


AISC Night School

Basic Steel Design -- Session 4: Bending Members
Louis F. Geschwindner





Smarter.
Stronger.
Steel.



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Session Description

22.4 Bending Members February 25, 2020

This session will discuss the design of structural steel beams and application of Chapter F of the AISC Specification. The session will review plastic vs. elastic moment strength and the various limit states of bending members. The lecture will address C_b , the lateral-torsional buckling modification factor for non-uniform moments and its effect on beam designs. The session will also review the design of single angles and WT shapes. Design examples will be presented.



Learning Objectives:

- List the AISC Specification requirements for the design of flexural members.
- List the applicable limit states for the design of flexural members to ensure a safe design.
- Define the lateral-torsional buckling modification factor, C_b , for non-uniform moment diagrams, and how it affects beam design.
- List the design steps for a structural T-shape in flexure.



Basic Steel Design: A review of the principles of steel design according to ANSI/AISC 360-16

Night School 22 Lesson 4 Bending Members



Smarter.
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Lesson 4 – Bending

- Bending Members
 - Plastic vs. elastic moment strength
 - Lateral-torsional buckling
 - Local buckling
 - Beam design
 - Tees and double angles
 - Shear strength



4.9

Bending Members

B3.1. For LRFD, design shall be performed in accordance with:

Required Strength \leq Available Strength

$$R_u \leq \phi R_n \quad (\text{B3-1})$$

where

R_u = required strength (LRFD) defined in Chapter C

R_n = nominal strength specified in Chapters F & G

ϕ = resistance factor specified in Chapters F & G

ϕR_n = design strength = resistance factor (nominal strength)



4.10

Bending Members

B3.2. For ASD, design shall be performed in accordance with:

Required Strength \leq Available Strength

$$R_a \leq R_n / \Omega \quad (\text{B3-2})$$

where

R_a = required strength (ASD) defined in Chapter C

R_n = nominal strength specified in Chapters F & G

Ω = safety factor specified in Chapters F & G

R_n / Ω = allowable strength = $\frac{\text{nominal strength}}{\text{safety factor}}$



4.11

Bending Members

F1. 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

$$\phi_b = 0.90 \text{ (LRFD)} \quad \Omega_b = 1.67 \text{ (ASD)}$$

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 are restrained against rotation about their longitudinal axis.



4.12

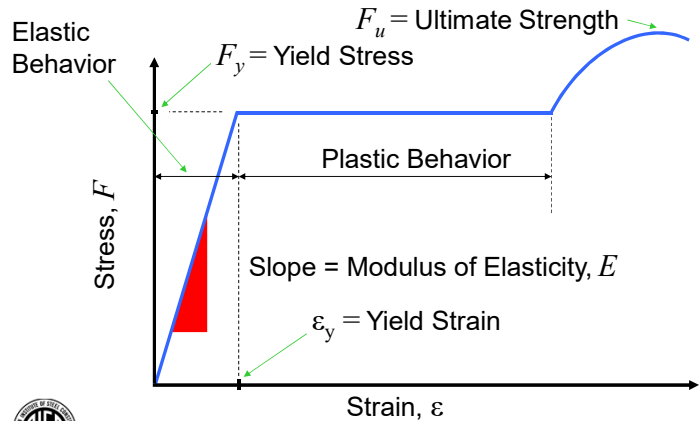
Bending Members

- Limit States
 - **Yielding**: as seen in the discussion of compression, yielding is the upper limit for all shapes.
 - **Lateral-torsional buckling**: a combination of lateral buckling and twist.
 - **Local buckling**: buckling of elements before they are able to reach yield.



4.13

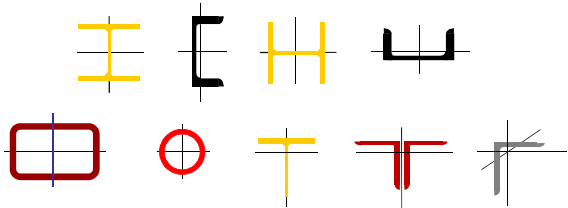
Steel as a Material



4.14

Bending Members

- Limit States
 - The buckling limit states influence strength differently depending on the shape



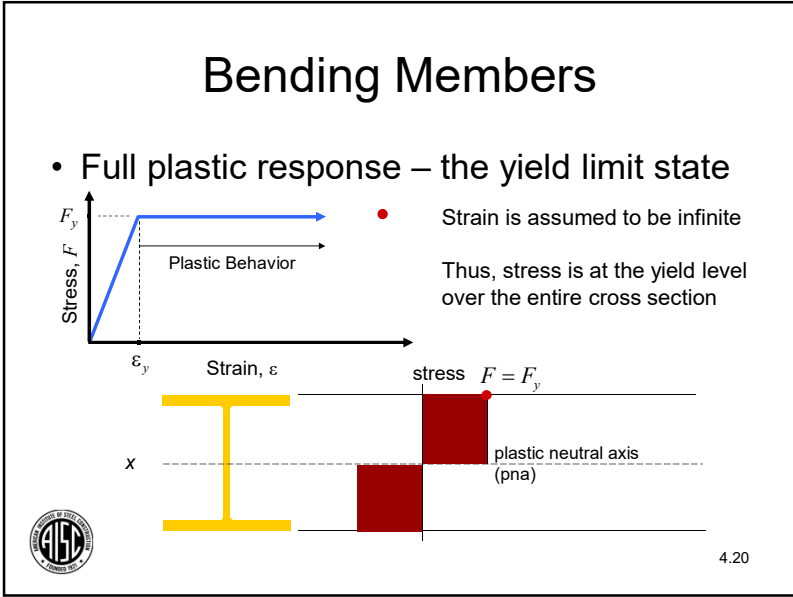
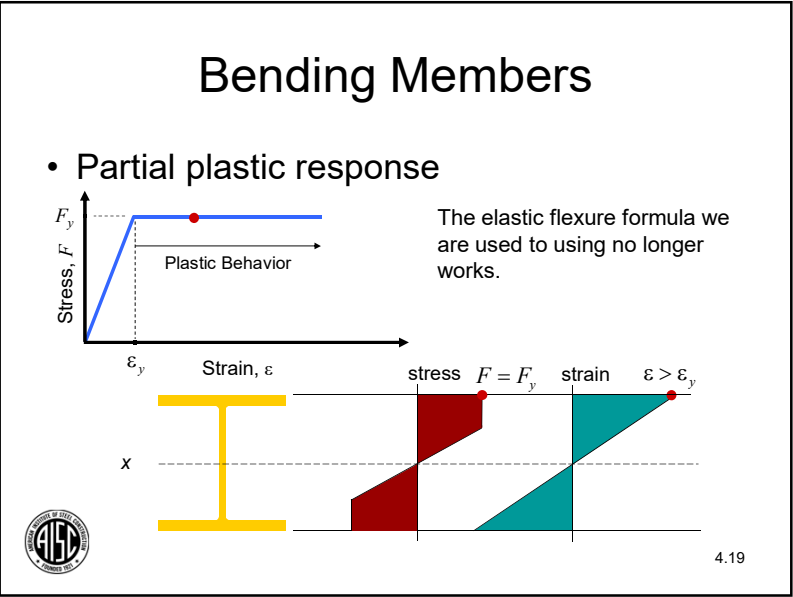
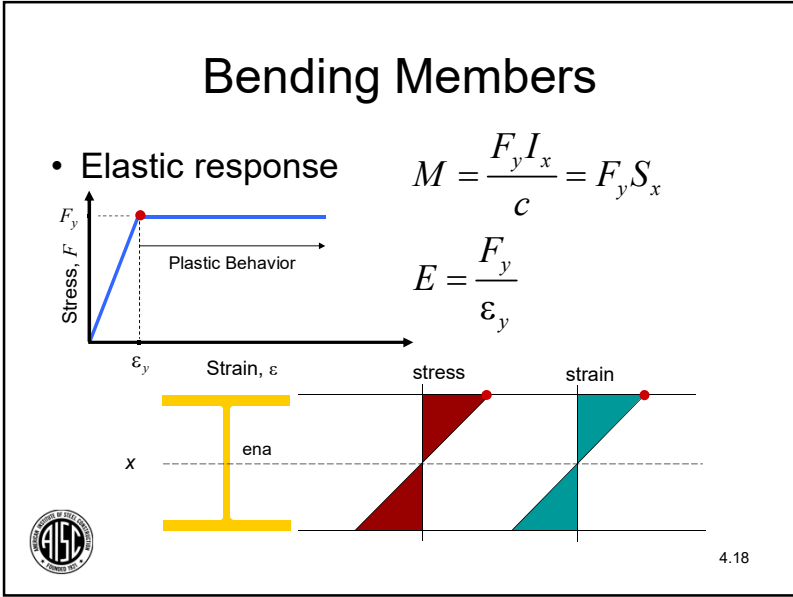
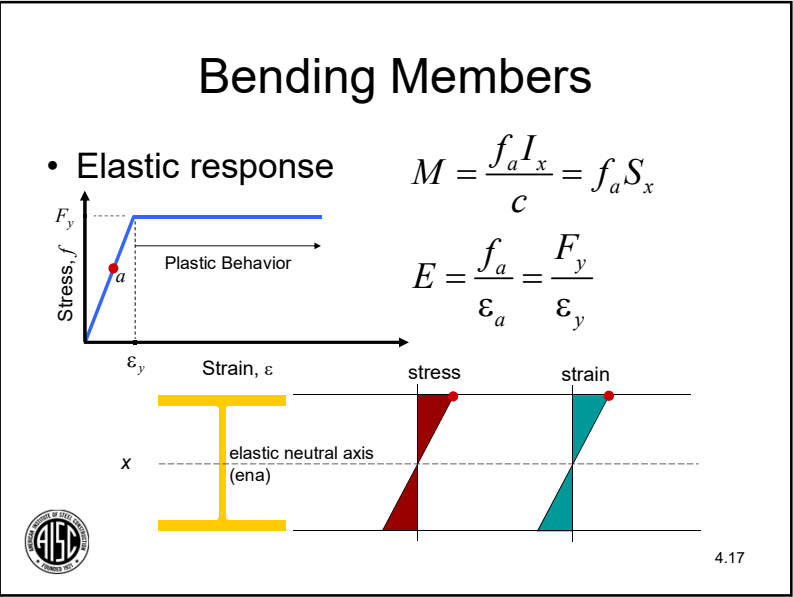
4.15

