2)[GJ + (\pi^2EC_w/L^2)]$ (*Eq.* C3.1.2-14)
   $\sigma_t = [1/(1.003)(3.57)^2]\{(11300)(0.00188)$
   $+ [\pi^2(29500)(9.94)/(15 \times 12)^2]\} = 8.65$ ksi

   $\bar{Q}_t = (\bar{Q}\,d^2)/(4Ar_o^2)$ (*Eq.* D4.1-14)

   $\bar{Q} = \bar{q}B$ (*Eq.* D4.1-12)

   $B = 24$ in. O.C.
   $\bar{q} = \bar{q}_o(2 - s/12)$ (*Eq.* D4.1-26)
   $\bar{q}_o = 2.0$ kip-in.
   $S = 12$ in.
   $\bar{q} = 2.0(2 - 12/12) = 2.0$ kip-in.
   $\bar{Q} = (2.0)(24) = 48$ kips
   $\bar{Q}_t = (48)(7)^2/(4)(1.003)(3.57)^2 = 46.0$ ksi
   $\sigma_{tQ} = 8.65 + 46.0 = 54.6$ ksi
   $\sigma_{ey} = \pi^2E/(L/r_y)^2 = \pi^2(29500)/[(15)(12)/0.999]^2 = 8.97$ ksi (*Eq.* C3.1.2-13)
   $\bar{Q}_a = \bar{Q}/A = 48/1.003 = 47.9$ ksi (*Eq.* D4.1-13)
   $\beta = 0.677$ (calculated previously)

$\sigma_{CR} = 8.97 + 47.9 = 56.9$ ksi

OR

$\sigma_{CR} = [1/(2 \times 0.677)][(68.5 + 54.6) - \sqrt{(68.5 + 54.6)^2 - 4(0.677)(68.5)(54.6)}]$
$= 38.6$ ksi

Use $\sigma_{CR} = 38.6$ ksi $= F_e$

$F_e > (F_y/2 = 25$ ksi$)$ so

$F_{n2} = F_y(1 - F_y/4F_e)$ (*Eq.* C4-3)

$F_{n2} = 50[1 - 50/(4 \times 38.6)] = 33.8$ ksi

(c) Check shear strain of wall material

$C_o = L/350 = (12)(15)/350 = 0.514$ in. (*Eq.* D4.1-21)

$D_o = L/700 = (12)(15)/700 = 0.257$ in. (*Eq.* D4.1-22)

$E_o = L/(d \times 10{,}000) = (12)(15)/(7 \times 10{,}000) = 0.00257$ rad (*Eq.* D4.1-23)

Let initial trial value of F_{n3} be based on largest stress for elastic values of E and G

Assume $F_n = 0.5F_y = 0.5(50) = 25$ ksi

$E = 29500$ ksi
$G = 11300$ ksi
$C_1 = (F_nC_o)/(\sigma_{ey} - F_n + \overline{Q}_a)$ (*Eq.* D4.1-20)
$\sigma_{ey} = 8.97$ ksi
$\overline{Q}_a = 47.9$ ksi
$C_1 = (25)(0.514)/(8.97 - 25 + 47.9) = 0.403$

$E_1 = \{F_n[(\sigma_{ex} - F_n)(r_o^2E_o - x_oD_o)$
$- F_nx_o(D_o - x_oE_o)]\}/[(\sigma_{ex} - F_n)r_o^2(\sigma_{tQ} - F_n) - (F_nx_o)^2]$ (*Eq.* D4.1-17)

$\sigma_{ex} = 68.5$ ksi
$\sigma_{tQ} = 54.6$ ksi

$E_1 = (25)\{(68.5 - 25)[(3.57)^2(0.00257) - (2.03)(0.257)]$
$-(25)(2.03)[0.257 - (2.03)(0.00257)]\}/(68.5 - 25)(3.57)^2(54.6 - 25)$
$-[(25)(2.03)]^2$

$E_1 = -0.0615$, use absolute value of 0.0615

$\overline{\gamma} = 0.008$ in/in (Table D4)

$\gamma = (\pi/L)[C_1 + (E_1 d/2)]$ (*Eq.* D4.1-15)

$\gamma = [\pi/(15)(12)][0.403 + (0.0615)(7)/2] = 0.0108 > \overline{\gamma} = 0.008$

Now, try a new value of $F_{n3} = 21$ ksi

Recalculating C_1 and E_1,

$C_1 = 0.301$
$E_1 = -0.0385$, use absolute value of 0.0385
$\gamma = [\pi/(15)(12)][0.301 + (0.0385)(7)/2] = 0.0076 < \overline{\gamma} = 0.008$

Interpolating, and trying a value of $F_{n3} = 21.5$ ksi

Recalculating C_1 and E_1,

$C_1 = 0.312$
$E_1 = -0.0408$, use absolute value of 0.0408
$\gamma = [\pi/(15)(12)][0.312 + (0.0408)(7)/2] = 0.0079 < \bar{\gamma} = 0.008$

Trying one final value of $F_{n3} = 21.6$ ksi

$C_1 = 0.317$
$E_1 = -0.0412$, use absolute value of 0.0412
$\gamma = [\pi/(15)(12)][0.317 + (0.0412)(7)/2] = 0.008$ *OK*

Calculate A_e at $F_n = 21.6$ ksi, the smallest value of F_{n1}, F_{n2}, and F_{n3}

Web:
$w = 6.663$ in.
$\lambda = (1.052/\sqrt{4})(6.663/0.075)\sqrt{21.6/29500} = 1.264$ (*Eq.* B2.1-4)

For $\lambda > 0.673$,

$\rho = (1 - 0.22/1.264)/1.264 = 0.653$ (*Eq.* B2.1-3)

$b = \rho w = (0.653)(6.663) = 4.351$ in. (*Eq.* B2.1-2)

Flange:
$w = 2.413$ in.
$w/t = 32.17$
$S = 1.28\sqrt{29500/21.6} = 47.3$ (*Eq.* B4-1)

$S/3 < w/t < S$ so,

$I_a/t^4 = 399\{[(w/t)/S] - 0.33\}^3$ (*Eq.* B4.2-6)
$I_a = (0.075)^4(399)\{[(32.17)/47.3] - 0.33\}^3$
$I_a = 0.000542$

Stiffener:
$w = 0.531$ in.
$I_s = 0.000936$ in.^{4}
$\lambda = (1.052/\sqrt{0.43})(0.531/0.075)\sqrt{21.6/29500} = 0.307$ (≤ 0.673) (*Eq.* B2.1-4)
$b = d_s' = 0.531$ in.
$d_s = d_s' = 0.531$ in. (for $I_s \geq I_a$) (*Eq.* B4.2-11)
$D/w = 0.275$
$k = [4.82 - 5(0.275)](0.000936/0.000542)^{1/2} + 0.43$ (*Eq.* B4.2-9)
$k = 4.96$

$5.25 - 5(0.275) = 3.88 < 4.96$, so use $k = 3.88$

$\lambda = (1.052/\sqrt{3.88})(32.17)\sqrt{21.6/29500} = 0.465 (\leq 0.673)$

$b = w = 2.413$ (*Eq.* B2.1-1)

$A_e = 1.003 - (6.663 - 4.351)(0.075) = 0.830$ in.^{2}

$P_a = A_e F_n/\Omega_c$ (*Eq.* D4.1-1)

$P_a = (0.830)(21.6)/1.92 = 9.34$ kips

2. Allowable Axial Load with 5 psf Lateral Load (Section D4.3)

$M_x = (5 \text{ psf})(2 \text{ ft. O.C.})(15 \text{ ft.})^2(12 \text{ in./ft.})/8 = 3375 \text{ in. lbs.}$

$(P/P_a) + (C_{mx}M_x/M_{ax}\alpha_x) + (C_{my}M_y/M_{ay}\alpha_y) \le 1.0$ (*Eq.* C5-1)

$M_y = 0$

Assume $C_{mx} = 1.0$ (braced against joint translation in the plane of loading, subject to transverse loading between supports with member ends unrestrained)

$1/\alpha_x = 1/[1 - (\Omega_c P/P_{cr})]$ (*Eq.* C5-4)

$\Omega_c = 1.92$

$P_{cr} = \pi^2 E I_b/(K_b L_b)^2 = [\pi^2(29500)(7.66)]/[(12)(15)]^2 = 68.8 \text{ kips}$

$P_a = 9.34 \text{ kips}$

$M_{ax} = M_{axo}$ = Allowable moments about the centroidal axes determined in accordance with Section C3.1 excepting lateral buckling provisions

Following Procedure I—Based on Initiation of Yielding

$M_{ax} = M_{nx}/\Omega_f$ where $\Omega_f = 1.67$ (*Eq.* C3.1-1)

$M_{nx} = S_{ex}F_y$ (*Eq.* C3.1.1-1)

$F_y = 50 \text{ ksi}$

Calculation of S_{ex} with extreme compression fiber at F_y

Stiffener (compression): Assume maximum stress is F_y initially, although actually it will be less.

$w = 0.531 \text{ in.}$

$\lambda = (1.052/\sqrt{0.43})(0.531/0.075)\sqrt{50/29500} = 0.468 \quad (\le 0.673)$ (*Eq.* B2.1-4)

$I_s = 0.000936 \text{ in.}^4$

$b = d'_s = 0.531 \text{ in.}$ (max. stress assumption OK)

$d_s = d'_s(I_s/I_a) = (0.531)(0.000936/0.00392) = 0.127 \text{ in.}$ (*Eq.* B4.2-11)

See below for calculation of I_a.

Flange (compression):

$w = 2.413 \text{ in.}$
$w/t = 32.17$

$S = 1.28\sqrt{29500/50} = 31.1$ (*Eq.* B4-1)

$w/t \ge S$ so,

$I_a/t^4 = [115(w/t)/S] + 5$ (*Eq.* B4.2-13)
$I_a = (0.075)^4\{[115(32.17)/31.1] + 5\} = 0.00392 \text{ in.}^4$
$D/w = 0.275$

$k = [4.82 - 5(0.275)](0.000936/0.00392)^{1/3} + 0.43$ (*Eq.* B4.2-9)
$k = 2.57$

$5.25-5(0.275)=3.88>2.57$ *OK*

$\lambda = (1.052/\sqrt{2.57})(32.17)\sqrt{50/29500}=0.869$ (*Eq.* B2.1-4)

$\rho = (1-0.22/0.869)/0.869=0.859$ (*Eq.* B2.1-3)

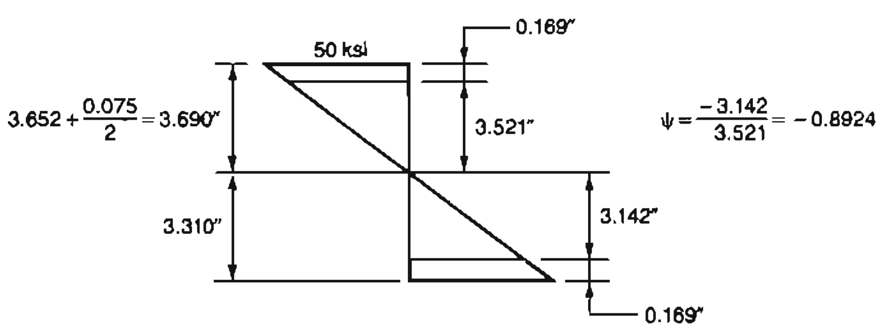
$b = (0.859)(2.413)=2.073$ in. (*Eq.* B2.1-2)

Assume the web is fully effective initially.

Segment	L (in.)	L_{eff} (in.)	y_{top} (in.) Distance to Center of Top Flange	$L_{eff}\,y_{top}$ (in.^{2})	$L_{eff}\,y_{top}^2$ (in.^{3})	I'_1 (in.^{3})
1	0.531	0.127	0.195	0.025	0.005	—
2	0.206	0.206	0.048	0.010	—	—
3	2.413	2.073	0	0	0	—
4	0.206	0.206	0.048	0.010	—	—
5	6.663	6.663	3.463	23.074	79.905	24.651
6	0.206	0.206	6.877	1.417	9.742	—
7	2.413	2.413	6.925	16.710	115.717	—
8	0.206	0.206	6.877	1.417	9.742	—
9	0.531	0.531	6.529	3.467	22.635	0.012
Sum	13.375	12.631		46.130	237.746	24.663

$\bar{y} = 46.130/12.631=3.652$ in.

$I_{x_{eff}} = 0.075(237.746+24.663-12.631\times 3.652^2)$
$= 7.05$ in.^{4}



Now, check to see if the web is fully effective.

$k = 4 + 2(1 - \psi)^3 + 2(1 - \psi)$
$k = 4 + 2(1 + 0.8924)^3 + 2(1 + 0.8924) = 21.34$

$f_1 = 3.521/3.690 \times 50 = 47.71$ ksi

$\lambda = (1.052/\sqrt{21.34})\,(6.663/0.075)\,\sqrt{47.71/29500} = 0.814$ (*Eq.* B2.1-4)

$\rho = (1 - 0.22/0.814)/0.814 = 0.896$ (*Eq.* B2.1-3)

$b_e = (0.896)\,(6.663) = 5.970$ in.

$b_2 = b_e/2 = 5.970/2 = 2.985$ in. (*Eq.* B2.3-2)

$b_1 = b_e/(3 - \psi) = 5.970/(3 + 0.8924) = 1.534$ in. (*Eq.* B2.3-1)

$b_1 + b_2 = 4.519 > 3.521$ (compressed portion of the web)

So, the web is fully effective.

$S_{ex} = 7.05/3.690 = 1.911$ in.^{3}
$M_{ax} = (1.911)\,(50) = 95.6$ kip-in.
$M_{ax} = (95.6)/1.67 = 57.2$ kip-in.

Using the interaction equation, and multiplying the combined loads by 0.75,

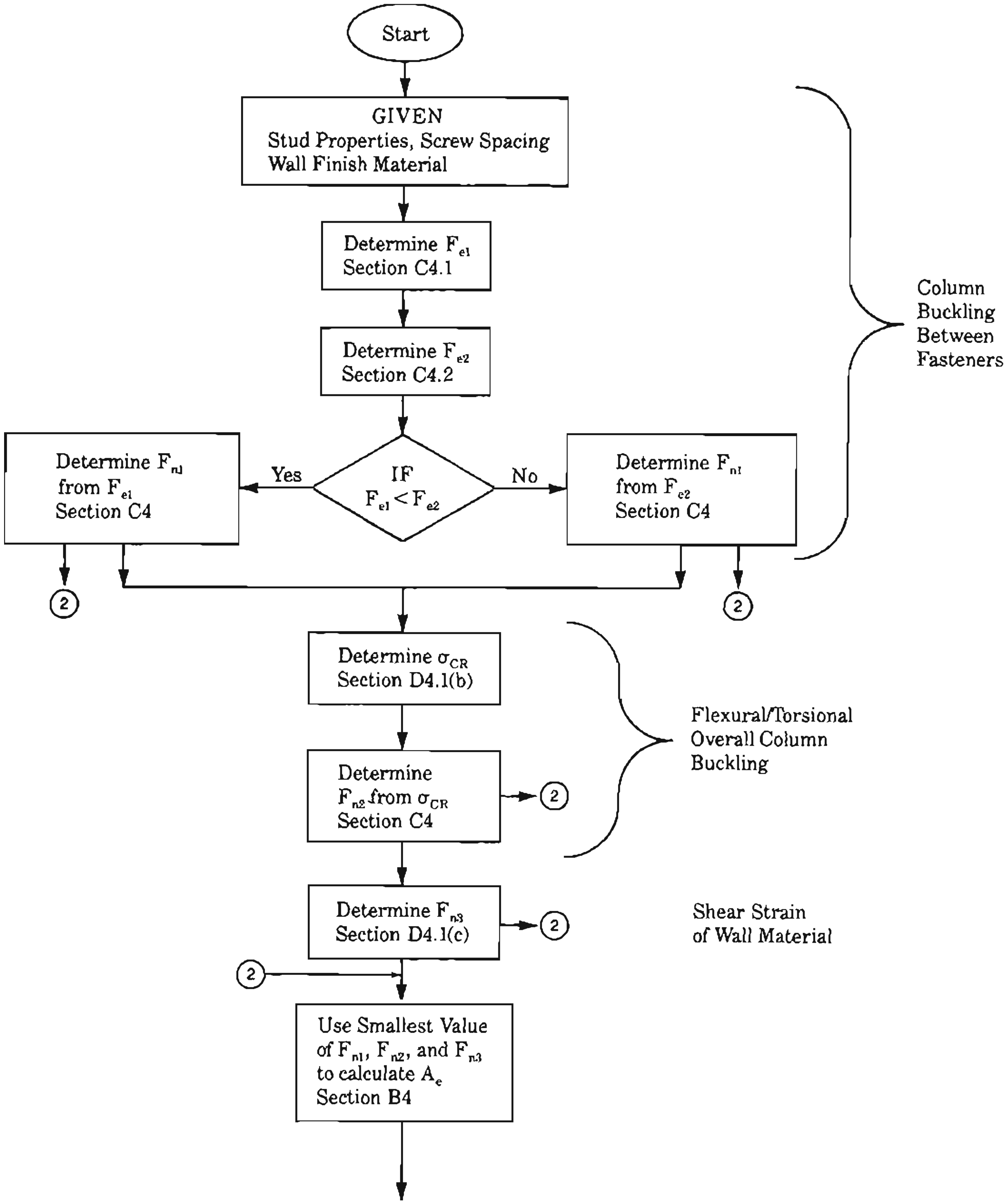
$$\frac{(0.75)P}{9.34} + \frac{[(3375)/1000]\,(0.75)}{(57.2)\,\{1 - [1.92P\,(0.75)/68.8]\}} \leq 1.0$$

Iterating with different values of P on the left hand side yields an allowable load,

$P_{allowable}$ (with 5 psf wind load) = 11.8 kips

The axial load alone controls, so $P_{allowable} = 9.34$ kips

Note that (*Eq.* C5-2) does not control in this case since the simply supported stud has zero end moments ($M_x = 0$) and $P_a < P_{ao}$ (allowable axial load for zero length stud).



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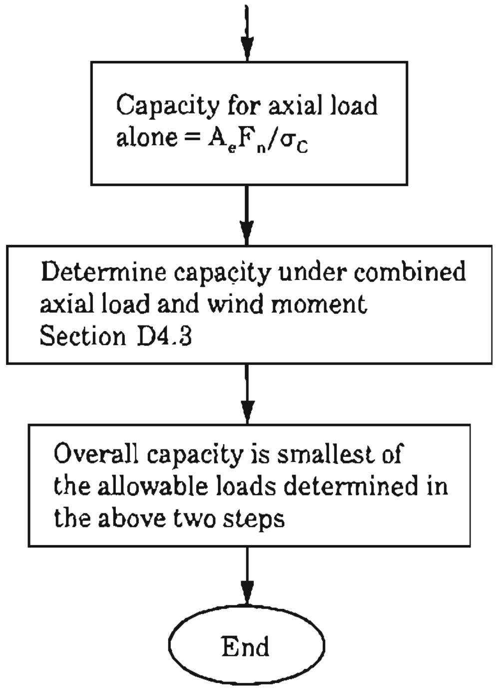
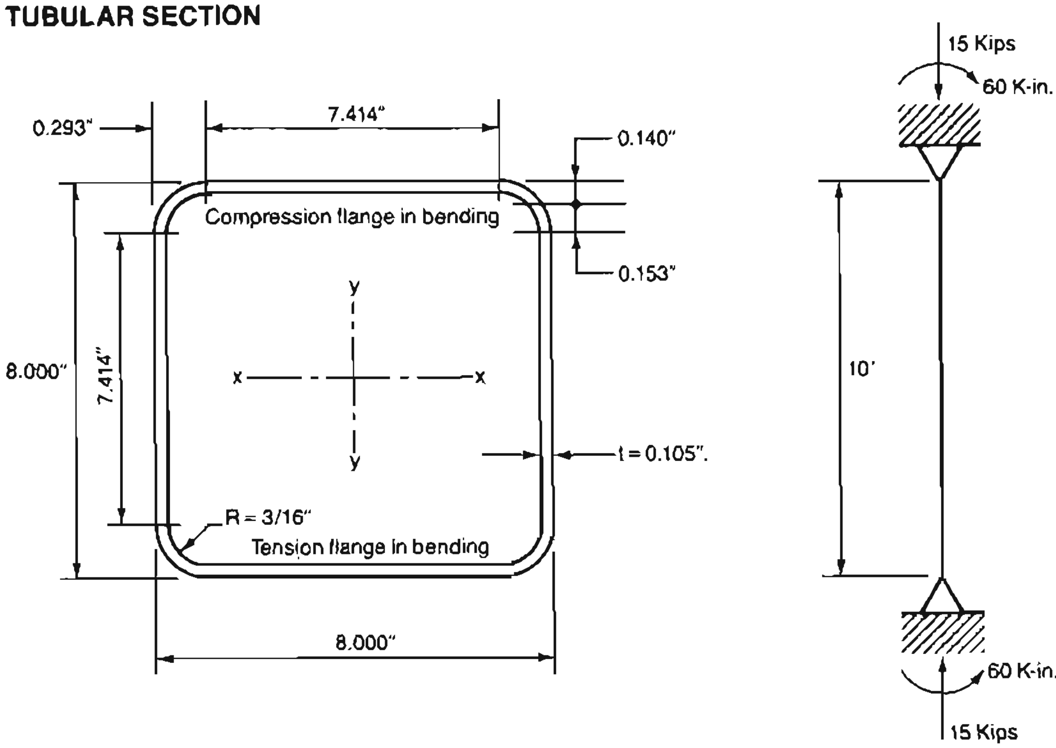


EXAMPLE NO. 17

TUBULAR SECTION



Given:

1. Steel: $F_y = 50$ ksi.
2. Section: $8 \times 8 \times 0.105$ Square Tube.
3. Unbraced length of column: 10 ft.
4. $K_x = K_y = 1.0$.
5. Axial load: P = 15 kips.
6. The eccentricity of axial load at each end of member, e_y, is 4 in. and member is bent in single curvature about x-axis.
7. $e_x = 0$.

Required: Check the adequacy of the given section.

Solution:

1. Full section properties:

   $r = R + t/2 = 3/16 + 0.105/2 = 0.240$ in.
   Length of arc, $u = 1.57\, r = 1.57 \times 0.240 = 0.377$ in.
   Distance of c.g. from center of radius, $c = 0.637r = 0.637 \times 0.240 = 0.153$ in.

   $I_x = I_y$ (doubly symmetric section)

Element	L (in.)	y Distance to Center of Section (in.)	Ly^2 (in.^{3})	I'_1 About Own Axis (in.^{3})
Flanges	$2 \times 7.414 = 14.828$	3.948	231.120	—
Corners	$4 \times 0.377 = 1.508$	3.860	22.469	—
Webs	$2 \times 7.414 = 14.828$	—	—	67.921
Sum	31.164		253.589	67.921

$A = Lt = 31.164 \times 0.105 = 3.272$ in.^{2}
$I' = L_y{}^2 + I'_1 = 253.589 + 67.921 = 321.510$ in.^{3}
$I_x = I_y = I't = 321.510 \times 0.105 = 33.759$ in.^{4}
$r_x = r_y = \sqrt{33.759/3.272} = 3.212$ in.
$S_x = I_x/4.000 = 33.759/4.000 = 8.440$ in.^{3}
$K_xL_x/r_x = 1.0(10 \times 12)/3.212 = 37.36 < 200$ *OK* [Section C4-(d)]

2. Determination of P_a (Section C4):

   Since the square tube is a doubly symmetric closed section, provisions of Section C4.1 apply, i.e., section is not subjected to torsional flexural buckling.

$F_e = \pi^2E/(K_xL_x/r_x)^2$ (*Eq.* C4.1-1)
$= (\pi^2 \times 29500)/(37.36)^2 = 208.60$ ksi

$F_y/2 = 50/2 = 25.00$ ksi

For $F_e > F_y/2$:
$F_n = F_y(1 - F_y/4F_e)$ (*Eq.* C4-3)
$= 50[1 - 50.00/(4 \times 208.60)]$
$= 47.00$ ksi

$w = 7.414$ in., $w/t = 7.414/0.105 = 70.61 < 500$ *OK* [Section B1.1-(a)-(2)]

$k = 4.00$ [Section B2.1-(a)]

$\lambda = (1.052/\sqrt{k})\,(w/t)\,\sqrt{f/E}$, $f = F_n$ (*Eq.* B2.1-4)
$= (1.052/\sqrt{4.00})\,(70.61)\,\sqrt{47/29500} = 1.482 > 0.673$

$\rho = (1 - 0.22/\lambda)/\lambda$ (*Eq.* B2.1-3)
$= (1 - 0.22/1.482)/1.482 = 0.575$

$b = \rho w$ (*Eq.* B2.1-2)
$= 0.575 \times 7.414 = 4.263$ in.

$A_e = A - 4(w - b)t$
$= 3.272 - 4(7.414 - 4.263)(0.105)$
$= 1.949$ in.^{2}

$P_n = A_eF_n$ (*Eq.* C4-2)
$= 1.949 \times 47.00$
$= 91.60$ kips

$\Omega_c = 1.92$

$P_a = P_n/\Omega_c$ (*Eq.* C4-1)
$= 91.60/1.92$
$= 47.71$ kips

3. $P/P_a = 15/47.71 = 0.314 > 0.15$

   Must check both interaction equations (*Eq.* C5-1), (*Eq.* C5-2).

4. Determination of P_{ao} (Section C4 for L = 0):

   For this square tube with L = 0, $K_xL_x/r_x = 0$, therefore, $F_e = \infty$

   $F_e > F_y/2 = 25.00$ ksi, $F_y/4F_e = 0$

   $F_n = 50(1-0) = 50$ ksi

   $\lambda = (1.052/\sqrt{4.00})(70.61)\sqrt{50/29500} = 1.529 > 0.673$

   $\rho = (1-0.22/1.529)/1.529 = 0.560$

   $b = 0.560 \times 7.414 = 4.152$ in.

   $A_e = 3.272 - 4(7.414 - 4.152)(0.105)$
   $= 1.902$ in.^{2}

   $P_n = 1.902 \times 50 = 95.10$ kips

   $P_{ao} = P_n/\Omega_c = 95.10/1.92$
   $= 49.53$ kips.

5. Determination of M_x, M_y (Section C5):

   Since the section is doubly symmetric, the centroidal axes of the effective section at P_a are the same as those of the full section.

   $M_x = P_{ey} = 15 \times 4 = 60$ kips-in.
   $M_y = P_{ex} = 0$

   Since $M_y = 0$, the interaction equations (*Eq.* C5-1) and (*Eq.* C5-2) reduce to the following

   $(P/P_a) + (C_{mx}/M_{ax}) \times (M_x/\alpha_x) \leq 1.0$ (*Eq.* C5-1)

   $(P/P_{ao}) + (M_x/M_{axo}) \leq 1.0$ (*Eq.* C5-2)

6. Determination of M_{nx} (Section C3):

   M_{nx} shall be taken as the smaller of the ultimate moments calculated according to Section C3.1.1 and C3.1.2:

   (a) Section C3.1.1: M_{nx} will be calculated on the basis of initiation of yielding.

   Computation of I_x:
   For the first approximation, assume a compression stress of $f = F_y = 50$ ksi in the compression flange, and that the web is fully effective.

   Compression flange: k = 4.00 (stiffened compression element supported by a web on each longitudinal edge)

   $w/t = 7.414/0.105 = 70.61 < 500$ *OK* [Section B1.1-(a)-(2)]

   $\lambda = (1.052/\sqrt{4.00})(70.61)\sqrt{50/29500} = 1.529 > 0.673$

   $\rho = (1-0.22/1.529)/1.529 = 0.560$

   $b = 0.560 \times 7.414$
   $= 4.152$ in.

Effective section properties about x-axis:

Element	L Effective Length (in.)	y Distance from Top Fiber (in.)	Ly (in.^{2})	Ly^2 (in.^{3})	I'_1 About Own Axis (in.^{3})
Webs	14.828	4.000	59.312	237.248	67.921
Upper Corners	0.754	0.140	0.106	0.015	—
Lower Corners	0.754	7.860	5.926	46.582	—
Compression Flange	4.152	0.053	0.220	0.012	—
Tension Flange	7.414	7.948	58.926	468.348	—
Sum	27.902		124.490	752.205	67.921

Distance from top fiber to x-axis is $y_{cg} = 124.490/27.902 = 4.462$ in.

Since distance of top compression fiber from neutral axis is greater than one half the section depth (i.e. $4.462 > 4.000$), a compression stress of 50 ksi will govern as assumed (i.e., initial yielding is in compression).

To check if the web is fully effective (Section B2.3)

$f_1 = [(4.462 - 0.293)/4.462](50) = 46.72$ ksi (compression)

$f_2 = -[(3.538 - 0.293)/4.462](50) = -36.36$ ksi (tension)

$$\psi = f_2/f_1 = -36.36/46.72 = -0.778$$

$$k = 4 + 2(1-\psi)^3 + 2(1-\psi) = 4 + 2[1-(-0.778)]^3 + 2[1-(-0.778)] = 18.798$$

$h = w = 7.414$ in., $h/t = w/t = 7.414/0.105 = 70.61$
$h/t = 70.61 < 200$ *OK* [Section B1.2-(a)]

$$\lambda = (1.052/\sqrt{18.798})(70.61)\sqrt{46.72/29500} = 0.682 > 0.673$$

$$\rho = (1 - 0.22/0.682)/0.682 = 0.993$$

$$b_e = 0.993 \times 7.414 = 7.362 \text{ in.}$$

$$b_2 = b_e/2 = 7.362/2 = 3.681 \text{ in.} \qquad (Eq.\ B2.3\text{-}2)$$

$$b_1 = b_e/(3-\psi) = 7.362/[3-(-0.778)] = 1.949 \text{ in.} \qquad (Eq.\ B2.3\text{-}1)$$

Compression portion of the web calculated on the basis of the effective section = $y_{cg} - 0.293 = 4.462 - 0.293 = 4.169$ in.

Since $b_1 + b_2 = 5.630$ in. > 4.169 in., $b_1 + b_2$ shall be taken as 4.169 in. This verifies the assumption that the web is fully effective.

$$I'_y = Ly^2 + I'_1 - Ly_{cg}^2 = 752.205 + 67.921 - 27.902(4.462)^2 = 264.613 \text{ in.}^3$$

$$\text{Actual } I_y = I'_x t = 264.613 \times 0.105 = 27.784 \text{ in.}^4$$

$$S_e = I_x/y_{cg} = 27.784/4.462 = 6.227 \text{ in.}^3$$

$M_{nx} = S_eF_y$ (*Eq.* C3.1.1-1)
$= 6.227 \times 50$
$= 311.35$ kips-in.

(b) Section C3.1.2: M_{nx} will be calculated on the basis of lateral buckling strength. However for this square tube (closed box-type member) the provisions of Section C3.1.2 do not apply.

Therefore
$M_{nx} = 311.35$ kips-in.
$\Omega_f = 1.67$

$M_{ax} = M_{nx}/\Omega_f$ (*Eq.* C3.1-1)
$= 311.35/1.67$
$= 186.44$ kips-in.

7. Determination of M_{axo}:

M_{axo} is determined according to Section C3.1 excepting the provisions of Section C3.1.2. Thus for our example (where section strength is based on the initiation of yielding) $M_{axo} = 186.44$ kips-in.

8. $C_{mx} = 0.6 - 0.4(M_1/M_2) \geq 0.4$

$M_1/M_2 = -(60/60) = -1.0$ (single curvature)

$0.6 - 0.4(M_1/M_2) = 0.6 - 0.4(-1.0)$
$= 1.0 > 0.4$

therefore $C_{mx} = 1.0$

9. Determination of $1/\alpha_x$:

$\Omega_c = 1.92$

$P_{cr} = \pi^2 EI_x/(K_xL_x)^2$ (*Eq.* C5-5)

$I_x = 33.759$ in.^{4}

$K_xL_x = 1.0(10 \times 12) = 120$ in.

$P_{cr} = [\pi^2(29500)(33.759)]/(120)^2 = 682.57$ kips

$1/\alpha_x = 1/[1 - (\Omega_cP/P_{cr})]$ (*Eq.* C5-4)
$= 1/[1 - (1.92 \times 15/682.57)] = 1.044$

$\alpha_x = 0.958$

10. Check interaction equations:

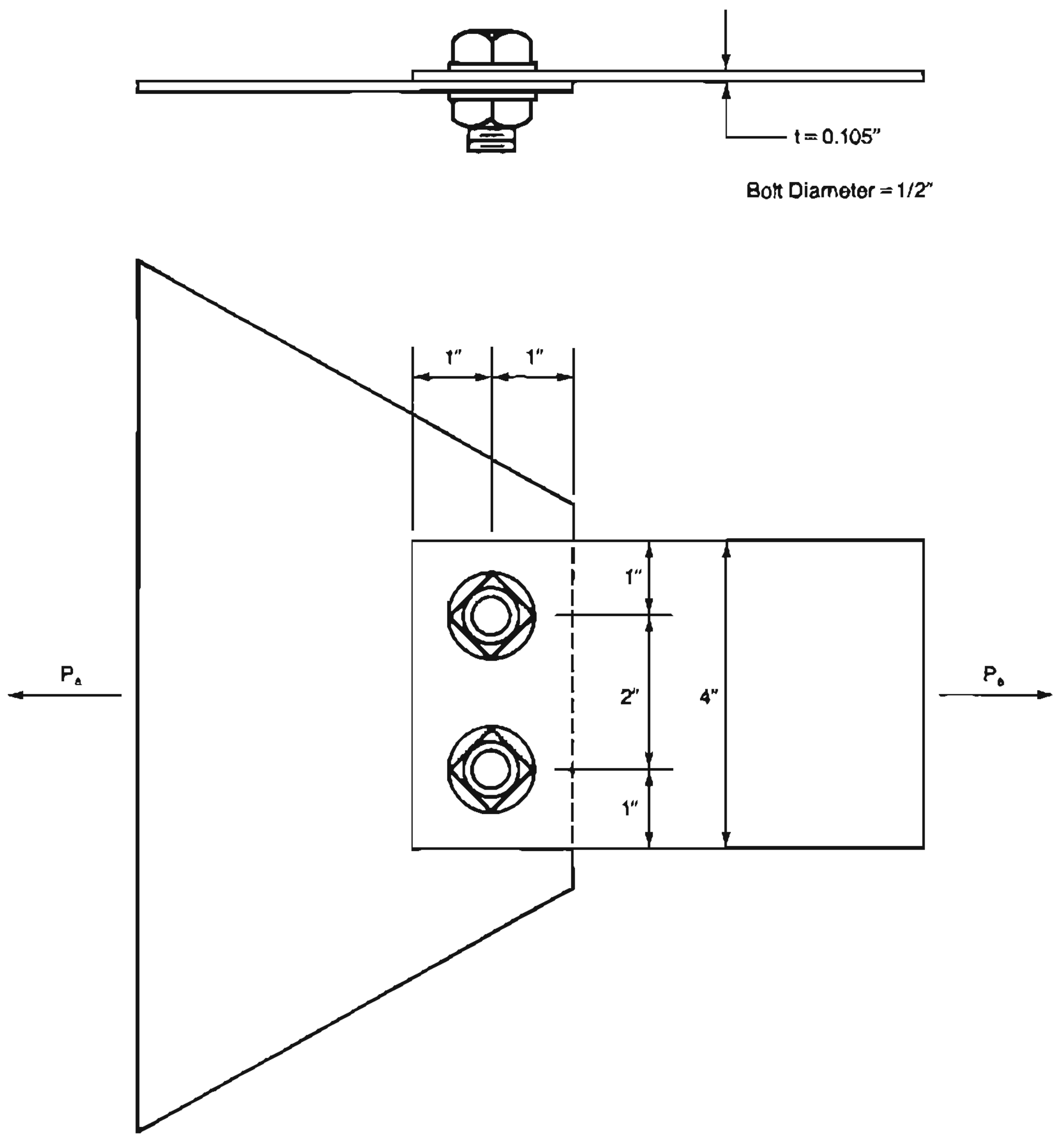
$(P/P_a) + (C_{mx}/M_{ax}) \times (M_x/\alpha_x) = (15/47.71) + (1.0/186.44) \times (60/0.958)$ (*Eq.* C5-1)
$= 0.314 + 0.336$
$= 0.650 < 1.0$ *OK*

$(P/P_{ao}) + (M_x/M_{axo}) = (15/49.53) + (60/186.44)$ (*Eq.* C5-2)
$= 0.303 + 0.322 = 0.625 < 1.0$ *OK*

Therefore the section is adequate for the applied loads.

EXAMPLE NO. 18

FLAT SECTION w/BOLTED CONNECTION



Given:
1. Steel: $F_y = 33$ ksi, $F_u = 45$ ksi.
2. Bolts conforming to ASTM A307 with washers under bolt head and nut.
3. Detail of connection shown in sketch.

Required: Determine the maximum allowable load, P_a.

Solution:

Thickness of thinnest part connected, t

$t = 0.105 < 3/16 = 0.188$ in.

Therefore, Section E3 applies.

1. Allowable load based on tension on net section.

Tension force on net section of bolted connection shall not exceed T_a from Section C2:

A_n—based on Table E3

$$A_n = 0.105[4 - 2(1/2 + 1/16)] = 0.302 \text{ in.}^2$$

$$T_n = A_n F_y = (0.302)(33) = 9.97\text{k} \qquad (Eq.\ C2\text{-}2)$$

$$T_a = T_n/\Omega_t = 9.97\text{k}/1.67 \qquad (Eq.\ C2\text{-}1)$$

$T_a = 5.97$k trial value

or P_a from Section E3.2:

Since $t = 0.105 < 3/16$ and washers are provided under both bolt head and nut

$$F_t = (1.0 - 0.9r + 3rd/s)F_u \le F_u \qquad (Eq.\ E3.2\text{-}2)$$

where in this case:

$r = 2(P_a/2)/P_a = 1$
$d = 0.5$ in.
$s = 2$ in.

$$F_t = [1.0 - 0.9(1) + 3(1)(0.5)/2](45) = 38.25 \text{ ksi} < 45 \text{ ksi} \quad OK$$

$\Omega_t = 2.22$ for single shear connection

$P_n = A_n F_t = (0.302)(38.25) = 11.55$k

$$P_a = P_n/\Omega_t = 11.55\text{k}/2.22 = 5.20\text{k} \qquad (Eq.\ E3.2\text{-}1)$$

$P_a = 5.20$k trial value

2. Allowable load based on bearing (Section E3.3)

For single shear with washers under bolt head and nut; $0.024 \text{ in.} \le t = 0.105 \text{ in.} < 3/16$ in.

From Table E3.3(A)

$F_p = 3.00\,F_u = 3.00(45) = 135$ ksi

$\Omega_b = 2.22$

$$P_n = F_p dt = (135)(0.5)(0.105) = 7.09\text{k/bolt} \qquad (Eq.\ E3.3\text{-}2)$$

$$P_a = P_n/\Omega_b = 7.09/2.22 = 3.19\text{k/bolt} \qquad (Eq.\ E3.3\text{-}1)$$

$P_a = (2 \text{ bolts})(3.19\text{k/bolt}) = 6.38$k trial value.

3. Allowable load based on bolt shear (Section E3.4)

$P_a = A_b F$ (*Eq.* E3.4-1)

$A_b = (\pi/4)(0.5)^2 = 0.196 \text{ in.}^2$

$F = F_v = 10$ ksi [Section E3.4(a), $d \leq ½$ in.]

$P_a = (10)(0.196)$
$= 1.96$k/bolt

$P_a = (2 \text{ bolts})(1.96\text{k/bolt})$
$= 3.92$k trial value.

4. Comparing the values from 1, 2, and 3 above, the shear stress in the bolts controls and

$P_a = 3.92$k

5. Check requirements for spacing and edge distance (Section E3.1)

(a) $F_u/F_y = 45/33 = 1.36 > 1.15$

For $F_u/F_y > 1.15$, $\Omega_e = 2.0$

$e = P/F_u t = (3.92\text{k}/2)/(45 \times 0.105) = 0.41$ in. (*Eq.* 3.1-2)

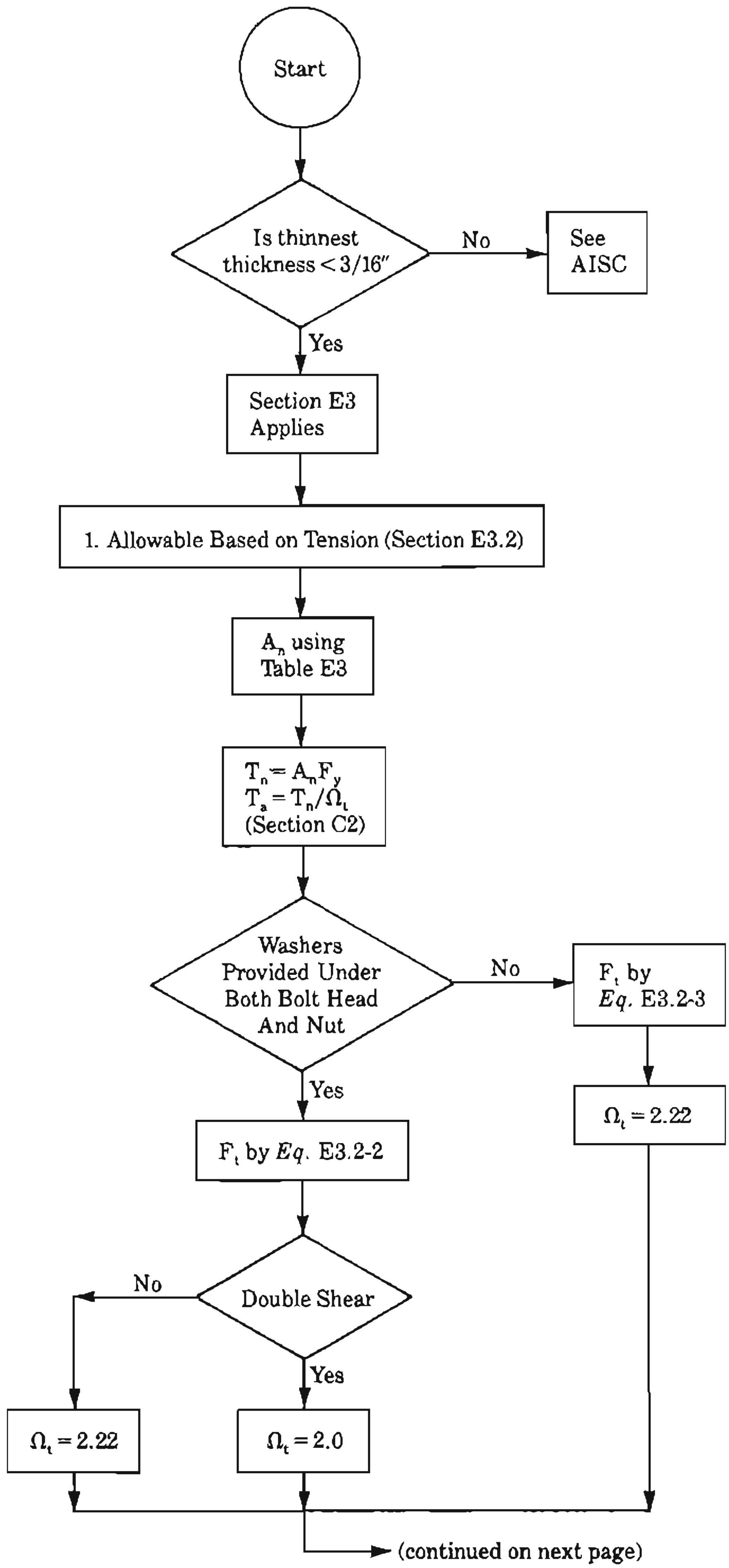
$e_{min} = e\Omega_e$ (*Eq.* 3.1-1)
$= (0.41)(2.0)$
$= 0.82 < 1.00$ *OK*

(b) Distance between bolt hole centers must be greater than 3d.

$3d = 3(0.5) = 1.5,\ 2 > 1.5$ *OK*

(c) Distance between bolt hole center and edge of connecting member must be greater than 1.5d.

$1.5d = 1.5(0.5) = 0.75,\ 1 > 0.75$ *OK*



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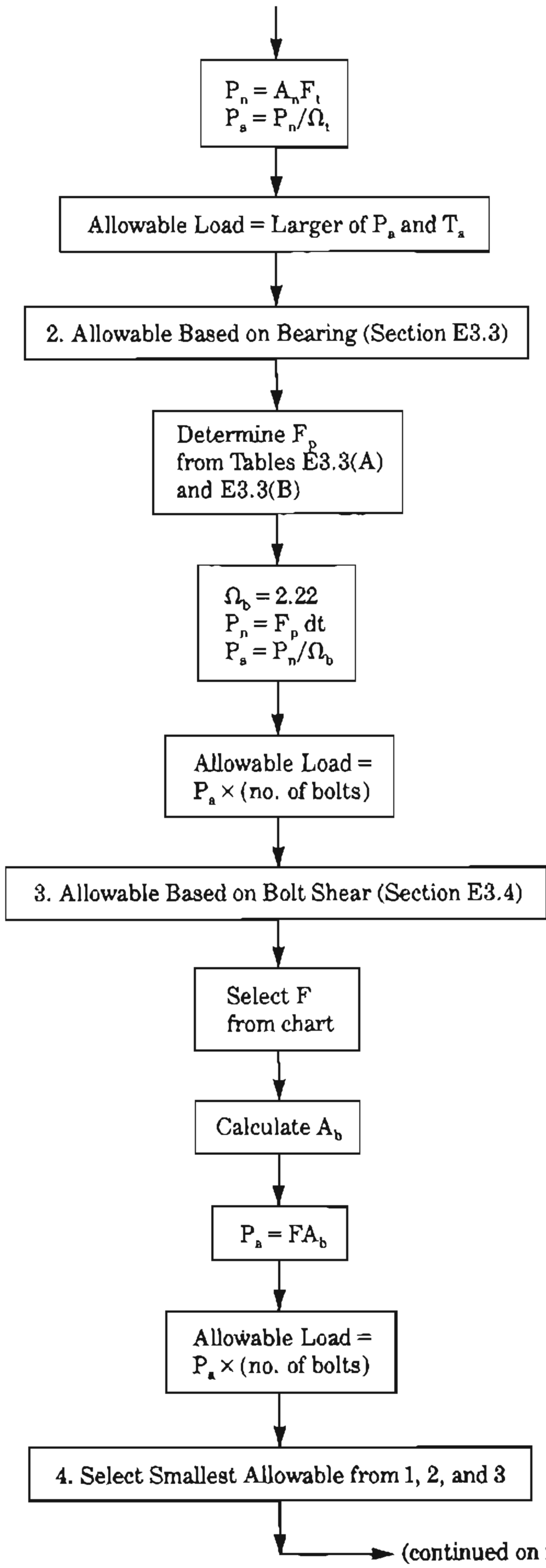

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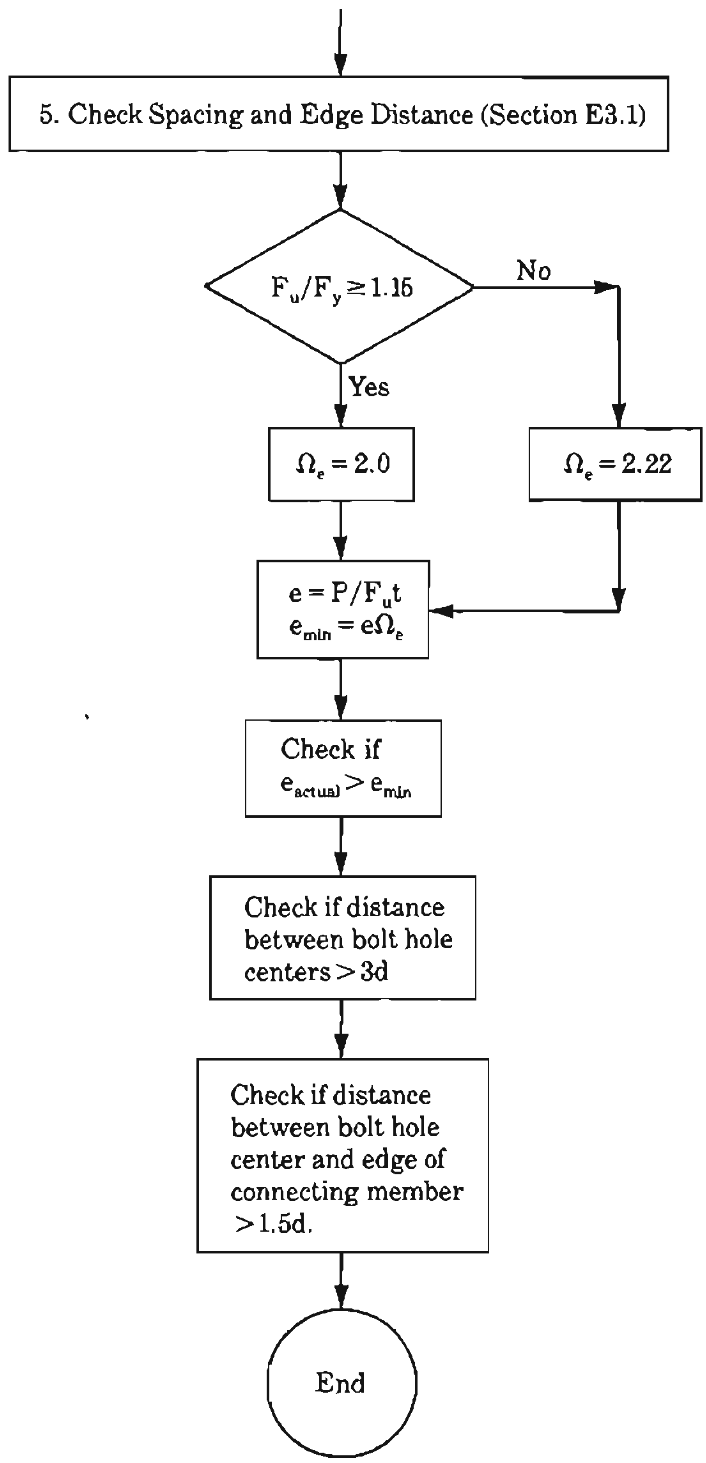
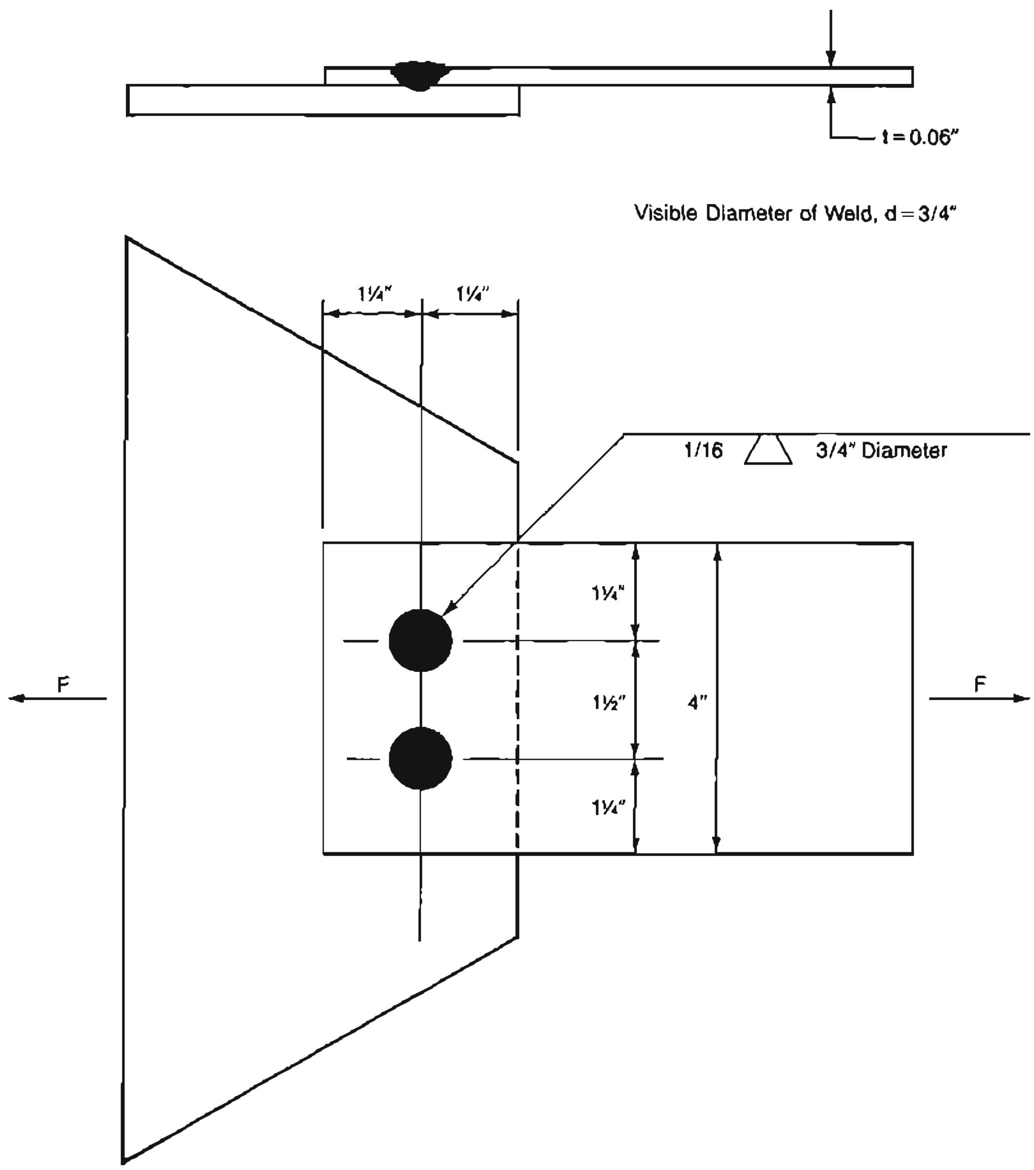

5. Check Spacing and Edge Distance (Section E3.1)
$F_u/F_y \geq 1.15$
No
Yes
$\Omega_e = 2.0$
$\Omega_e = 2.22$
$e = P/F_u t$
$e_{min} = e\Omega_e$
Check if $e_{actual} > e_{min}$
Check if distance between bolt hole centers > 3d
Check if distance between bolt hole center and edge of connecting member > 1.5d.
End

EXAMPLE NO. 19

FLAT SECTION w/ARC SPOT WELDED CONNECTION



Given:
1. Steel: $F_y = 50$ ksi, $F_u = 65$ ksi.
2. Total Load, F = 4.6k.
3. Detail of connection shown in sketch.

