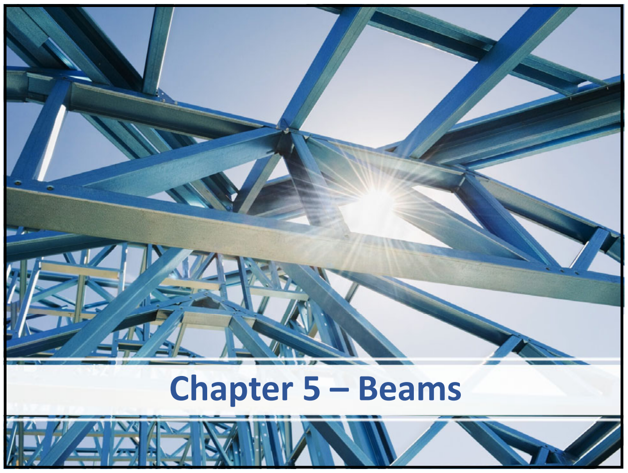

Chapter 5 – Beams

1

Chapter 5 – Compact Shapes

- A beam can fail by reaching M_p and becoming fully plastic, or it can fail by:
  1. ***lateral-torsional buckling*** (LTB), elastically or inelastically;
  2. ***flange local buckling*** (FLB), elastically or inelastically; or
  3. ***web local buckling*** (WLB), elastically or inelastically.
- If the ***maximum bending stress*** is less than the proportional limit when buckling occurs, the failure is said to be ***elastic***.
- Otherwise, it is ***inelastic***

2

Chapter 5 – Compact Shapes

- First, we will categorie beams as ***compact, noncompact, or slender.*** Then determine the moment resistance based on the degree of lateral support.
- The discussion in this section applies to two types of ***beams***:
  1. ***hot-rolled I shapes*** bent about the strong axis and loaded in the plane of the weak axis, and
  2. ***channels*** bent about the strong axis and either loaded through the shear center or restrained against twisting.
- The ***shear center*** is the point on the cross section through which a transverse load must pass if the beam is to bend ***without twisting***.

3

Chapter 5 – Compact Shapes

- Emphasis will be on ***I*** shapes.
- ***C***-shapes are different only in that the width-to-thickness ratio of the flange is b_f/t_f rather than $b_f/2t_f$
- We begin with ***compact*** shapes, defined as those whose webs are continuously connected to the flanges

4

Chapter 5 – Compact Shapes

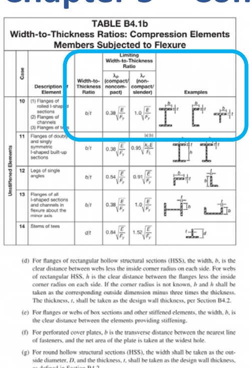
TABLE B4.1b
Width-to-Thickness Ratios: Compression Elements Members Subjected to Flexure

$$\frac{b_f}{2t_f} = 0.38\sqrt{\frac{E}{F_y}}$$

5

Chapter 5 – Compact Shapes

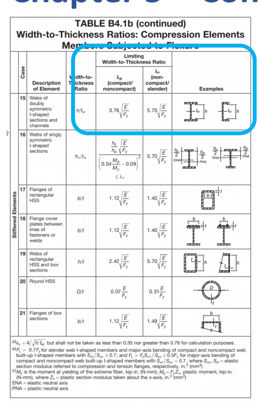
TABLE B4.1b (continued)
Width-to-Thickness Ratios: Compression Elements Members Subjected to Flexure

$$\frac{h}{t_w} = 3.76\sqrt{\frac{E}{F_y}}$$

6

Chapter 5 – Compact Shapes

- The web criterion is met by all standard ***I*** and ***C*** shapes listed in the Manual for $F_y \leq 70$ *ksi*.
- In most cases, ***only the flange ratio*** needs to be checked.
- Note, built-up welded ***I*** shapes can have noncompact or slender webs.
- Most shapes will also satisfy the flange requirement and will therefore be classified as ***compact***.