Bending Members

- First concentrate on doubly symmetric I-shaped members
 - Limit States
 - Yielding
 - Local Buckling
 - Lateral-Torsional Buckling

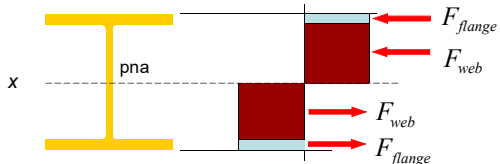


4.16



Bending Members

- Determine the force in the flange and web



$$F_{flange} = F_y A_{flange} = F_y b_f t_f$$

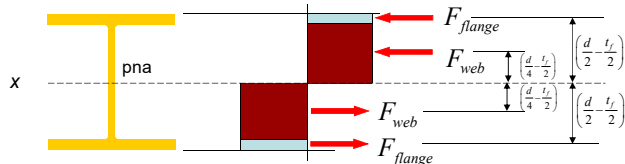
$$F_{web} = F_y \frac{A_{web}}{2} = F_y t_w \left(\frac{d}{2} - t_f \right)$$



4.21

Bending Members

- Take moments about the plastic neutral axis



$$M_p = 2 \left[F_y A_{flange} \left(\frac{d}{2} - \frac{t_f}{2} \right) + F_y \frac{A_{web}}{2} \left(\frac{d}{4} - \frac{t_f}{2} \right) \right]$$



4.22

Bending Members

- Simplifying

$$M_p = 2 \left[A_{flange} \left(\frac{d}{2} - \frac{t_f}{2} \right) + \frac{A_{web}}{2} \left(\frac{d}{4} - \frac{t_f}{2} \right) \right] F_y$$

$$= F_y Z$$

where Z = the plastic section modulus
= the moment of the area about pna



4.23

Bending Members

- How do we define the plastic neutral axis?

$$T = C$$

$$F_y A_T = F_y A_C$$

$$A_T = A_C$$

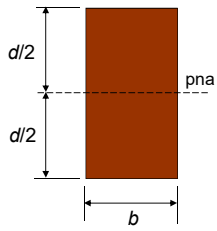
- Thus, the area above the pna must equal the area below the pna.



4.24

Bending Members

- For a rectangle the pna is in the middle.



$$I = \frac{bd^3}{12}$$

$$S = \frac{bd^2}{6}$$

$$Z = 2 \left(b \left(\frac{d}{2} \right) \frac{d}{4} \right) = \frac{bd^2}{4}$$

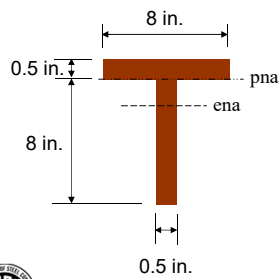
$$\beta = \frac{Z}{S} = \frac{\frac{bd^2}{4}}{\frac{bd^2}{6}} = 1.5 \quad \text{Shape Factor}$$



4.25

Bending Members

- For a shape not symmetric about the bending axis, $A_T = A_C$



$$A_{total} = 0.5(8) + 0.5(8) = 8.0 \text{ in.}^2$$

$$A_T = A_C = \frac{A_{total}}{2} = 4.0 \text{ in.}^2$$

Thus the pna is at the flange-stem juncture.

$$S = 9.39 \text{ in.}^3$$

$$Z = 17.0 \text{ in.}^3$$

$$\beta = \frac{17.0}{9.39} = 1.81$$



4.26

Bending Members

- The shape factor relates the plastic moment to the elastic moment.
- It illustrates the extra strength that is available if we consider the limit state of yielding rather than the elastic limit.
- Throughout the specification the shape factor is limited to a maximum of 1.6 in order to limit the strain to something less than the initiation of strain hardening.



4.27

Bending Members

- If a shape is capable of reaching the plastic moment without local buckling it is said to be a compact shape
 - Yielding is the upper limit on strength
 - However, lateral-torsional buckling based on unbraced length may still control strength

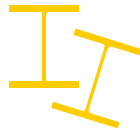
F2. User note: All current ASTM A6 W, S, M, C and MC shapes except W21x48, W14x99, W14x90, W12x65, W10x12, W8x31, W8x10, W6x15, W6x9, W6x8.5, and M4x6 have compact flanges for $F_y = 50$ ksi (345 MPa); all current ASTM A6 W, S, M, HP, C and MC shapes have compact webs at $F_y \leq 70$ ksi (485 MPa).



4.28

Bending Members

- Lateral-torsional buckling
 - Compression portion of the bending member tries to behave like a column but can't.
 - Tension region resists buckling down
 - Tension region also resists buckling laterally
 - Thus, the shape twists as it buckles laterally



4.29

Bending Members

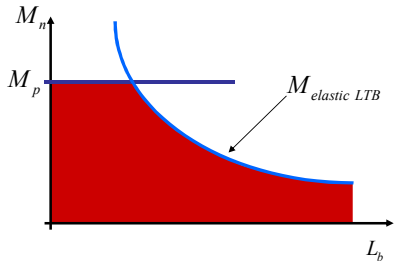
- To control lateral-torsional buckling the beam must be properly braced.
 - Supports must be restrained against twisting (a given)
 - Intermediate points along the span may have the compression flange braced against lateral translation.
 - Similar to column bracing but treated differently.
 - Intermediate points may be braced against twisting by torsional braces.
 - The distance between braced points is referred to as the unbraced length, L_b .



4.30

Bending Members

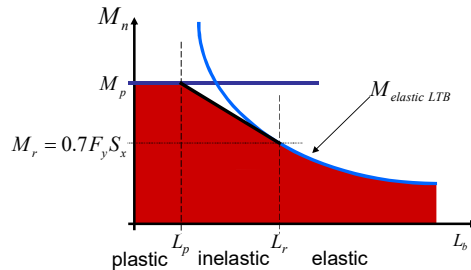
- If the member behaves either plastically or elastically



4.31

Bending Members

- But we know it will behave inelastically, just like columns thus, there is a transition



4.32

Lateral-Torsional Buckling

- If $L_b \leq L_p$, the limit state of yielding controls bending strength.

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




4.33

Based on an unbraced length less than or equal to L_p

Design Aid

Table 3-6 (continued)
Maximum Total Uniform Load, kips
W-Shapes



$F_y = 50$ ksi

Shape	W21<									
	57		55		50		48 ^f		44	
Design	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
6					316	474			290	435
7	342	513			314	471	288	433	272	408
8	322	484	312	468	274	413	285	398	238	358
9	286	430	279	420	244	367	235	354	212	318
10	257	387	251	373	220	330	212	318	190	286
11	234	352	229	344	200	300	193	289	173	260
12	215	323	210	315	183	275	176	265	159	239
13	198	298	193	291	169	254	163	245	146	220
14	184	276	180	270	157	236	151	227	136	204
15	172	258	168	252	146	220	141	212	127	191
16	161	242	157	236	137	206	132	199	119	179
17	151	228	148	222	129	194	125	187	112	168
18	143	215	140	210	122	185	119	177	106	159
19	136	204	132	199	116	174	111	168	100	151
20	129	194	126	189	110	165	106	159	95.2	143
21	123	184	120	180	105	157	101	152	90.7	136
22	117	176	114	172	99.8	150	96.3	145	86.6	130
23	112	168	109	164	95.5	143	92.1	138	82.8	124
24	107	161	105	159	91.5	138	88.2	133	79.3	119
25	103	155	101	151	87.8	132	84.7	127	76.2	114
26	99.0	149	96.7	145	84.4	127	81.6	122	73.2	110