Required: Design the connection to transmit F = 4.6k using arc spot welds having ¾ in. visible diameter.

Solution:

1. Weld Dimensions

$$\begin{aligned} d &= 0.75 \text{ in.} \\ d_a &= d - t \\ &= 0.75 - 0.06 \\ &= 0.69 \text{ in.} \\ d_e &= 0.7d - 1.5t \quad \text{but} \le 0.55d \qquad (Eq.\ E2.2\text{-}5) \\ &= 0.7(0.75) - 1.5(0.06) = 0.44 \end{aligned}$$

$0.55d = 0.55(0.75) = 0.41$ in.

$0.44 > 0.41$, use $d_e = 0.41 > 3/8$ in. *OK*

2. Determine number of arc spot welds required.

(a) $P = 0.625\, d_e^2\, F_{xx}$ (*Eq.* E2.2-1)

Using E60 electrode, $F_{xx} = 60$ ksi

$P = 0.625(0.41)^2(60) = 6.30$k/weld

(b) Compute $d_a/t = 0.69/0.06 = 11.5$

Compute $\sqrt{E/F_u} = \sqrt{29500/65} = 21.3$

For $d_a/t = 11.5 < 0.815\sqrt{E/F_u} = 17.4$

$$\begin{aligned} P &= 2.20\, t d_a F_u \qquad (Eq.\ E2.2\text{-}2) \\ &= 2.20(0.06)(0.69)(65) \\ &= 5.92\text{k/weld} \end{aligned}$$

$P = 5.92$k/weld controls

$P_a = P/\Omega_w = 5.92/2.50 = 2.37$k/weld (*Eq.* E2-1)

Number of welds = 4.6k/(2.37k/weld) = 1.94 welds, use 2.

3. Check edge distance and spacing requirements

(a) $F_u/F_y = 65/50 = 1.3 > 1.15$

For $F_u/F_y > 1.15$, $\Omega_e = 2.0$

$e = P/F_u t = (4.6k/2)/[(65)(0.06)] = 0.59$ in. (*Eq.* E2.2-7)

$e_{min} = e\Omega_e = (0.59)(2.0) = 1.18$ in.

1.25 edge distance > 1.18 *OK*

(b) Edge distance shall not be less than 1.5d.

$1.5d = 1.5(0.75) = 1.13$, $1.25 > 1.13$ *OK*

(c) Clear distance between weld and end of member shall not be less than 1.0d.

$1.0d = 1(0.75) = 0.75$ in.

Clear distance $= 1.25 - 0.375 = 0.875 > 0.75$ *OK*

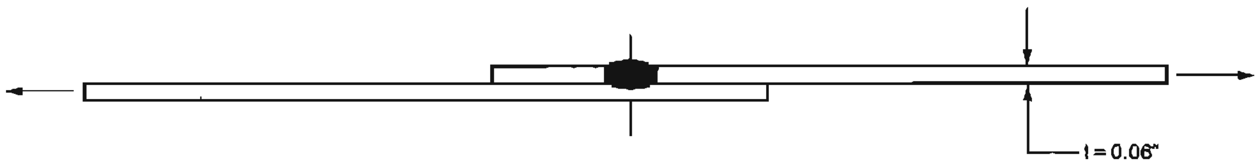
(d) Thinnest connected part, $t = 0.06 < 0.15$ *OK*

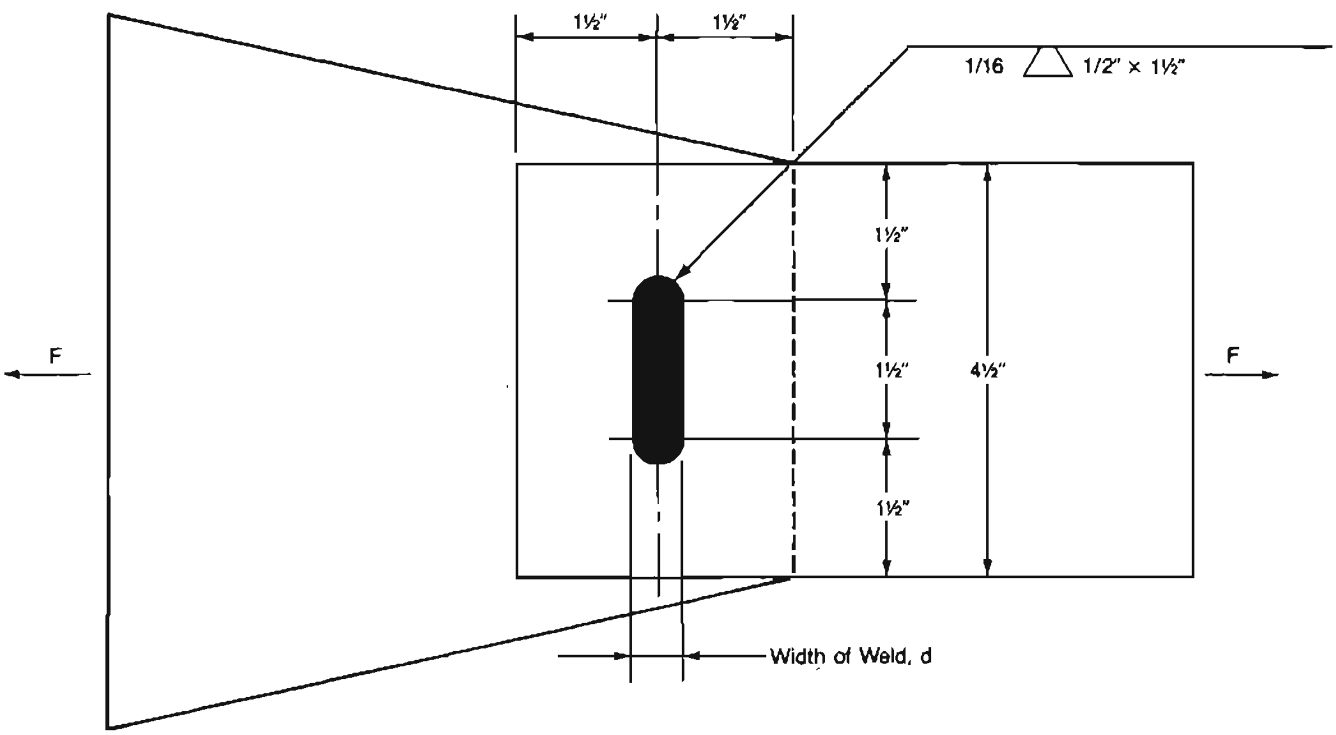
4. Use 2-3/4 in. diameter spot welds in the configuration shown. No weld washers required because

$t = 0.06 > 0.028$

EXAMPLE NO. 20

FLAT SECTION w/ARC SEAM WELDED CONNECTION





Given: 1. Steel: $F_y = 50$ ksi, $F_u = 65$ ksi.
2. Total Load, $F = 2.8k$.
3. Detail of connection shown in above sketch.

Required: Design the connection to transmit $F = 2.8k$ using Arc Seam Welds. Try d = ½ in.

Solution:

1. Load shall not exceed either

$P = [(d_e^2/4) + (Ld_e/3)]2.5F_{xx}$ (*Eq.* E2.3-1)

Try E60 electrode, $F_{xx} = 60$ ksi

$L = 1.5$, or maximum 3d, $3(0.5) = 1.5$ in. *OK*

$d_a = 0.5 - 0.06 = 0.44$ in. (*Eq.* E2.3-3)

$d_e = 0.7d - 1.5t$ (*Eq.* E2.3-5)
$= 0.7(0.5) - 1.5(0.06)$
$= 0.260$ in.

$P = \{[(0.26)^2/4] + [1.5(0.26)/3]\}2.5(60) = 22.0k$
OR
$P = 2.5tF_u(0.25L + 0.96d_a)$ (*Eq.* E2.3-2)
$= 2.5(0.06)(65)[0.25(1.5) + 0.96(0.44)]$
$= 7.77k$

Using the lesser $P = 7.77k$

$P_a = 7.77/2.5 = 3.11k > 2.8k$ req'd *OK* (*Eq.* E2-1)

2. Determine minimum edge distance in line of force.

(a) $F_u/F_y = 65/50 = 1.3 > 1.15$

$e = P/F_ut = 2.8k/[(65)(0.06)] = 0.72$ in. (*Eq.* E2.2-7)

$e_{min} = e\Omega_e = (0.72 \times 2.0) = 1.44$ in.

1.5 edge distance > 1.44 in. *OK*

(b) Edge distance shall not be less than 1.5d.

$1.5d = 1.5(0.5) = 0.75$, $1.5 > 0.75$ in. *OK*

(c) Clear distance between weld and end of member shall not be less than 1.0d

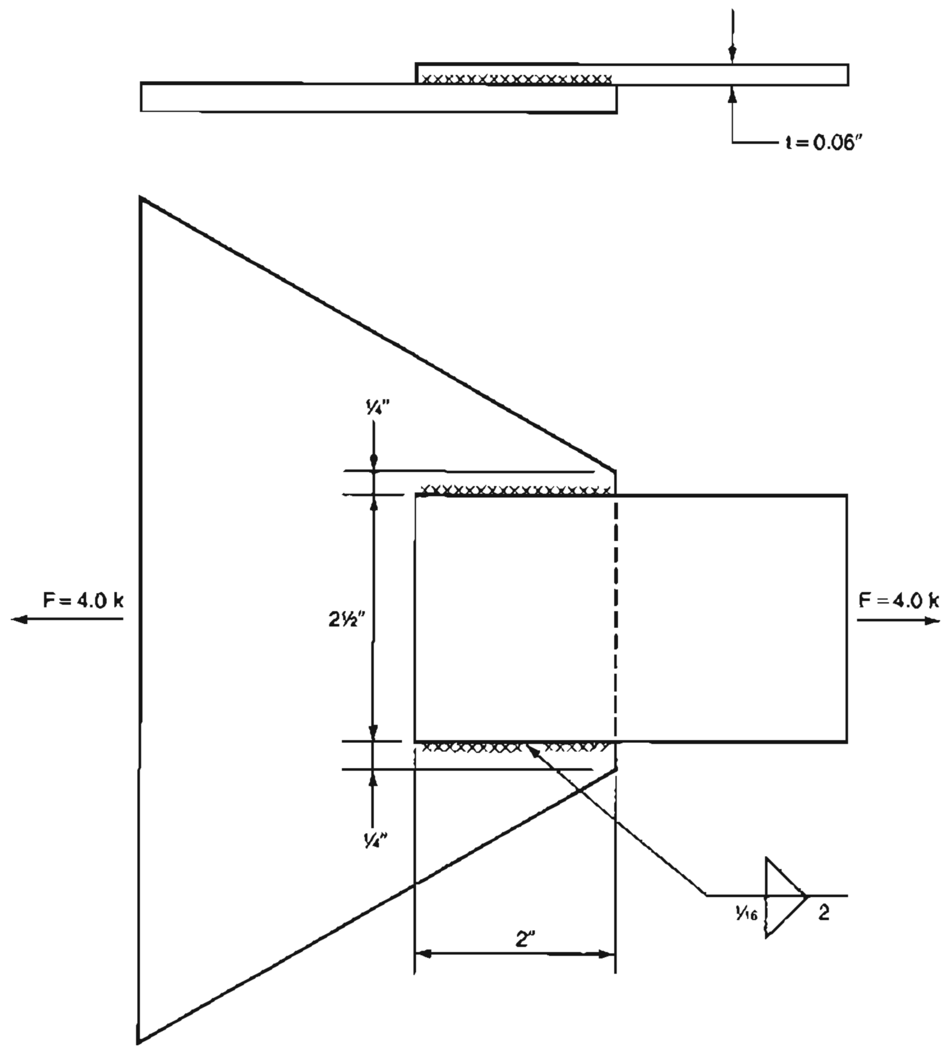
$1.0d = 1.0(0.5) = 0.5$ in.

Clear distance $= 1.5 - 0.25 = 1.25 > 0.5$ in. *OK*

3. Use arc seam welded connection per sketch with E60 electrode and d = ½ in.

EXAMPLE NO. 21

FLAT SECTION w/LAP FILLET WELDED CONNECTION



Given:
1. Steel: $F_y = 50$ ksi, $F_u = 65$ ksi.
2. Total Load, F = 4.0k.
3. Detail of connection shown above in sketch.

Required: Check to see if longitudinal fillet welded connection is adequate to transmit F = 4.0k.

Solution:

1. $L/t = 2/0.06 = 33.33 > 25$

   For $L/t \geq 25$,

   $$P = 0.75tLF_u \qquad (Eq.\ E2.4\text{-}2)$$
   $$= 0.75(0.06)(2)(65)$$
   $$= 5.85k$$

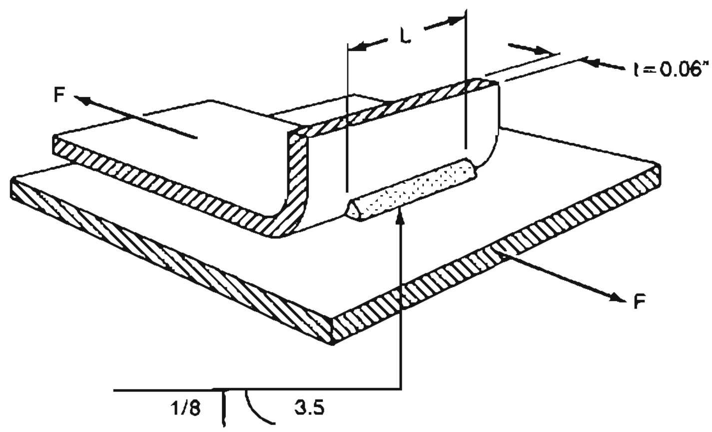
2. Note: $t = 0.06 < 0.150$. Therefore, (*Eq.* E2.4-4) does not apply.

3. $P_a = P/\Omega_w = 5.85/2.50 = 2.34k$ (*Eq.* E2-1)

   (2.34k/weld) (2 welds) = 4.68k > 4.0k *OK*

EXAMPLE NO. 22

FLAT SECTION w/SINGLE FLARE BEVEL GROOVE WELDED CONNECTION



Given:
1. Steel: $F_y = 50$ ksi, $F_u = 65$ ksi.
   Total Load, F = 4.0 kips.
2. Detail of connection shown in above sketch.
3. Transverse loading.

Required: Design the welded connection to transfer F = 4.0 kips.

Solution:
1. Solve for P using $P_a = P/\Omega_w = F$ (*Eq.* E2-1)

   $P = (4.0\text{k})(2.5)$
   $= 10.0\text{k}$

2. For flare-bevel groove welds, transverse loading, the load P shall not exceed

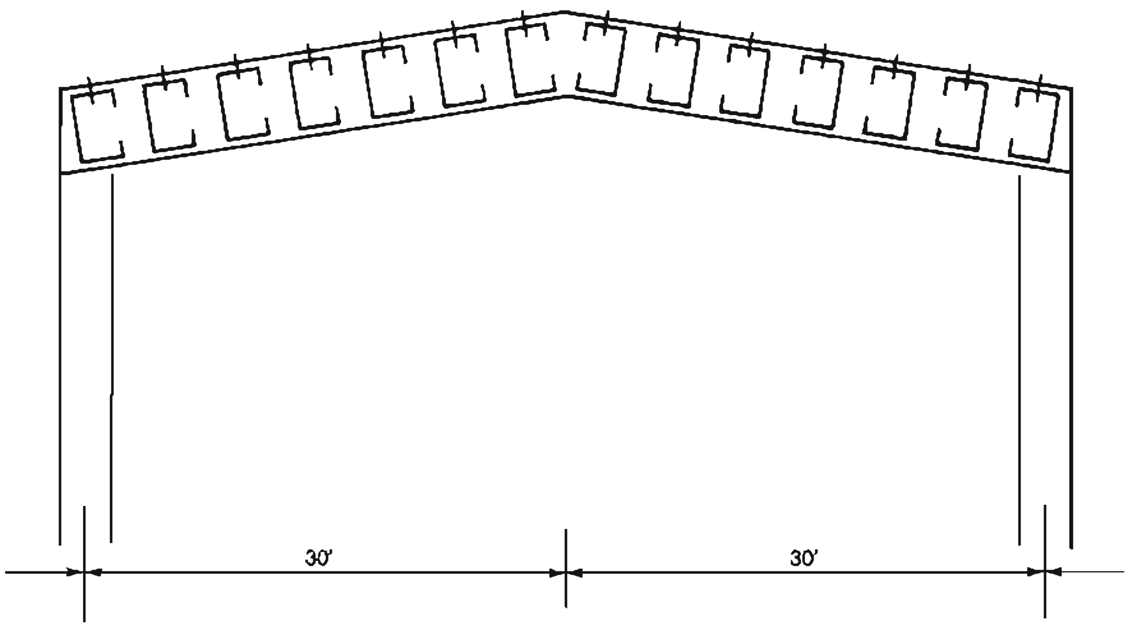
   $P = 0.833tLF_u$ (*Eq.* E2.5-1)

   Solve for L
   $L = P/0.833tF_u = 10.0\text{k}/[0.833(0.06)(65)]$
   $L = 3.08$ in.

3. Use 3.5 in. long flare bevel groove weld per sketch.
4. Size of weld ⅛" (¹⁄₁₆ min)

EXAMPLE NO. 23

C-SECTION BRACING UNDER GRAVITY LOADING



Given:
1. Steel: $F_y = 50$ ksi.
2. 60 ft. wide building, 20 ft. bays, simply supported purlins on 5-foot centers.
3. Roof slope 1:12.
4. Same channel as in ex. # 15.
5. Dead and live load: 17.6 psf

Required: Design of bracings of the roof system under gravity loads, using D3.2.1 of Specification.

Solution:

$P = 0.05W$

$W = 17.6 \times 30 \times 20 = 10560$ lbs.

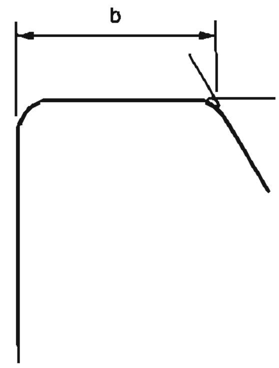
$P = 0.05(10560) = 528$ lbs.

A total restraint force of 528 lbs. must be supplied in each bay. It is up to the designer to decide what devices can be used to supply this force.

EXAMPLE NO. 24

Given: Same roof system as Example 23, with Z-section instead of channel. Z-section is the one used in Example 4.

Required: Design of bracings of the roof system under gravity loads, using D3.2.1 of Specification.



Solution:

$b = 1.689$ in.
$d = 6.000$ in.
$t = 0.060$ in.
$n_p = 7$
$\theta = 4.76°$, $\tan\theta = 0.083$
$W = 17.6 \times 30 \times 20 = 10560$ lbs.

1. Single span system with restraints at supports.

$$P_L = 0.5\left[\frac{0.220b^{1.5}}{n_p^{0.72}\, d^{0.9}\, t^{0.6}} - \tan\theta\right]W \qquad (Eq.\ D3.2.1\text{-}1)$$

$$P_L = 0.5\left[\frac{0.22(1.689)^{1.5}}{7^{0.72} \times 6.000^{0.9} \times 0.060^{0.6}} - 0.0833\right]10560$$

$P_L = 238$ lbs.

2. Single span system with third point restraints:

$$P_L = 0.5\left[\frac{0.474b^{1.22}}{n_p^{0.57}\, d^{0.89}\, t^{0.33}} - \tan\theta\right]W \qquad (Eq.\ D3.2.1\text{-}2)$$

$$P_L = 0.5\left[\frac{0.474(1.689)^{1.22}}{7^{0.57} \times 6.000^{0.89} \times 0.060^{0.33}} - 0.0833\right]10560$$

$P_L = 365$ lbs.

3. Single span system with midspan restraint:

$$P_L = 0.5\left[\frac{0.224b^{1.32}}{n_p^{0.65}\, d^{0.83}\, t^{0.50}} - \tan\theta\right]W \qquad (Eq.\ D3.2.1\text{-}3)$$

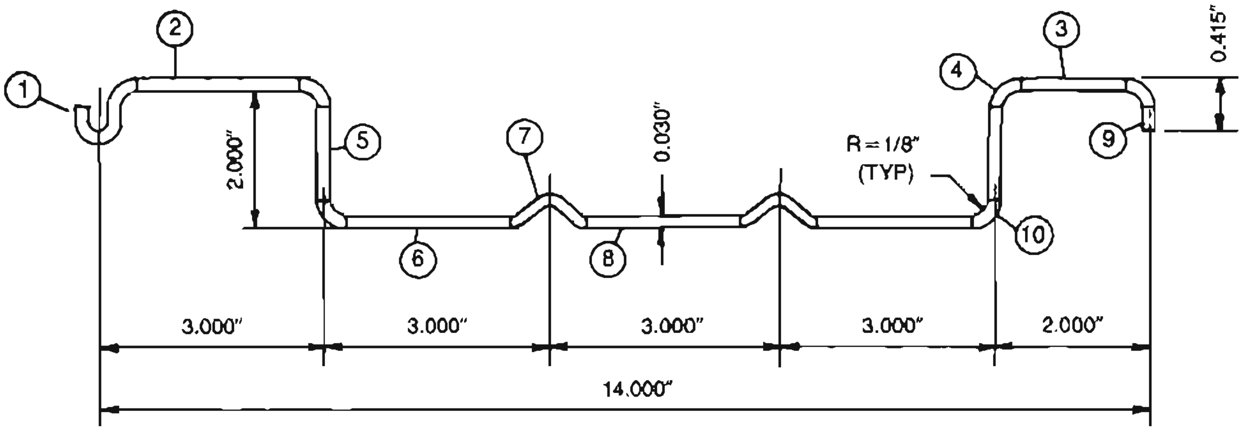
$$P_L = 0.5\left[\frac{0.224(1.689)^{1.32}}{7^{0.65} \times 6.000^{0.83} \times 0.060^{0.50}} - 0.0833\right]10560$$

$P_L = 354$ lbs.

Each brace will be designed to resist one of the P_L forces, determined above, depending on the restraint condition of the span.

EXAMPLE NO. 25

WALL PANEL

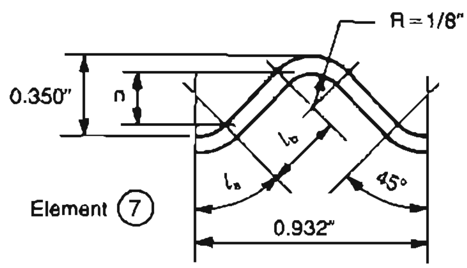


Given: 1. Steel: $F_y = 50$ ksi.
2. Section: as shown in the sketches.

Required: Section properties for positive and negative bending.

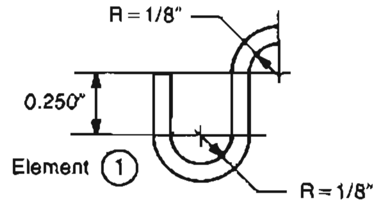
Solution:

1. Linear Properties. Refer to Part III, Section 1.
   Elements ④ and ⑩
   90° corners, $r = R + t/2 = 0.125 + 0.030/2 = 0.140$ in.
   Length of arc, $u = 1.57r = 1.57 \times 0.140 = 0.220$ in.
   Distance of c.g. from center of radius, $c_1 = 0.637r = 0.637 \times 0.140 = 0.089$ in.



Element ⑦
$r = 0.140$ in. $\theta = 45° = 0.785$ rad.
$c_1 = r \sin\theta/\theta = 0.140 \times 0.707/0.785 = 0.126$ in.
$n = 0.350 - 2 \times 0.140(1 - \cos 45°) = 0.350 - 0.082 = 0.268$ in.
$l_b = 0.268/\sin 45° = 0.379$ in.
$l_a = \theta r = 0.785 \times 0.140 = 0.110$ in.
I' (straight portions) $= 2 \times 1/12 \times l_b \times n^2 = 2 \times 1/12 \times 0.379 \times 0.268^2 = 0.0045$ in.^{3}
I' (Arcs) $= 4 \times 0.110 \times (0.350/2 - 0.140 + 0.126)^2 = 0.0114$ in.^{3}
$I' = I'$ (straight portions) $+ I'$ (Arcs) $= 0.0045 + 0.0114 = 0.0159$ in.^{3}
$I = I't = 0.0159 \times 0.030 = 0.000477$ in.^{4}
Check adequacy of intermediate stiffener according to Section B5.

For $w/t = (3.000 - 0.140 - 0.932/2)/0.030 = 79.8$ (Element ⑥)
$I_{min} = [3.66t^4 \sqrt{(w/t)^2 - 4000/F_y}]$ but not less than $(18.4t^4) = 0.000015$ in.^{4} (*Eq.* B5-1)
$I_{min} = [3.66 \times (0.030)^4 \sqrt{(79.8)^2 - 4000/50}] = 0.000235$ in.$^4 < 0.00477$ in.^{4}
satisfactory
$L_{st} = 4l_a + 2l_b = 0.440 + 0.758 = 1.198$ in.



Element ①

	L Length (in.)	y Distance from Center of Top Flange (in.)	Ly (in.^{2})	Ly^2 (in.^{3})	I_1' About Own Axis (in.^{3})
90° Corner	0.220	0.140 − 0.089 = 0.051	0.011	—	—
Straight Segments	0.500	0.265	0.133	0.035	0.003
Semi-Circle	0.440	0.390 + 0.089 = 0.479	0.211	0.101	—
Sum	1.160		0.355	0.136	0.003

$y_{cg} = 0.355/1.160 = 0.306$ in.
$I_x' = I_1' + Ly^2 - Ly_{cg}^2 = 0.003 + 0.136 - 1.160 \times 0.306^2 = 0.139 - 0.109 = 0.030$ in.^{3}
$I_x = I_x't = 0.030 \times 0.030 = 0.00090$ in.^{4}

2. Section Modulus for Load Determination—Positive Bending

Since the neutral axis will be below the center of the cross section, the compression stress will govern.

Element ① from Section B3.1 (a) since stress gradient is in opposite direction

$w = 0.25$ in.
$k = 0.43$
$f = F_y$ [see B2.1a(1)]
$\lambda = (1.052/\sqrt{0.43})\,(0.25/0.030)\,\sqrt{50/29500} = 0.550$ (*Eq.* B2.1-4)
$\rho = 1$ for $\lambda \le 0.673$
$b = w = 0.25$ in. (*Eq.* B2.1-1)

Element ② from Section B4.2 (a)
$w/t = [3-3(0.140)]/0.030 = 86 \qquad f = F_y$
$S = 1.28\sqrt{E/f} = 1.28\sqrt{29500/0.6(50)} = 40.138$ (*Eq.* B4-1)
$D/w = [0.25 + 2(0.125) + 1.5(0.030)]/[3-3(0.140)] = 0.211$
$n = 1/3 \qquad$ for $w/t > S$
$I_a = (0.030)^4\,[115(86/40.138) + 5] = 0.000204$ (*Eq.* B4.2-13)
$I_s = I_1' = 0.003$
$k = 3.57(0.003/0.000204)^{1/3} + 0.43 = 9.176 > 4$ (*Eq.* B4.2-10)
$\lambda = (1.052/\sqrt{4})\,(86)\,\sqrt{50/29500} = 1.862$ (*Eq.* B2.1-4)
$\rho = (1-0.22/1.862)\,(1/1.862) = 0.474$ (*Eq.* B2.1-3)
$b = \rho w = 0.474[3-3(0.140)] = 1.233$ in. (*Eq.* B2.1-2)
$A_s = A_s' = 0.348$ in.$^2 \qquad$ for $I_s \ge I_a$ (*Eq.* B4.2-12)

Element ⑨ from Section B3.2 (a)

$w = 0.415 - 0.030 - 0.125 = 0.26$ in.
$k = 0.43$

$f < F_y$ Use F_y as conservative value.

$\lambda = (1.052/\sqrt{0.43})(0.26/0.030)\sqrt{50/29500} = 0.572$ (*Eq.* B2.1-4)
$\rho = 1$ for $\lambda \leq 0.673$
$b = w = 0.26$ in. (*Eq.* B2.1-1)

Element ③ from Section B4.2 (a)

$w/t = [2-2(0.140)]/0.030 = 57.333$ $f = F_y$ [see B2.1a(1)] (*Eq.* B4-1)
$S = 1.28\sqrt{E/f} = 1.28\sqrt{29500/0.6(50)} = 40.138$
$D/w = [0.415-0.5(0.030)]/[2-2(0.140)] = 0.233$
$n = 1/3$ for $w/t > S$
$I_a = (0.030)^4[(57.333/40.138)(115)+5] = 0.000137$ for $w/t \geq S$ (*Eq.* B4.2-13)
$I_s = (1/12)bh^3 = (1/12)(0.030)(0.415-0.125-0.030)^3 = 0.000044$
$k = 3.57(0.000044/0.000137)^{1/3} + 0.43 = 2.875 < 4$ (*Eq.* B4.2-10)
$\lambda = (1.052/\sqrt{2.875})(57.333)\sqrt{50/29500} = 1.464$ (*Eq.* B2.1-4)
$\rho = [1-(0.22/1.464)](1/1.464) = 0.580$ (*Eq.* B2.1-3)
$b = \rho w = 0.580[2-2(0.140)] = 0.998$ in.
$d_s = (I_s/I_a)d'_s = (0.000044/0.000137)(0.26) = 0.084$ in. (*Eq.* B4.2-11)

Element	L	y	Ly	Ly²	I'_1
1	1.160	0.321	0.372	0.120	0.030
2	1.233	0.015	0.018	—	—
3	0.998	0.015	0.015	—	—
4	0.660	0.066	0.044	0.003	—
5	3.440	1.015	3.490	3.550	0.849
6	4.788	2.015	9.650	19.440	—
7	2.396	1.840	4.410	8.110	—
8	2.068	2.015	4.170	8.400	—
9	0.260	0.285	0.074	0.021	—
10	0.440	1.964	0.864	1.700	—
	17.443		23.107	41.344	0.879

$y_{cg} = 23.107/17.443 = 1.325$ in.
$I'_x = 41.344 + 0.879 - 17.443(1.325)^2 = 11.600$ in.^{3}
$I_x = I'_x t = 11.600(0.030) = 0.348$ in.^{4}
$S_x = I_x/y_{cg} = 0.163/1.325 = 0.123$ in.^{3}

$M_n = S_e F_y = 0.123(50) = 6.150$ kip-in.
$M_a = M_n/\Omega_f = 6.150/1.67 = 3.683$ kip-in. (*Eq.* C3.1-1)

Element ⑤ from Section B2.3 (a)

$y_{cg} = 1.325$ in.
$f_1 = [(1.325-0.125-0.030)/1.325](50) = 44.199$
$f_2 = -[(2.030-0.125-0.030-1.325)/1.325](50) = 20.755$

$\psi = f_2/f_1 = -20.755/44.199 = -0.470$
$k = 4+2(1+0.470)^3+2(1+0.470) = 13.293 > 4$ (*Eq.* B2.3-4)

$\lambda = (1.052/\sqrt{13.293})\{[2.030-2(0.155)]/0.030\}\sqrt{44.151/29500} = 1.640$ (*Eq.* B2.1-4)

$\rho = (1-0.22/1.640)(1/1.640) = 1.025$ (*Eq.* B2.1-3)

$b_e = \rho w = 1.025(1.720) = 1.763$, use $1.72 = w$ (*Eq.* B2.1-2)

Thus element ⑤ is fully effective so properties above are correct. If $b_e < 1.72$ then properties should be recomputed for an exact solution.

$b_1 + b_2 = 0.947$ in.

$w_c \quad = 1.370 - 0.030 - 0.125 = 1.215$

Revise section properties for loss of $(1.215 - 0.947)(0.030)(2) = 0.0161$ in.^{2} for element ⑤ if greater accuracy is required.

3. Moment of Inertia for Deflection Determination—Positive Bending

Element ② from Section B4.2 (b)

$f \quad = F_y/1.67 = 30$ ksi
$w \quad = 3 - 3(0.140) = 2.580$ in.
$\lambda_c \quad = 0.256 + 0.328(2.580/0.030)\sqrt{50/29500} = 1.417$ (*Eq.* B2.1-10)
$k \quad = 4$
$\lambda \quad = (1.052/\sqrt{4})(2.580/0.030)\sqrt{30/29500} = 1.443$ (*Eq.* B2.1-4)
$\rho \quad = [0.41 + 0.59\sqrt{50/30} - (0.22/1.443)](1/1.443) = 0.706$ (*Eq.* B2.1-9)
$b \quad = \rho w = 0.706(2.580) = 1.821$ in. (*Eq.* B2.1-2)
$A_s \quad = A_s' = 0.348$ in.^{2} (*Eq.* B4.2-12)

Element ③ from Section 4.2 (b)

$f \quad = 50/1.67 = 30$ ksi
$w \quad = 2 - 2(0.140) = 1.720$ in.
$\lambda_c \quad = 0.256 + 0.328(1.720/0.030)\sqrt{50/29500} = 1.030$ (*Eq.* B2.1-10)
$k \quad = 2.875$
$\lambda \quad = (1.052/\sqrt{2.875})(1.720/0.030)\sqrt{30/29500} = 1.134$ (*Eq.* B2.1-4)
$\rho \quad = [0.41 + 0.59\sqrt{50/30} - (0.22/1.134)](1/1.134) = 0.862$ (*Eq.* B2.1-9)
$b \quad = \rho w = 0.862(1.720) = 1.483$ in. (*Eq.* B2.1-2)
$d_s \quad = 0.084$ in.

Element	L	y	Ly	Ly²	I_1'
1	1.160	0.321	0.372	0.120	0.030
2	1.821	0.015	0.027	—	—
3	1.483	0.015	0.022	—	—
4	0.660	0.066	0.044	0.003	—
5	3.440	1.015	3.490	3.550	0.849
6	4.788	2.015	9.650	19.440	—
7	2.396	1.840	4.410	8.110	—
8	2.068	2.015	4.170	8.400	—
9	0.260	0.285	0.074	0.021	—
10	0.440	1.964	0.864	1.700	—
	18.516		23.123	41.344	0.879

$y_{cg} = 23.123/18.516 = 1.249$ in.
$I_x' = 41.344 + 0.879 - 18.516(1.249)^2 = 13.338$ in.^{3}
$I_x = I_x' t = 13.338(0.030) = 0.400$ in.^{4}

4. Section Modulus for Load Determination—Negative Bending

Since the N.A. may be closer to the compression flange than to the tension flange, the compression stress is unknown, and therefore the effective width of the compression flange and section properties must be determined by an iterative method.

Elements ①, ②, ③, ④, ⑤, ⑨ and ⑩ do not vary with stress level.

Element	L (in.)	y Distance from Top Fiber (in.)	Ly ($in.^2$)	Ly^2 ($in.^3$)	I_1' About Own Axis ($in.^3$)
1	1.160	0.321	0.372	0.119	0.030
2	2.580	0.015	0.039	0.001	—
3	1.720	0.015	0.026	—	—
4	$3 \times 0.220 = 0.660$	0.066	0.044	0.003	—
5	$2 \times 1.720 = 3.440$	1.015	3.490	3.550	0.849
9	0.260	0.285	0.074	0.021	—
10	$2 \times 0.220 = 0.440$	1.964	0.864	1.700	—
Sum	10.260		4.909	5.394	0.879

Element ⑥ from Section B5 (d)

Assume $y_{cg} = 1.300$ in.
$f \quad = [(2.030 - 1.300)/1.300](50) = 28.077$ ksi
$w/t = [3 - 0.140 - 0.5(0.932)]/0.030 = 79.8$
$k = 4$

$\lambda \quad = (1.052/\sqrt{4})(79.8)\sqrt{28.077/29500} = 1.295$ (*Eq*. B2.1-4)

$\rho \quad = (1 - 0.22/1.295)(1/1.295) = 0.641$ (*Eq*. B2.1-3)

$b \quad = \rho w = 0.641(2.394) = 1.535$ in. (*Eq*. B2.1-2)

$b_e \quad = 0.030[(1.535/0.030) - 0.10(79.8 - 60)] = 1.476$ in. (*Eq*. B5-3)

Element ⑦ from Section B5 (d)

$60 < w/t = 79.8 < 90$

$\alpha \quad = 3 - [2(1.476)/2.394] - \{(1/30)[1 - (1.476/2.394)](79.8)\} = 0.747$ (*Eq*. B5.5)

$L_{ef} \quad = \alpha L_{st} = 0.747(2.396) = 1.790$ in. (*Eq*. B5-4)

Element ⑧ from Section B5 (d)

Assume $y_{cg} = 1.300$ in.
$f \quad = 28.077$ ksi
$w/t = (3 - 0.932)/0.030 = 68.933$
$k \quad = 4$

$\lambda \quad = (1.052/\sqrt{4})(68.933)\sqrt{28.077/29500} = 1.119$ (*Eq*. B2.1-4)

$\rho \quad = (1 - 0.22/1.119)(1/1.119) = 0.718$ (*Eq*. B2.1-3)

$b \quad = \rho w = 0.718(2.068) = 1.485$ in. (*Eq*. B2.1-2)

$b_e \quad = 0.030[(1.485/0.030) - 0.10(68.033 - 60)] = 1.458$ in. (*Eq*. B5.3)

Element	L	y	Ly
1	1.160	0.321	0.372
2	2.580	0.015	0.039
3	1.720	0.015	0.026
4	0.660	0.066	0.044
5	3.440	1.015	3.490
6	2.952	2.015	5.948
7	1.790	1.840	3.294
8	1.458	2.015	2.938
9	0.260	0.285	0.074
10	0.440	1.964	0.864
	16.460		17.089

$y_{cg} = 17.089/16.460 = 1.038$ in.

Recalculate b_c and α for elements 6, 7, and 8 based on

$f = F_y$ since $y_{cg} < 2.030/2 = 1.015$

As with the positive bending, the effective width of element 5 needs to be checked.

5. Moment of Inertia for Deflection Determination—Negative Bending

For deflection determination, calculate I_x based on

$f = F_y/1.67$ for elements 6, 7, and 8.

(Refer to Example No. 8, page IV-63 for example of procedure to follow.)

CHARTS & TABLES

CHARTS AND TABLES

FOR USE WITH THE
AUGUST 19, 1986, EDITION OF THE

SPECIFICATION FOR THE DESIGN OF COLD-FORMED STEEL STRUCTURAL MEMBERS

Cold-Formed Steel Design Manual - Part V

AMERICAN IRON AND STEEL INSTITUTE
1000 16th STREET, NW
WASHINGTON, DC 20036

1st Printing—March 1987

Produced by W. P. Reyman Associates, Inc. New York

PREFACE

This document, *Part V of the Cold-Formed Steel Design Manual*, contains two groups of design aids: (A) charts and tables prepared to assist in the use of particular design provisions of the *Specification*, and (B) tables of section properties. Included in Group A is an extensive series of graphs related to the compression member design procedures contained in *Part III, Supplementary Information*.

These *Charts and Tables* should be used in conjunction with the other parts of the *Design Manual*, which include *Commentary (Part II), Supplementary Information (Part III)*, and *Illustrative Examples (Part IV)*, in addition to the *Specification (Part I)*.

American Iron and Steel Institute
August 1986

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PART V

CHARTS AND TABLES FOR USE WITH THE AUGUST 19, 1986, EDITION OF THE SPECIFICATION FOR THE DESIGN OF COLD-FORMED STEEL STRUCTURAL MEMBERS

GROUP B—TABLES OF SECTION PROPERTIES

GROUP A

CHARTS RELATED TO PARTICULAR SPECIFICATION PROVISIONS

GENERAL NOTES

(a) The appropriate equations from the Specification are generally shown on each design aid.

(b) The definitions of the terms used in these charts and tables can be found in the Specification.

(c) The torsional-flexural buckling charts are grouped together by cross-section type for convenience; that is, singly-symmetric angle sections, singly-symmetric channel sections, and singly-symmetric hat sections.

(d) The torsional-flexural buckling design charts are based on a square corner approximation for all section properties.

(e) In the titles and labels for the torsional-flexural buckling design charts, "**a**" and "σ" are used interchangeably.

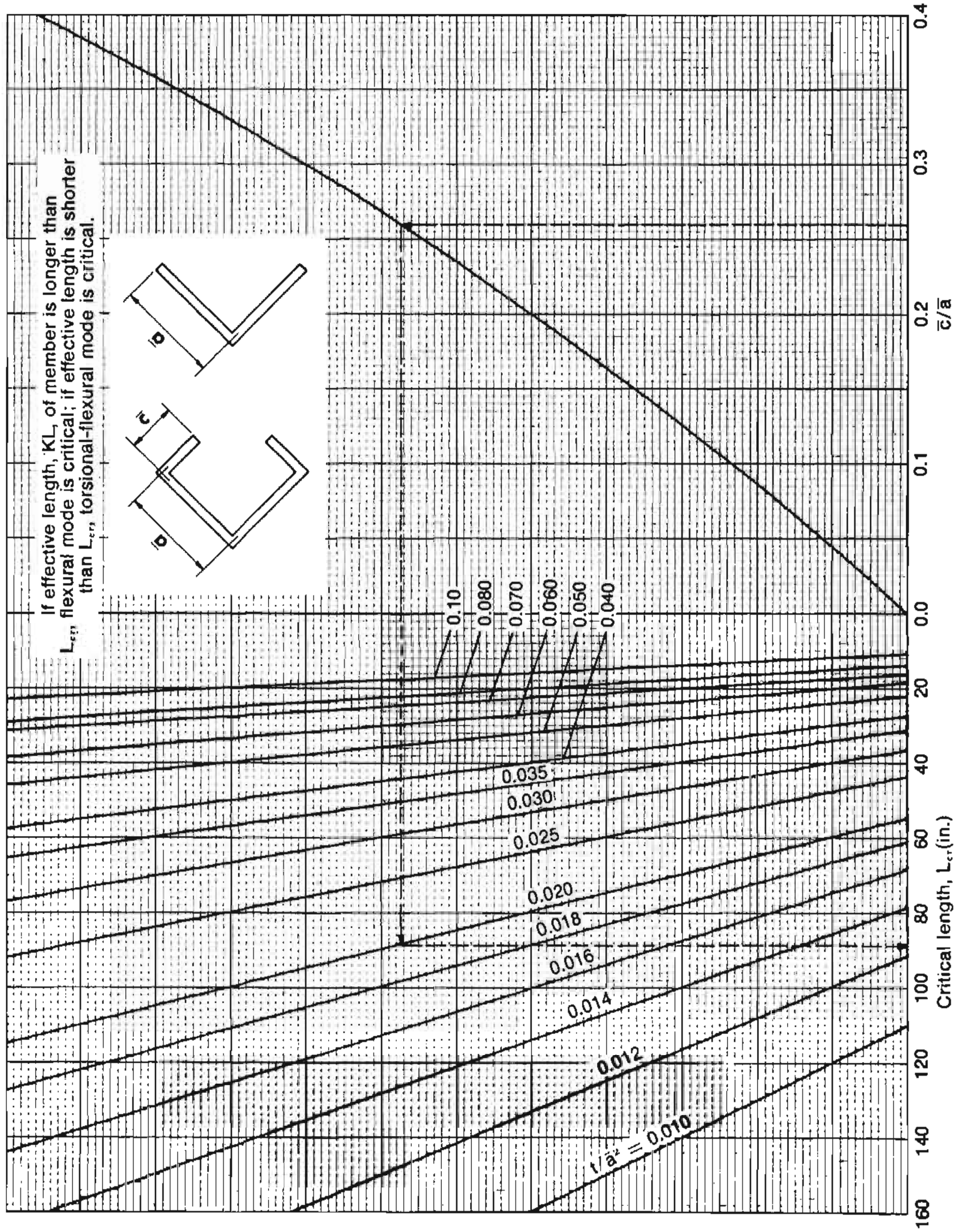


CHART V-1.1
Torsional-Flexural Buckling
(See Part III, Section 2 for application)
Buckling Mode for Equal Angles (Singly-Symmetric), With and Without Lips

CHART V-1.2
$Cx/\bar{a}^2$ for Equal Angles (Singly-Symmetric), With and Without Lips

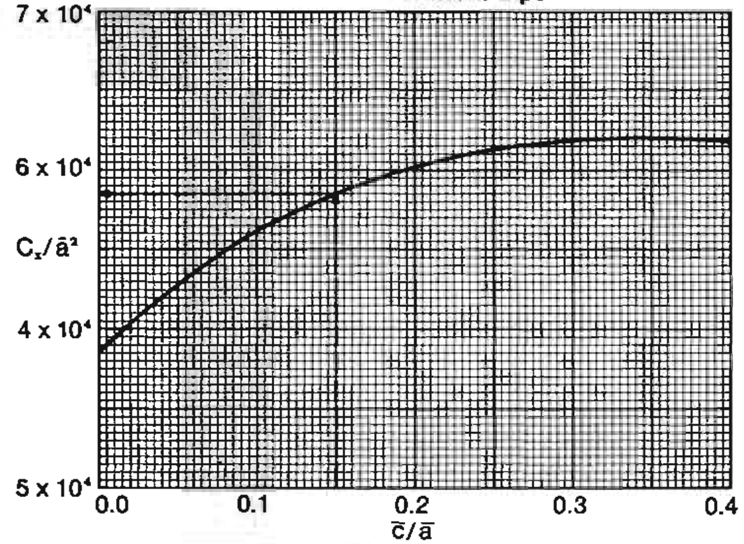


CHART V-1.3
$\sigma_{to}\ (\bar{a}/t)^2$ for Equal Angles (Singly-Symmetric), With and Without Lips

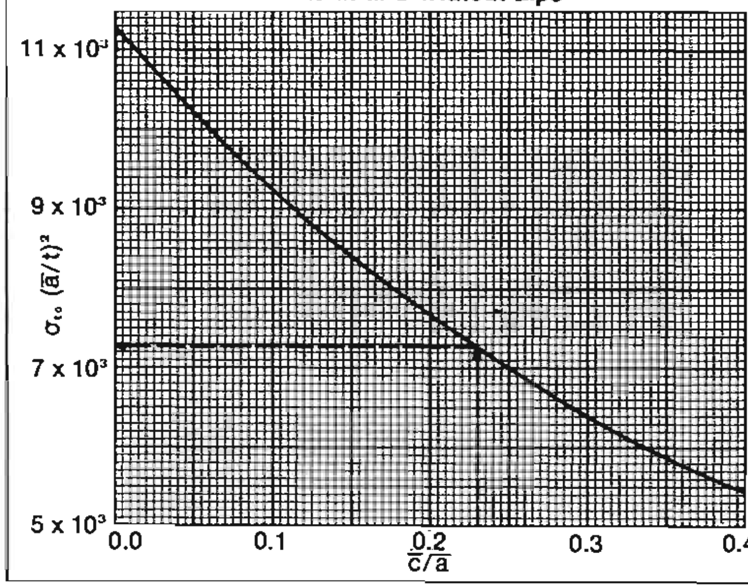
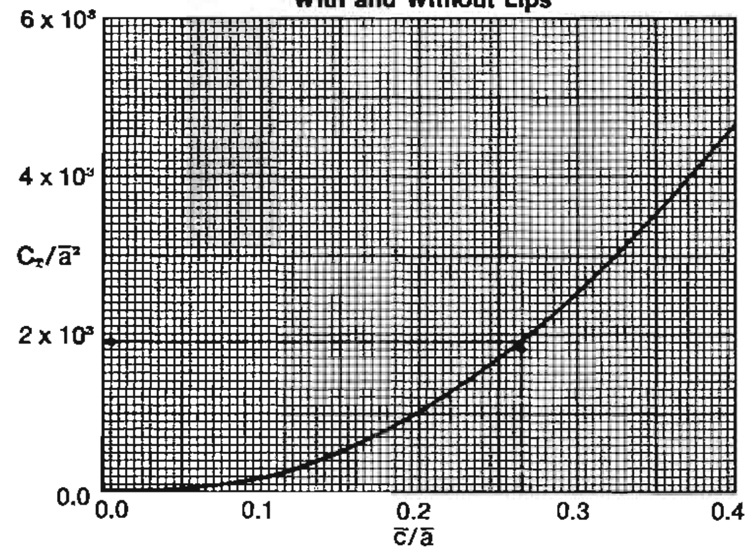


CHART V-1.4
$C_T/\bar{a}^2$ for Equal Angles (Singly-Symmetric), With and Without Lips