7

Chapter 5 – Compact Shapes

- The ***noncompact*** shapes are identified in the dimensions and properties table with a footnote (***footnote f***).
- Note that compression members have different criteria than flexural members, so a shape could be ***compact*** for flexure but ***slender*** for compression.
- As discussed in Chapter 4, shapes with slender compression elements are identified with a footnote (***footnote c***).

8

Chapter 5 – Compact Shapes

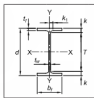

Table 1-1 (continued)
W-Shapes
Dimensions

Shape	Area, A	Depth, d		Web Thickness, t_w		Web $\frac{t_w}{2}$	Flange Width, b_f		Flange Thickness, t_f		Distance k_{des}	Distance k_{det}	k_1	T	Workable Gage
	in.^{2}	in.		in.		in.	in.		in.		in.	in.	in.	in.	in.
W14×132	38.8	14.7	14⅝	0.645	⅝	5/16	14.7	14¾	1.03	1	1.63	2 5/16	1 9/16	10	5½
×120	35.3	14.5	14½	0.590	9/16	5/16	14.7	14⅝	0.940	15/16	1.54	2¼	1½		
[illegible]	32.0	14.3	14⅜	0.525	½	¼	14.6	14⅝	0.860	⅞	1.46	2 3/16	1½		
×99[f]	29.1	14.2	14⅛	0.485	½	¼	14.6	14⅝	0.780	¾	1.38	2 1/16	1 7/16		
×90[f]	26.5	14.0	14	0.440	7/16	¼	14.5	14½	0.710	11/16	1.31	2	1 7/16		
W14×82	24.0	14.3	14¼	0.510	½	¼	10.1	10⅛	0.855	⅞	1.45	1 11/16	1 1/16	10⅞	5½
×74	21.8	14.2	14⅛	0.450	7/16	¼	10.1	10⅛	0.785	13/16	1.38	1⅝	1 1/16		
×68	20.0	14.0	14	0.415	7/16	¼	10.0	10	0.720	¾	1.31	1 9/16	1 1/16		
×61	17.9	13.9	13⅞	0.375	⅜	3/16	10.0	10	0.645	⅝	1.24	1½	1		
W14×53	15.6	13.9	13⅞	0.370	⅜	3/16	8.06	8	0.660	11/16	1.25	1½	1	10⅞	5½
[illegible]	14.1	13.8	13¾	0.340	5/16	3/16	8.03	8	0.595	⅝	1.19	1 7/16	1		
×43[c]	12.6	13.7	13⅝	0.305	5/16	3/16	8.00	8	0.530	½	1.12	1⅜	1		

[c] Shape is slender for compression with $F_y = 50$ ksi.
[f] Shape exceeds the compact limit for flexure with $F_y = 50$ ksi.
[g] The actual size, combination, and orientation of fastener components should be compared with the geometry of the cross section to ensure compatibility.
[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1d.

9

Chapter 5 – Compact Shapes

- If the beam is ***compact*** and has ***continuous lateral support***, or if the unbraced length is very short, the nominal moment strength, M_n, is the full plastic moment capacity of the shape, M_p.
- For members with ***inadequate lateral support***, the moment resistance is limited by the ***lateral-torsional buckling strength***, either inelastic or elastic.

10

Chapter 5 – Compact Shapes

- The first category, ***laterally supported compact beams***, is quite common and is the simplest case.
- For a doubly symmetric, compact ***I***- or ***C-shaped*** section bent about its major axis, **AISC F2.1** gives the nominal strength as:

$$M_n = M_p = F_y Z_x \qquad \textbf{\textit{AISC Equation F2-1}}$$

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Chapter 5 – Compact Shapes

16.1-50

CHAPTER F

DESIGN OF MEMBERS FOR FLEXURE

This chapter applies to members subjected to simple bending about one principal axis. For simple bending, the member is loaded in a plane parallel to a principal axis that passes through the shear center or is restrained against twisting at load points and supports.