4.34

Based on an unbraced length less than or equal to L_p

Design Aid

22	117	176	114	172	99.8	150	96.3	145	86.6	130	
23	112	168	109	164	95.5	143	92.1	138	82.8	124	
24	107	161	105	159	91.5	138	88.2	133	79.3	119	
25	103	155	101	151	87.8	132	84.7	127	76.2	114	
26	99.0	149	96.7	145	84.4	127	81.6	122	73.2	110	
27	95.4	143	93.1	140	81.3	122	78.4	118	70.5	106	
28	92.0	138	89.8	135	78.4	118	75.6	114	68.0	102	
29	88.8	133	86.7	130	75.7	114	73.0	110	65.7	98.7	
30	85.8	129	83.8	126	73.2	110	70.6	106	63.5	95.4	
32	80.5	121	78.6	118	68.6	103	66.2	99.5	59.5	89.4	
34	75.7	114	74.0	111	64.6	97.1	62.3	93.6	56.0	84.2	
36	71.5	108	69.9	106	61.0	91.7	58.0	89.4	52.9	79.5	
38	67.8	102	66.2	99.5	57.8	86.8	55.7	83.8	50.1	75.3	
40	64.4	96.8	62.9	94.5	54.9	82.5	52.9	79.6	47.6	71.6	
42	61.3	92.1	59.9	90.0	52.3	78.6	50.4	75.8	45.3	68.1	
44	58.5	88.0	57.2	85.9	49.9	75.0	48.1	72.3	43.3	65.0	
46	56.0	84.1	54.7	82.2	47.7	71.7	46.0	69.2	41.4	62.2	
48	53.6	80.6	52.4	78.9	45.7	68.5	44.1	66.3	39.7	59.6	
50	51.5	77.4	50.3	75.6	43.9	66.0	42.4	63.7	38.1	57.2	
52	49.5	74.4	48.4	72.7	42.2	63.5					
Beam Properties											
W_x/I_x	$\phi_b M_n$, kip-ft	2570	3870	2510	3780	2200	3300	2120	3180	1900	2860
M_p/I_x	$\phi_b M_p$, kip-ft	322	484	314	473	274	413	265	398	238	358
M_u/I_x	$\phi_b M_u$, kip-ft	194	291	192	289	165	248	162	244	143	214
BF/I_x	$\phi_b BF$, kips	13.4	20.3	10.8	16.3	12.1	18.3	9.89	14.8	11.1	16.8
W_x/S_x	$\phi_b W_x$, kips	171	256	156	234	158	237	144	216	145	217
Z_x , in ³		128	176	110	160	107	157	95.4			
L_p , ft		4.77	6.11	4.59	6.09	4.45					
L_b , ft		14.3	17.4	13.6	18.5	13.0					
ASD	LRFD	Shape does not meet compact limit for flexure with $F_y = 50$ ksi; tabulated values have been adjusted accordingly.									
$\Omega_b = 1.67$	$\phi_b = 0.90$	Notes: For beams laterally unsupported, see Table 3-10.									
$\Omega_b = 1.50$	$\phi_b = 1.00$	Available strength tabulated above heavy line is limited by available shear strength.									



4.35

Lateral-Torsional Buckling

- If $L_b > L_r$ the limit state of elastic lateral-torsional buckling controls

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

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



4.36

Lateral-Torsional Buckling

- If $L_p < L_b < L_r$ the limit state of inelastic lateral-torsional buckling controls
 - Represented by a straight line between M_p and the moment that can be reached elastically, assuming $0.3F_y$ is due to residual stresses.

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



4.37

Lateral-Torsional Buckling

- The dividing line between yielding and inelastic lateral-torsional buckling is given by L_p :

$$L_p = 1.76 r_y \sqrt{\frac{E}{F_y}} \quad (F2-5)$$



4.38

Lateral-Torsional Buckling

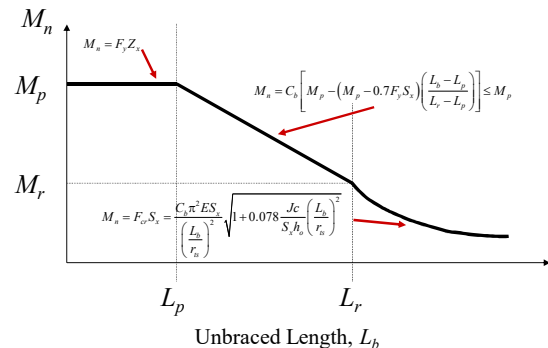
- The dividing line between inelastic and elastic lateral-torsional buckling is given by L_r .

$$L_r = 1.95 r_{ts} \frac{E}{0.7F_y} \sqrt{\frac{Jc}{S_x h_o} + \sqrt{\left(\frac{Jc}{S_x h_o} \right)^2 + 6.76 \left(\frac{0.7F_y}{E} \right)^2}} \quad (F2-6)$$



4.39

Lateral-Torsional Buckling



4.40

Lateral-Torsional Buckling

Rewrite Eq. F2-2 to simplify

$$M_n = C_b \left[M_p - (M_p - 0.7F_y S_x) \left(\frac{L_b - L_p}{L_r - L_p} \right) \right] \leq M_p$$

becomes

$$M_n = C_b \left[M_p - \frac{(M_p - 0.7F_y S_x)}{(L_r - L_p)} (L_b - L_p) \right] \leq M_p$$

Taking

$$BF = \frac{(M_p - 0.7F_y S_x)}{(L_r - L_p)}$$



4.41

Lateral-Torsional Buckling

• Thus,

$$M_n = C_b \left[M_p - BF(L_b - L_p) \right] \leq M_p$$

and the available strength is

$$\phi M_n = C_b \left[\phi M_p - \phi BF(L_b - L_p) \right] \quad (\text{LRFD})$$

$$\frac{M_n}{\Omega_b} = C_b \left[\frac{M_p}{\Omega_b} - \frac{BF}{\Omega_b} (L_b - L_p) \right] \quad (\text{ASD})$$



4.42

Design Aid

Tabulate the terms in these equations

Table 3-2 (continued)
W-Shapes
Selection by Z_x $F_y = 50$ ksi

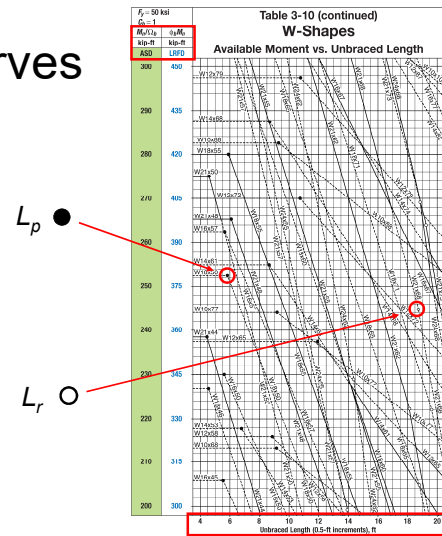
Shape	Z_x in. ³	M_{pr}/Ω_b		ϕM_{pr}		M_p/Ω_b		ϕM_p		BF/Ω_b		L_p ft	L_r ft	I_x in. ⁴	M_{pr}/Ω_b		ϕM_{pr}	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD							
W21-44	95.4	230	350	143	214	11.1	16.0	4.45	13.0	0.43	1.45	217						
W16-50	92.0	230	345	141	213	7.69	11.4	5.62	17.2	6.59	12.4	186						
W10-46	90.7	220	340	130	207	9.03	14.0	4.50	13.7	7.12	13.0	195						
W14-53	87.1	217	327	136	204	5.22	7.93	6.78	22.3	5.41	10.3	154						
W12-58	86.4	216	324	136	205	3.82	5.69	6.67	29.8	4.75	87.8	132						
W10-68	85.3	213	320	132	199	2.58	3.85	9.15	40.6	3.94	97.8	147						
W10-45	82.3	205	309	127	191	7.12	10.8	5.55	16.5	5.86	111	167						
W18-40	78.4	196	294	119	180	8.94	13.2	4.49	13.1	6.12	113	169						
W14-48	78.4	196	294	123	184	5.09	7.67	6.75	21.1	4.84	93.8	141						
W12-53	77.9	194	292	123	185	3.65	5.50	8.76	28.2	4.25	83.5	125						
W10-60	74.6	186	280	116	175	2.54	3.82	9.08	36.6	3.41	85.7	129						
W10-40	73.0	182	274	113	170	6.67	10.0	5.55	15.9	5.18	97.6	146						
W12-50	71.9	179	270	112	169	3.97	5.98	6.92	23.8	3.91	90.3	135						
W8-67	70.1	175	263	105	159	1.75	2.59	7.49	47.6	2.72	103	154						
W14-43	69.6	174	261	109	164	4.88	7.28	6.88	20.0	4.28	83.6	123						



4.43

Beam Curves

Another design aid that will assist in design considering unbraced length



4.44

Slender Elements

$$\lambda_f = \frac{b}{t} = \frac{b_f}{2} \left(\frac{1}{t_f} \right) = \frac{b_f}{2t_f}$$

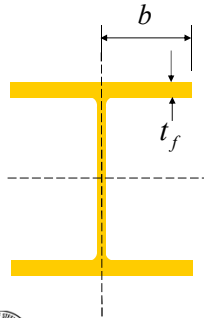


TABLE B4.1b
Width-to-Thickness Ratios: Compression Elements Members Subject to Flexure