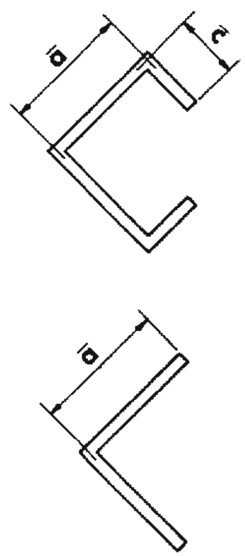
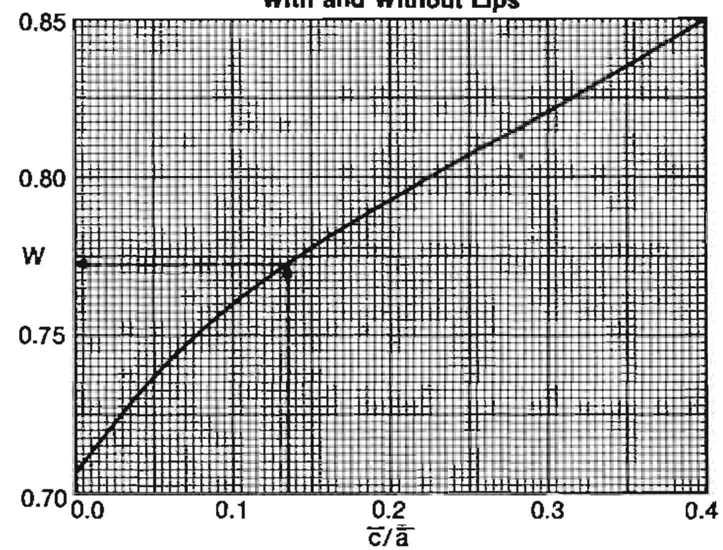


CHART V-1.6
W-Factor for Equal Angles (Singly-Symmetric), With and Without Lips



TORSIONAL-FLEXURAL BUCKLING
(See Part III, Section 2 for application)

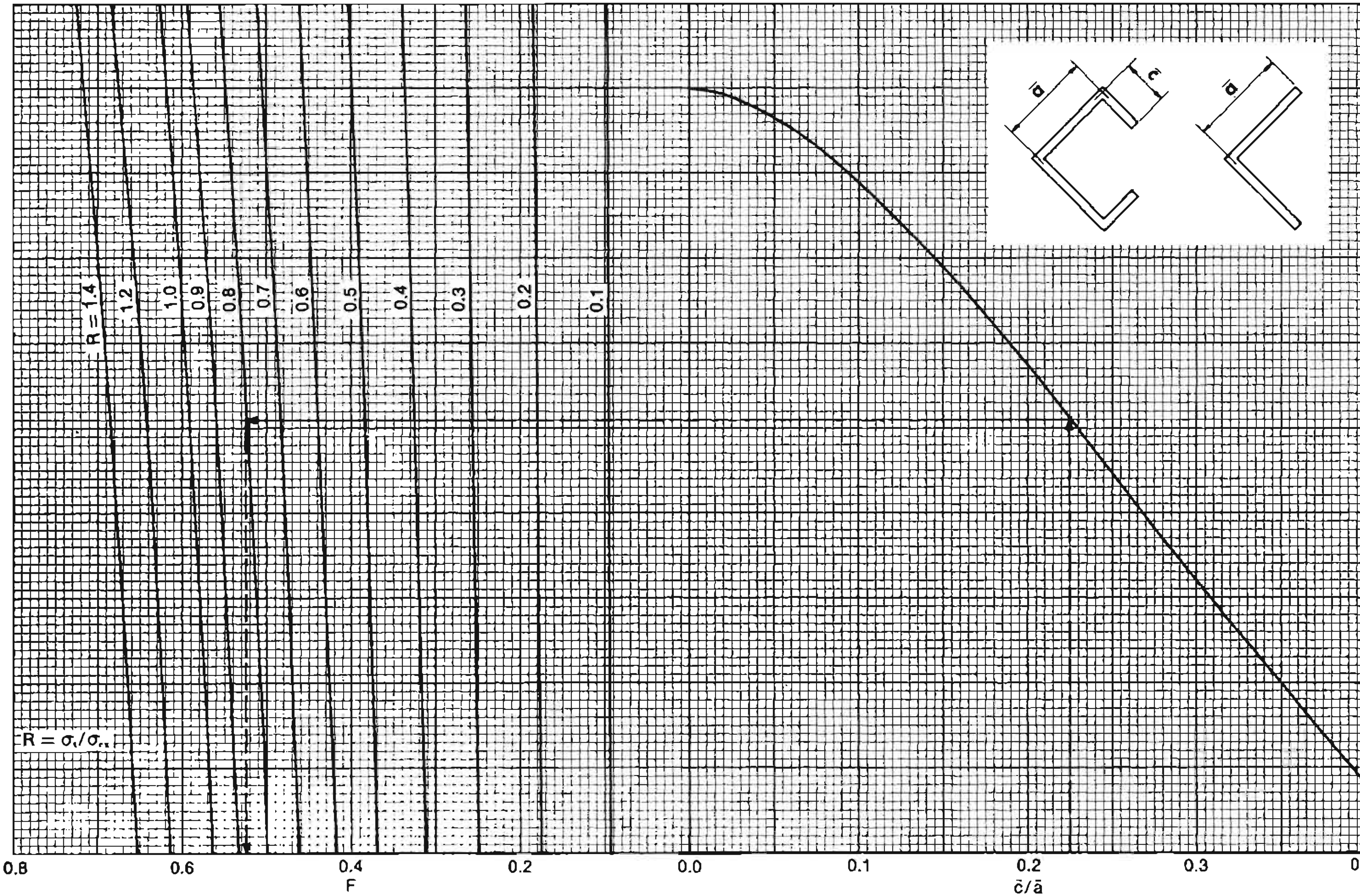


CHART V-1.5
Torsional-Flexural Buckling
(See Part III, Section 2 for application)
F-Factor for Equal Angles (Singly-Symmetric), With and Without Lips

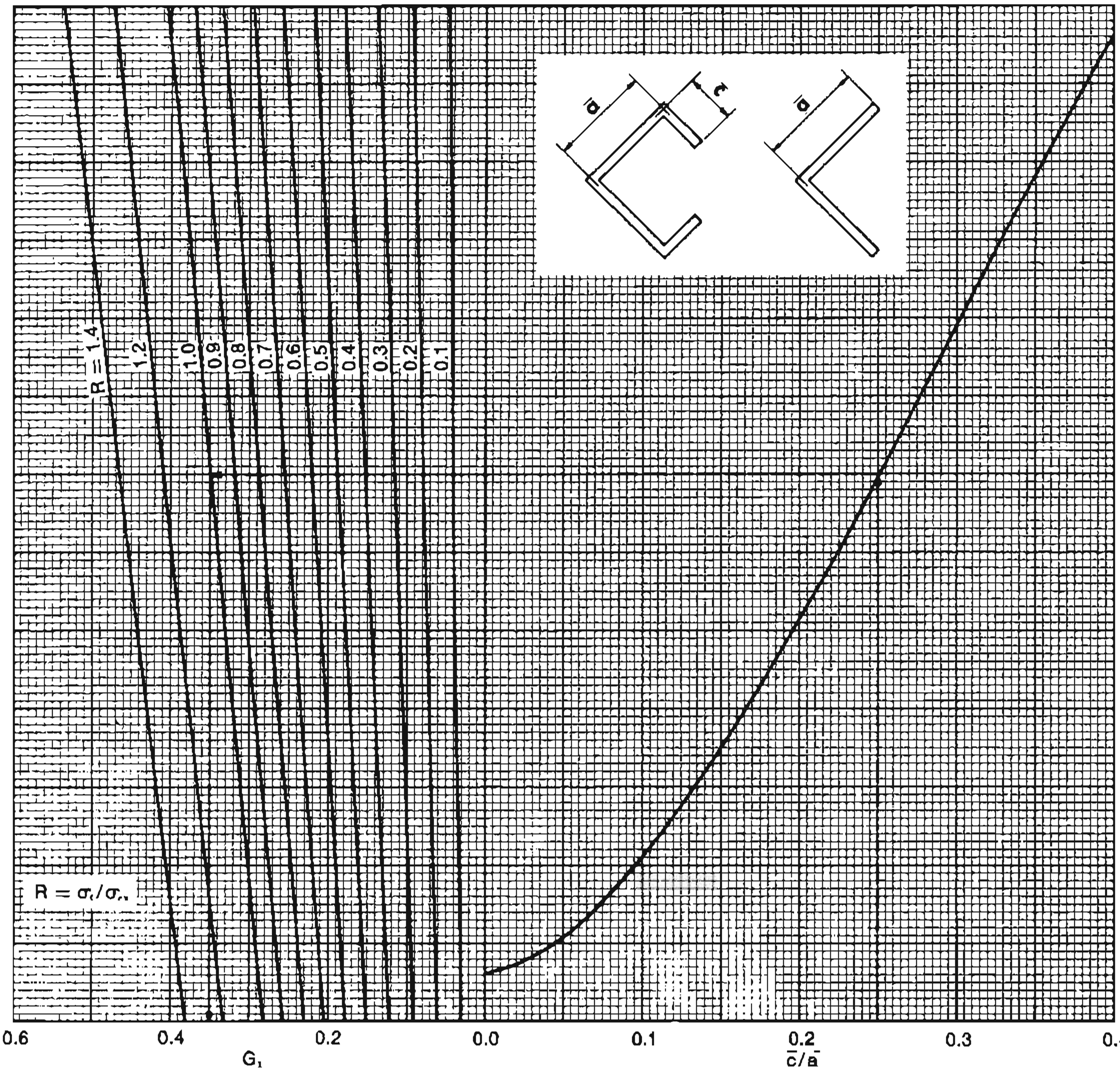


CHART V-1.7

Torsional-Flexural Buckling
(See Part III, Section 2 for application)
G_1-Factor for Equal Angles (Single-Symmetric), With and Without Lips

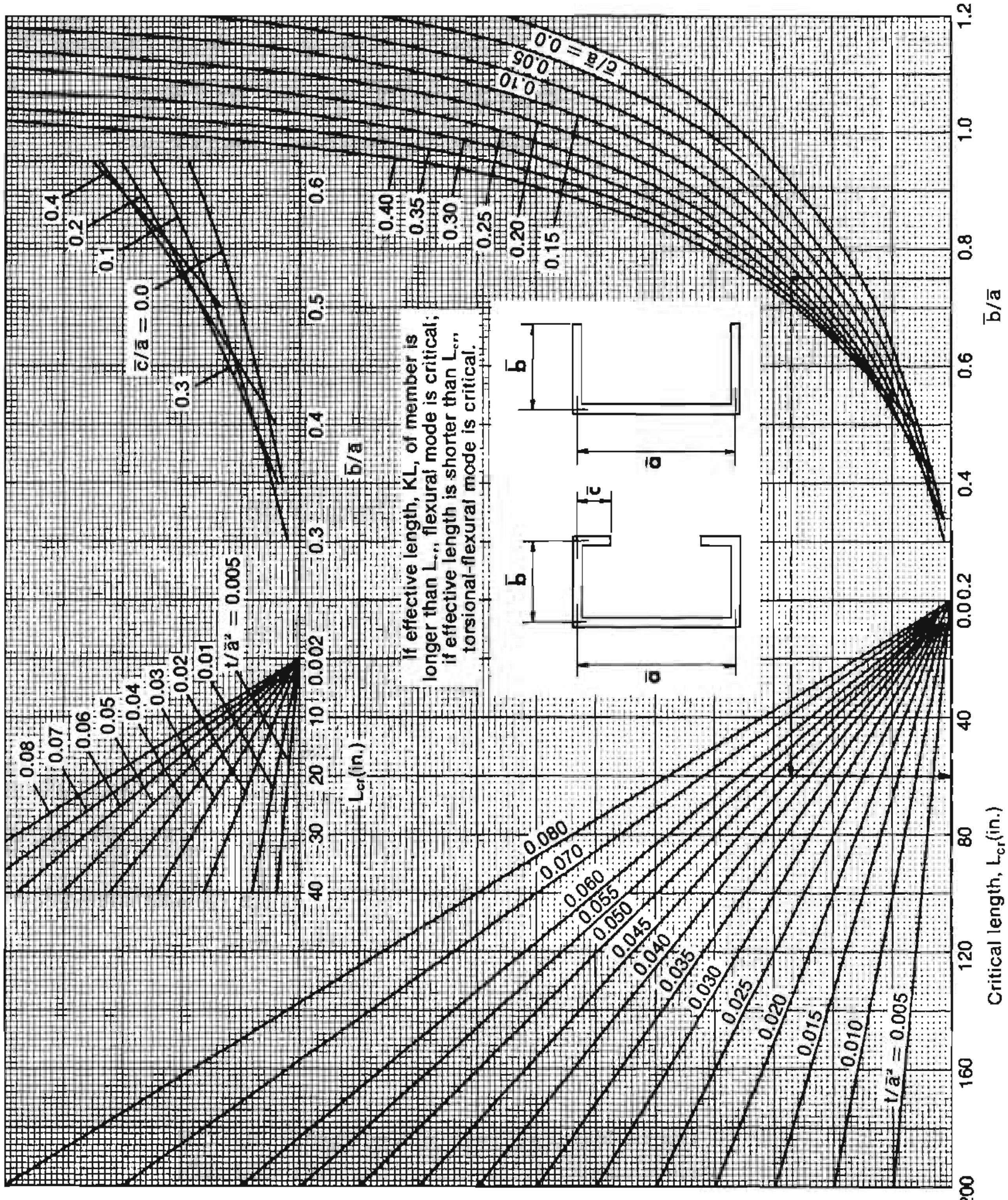


CHART V-2.1
Torsional-Flexural Buckling
(See Part III, Section 2 for application)
Buckling Mode for Channels (Singly-Symmetric), With and Without Lips

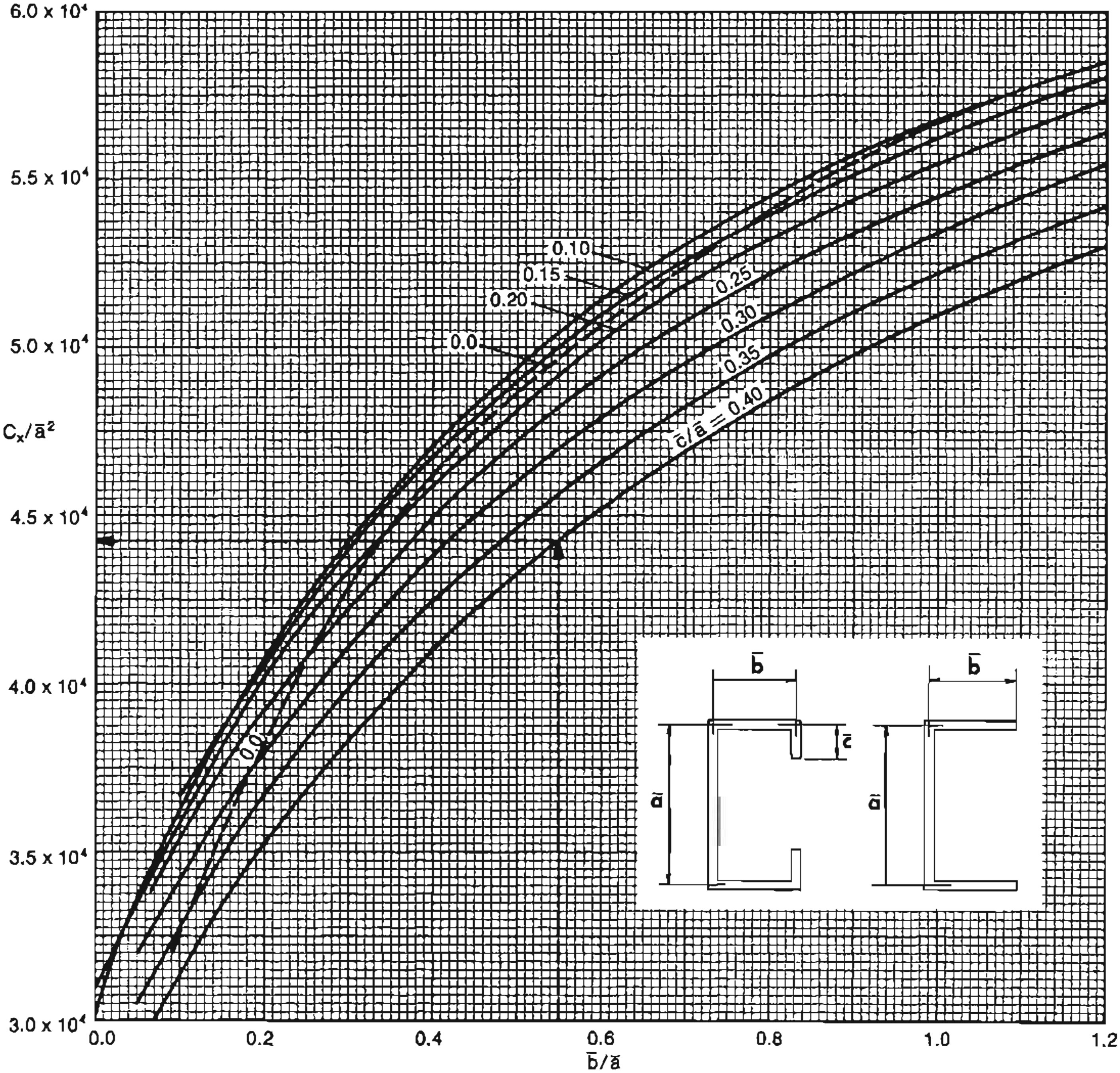


CHART V-2.2
Torsional-Flexural Buckling
(See Part III, Section 2 for application)
$C_x/\bar{a}^2$ for Channels (Singly-Symmetric), With and Without Lips

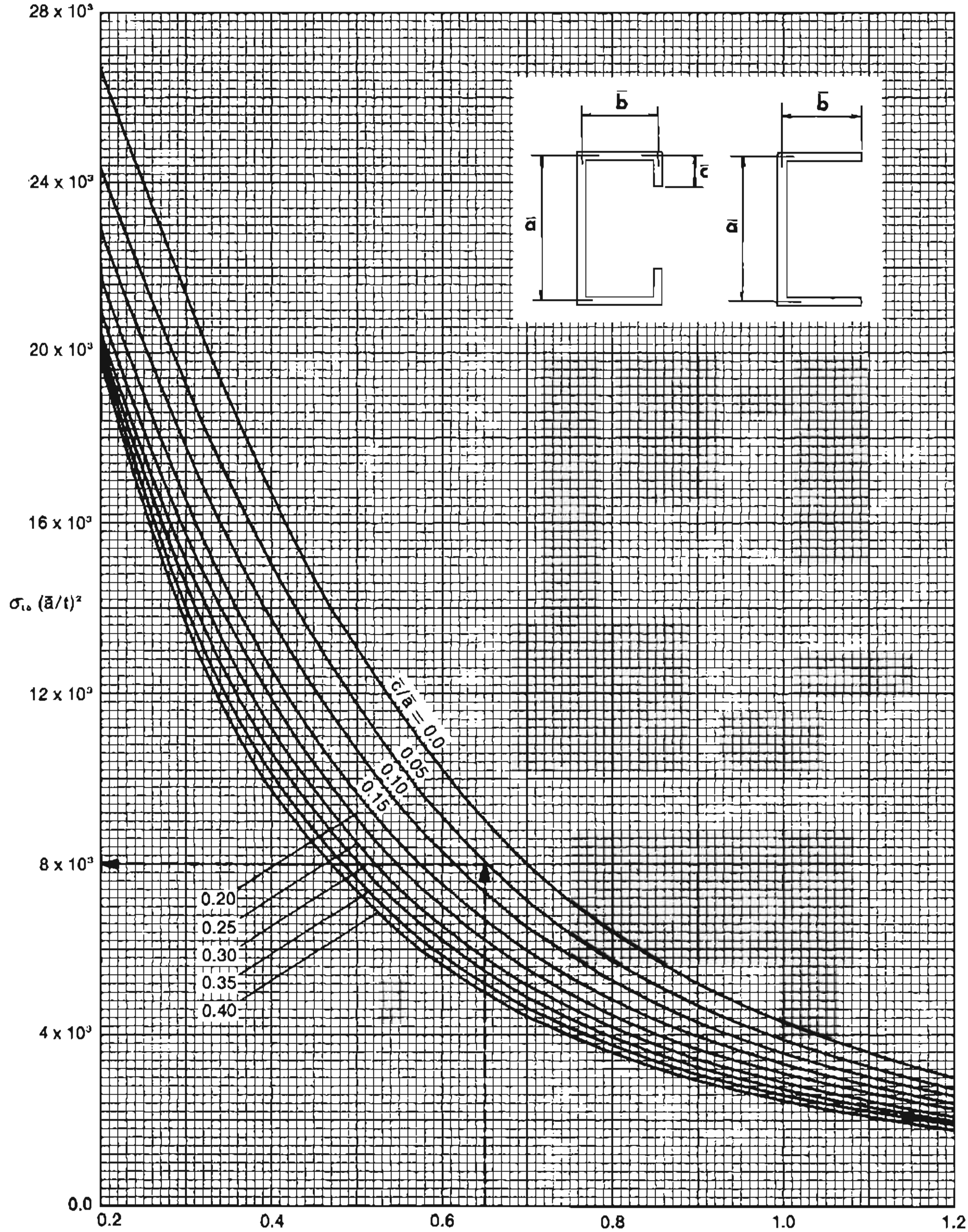


CHART V-2.3
Torsional-Flexural Buckling
(See Part III, Section 2 for application)
$\sigma_{to}\,(\bar{a}/t)^2$ for Channels (Singly-Symmetric), With and Without Lips

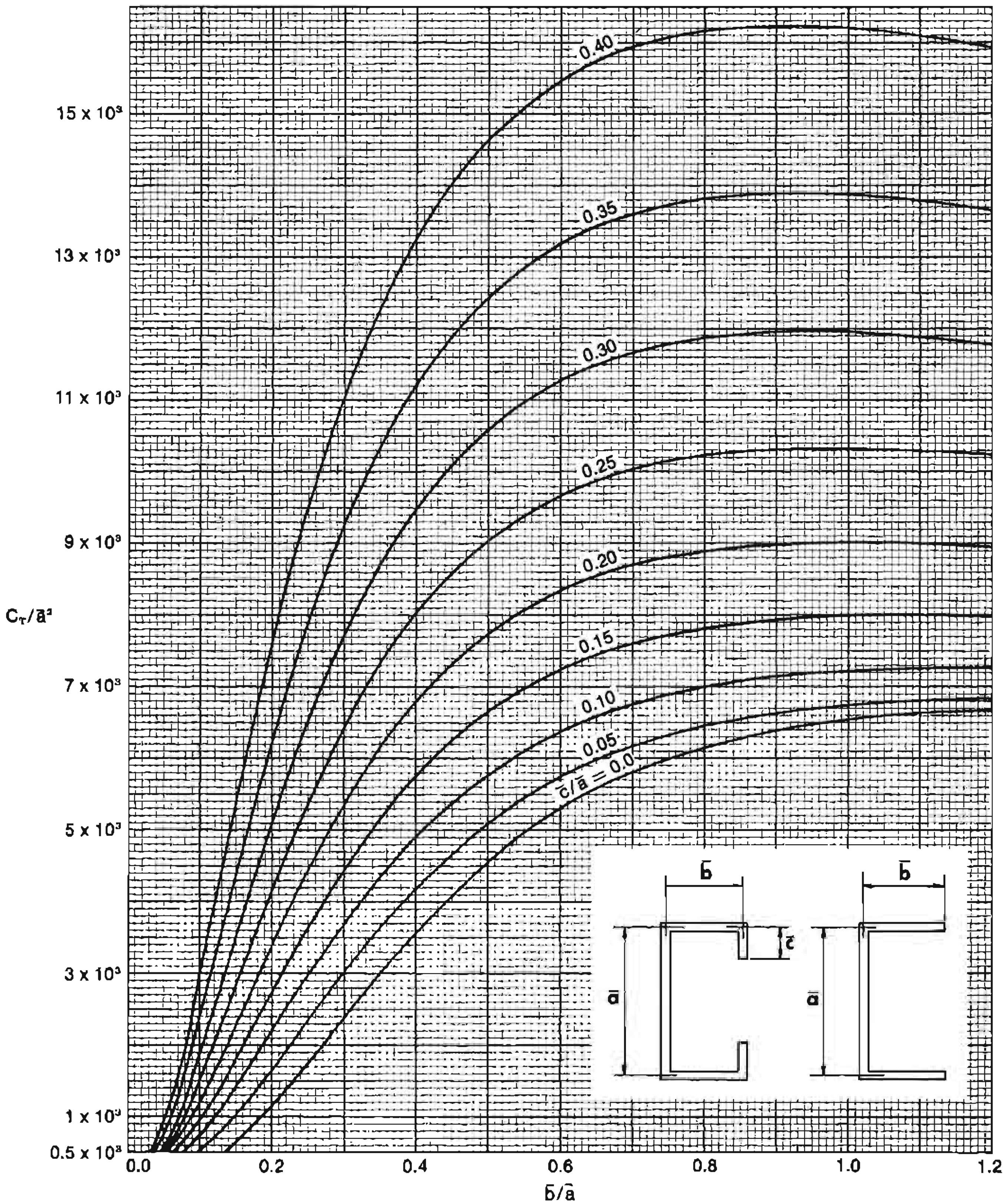


CHART V-2.4
Torsional-Flexural Buckling
(See Part III, Section 2 for application)
$C_t/\bar{a}^2$ for Channels (Singly-Symmetric), With and Without Lips

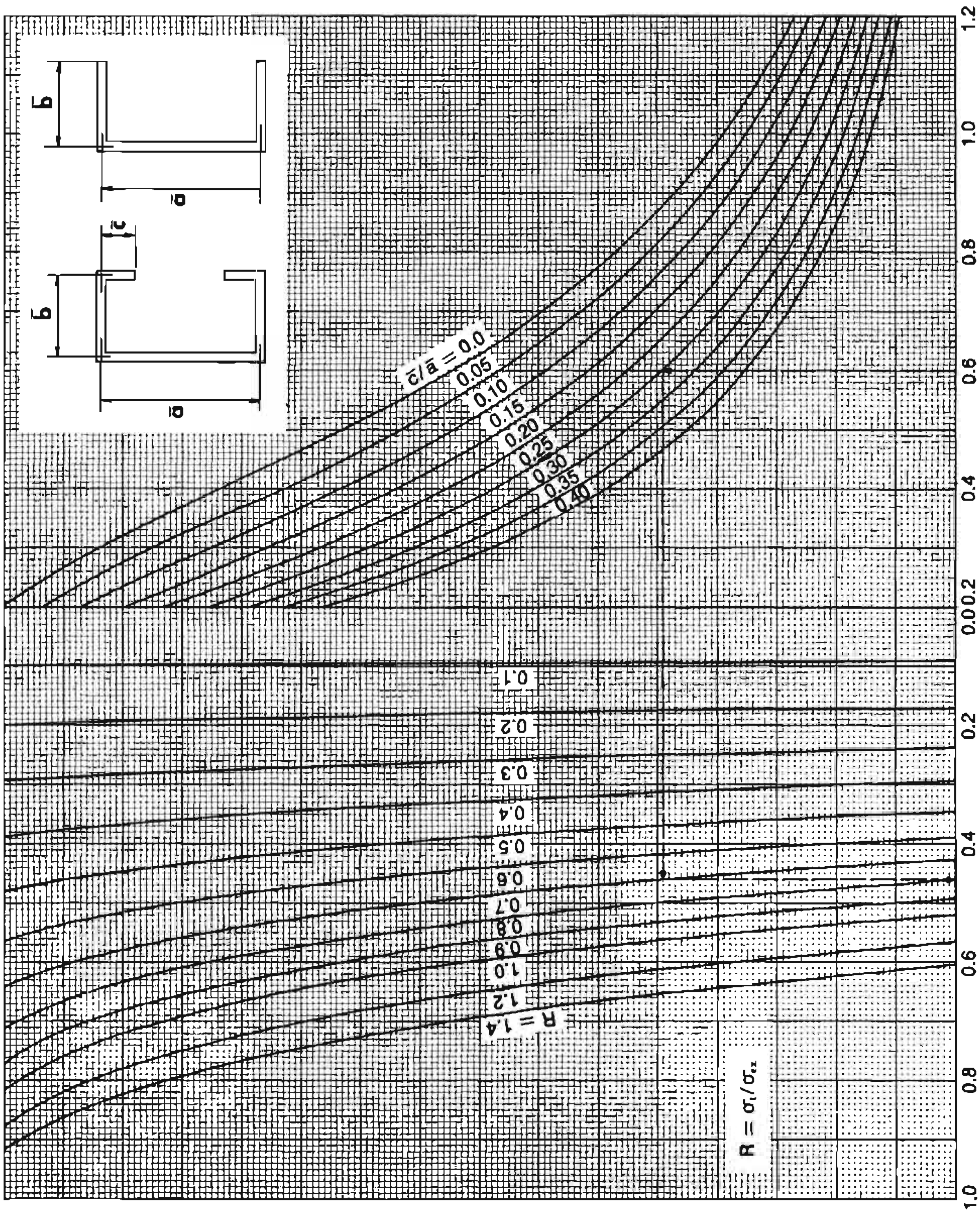


CHART V-2.5
Torsional-Flexural Buckling
(See Part III, Section 2 for application)
F-Factor for Channels (Singly-Symmetric), With and Without Lips

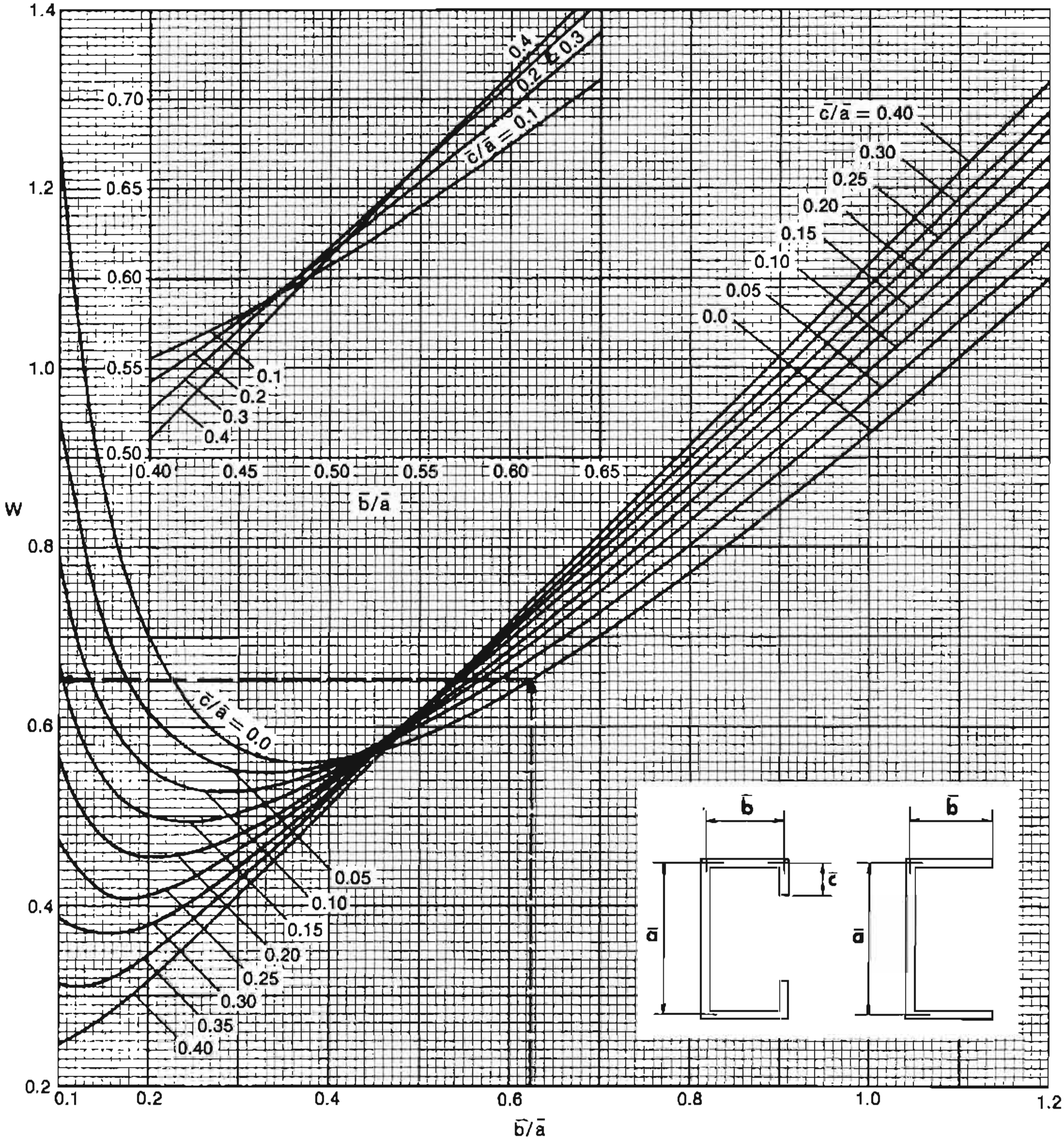


CHART V-2.6
Torsional-Flexural Buckling
(See Part III, Section 2 for application)
W-Factor for Channels (Singly-Symmetric), With and Without Lips

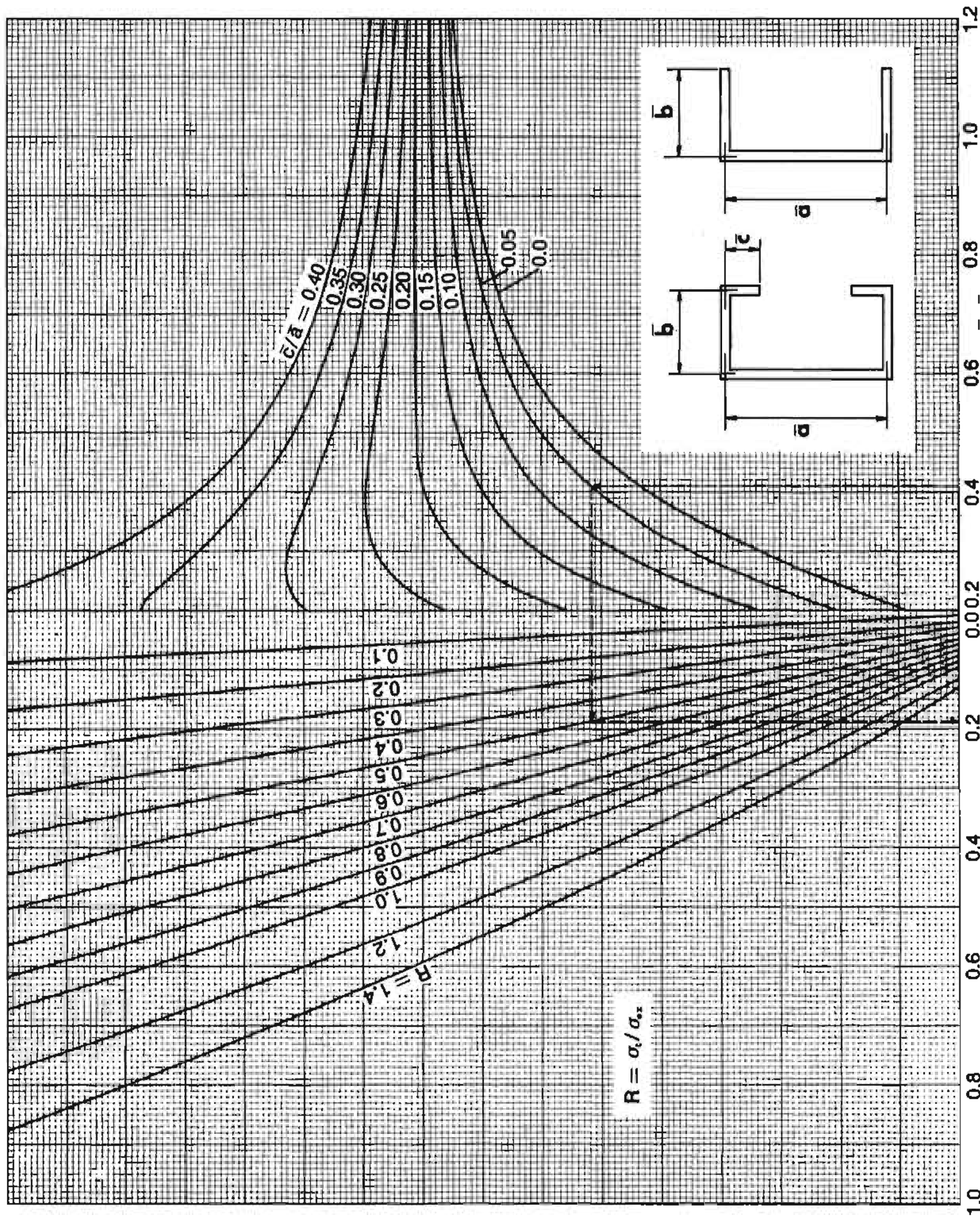


CHART V-2.7
Torsional-Flexural Buckling
(See Part III, Section 2 for application)
G_1-Factor for Channels (Singly-Symmetric), With and Without Lips

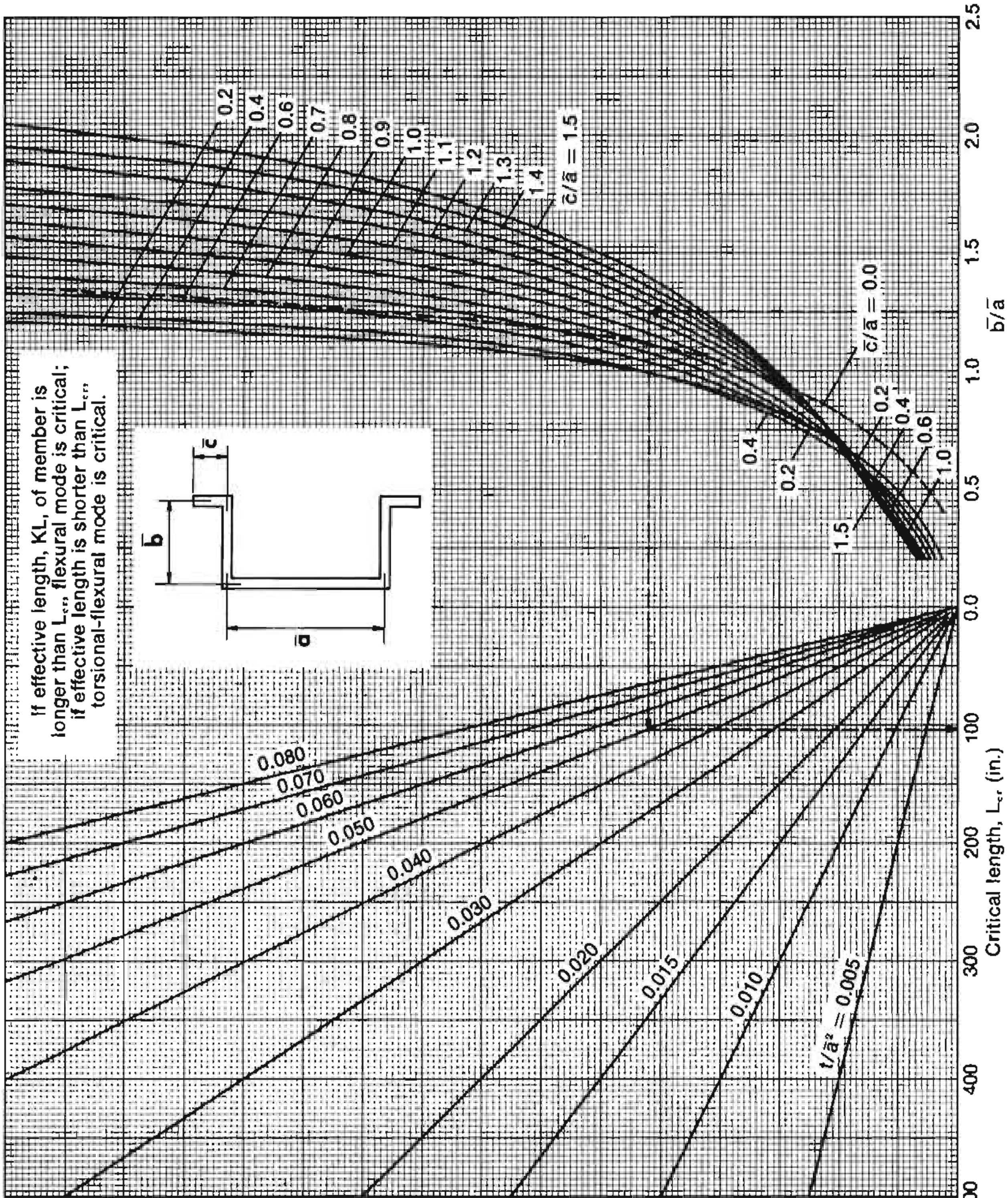


CHART V-3.1
Torsional-Flexural Buckling
(See Part III, Section 2 for application)
Buckling Mode for Hat Sections (Singly-Symmetric)

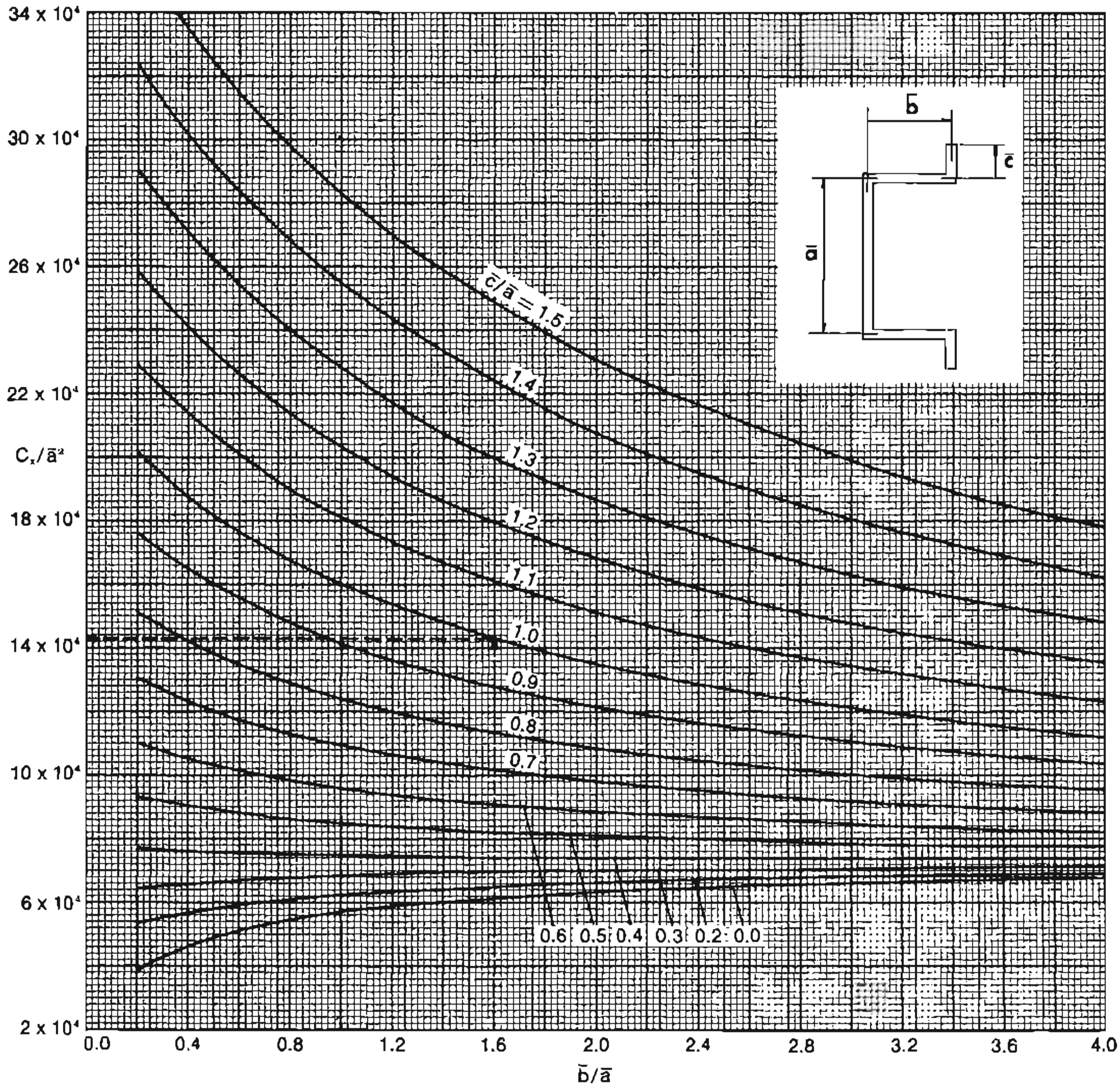


CHART V-3.2

Torsional-Flexural Buckling
(See Part III, Section 2 for application)
$C_x/\bar{a}^2$ for Hat Sections (Singly-Symmetric)

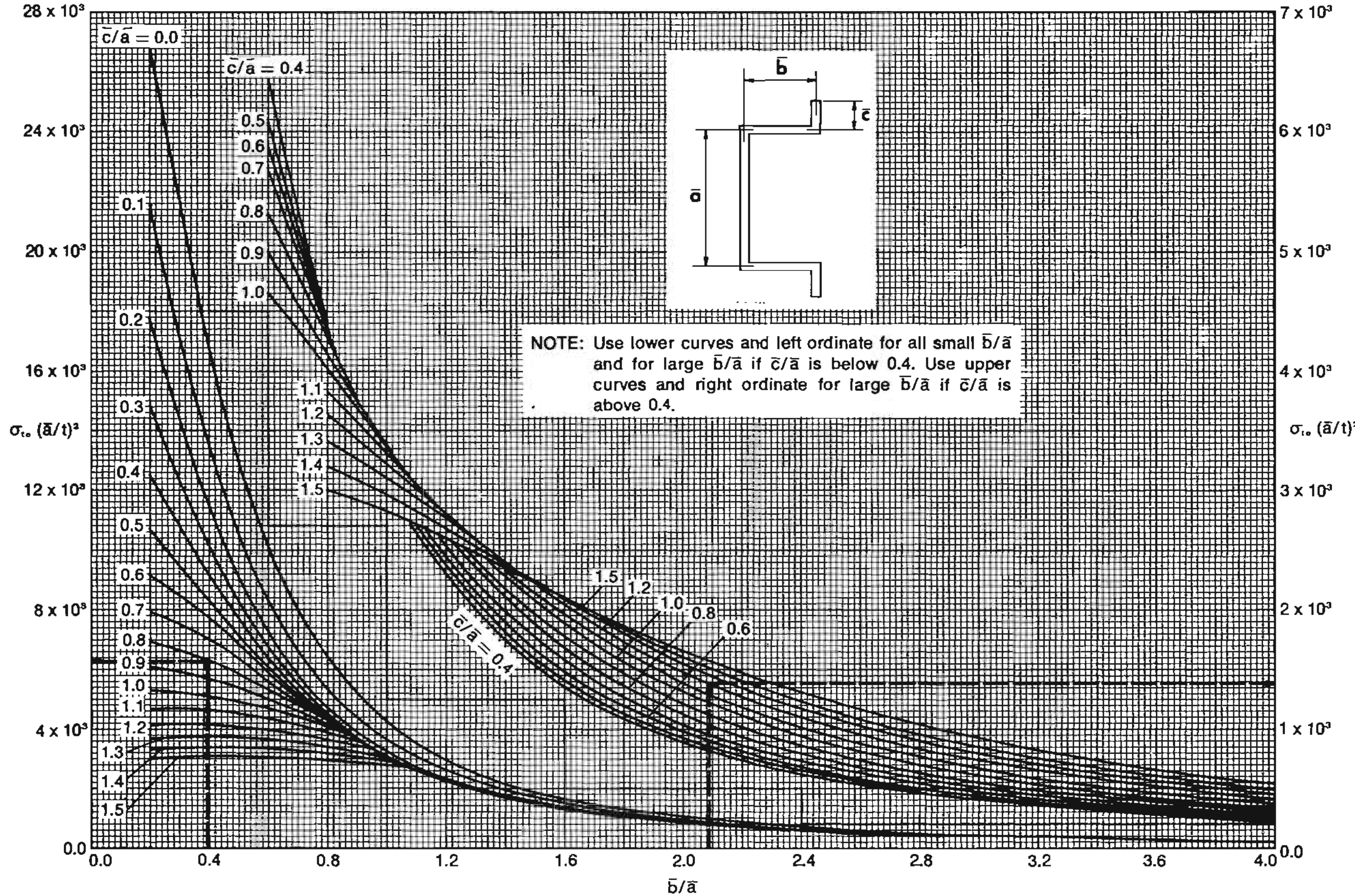


CHART V-3.3
Torsional-Flexural Buckling
(See Part III, Section 2 for application)
$\sigma_{to}\,(\bar{a}/t)^2$ for Hat Sections (Singly-Symmetric)

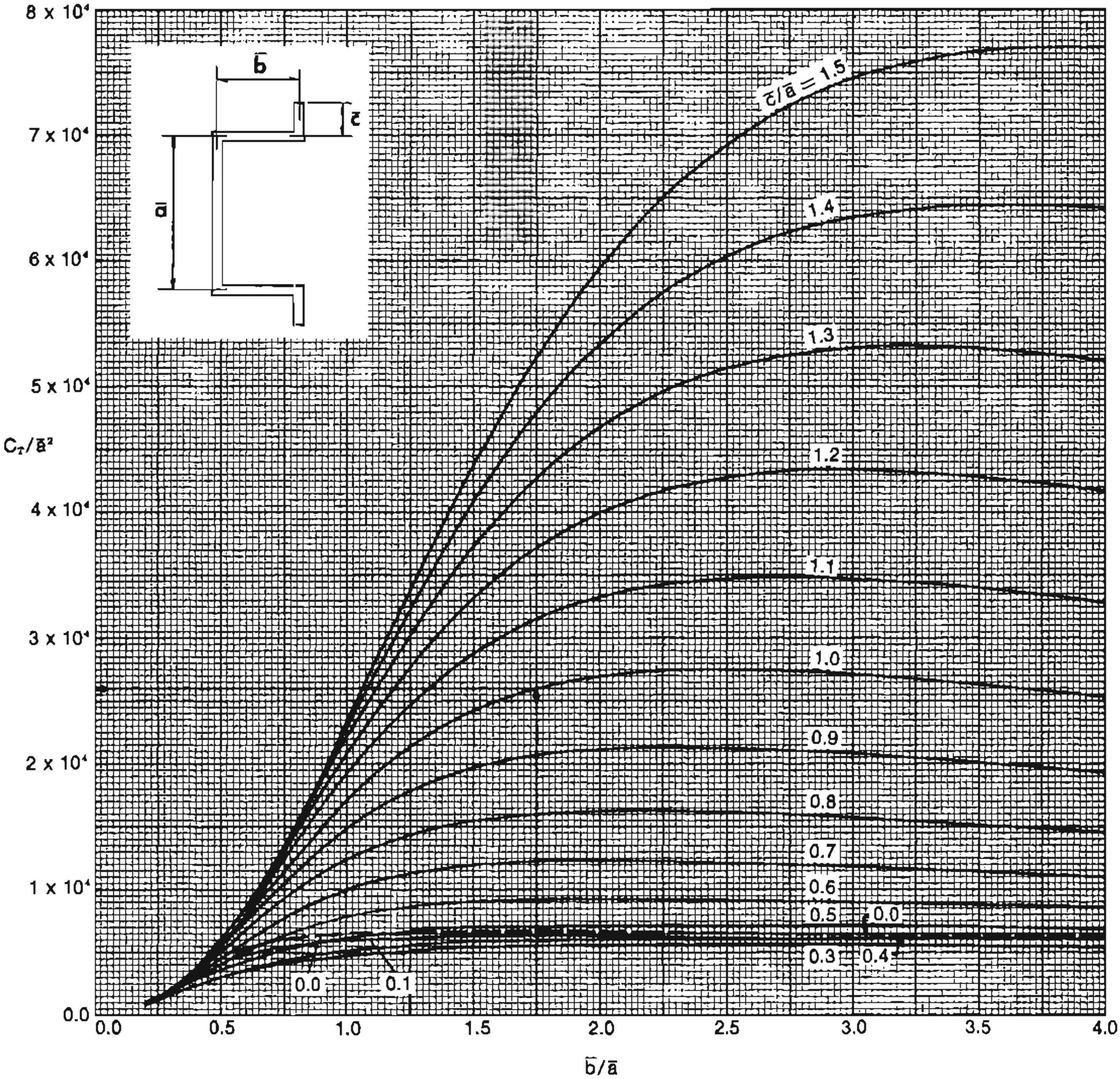


CHART V-3.4
Torsional-Flexural Buckling
(See Part III, Section 2 for application)
$C_T/\bar{a}^2$ for Hat Sections (Singly-Symmetric)