The chapter is organized as follows:

- F1. General Provisions
- F2. Doubly Symmetric Compact I-Shaped Members and Channels Bent About Their Major Axis
- F3. Doubly Symmetric I-Shaped Members with Compact Webs and Noncompact or Slender Flanges Bent About Their Major Axis
- F4. Other I-Shaped Members with Compact or Noncompact Webs Bent About Their Major Axis
- F5. Doubly Symmetric and Singly Symmetric I-Shaped Members with Slender Webs Bent About Their Major Axis
- F6. I-Shaped Members and Channels Bent About Their Minor Axis
- F7. Square and Rectangular HSS and Box Sections
- F8. Round HSS
- F9. Tees and Double Angles Loaded in the Plane of Symmetry
- F10. Single Angles
- F11. Rectangular Bars and Rounds
- F12. Unsymmetrical Shapes
- F13. Proportions of Beams and Girders

User Note: For cases not included in this chapter, the following sections apply:
- Chapter G Design provisions for shear
- H1–H3 Members subjected to biaxial flexure or to combined flexure and axial force
- H3 Members subjected to flexure and torsion
- Appendix 3 Members subjected to fatigue

For guidance in determining the appropriate sections of this chapter to apply, Table User Note F1.1 may be used.

Chap. F.] DESIGN OF MEMBERS FOR FLEXURE 16.1-51

TABLE USER NOTE F1.1
Selection Table for the Application of Chapter F Sections

Section in Chapter F	Cross Section	Flange Slenderness	Web Slenderness	Limit States
F2		C	C	Y, LTB
F3		NC, S	C	LTB, FLB
F4		C, NC, S	C, NC	CFY, LTB, FLB, TFY
F5		C, NC, S	S	CFY, LTB, FLB, TFY
F6		C, NC, S	NA	Y, FLB
F7		C, NC, S	C, NC, S	Y, FLB, WLB, LTB
F8		NA	NA	Y, LB
F9		C, NC, S	NA	Y, LTB, FLB, WLB
F10		NA	NA	Y, LTB, LLB
F11		NA	NA	Y, LTB
F12	Unsymmetrical shapes, other than single angles	NA	NA	All limit states

C = compact; CFY = compression flange yielding; FLB = flange local buckling; LB = local buckling; LLB = leg local buckling; LTB = lateral-torsional buckling; NC = noncompact; S = slender; TFY = tension flange yielding; WLB = web local buckling; Y = yielding; NA = not applicable

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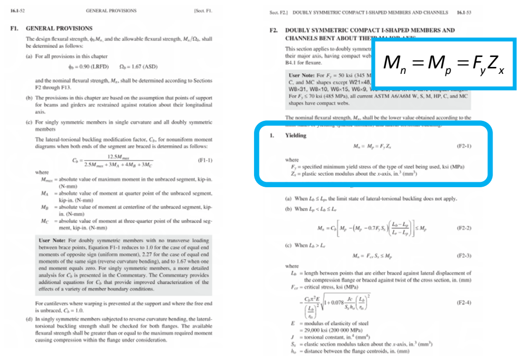
Chapter 5 – Compact Shapes



16.1-52 GENERAL PROVISIONS [Sect. F1.

F1. GENERAL PROVISIONS

The design flexural strength, $\phi_b M_n$, and the allowable flexural strength, M_n/Ω_b, shall be determined as follows:

(a) For all provisions in this chapter

$\phi_b = 0.90$ (LRFD) $\Omega_b = 1.67$ (ASD)

and the nominal flexural strength, M_n, shall be determined according to Sections F2 through F13.

(b) The provisions in this chapter are based on the assumption that points of support for beams and girders are restrained against rotation about their longitudinal axis.