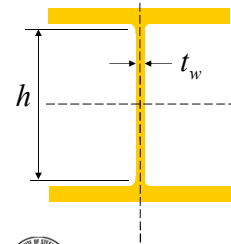
Case	Description of Element	Width-to-Thickness Ratio	Limiting Width-to-Thickness Ratio		Examples
			λ_p (compact/noncompact)	λ_r (noncompact/slender)	
Unstiffened Elements	10 Flanges of rolled I-shaped sections, channels, and tees	b/t	$0.38 \sqrt{\frac{E}{F_y}}$	$1.0 \sqrt{\frac{E}{F_y}}$	
	11 Flanges of doubly and singly symmetric I-shaped built-up sections	b/t	$0.38 \sqrt{\frac{E}{F_y}}$	$0.96 \sqrt{\frac{K_y E}{F_y}}$ (a) (b)	
	12 Legs of single angles	b/t	$0.54 \sqrt{\frac{E}{F_y}}$	$0.91 \sqrt{\frac{E}{F_y}}$	
	13 Flanges of all I-shaped sections and channels in flexure about the minor axis	b/t	$0.38 \sqrt{\frac{E}{F_y}}$	$1.0 \sqrt{\frac{E}{F_y}}$	
	14 Stems of tees	d/t	$0.84 \sqrt{\frac{E}{F_y}}$	$1.52 \sqrt{\frac{E}{F_y}}$	

4.45



Slender Elements

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



Stiffened Elements	15 Webs of doubly symmetric I-shaped sections and channels	h/t_w	$3.76 \sqrt{\frac{E}{F_y}}$	$5.70 \sqrt{\frac{E}{F_y}}$	
	16 Webs of singly symmetric I-shaped sections	h_w/t_w	$\frac{h_w}{0.64 M_p} \sqrt{\frac{E}{F_y}}$ (a) $(0.00 < \lambda < 1)$	$5.70 \sqrt{\frac{E}{F_y}}$	
	17 Flanges of rectangular HSS	b/t	$1.12 \sqrt{\frac{E}{F_y}}$	$1.40 \sqrt{\frac{E}{F_y}}$	
	Flange cover plates and diaphragm plates between lines of fasteners or welds	b/t	$1.12 \sqrt{\frac{E}{F_y}}$	$1.40 \sqrt{\frac{E}{F_y}}$	
	19 Webs of rectangular HSS and box sections	h/t	$2.42 \sqrt{\frac{E}{F_y}}$	$5.70 \sqrt{\frac{E}{F_y}}$	
20 Round HSS	D/t	$0.07 \frac{E}{F_y}$	$0.3 \frac{E}{F_y}$		
24 Flanges of box sections	b/t	$1.12 \sqrt{\frac{E}{F_y}}$	$1.40 \sqrt{\frac{E}{F_y}}$		

(a) $K_y = 4.0(h/t_w)$, shall not be taken less than 0.35 nor greater than 0.76 for calculation purposes.
 (b) $F_y = 0.7F_u$ for slender web I-shaped members and major-axis bending of compact and noncompact web built-up I-shaped members with $d_w/b_w \geq 0.7$; $F_y = F_u d_w/b_w \geq 0.65$ for major-axis bending of compact and noncompact web built-up I-shaped members with $S_u/S_m < 0.7$, where S_u , S_m = elastic section modulus referred to compression and tension flanges, respectively, in in^3 .
 (c) M_p is the moment at yielding of the extreme fiber, $M_p = F_y Z_x$, plastic bending moment, kip-in. (N-mm), where Z_x = plastic section modulus taken about x-axis, in³ (cm³).
 E = modulus of elasticity of steel = 29,000 ksi (200,000 MPa) CNA - elastic neutral axis PNA - plastic neutral axis
 F_y = specified minimum yield stress, ksi (MPa)

4.46



Flange Local Buckling

- Compact W-shape

$$\lambda = \frac{b_f}{2t_f} \leq \lambda_{pf} = 0.38 \sqrt{\frac{E}{F_y}}$$

- Noncompact W-shape

$$\lambda_{pf} = 0.38 \sqrt{\frac{E}{F_y}} < \frac{b_f}{2t_f} \leq \lambda_{rf} = 1.0 \sqrt{\frac{E}{F_y}}$$

4.47



Flange Local Buckling

- Slender W-shape

$$\lambda_{rf} = 1.0 \sqrt{\frac{E}{F_y}} < \frac{b_f}{2t_f}$$

4.48



Flange Local Buckling

Nominal Strength

- Compact $M_n = M_p = F_y Z_x$ (F2-1)

- Slender $M_n = \frac{0.9Ek_c S_x}{\lambda^2}$ (F3-2)

with $k_c = \frac{4}{\sqrt{h/t_w}}$ But shall not be taken less than 0.35 nor greater than 0.76 for calculation purposes



4.49

Flange Local Buckling

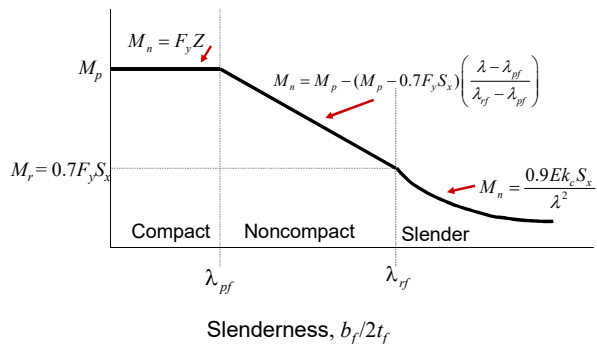
- Noncompact

$$M_n = M_p - (M_p - 0.7F_y S_x) \left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}} \right) \quad (F3-1)$$



4.50

Flange Local Buckling



4.51

Flange Local Buckling

- Limits W-shape, A992

$$\lambda_p = 0.38 \sqrt{\frac{E}{F_y}}$$

$$\lambda_p = 9.15$$

– Ten W-shapes have noncompact flange for A992

F3. User Note: The following shapes have noncompact flanges for $F_y = 50$ ksi (345 MPa): W21x48, W14x99, W14x90, W12x65, W10x12, W8x31, W8x10, W6x15, W6x9, W6x8.5, and M4x6. All other ASTM A6 W, S and M shapes have compact flanges for $F_y \leq 50$ ksi (345 MPa).



4.52

Flange Local Buckling

Table 1-1 (continued)
W-Shapes
Dimensions

Shape	Area, A in. ²	Depth, d in.	Web		Flange		Distance			Workable Gage in.								
			Thickness, t _w in.	t _w /2 in.	Width, b _f in.	Thickness, t _f in.	k _{flange} in.	k _{web} in.	γ in.									
W14x132	38.8	14.3	14.3	7.15	14.3	14.3	0.525	1/2	3/4	14.6	14.5%	0.860	7/8	1.46	2 1/16	1 1/2	0	5 1/2
W14x120	35.3	14.2	14.2	7.1	14.2	14.2	0.485	1/2	3/4	14.6	14.5%	0.780	3/4	1.38	2 1/16	1 7/8	0	5 1/2
W14x99	29.1	14.2	14.2	7.1	14.2	14.2	0.485	1/2	3/4	14.6	14.5%	0.780	3/4	1.38	2 1/16	1 7/8	0	5 1/2
W14x82	26.5	14.0	14.0	7.0	14.0	14.0	0.440	1/2	3/4	14.5	14 1/2	0.710	1 1/16	1.31	2	1 7/8	0	5 1/2
W14x74	24.0	14.3	14 1/4	7.15	14 1/4	14 1/4	0.510	1/2	3/4	10.1	10 7/8	0.855	7/8	1.45	1 11/16	1 7/8	10 7/8	5 1/2
W14x62	20.0	14.0	14.0	7.0	14.0	14.0	0.440	1/2	3/4	10.1	10 7/8	0.855	7/8	1.45	1 11/16	1 7/8	10 7/8	5 1/2
W14x54	17.9	14.0	14.0	7.0	14.0	14.0	0.440	1/2	3/4	10.1	10 7/8	0.855	7/8	1.45	1 11/16	1 7/8	10 7/8	5 1/2
W14x44	15.7	13.7	13 7/8	6.875	13 7/8	13 7/8	0.300	1/2	3/4	6.00	6	0.530	7/2	1.12	1 7/8	1	1	5 1/2