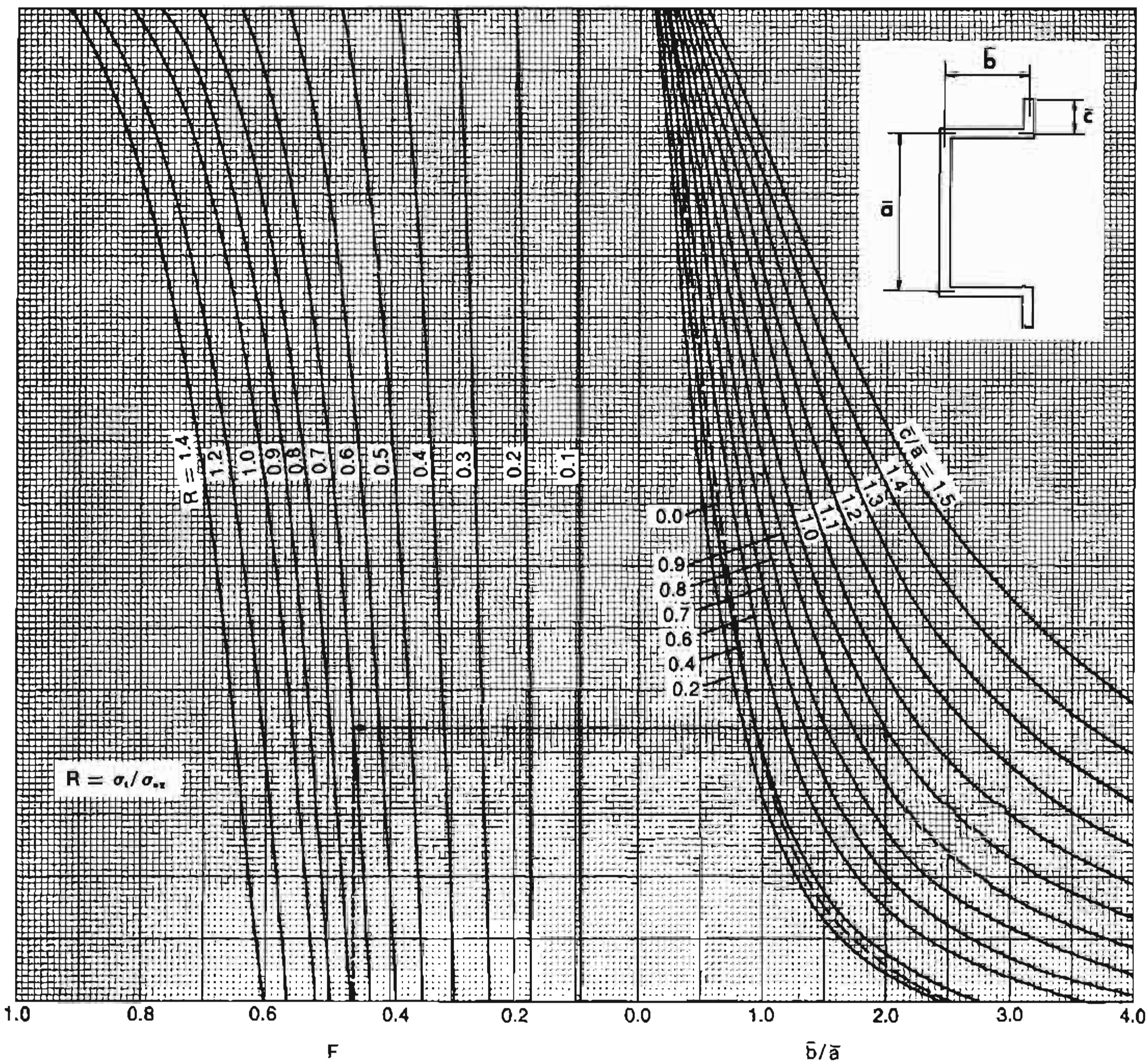


CHART V-3.5
Torsional-Flexural Buckling
(See Part III, Section 2 for application)
F-Factor for Hat Sections (Singly-Symmetric)

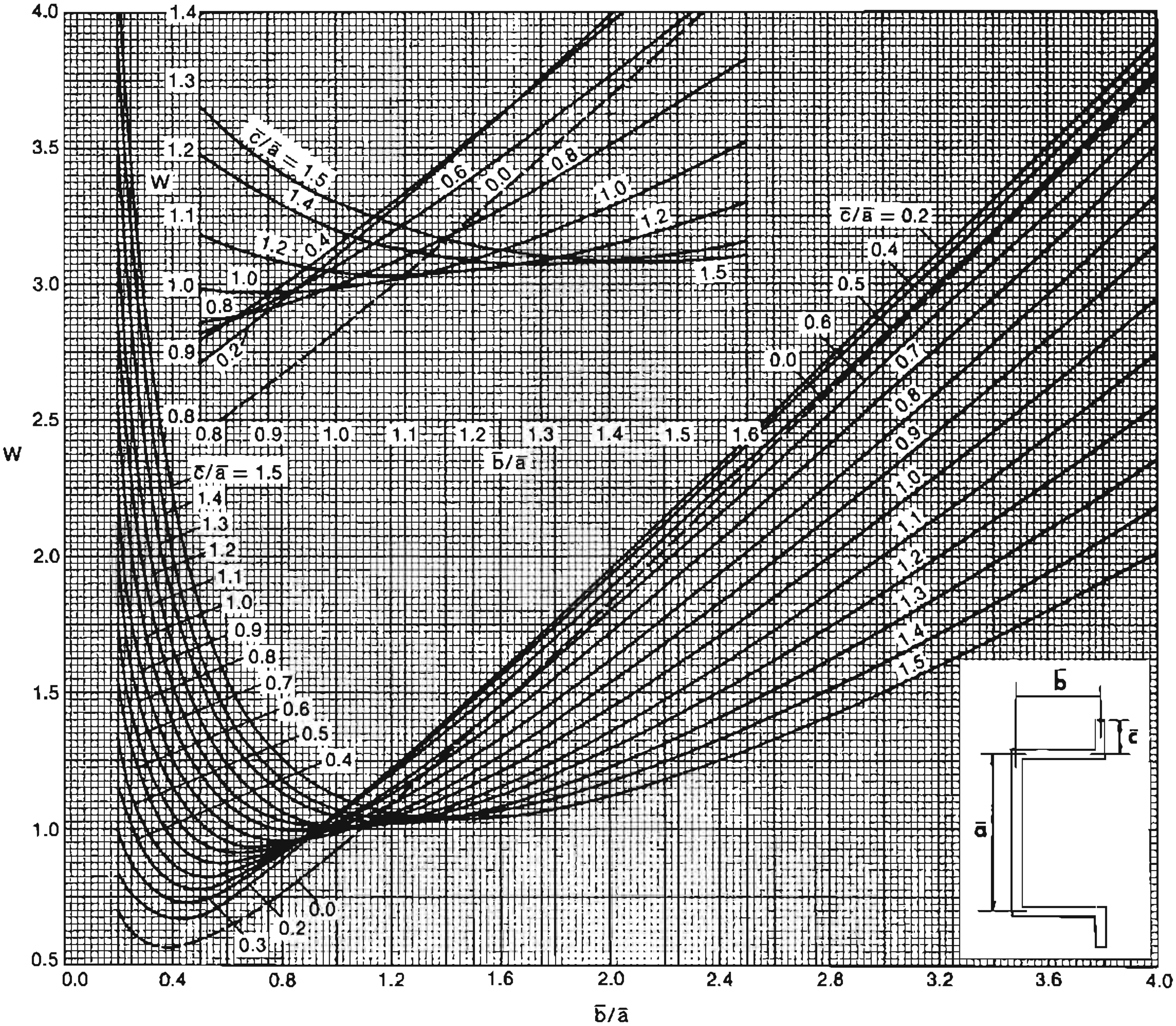


CHART V-3.6
Torsional-Flexural Buckling
(See Part III, Section 2 for application)
W-Factor for Hat Sections (Singly-Symmetric)

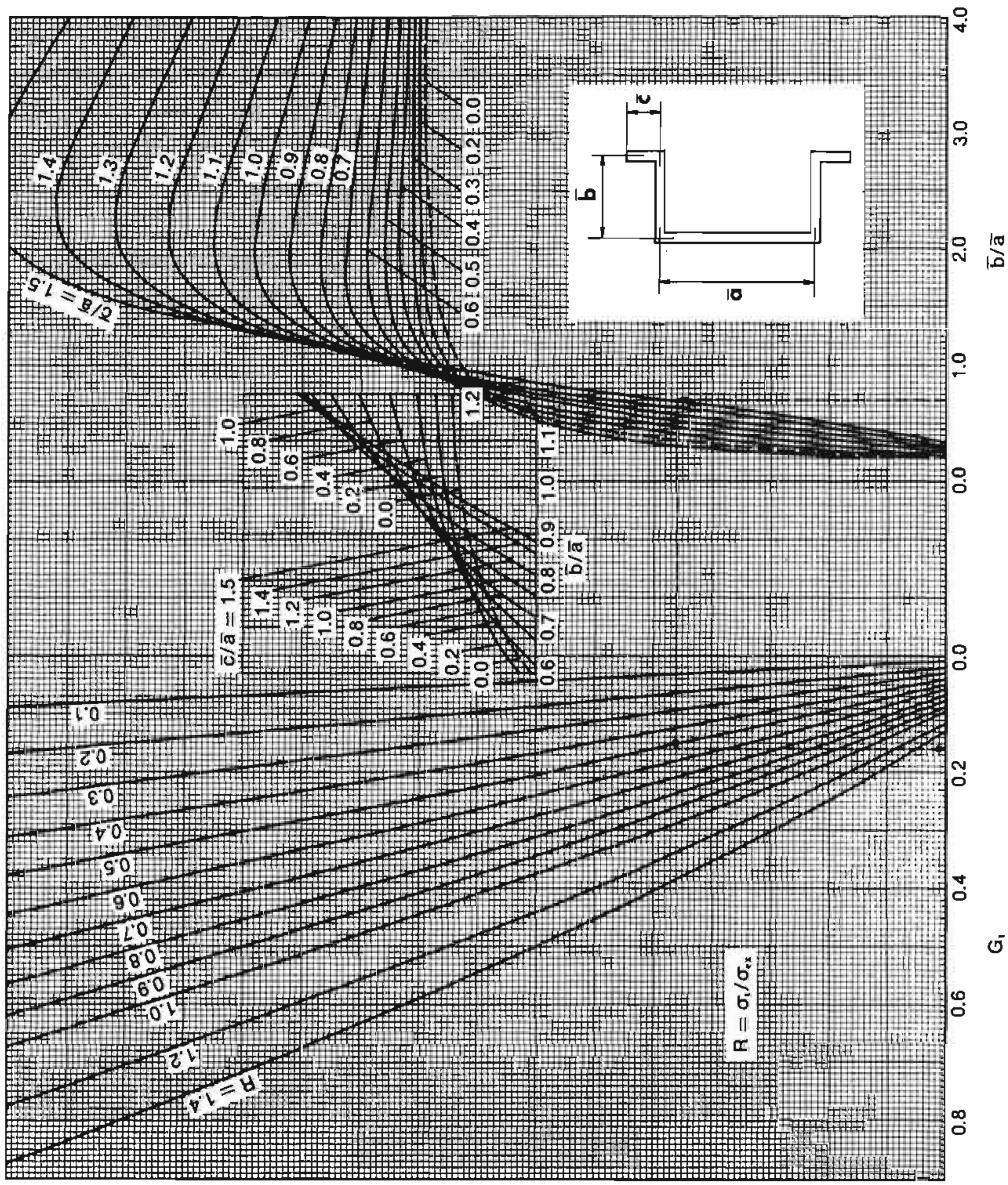


CHART V-3.7
Torsional-Flexural Buckling
(See Part III, Section 2 for application)
G_{1}-Factor for Hat Sections (Singly-Symmetric)

GROUP B
TABLES OF SECTION PROPERTIES
GENERAL NOTES

(a) The specific sections listed in these tables are not necessarily stock sections. They are included primarily as a guide in the design of cold-formed steel structural members.

(b) The effective section modulus values are calculated as the ratio of effective moment of inertia at the indicated stress level and the distance to the extreme fiber. In calculating the maximum moment capacity of these sections, additional checks such as the provisions of Chapter C of the *Specification* and the information on laterally unbraced compression flanges in Part III should also be taken into account where applicable.

(c) As a general rule, tabulated section properties are shown to three significant figures, while dimensions are given to three decimal places. However, in some cases space limitations made it impractical to adhere strictly to this guideline.

(d) The weight of these sections is calculated based on steel as weighing 40.8 pounds per square foot per inch thickness.

(e) Where they apply, the algebraic formulae presented in Part III formed the basis of the calculations for these tables.

(f) The properties of Tables 5 & 6 apply only when the channels are adequately joined together. See Chapter D of the *Specification*.

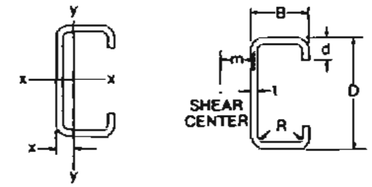
(g) The allowable bending moments shown in these tables apply only when the sections have adequate lateral bracing.

(h) Tables 1-9 incl. are Full Area Tables. Tables 10-15 incl. are Effective Area Tables. ($F_y = 50$ksi)

(i) The Full Area Tables were prepared at Cornell University by Shyh Hann Ji.

TABLE 1 TABLE 1

CHANNEL WITH STIFFENED FLANGES



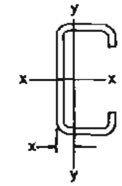
See notes on page V-25

Size D	Size B	t	d	R	Area	Wgt. per Foot	Axis x-x I_x	Axis x-x S_x	Axis x-x r_x	Axis y-y I_y	Axis y-y S_y	Axis y-y r_y	Axis y-y x	m	J	C_w	j	r_o	x_o
In.	In.	In.	In.	In.	In.²	Lb.	In.⁴	In.³	In.	In.⁴	In.³	In.	In.	In.	In.⁴	In.⁶	In.	In.	In.
12.0	3.50	0.135	1.01	0.1875	2.706	9.20	56.267	9.378	4.56	4.037	1.560	1.222	0.912	1.487	0.01640	119.00	6.78	5.27	−2.33
		0.105	0.90	0.1875	2.097	7.13	43.836	7.306	4.57	3.090	1.181	1.214	0.883	1.461	0.00771	89.90	6.84	5.26	−2.29
10.0	3.50	0.135	1.01	0.1875	2.436	8.28	36.526	7.305	3.87	3.823	1.533	1.253	1.006	1.582	0.01480	79.90	5.59	4.79	−2.52
		0.105	0.90	0.1875	1.887	6.41	28.506	5.701	3.89	2.929	1.160	1.246	0.975	1.553	0.00693	60.10	5.64	4.77	−2.48
		0.075	0.72	0.09375	1.344	4.57	20.533	4.107	3.91	2.035	0.792	1.231	0.932	1.473	0.00252	39.60	5.58	4.73	−2.37
9.0	3.25	0.135	1.00	0.1875	2.230	7.58	27.158	6.035	3.49	3.072	1.344	1.174	0.965	1.507	0.01360	53.20	5.03	4.40	−2.40
		0.105	0.84	0.1875	1.717	5.84	21.077	4.684	3.50	2.300	0.987	1.157	0.921	1.458	0.00631	38.50	5.10	4.36	−2.33
		0.075	0.70	0.09375	1.228	4.18	15.287	3.397	3.53	1.627	0.689	1.151	0.888	1.393	0.00230	25.90	5.03	4.34	−2.24
		0.060	0.61	0.09375	0.976	3.32	12.182	2.707	3.53	1.265	0.530	1.138	0.862	1.364	0.00117	19.80	5.06	4.31	−2.20
8.0	3.00	0.135	0.93	0.1875	2.009	6.83	19.372	4.843	3.11	2.356	1.127	1.083	0.909	1.409	0.01220	32.60	4.51	3.98	−2.25
		0.105	0.81	0.1875	1.553	5.28	15.125	3.781	3.12	1.794	0.844	1.075	0.876	1.375	0.00571	24.10	4.56	3.97	−2.20
		0.075	0.70	0.09375	1.116	3.79	11.046	2.761	3.15	1.290	0.600	1.075	0.851	1.321	0.00209	16.50	4.48	3.95	−2.14
		0.060	0.60	0.09375	0.885	3.01	8.791	2.198	3.15	0.997	0.458	1.061	0.822	1.288	0.00106	12.50	4.51	3.92	−2.08
8.0	1.625	0.105	0.82	0.1875	1.266	4.31	10.646	2.662	2.90	0.391	0.331	0.556	0.411	0.707	0.00465	5.79	4.95	3.12	−1.07
		0.075	0.82	0.09375	0.927	3.15	7.996	1.999	2.94	0.319	0.261	0.587	0.415	0.707	0.00174	4.43	4.69	3.18	−1.08
		0.060	0.60	0.09375	0.720	2.45	6.190	1.548	2.93	0.222	0.176	0.555	0.369	0.645	0.000864	2.96	5.00	3.14	−0.986
		0.048	0.50	0.09375	0.569	1.94	4.889	1.222	2.93	0.166	0.135	0.541	0.347	0.618	0.000437	2.20	5.13	3.13	−0.943
7.0	2.75	0.135	0.88	0.1875	1.793	6.10	13.266	3.790	2.72	1.777	0.940	0.996	0.859	1.320	0.01090	19.20	3.99	3.58	−2.11
		0.105	0.88	0.1875	1.410	4.79	10.553	3.015	2.74	1.438	0.761	1.010	0.859	1.335	0.00518	15.70	3.99	3.62	−2.14
		0.075	0.70	0.09375	1.003	3.41	7.659	2.188	2.76	1.000	0.517	0.999	0.815	1.251	0.00188	10.10	3.95	3.57	−2.03
		0.060	0.60	0.09375	0.795	2.70	6.099	1.742	2.77	0.772	0.393	0.986	0.786	1.217	0.000954	7.56	3.98	3.54	−1.97
6.0	2.50	0.135	0.82	0.1875	1.574	5.35	8.575	2.858	2.33	1.293	0.764	0.906	0.808	1.228	0.00956	10.50	3.48	3.19	−1.97
		0.105	0.82	0.1875	1.240	4.22	6.843	2.281	2.35	1.051	0.621	0.921	0.808	1.244	0.00456	8.64	3.49	3.22	−2.00
		0.075	0.82	0.09375	0.909	3.09	5.124	1.708	2.37	0.805	0.478	0.941	0.814	1.232	0.00170	6.51	3.41	3.25	−2.01
		0.060	0.60	0.09375	0.705	2.40	4.012	1.337	2.39	0.583	0.333	0.909	0.752	1.148	0.000846	4.31	3.46	3.16	−1.87
6.0	1.625	0.105	0.82	0.1875	1.056	3.59	5.246	1.748	2.23	0.360	0.323	0.584	0.481	0.795	0.00388	3.16	3.29	2.60	−1.22
		0.075	0.82	0.09375	0.777	2.79	3.972	1.324	2.26	0.293	0.256	0.615	0.486	0.791	0.00146	2.43	3.15	2.65	−1.24
		0.060	0.60	0.09375	0.600	1.68	3.085	1.028	2.27	0.205	0.172	0.585	0.437	0.722	0.000720	1.58	3.33	2.60	−1.12
		0.048	0.50	0.09375	0.473	1.61	2.441	0.814	2.27	0.154	0.128	0.571	0.412	0.693	0.000363	1.56	3.41	2.58	−1.08
5.0	2.00	0.135	0.70	0.1875	1.272	4.32	4.684	1.874	1.92	0.648	0.478	0.714	0.644	0.986	0.00773	3.73	2.89	2.58	−1.56
		0.105	0.70	0.1875	1.005	3.42	3.761	1.504	1.93	0.533	0.393	0.728	0.643	1.001	0.00369	3.10	2.90	2.61	−1.59
		0.075	0.60	0.09375	0.726	2.47	2.797	1.119	1.96	0.389	0.283	0.733	0.622	0.949	0.00136	2.10	2.83	2.60	−1.53
		0.060	0.50	0.09375	0.573	1.95	2.227	0.891	1.97	0.298	0.212	0.721	0.594	0.915	0.000687	1.54	2.86	2.57	−1.48
		0.048	0.50	0.09375	0.461	1.57	1.804	0.722	1.98	0.244	0.173	0.727	0.594	0.921	0.000354	1.26	2.86	2.58	−1.49
4.0	2.00	0.135	0.70	0.1875	1.137	3.87	2.752	1.376	1.56	0.598	0.464	0.725	0.712	1.054	0.00691	2.38	2.52	2.41	−1.70
		0.105	0.70	0.1875	0.900	3.06	2.219	1.110	1.57	0.492	0.382	0.740	0.712	1.069	0.00331	1.99	2.53	2.45	−1.73
		0.075	0.60	0.09375	0.651	2.21	1.664	0.832	1.60	0.361	0.275	0.745	0.689	1.009	0.00122	1.33	2.48	2.42	−1.66
		0.060	0.50	0.09375	0.513	1.74	1.330	0.665	1.61	0.277	0.206	0.734	0.660	0.972	0.000615	0.958	2.49	2.39	−1.60
		0.048	0.50	0.09375	0.413	1.40	1.079	0.539	1.62	0.226	0.169	0.740	0.660	0.979	0.000317	0.789	2.49	2.40	−1.61

TABLE 1 (continued)

CHANNEL WITH STIFFENED FLANGES

See notes on page V-25



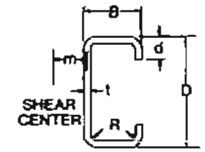


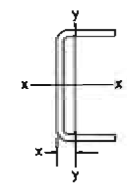
TABLE 1 (continued)

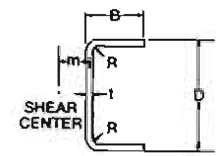
Size D	Size B	t	d	R	Area	Wgt. per Foot	Axis x-x I_x	Axis x-x S_x	Axis x-x r_x	Axis y-y I_y	Axis y-y S_y	Axis y-y r_y	Axis y-y x	m	J	C_w	j	r_o	x_o
In.	In.	In.	In.	In.	In.^{2}	Lb.	In.^{4}	In.^{3}	In.	In.^{4}	In.^{3}	In.	In.	In.	In.^{4}	In.^{6}	In.	In.	In.
4.0	1.625	0.075	0.60	0.09375	0.594	2.02	1.443	0.722	1.56	0.218	0.202	0.608	0.539	0.818	0.00113	0.817	2.26	2.12	−1.32
		0.060	0.50	0.09375	0.468	1.59	1.151	0.576	1.57	0.166	0.150	0.599	0.511	0.758	0.000558	0.586	2.29	2.10	−1.27
		0.048	0.50	0.09375	0.377	1.28	0.936	0.467	1.58	0.137	0.123	0.603	0.510	0.791	0.000290	0.485	2.29	2.12	−1.28
		0.036	0.50	0.09375	0.285	0.97	0.713	0.356	1.58	0.106	0.0949	0.609	0.511	0.799	0.000123	0.376	2.30	2.13	−1.29
3.625	1.625	0.075	0.60	0.09375	0.566	1.93	1.146	0.631	1.42	0.211	0.199	0.611	0.564	0.843	0.00107	0.672	2.13	2.07	−1.37
		0.060	0.50	0.09375	0.445	1.51	0.915	0.505	1.45	0.161	0.147	0.602	0.536	0.807	0.000531	0.479	2.15	2.04	−1.31
		0.048	0.50	0.09375	0.357	1.21	0.744	0.410	1.44	0.132	0.121	0.608	0.536	0.813	0.000274	0.395	2.15	2.05	−1.33
		0.036	0.50	0.09375	0.271	0.922	0.567	0.314	1.45	0.102	0.0939	0.615	0.536	0.819	0.000118	0.307	2.16	2.05	−1.34
3.5	2.00	0.135	0.70	0.1875	1.069	3.64	2.003	1.145	1.37	0.568	0.456	0.729	0.753	1.092	0.00650	1.85	2.39	2.36	−1.78
		0.105	0.70	0.1875	0.847	2.88	1.620	0.926	1.38	0.468	0.375	0.743	0.753	1.107	0.00311	1.55	2.40	2.39	−1.81
		0.075	0.60	0.09375	0.613	2.08	1.222	0.698	1.41	0.344	0.271	0.749	0.729	1.043	0.00115	1.02	2.35	2.36	−1.73
		0.060	0.50	0.09375	0.483	1.64	0.979	0.559	1.42	0.264	0.203	0.739	0.699	1.005	0.000579	0.728	2.35	2.32	−1.67
		0.048	0.50	0.09375	0.389	1.32	0.795	0.454	1.43	0.216	0.166	0.745	0.699	1.011	0.000299	0.601	2.36	2.33	−1.69
3.0	1.75	0.105	0.70	0.1875	0.742	2.52	1.017	0.678	1.17	0.318	0.300	0.654	0.689	1.015	0.00273	0.861	2.10	2.13	−1.65
		0.075	0.53	0.09375	0.528	1.79	0.767	0.512	1.21	0.224	0.202	0.651	0.644	0.918	0.000989	0.495	2.04	2.05	−1.52
		0.060	0.53	0.09375	0.426	1.45	0.628	0.418	1.21	0.185	0.167	0.658	0.644	0.926	0.000512	0.413	2.05	2.07	−1.54
		0.048	0.41	0.09375	0.332	1.13	0.499	0.332	1.22	0.138	0.120	0.644	0.607	0.877	0.000255	0.277	2.05	2.01	−1.46
2.5	1.625	0.075	0.60	0.09375	0.482	1.64	0.480	0.383	1.00	0.183	0.189	0.618	0.656	0.924	0.000895	0.343	1.86	1.94	−1.54
		0.060	0.50	0.09375	0.378	1.29	0.387	0.310	1.01	0.141	0.142	0.612	0.626	0.886	0.000450	0.233	1.86	1.90	−1.48
		0.048	0.50	0.09375	0.305	1.04	0.316	0.253	1.02	0.116	0.116	0.616	0.626	0.890	0.000233	0.195	1.87	1.92	−1.50
		0.036	0.50	0.09375	0.231	0.786	0.242	0.193	1.02	0.0895	0.0896	0.622	0.626	0.897	0.0000991	0.153	1.87	1.92	−1.51

All columns from I_x through x_o are under the heading "Properties of Full Section".

TABLE 2 TABLE 2

**CHANNEL
WITH UNSTIFFENED FLANGES**





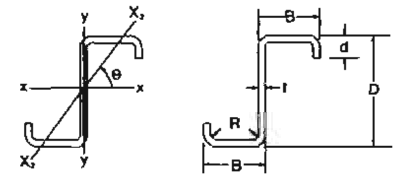
See notes on page V-25

Size D	Size B	t	R	Area	Wgt. per Foot	Axis x-x I_x	Axis x-x S_x	Axis x-x r_x	Axis y-y I_y	Axis y-y S_y	Axis y-y r_y	x	m	J	C_w	j	r_o	x_o
In.	In.	In.	In.	In.^{2}	Lb.	In.^{4}	In.^{3}	In.	In.^{4}	In.^{3}	In.	In.	In.	In.^{4}	In.^{6}	In.	In.	In.
8.0	2.00	0.135	0.18750	1.554	5.284	13.076	3.269	2.901	0.4853	0.3019	0.559	0.393	0.596	0.00944	5.56	4.84	3.09	−0.922
		0.105	0.18750	1.216	4.135	10.335	2.584	2.915	0.3862	0.2385	0.563	0.381	0.600	0.00447	4.45	4.86	3.11	−0.929
		0.075	0.09375	0.880	2.993	7.599	1.900	2.938	0.2830	0.1732	0.567	0.366	0.597	0.00165	3.27	4.89	3.13	−0.925
		0.060	0.09375	0.706	2.402	6.127	1.532	2.945	0.2290	0.1396	0.569	0.360	0.599	0.000848	2.66	4.89	3.14	−0.929
7.0	1.50	0.135	0.18750	1.284	4.366	7.840	2.240	2.471	0.2044	0.1681	0.399	0.284	0.416	0.00780	1.82	4.61	2.58	−0.633
		0.105	0.18750	1.006	3.421	6.218	1.777	2.486	0.1639	0.1335	0.404	0.272	0.421	0.00370	1.47	4.62	2.60	−0.640
		0.075	0.09375	0.730	2.483	4.603	1.315	2.511	0.1211	0.0975	0.407	0.257	0.418	0.00137	1.09	4.65	2.62	−0.638
		0.060	0.09375	0.586	1.994	3.718	1.062	2.518	0.0983	0.0788	0.410	0.251	0.420	0.000704	0.888	4.66	2.63	−0.641
6.0	1.50	0.135	0.18750	1.149	3.907	5.334	1.778	2.155	0.1973	0.1657	0.414	0.310	0.447	0.00698	1.26	3.61	2.30	−0.689
		0.105	0.18750	0.901	3.064	4.240	1.413	2.169	0.1582	0.1316	0.419	0.298	0.451	0.00331	1.02	3.62	2.32	−0.696
		0.075	0.09375	0.655	2.228	3.150	1.050	2.193	0.1171	0.0962	0.423	0.283	0.447	0.00123	0.758	3.65	2.34	−0.692
		0.060	0.09375	0.526	1.790	2.547	0.849	2.200	0.0951	0.0777	0.425	0.277	0.449	0.000632	0.618	3.66	2.35	−0.696
		0.048	0.09375	0.423	1.437	2.055	0.685	2.205	0.0770	0.0627	0.427	0.272	0.451	0.000325	0.503	3.66	2.35	−0.698
5.0	1.25	0.105	0.18750	0.744	2.529	2.399	0.960	1.796	0.0894	0.0900	0.347	0.256	0.376	0.00273	0.399	3.00	1.92	−0.580
		0.075	0.09375	0.543	1.846	1.797	0.719	1.820	0.0667	0.0661	0.350	0.241	0.372	0.00102	0.299	3.04	1.94	−0.575
		0.060	0.09375	0.436	1.484	1.456	0.583	1.827	0.0543	0.0535	0.353	0.235	0.374	0.000524	0.245	3.04	1.95	−0.579
		0.048	0.09375	0.351	1.192	1.177	0.471	1.832	0.0441	0.0432	0.355	0.230	0.376	0.000269	0.199	3.05	1.96	−0.582
4.0	1.125	0.105	0.18750	0.613	2.083	1.286	0.643	1.449	0.0623	0.0713	0.319	0.251	0.356	0.00225	0.175	2.27	1.58	−0.555
		0.060	0.09375	0.449	1.527	0.973	0.486	1.472	0.0467	0.0525	0.323	0.235	0.351	0.000842	0.132	2.30	1.60	−0.549
		0.075	0.09375	0.361	1.229	0.791	0.395	1.479	0.0382	0.0426	0.325	0.229	0.353	0.000434	0.108	2.31	1.61	−0.553
		0.048	0.09375	0.291	0.988	0.640	0.320	1.485	0.0310	0.0345	0.327	0.225	0.355	0.000223	0.0885	2.31	1.62	−0.555
3.0	1.125	0.105	0.18750	0.508	1.726	0.636	0.424	1.120	0.0573	0.0688	0.336	0.292	0.398	0.00187	0.0873	1.60	1.33	−0.637
		0.075	0.09375	0.374	1.272	0.487	0.324	1.141	0.0432	0.0508	0.340	0.275	0.390	0.000701	0.0663	1.62	1.35	−0.627
		0.060	0.09375	0.301	1.025	0.397	0.265	1.147	0.0353	0.0412	0.342	0.269	0.392	0.000362	0.0547	1.63	1.35	−0.631
		0.048	0.09375	0.243	0.825	0.322	0.215	1.153	0.0287	0.0334	0.344	0.264	0.393	0.000186	0.0448	1.64	1.36	−0.634
2.0	1.125	0.105	0.18750	0.403	1.369	0.241	0.241	0.773	0.0497	0.0645	0.351	0.355	0.450	0.00148	0.0326	1.17	1.13	−0.752
		0.075	0.09375	0.299	1.017	0.188	0.188	0.792	0.0379	0.0480	0.356	0.335	0.438	0.000561	0.0251	1.19	1.14	−0.735
		0.060	0.09375	0.241	0.821	0.154	0.154	0.798	0.0310	0.0389	0.358	0.329	0.440	0.000290	0.0208	1.20	1.15	−0.738
		0.048	0.09375	0.195	0.661	0.126	0.126	0.804	0.0253	0.0315	0.360	0.324	0.441	0.000149	0.0171	1.21	1.15	−0.741

TABLE 3 TABLE 3

Z-SECTION WITH STIFFENED FLANGES

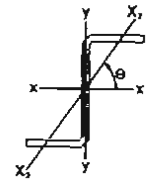
See notes on page V-25

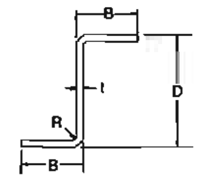


Size D	Size B	t	d	R	Area	Wgt. per Foot	Axis x-x I_x	Axis x-x S_x	Axis x-x r_x	Axis y-y I_y	Axis y-y S_y	Axis y-y r_y	I_{xy}	Axis x_2-x_2 r_{min}	$90°-\theta$	J	C_w
In.	In.	In.	In.	In.	In.²	Lb.	In.⁴	In.³	In.	In.⁴	In.³	In.	In.⁴	In.	Deg.	In.⁴	In.⁶
12.0	3.50	0.135	1.01	0.1875	2.706	9.20	56.267	9.378	4.56	5.968	1.739	1.485	13.152	1.006	13.80	0.0164	162.00
		0.105	0.90	0.1875	2.097	7.13	43.836	7.306	4.57	4.535	1.316	1.471	10.097	0.999	13.60	0.00771	123.00
10.0	3.50	0.135	1.01	0.1875	2.436	8.28	36.526	7.305	3.87	5.968	1.739	1.565	10.867	1.013	17.71	0.0148	108.00
		0.105	0.90	0.1875	1.887	6.41	28.506	5.701	3.89	4.535	1.316	1.550	8.356	1.006	17.44	0.00693	82.20
		0.075	0.72	0.09375	1.344	4.57	20.533	4.107	3.91	3.109	0.898	1.521	5.842	0.996	16.92	0.00252	54.80
9.0	3.25	0.135	1.00	0.1875	2.230	7.58	27.158	6.035	3.49	4.869	1.530	1.477	8.506	0.945	18.68	0.0136	71.60
		0.105	0.84	0.1875	1.717	5.84	21.077	4.684	3.50	3.594	1.124	1.447	6.423	0.931	18.15	0.00631	52.50
		0.075	0.70	0.09375	1.228	4.18	15.287	3.397	3.53	2.516	0.783	1.431	4.557	0.927	17.76	0.00230	35.70
		0.060	0.61	0.09375	0.976	3.32	12.182	2.707	3.53	1.941	0.603	1.410	3.566	0.918	17.43	0.00117	27.50
8.0	3.00	0.135	0.93	0.1875	2.009	6.83	19.372	4.843	3.11	3.779	1.288	1.371	6.358	0,868	19.60	0.0122	43.70
		0.105	0.81	0.1875	1.553	5.28	15.125	3.781	3.12	2.846	0.965	1.354	4.868	0.860	19.21	0.00571	32.80
		0.075	0.70	0.09375	1.116	3.79	11.046	2.761	3.15	2.028	0.685	1.348	3.500	0.862	18.91	0.00209	22.70
		0.060	0.60	0.09375	0.885	3.01	8.791	2.198	3.15	1.552	0.523	1.324	2.725	0.851	18.49	0.00106	17.30
7.0	2.75	0.135	0.88	0.1875	1.793	6.10	13.266	3.790	2.72	2.901	1.082	1.272	4.638	0.794	20.91	0.0109	25.60
		0.105	0.88	0.1875	1.410	4.79	10.553	3.015	2.74	2.355	0.873	1.292	3.728	0.805	21.14	0.00518	20.80
		0.075	0.70	0.09375	1.003	3.41	7.659	2.188	2.76	1.607	0.592	1.266	2.612	0.796	20.40	0.00188	13.70
		0.060	0.60	0.09375	0.795	2.70	6.099	1.742	2.77	1.227	0.451	1.242	2.033	0.785	19.92	0.000954	10.40
6.0	2.50	0.135	0.82	0.1875	1.574	5.35	8.575	2.858	2.33	2.156	0.886	1.170	3.236	0.716	22.62	0.00956	13.90
		0.105	0.82	0.1875	1.240	4.22	6.843	2.281	2.35	1.758	0.718	1.191	2.611	0.728	22.88	0.00456	11.40
		0.075	0.82	0.09375	0.909	3.09	5.124	1.708	2.37	1.353	0.549	1.220	1.977	0.746	23.18	0.00170	8.53
		0.060	0.60	0.09375	0.705	2.40	4.012	1.337	2.39	0.950	0.385	1.161	1.463	0.718	21.85	0.000846	5.87
5.0	2.00	0.135	0.70	0.1875	1.272	4.32	4.684	1.874	1.92	1.071	0.554	0.917	1.681	0.567	21.47	0.00773	4.90
		0.105	0.70	0.1875	1.005	3.42	3.761	1.504	1.93	0.884	0.454	0.938	1.369	0.579	21.79	0.00369	4.06
		0.075	0.60	0.09375	0.726	2.47	2.797	1.119	1.96	0.637	0.325	0.937	0.998	0.583	21.38	0.00136	2.80
		0.060	0.50	0.09375	0.573	1.95	2.227	0.891	1.97	0.480	0.244	0.915	0.772	0.573	20.73	0.000687	2.09
		0.048	0.50	0.09375	0.461	1.57	1.804	0.722	1.98	0.393	0.199	0.924	0.629	0.577	20.86	0.000354	1.72
4.0	2.00	0.135	0.70	0.1875	1.137	3.87	2.752	1.376	1.56	1.071	0.554	0.970	1.315	0.555	28.70	0.00691	3.00
		0.105	0.70	0.1875	0.900	3.06	2.219	1.110	1.57	0.884	0.454	0.991	1.072	0.566	29.04	0.00331	2.50
		0.075	0.60	0.09375	0.651	2.21	1.664	0.832	1.60	0.637	0.325	0.990	0.786	0.570	28.43	0.00122	1.72
		0.060	0.50	0.09375	0.513	1.74	1.330	0.665	1.61	0.480	0.244	0.967	0.610	0.561	27.57	0.000615	1.27
		0.048	0.50	0.09375	0.413	1.40	1.079	0.539	1.62	0.393	0.199	0.976	0.498	0.565	27.72	0.000317	1.05
3.5	2.00	0.135	0.70	0.1875	1.069	3.64	2.003	1.145	1.37	1.071	0.554	1.001	1.131	0.541	33.80	0.00650	2.24
		0.105	0.70	0.1875	0.847	2.88	1.620	0.926	1.38	0.884	0.454	1.021	0.924	0.552	34.13	0.00311	1.87
		0.075	0.60	0.09375	0.613	2.08	1.222	0.698	1.41	0.637	0.325	1.020	0.680	0.556	33.37	0.00115	1.28
		0.060	0.50	0.09375	0.483	1.64	0.979	0.559	1.42	0.480	0.244	0.997	0.529	0.546	32.38	0.000579	0.947
		0.048	0.50	0.09375	0.389	1.32	0.795	0.454	1.43	0.393	0.199	1.005	0.432	0.550	32.52	0.000299	0.780
3.0	1.75	0.105	0.70	0.1875	0.742	2.52	1.017	0.678	1.17	0.618	0.364	0.913	0.611	0.486	35.96	0.00273	0.990
		0.075	0.53	0.09375	0.526	1.79	0.767	0.512	1.21	0.418	0.244	0.890	0.437	0.480	34.11	0.000989	0.617
		0.060	0.53	0.09375	0.426	1.45	0.628	0.418	1.21	0.346	0.201	0.900	0.360	0.485	34.30	0.000512	0.514
		0.048	0.41	0.09375	0.332	1.13	0.499	0.332	1.22	0.251	0.145	0.868	0.274	0.472	32.82	0.000255	0.362

TABLE 4 TABLE 4

Z-SECTION WITH UNSTIFFENED FLANGES



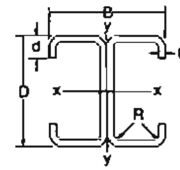


See notes on page V-25

Size		t	R	Area	Wgt. per Foot	Properties of Full Section										
						Axis x-x			Axis y-y			I_{xy}	Axis x_2-x_2	90°-θ	J	C_w
D	B					I_x	S_x	r_x	I_y	S_y	r_y		r_{min}			
In.	In.	In.	In.	In.^{2}	Lb.	In.^{4}	In.^{3}	In.	In.^{4}	In.^{3}	In.	In.^{4}	In.	Deg.	In.^{4}	In.^{6}
8.0	2.000	0.135	0.1875	1.554	5.284	13.076	3.269	2.901	0.6496	0.3362	0.647	1.9872	0.467	8.87	0.00944	7.56
		0.105	0.1875	1.216	4.135	10.335	2.584	2.915	0.5171	0.2655	0.652	1.5752	0.472	8.90	0.00447	6.06
		0.075	0.09375	0.880	2.993	7.599	1.900	2.938	0.3779	0.1926	0.655	1.1453	0.477	8.80	0.00165	4.46
		0.060	0.09375	0.706	2.402	6.127	1.532	2.945	0.3058	0.1552	0.658	0.9249	0.479	8.81	0.000848	3.62
7.0	1.500	0.135	0.1875	1.284	4.366	7.840	2.240	2.471	0.2647	0.1848	0.454	0.9548	0.337	7.07	0.00780	2.43
		0.105	0.1875	1.006	3.421	6.218	1.777	2.486	0.2124	0.1467	0.459	0.7612	0.342	7.11	0.00370	1.96
		0.075	0.09375	0.730	2.483	4.603	1.315	2.511	0.1564	0.1070	0.463	0.5561	0.347	7.02	0.00137	1.46
		0.060	0.09375	0.586	1.994	3.718	1.062	2.518	0.1271	0.0864	0.465	0.4503	0.349	7.04	0.000704	1.19
6.0	1.500	0.135	0.1875	1.149	3.907	5.334	1.778	2.155	0.2647	0.1840	0.480	0.8157	0.345	8.92	0.00698	1.72
		0.105	0.1875	0.901	3.064	4.240	1.413	2.169	0.2124	0.1467	0.485	0.6508	0.349	8.95	0.00331	1.39
		0.075	0.09375	0.655	2.228	3.150	1.050	2.193	0.1564	0.1070	0.489	0.4758	0.355	8.82	0.00123	1.03
		0.060	0.09375	0.526	1.790	2.547	0.849	2.200	0.1271	0.0864	0.491	0.3854	0.357	8.84	0.000632	0.842
		0.048	0.09375	0.423	1.437	2.055	0.685	2.205	0.1029	0.0697	0.493	0.3115	0.359	8.85	0.000325	0.685
5.0	1.250	0.105	0.1875	0.744	2.529	2.399	0.960	1.796	0.1203	0.1004	0.402	0.3704	0.288	9.00	0.00273	0.543
		0.075	0.09375	0.543	1.846	1.797	0.719	1.820	0.0891	0.0735	0.405	0.2719	0.294	8.83	0.00102	0.407
		0.060	0.09375	0.436	1.484	1.456	0.583	1.827	0.0726	0.0595	0.408	0.2209	0.296	8.85	0.000524	0.333
		0.048	0.09375	0.351	1.192	1.177	0.471	1.832	0.0590	0.0481	0.410	0.1789	0.298	8.87	0.000269	0.272
4.0	1.500	0.060	0.09375	0.406	1.382	0.965	0.483	1.546	0.1271	0.0865	0.561	0.2558	0.370	15.74	0.000485	0.335
4.0	1.125	0.105	0.1875	0.613	2.083	1.286	0.643	1.449	0.0864	0.0806	0.376	0.2367	0.260	10.77	0.00225	0.240
		0.075	0.09375	0.449	1.527	0.973	0.486	1.472	0.0643	0.0591	0.378	0.1744	0.267	10.50	0.000842	0.181
		0.060	0.09375	0.361	1.229	0.791	0.395	1.479	0.0525	0.0480	0.381	0.1420	0.269	10.52	0.000434	0.149
		0.048	0.09375	0.291	0.988	0.640	0.320	1.485	0.0427	0.0388	0.383	0.1151	0.271	10.54	0.000223	0.122
3.0	1.500	0.060	0.09375	0.346	1.178	0.494	0.330	1.198	0.1271	0.0865	0.607	0.1906	0.365	23.06	0.000413	0.171
3.0	1.125	0.105	0.1875	0.508	1.726	0.636	0.424	1.120	0.0864	0.0806	0.413	0.1759	0.263	16.30	0.00187	0.123
		0.075	0.09375	0.374	1.272	0.487	0.324	1.141	0.0643	0.0591	0.415	0.1300	0.271	15.80	0.000701	0.0936
		0.060	0.09375	0.301	1.025	0.397	0.265	1.147	0.0525	0.0480	0.417	0.1059	0.273	15.80	0.000362	0.0771
		0.048	0.09375	0.243	0.825	0.322	0.215	1.153	0.0427	0.0388	0.420	0.0860	0.275	15.80	0.000186	0.0632
2.0	1.125	0.105	0.1875	0.403	1.369	0.241	0.241	0.773	0.0864	0.0806	0.463	0.1150	0.250	28.07	0.00148	0.0467
		0.075	0.09375	0.299	1.017	0.188	0.188	0.792	0.0643	0.0591	0.464	0.0855	0.262	27.12	0.000561	0.0359
		0.060	0.09375	0.241	0.821	0.154	0.154	0.798	0.0525	0.0480	0.466	0.0699	0.264	27.03	0.000290	0.0298
		0.048	0.09375	0.195	0.661	0.126	0.126	0.804	0.0427	0.0388	0.469	0.0569	0.266	26.95	0.000149	0.0245
1.5	1.500	0.048	0.09375	0.207	0.702	0.084	0.112	0.641	0.1029	0.0697	0.709	0.0760	0.287	41.53	0.000159	0.0270
		0.036	0.09375	0.156	0.530	0.0652	0.0862	0.646	0.0781	0.0527	0.710	0.0580	0.291	41.70	0.0000670	0.0209

TABLE 5 TABLE 5

TWO CHANNELS WITH STIFFENED FLANGES BACK-TO-BACK



See notes on page V-25

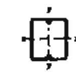
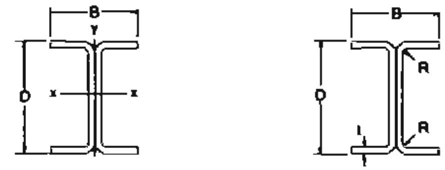
Size		t	d	R	Area	Wgt. per Foot	Properties of Full Section									
							Axis x-x			Axis y-y			J	C_w		
D	B						I_x	S_x	r_x	I_y	S_y	r_y			I_y	r_y
In.	In.	In.	In.	In.	$In.^2$	Lb.	$In.^4$	$In.^3$	In.	$In.^4$	$In.^3$	In.	$In.^4$	$In.^6$	$In.^4$	In.
12.0	7.00	0.135	1.01	0.1875	5.411	18.40	113.0	18.756	4.56	12.578	3.594	1.525	0.0329	478.00	44.31	2.86
		0.105	0.90	0.1875	4.193	14.26	87.7	14.612	4.57	9.448	2.699	1.501	0.0154	361.00	34.90	2.88
10.0	7.00	0.135	1.01	0.1875	4.871	16.56	73.1	14.611	3.87	12.575	3.593	1.607	0.0296	336.00	37.95	2.79
		0.105	0.90	0.1875	3.773	12.83	57.0	11.402	3.89	9.447	2.699	1.582	0.0139	253.00	29.91	2.82
		0.075	0.72	0.09375	2.687	9.14	41.1	8.213	3.91	6.403	1.829	1.544	0.00504	167.00	21.79	2.85
9.0	6.50	0.135	1.00	0.1875	4.461	15.17	54.3	12.070	3.49	10.299	3.169	1.519	0.0271	225.00	29.43	2.57
		0.105	0.84	0.1875	3.433	11.67	42.2	9.368	3.50	7.511	2.311	1.479	0.0126	163.00	23.22	2.60
		0.075	0.70	0.09375	2.456	8.35	30.6	6.794	3.53	5.192	1.597	1.454	0.00461	110.00	16.96	2.63
		0.060	0.61	0.09375	1.952	6.64	24.4	5.414	3.53	3.981	1.225	1.428	0.00234	84.00	13.66	2.65
8.0	6.00	0.135	0.93	0.1875	4.018	13.66	38.7	9.686	3.11	8.032	2.677	1.414	0.0244	138.00	22.28	2.35
		0.105	0.81	0.1875	3.105	10.56	30.3	7.563	3.12	5.968	1.989	1.386	0.0114	103.00	17.60	2.38
		0.075	0.70	0.09375	2.231	7.59	22.1	5.523	3.15	4.196	1.399	1.371	0.00418	70.70	12.88	2.40
		0.060	0.60	0.09375	1.770	6.02	17.6	4.395	3.15	3.189	1.063	1.342	0.00212	53.50	10.39	2.42
7.0	5.50	0.135	0.88	0.1875	3.586	12.19	26.5	7.581	2.72	6.202	2.255	1.315	0.0218	82.30	16.38	2.14
		0.105	0.88	0.1875	2.820	9.59	21.1	6.030	2.74	4.957	1.803	1.326	0.0104	67.20	12.96	2.14
		0.075	0.70	0.09375	2.006	6.82	15.3	4.377	2.76	3.333	1.212	1.289	0.00376	43.50	9.51	2.18
		0.060	0.60	0.09375	1.590	5.40	12.2	3.485	2.77	2.527	0.919	1.261	0.00191	32.70	7.68	2.20
6.0	5.00	0.135	0.82	0.1875	3.149	10.71	17.2	5.717	2.33	4.641	1.856	1.214	0.0191	45.50	11.60	1.92
		0.105	0.82	0.1875	2.480	8.43	13.7	4.562	2.35	3.720	1.488	1.225	0.00911	37.30	9.20	1.93
		0.075	0.82	0.09375	1.817	6.18	10.2	3.416	2.37	2.814	1.125	1.244	0.00341	28.10	6.78	1.93
		0.060	0.60	0.09375	1.410	4.79	8.02	2.674	2.39	1.962	0.785	1.180	0.00169	18.90	5.48	1.97
5.0	4.00	0.135	0.70	0.1875	2.544	8.65	9.37	3.747	1.92	2.351	1.175	0.961	0.0155	15.90	5.98	1.53
		0.105	0.70	0.1875	2.009	6.83	7.52	3.009	1.93	1.898	0.949	0.972	0.00738	13.20	4.76	1.54
		0.075	0.60	0.09375	1.451	4.93	5.59	2.238	1.96	1.340	0.670	0.961	0.00272	9.05	3.53	1.56
		0.060	0.50	0.09375	1.146	3.90	4.45	1.781	1.97	0.999	0.500	0.934	0.00137	6.68	2.86	1.58
		0.048	0.50	0.09375	0.922	3.14	3.61	1.443	1.98	0.812	0.406	0.939	0.000708	5.49	2.31	1.58
4.0	4.00	0.135	0.70	0.1875	2.274	7.73	5.50	2.752	1.56	2.349	1.175	1.016	0.0138	10.40	4.97	1.48
		0.105	0.70	0.1875	1.799	6.12	4.44	2.219	1.57	1.897	0.949	1.027	0.00661	8.69	3.97	1.48
		0.075	0.60	0.09375	1.301	4.42	3.33	1.664	1.60	1.340	0.670	1.015	0.00244	5.91	2.96	1.51
		0.060	0.50	0.09375	1.026	3.49	2.66	1.330	1.61	0.999	0.500	0.987	0.00123	4.34	2.40	1.53
		0.048	0.50	0.09375	0.826	2.81	2.16	1.079	1.62	0.812	0.406	0.992	0.000634	3.57	1.94	1.53
3.5	4.00	0.135	0.70	0.1875	2.139	7.27	4.01	2.289	1.37	2.349	1.174	1.048	0.0130	8.09	4.46	1.44
		0.105	0.70	0.1875	1.694	5.76	3.24	1.852	1.38	1.897	0.949	1.058	0.00623	6.77	3.57	1.45
		0.075	0.60	0.09375	1.226	4.17	2.44	1.397	1.41	1.340	0.670	1.045	0.00230	4.59	2.67	1.48
		0.060	0.50	0.09375	0.966	3.28	1.96	1.119	1.42	0.999	0.500	1.017	0.00116	3.36	2.16	1.50
		0.048	0.50	0.09375	0.778	2.65	1.59	0.909	1.43	0.812	0.406	1.022	0.000598	2.77	1.75	1.50
3.0	3.50	0.105	0.70	0.1875	1.484	5.05	2.03	1.356	1.17	1.340	0.766	0.950	0.00545	3.64	2.31	1.25
		0.075	0.53	0.09375	1.055	3.59	1.53	1.023	1.21	0.885	0.506	0.916	0.00198	2.22	1.74	1.28
		0.060	0.53	0.09375	0.853	2.90	1.26	0.837	1.21	0.724	0.414	0.921	0.00102	1.86	1.41	1.29
		0.048	0.41	0.09375	0.665	2.26	0.997	0.665	1.22	0.520	0.297	0.885	0.000511	1.29	1.14	1.31