(c) For singly symmetric members in single curvature and all doubly symmetric members

The lateral-torsional buckling modification factor, C_b, for nonuniform moment diagrams when both ends of the segment are braced is determined as follows:

$$C_b = \frac{12.5M_{max}}{2.5M_{max} + 3M_A + 4M_B + 3M_C} \quad \text{(F1-1)}$$

where

M_{max} = absolute value of maximum moment in the unbraced segment, kip-in. (N-mm)

M_A = absolute value of moment at quarter point of the unbraced segment, kip-in. (N-mm)

M_B = absolute value of moment at centerline of the unbraced segment, kip-in. (N-mm)

M_C = absolute value of moment at three-quarter point of the unbraced segment, kip-in. (N-mm)

User Note: For doubly symmetric members with no transverse loading between brace points, Equation F1-1 reduces to 1.0 for the case of equal end moments of opposite sign (uniform moment), 2.27 for the case of equal end moments of the same sign (reverse curvature bending), and to 1.67 when one end moment equals zero. For singly symmetric members, a more detailed analysis for C_b is presented in the Commentary. The Commentary provides additional equations for C_b that provide improved characterization of the effects of a variety of member boundary conditions.

For cantilevers where warping is prevented at the support and where the free end is unbraced, $C_b = 1.0$.

(d) In singly symmetric members subjected to reverse curvature bending, the lateral-torsional buckling strength shall be checked for both flanges. The available flexural strength shall be greater than or equal to the maximum required moment causing compression within the flange under consideration.

Sect. F2.] DOUBLY SYMMETRIC COMPACT I-SHAPED MEMBERS AND CHANNELS 16.1-53

F2. DOUBLY SYMMETRIC COMPACT I-SHAPED MEMBERS AND CHANNELS BENT ABOUT THEIR MAJOR AXIS

This section applies to doubly symmetric ... their major axis, having compact web ... B4.1 for flexure.

$$M_n = M_p = F_y Z_x$$

User Note: For F_y = 50 ksi (345 M... C, and MC shapes except W21×48, ... W8×31, W8×10, W6×15, W6×9, ... For $F_y \le 70$ ksi (485 MPa), all current ASTM A6/A6M W, S, M, HP, C, and MC shapes have compact webs.

The nominal flexural strength, M_n, shall be the lower value obtained according to the ...

1. Yielding

$$M_n = M_p = F_y Z_x \quad \text{(F2-1)}$$

where

F_y = specified minimum yield stress of the type of steel being used, ksi (MPa)

Z_x = plastic section modulus about the x-axis, in.³ (mm³)

(a) When $L_b \le L_p$, the limit state of lateral-torsional buckling does not apply.

(b) When $L_p < L_b \le L_r$

$$M_n = C_b\left[M_p - \left(M_p - 0.7F_y S_x\right)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \le M_p \quad \text{(F2-2)}$$

(c) When $L_b > L_r$

$$M_n = F_{cr} S_x \le M_p \quad \text{(F2-3)}$$

where

L_b = length between points that are either braced against lateral displacement of the compression flange or braced against twist of the cross section, in. (mm)

F_{cr} = critical stress, ksi (MPa)

$$= \frac{C_b \pi^2 E}{\left(\frac{L_b}{r_{ts}}\right)^2}\sqrt{1 + 0.078\frac{Jc}{S_x h_o}\left(\frac{L_b}{r_{ts}}\right)^2} \quad \text{(F2-4)}$$

E = modulus of elasticity of steel = 29,000 ksi (200 000 MPa)

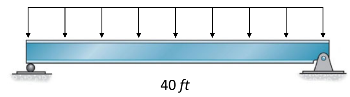
J = torsional constant, in.⁴ (mm⁴)

S_x = elastic section modulus taken about the x-axis, in.³ (mm³)

h_o = distance between the flange centroids, in. (mm)

Chapter 5 – Compact Shapes

- **Example 5.3**: The beam shown is a **W18 x 60** of **A992** steel with F_y = 50 *ksi*; F_u = 65 *ksi*. It supports a reinforced concrete floor slab that provides continuous lateral support of the compression flange.
- The service dead load is 600 *lb/ft*. This load is superimposed on the beam; it does not include the beam's weight. The service live load is 800 *lb/ft*.
- Does this beam have adequate moment strength?



Chapter 5 – Compact Shapes

- **Example 5.3**: Check for compactness.