Note the footnote on the weight, f

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

4.53

Flange Local Buckling

Table 1-1 (continued)
W-Shapes
Properties

Nominal Wt. lb/ft	Compact Section Criteria			Axis X-X				Axis Y-Y				Torsional Properties			
	b _f /2t _f	t _w /t _f	d/t _w	I	S	r	Z	I	S	r	Z	r _p	h _b	J	C _w
				in. ⁴	in. ³	in.	in. ³	in. ⁴	in. ³	in.	in. ³	in.	in.	in. ⁴	in. ⁶
132	7.15	17.7	1530	1390	173	6.22	192	447	61.2	3.73	92.7	4.17	13.4	7.12	22700
120	7.80	19.3	1390	1240	173	6.22	192	447	61.2	3.73	92.7	4.17	13.4	7.12	20200
109	8.40	21.7	1240	1110	157	6.17	173	402	53.2	3.71	83.6	4.14	13.4	5.37	18000
99	10.2	25.9	999	999	143	6.14	157	362	49.9	3.70	75.6	4.10	13.3	4.06	16000
82	5.92	22.4	881	881	123	6.05	139	348	29.3	2.48	44.8	2.85	13.4	5.07	6710
74	6.34	21.4	800	800	123	6.05	139	348	29.3	2.48	44.8	2.85	13.4	5.07	6710
62	6.34	21.4	720	720	123	6.05	139	348	29.3	2.48	44.8	2.85	13.4	5.07	6710
54	6.34	21.4	640	640	123	6.05	139	348	29.3	2.48	44.8	2.85	13.4	5.07	6710

Note that (b_f/2t_f) exceeds 9.15

4.54

Flange Local Buckling Impact on Lateral-Torsional Buckling

Table 3-2 (continued)
W-Shapes
Selection by Z_x

F_y = 50 ksi

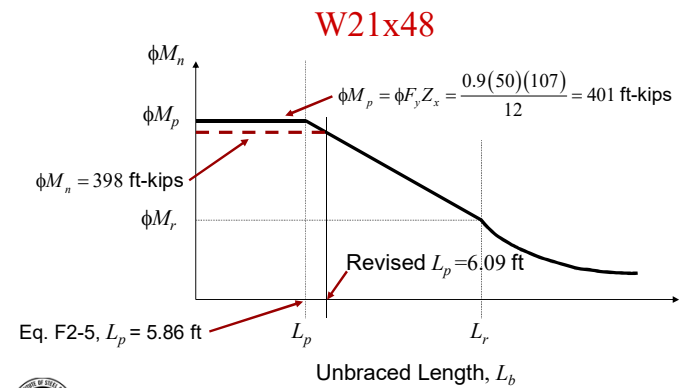
Shape	Z _x in. ³	M _{px} /A _g		M _{py} /A _g		M _{px} /A _g		M _{py} /A _g		L _p ft	L _r ft	L _x in. ⁴	M _{px} /A _g	M _{py} /A _g
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD					
W24x62	153	282	374	229	344	16.1	24.1	4.97	14.4	1660	204	306		
W16x77	150	374	563	234	352	7.34	11.1	8.72	27.8	1110	150	225		
W12x14	100	209	400	170	230	3.09	3.00	10.7	31.0	397	100	109		
W21x48	107	265	398	162	244	9.89	14.8	6.09	16.5	959	144	216		
W16x57	165	306	391	161	242	7.98	12.0	5.65	18.3	758	141	212		
W14x61	109	254	382	161	242	4.83	7.60	8.65	27.5	640	104	156		
W18x50	101	252	370	155	233	8.76	13.2	5.83	16.9	800	128	192		
W10x77	97.6	244	366	150	225	2.60	3.00	9.18	45.3	455	112	169		
W12x65	96.8	237	356	154	231	3.58	5.39	11.9	35.1	533	94.4	142		

Not compact

Shape exceeds compact limit for flexure with F_y = 50 ksi. Tabulated values have been adjusted accordingly.
 Shape does not meet the M_{px}/A_g limit for shear in AISC Specification Section G2.1(a) with F_y = 50 ksi; therefore, φ_v = 0.50 and λ_{sv} = 1.12.

4.55

Flange Local Buckling Impact on Lateral-Torsional Buckling



4.56

Lateral-Torsional Buckling

- F1.(c) For singly symmetric members in single curvature and all doubly symmetric members

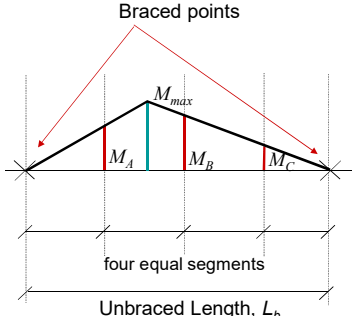
$$C_b = \frac{12.5M_{max}}{2.5M_{max} + 3M_A + 4M_B + 3M_C} \quad (F1-1)$$



4.61

Lateral-Torsional Buckling

- Moment diagram over unbraced length



4.62

Table 3-1
Values of C_b for Simply Supported Beams

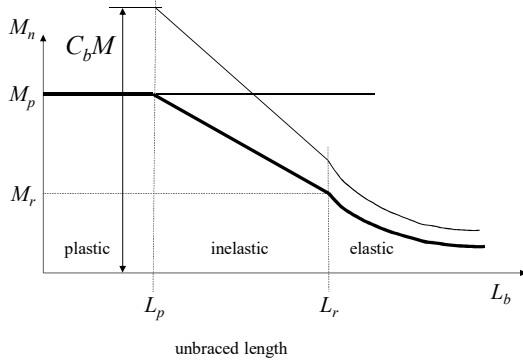
Load	Lateral Bracing Along Span	C_b
	None Load at midpoint	1.32
	At load point	1.67, 1.67
	None Loads at third points	1.14
	At load points Loads symmetrically placed	1.67, 1.00, 1.67
	None Loads at quarter points	1.14
	At load points Loads at quarter points	1.67, 1.11, 1.11, 1.67
	None	1.14
	At midpoint	1.30, 1.30
	At third points	1.45, 1.01, 1.45
	At quarter points	1.52, 1.06, 1.06, 1.52
	At fifth points	1.58, 1.12, 1.00, 1.12, 1.58

Note: Lateral bracing must always be provided at points of support per AISC Specification Chapter 1.

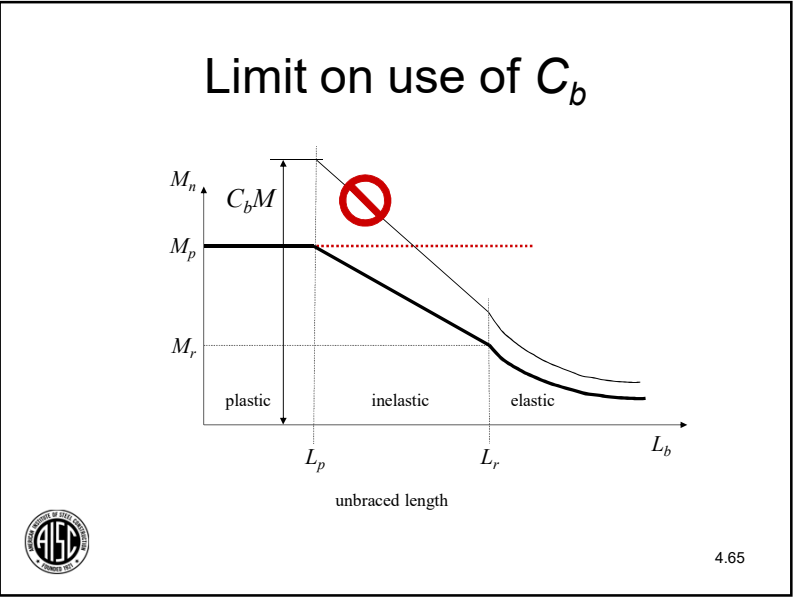


4.63

Limit on use of C_b



4.64



Example 1(ASD)

Simply supported 20 ft span beam with full lateral support and concentrated loads at midspan

$P_L = 24$ kips
 $P_D = 8$ kips

4.66

Example 1(ASD)

$P_a = 8.0 + 24.0 = 32.0$ kips

$M_a = \frac{32.0(20)}{4} = 160$ ft-kips

for a compact, fully braced section

$Z_{req} = \frac{160(12)}{(50/1.67)} = 64.0$ in³

4.67

Example 1(ASD)

Table 3-2 (continued)
W-Shapes
Selection by Z_x

$F_y = 50$ ksi

Shape	Z_x in ³	M_{px}/Ω_b		$\phi_b M_{px}$		M_{rx}/Ω_b		$\phi_b M_{rx}$		BF/Ω_b kips	$\phi_b BF$ kips	L_p ft	L_r ft	I_x in ⁴	V_{nx}/Ω_v		$\phi_v V_{nx}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD									
W21x44	95.4	238	358	143	214	11.1	16.8	4.45	13.0	843	145	217						
W16x50	92.0	230	345	141	213	7.69	11.4	5.62	17.2	659	124	186						
W18x46	90.7	226	340	138	207	9.63	14.6	4.56	13.7	712	130	195						
W14x53	87.1	217	327	136	204	5.22	7.93	6.78	22.3	541	103	154						
W10x69	86.0	190	280	100	136	2.40	3.70	9.04	33.0	303	74.7	112						
W18x35	66.5	166	249	101	151	8.14	12.3	4.31	12.3	510	106	159						
W12x45	64.2	160	241	101	151	3.80	5.80	6.89	22.4	348	81.1	122						
W16x36	64.0	160	240	98.7	148	6.24	9.36	5.37	15.2	448	93.8	141						
W14x38	65	231	354	143	214	5.37	8.20	5.47	16.2	385	87.4	131						
W10x49	64	227	343	143	214	2.46	3.71	8.97	31.6	272	68.0	102						
W8x58	63.8	224	343	143	214	1.70	2.55	7.42	41.6	228	89.3	134						
W12x40	57.0	142	214	89.9	135	3.66	5.54	6.85	21.1	307	70.2	105						
W10x45	54.9	137	206	85.8	129	2.59	3.89	7.10	26.9	248	70.7	106						
W14x34	54.6	136	205	84.9	128	5.01	7.55	5.40	15.6	340	79.8	120						