TABLE 6 TABLE 6

2 CHANNELS WITH UNSTIFFENED FLANGES BACK-TO-BACK



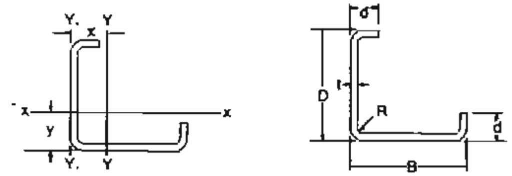
See notes on page V-25

Size D	Size B	t	R	Area	Weight per Foot	Axis x-x I_x	Axis x-x S_x	Axis x-x r_x	Axis y-y I_y	Axis y-y S_y	Axis y-y r_y	J	C_w
In.	In.	In.	In.	In.^{2}	Lb.	In.^{4}	In.^{3}	In.	In.^{4}	In.^{3}	In.	In.^{4}	In.^{6}
8.0	4.00	0.135	0.1875	3.108	10.57	26.151	6.538	2.901	1.4499	0.7249	0.683	0.0189	20.10
		0.105	0.1875	2.433	8.27	20.670	5.168	2.915	1.1247	0.5624	0.680	0.00894	16.10
		0.075	0.09375	1.761	5.99	15.198	3.799	2.938	0.8017	0.4008	0.675	0.00330	11.90
		0.060	0.09375	1.413	4.80	12.254	3.064	2.945	0.6409	0.3204	0.673	0.00170	9.64
7.0	3.00	0.135	0.1875	2.568	8.73	15.680	4.480	2.471	0.6162	0.4108	0.490	0.0156	6.23
		0.105	0.1875	2.013	6.84	12.436	3.553	2.486	0.4767	0.3178	0.487	0.00740	5.05
		0.075	0.09375	1.461	4.97	9.206	2.630	2.511	0.3390	0.2260	0.482	0.00274	3.75
		0.060	0.09375	1.173	3.99	7.435	2.124	2.518	0.2708	0.1805	0.480	0.00141	3.06
6.0	3.00	0.135	0.1875	2.298	7.81	10.669	3.556	2.155	0.6149	0.4100	0.517	0.0140	4.55
		0.105	0.1875	1.803	6.13	8.481	2.827	2.169	0.4761	0.3174	0.514	0.00662	3.69
		0.075	0.09375	1.311	4.46	6.300	2.100	2.193	0.3388	0.2258	0.508	0.00246	2.75
		0.060	0.09375	1.053	3.58	5.094	1.698	2.200	0.2707	0.1804	0.507	0.00126	2.24
		0.048	0.09375	0.845	2.87	4.110	1.370	2.205	0.2163	0.1442	0.506	0.000649	1.82
5.0	2.50	0.105	0.1875	1.488	5.06	4.798	1.919	1.796	0.2764	0.2212	0.431	0.00547	1.44
		0.075	0.09375	1.086	3.69	3.595	1.438	1.820	0.1964	0.1571	0.425	0.00204	1.08
		0.060	0.09375	0.873	2.97	2.913	1.165	1.827	0.1568	0.1254	0.424	0.00105	0.886
		0.048	0.09375	0.701	2.38	2.354	0.942	1.832	0.1253	0.1002	0.423	0.000538	0.723
4.0	2.25	0.105	0.1875	1.225	4.17	2.572	1.286	1.449	0.2018	0.1793	0.406	0.00450	0.655
		0.075	0.09375	0.898	3.05	1.946	0.973	1.472	0.1432	0.1273	0.399	0.00168	0.495
		0.060	0.09375	0.723	2.46	1.581	0.791	1.479	0.1143	0.1016	0.398	0.000867	0.408
		0.048	0.09375	0.581	1.98	1.281	0.640	1.485	0.0914	0.0812	0.397	0.000446	0.334
3.0	2.25	0.105	0.1875	1.015	3.45	1.273	0.849	1.120	0.2012	0.1788	0.445	0.00373	0.362
		0.075	0.09375	0.748	2.54	0.973	0.649	1.141	0.1430	0.1271	0.437	0.00140	0.275
		0.060	0.09375	0.603	2.05	0.794	0.529	1.147	0.1142	0.1015	0.435	0.000723	0.227
		0.048	0.09375	0.485	1.65	0.645	0.430	1.153	0.0913	0.0812	0.434	0.000373	0.186
2.0	2.25	0.105	0.1875	0.806	2.74	0.482	0.482	0.773	0.2006	0.1783	0.499	0.00296	0.155
		0.075	0.09375	0.598	2.03	0.375	0.375	0.792	0.1428	0.1269	0.489	0.00112	0.119
		0.060	0.09375	0.483	1.64	0.308	0.308	0.798	0.1141	0.1014	0.486	0.000579	0.0988
		0.048	0.09375	0.389	1.32	0.251	0.251	0.804	0.0912	0.0811	0.484	0.000299	0.0814

TABLE 7

EQUAL LEG ANGLE WITH STIFFENED FLANGES

See notes on page V-25



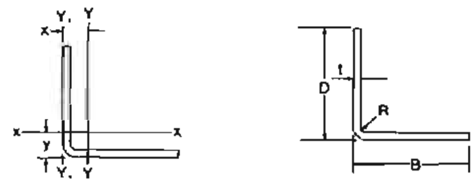
Size D	Size B	t	d	R	Area	Wgt. per Foot	Axis x-x and Axis y-y I	S	r	x = y	r_{y_1}	J	C_w	r_o	I_{y_2}	r_{y_2}	j	x_o
In.	In.	In.	In.	In.	In.^{2}	Lb.	In.^{4}	In.^{3}	In.	In.	In.	In.^{4}	In.^{6}	In.	In.^{4}	In.	In.	In.
4.0	4.0	0.135	1.10	0.1875	1.278	4.345	2.594	0.957	1.425	1.290	1.922	0.00776	1.1058	2.80	1.2445	0.987	3.11	−1.940
3.0	3.0	0.135	0.93	0.1875	0.962	3.271	1.083	0.541	1.061	0.999	1.457	0.00584	0.3503	2.12	0.5399	0.749	2.33	−1.498
		0.105	0.81	0.1875	0.735	2.498	0.828	0.407	1.062	0.964	1.434	0.00270	0.1910	2.07	0.3944	0.733	2.33	−1.423
2.5	2.5	0.135	0.82	0.1875	0.797	2.711	0.613	0.371	0.876	0.847	1.219	0.00484	0.1579	1.77	0.3114	0.625	1.93	−1.258
		0.105	0.82	0.1875	0.632	2.148	0.500	0.302	0.889	0.843	1.226	0.00232	0.1335	1.80	0.2547	0.635	1.95	−1.284
2.0	2.0	0.135	0.82	0.1875	0.662	2.252	0.325	0.255	0.700	0.725	1.008	0.00402	0.0972	1.49	0.1829	0.525	1.56	−1.119
		0.105	0.82	0.1875	0.527	1.791	0.268	0.209	0.713	0.721	1.014	0.00194	0.0828	1.53	0.1510	0.535	1.58	−1.144
		0.075	0.60	0.09375	0.360	1.226	0.184	0.137	0.714	0.659	0.972	0.000676	0.0244	1.43	0.0907	0.502	1.56	−1.005
		0.060	0.50	0.09375	0.280	0.951	0.141	0.103	0.710	0.631	0.950	0.000336	0.0118	1.37	0.0658	0.485	1.55	−0.936

Columns from r_{y_1} through x_o: Properties of Full Section.

TABLE 8

EQUAL LEG ANGLE WITH UNSTIFFENED FLANGES

See notes on page V-25

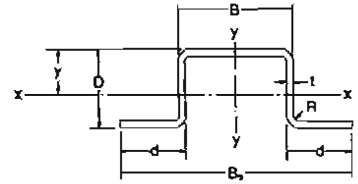


Size D	Size B	t	R	Area	Wgt. per Foot	Axis x-x and Axis y-y I	S	r	x = y	r_{y_1}	J	C_w	r_o	Axis y I_{y_2}	r_{y_2}	j	x_o
In.	In.	In.	In.	In.^{2}	Lb.	In.^{4}	In.^{3}	In.	In.	In.	In.^{4}	In.^{6}	In.	In.^{4}	In.	In.	In.
4.0	4.0	0.135	0.1875	1.047	3.560	1.6947	0.5774	1.272	1.065	1.659	0.00636	0.000	2.22	0.6529	0.790	2.76	−1.305
3.0	3.0	0.135	0.1875	0.777	2.642	0.7003	0.3206	0.949	0.815	1.251	0.00472	0.000	1.65	0.2659	0.585	2.06	−0.952
		0.105	0.1875	0.608	2.068	0.5536	0.2520	0.954	0.803	1.247	0.00223	0.000	1.66	0.2110	0.589	2.07	−0.962
2.5	2.5	0.135	0.1875	0.642	2.183	0.3985	0.2202	0.788	0.691	1.048	0.00390	0.000	1.36	0.1494	0.482	1.71	−0.776
		0.105	0.1875	0.503	1.711	0.3161	0.1735	0.793	0.678	1.043	0.00185	0.000	1.37	0.1191	0.486	1.72	−0.786
2.0	2.0	0.135	0.1875	0.507	1.724	0.1989	0.1387	0.626	0.566	0.844	0.00308	0.000	1.07	0.0731	0.380	1.36	−0.599
		0.105	0.1875	0.398	1.354	0.1585	0.1096	0.631	0.554	0.840	0.00146	0.000	1.08	0.0586	0.384	1.37	−0.609
		0.075	0.09375	0.290	0.987	0.1170	0.0799	0.635	0.536	0.831	0.000544	0.000	1.11	0.0450	0.394	1.38	−0.650
		0.060	0.09375	0.233	0.793	0.0947	0.0644	0.637	0.530	0.829	0.000280	0.000	1.11	0.0365	0.396	1.38	−0.655

Columns from r_{y_1} through x_o: Properties of Full Section.

TABLE 9 TABLE 9

HAT SECTIONS

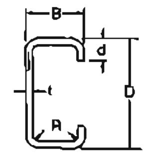


See notes on page V-25

Size D	Size B	t	d	R	Area	Wgt per Foot	I_x	S_x	r_x	I_y	S_y	r_y	y	J	C_w	j	r_o	x_o
In.	In.	In.	In.	In.	In.^{2}	Lb.	In.^{4}	In.^{3}	In.	In.^{4}	In.^{3}	In.	In.	In.^{4}	In.^{6}	In.	In.	In.
10.0	15.00	0.135	1.670	0.1875	5.044	17.149	67.50	10.3279	3.659	210.00	23.2710	6.456	3.461	0.0306	2340.00	11.21	10.87	-7.95
		0.105	1.340	0.1875	3.869	13.155	50.20	7.5430	3.602	160.00	18.2608	6.421	3.346	0.0142	1810.00	11.06	10.72	-7.79
10.0	10.00	0.105	1.340	0.1875	3.344	11.370	43.60	7.1050	3.611	66.80	10.7092	4.468	3.863	0.0123	685.00	10.16	10.29	-8.54
		0.075	1.050	0.09375	2.368	8.052	30.20	4.8420	3.572	46.90	7.8514	4.451	3.758	0.00444	493.00	9.99	10.14	-8.38
10.0	5.00	0.075	1.050	0.09375	1.993	6.777	24.10	4.3405	3.474	11.00	3.1722	2.352	4.458	0.00374	98.40	9.74	10.17	-9.27
8.0	12.00	0.135	1.670	0.1875	4.099	13.936	36.30	7.0912	2.977	111.00	14.7525	5.208	2.878	0.0249	762.00	9.13	8.85	-6.51
		0.105	1.340	0.1875	3.134	10.656	26.80	5.1291	2.926	83.60	11.5586	5.166	2.767	0.0115	597.00	9.00	8.71	-6.37
8.0	8.00	0.105	1.340	0.1875	2.714	9.228	23.30	4.8340	2.928	35.20	6.7247	3.602	3.188	0.00997	222.00	8.24	8.35	-6.94
		0.075	0.980	0.09375	1.908	6.486	15.80	3.2011	2.879	24.30	4.9542	3.569	3.059	0.00358	161.00	8.06	8.18	-6.78
8.0	4.00	0.075	0.980	0.09375	1.608	5.466	12.60	2.8710	2.796	5.74	1.9756	1.889	3.623	0.00301	319.00	7.80	8.17	-7.44
		0.060	0.840	0.09375	1.274	4.330	9.82	2.2149	2.777	4.51	1.6215	1.881	3.566	0.00153	25.80	7.80	8.14	-7.42
6.0	9.00	0.135	1.670	0.1875	3.154	10.723	16.40	4.4300	2.283	49.70	8.2408	3.971	2.290	0.0192	177.00	7.03	6.79	-5.02
		0.105	1.340	0.1875	2.399	8.157	12.10	3.1629	2.243	36.80	6.4229	3.918	2.185	0.00882	140.00	6.92	6.68	-4.92
6.0	6.00	0.105	1.340	0.1875	2.084	7.086	10.40	2.9828	2.236	15.70	3.7063	2.744	2.507	0.00766	51.60	6.28	6.37	-5.30
		0.075	0.915	0.09375	1.448	4.923	6.91	1.8989	2.185	10.50	2.7299	2.691	2.360	0.00271	37.70	6.12	6.22	-5.16
6.0	3.00	0.075	0.915	0.09375	1.223	4.158	5.47	1.7042	2.116	2.51	1.0707	1.431	2.787	0.00229	75.50	5.84	6.14	-5.58
		0.060	0.760	0.09375	0.964	3.278	4.24	1.2947	2.097	1.94	0.8812	1.418	2.726	0.00116	60.40	5.86	6.13	-5.59
		0.048	0.660	0.09375	0.764	2.599	3.32	1.0019	2.084	1.53	0.7221	1.412	2.686	0.000587	48.80	5.86	6.12	-5.58
4.0	6.00	0.135	1.670	0.1875	2.209	7.510	5.42	2.3437	1.567	16.90	3.7178	2.763	1.686	0.0134	22.60	4.86	4.66	-3.41
		0.105	1.340	0.1875	1.664	5.658	3.96	1.6444	1.543	12.00	2.8447	2.691	1.592	0.00612	17.90	4.79	4.60	-3.40
4.0	4.00	0.105	1.340	0.1875	1.454	4.944	3.39	1.5510	1.527	5.31	1.6409	1.911	1.814	0.00534	6.89	4.22	4.31	-3.54
		0.075	0.915	0.09375	0.998	3.393	2.23	0.9602	1.494	3.35	1.1788	1.832	1.680	0.00187	4.83	4.17	4.25	-3.53
4.0	2.00	0.075	0.915	0.09375	0.848	2.883	1.75	0.8631	1.437	0.841	0.4572	0.996	1.970	0.00159	1.15	3.77	4.02	-3.62
		0.060	0.750	0.09375	0.663	2.254	1.34	0.6428	1.423	0.626	0.3704	0.972	1.911	0.000795	0.829	3.86	4.08	-3.70
		0.048	0.618	0.09375	0.520	1.769	1.03	0.4831	1.409	0.476	0.3032	0.956	1.861	0.000400	0.640	3.91	4.10	-3.73
3.0	4.50	0.135	1.670	0.1875	1.736	5.904	2.47	1.5155	1.192	8.28	2.1885	2.184	1.371	0.0105	5.66	3.73	3.55	-2.53
		0.105	1.340	0.1875	1.297	4.408	1.80	1.0526	1.179	5.69	1.6327	2.095	1.287	0.00476	4.20	3.69	3.52	-2.57
3.0	3.00	0.105	1.340	0.1875	1.139	3.873	1.53	0.9921	1.159	2.62	0.9563	1.515	1.457	0.00419	1.89	3.12	3.20	-2.57
		0.075	0.915	0.09375	0.773	2.628	1.01	0.6033	1.140	1.54	0.6594	1.413	1.334	0.00145	1.14	3.15	3.22	-2.66
3.0	1.50	0.075	0.915	0.09375	0.660	2.245	0.784	0.5423	1.089	0.415	0.2611	0.793	1.555	0.00124	0.371	2.64	2.87	-2.53
		0.060	0.750	0.09375	0.513	1.744	0.599	0.3995	1.081	0.296	0.2054	0.759	1.500	0.000615	0.235	2.79	2.98	-2.68
		0.048	0.618	0.09375	0.400	1.361	0.459	0.2970	1.071	0.218	0.1650	0.738	1.453	0.000308	0.164	2.88	3.05	-2.76
2.0	4.00	0.105	1.340	0.1875	1.034	3.516	0.670	0.5948	0.805	3.72	1.1484	1.896	0.873	0.00380	1.13	3.08	2.64	-1.65
		0.075	0.915	0.09375	0.698	2.373	0.432	0.3531	0.787	2.19	0.7719	1.772	0.776	0.00131	0.799	2.85	2.54	-1.64
2.0	2.00	0.075	0.915	0.09375	0.548	1.863	0.328	0.3210	0.774	0.563	0.3062	1.014	0.978	0.00103	0.180	2.06	2.12	-1.70
		0.060	0.750	0.09375	0.423	1.438	0.249	0.2333	0.768	0.400	0.2368	0.973	0.931	0.000507	0.127	2.10	2.15	-1.75
2.0	1.00	0.060	0.750	0.09375	0.363	1.234	0.193	0.2093	0.728	0.116	0.0975	0.565	1.080	0.000435	0.0542	1.62	1.80	-1.55
		0.048	0.618	0.09375	0.280	0.953	0.147	0.1534	0.725	0.0795	0.0743	0.532	1.039	0.000215	0.0321	1.76	1.91	-1.69
1.5	3.00	0.105	1.340	0.1875	0.824	2.802	0.303	0.3892	0.606	1.96	0.7149	1.540	0.722	0.00303	0.312	2.47	2.04	-1.19
		0.075	0.915	0.09375	0.548	1.863	0.198	0.2293	0.601	1.06	0.4537	1.392	0.636	0.00103	0.187	2.24	1.96	-1.24
1.5	1.50	0.075	0.915	0.09375	0.435	1.480	0.147	0.2078	0.582	0.301	0.1893	0.831	0.791	0.000816	0.0606	1.49	1.55	-1.17
		0.060	0.750	0.09375	0.333	1.132	0.112	0.1500	0.581	0.202	0.1406	0.780	0.750	0.000399	0.0376	1.54	1.58	-1.25
1.5	0.75	0.060	0.750	0.09375	0.288	0.979	0.0855	0.1341	0.545	0.0674	0.0633	0.484	0.863	0.000345	0.0210	1.01	1.20	-0.953
		0.048	0.618	0.09375	0.220	0.749	0.0656	0.0975	0.546	0.0430	0.0455	0.442	0.827	0.000169	0.0122	1.15	1.31	-1.10

TABLE 10 TABLE 10

**CHANNEL
WITH
STIFFENED FLANGES**
Effective Area
($F_y = 50$ksi)



See notes on page V-25

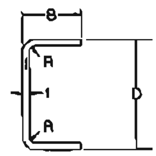
Size		t	d	R	Effective Section Properties		
D	B				I_x	S_x	Area
In.	In.	In.	In.	In.	$In.^4$	$In.^3$	$In.^2$
12.0	3.50	0.135	1.01	0.1875	55.386	9.134	2.672
		0.105	0.90	0.1875	40.903	6.546	2.008
10.0	3.50	0.135	1.01	0.1875	35.951	7.109	2.402
		0.105	0.90	0.1875	26.513	5.071	1.798
		0.075	0.72	0.09375	17.462	3.225	1.207
9.0	3.25	0.135	1.00	0.1875	27.209	6.056	2.235
		0.105	0.84	0.1875	20.029	4.307	1.655
		0.075	0.70	0.09375	13.557	2.809	1.141
		0.060	0.61	0.09375	9.635	1.885	0.817
8.0	3.00	0.135	0.93	0.1875	19.542	4.920	2.028
		0.105	0.81	0.1875	14.714	3.607	1.517
		0.075	0.70	0.09375	9.969	2.343	1.045
		0.060	0.60	0.09375	7.349	1.681	0.785
8.0	1.625	0.105	0.82	0.1875	9.958	2.395	1.210
		0.075	0.82	0.09375	7.996	1.999	0.927
		0.060	0.60	0.09375	6.361	1.620	0.736
		0.048	0.50	0.09375	4.419	1.057	0.521
7.0	2.75	0.135	0.88	0.1875	13.572	3.957	1.847
		0.105	0.88	0.1875	10.548	3.013	1.409
		0.075	0.70	0.09375	7.036	1.906	0.947
		0.060	0.60	0.09375	5.370	1.423	0.734
6.0	2.50	0.135	0.82	0.1875	8.685	2.928	1.600
		0.105	0.82	0.1875	6.957	2.354	1.268
		0.075	0.82	0.09375	4.944	1.607	0.882
		0.060	0.60	0.09375	3.607	1.126	0.657
6.0	1.625	0.105	0.82	0.1875	5.077	1.655	1.026
		0.075	0.82	0.09375	7.369	2.018	1.338
		0.060	0.60	0.09375	3.167	1.077	0.616
		0.048	0.50	0.09375	2.334	0.757	0.459
5.0	2.00	0.135	0.70	0.1875	4.006	1.465	1.152
		0.105	0.70	0.1875	3.935	0.661	1.126
		0.075	0.60	0.09375	2.760	1.093	0.716
		0.060	0.50	0.09375	2.053	0.780	0.542
		0.048	0.50	0.09375	1.633	0.614	0.432
4.0	2.00	0.135	0.70	0.1875	2.555	1.212	1.064
		0.105	0.70	0.1875	2.162	1.133	0.924
		0.075	0.60	0.09375	1.644	0.813	0.641
		0.060	0.50	0.09375	1.222	0.578	0.482
		0.048	0.50	0.09375	0.972	0.454	0.384
4.0	1.625	0.075	0.60	0.09375	1.508	0.796	0.679
		0.060	0.50	0.09375	1.143	0.566	0.463
		0.048	0.50	0.09375	0.895	0.433	0.363
		0.036	0.50	0.09375	0.663	0.316	0.270
3.625	1.625	0.075	0.60	0.09375	1.188	0.687	0.651
		0.060	0.50	0.09375	0.908	0.497	0.440
		0.048	0.50	0.09375	0.712	0.380	0.345
		0.036	0.50	0.09375	0.527	0.276	0.257
3.5	2.00	0.135	0.70	0.1875	1.858	1.005	0.995
		0.105	0.70	0.1875	1.510	0.857	0.759
		0.075	0.60	0.09375	1.208	0.683	0.604
		0.060	0.50	0.09375	0.899	0.484	0.462
		0.048	0.50	0.09375	0.715	0.381	0.360
3.0	1.75	0.105	0.70	0.1875	1.016	0.677	0.742
		0.075	0.53	0.09375	0.767	0.511	0.527
		0.060	0.53	0.09375	0.617	0.405	0.417
		0.048	0.41	0.09375	0.442	0.272	0.305
2.5	1.625	0.075	0.60	0.09375	0.481	0.384	0.482
		0.060	0.50	0.09375	0.385	0.305	0.373
		0.048	0.50	0.09375	0.303	0.233	0.291
		0.036	0.50	0.09375	0.224	0.169	0.216

TABLE 11 TABLE 11

CHANNEL WITH UNSTIFFENED FLANGES

Effective Area

($F_y = 50$ksi)



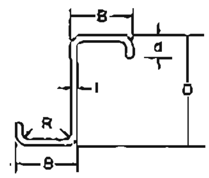
Size		t	d	R	Effective Section Properties		
D	B				I_x	S_x	Area
In.	In.	In.	In.	In.	$In.^4$	$In.^3$	$In.^2$
8.0	2.0	0.135	0	0.1875	12.690	3.123	1.529
		0.105	0	0.1875	9.580	2.304	1.170
		0.075	0	0.09375	6.519	1.513	0.816
		0.060	0	0.09375	4.783	1.080	0.613
7.0	1.5	0.135	0	0.1875	7.839	2.240	1.284
		0.105	0	0.1875	6.119	1.734	0.998
		0.075	0	0.09375	4.219	1.155	0.700
		0.060	0	0.09375	3.278	0.882	0.552
6.0	1.5	0.135	0	0.1875	5.333	1.778	1.149
		0.105	0	0.1875	4.168	1.377	0.893
		0.075	0	0.09375	2.868	0.917	0.625
		0.060	0	0.09375	2.223	0.693	0.492
		0.048	0	0.09375	1.675	0.519	0.380
5.0	1.25	0.105	0	0.1875	2.398	0.959	0.744
		0.075	0	0.09375	1.700	0.661	0.527
		0.060	0	0.09375	1.320	0.508	0.415
		0.048	0	0.09375	1.024	0.382	0.327
4.0	1.125	0.105	0	0.1875	1.186	0.607	0.563
		0.075	0	0.09375	0.938	0.460	0.440
		0.060	0	0.09375	0.730	0.350	0.346
		0.048	0	0.09375	0.565	0.265	0.273
3.0	1.125	0.105	0	0.1875	0.636	0.424	0.508
		0.075	0	0.09375	0.467	0.304	0.365
		0.060	0	0.09375	0.363	0.230	0.286
		0.048	0	0.09375	0.280	0.173	0.225
2.0	1.125		0	0.1875	0.240	0.240	0.403
			0	0.09375	0.179	0.174	0.290
			0	0.09375	0.129	0.120	0.201
			0	0.09375	0.107	0.097	0.177

TABLE 12 TABLE 12

Z-SECTION WITH STIFFENED FLANGES

Effective Area

($F_y = 50$ksi)



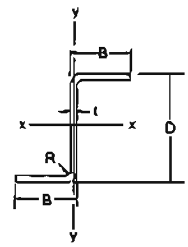
Size		t	d	R	Effective Section Properties		
D	B				I_x	S_x	Area
In.	In.	In.	In.	In.	In.^{4}	In.^{3}	In.^{2}
12.0	3.5	0.135	1.01	0.1875	55.386	9.134	2.672
		0.105	0.90	0.1875	40.903	6.546	2.008
10.0	3.5	0.135	1.01	0.1875	35.951	7.109	2.402
		0.105	0.90	0.1875	26.513	5.071	1.798
		0.075	0.72	0.09375	17.462	3.225	1.207
9.0	3.25	0.135	1.00	0.1875	27.209	6.056	2.235
		0.105	0.84	0.1875	20.029	4.307	1.655
		0.075	0.70	0.09375	13.557	2.809	1.141
		0.060	0.61	0.09375	9.6350	1.885	0.817
8.0	3.0	0.135	0.93	0.1875	19.542	4.920	2.028
		0.105	0.81	0.1875	14.714	3.607	1.517
		0.075	0.70	0.09375	9.969	2.343	1.045
		0.060	0.60	0.09375	7.349	1.681	0.785
7.0	2.75	0.135	0.88	0.1875	13.572	3.957	1.847
		0.105	0.88	0.1875	10.548	3.013	1.409
		0.075	0.70	0.09375	7.036	1.906	0.947
		0.060	0.60	0.09375	5.370	1.423	0.734
6.0	2.5	0.135	0.82	0.1875	8.685	2.928	1.600
		0.105	0.82	0.1875	6.957	2.354	1.268
		0.075	0.82	0.09375	4.944	1.607	0.882
		0.060	0.60	0.09375	3.607	1.126	0.657
5.0	2.0	0.135	0.70	0.1875	4.006	1.465	1.152
		0.105	0.70	0.1875	3.935	1.661	1.125
		0.075	0.60	0.09375	2.760	1.093	0.716
		0.060	0.50	0.09375	2.053	0.780	0.542
		0.048	0.50	0.09375	1.633	0.614	0.432
4.0	2.0	0.135	0.70	0.1875	2.565	1.212	1.064
		0.105	0.70	0.1875	2.162	1.133	0.924
		0.075	0.60	0.09375	1.644	0.813	0.641
		0.060	0.50	0.09375	1.222	0.578	0.482
		0.048	0.50	0.09375	0.972	0.454	0.384
3.5	2.0	0.135	0.70	0.1875	1.858	1.005	0.995
		0.105	0.70	0.1875	1.510	0.857	0.759
		0.075	0.60	0.09375	1.208	0.683	0.604
		0.060	0.50	0.09375	0.899	0.484	0.452
		0.048	0.50	0.09375	0.715	0.381	0.360
3.0	1.75	0.105	0.70	0.1875	1.011	0.672	0.040
		0.075	0.53	0.09375	0.767	0.511	0.527
		0.060	0.53	0.09375	0.617	0.405	0.417
		0.048	0.41	0.09375	0.442	0.272	0.305

TABLE 13 TABLE 13

Z-SECTION WITH UNSTIFFENED FLANGES

Effective Area

($F_y = 50$ksi)



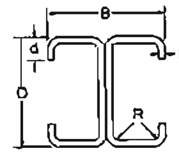
Size D	Size B	t	d	R	Effective Section Properties I_x	Effective Section Properties S_x	Effective Section Properties Area
In.	In.	In.	In.	In.	$In.^4$	$In.^3$	$In.^2$
8.0	2.0	0.135	0	0.1875	12.689	3.123	1.530
		0.105	0	0.1875	9.580	2.304	1.170
		0.075	0	0.09375	6.5192	1.513	0.816
		0.060	0	0.09375	4.783	1.080	0.613
7.0	1.5	0.135	0	0.1875	7.839	2.240	1.284
		0.105	0	0.1875	6.119	1.734	0.998
		0.075	0	0.09375	4.219	1.155	0.700
		0.060	0	0.09375	3.278	0.882	0.552
6.0	1.5	0.135	0	0.1875	5.333	1.778	1.149
		0.105	0	0.1875	4.168	1.377	0.893
		0.075	0	0.09375	2.868	0.912	0.625
		0.060	0	0.09375	2.223	0.693	0.492
6.0	1.5	0.060	0	0.09375	2.223	0.693	0.492
		0.048	0	0.09375	1.675	0.519	0.380
5.0	1.25	0.105	0	0.1875	2.398	0.959	0.744
		0.075	0	0.09375	1.700	0.661	0.527
		0.060	0	0.09375	1.320	0.503	0.415
		0.048	0	0.09375	1.024	0.382	0.327
4.0	1.5	0.060	0	0.09375	0.819	0.376	0.372
4.0	1.125	0.105	0	0.1875	1.186	0.607	0.563
		0.075	0	0.09375	0.938	0.460	0.440
		0.060	0	0.09375	0.730	0.350	0.346
		0.048	0	0.09375	0.565	0.265	0.273
3.0	1.5	0.060	0	0.09375	0.412	0.248	0.312
3.0	1.125	0.105	0	0.1875	0.636	0.424	0.508
		0.075	0	0.09375	0.467	0.304	0.365
		0.060	0	0.09375	0.363	0.230	0.286
		0.048	0	0.09375	0.280	0.173	0.225
2.0	1.125	0.105	0	0.1875	0.240	0.240	0.403
		0.075	0	0.09375	0.179	0.174	0.290
		0.060	0	0.09375	0.129	0.120	0.201
		0.048	0	0.09375	0.007	0.097	0.177
1.5	1.5	0.048	0	0.09375	0.059	0.064	0.153
		0.036	0	0.09375	0.044	0.047	0.125

TABLE 14 TABLE 14

TWO CHANNELS WITH FLANGES BACK TO BACK

Effective Area

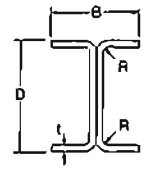
($F_y = 50$ksi)



Size		t	d	R	Effective Section Properties		
D	B				I_x	S_x	Area
In.	In.	In.	In.	In.	$In.^4$	$In.^3$	$In.^2$
12.0	7.0	0.135	1.01	0.18750	110.77	18.269	5.345
		0.105	0.90	0.18750	81.806	13.092	4.015
10.0	7.0	0.135	1.01	0.18750	71.901	14.217	4.804
		0.105	0.90	0.18750	53.025	10.142	3.595
		0.075	0.72	0.09375	34.876	6.435	2.409
9.0	6.5	0.135	1.00	0.18750	54.418	12.111	4.470
		0.105	0.84	0.18750	40.058	8.614	3.310
		0.075	0.70	0.09375	27.114	5.617	2.283
		0.060	0.61	0.09375	19.249	3.762	1.630
8.0	6.0	0.135	0.93	0.18750	38.740	9.685	4.018
		0.105	0.81	0.18750	29.427	7.215	3.034
		0.075	0.70	0.09375	19.939	4.686	2.089
		0.060	0.60	0.09375	14.670	3.351	1.566
7.0	5.5	0.135	0.88	0.18750	27.144	7.915	3.694
		0.105	0.88	0.18750	21.095	6.025	2.819
		0.075	0.70	0.09375	14.071	3.811	1.893
		0.060	0.60	0.09375	10.740	2.846	1.469
6.0	5.0	0.135	0.82	0.18750	17.370	5.856	3.201
		0.105	0.82	0.18750	13.913	4.708	2.535
		0.075	0.82	0.09375	9.888	3.213	1.763
		0.060	0.60	0.09375	7.215	2.252	1.314
5.0	4.0	0.135	0.70	0.18750	8.012	2.930	2.303
		0.105	0.70	0.18750	7.870	3.323	2.252
		0.075	0.60	0.09375	5.520	2.185	1.432
		0.060	0.50	0.09375	4.107	1.561	1.084
		0.048	0.50	0.09375	3.266	1.228	0.863
4.0	4.0	0.135	0.70	0.18750	5.111	2.424	2.128
		0.105	0.70	0.18750	4.316	2.257	1.838
		0.075	0.60	0.09375	3.287	1.626	1.282
		0.060	0.50	0.09375	2.445	1.156	0.964
		0.048	0.50	0.09375	1.944	0.909	0.767
3.5	4.0	0.135	0.70	0.18750	3.717	2.01	1.989
		0.105	0.70	0.18750	3.028	1.723	1.531
		0.075	0.60	0.09375	2.416	1.366	1.207
		0.060	0.50	0.09375	1.798	0.969	0.904
		0.048	0.50	0.09375	1.430	0.761	0.719
3.0	3.5	0.105	0.70	0.18750	2.022	1.343	1.479
		0.075	0.53	0.09375	1.534	1.022	1.055
		0.060	0.53	0.09375	1.234	0.810	0.834
		0.048	0.41	0.09375	0.884	0.544	0.610

TABLE 15 TABLE 15

TWO CHANNELS
WITH
UNSTIFFENED FLANGES
BACK TO BACK
Effective Area
($F_y = 50$ksi)



Size		t	d	R	Effective Section Properties		
D	B				I_x	S_x	Area
In.	In.	In.	In.	In.	In.^{4}	In.^{3}	In.^{2}
8.0	4.0	0.135	0	0.1875	25.379	6.246	3.059
		0.105	0	0.1875	19.160	4.609	2.339
		0.075	0	0.09375	13.037	3.025	1.633
		0.060	0	0.09375	9.543	2.151	1.222
7.0	3.0	0.135	0	0.1875	15.678	4.479	2.568
		0.105	0	0.1875	12.238	3.468	1.996
		0.075	0	0.09375	8.438	2.311	1.399
		0.060	0	0.09375	6.556	1.764	1.104
6.0	3.0	0.135	0	0.1875	10.667	3.556	2.298
		0.105	0	0.1875	8.335	2.754	1.786
		0.075	0	0.09375	5.735	1.823	1.249
		0.060	0	0.09375	4.445	1.385	0.984
		0.048	0	0.09375	3.349	1.039	0.761
5.0	2.5	0.105	0	0.1875	4.79	1.919	1.488
		0.075	0	0.09375	3.400	1.321	1.054
		0.060	0	0.09375	2.641	1.006	0.830
		0.048	0	0.09375	2.047	0.765	0.654
4.0	2.25	0.105	0	0.1875	2.363	1.206	1.119
		0.075	0	0.09375	1.876	0.920	0.880
		0.060	0	0.09375	1.460	0.700	0.693
		0.048	0	0.09375	1.131	0.531	0.545
3.0	2.25	0.105	0	0.1875	1.272	0.848	1.015
		0.075	0	0.09375	0.935	0.609	0.730
		0.060	0	0.09375	0.725	0.460	0.573
		0.048	0	0.09375	0.560	0.346	0.449
2.0	2.25	0.105	0	0.1875	0.481	0.481	0.805
		0.075	0	0.09375	0.358	0.348	0.580
		0.060	0	0.09375	0.258	0.240	0.402
		0.048	0	0.09375	0.213	0.194	0.353

COMPUTER AIDS

COMPUTER AIDS

FOR USE WITH THE
AUGUST 19, 1986, EDITION OF THE

SPECIFICATION FOR THE DESIGN OF COLD-FORMED STEEL STRUCTURAL MEMBERS

Cold-Formed Steel Design Manual - Part VI

AMERICAN IRON AND STEEL INSTITUTE
1000 16th STREET, NW
WASHINGTON, DC 20036

1st Printing—March 1987

Produced by W. P. Reyman Associates, Inc. New York

PREFACE

This document, *Part VI of the Cold-Formed Steel Design Manual*, contains many useful design aids; flow charts for many sections of the *Specification* are included. These charts will be very helpful in getting all users familiar with the new Specification. The charts serve to direct the user to the appropriate sections. They will also be useful to programmers.

These *Computer Aids* should be used in conjunction with the other parts of the *Design Manual*, which include *Commentary* (Part II), *Supplementary Information* (Part III), *Illustrative Examples* (Part IV), *Design Aids* (Part V), and *Test Procedures* (Part VII).

American Iron and Steel Institute
August 1986

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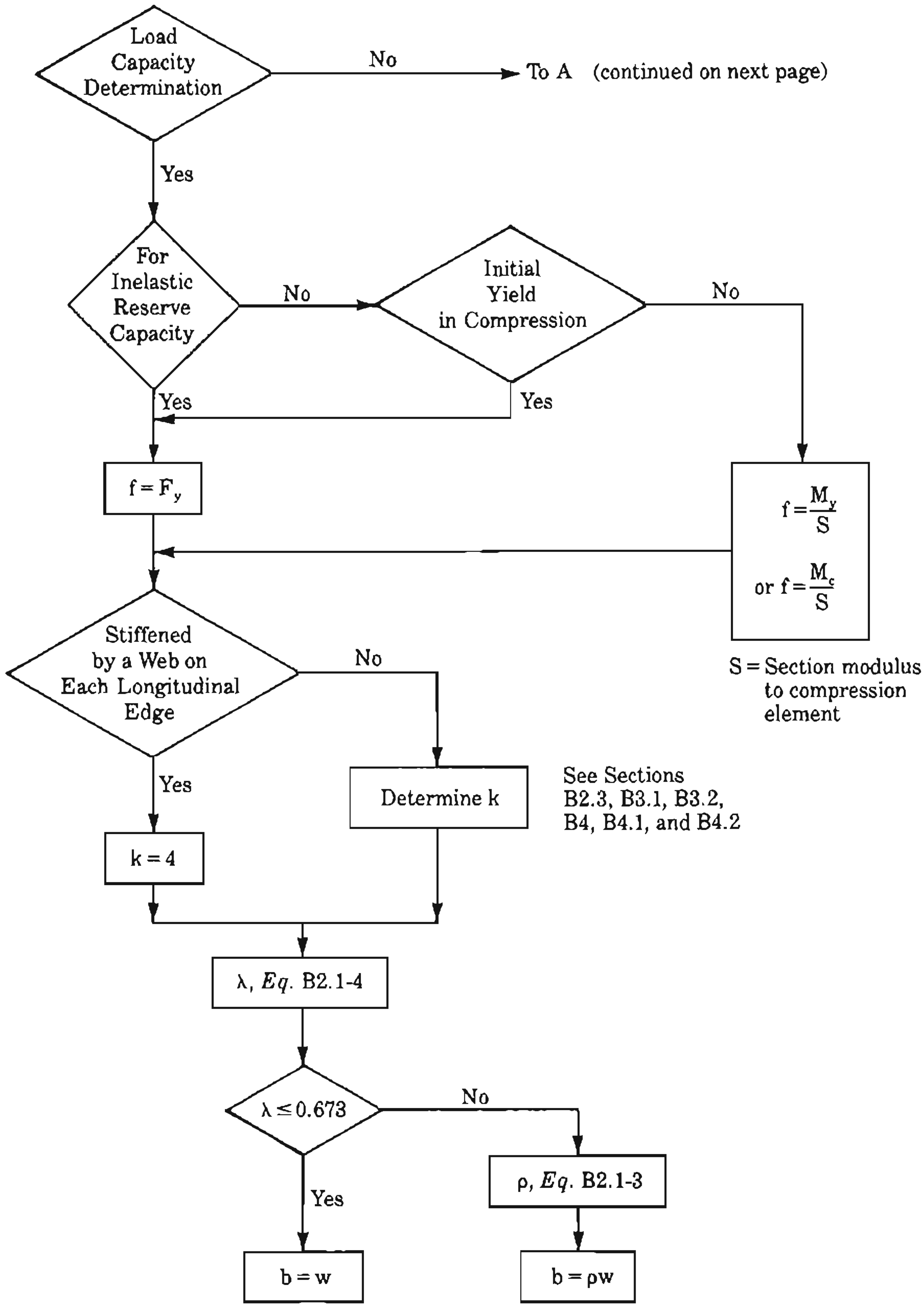
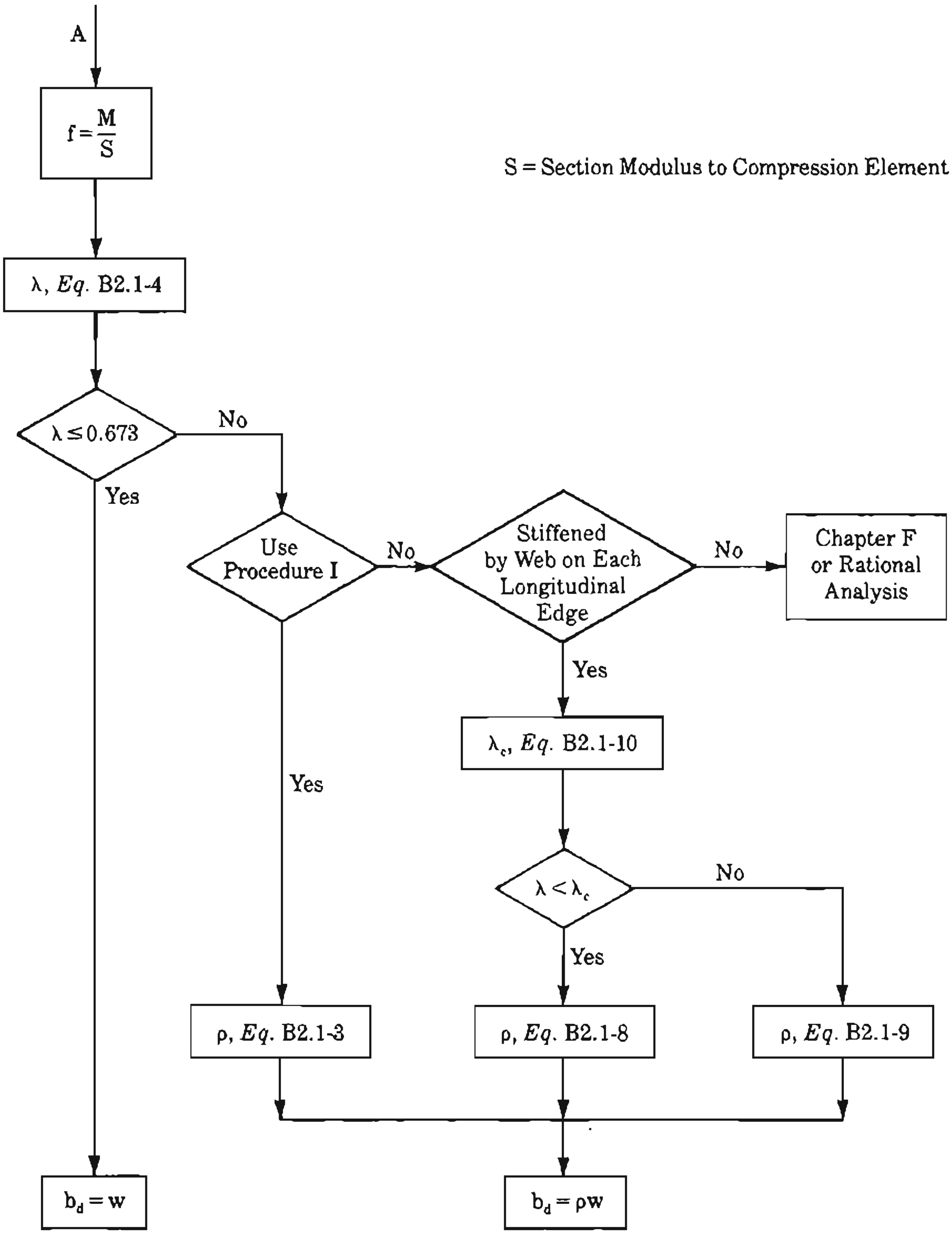
B2.1 Uniformly Compressed Stiffened Elements

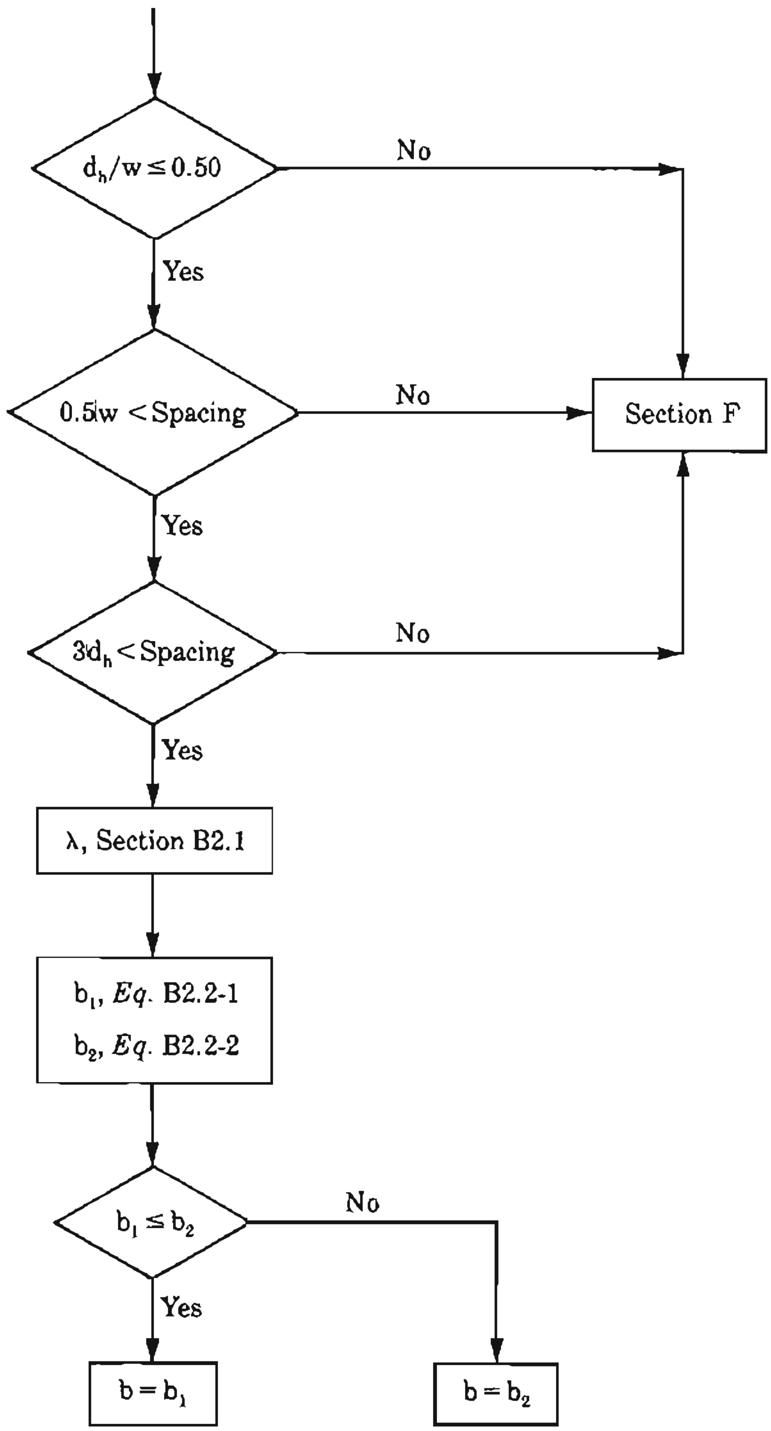


B2.1 Uniformly Compressed Stiffened Elements (continued)