From Table 1-1 (**1**-23) for a **W18 x 60**

Table 1-1 (continued)
W-Shapes
Properties
W18–W16

Nominal Wt.	Compact Section Criteria		Axis X-X				Axis Y-Y				r_{ts}	h_o	Torsional Properties	
	$\frac{b_f}{2t_f}$	$\frac{h}{t_w}$	I	S	r	Z	I	S	r	Z			J	C_w
lb/ft			in.⁴	in.³	in.	in.³	in.⁴	in.³	in.	in.³	in.	in.	in.⁴	in.⁶
130	4.65	23.9	2460	256	8.03	290	278	49.9	2.70	76.7	3.13	18.1	14.5	22700
119	5.31	24.5	2190	231	7.90	262	253	44.9	2.69	69.1	3.13	17.9	10.6	20300
106	5.96	27.2	1910	204	7.84	230	220	39.4	2.66	60.5	3.10	17.8	7.48	17400
97	6.41	30.0	1750	188	7.82	211	201	36.1	2.65	55.3	3.08	17.7	5.86	15800
86	7.20	33.4	1530	166	7.77	186	175	31.6	2.63	48.4	3.05	17.6	4.10	13600
76	8.11	37.8	1330	146	7.73	163	152	27.6	2.61	42.2	3.02	17.5	2.83	11700
71	4.71	32.4	1170	127	7.50	146	60.3	15.8	1.70	24.7	2.05	17.7	3.49	4700
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
60	5.44	38.7	984	108	7.47	123	50.1	13.3	1.68	20.6	2.02	17.5	2.17	3850
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
50	6.57	45.2	800	88.9	7.38	101	40.1	10.7	1.65	16.6	1.98	17.4	1.24	3040

$$\frac{b_f}{2t_f} = 5.44$$

$$\frac{h}{t_w} = 38.7$$

$$Z_x = 123in^3$$

Chapter 5 – Compact Shapes

- **Example 5.3**: Check for compactness.

$$\frac{b_f}{2t_f} = 5.44 \qquad 0.38\sqrt{\frac{E}{F_y}} = 0.38\sqrt{\frac{29{,}000ksi}{50ksi}} = 9.15$$

$$\frac{b_f}{2t_f} < 0.38\sqrt{\frac{E}{F_y}}$$ The flange is ***compact***

$$\frac{h}{t_w} = 38.7 \qquad 3.76\sqrt{\frac{E}{F_y}} = 3.76\sqrt{\frac{29{,}000ksi}{50ksi}} = 90.55$$

$$\frac{h}{t_w} < 3.76\sqrt{\frac{E}{F_y}}$$ The web is ***compact***

The web is ***compact*** for all shapes in the Manual for $F_y \le 70$ *ksi*

Chapter 5 – Compact Shapes

- **Example 5.3**: Because the beam is compact and laterally supported, the nominal flexural strength is:

$$M_n = M_p = F_y Z_x = 50ksi\left(123in^3\right) = 6{,}150.0k\,in = 512.5k\,ft$$

For a simply supported, uniformly loaded beam, the maximum bending moment occurs at midspan and is equal to:

$$M_{max} = \frac{wL^2}{8}$$ **AISC Table 3-22** (**3**-215)

$$M_D = \frac{(0.6k/ft + 0.06k/ft)(40ft)^2}{8} = 132.0k\,ft$$

$$M_L = \frac{(0.8k/ft)(40ft)^2}{8} = 160.0k\,ft$$

Chapter 5 – Compact Shapes

- **Example 5.3**: The dead load is less than 8 times the live load, so load combination 2 controls:

$$M_u = 1.2M_D + 1.6M_L = 1.2(132.0kft) + 1.6(160.0kft)$$
$$= 414.4k\,ft$$

The design strength is $\phi_b M_n = 0.9(512.5k\,ft) = 461.25k\,ft$

$\phi_b M_n > M_u$ **OK**

The design moment exceeds the factored-load moment, so the **W18 x 60** is satisfactory.

Chapter 5 – Compact Shapes

Let's work on some problems

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Chapter 5 – Beams

Any questions?

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