4.68

Example 1(ASD)

Select W18x35

$Z_x = 66.5 \text{ in.}^3 > 64.0 \text{ in.}^3$
 $M_n / \Omega_b = 166 \text{ ft-kips} > 160 \text{ ft-kips}$
 $L_p = 4.31 \text{ ft}$ is the maximum unbraced length



4.69

Example 1(LRFD)

Simply supported 20 ft span beam with full lateral support and concentrated loads at midspan

$P_L = 24 \text{ kips}$
 $P_D = 8 \text{ kips}$



4.70

Example 1(LRFD)

$P_u = 1.2(8.0) + 1.6(24.0) = 48.0 \text{ kips}$
 $M_u = \frac{48.0(20)}{4} = 240 \text{ ft-kips}$
 for a compact, fully braced section

$Z_{req} = \frac{240(12)}{0.9(50)} = 64.0 \text{ in.}^3$



4.71

Example 1(LRFD)

Table 3-2 (continued)
W-Shapes
Selection by Z_x $F_y = 50 \text{ ksi}$

Shape	Z_x in. ³	M_{px}/Ω_b		M_{ny}/Ω_b		BF/Ω_b		L_p ft	L_r ft	I_x in. ⁴	V_{nx}/Ω_v	
		ASD	LRFD	ASD	LRFD	ASD	LRFD				ASD	LRFD
W21x44	95.4	238	358	143	214	11.1	16.8	4.45	13.0	843	145	217
W16x50	92.0	230	345	141	213	7.69	11.4	5.62	17.2	659	124	186
W18x46	90.7	226	340	138	207	9.63	14.6	4.56	13.7	712	130	195
W14x53	87.1	217	327	136	204	5.22	7.93	6.78	22.3	541	103	154
W10x49	86.0	190	280	100	136	4.40	3.70	9.04	33.0	303	74.7	112
W18x35	66.5	166	249	101	151	8.14	12.3	4.31	12.3	510	106	159
W12x45	64.2	160	241	101	151	3.80	5.80	6.89	22.4	348	81.1	122
W16x36	64.0	160	240	98.7	148	6.24	9.36	5.37	15.2	448	93.8	141
W14x38	65	153	227	95.4	143	5.37	8.20	5.47	16.2	385	87.4	131
W10x49	64	151	227	95.4	143	2.46	3.71	8.97	31.6	272	68.0	102
W8x58	68	149	214	90.8	137	1.70	2.55	7.42	41.6	228	89.3	134
W12x40	57.0	142	214	89.9	135	3.66	5.54	6.85	21.1	307	70.2	105
W10x45	54.9	137	206	85.8	129	2.59	3.89	7.10	26.9	248	70.7	106
W14x34	54.6	136	205	84.9	128	5.01	7.55	5.40	15.6	340	79.8	120



4.72

Example 1 (LRFD)

Select W18x35

$$Z_x = 66.5 \text{ in.}^3 > 64.0 \text{ in.}^3$$

$$\phi_b M_p = 249 \text{ ft-kips} > 240 \text{ ft-kips}$$

$L_p = 4.31 \text{ ft}$ is the maximum unbraced length

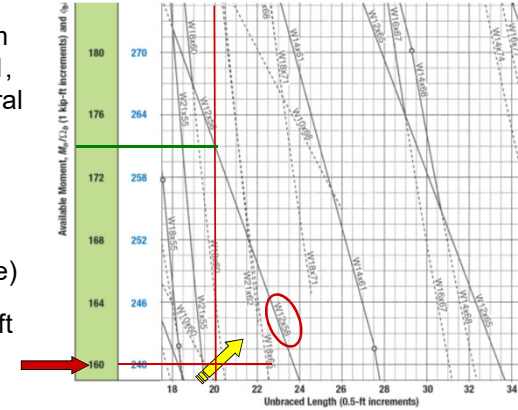


4.73

Example 2 (ASD)

- For the beam of Example 1, assume lateral supports at ends only.
- $L_b = 20 \text{ ft}$
- $C_b = 1.0$ (conservative)

$$M_a = 160 \text{ kip-ft}$$



4.74

Example 2 (ASD)

Select W12x58.

Check using Table 3-2

Z_x Table 3-2 (continued)
W-Shapes
Selection by Z_x $F_y = 50 \text{ ksi}$

Shape	Z_x in. ³	M_p/Ω_c (kip-ft)		M_u/ϕ_b (kip-ft)		$\phi_b M_p$ (kip-ft)		$\phi_b M_n$ (kip-ft)		L_p ft	L_r ft	L_c ft	K_x/Ω_c (kips)	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD					
W21-44	95.4	230	308	143	214	11.1	16.8	4.45	13.0	843	145	217		
W16-50	92.0	230	305	141	213	7.69	11.4	5.82	17.2	659	124	186		
W18-46	90.7	228	300	138	207	9.63	14.6	4.56	13.7	712	130	195		
W14-63	97.7	212	282	136	200	8.22	12.4	6.29	22.4	411	163	154		
W12-58	86.4	216	294	136	200	9.39	13.9	5.97	20.8	475	172	172		
W10-68	85.3	213	290	132	199	2.58	3.83	9.15	40.6	394	97.8	147		
W18-45	82.3	205	309	127	191	7.12	10.8	5.55	16.5	588	111	167		

$$\frac{M_p}{\Omega} = 216 \text{ kip-ft}, \quad \frac{BF}{\Omega} = 3.82 \text{ kips}, \quad L_p = 8.87 \text{ ft}$$

$$\frac{M_n}{\Omega_b} = 1.0 [216 - 3.82(20 - 8.87)] = 173 \text{ ft-kips}$$

$$> M_a = 160 \text{ ft-kips}$$

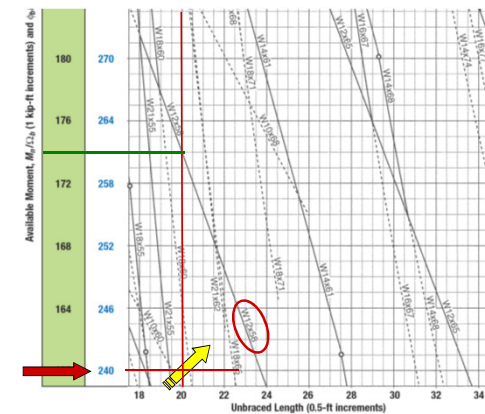


4.75

Example 2 (LRFD)

- For the beam of Example 1, assume lateral supports at ends only.
- $L_b = 20 \text{ ft}$
- $C_b = 1.0$ (conservative)

$$M_u = 240 \text{ kip-ft}$$



4.76

Example 2 (LRFD)

Select W12x58.
Check using Table 3-2

Z_x Table 3-2 (continued)
W-Shapes
Selection by Z_x $F_y = 50$ ksi

Shape	Z_x in. ³	M_p/L_b		M_u/L_b		M_p/L_b		M_u/L_b		BF/L_b	$\phi_B F$	L_c	L_u	M_p/L_b		M_u/L_b	
		ASD kip-ft	LRFD kip-ft	ASD kip-ft	LRFD kip-ft	ASD kip-ft	LRFD kip-ft	ASD kip-ft	LRFD kip-ft								
W21-44	95.4	228	398	143	244	11.1	18.9	4.45	13.0	843	145	217					
W16-50	92.0	330	546	141	213	7.60	11.4	5.62	17.2	680	124	186					
W18-46	90.7	226	340	138	207	9.63	14.6	4.56	13.7	712	130	195					
W14-53	87.1	212	327	136	204	9.95	13.8	6.79	22.3	541	109	154					
W12-58	86.4	211	324	136	205	8.82	5.69	8.87	29.4	479	87.8	132					
W10-68	85.3	213	320	132	199	2.58	3.95	9.15	40.6	394	97.8	147					
W16-45	82.3	208	308	127	191	7.12	10.9	5.55	18.5	586	111	167					

$\phi M_p = 324$ kip-ft, $\phi BF = 5.69$ kips, $L_p = 8.87$ ft

$\phi M_n = 1.0 [324 - 5.69(20 - 8.87)] = 261$ ft-kips

$> M_u = 240$ ft-kips



4.77

Example 3 (ASD)

For the beam of Example 1, assume lateral supports at ends only, $L_b = 20$ ft, and use the correct C_b to determine the lightest shape

Table 3-1
Values of C_b for Simply Supported Beams

Load	Lateral Bracing Along Span	C_b
	None Load at midpoint	1.32
	At load point	1.67
	None	1.0

$C_b = 1.32$



4.78

Example 3 (ASD)

- Using C_b , determine a modified required moment strength

$C_b = 1.32$

$M_a = 160/1.32 = 121$ ft-kips

From Table 3-10 select, at $L_b = 20$ ft
W14x48

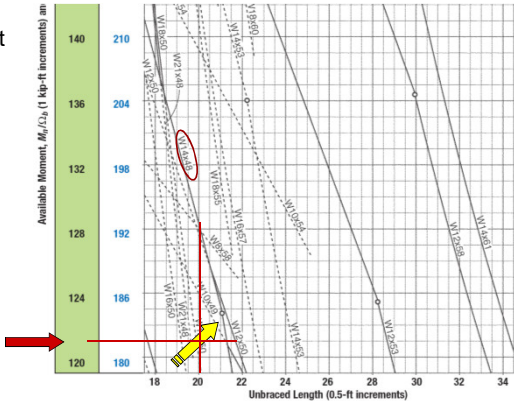


4.79

Example 3 (ASD)

Check to be sure that the required strength does not exceed the plastic moment strength.