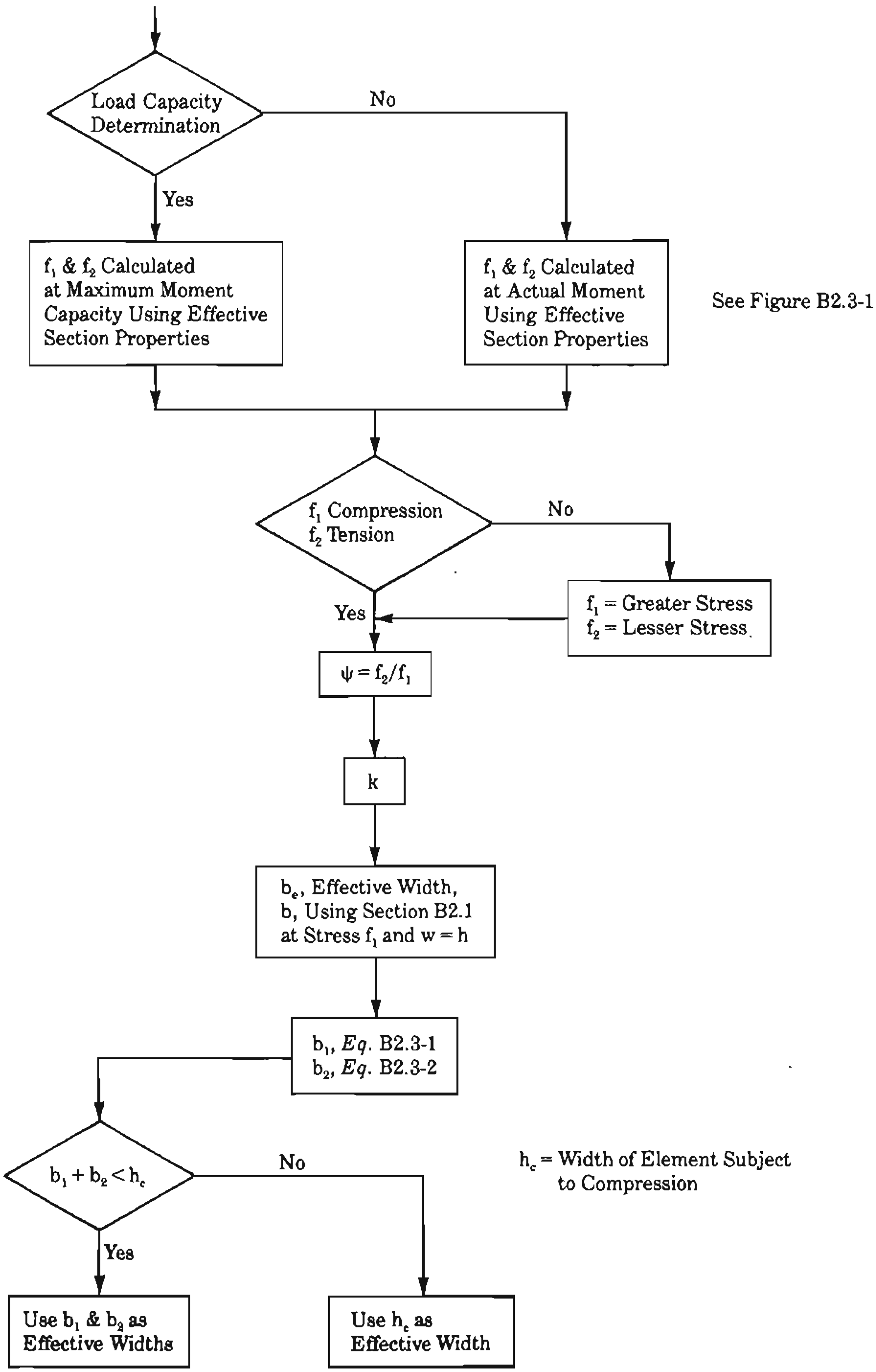


S = Section Modulus to Compression Element

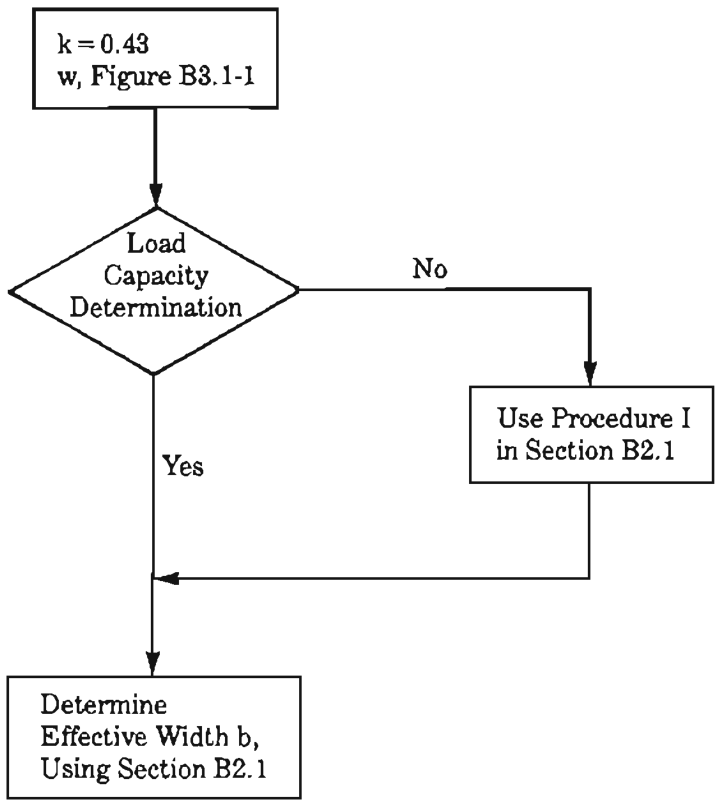
B2.2 Uniformly Compressed Stiffened Elements with Circular Holes



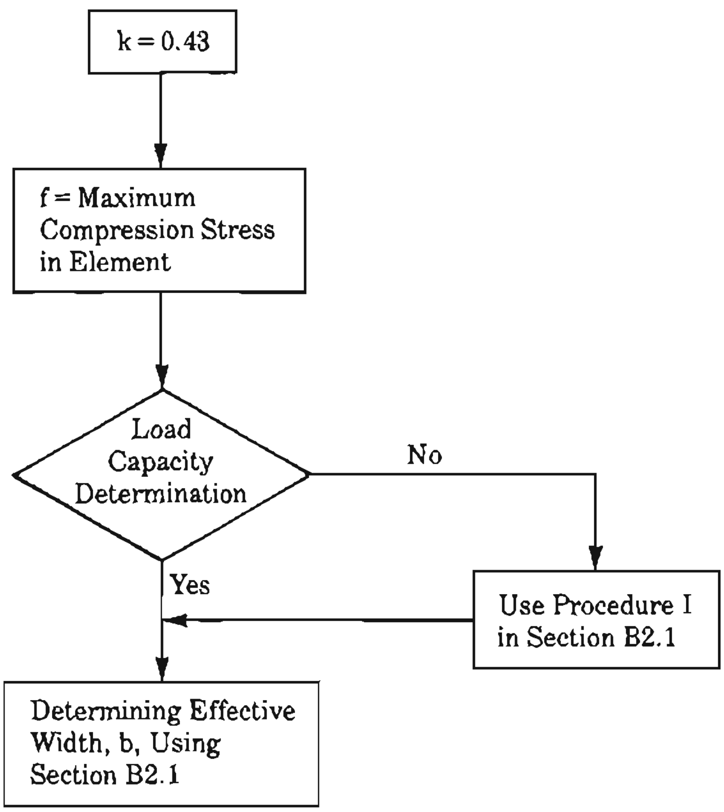
B2.3 Effective Width of Webs and Stiffened Elements with Stress Gradient



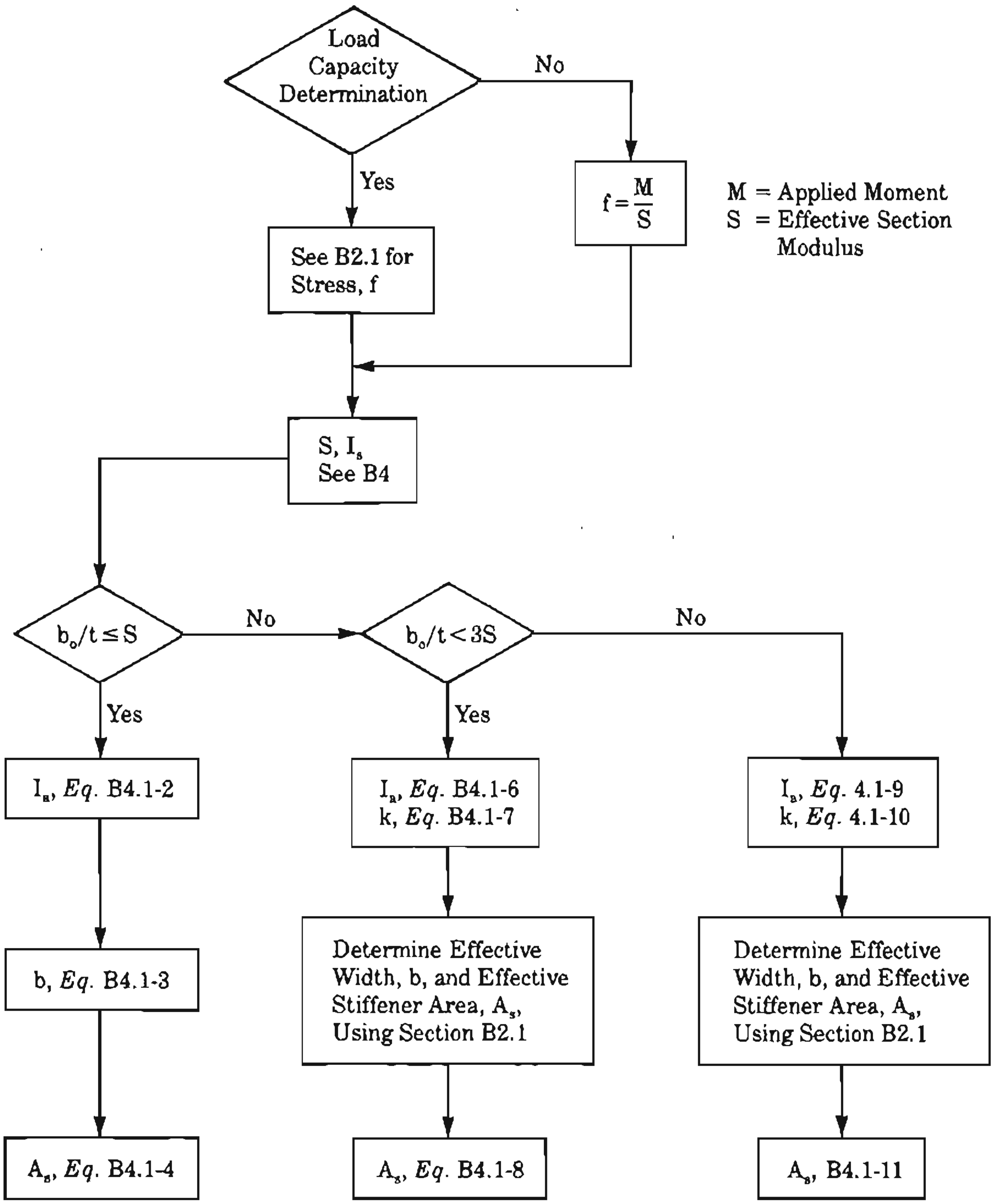
B3.1 Uniformly Compressed Unstiffened Elements



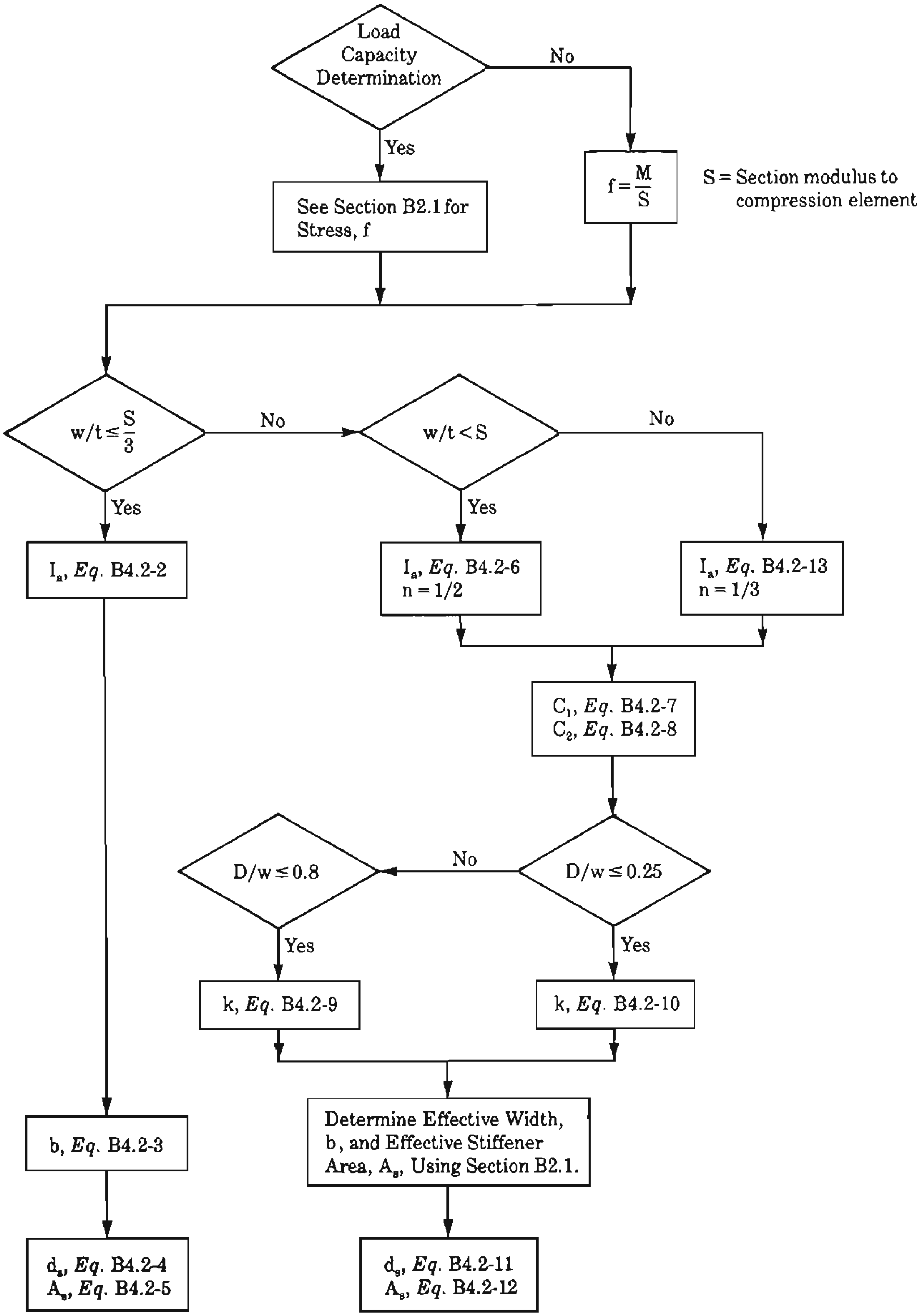
B3.2 Unstiffened Elements and Edge Stiffeners with Stress Gradient



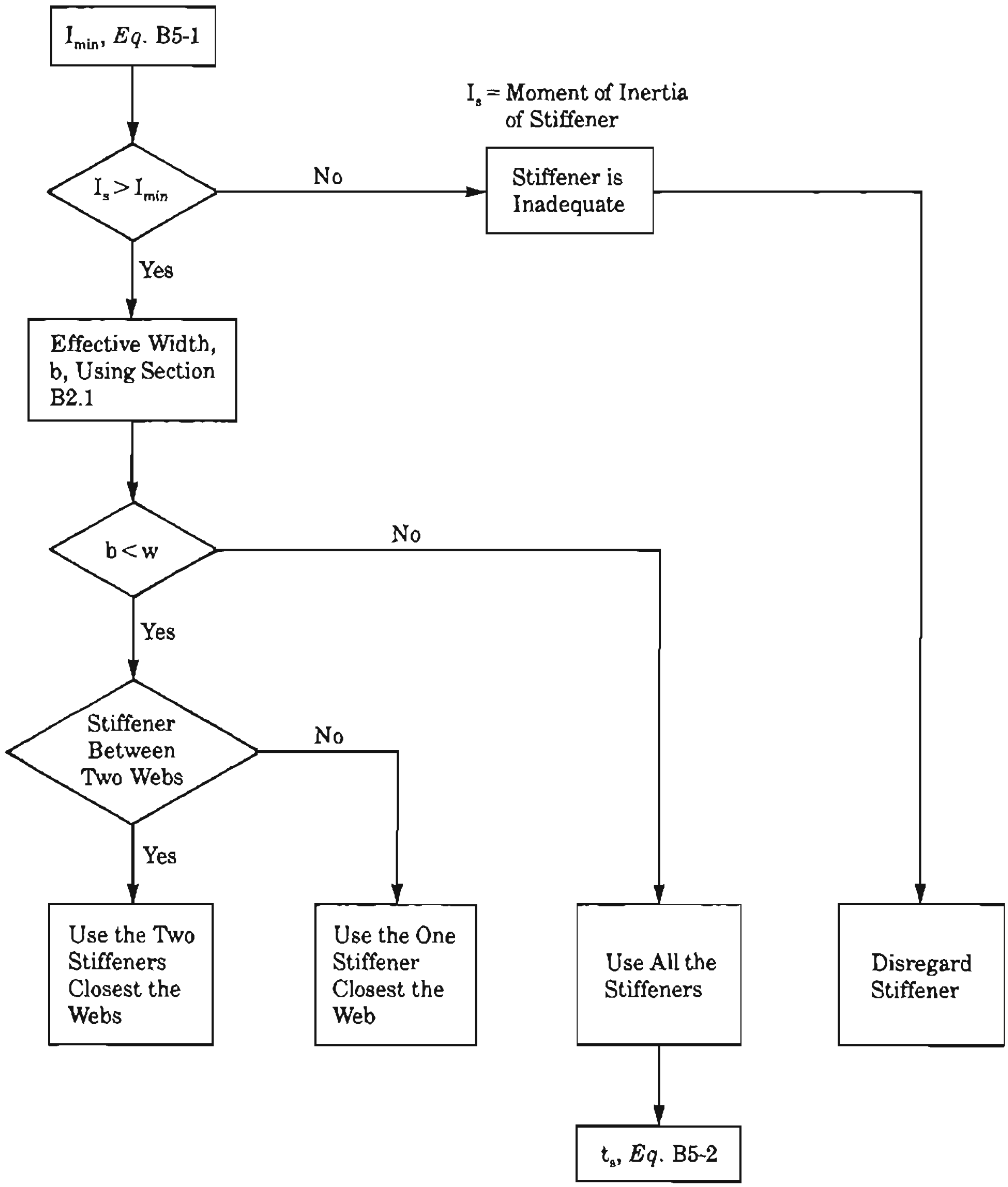
B4.1 Uniformly Compressed Elements with an Intermediate Stiffener



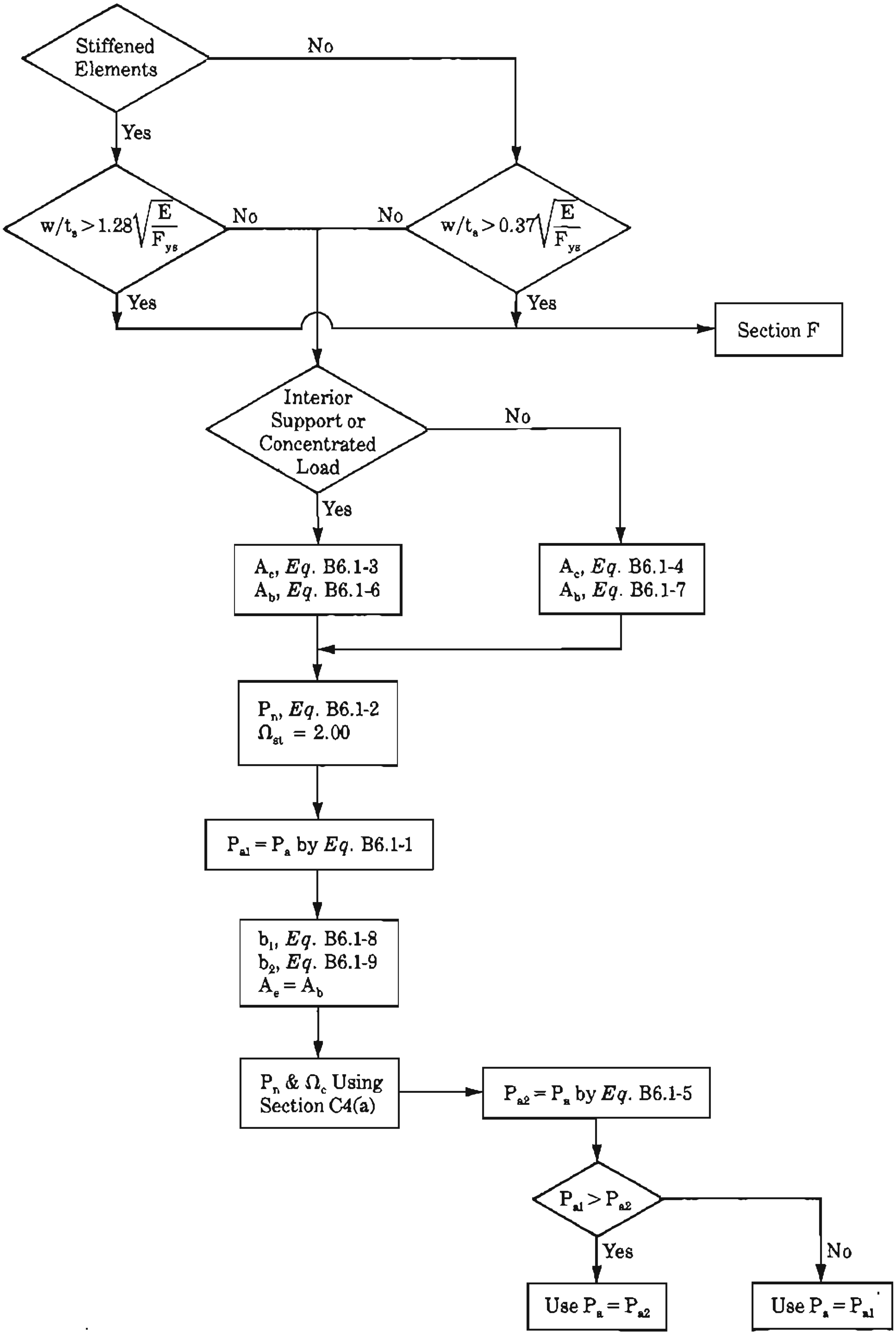
B4.2 Uniformly Compressed Elements with an Edge Stiffener



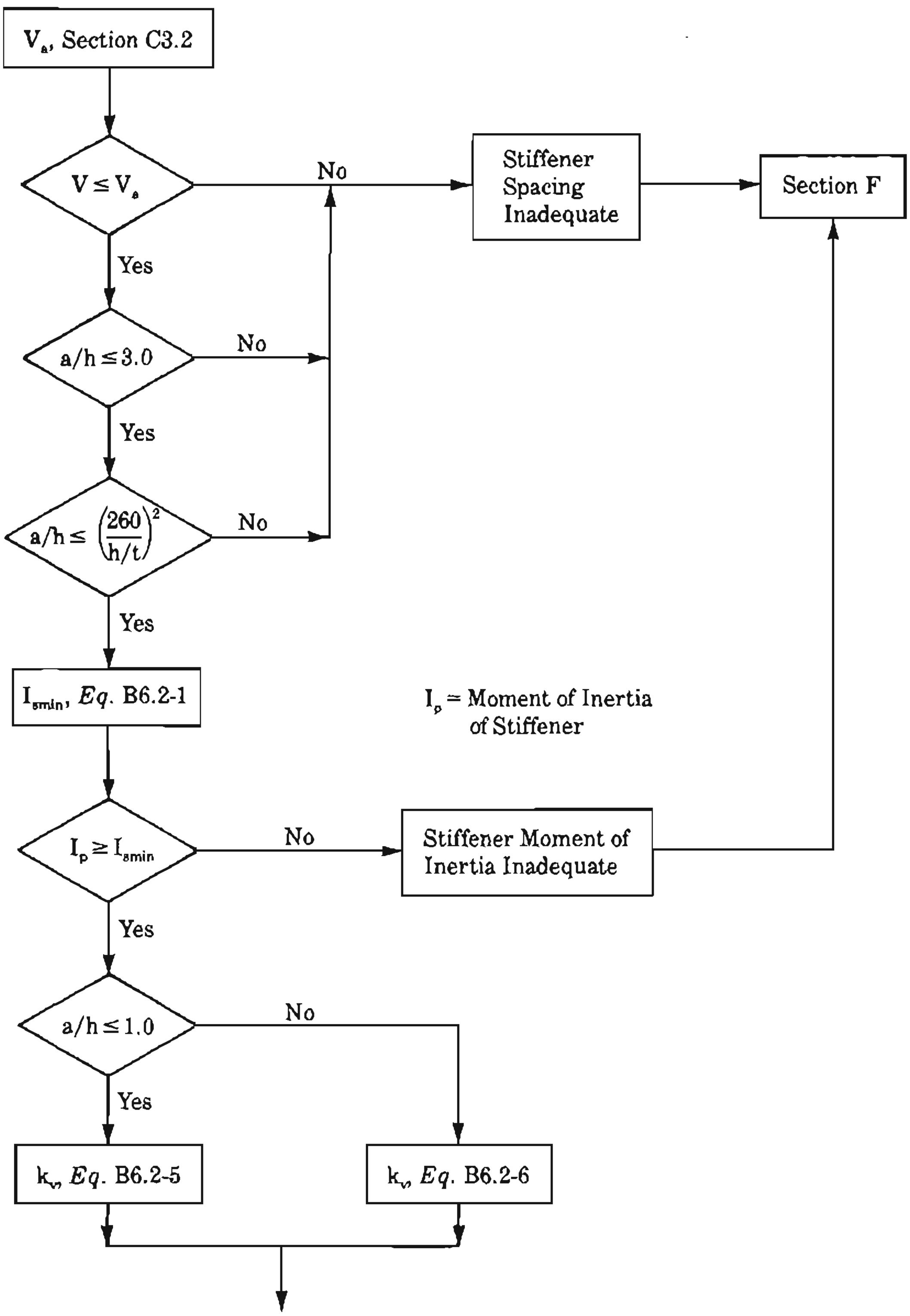
B5 Effective Widths of Edge Stiffened Elements with Intermediate Stiffeners or Stiffened Elements with More than One Intermediate Stiffener



B6.1 Transverse Stiffeners



B6.2 Shear Stiffeners



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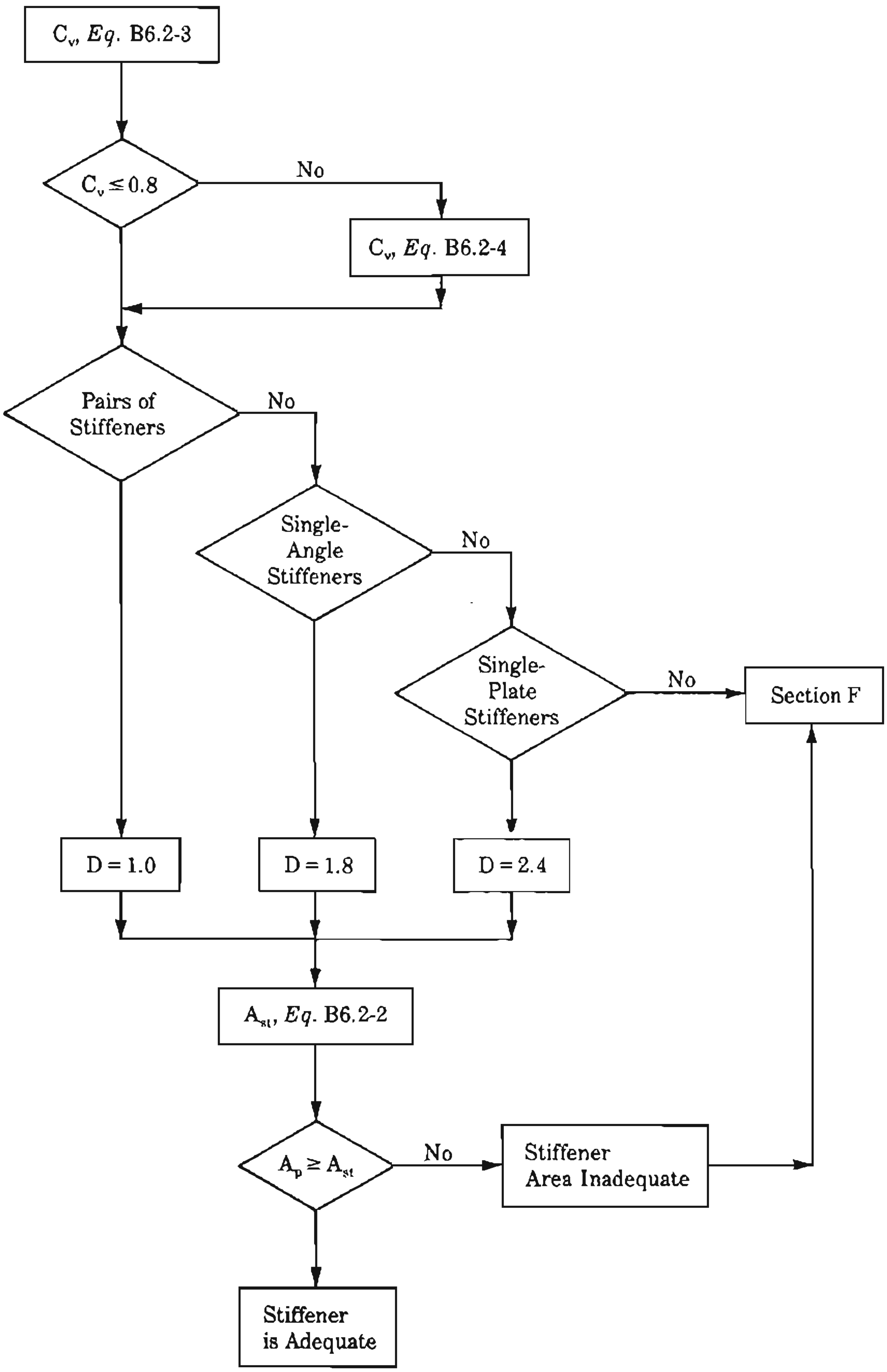
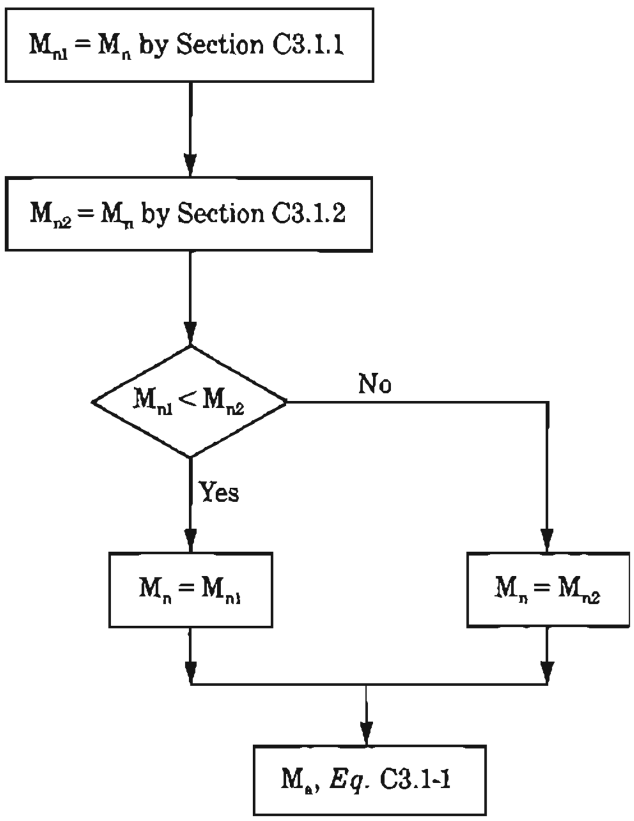
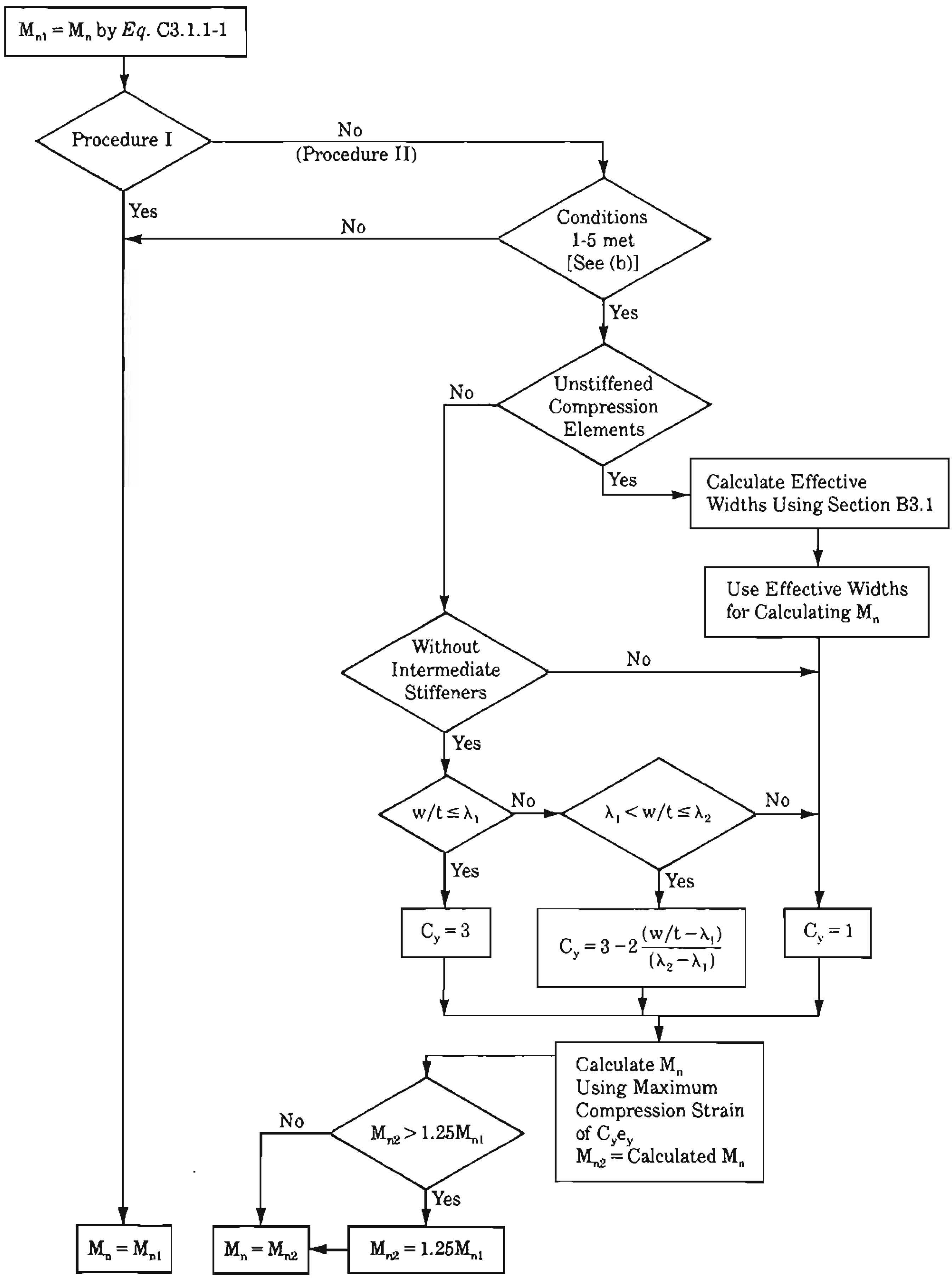
B6.2 Shear Stiffeners (continued)



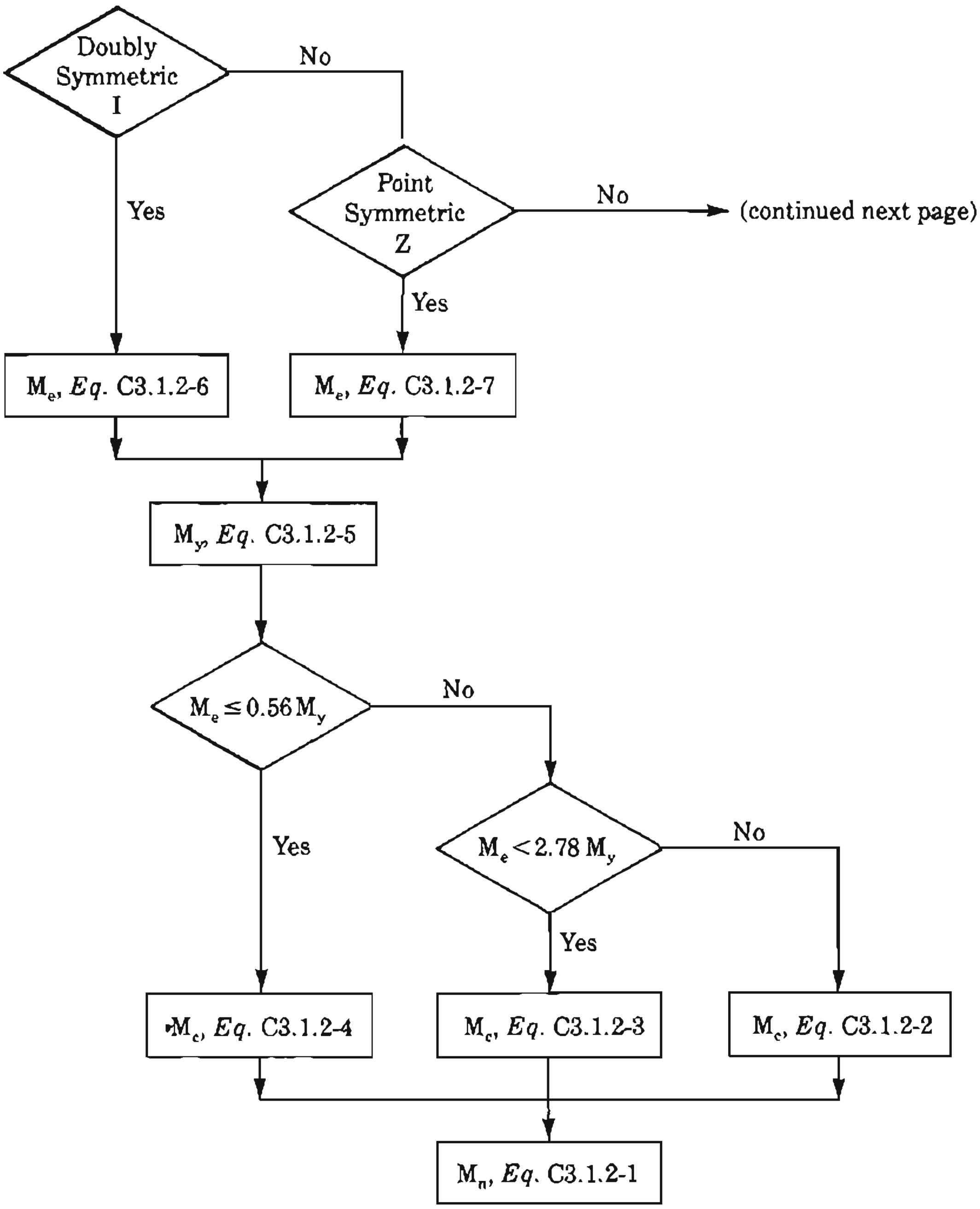
C3.1 Strength for Bending Only



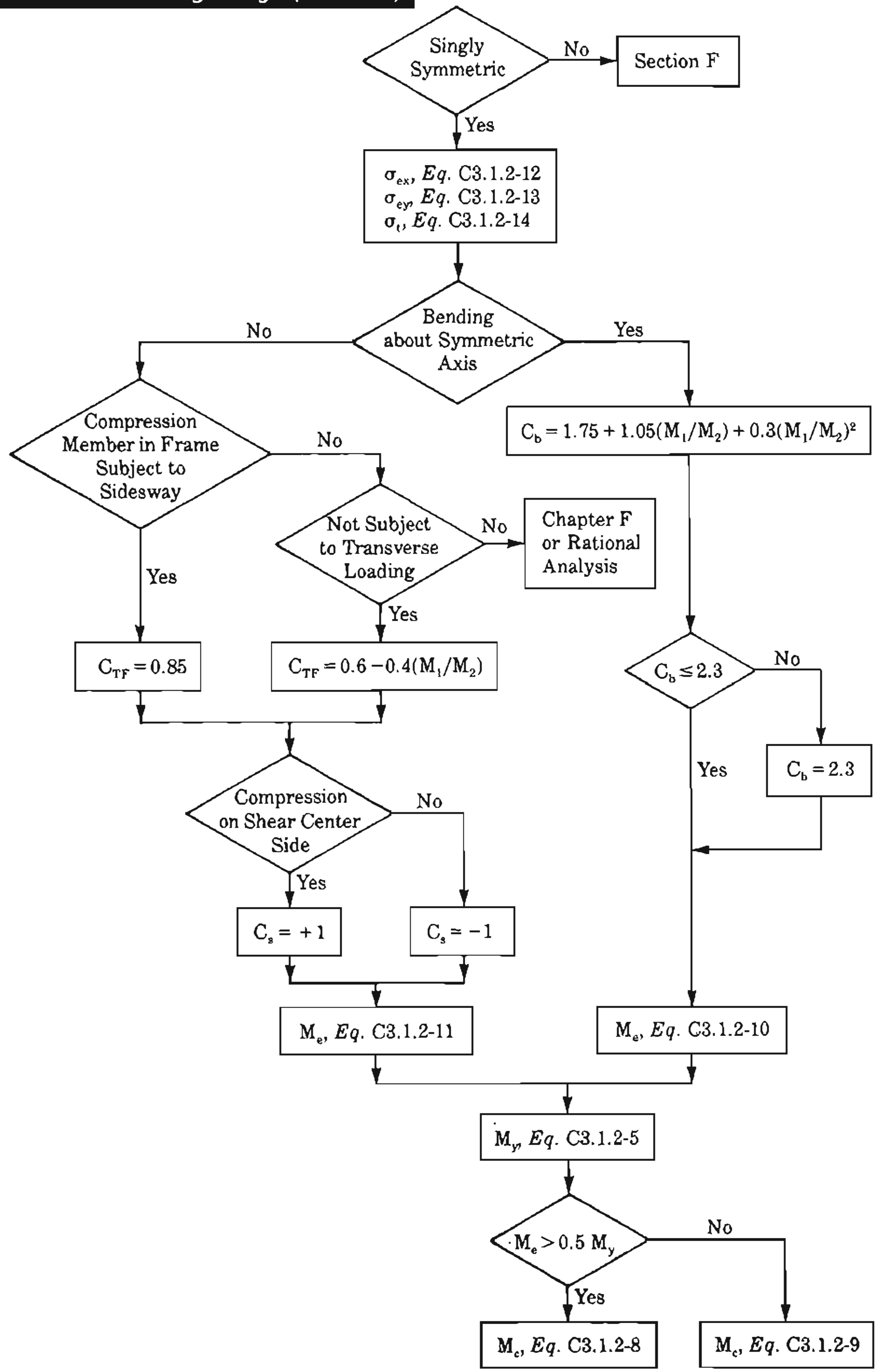
C3.1.1 Nominal Section Strength



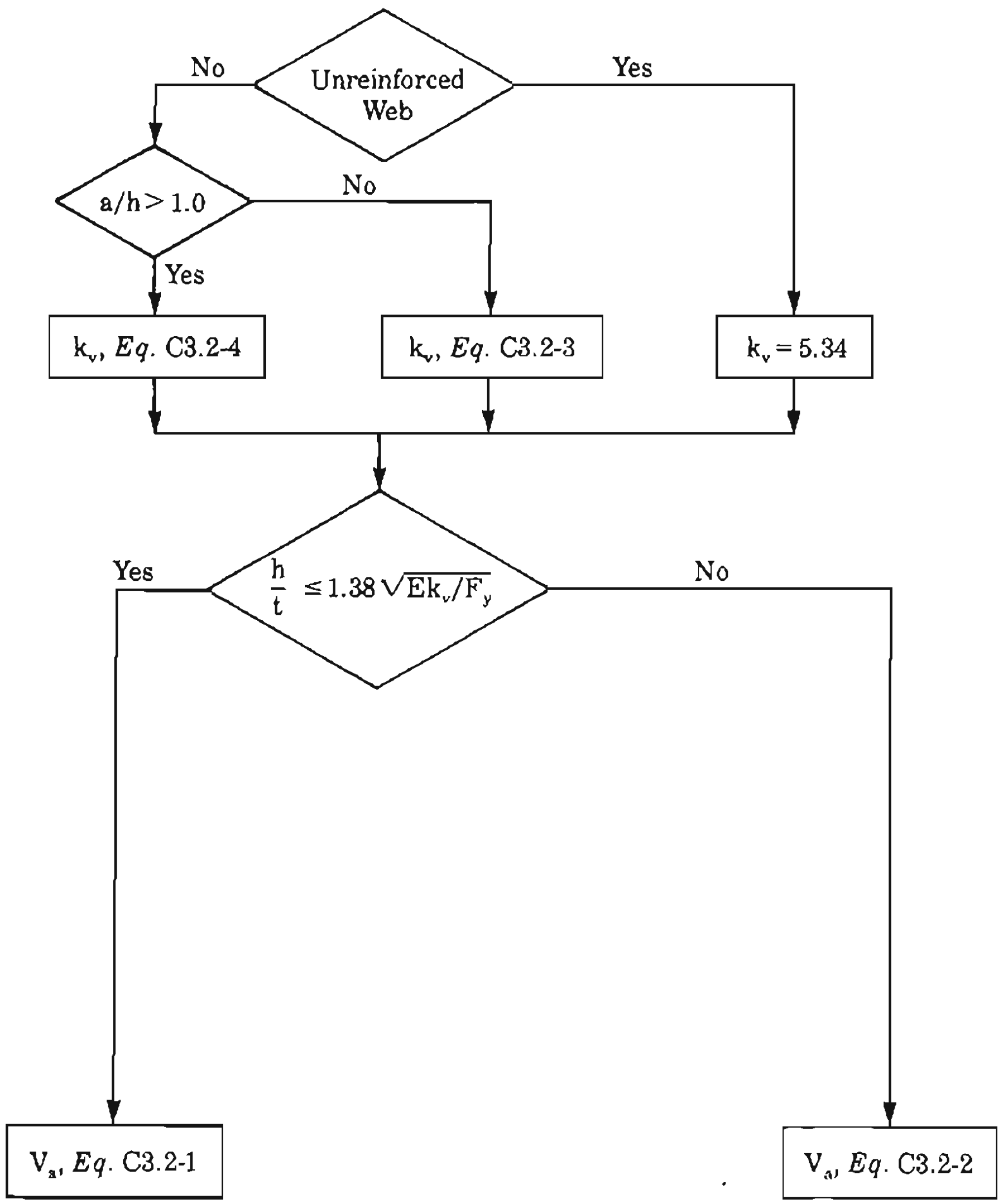
C3.1.2 Lateral Buckling Strength



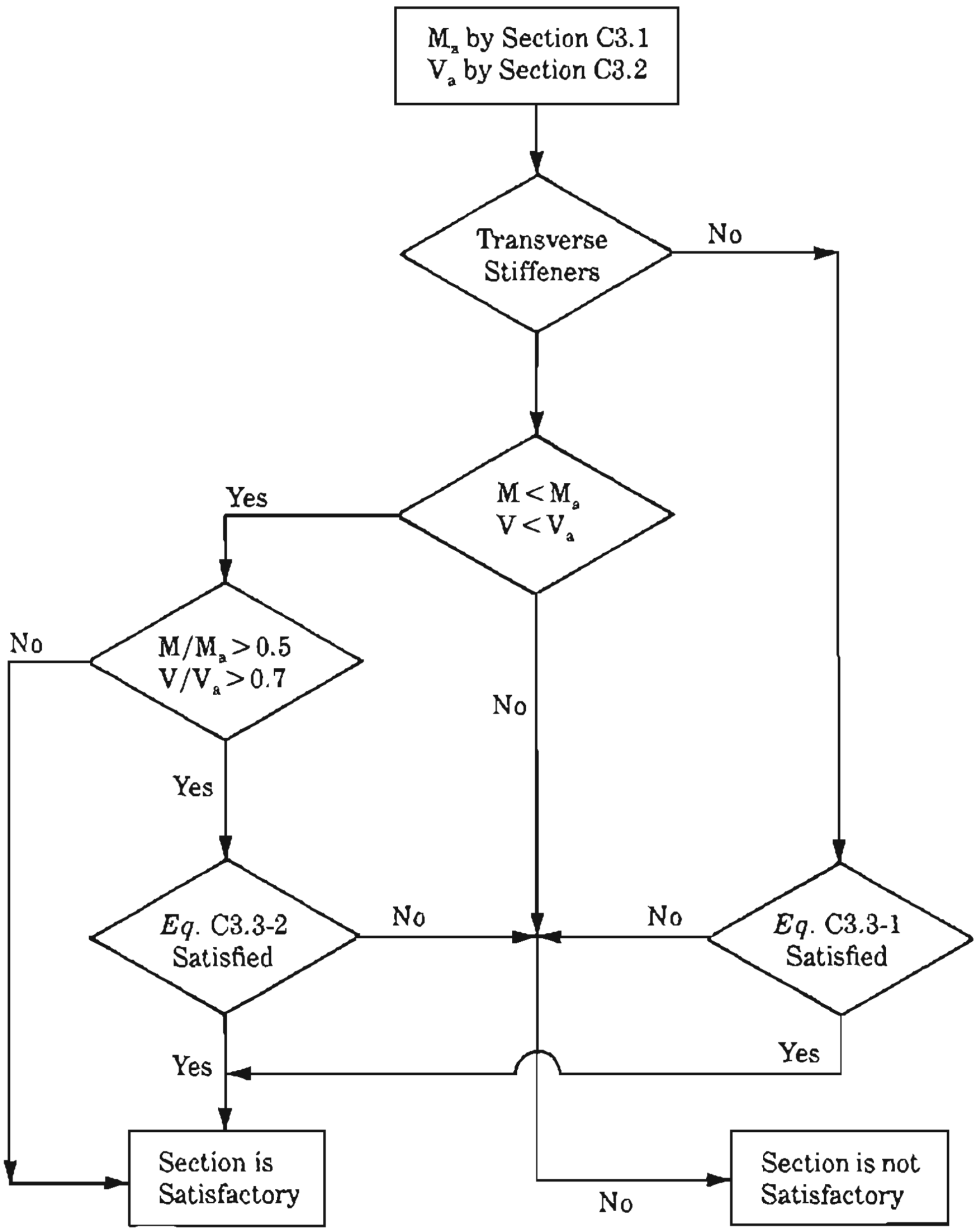
C3.1.2 Lateral Buckling Strength (continued)