$\frac{M_p}{\Omega_b} = 196$
 > 160 ft-kips
 \therefore ok

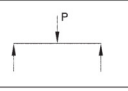



4.80

Example 3 (LRFD)

For the beam of Example 1, assume lateral supports at ends only, $L_b = 20$ ft, and use the correct C_b to determine the lightest shape

Table 3-1
Values of C_b for Simply Supported Beams

Load	Lateral Bracing Along Span	C_b
	None Load at midpoint	1.32
	At load point	1.67 1.87
	None	

$C_b = 1.32$



4.81

Example 3 (LRFD)

- Using C_b , determine a modified required moment strength

$$C_b = 1.32$$

$$M_a = 240/1.32 = 182 \text{ ft-kips}$$

From Table 3-10 select, at $L_b = 20$ ft
W14x48



4.82

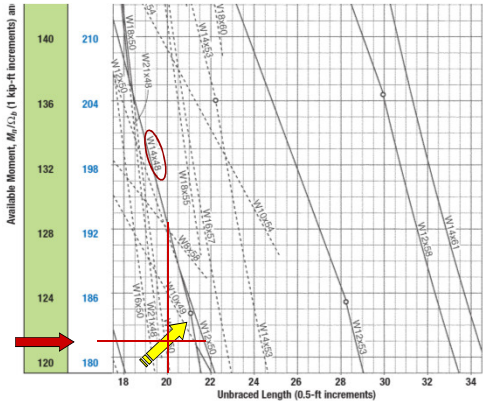
Example 3 (LRFD)

Check to be sure that the required strength does not exceed the plastic moment strength.

$$\phi M_p = 294$$

$$> 240 \text{ ft-kips}$$

$$\therefore \text{ok}$$



4.83





Design for Flexure

- Chapter F also includes:
 - F4, F5. Built-up I-shapes with non-compact or slender webs
 - F6. Minor axis bending
 - 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




4.84

Tees and Double Angles

- Limit States
 - Yielding 
 - Lateral-Torsional Buckling 
 - Flange Local Buckling 
 - Stem Local Buckling 


For this presentation assume a simple beam, thus compression is on top



4.85

Tees and Double Angles


- F9. Tees and Double Angles Loaded in the Plane of Symmetry
 - F9.1 Yielding
 - (a) tee stems and web leg in tension
 - (b) tee stems in compression
 - (c) web legs in compression
 - F9.2 Lateral-Torsional Buckling
 - (a) stems and web legs in tension
 - (b) stems and web legs in compression



4.86



Tees and Double Angles


- F9. Tees and Double Angles Loaded in the Plane of Symmetry
 - F9.3 Flange Local Buckling
 - (a) for tee flanges
 - (b) for double angle flange legs
 - Use single angle Section F10.3
 - F9.4 Tee stems and Web Legs Local Buckling
 - (a) for tee stems
 - (b) for double angle web legs
 - Use single angle Section F10.3



4.87

Tees

- F9.1. Yielding
 - $M_n = M_p$ (F9-1)
 - (a) For tee stems and web legs in tension 
 - $M_p = F_y Z_x \leq 1.6 M_y$ (F9-2)
 - (b) For tee stems in compression 
 - $M_p = M_y$ (F9-4)



4.88

Tees

- F9.2 Lateral-Torsional Buckling

(a) For stems and web legs in tension **T**

(1) When $L_b \leq L_p$

LTB does not apply

(2) When $L_p < L_b \leq L_r$

$$M_n = M_p - (M_p - M_y) \left[\frac{L_b - L_p}{L_r - L_p} \right] \quad (\text{F9-6})$$



4.89

Tees

- F9.2 Lateral-Torsional Buckling

(a) For stems and web legs in tension **T**

(3) When $L_b > L_r$

$$M_n = M_{cr} \quad (\text{F9-7})$$

where

$$M_{cr} = \frac{1.95E}{L_b} \sqrt{I_y J} \left[B + \sqrt{1 + B^2} \right] \quad (\text{F9-10})$$

and

$$B = 2.3 \left(\frac{d}{L_b} \right) \sqrt{\frac{I_y}{J}} \quad (\text{F9-11})$$

Note that with the stem in tension, B is positive



4.90

Tees

- F9.2 Lateral-Torsional Buckling

(b) For stems and web legs in compression **L**

(1) For tee stems

$$M_n = M_{cr} \leq M_y \quad (\text{F9-13})$$

where

$$M_{cr} = \frac{1.95E}{L_b} \sqrt{I_y J} \left[B + \sqrt{1 + B^2} \right] \quad (\text{F9-10})$$

$$B = -2.3 \left(\frac{d}{L_b} \right) \sqrt{\frac{I_y}{J}} \quad (\text{F9-12})$$

Note that with the stem in compression, B is negative



4.91

Tees

- F9.3. Flange Local Buckling **T**

(a) For tee flanges

(1) Compact flange

FLB does not apply

(2) Noncompact flange

$$M_n = M_p - (M_p - 0.7F_y S_{xc}) \left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}} \right) \leq 1.6M_y \quad (\text{F9-14})$$

(3) Slender flange

$$M_n = \frac{0.7ES_{xc}}{\left(\frac{b_f}{2t_f} \right)^2} \quad (\text{F9-15})$$



4.92

Tees

- F9.4. Stem Local Buckling

(a) For tee stems

$$M_n = F_{cr} S_x \quad (F9-16)$$

(1) when $\frac{d}{t_w} \leq 0.84 \sqrt{\frac{E}{F_y}}$

$$F_{cr} = F_y \quad (F9-17)$$

(2) when $0.84 \sqrt{\frac{E}{F_y}} < \frac{d}{t_w} \leq 1.52 \sqrt{\frac{E}{F_y}}$

$$F_{cr} = \left[1.43 - 0.515 \frac{d}{t_w} \sqrt{\frac{F_y}{E}} \right] F_y \quad (F9-18)$$



4.93

Tees

- F9.4. Stem Local Buckling

(a) For tee stems

(3) when $\frac{d}{t_w} > 1.52 \sqrt{\frac{E}{F_y}}$

$$F_{cr} = \frac{1.52E}{\left(\frac{d}{t_w}\right)^2} \quad (F9-19)$$



4.94

Example 4

- Determine the nominal flexural strength of a WT16.5x59 with an unbraced length, $L_b = 10$ ft and the stem in tension.

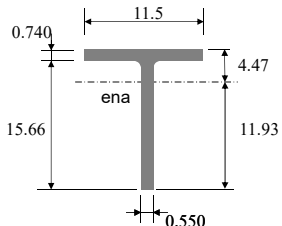
$$\begin{aligned} A_g &= 17.4 \text{ in.}^2 & d &= 16.4 \text{ in.} \\ S_x &= 39.2 \text{ in.}^3 & t_w &= 0.550 \text{ in.} \\ Z_x &= 70.8 \text{ in.}^3 & J &= 2.64 \text{ in.}^4 \\ I_y &= 93.5 \text{ in.}^4 & r_y &= 2.32 \text{ in.} \end{aligned}$$



4.95

Example 4

- Elastic Neutral Axis (ena)
 - Moment of Area above ena = Moment of Area below ena