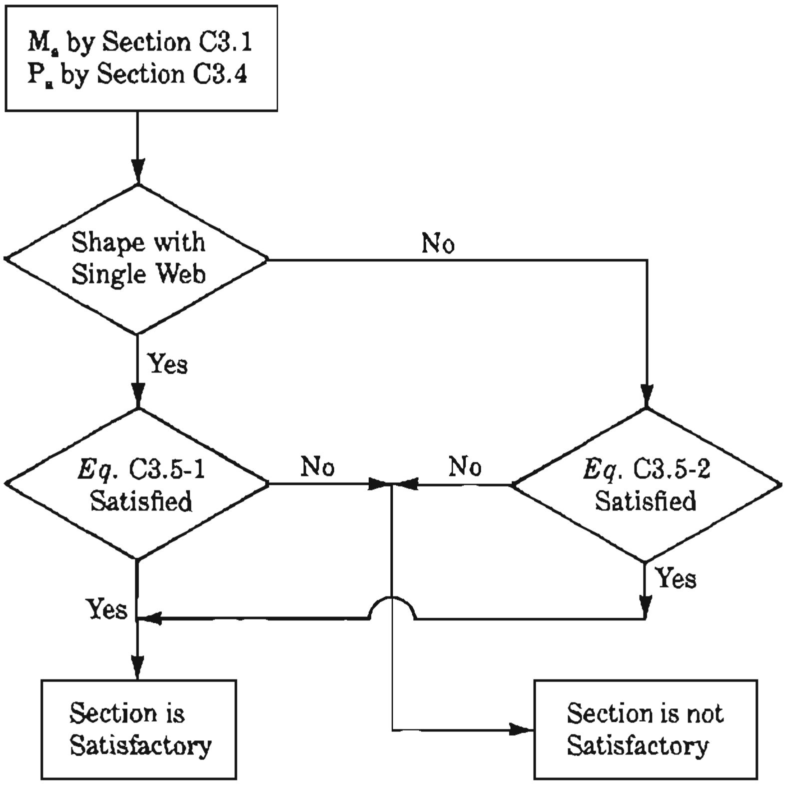
C3.2 Strength for Shear Only



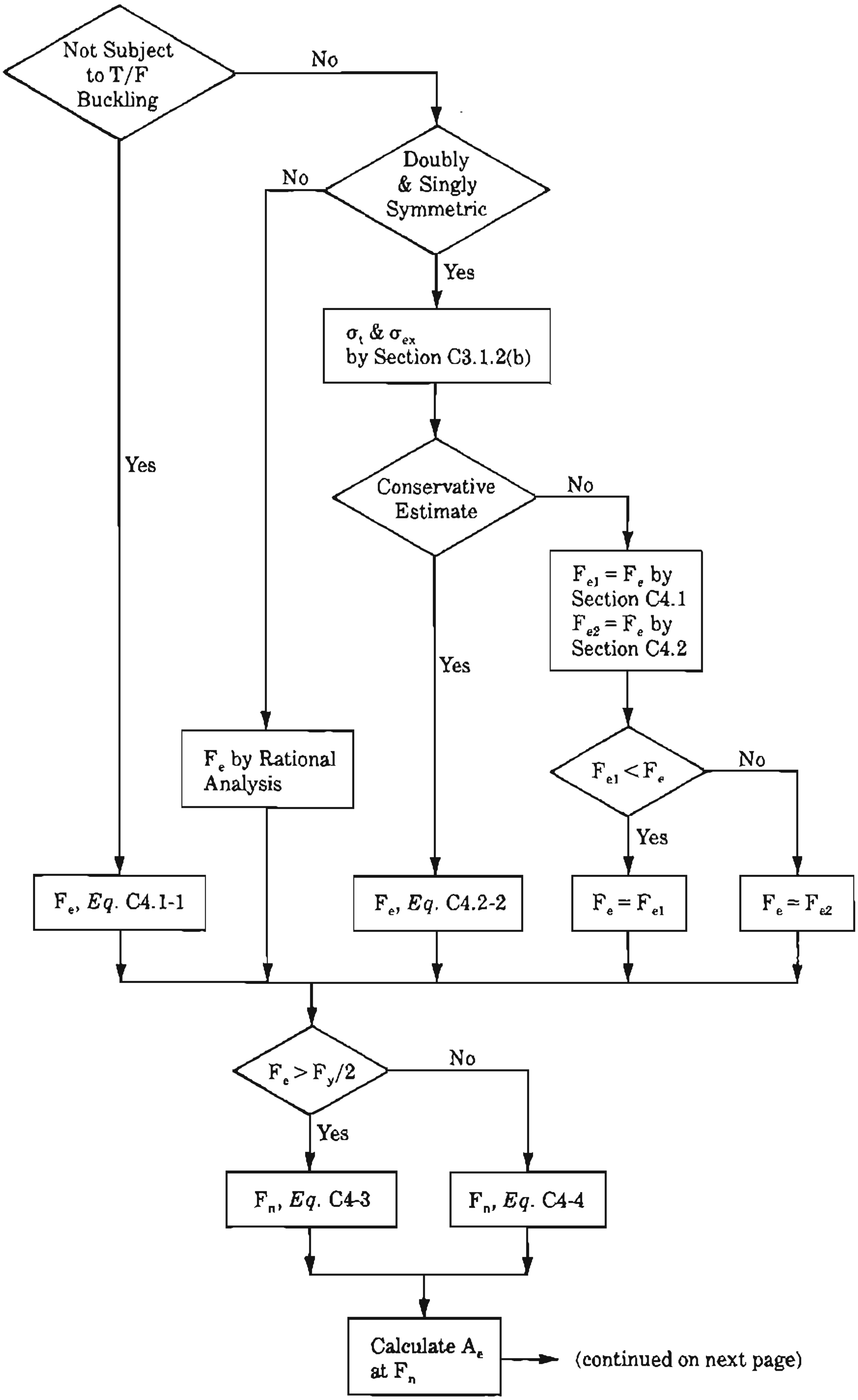
C3.3 Strength for Combined Bending and Shear



C3.5 Combined Bending and Web Crippling Strength



C4 Concentrically Loaded Compression Members



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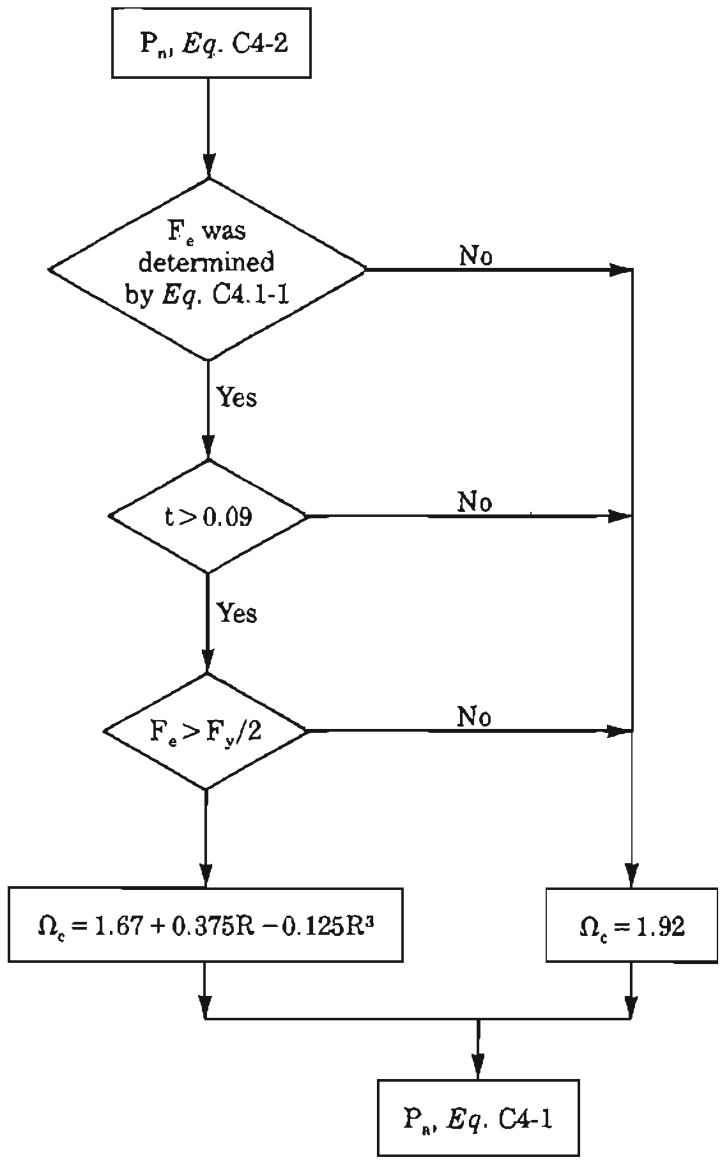
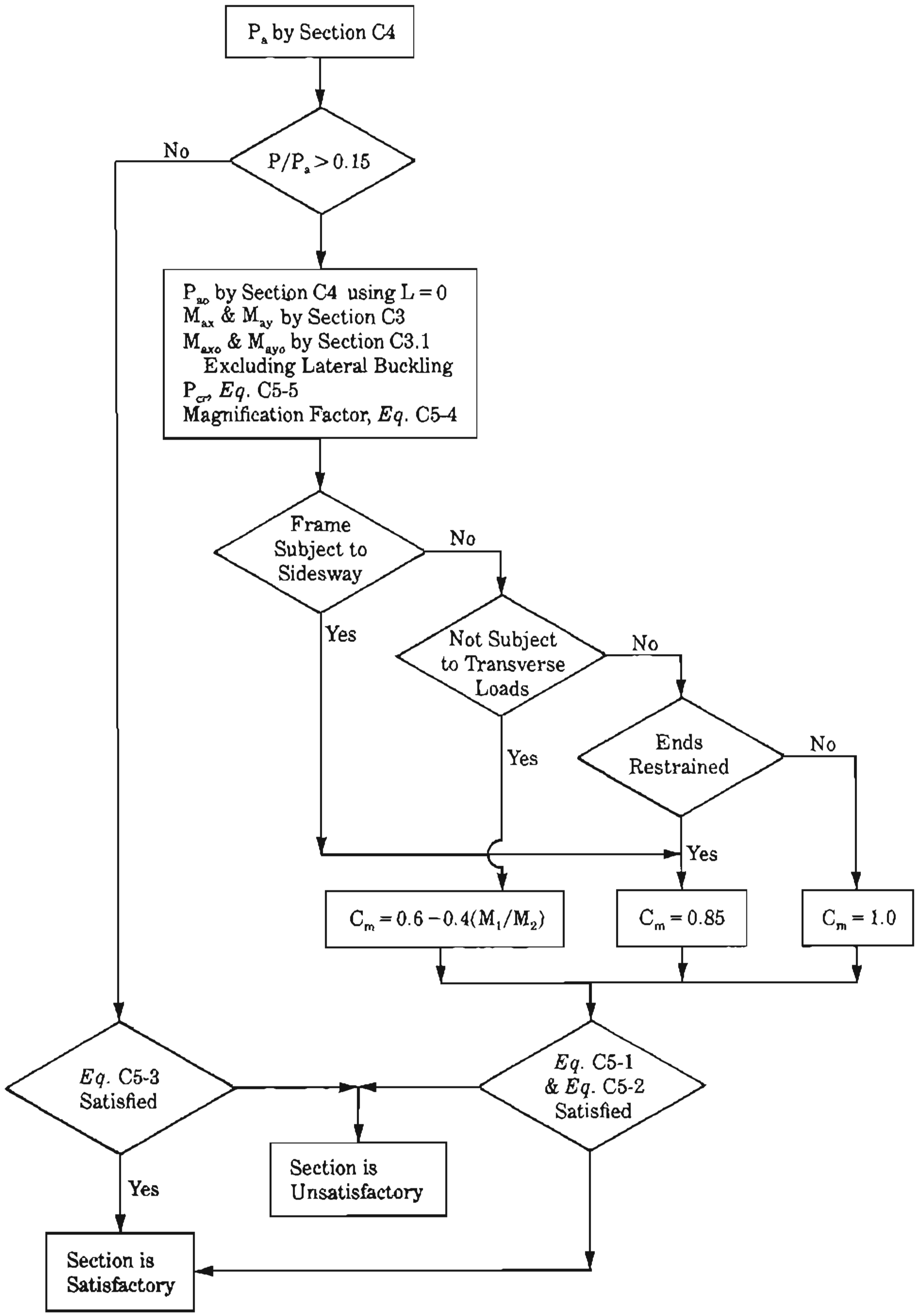
C4 Concentrically Loaded Compression Members (continued)



C5 Combined Axial Load and Bending



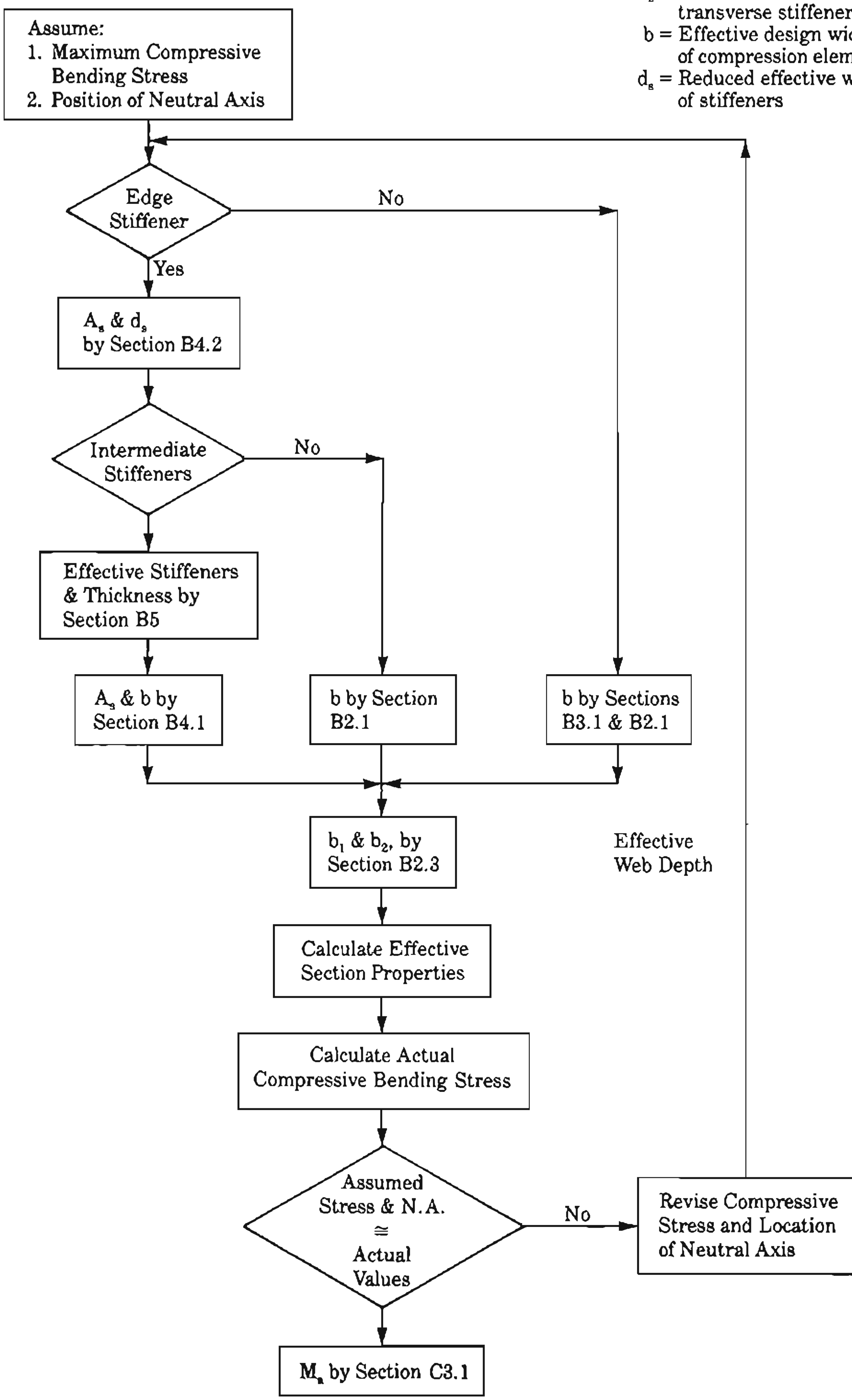
Moment Capacity of Flexural Members

Definition of Symbols

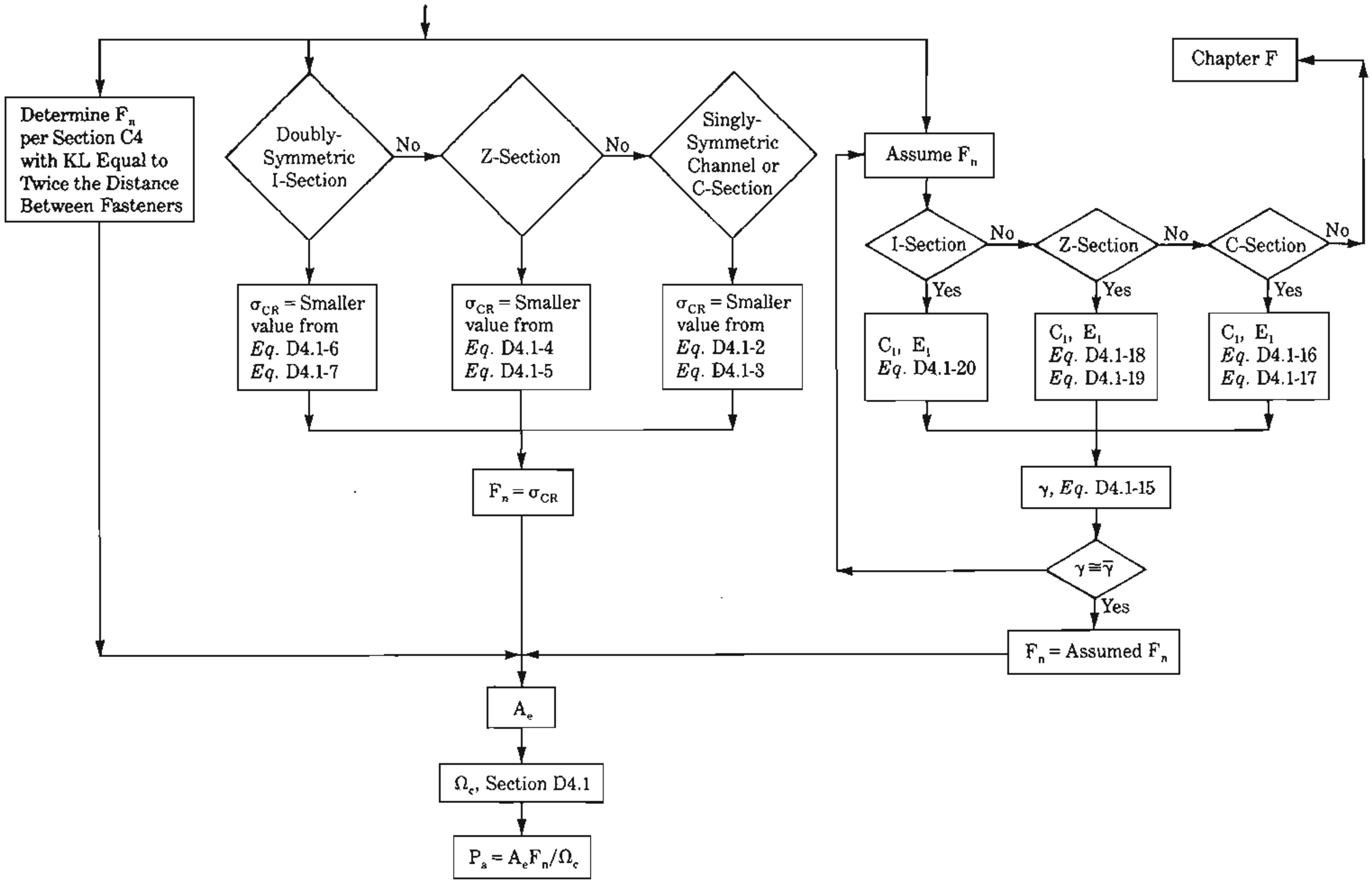
A_s = Cross-sectional area of transverse stiffeners

b = Effective design width of compression element

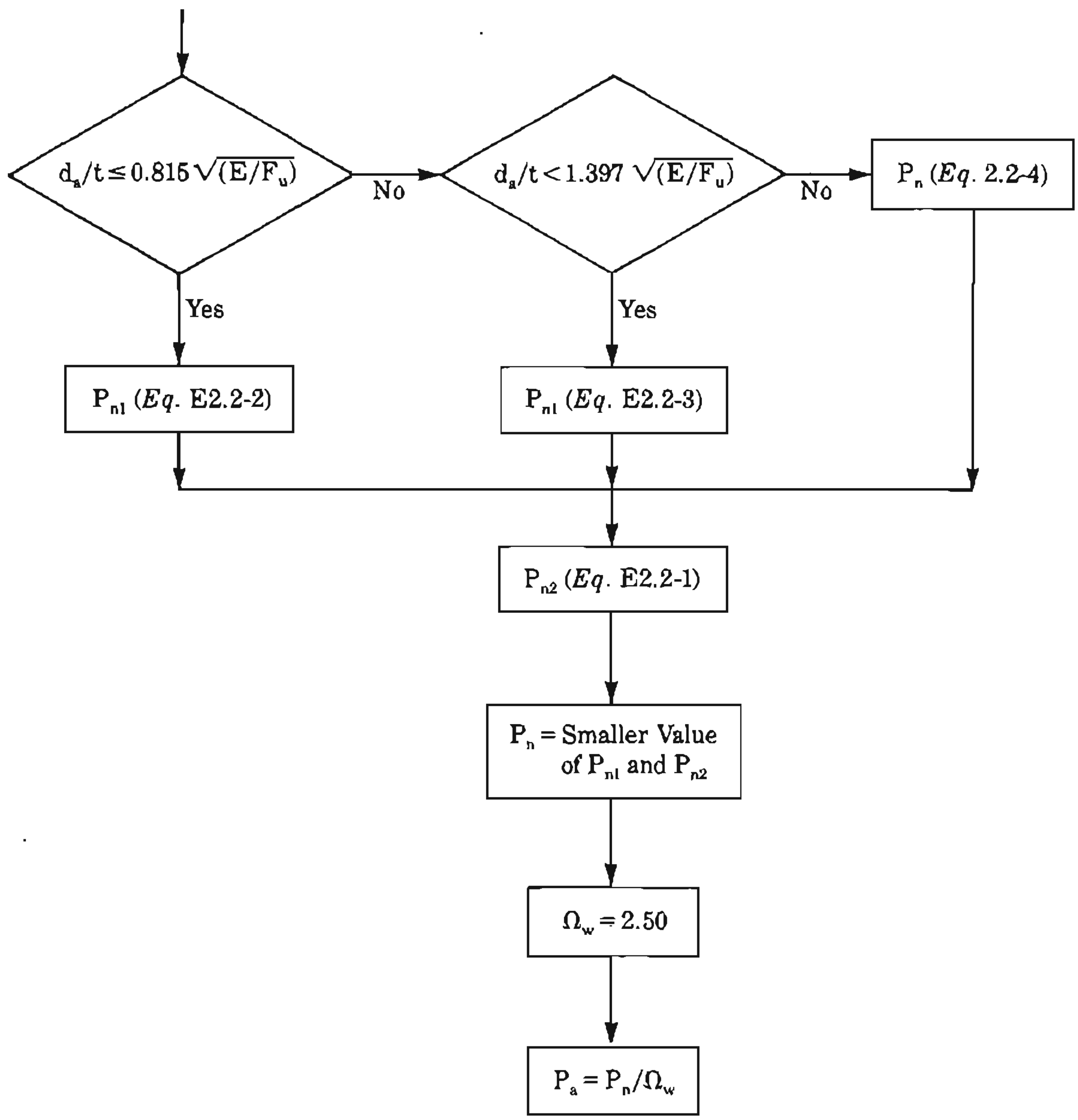
d_s = Reduced effective width of stiffeners



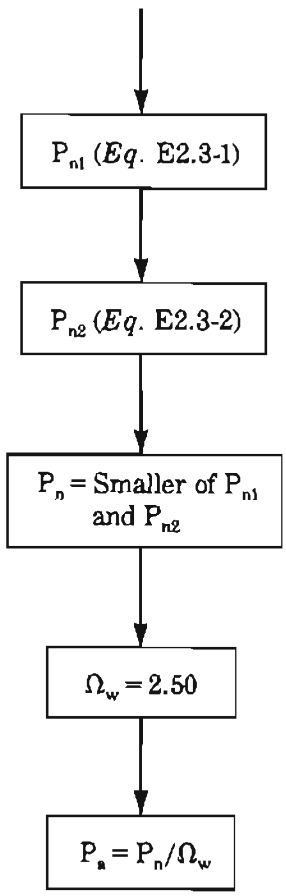
D4.1 Capacity of a Wall Stud



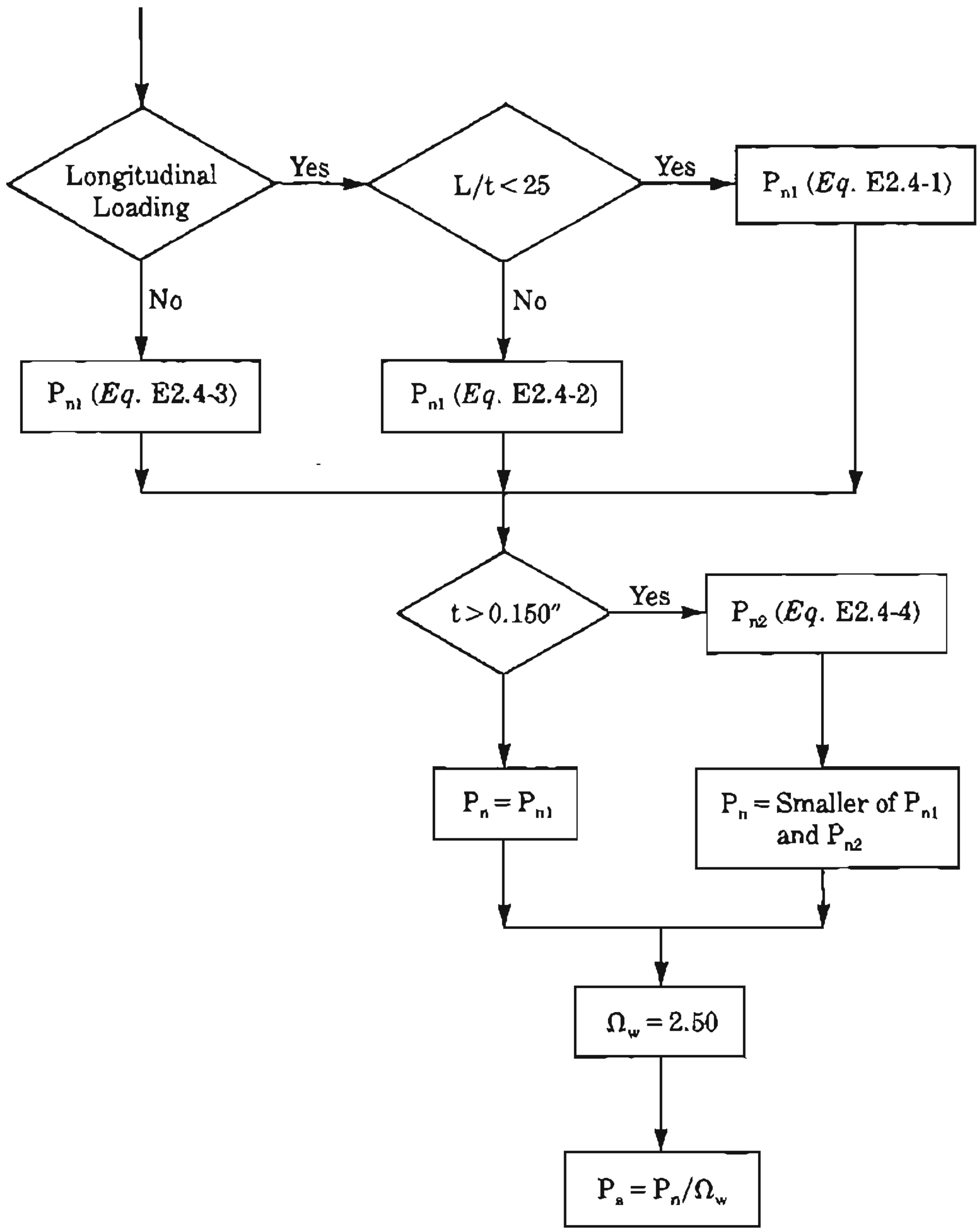
E2.2 Arc Spot Welds



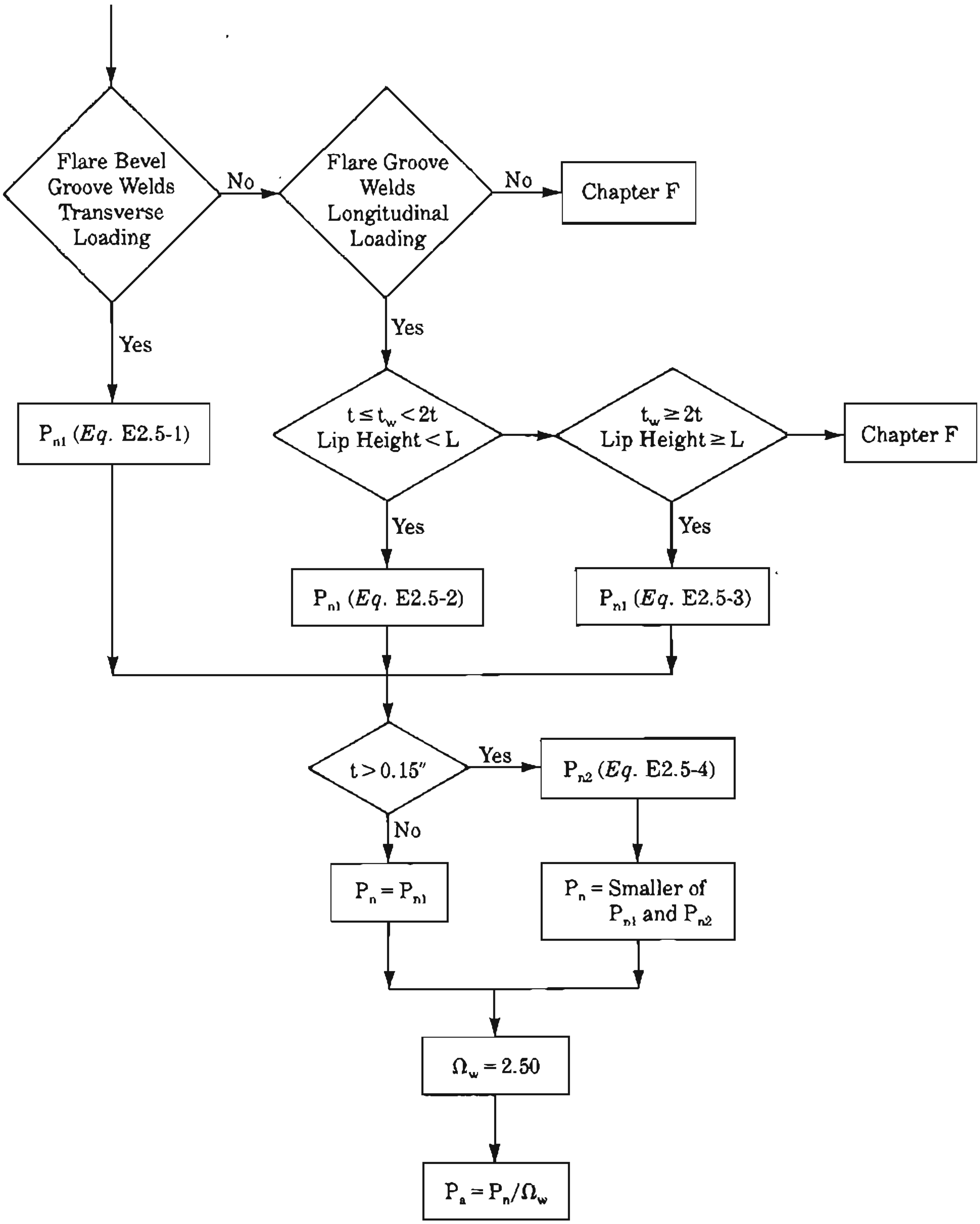
E2.3 Arc Seam Welds



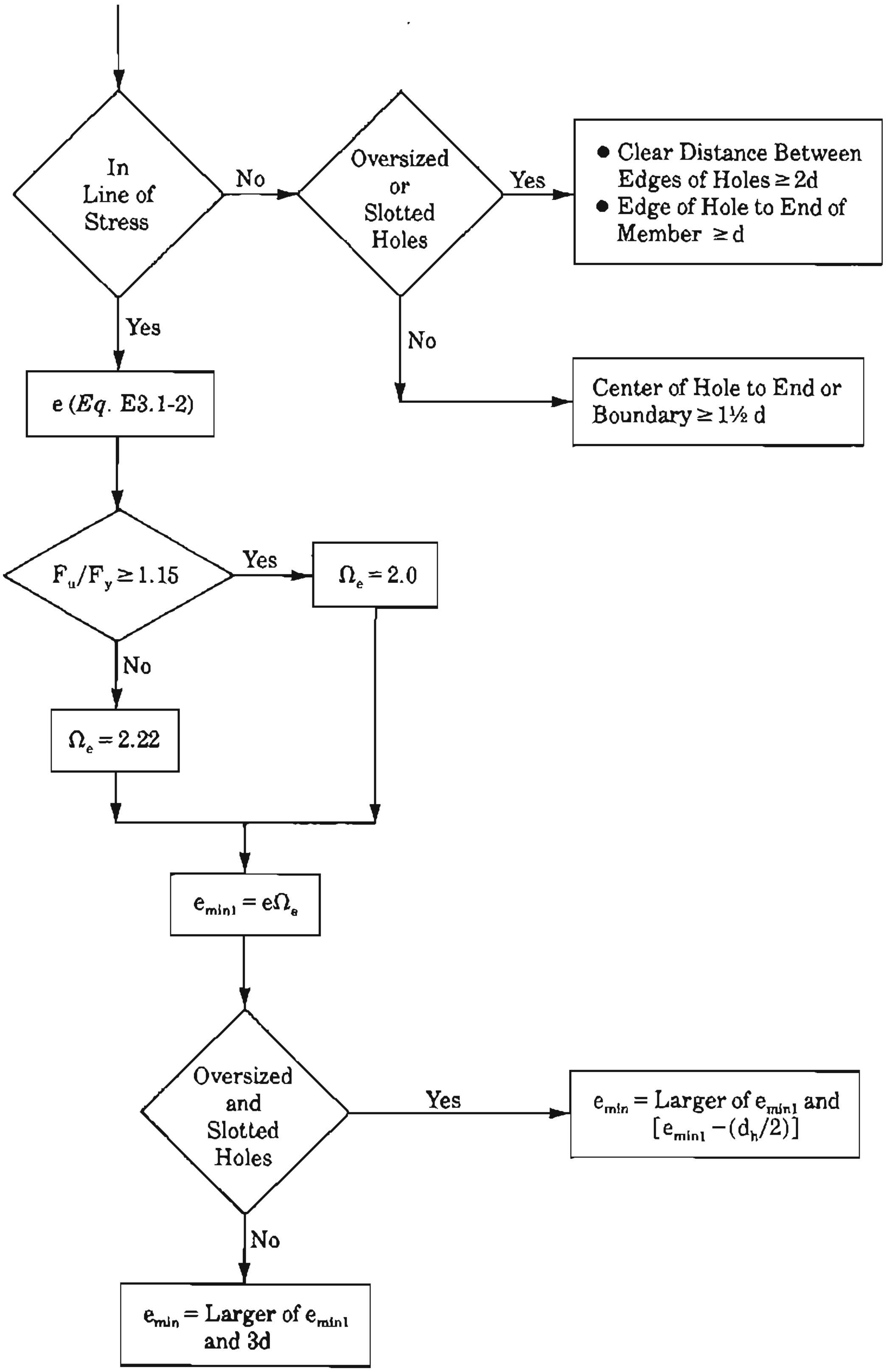
E2.4 Fillet Welds



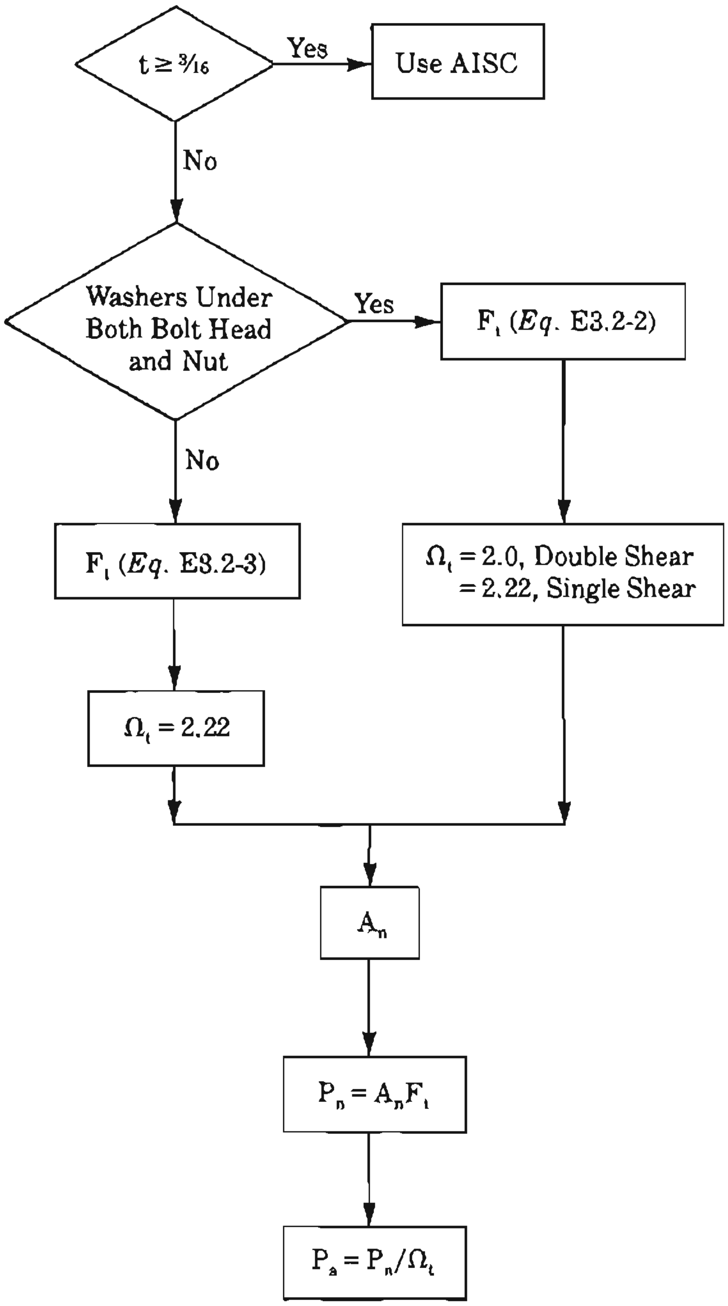
E2.5 Flare Groove Welds



E3.1 Spacing and Edge Distance



E3.2 Tension in Connected Part



TEST PROCEDURES

TEST PROCEDURES

FOR USE WITH THE
AUGUST 19, 1986, EDITION OF THE

SPECIFICATION FOR THE DESIGN OF COLD-FORMED STEEL STRUCTURAL MEMBERS

Cold-Formed Steel Design Manual - Part VII

AMERICAN IRON AND STEEL INSTITUTE
1000 16th STREET, NW
WASHINGTON, DC 20036

1st Printing—March 1987

Produced by W. P. Reyman Associates, Inc. New York

PREFACE

This document, *Part VII of the Cold-formed Steel Design Manual,* contains test procedures applicable to various provisions of the *Specification*. It consists of two parts:
Test Method for Rotational-Lateral Stiffness of Beam-to-Panel Assemblies, and Test Method for Stub Columns.

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August 1986

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TEST PROCEDURES
BASED ON THE
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ROTATIONAL-LATERAL STIFFNESS TEST METHOD FOR BEAM-TO-PANEL ASSEMBLIES

1. Scope

1.1 The purpose of this test is to determine the rotational-lateral stiffness of beam-to-panel assemblies. This test method is used primarily in determining the strength of beams connected to panels as part of a structural assembly.* The unattached "free" flange of the beam is restrained from lateral displacements and twisting by the bending stiffness of the beam elements, the connection between the "attached" flange of the beam and the panel, and the bending stiffness of the panel.

1.2 This test method applies to structural subassemblies consisting of panel, beam, and joint components, or of the joint between a wall, floor, ceiling, or roof panel and the supporting beam (purlin, girt, joist, stud).

1.3 This test method is also used to establish a limit of the displacements for avoiding joint failure.

1.4 The combined stiffness of the assembly determined by this method, K, consists of: (a) the lateral stiffness of the beam, K_a, which is a function of the geometry of the beam and geometric details of the beam-to-panel connection, (b) the local stiffness of the joint components in the immediate vicinity of the connection, K_b, which is affected by the type of fasteners, the fastener spacing used, and the geometry of the elements connected, and (c) the bending stiffness of the panel, K_c, which is a function of the moment of inertia of the panel, the beam spacing, and the beam location (edge vs interior). The latter stiffness shall be taken into account by theoretical analysis or by using the alternate test procedure described in Section 10.

1.5 For specific geometric conditions the design engineer may require duplicate testing using a new specimen with the beam orientation, or the force direction, reversed.

2. Description of Terms

2.1 *Subassembly*—A subassembly is a representative portion of a larger structural assembly consisting of a wall, floor, ceiling, or roof panel with one beam connected to the panel either continuously or at regular intervals (Figure 1).

2.2 *Panel*—The panel used in the subassembly may be made of any structural material, for example: aluminum, reinforced concrete, fiberboard, gypsum board, plastic, plywood, steel, etc. (Figure 1).

2.3 *Beam*—A beam may have an open or a closed cross section. One flange of the beam is connected to the panel, and is called the "attached" flange. The other is the "unattached" flange (Figure 1).

*1986-Edition of the AISI *Specification*, Section C.3.1.3.

2.4 *Joint or Connection*—A joint or connection includes the local area around a mechanical fastener, weld, or adhesively bonded area that connects the beam with the panel. The local area also includes filler material such as insulation located between the panel and the beam flange.

2.5 *Lateral Load*—The total lateral load, P (in kips), is applied to the unattached flange of the beam (Figure 2) in a plane parallel to that of the original panel position.

2.6 *Lateral Deflection*—The lateral deflection (Figure 2) is the lateral displacement, D (in inches), of the unattached flange due to the lateral load, P.

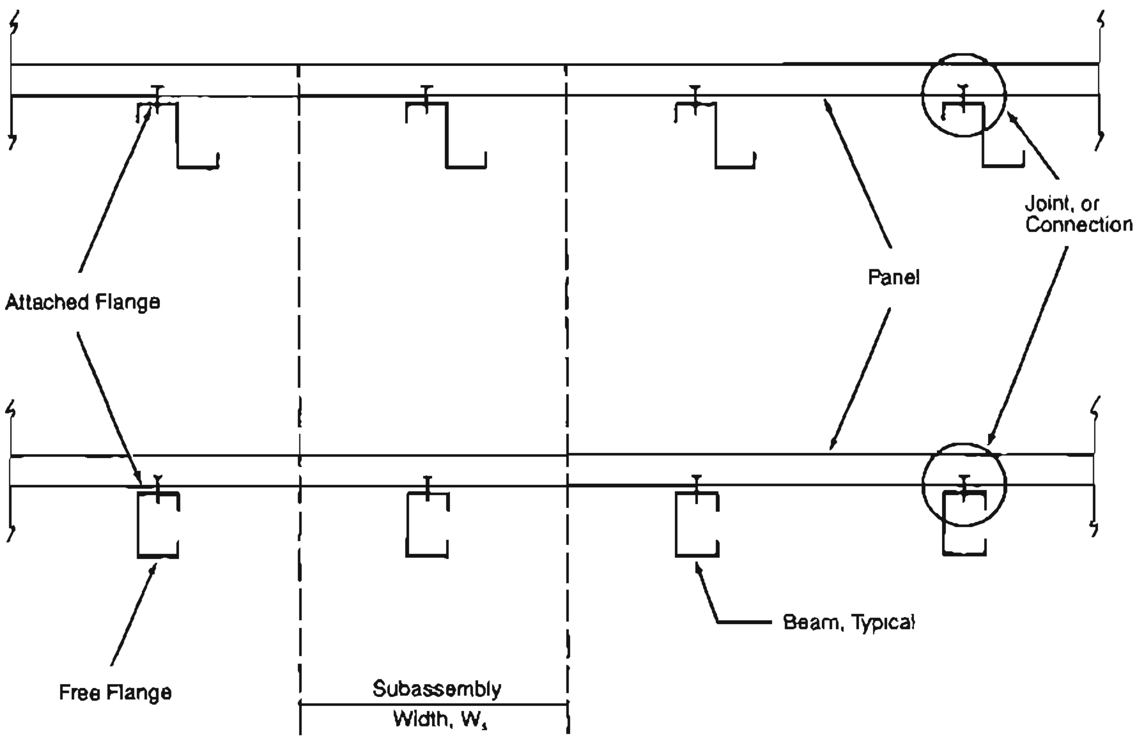
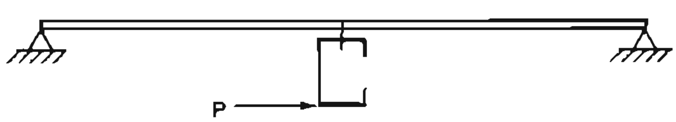
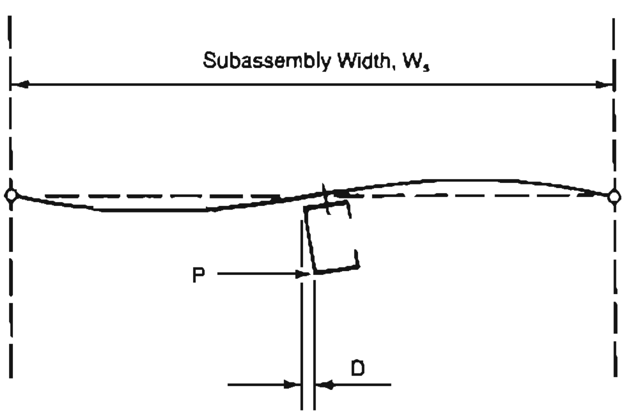


Figure 1 Wall, Floor, Ceiling, or Roof Assembly



(a) Loading Diagram



(b) Deflected Subassembly

Figure 2 Loaded and Deflected Subassembly

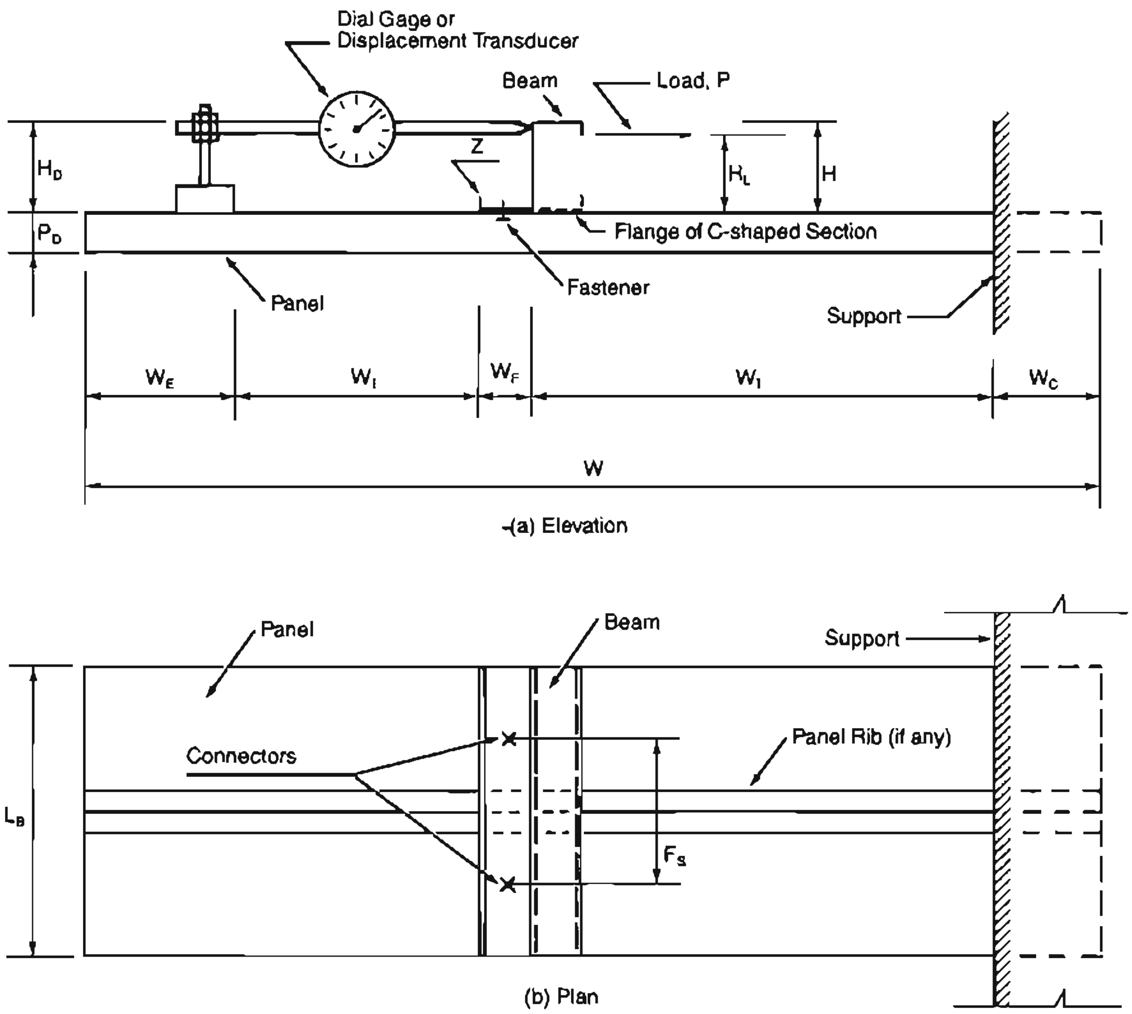


Figure 3 Test Specimen and Horizontal Test Setup

2.7 *Rotational-Lateral Stiffness*—The rotational-lateral stiffness, K, is equal to the total lateral load applied on the unattached flange of the test beam, divided by the length dimension of the beam, L_B (Figure 3b), and divided by the lateral deflection of the unattached flange of the beam at that load level. Thus, the units of K are: kips of lateral load per inch of beam length per inch of deflection, or k/in./in.

3. Materials

3.1 Components of the test specimen(s) shall be measured, and the component suppliers shall be identified.

3.2 Physical and material properties of the panel and beam shall be determined according to the latest edition of Specification ASTM E370 or other applicable standards.

4. Test Specimens

4.1 The overall panel width, W (Figure 3), of the specimen shall be such that the dial-gage support and the specimen support are each separated from the beam by a distance, W_I, not less than the largest of the following distances: (a) 1.5 times the overall panel depth P_D, (b) the overall width of the attached beam flange W_F, and (c) the fastener spacing along the flange of the beam, F_S. For ribbed panels, W_I shall also exceed two times the width of the attached flat of the panel.

4.2 The clamped width of the specimen, W_C, shall be at least equal to two times the panel depth, but not less than 2 inches.

4.3 The end dimension, W_E, shall be long enough to conveniently attach a dial gage or an extensometer to the end of the panel.

4.4 The minimum overall panel width shall be equal to:

$$W = W_E + 2W_J + W_F + W_C \tag{1}$$

4.5 The minimum beam and panel length, L_B, of the test specimen shall not be less than the largest of (a) two times the maximum connector spacing, F_S, used in actual field installations, or (b) the nominal coverage width of the panel. The specimen shall contain at least two fasteners in each line of connections along the beam.

4.6 Each specimen shall be assembled under the supervision of a representative of the testing laboratory, either at the manufacturer's facilities or at the testing laboratory.

4.7 Each specimen shall be assembled from new material; i.e., materials not used in previous test specimens, and in accordance with manufacturer's specifications.

4.8 The fabrication and field installation procedures specified for the overall assembly, and the tools used, shall also be used in the specimen construction as much as possible.

4.9 Drilled or punched pilot holes in the panels or beams shall be the same as those used in field installations.

5. Test Setup

5.1 The test specimens may be tested in a horizontal or vertical position (Figure 3 and Figure 4, respectively). The zero-load readings of the deflection-measuring device(s) shall be recorded.

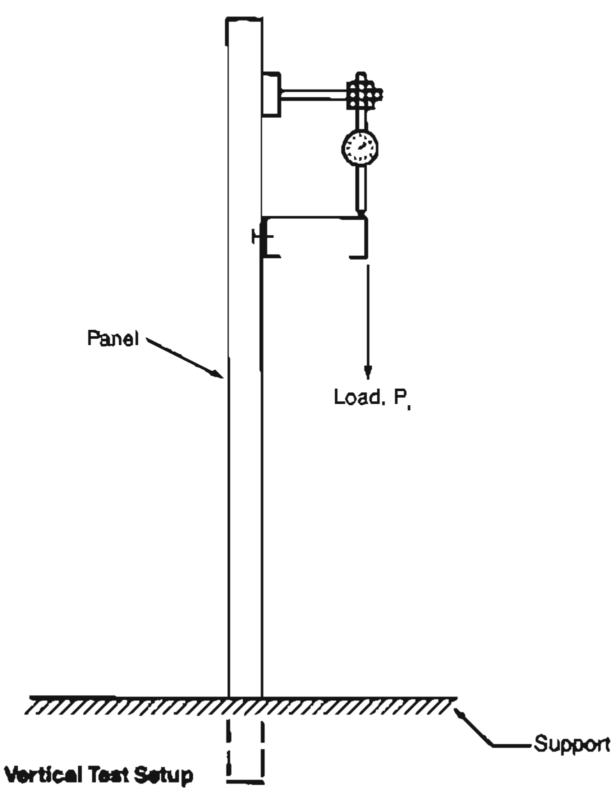


Figure 4 Vertical Test Setup

5.2 The clamped end of the panel shall be the only support of the test specimen.

5.3 When the test specimen panel is a hollow-core, corrugated, or trapezoidal panel, voids of the clamped regions shall be filled with filler materials such as wood, gypsum, or similar filler materials to ensure that the clamped overall depth of the panel is reasonably maintained. For foam-filled sandwich panels, if necessary, the filler material over the distance W_C may be replaced with wood, gypsum, or similar filler materials.

5.4 Loads applied to the unattached flange shall be introduced as close as possible to the extreme fiber of the beam, or at the intersection of the outer faces of the unattached flange and the web.

5.5 If the beam does not have a flat face perpendicular to the panel at the locations where the load is to be applied and the lateral displacement is to be measured, brackets are to be mechanically attached to the beam web to provide a flat surface. Figure 5 shows a typical application of a load bracket and/or dial gage bracket. The attachment of either bracket shall be accomplished such that the bracket does not stiffen the beam, or reduce its distortion.

5.6 The total lateral load applied, P, shall be distributed over several locations, if necessary, to reduce variations in the lateral deflection along the length of the unattached flange.

5.7 The load application shall be accomplished by chain or wire, and the necessary precautions shall be taken to ensure that the direction of the applied load remains essentially parallel to the original plane of the panel (Figure 5).

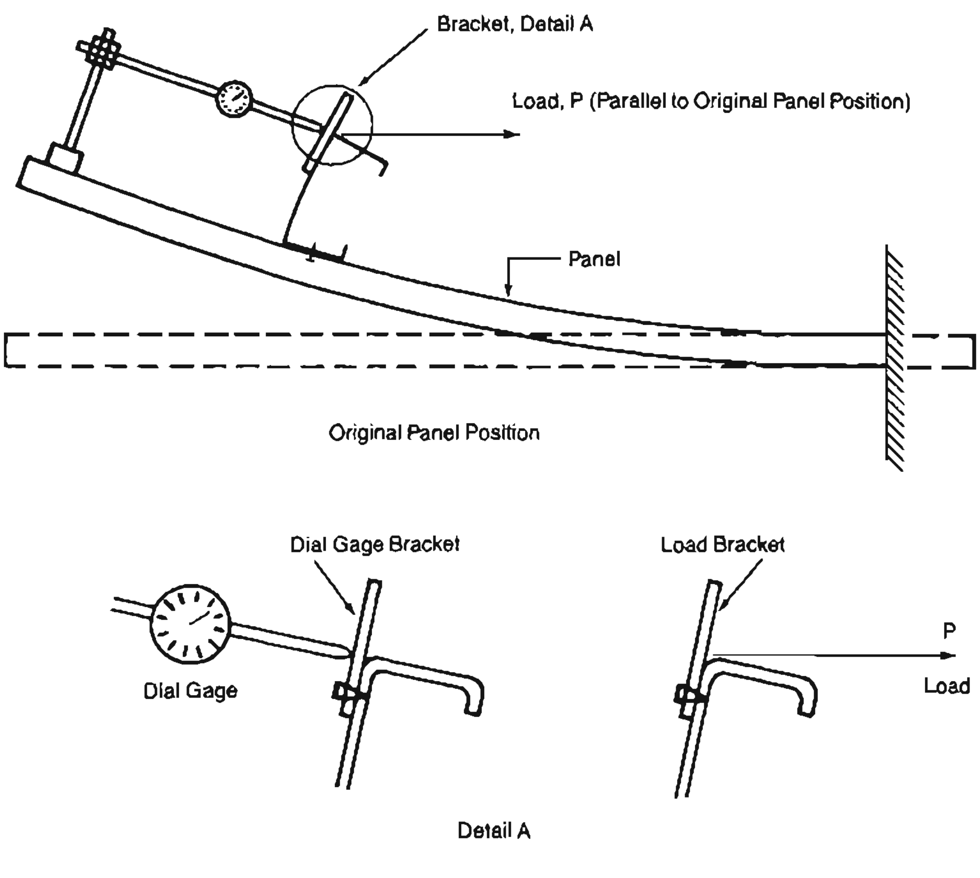


Figure 5 Dial Gage and Load Bracket

5.8 One or more dial gages or displacement transducers shall be used to measure the lateral displacements during loading. The gages shall be arranged symmetrically about the mid-width point, and have graduations at not greater than 0.001-inch intervals.

6. Test Procedures

6.1 The dial-gage height, H_D, and load height, H_L, as shown in Figure 3, shall be arranged such as to equal as close as possible the overall beam depth, H. Prior to loading the test specimen, the dimensions H_D and H_L, and the dial-gage readings shall be recorded.

6.2 No preload is to be used. The load shall be applied in a direction which is critical for the intended use of the results.

6.3 The applied load shall be increased in five or more equal increments to the maximum expected value, in order to produce deflection increments of not more than 5 percent of the beam depth.

6.4 If the specimen includes fiberglass insulation or other non-metallic elements in the joint between panel and beam, the load shall be held at each increment for 5 minutes before reading the lateral movement.

6.5 After each load increment is added, and the deflection has stabilized, the load and lateral movement of the unattached flange shall be measured and recorded.

6.6 A test shall be terminated at failure (fastener pullout, fastener failure, panel buckling, panel failure, beam failure, etc.) and the mode of failure recorded, unless the design engineer has determined that the application of the rotational-lateral stiffness, K, occurs at lower load or displacement levels and that the test may be terminated earlier.

7. Number of Tests

7.1 The minimum number of tests for one set of parameters shall be three. For parametric studies using multiple values of one or more parameters a smaller number of tests may be used.

7.2 If used as part of a series of at least three tests, one test is sufficient for a specific condition of an all-metallic mechanically-fastened specimen using the same basic components, but using unique geometrical or physical-property differences such as fastener spacings, different beam or panel yield strengths, etc.

7.3 Three tests are required for any specific condition of welded or adhesively-bonded specimens, or for specimens using non-metallic materials.

7.4 When the rotational-lateral stiffness for three or more panel or beam thicknesses with otherwise identical parameters is to be determined, at least two specimens each with the minimum and the maximum thickness shall be tested. For a ratio of maximum-to-minimum thicknesses greater than 2.5, additional specimens with intermediate thicknesses must be tested. One test of every thickness may be used in accordance with Section 7.2.

7.5 When the rotational-lateral stiffness for a range of screw spacings is to be determined, the minimum number of specimens shall be as follows: For a ratio of maximum-to-minimum screw spacings equal to or less than 2, at least two specimens each with the minimum and the maximum screw spacing shall be tested. For a range of five or more different screw spacings, or for a ratio of maximum-to-minimum screw spacings greater than 2, additional specimens with intermediate spacings must be tested. One test of every screw spacing may be used in accordance with Section 7.2

7.6 Where the rotational-lateral stiffness for a range of other panel parameters—such as

yield or ultimate strength, changes in geometry, etc.—are to be determined, a number of tests similar to the requirements under Sections 7.2 through 7.5 shall be performed.

7.7 For unsymmetric or staggered fastener arrays and/or beams unsymmetric about a plane parallel to the web, duplicate tests may be required by the design engineer using new specimens with the beam orientation, or the force direction, reversed.

8. Test Evaluation Procedure

8.1 Typical load-displacement curves (P vs. D) obtained from the tests are as shown in Figure 6. For multiple tests of one set of test parameters, the curve resulting in the lowest value of K_t, as defined in Section 8.2, shall be used for the test evaluation procedure.*

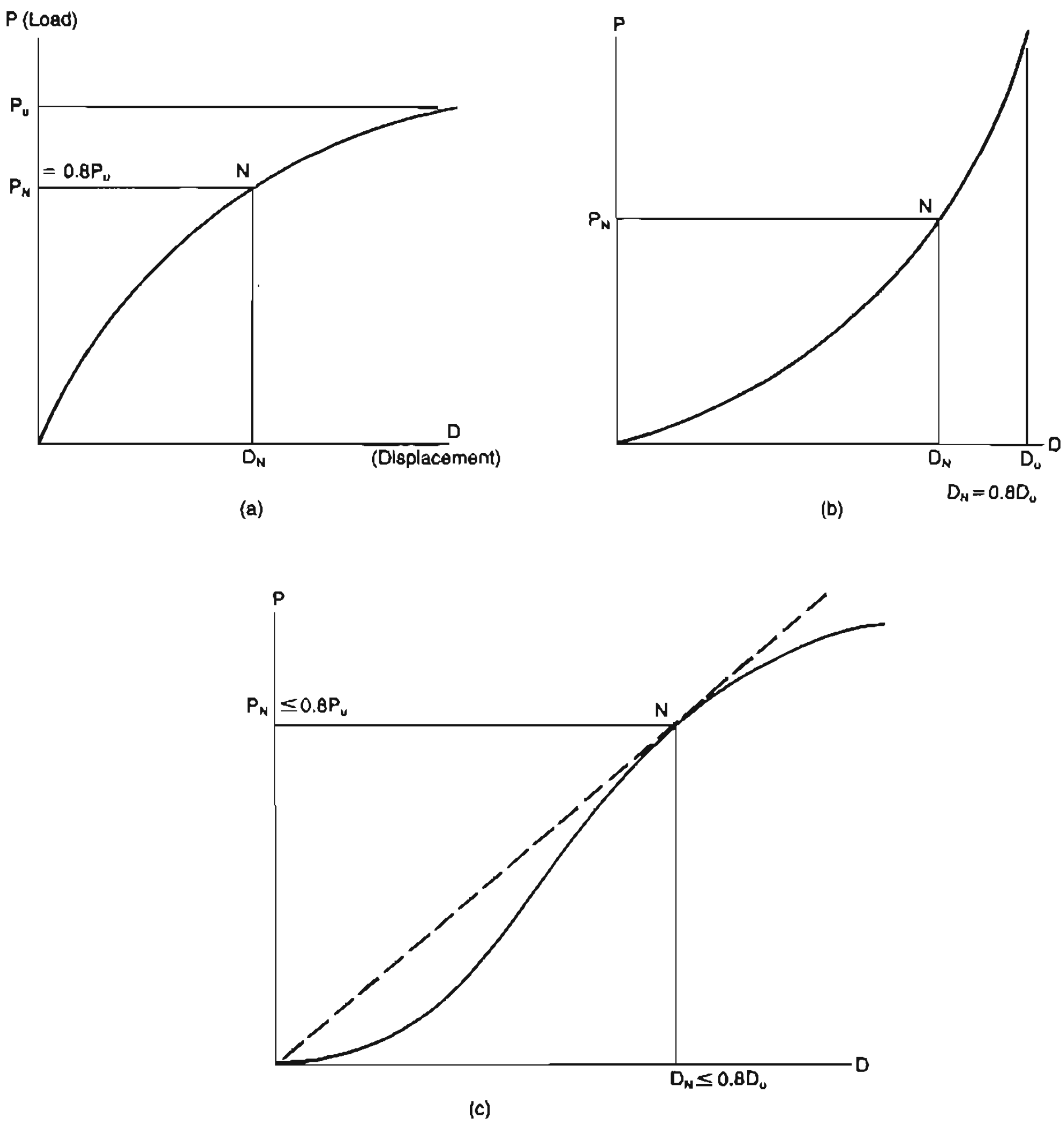


Figure 6 Typical Load-displacement Curves

*The test stiffness, K_t, includes the stiffness effects of the beam, K_a, and the beam-to-panel connection, K_b, but excludes the effects of the bending stiffness of the panel, K_c, and follows the relationship $K_t = (1/K_a + 1/K_b)^{-1}$.

8.2 The test stiffness, K_t, at any load level is determined by

$$K_t = P/D/L_B \tag{2}$$

8.3 The nominal test stiffness. K_N, shall be determined by

$$K_N = P_N/D_N/L_B \tag{3}$$

where P_N and D_N shall be determined for a point, N, such that either P_N shall be equal to 0.8 times the ultimate load, P_u, for load-displacement curves as shown in Figure 6(a), or the displacement D_N shall be equal to 0.8 times the ultimate displacement, D_u, for load-displacement curves as shown in Figure 6(b), or by a tangent drawn from the origin to the P-D curve as shown in Figure 6(c), resulting in $P_N \leq 0.8P_u$ and $D_N \leq 0.8D_u$.

8.4 When the design engineer specifies in advance a desired maximum lateral displacement limit of D_{NL}, the test may be discontinued when D_{NL} is reached, and K_N may be determined from P_N at D_{NL}, as long as the limits under Section 8.3 are observed and D_{NL} is not exceeded in actual design applications.

8.5 Where either H_D or H_L are not equal to the overall beam height, H, K_t and K_N shall be corrected by the factor H_DH_L/H^2.

8.6 In addition, K_t and K_N shall be adjusted by the stiffness contributions of the panel, K_c, derived from the linear-elastic displacement analysis representing the actual design applications, unless such an analysis shows that these contributions are insignificant. Alternately, the panel stiffness may be included by using the alternate test method under Section 10.

8.7 For subassemblies such as shown in Figure 2, the applied lateral test loads cause a bending moment distribution in the panel similar to that shown in Figure 7, and a lateral displacement of the unattached flange of the beam, D_c, equal to

$$D_c = PH_L^2W_S/(12EI) \tag{4}$$

where W_S is the width of the subassembly (Figure 2 and Figure 7), E is the modulus of elasticity of the panel material, and I is the effective moment of inertia of the panel cross section (obtained from deflection determination calculations for cold-formed metal deck panels).
The panel stiffness is equal to

$$K_c = 1/D_c \tag{5}$$

8.8 The overall rotational-lateral stiffness of the subassembly shall be determined as

$$K = (1/K_t + 1/K_c)^{-1} \tag{7}$$

8.9 When tests covering ranges of parameters (thickness, yield strengths, screw spacings, etc.) are conducted according to Section 7, a linear interpolation may be used to determine intermediate K values.

9. Test Report

9.1 The test report shall consist of a description of all specimen components, including drawings defining the actual and nominal geometry, material specifications, material properties test results describing the actual physical properties of each component, and the sources of supply. Differences between the actual and the nominal dimensions and material properties shall be noted in the report.

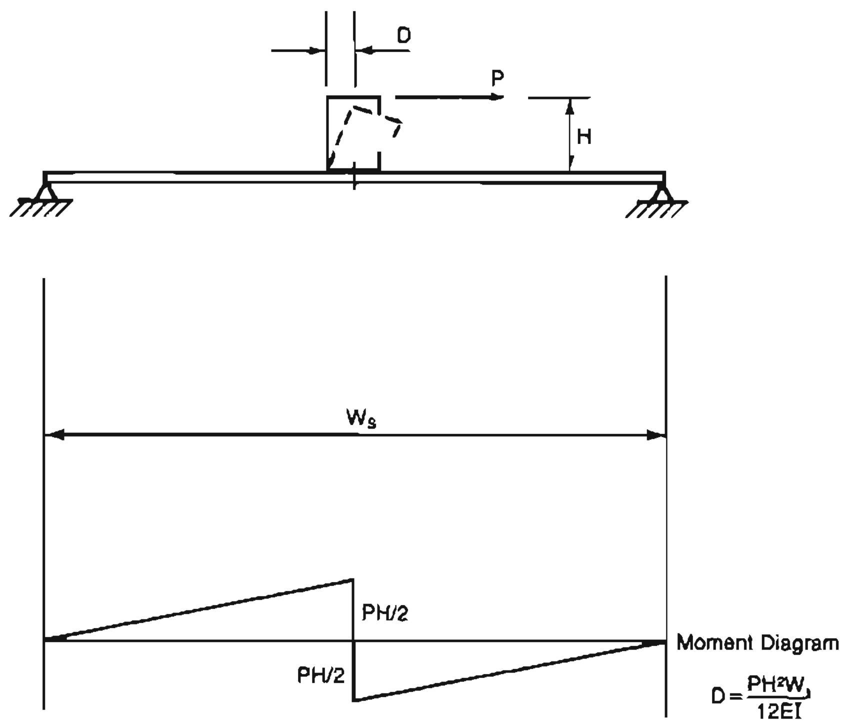


Figure 7 Bending Moment Diagram for Panel with an Interior Beam

9.2 In addition, the test report shall contain a sketch or photograph of the test setup, the latest calibration date and accuracy of the equipment used, the signature of the person responsible for the tests, and a tabulation of all raw and evaluated test data.

9.3 All graphs resulting from the test evaluation procedure shall be included in the test report.

9.4 A summary statement, or tabulation, shall be included in the summary of the report to define the actual and nominal rotational-lateral stiffness derived from the tests conducted, including all limitations.

10. Alternate Rotational-Lateral Stiffness Test*

10.1 To include the panel-stiffness contribution in the test, rather than by linear-elastic analysis, the design engineer may request a test specimen and setup as shown in Figure 8 and Figure 9, respectively.

10.2 The test specimens shall be as described under Section 4 except as follows.

10.2.1 The minimum overall panel width of the specimen, W (Figure 8), shall be

$$W = W_E + W_I + W_C \tag{6}$$

10.2.2 The minimum end dimension, W_E, shall equal the width of the attached beam flange plus 4 inches to allow the development of local deformation patterns around the fasteners as they would develop in a real structure.

*This method is conservative as compared to the basic methods which analytically account for the stiffness of the panel.

10.2.3 For specimens representing interior-beam subassemblies, as shown in Figures 1 and 2, the dimension W_1 of the test specimen (Figure 8) shall be equal to 1/12 of the subassembly width, W_S (Figures 1 and 2), to assure that the overall rotational-lateral stiffness contribution of the test-specimen panel is the same as that of the subassembly.

10.2.4 For other subassembly conditions, W_1 shall be determined to represent the actual conditions.

10.3 The test-setup shall be as described under Section 5 except as follows.

10.3.1 The clamped support as shown in Figures 8 and 9 shall be sufficiently rigid to minimize the rotation and translation of the test specimen at the support.

10.3.2 The lateral-displacement measuring device shall be located on a support fixed relative to the clamped support of the test panel, as shown in Figure 9.

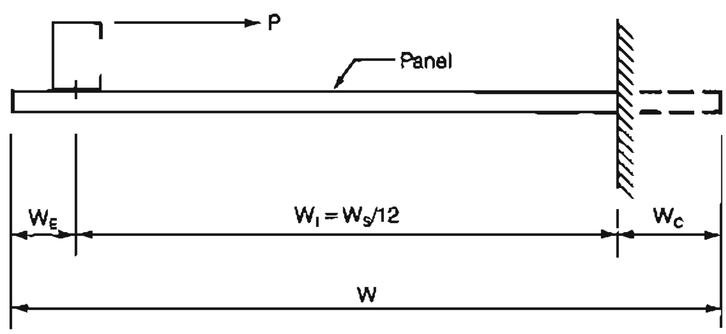


Figure 8 Panel Width for Alternate Test Procedure

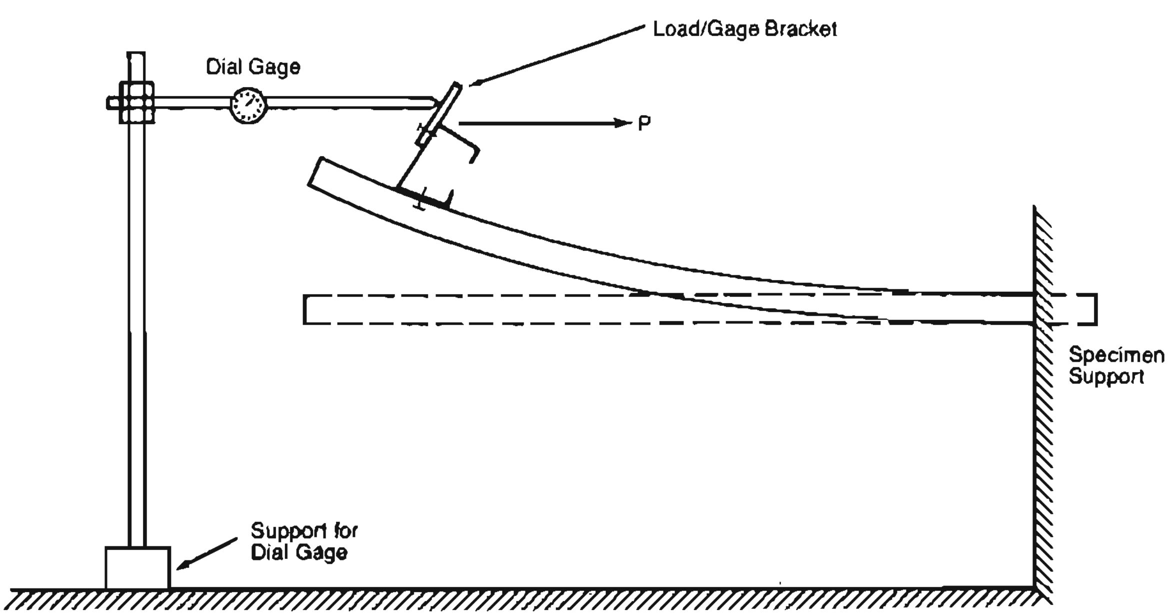


Figure 9 Test Setup For Alternate Test

10.4 Test procedures shall be the same as described under Section 6.

10.5 The number of tests shall be determined as described in Section 7.

10.6 The test-evaluation procedure shall follow the underlying principles used to develop Section 8. The test stiffness at any load level shall be determined according to Equation 2 and the nominal test stiffness shall be determined according to Equation 3. No further adjustments are needed.

10.7 For other interior-beam spacings, for exterior-beam conditions, or for other geometrical conditions, the measured displacements shall be adjusted by a linear-elastic analysis to represent the actual field conditions, unless such an analysis shows that these displacements and their effect on K are insignificant.

STUB-COLUMN TEST METHOD[1] FOR EFFECTIVE AREA OF COLD-FORMED STEEL COLUMNS

1. Scope

1.1 This test method covers the determination of the effective cross-sectional area of cold-formed steel columns. It primarily considers the effects of local buckling and residual stresses and applies to solid or perforated columns that have holes (or hole patterns) in the flat and/or curved elements of the cross section (1).[2]

1.2 The effective area is used to determine the allowable axial loads of cold-formed column sections in accordance with the *AISI Specification For The Design Of Cold-Formed Steel Structural Members*, hereafter called AISI *Specification.*

1.3 The effective area is a variable section property of columns. It reflects the effects of local buckling in relatively thin area elements caused by axial stresses, or loads. When the axial load is zero, the effective area is equal to the gross cross-sectional area; however, when an axial load is applied, the effective area may be less than the gross area. In such a case, the effective area will reduce with increasing load.

1.4 Local buckling reduces the axial load-carrying capacity that would otherwise be limited only by general yielding or overall column buckling. The amount of the reduction depends on the width-to-thickness ratio of the flat elements of the column cross section, the yield strength of the steel sheet from which the column is formed, and the size and frequency of holes or hole patterns, if present.

2. Applicable Documents

2.1 ASTM Standards:
A370—Tensile Test Method For Steel Sheets
E4—Verification of Testing Machines

2.2 *AISI Specification for the Design of Cold-Formed Steel Structural Members, 1986 Edition.*

[1]This test and evaluation method will be proposed to the appropriate ASTM Committee for review and adoption.
[2]Numbers in parentheses refer to references at the end of this test method.

3. Terminology

3.1 ASTM Definitions Standards:
E6—Definitions of Terms Relating to Methods of Mechanical Testing.
E380—Standard for Metric Practice.

3.2 Description of terms specific to this standard:
Elements = Straight or curved portions of the cross section of a column or stub column.
Local Buckling = The local buckling mode of a flat element of a column cross section, which influences the overall column-buckling behavior.
Overall Buckling = Buckling of a column as a function of its overall length.
Stub-Column = An axial compression member of the same cross section and material as the column for which the strength needs to be determined, but of sufficiently short length to preclude overall column buckling, if possible.

3.3 Symbols:

A = the gross cross-sectional area of a column without holes or perforations, or the minimum gross cross-sectional area of a column with holes or perforations.

A_a = the average of all gross cross-sectional areas of the stub columns in a test unit, or the average of gross cross-sectional areas of a stub column.

A_e = the effective cross-sectional area of a stub column at a load less than the ultimate test load, or the effective area of a full-length column.

A_{ei} = the effective cross-sectional area of a stub column at load P_i.

A_{eu} = the nominal effective cross-sectional area at ultimate load adjusted to the nominal thickness and the minimum specified yield strength.

A_{euR} = the average effective cross-sectional area of a test unit of stub columns at the ultimate axial load.

A_{eu1} = the effective cross-sectional area of a stub column with parameters of Test Unit 1 at ultimate load.

A_{eu2} = the effective cross-sectional area of a stub column with parameters of Test Unit 2 at ultimate load.

A_1 = the minimum gross cross-sectional area of a stub column with parameters of Test Unit 1 at ultimate load.

A_2 = the minimum gross cross-sectional area of a stub column with parameters of Test Unit 2 at ultimate load.

D = the axial shortening of a stub column at load P.

D_i = the axial shortening of a stub column at load P_i.

D_u = the axial shortening of a stub column at load P_u.

f = the average axial stress assumed to be uniformly distributed over the effective cross-sectional area, A_e.

f_i = the average axial stress assumed to be uniformly distributed over the effective cross-sectional area, A_{ei} at load P_i.

f_o = the average axial stress assumed to be uniformly distributed over the effective cross-sectional area, A_e, above which the section is not fully effective.

F_n = the nominal ultimate stress, assumed to be uniformly distributed over the effective cross section of a column as calculated from Section C4 of the AISI *Specification*, at which flexural, torsional, torsional-flexural, or local buckling, and/or yielding, may occur.

F_u = the ultimate stress, assumed to be uniformly distributed, at which local failure occurs in a tested stub column.

F_y = the minimum specified elastic limit or yield stress of column or stub-column material.

F_{ya} = the average elastic limit or yield stress of the sheet steel for a given test unit.

F_{yi} = the individual elastic limit or yield stress of the sheet-steel specimens in a test unit.

i = load-displacement-reading number for a particular stub-column test (load displacement D_i at load P_i).

j = total number of load-displacement readings taken for a particular stub-column test.

L = the length of the stub-column test specimen.

Lp = the pitch of a repeating pattern of perforations along the longitudinal column axis.

n = the ratio of the effective cross-sectional area at the ultimate load to the full cross-sectional area, A_{eu}/A.

P = the applied axial compression force (column load).

P_i = the applied load at load-increment i.

P_n = the nominal failure load of a column.

P_u = the ultimate stub-column load at which local failure occurs.

P_{ua} = the average of all ultimate stub-column loads within a test unit.

r = the minimum radius of gyration of the cross-sectional area, A.

t = the nominal base-steel thickness exclusive of coating.

t_a = the average of all base-steel thicknesses within a test unit, exclusive of coating.

W = the greatest overall width of the cross section including corner(s).

4. Significance

4.1 This test method provides requirements for testing, and equations to determine, the effective area of a cold-formed column section at ultimate load, A_{eu}, and the load- or stress-dependent effective area, A_e. These properties are used in the AISI *Specification* to determine the ultimate and less-than-ultimate column strengths. The ultimate column strength, P_u, is the product of the minimum specified yield stress, F_y, or the buckling stress F_n, and the corresponding effective cross-sectional area at that stress, A_{eu}. At an applied column strength of P less than P_u, the corresponding effective cross-sectional area shall be A_e.

4.2 The test method also provides a means to observe, measure, and account for local buckling deformations when the appearance of a column section under stress must be determined.

4.3 An inherent assumption of the test method is that true stub-column behavior (which considers local buckling effects only) is achieved when overall column-buckling effects are eliminated. For this condition the ultimate test load on a stub column, P_u, equals the product of the effective cross-sectional area at ultimate load, A_{eu}, times the stress that causes local buckling, or times the yield stress of the virgin steel sheet. In case overall buckling cannot be avoided because of geometrical constraints, the critical column-buckling stress must be used.

4.4 The determination of A_e may be conducted by either one of the two following methods:

(1) The basic, more simple, and conservative method:
This method is embodied in the main part of this document and is based on the measured test loads of stub columns and their measured and tested physical and mechanical properties.

(2) An alternate and less conservative method:
This method is based on the shortening of stub columns which occurs during testing. Also, this evaluation method requires more calculations. The results of this method lead to more accurate results for A_e, and to higher allowable axial loads at lower-than-ultimate stress levels. The evaluation procedure for this method is described in Appendix A.

5. Apparatus

5.1 The tests shall be conducted on a testing machine that complies with the requirements of ASTM E4.

5.2 Linear displacement devices for measuring lateral displacements shall have a 0.001-inch least-reading capability.

5.3 Measuring devices for determination of the actual geometry of a test specimen shall have a 0.001-inch least-reading capability.

5.4 If axial shortening is recorded, the measuring device shall have a 0.0001-inch least-reading capability.

6. Test Unit

6.1 A test unit shall include a minimum of three identical stub-column specimens and a minimum of two corresponding sheet-type tensile specimens.

6.2 The specimens within a unit shall represent one type of cold-formed steel section with the same specified geometrical, physical, and chemical properties. The specimens may be taken from the same column or from different production runs provided the source of the specimens is properly identified and recorded.

6.3 If stub-column specimens are taken from different production runs, at least two corresponding sheet-type specimens must be taken and tested from each production run.

6.4 The stub-column test specimens shall be used to determine:

(1) The actual geometry of each specimen.

(2) The ultimate stub-column test load.

(3) Axial shortenings at each load level if the alternate test-evaluation method described in Appendix A is used.

(4) Lateral displacements of the specimen at locations of interest (if desired).

6.5 The tensile test specimens shall be used to define the yield stress of each stub-column specimen acording to the requirements described in ASTM A370.

6.6 For each test specimen and test unit, the measured geometrical and tested physical properties of the individual specimens shall meet the requirements stated by the fabricator and material producer, respectively.

6.7 If the average area, thickness, or yield strength of a test unit varies by more than 20 percent from the respective nominal or specified-minimum value, the test unit is considered to be non-representative of the column section, and further evaluations of the effective area are considered to be invalid.

7. Stub-Column Specimens

The stub-column specimens shall meet length and end-flatness requirements as follows, depending on whether or not unconnected or welded endplates are used.

7.1 Stub-Column Length—The length requirements of the stub-column test specimen, L, as shown in Figures 1 and 2, are that it be (1) sufficiently short to eliminate overall column buckling effects, and (2) sufficiently long to minimize the end effects during loading, which means that its center portion be representative of the repetitive hole pattern in the full column.

7.1.1 To eliminate overall column-buckling effects, the stub-column length shall not exceed twenty times the minimum radius of gyration, r, of the cross section, A, except where necessary to meet the requirements of Sections 7.1.2 through 7.1.5.

7.1.2 For unperforated columns (Figure 1a) the stub-column length shall not be less than three times the greatest overall width of the cross section, W.

7.1.3 For perforated columns in which the pitch (gage length) of the perforation pattern, L_p, for a single hole or a group of holes, is smaller than, or equal to, the greatest overall width, W, of the cross section (Figures 1b and 1g), or for a single hole pattern with a gage length larger than the greatest overall width (Figure 1c), the specimen length shall not be less than three times the greatest overall width of the cross section, W. For widely spaced hole patterns (Figure 1c) the significant hole or hole pattern shall be located at or near the midlength of the stub column.

7.1.4 For perforated columns in which the pitch of the perforation pattern, L_p, is greater than the widest side, W, of the cross section (Figures 1d, 1e, 1f, and 1h), the specimen length shall not be less than three times the pitch of the perforation pattern.

7.1.5 For perforated sections in which the specimen end planes must pass through the normal perforation pattern (Figure 1i), a special section (Figure 1j) may be fabricated to obtain full cross-sectional surfaces at the specimen ends.

7.2 Stub-Column End Surface Preparation—The end planes of the stub-column test specimens shall be carefully cut to a flatness tolerance of plus or minus 0.002 inches. When the required flatness can be achieved, welding of the stub-column ends to the endplates is not required. However, when this flatness cannot be achieved, steel endplates shall be continuously welded to both ends of the specimen so that there shall be no gap between the ends of the stub column and the endplates.

7.3 Stub-Column Specimen Source—Stub-column test specimens may be cut from the commercially fabricated column product. Alternatively, stub columns may be specially

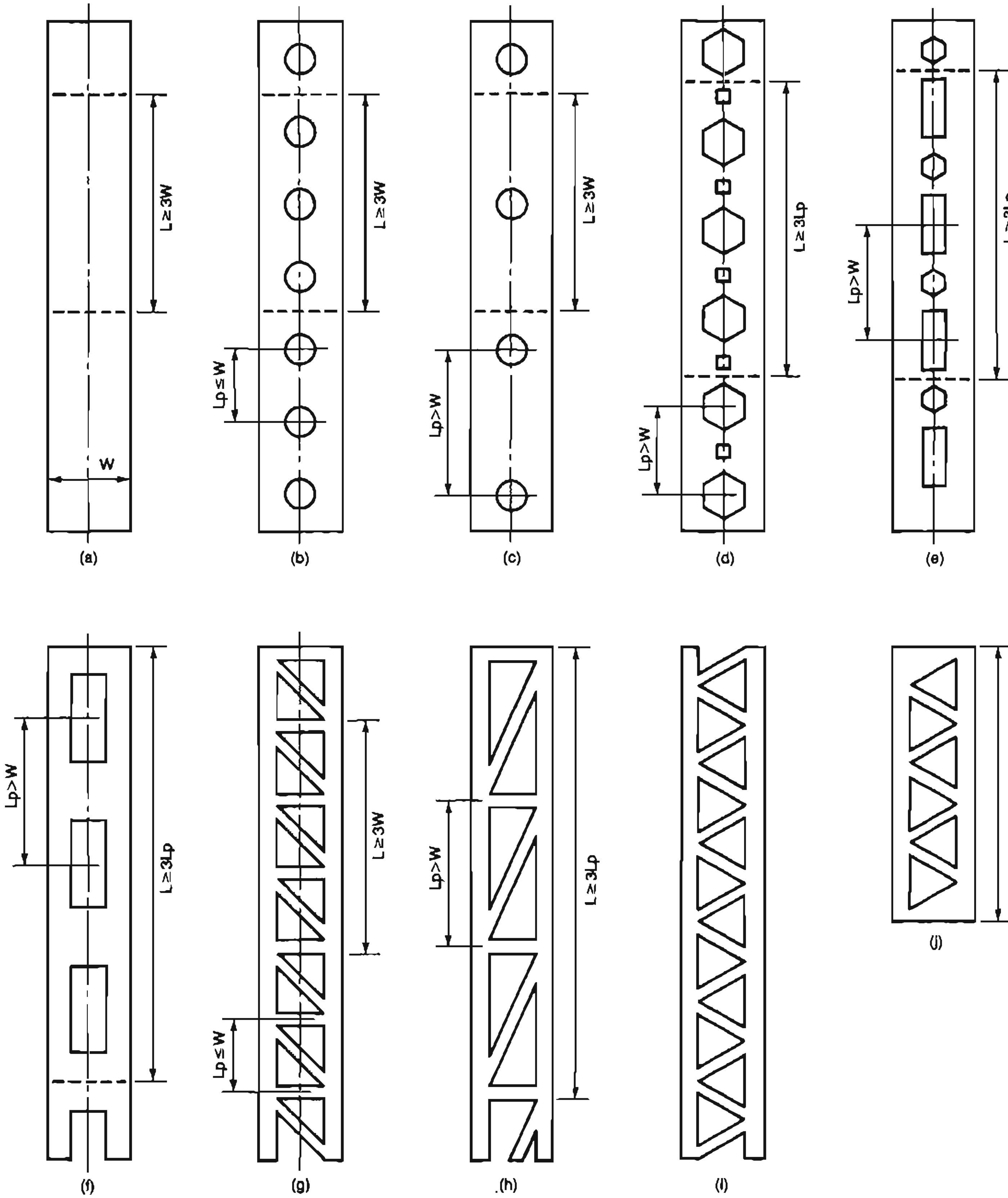


Figure 1 Hypothetical Perforation Patterns And Suggested Stub Column Lengths

NOTES: (1) Perforations shown are in a flat portion of a member with width W
(2) L = Length of Stub Column.
(3) L_p = Pitch Length of Perforation Pattern

fabricated provided care is taken not to exceed the cold work of forming expected in the commercial product; however, subsequent proof tests using specimens from commerically produced columns are recommended.

7.4 Tensile Specimen Source—Longitudinal tensile specimens shall be cut from the center of the widest flat of a formed section from which the stub-column specimens have been taken. If perforations are large and frequent in all flats of the formed section, the tensile specimens may be taken from the sheet or coil material used for the fabrication of the stub-column specimens. The tensile specimens shall not be taken from parts of a previously tested stub column.

7.5 Endplate Requirements—Steel endplates shall be at least 0.5 inch thick and have a flatness tolerance of plus or minus 0.002 inches.

8. Stub-Column Test Procedure

8.1 Vertical alignment of the stub column is essential to ensure that the applied load is uniformly distributed over the specimen end surfaces. Care should also be taken to center the specimen on the axis of the test machine.

8.1.1 Steel endplates shall be used to transfer the test loads uniformly into the stub columns (Figure 2).

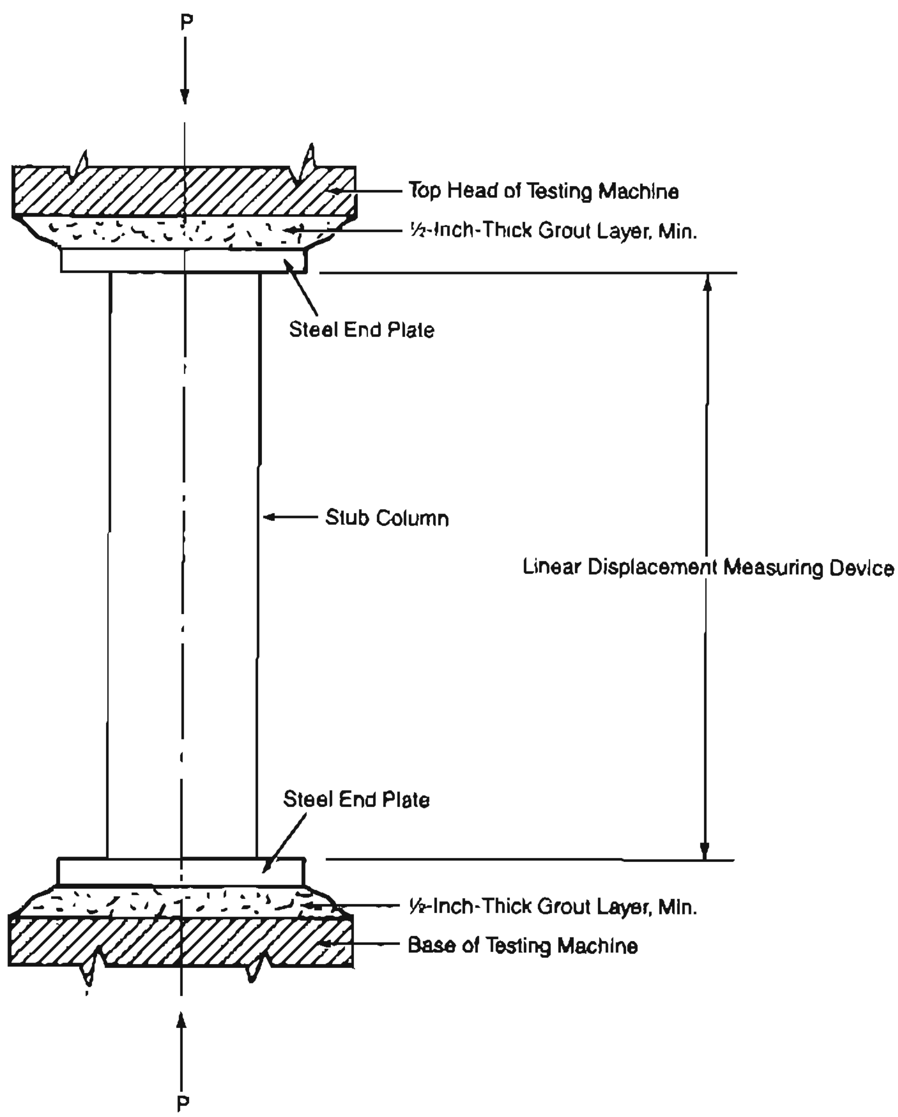


Figure 2 Test Setup

8.1.2 A ½-inch-thick layer of grout, similar to gypsum-based concrete capping compound used for fast setting, shall be placed between the stub-column endplates and the machine heads to facilitate aligning the test specimen (Figure 2).

8.2 When an axial compression load is applied to the test specimen as a result of grout expansion during curing, or if a small preload is purposely applied to ensure proper contact between the stub-column endplates and the machine heads, the load shall be treated as part of the applied test load.

8.3 The load increments applied during the test shall not exceed 10 percent of the estimated ultimate test load.

8.4 The maximum loading rate between load increments shall not exceed a corresponding applied stress rate of 3 kips per square inch of cross-sectional area per minute.

8.5 When axial shortening values are recorded, the following procedures shall be required:

(1) The change in the vertical distance between the inside surfaces of the endplates (Figure 2) shall be measured to the nearest 0.0001-inch at each load increment for each specimen.

(2) The load increments applied during the test shall be the same for each specimen within a test unit, with a variation not to exceed one percent.

9. Calculations

9.1 For a given test unit, all individual ultimate loads, P_u, derived from the stub-column tests shall be used to calculate the average ultimate load, P_{ua}. Similarly, all individual yield strengths, F_{yi}, derived from the tensile tests of the same unit shall be used to calculate the average yield stress of the same test unit, F_{ya}.

9.2 The effective areas A_{eua}, A_{eu}, and A_e shall be calculated as specified in Sections 9.3 through 9.6; however, the final value of these effective areas shall not exceed that of the minimum gross cross-sectional area, A.

9.3 For tests in which the length of the stub column does not exceed twenty times the minimum radius of gyration of the cross section, r, the average effective area at the ultimate load, A_{eua}, for a given test unit shall be calculated as

$$A_{eua} = P_{ua}/F_{ya}$$

9.4 For tests in which the length of the stub column exceeds twenty times the minimum radius of gyration of the cross section, the average effective area at the ultimate load shall be determined by iteration of the following equations:

$$A_{eua} = A_a - (A_a - P_{ua}/F_n)/(F_n/F_{ya})^n$$

where A_a is the average minimum gross area of the stub columns in the test unit, and F_n is the flexural or torsional-flexural buckling stress derived from Section C4 of the AISI Specification with K = 0.5 (using the average cross-sectional properties of the test unit). The exponent n is determined as follows:

$$n = A_{eua}/A_a$$

Assuming an initial value for n equal to less than 1.0, A_{eua} can be calculated from the first equation. Using this A_{eua} in the second equation will provide a new value for n. Repeating this process will lead to convergence of the above equations and an acceptable value of A_{eua} for one specific test unit.

9.5 The value of A_{eua} for a specific test unit shall be adjusted to A_{eu}, which is the effective cross-sectional area of a column at ultimate load with a nominal cross section of A and a specified minimum yield strength of F_y. The adjustment shall be performed in one or two steps as follows.

9.5.1 If the average area of the stub columns in the test unit, A_a, or the average base steel thickness, t_a, are diferent from the nominal area or thickness, respectively, the effective cross-sectional area at ultimate load shall be calculated as follows:

$$A_{eu} = A_{eua}(A/A_a)$$

or

$$A_{eu} = A_{eua}(t/t_a)$$

9.5.2 If the average yield strength of all stub columns in a test unit, F_{ya}, is different from the nominal yield strength, F_y, the effective cross-sectional area at ultimate load shall be the lower of the two values calculated as follows:

$$A_{eu} = A[1 - (1\text{-}A_{eua}/A)(F_y/F_{ya})]$$

or

$$A_{eu} = A_{eua}(F_{ya}/F_y)^{0.4}$$

9.5.3. If the average area and the minimum specified yield strength are different from the nominal values of a test unit, A_{eu} derived from the equation in Section 9.5.1 shall be used as A_{eua} in the equations of Section 9.5.2, which will lead to an acceptable value of A_{eu}.

9.6 The effective area at any working stress level, A_e, may be determined by

$$A_e = A - (A\text{-}A_{eu})(f/F_y)^n$$

9.7 For a series of sections, such as in a parameter study during which only one parameter (thickness, depth, width, yield strength, etc.) is changed, interpolations between test units, or extrapolations beyond test units, shall be acceptable as described in Appendix B.

9.8 Extrapolations beyond 20 percent of the extreme parameters tested shall not be permitted.

10. Report

10.1 Documentation—The report shall include a complete record of the sources and locations of all stub-column and tensile-test specimens and shall describe whether the specimens were taken from one or several columns, one or several production runs, coil stock, or other sources.

10.2 The documentation shall include all measurements taken for each stub-column test specimen, including (1) cross-section dimensions, (2) uncoated sheet thickness, (3) longitudinal yield strength, (4) end preparation procedure, (5) applicable material specification, and (6) test and evaluation procedure used.

10.3 The determination of the selected stub-column length shall be fully documented with appropriate calculations.

10.4 A description of the test setup—including the endplates, the grout layer used for alignment, and the instrumentation used to measure lateral displacements and axial shortening—shall be included.

10.5 The report shall include the load increments, rate of loading, and intermediate and ultimate loads for each stub column tested.

10.6 The report shall include complete calculations and results of the effective area, A_{eu}, for each test unit and calculations of A_e, if requested.

11. Precision

11.1 The following criteria shall be used to judge the acceptability of the test results.

11.1.1 Repeatability—Individual stub-column test results shall be considered suspect if they differ by more than 10 percent from the mean value for a test unit with at least three specimens.

11.1.2 Reproducability—The results of tests on stub-columns conducted at two or more laboratories should agree within ten (10) percent when adjusted for differences in cross sectional dimensions and yield strength.

REFERENCES

(1) T. Pekoz, "Development of a Unified Approach to the Design of Cold-Formed Steel Members, Committee of Sheet Steel Producers, American Iron and Steel Institute, 1000 16th Street, NW, Washington, DC 20036, 1986.

APPENDIX A

Use Of Axial Shortening Measurements In Design

A-1 Axial shortening measurements as part of thin-walled cold-formed steel stub-column tests may be used as an alternative method of determining the effective area of a column, A_e, at a certain design load or stress. This method provides a more accurate and less conservative alternative to design engineers to determine the effective area of a column section, A_e.
A-2 The calculations by this method shall be made separately for each stub-column specimen within a test unit. This shall result in a total of j calculations as a result of a total of j load-displacement tests for each test unit.
A-3 For a given specimen the effective area at ultimate load, A_{eu}, shall be calculated from Section 9.3 or 9.4 letting $A_{eus} = A_{eu}$, $A_s = A$, $F_{ys} = F_y$, and $P_{us} = P_u$.

A-3.1 Calculations at each load-displacement reading, i, shall be conducted according to the following procedure; however, at zero load, the effective area, A_e, shall be equal to the minimum gross cross-sectional area, A. This provides results for the effective area at each load point:

(1) Starting with the lowest load-displacement reading, the effective area, A_i, and the assumed uniformly distributed stress f_i, shall be calculated for each reading, i, from:

and $$A_{ei} = \frac{P_i D_u}{F_y D_i}$$

$$f_i = F_y D_i / D_u$$

where D_i an D_u are the axial shortening at loads P_i and P_u, respectively.

(2) If A_{ei} calculated is greater than A, A_{ei} shall be set equal to A.

(3) If A_{ei} calculated is less than A, A_{ei} shall be as calculated, and f_o, the stress above which the section is not fully effective, shall be set equal to f_{i-1}, as calculated for the previous load-displacement reading.

A.3.2 For specimens within a test unit, the lowest A_{ej} values shall be used for further evaluations.

A-4 For any load that causes a stress f higher than f_o, an exponential equation may be developed as follows.:

$$A_c = A[1-(1-A_{eu}/A)\,(f-f_o)/F_y-f_o)]^b$$

where $$b = \frac{\sum_{i=1}^{j} (X_i)(Y_i) - (a)\sum_{i=1}^{j} (X_i)}{\sum_{i=1}^{j} (X_i)^2}$$

and $$X = \ln[(f_i-f_o)/(F_y-f_o)]$$

$$Y_i = \ln(1-A'_{ei}/A)$$

$$a = \ln(1-A_{eu}/A)$$

and ln designates the natural logarithm.

A-5 If the effective areas for a section with specified dimensions and minimum yield strength are desired, which are different from the tested specimens, the Aeu and Aei values calculated under Section A-3 shall be normalized to the specified parameters according to Section 9.5 before the curve-fitting procedure of Section A-4 is employed.

A-6 All calculations pertaining to this procedure shall be included in the report, as discussed in Section 10.

APPENDIX B

Parametric Studies

B-1 For parametric studies intended to develop the effective area for a series of sections with the same basic cross section (either C, U, H, or any other shape) and the same hole pattern, but with one or more changing parameters, the required number of test units may be less than the sum of all sections with different geometries and yield strengths.

B-1.1 For a series of sections with three different values for one parameter only (dimension or nominal yield strength), at least two test units shall be chosen to include the minimum and the maximum value of the changing parameter. For the third value, A_{eu} may be interpolated according to Section B-2.

B-1.2 If more than three different values for one parameter are included in a series of sections, additional units with intermediate values shall be tested such that the ratio of the changing values in adjacent units is not greater than 1.5 or be less than 0.67. For intermediate values of the changing parameter, A_{eu} may be interpolated according to Section B-2.

B-1.3 For a series of sections with the same basic cross section that includes different values for several parameters (dimensions and/or yield strength), an appropriate factorial of test units shall be established by the responsible professional engineer in accordance with the guidelines for changes in an individual parameter, and in compliance with responsible code authorities. Interpolations and extrapolations may be made as mutually agreeable, following the general guidelines set forth in Section B-2 for changes of one parameter only.

B-1.4 For a section that falls outside a series of tested members with the same basic cross section, A_{eu} may be extrapolated provided the changing parameter does not exceed a value of 20 percent below or above the respective minimum or maximum values tested in the series.

B-2 Interpolations and extrapolations are allowed as part of a parametric study, and as defined under B-1.

B-2.1 For a section with a thickness different from the thicknesses tested, but with identical overall nominal cross-sectional dimensions and minimum specified yield strength, A_{eu} for a thickness t and an area A may be calculated provided t does not exceed the limits described under Section B-1.2 and B-1.4. Under these conditions, A_{eu} may be determined by interpolation or extrapolation from the results of the nearest two test units with thicknesses t_1 and t_2, respectively:

$$A_{eu} = A[A_{eu1}/A_1 + (A_{eu2}/A_2 - A_{eu1}/A_1)(t_1-t)/(t_1-t_2)]$$

where A_1 and A_2 are the minimum gross cross-sectional areas, and A_{eu1} and A_{eu2} are the nominal effective cross-sectional areas for Test Units 1 and 2, respectively.

B-2.2. For a section with a yield strength different from the yield strengths tested, but with identical cross-sectional dimensions, A_{eu} for a yield strength F_y may be calculated provided F_y does not exceed the limits described under Section B-1.2 and B-1.4. Under these conditions, A_{eu} may be determined by interpolation or extrapolation from the results of the nearest two test units with yield strengths F_{y1} and F_{y2}, and with effective areas A_{eu1} and A_{eu2}, respectively:

$$A_{eu} = A[A_{eu1}/A_1 + (A_{eu2}/A_2-A_{eu1}/A_1)(F_{y1}-F_y)/(F_{y1}-F_{y2})]$$

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AISI
SG-673
1186-5M-RI
Printed in U.S.A.