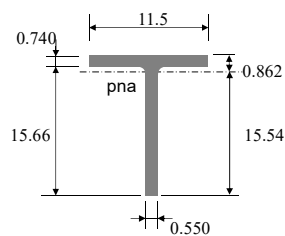
$$S_{bottom} = 39.2 \text{ in.}^3$$



4.96

Example 4

- Plastic Neutral Axis (pna)
 - Area above pna = Area below pna



$$Z_x = 70.8 \text{ in.}^3$$

$$\beta = \frac{70.8}{39.2} = 1.81$$



4.97

Example 4

- F9.1 Yielding
 - (a) Tee stem in tension **T**

$$M_p = F_y Z_x \leq 1.6 M_y \quad (\text{F9-2})$$

$$= 50(70.8) = 3540 \text{ in.-kips}$$

$$M_y = F_y S_x \quad (\text{F9-3})$$

$$= 50(39.2) = 1960 \text{ in.-kips}$$

$$1.6 M_y = 1.6(1960) = 3140 \text{ in.-kips}$$

thus,

$$M_n = M_p = 1.6 M_y = 3140 \text{ in.-kips}$$



4.98

Example 4

- For lateral torsional buckling with the stem in tension, check unbraced length

$$L_p = 1.76 r_y \sqrt{\frac{E}{F_y}} = 1.76(2.32) \sqrt{\frac{E}{50}} = 98.3 \text{ in.} \Rightarrow 8.19 \text{ ft.} \quad (\text{F9-8})$$

$$L_r = 1.95 \left(\frac{E}{F_y} \right) \frac{\sqrt{I_y J}}{S_x} \sqrt{2.36 \left(\frac{F_y}{E} \right) \frac{d S_x}{J} + 1} \quad (\text{F9-9})$$

$$= 1.95 \left(\frac{E}{50} \right) \frac{\sqrt{93.5(2.64)}}{39.2} \sqrt{2.36 \left(\frac{50}{E} \right) \frac{16.4(39.2)}{2.64} + 1}$$

$$= 640 \text{ in.} \Rightarrow 53.3 \text{ ft}$$



4.99

Example 4

- Since $L_p < L_b \leq L_r$
 - This is inelastic lateral-torsional buckling

$$M_n = M_p - (M_p - M_y) \left(\frac{L_b - L_p}{L_r - L_p} \right) \quad (\text{F9-6})$$

$$= 3140 - (3140 - 1960) \left(\frac{10.0 - 8.19}{53.3 - 8.19} \right)$$

$$= 3090 \text{ in.-kips}$$



4.100

Example 4

- Flange local buckling **T**

$$\frac{b_f}{2t_f} = 7.76 \leq \lambda_p = 0.38 \sqrt{\frac{E}{F_y}} = 9.15$$

– Therefore it is compact

- Stem local buckling **T**

– The stem is not in compression, therefore this limit state is not applicable



4.101

Example 4

Summary	Limit State	M_n	ϕM_n	M_n/Ω
		In-kips	In-kips	In-kips
T Stem in Tension	Yielding	3140		
	Lateral-torsional buckling	3090	2780	1850
	Flange local buckling	Compact		
	Stem local buckling	Not in comp.		



4.102

Shear

- Two methods of calculating shear strength are available.
 - The method presented in Section G2.1 utilizes the post buckling strength of the member without using tension field action. All rolled W-shapes fit here.
 - The method presented in Section G2.2 utilizes tension field action for interior stiffened panels.



4.103

Shear

- Limit States
 - Yielding
 - Buckling
 - Post buckling strength
 - Post buckling strength through Tension Field Action. Only applies to built-up I-shapes
 - Shear Rupture
 - Treated as a connection concern



4.104

Shear

- The nominal shear strength, V_n , of unstiffened or stiffened webs according to the limit states of shear yielding and shear buckling, is

$$V_n = 0.6F_y A_w C_{v1} \quad (\text{G2-1})$$



4.105

Shear

- G2.1.(a) For webs of rolled I-shaped members with

$$\frac{h}{t_w} \leq 2.24 \sqrt{\frac{E}{F_y}}$$

$$C_{v1} = 1.0 \quad (\text{G2-2})$$

This means the web will yield in shear.



4.106

Shear

- User Note:** All current ASTM A6 W, S, and HP shapes except W44x230, W40x149, W36x135, W33x118, W30x90, W24x55, W16x26 and W12x14 meet the criteria stated in Section G2.1(a) for $F_y = 50$ ksi (345 MPa).

For the shapes that meet the requirements of Section G2.1(a),

$$\phi_v = 1.00 \text{ (LRFD)} \quad \Omega_v = 1.50 \text{ (ASD)}$$

For others, there are only 8 W-shapes,



$$\phi_v = 0.90 \text{ (LRFD)} \quad \Omega_v = 1.67 \text{ (ASD)}$$

4.107

Shear

- G2.1.(b)(1) For all other I-shaped members and channels,

(i) when

$$\frac{h}{t_w} \leq 1.10 \sqrt{\frac{k_v E}{F_y}}$$

$$C_{v1} = 1.0 \quad (\text{G2-3})$$

This means the web will yield in shear.



4.108

Shear

- G2.1.(b)(1)

(ii) when

$$\frac{h}{t_w} > 1.10 \sqrt{\frac{k_v E}{F_y}}$$

$$C_{v1} = \frac{1.10 \sqrt{k_v E / F_y}}{h/t_w} \quad (G2-4)$$

This means the web will buckle inelastically.



4.109

Shear

- Web plate shear buckling coefficient
– For webs without transverse stiffeners

$$k_v = 5.34$$

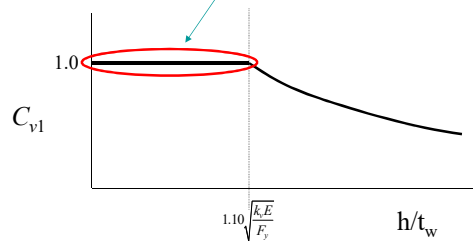
- **User Note:** For all ASTM A6 W, S, M and HP shapes except M12.5x12.4, M12.5x11.6, M12x11.8, M12x10.8, M12x10, M10x8 and M10x7.5, when $F_y = 50$ ksi (345 MPa), $C_{v1} = 1.0$.



4.110

Shear

All W-shapes will fall in this range



4.111

Shear

- G2.1.(a) For webs of rolled I-shaped members with

$$\frac{h}{t_w} \leq 2.24 \sqrt{\frac{E}{F_y}}$$

$$\phi_v = 1.00 \text{ (LRFD)} \quad \Omega_v = 1.50 \text{ (ASD)}$$



4.112

Shear

For W-shapes, $k_v = 5.34$

4.113

Manual

Z_x tables and uniform load tables show shear strength

Z_x

Table 3-2 (continued)
W-Shapes
Selection by Z_x

F_y = 50 ksi

Shape	Z _x in. ³	M _x /Ω _b		M _x /φ _b		M _y /Ω _b		M _y /φ _b		BF/Ω _b kips	BF/φ _b kips	L _p ft	L _r ft	I _x in. ⁴	S _x /Ω _b		S _x /φ _b	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD						ASD	LRFD		
W21×44	95.4	238	358	143	214	11.1	16.8	4.45	13.0	843	145	217						
W16×50	92.0	230	345	141	213	7.69	11.4	5.62	17.2	659	124	186						
W18×46	90.7	226	340	138	207	9.63	14.6	4.56	13.7	712	130	195						
W14×53	87.1	217	327	136	204	5.22	7.93	6.78	22.3	541	103	154						
W10×34	66.6	166	249	101	151	8.14	12.3	4.31	12.3	610	106	159						
W12×45	64.2	160	241	101	151	3.80	5.80	6.89	22.4	348	81.1	122						
W10×36	64.0	160	240	98.7	148	6.24	9.36	5.37	15.2	448	93.8	141						
W14×38	61.5	153	231	95.4	143	5.37	8.20	5.47	16.2	385	87.4	131						
W10×49	60.4	151	227	95.4	143	2.46	3.71	8.97	31.6	272	68.0	102						
W8×58	59.8	149	224	90.8	137	1.70	2.55	7.42	41.6	228	89.3	134						
W12×40	57.0	142	214	89.9	135	3.66	5.54	6.85	21.1	307	70.2	105						
W10×45	54.9	137	206	85.8	129	2.59	3.89	7.10	26.9	248	70.7	106						
W14×34	54.6	136	205	84.8	128	5.01	7.55	5.40	15.6	340	70.8	106						

F_y = 50 ksi

Table 3-6 (continued)
Maximum Total
Uniform Load, kips
W-Shapes

W10-W16

Shape	WB=						WT=					
	60	60	48	40	36	100	60	60	48	40	36	
Design	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
6	282	426	285	428	261	391	226	339	212	313	190	285
7	348	522	348	522	244	366	244	366	244	366	244	366
8	414	618	414	618	227	340	227	340	227	340	227	340
9	480	714	480	714	210	315	210	315	210	315	210	315
10	546	810	546	810	193	290	193	290	193	290	193	290
11	612	906	612	906	176	265	176	265	176	265	176	265
12	678	1002	678	1002	159	240	159	240	159	240	159	240
13	744	1098	744	1098	142	215	142	215	142	215	142	215
14	810	1194	810	1194	125	190	125	190	125	190	125	190
15	876	1290	876	1290	108	165	108	165	108	165	108	165
16	942	1386	942	1386	91	140	91	140	91	140	91	140
17	1008	1482	1008	1482	74	115	74	115	74	115	74	115
18	1074	1578	1074	1578	57	90	57	90	57	90	57	90
19	1140	1674	1140	1674	40	65	40	65	40	65	40	65
20	1206	1770	1206	1770	23	40	23	40	23	40	23	40

4.114

Example 4(ASD)

- For the beam designed in Example 1, confirm that it has sufficient strength in shear.

$$P_a = 8.0 + 24.0 = 32.0 \text{ kips}$$

$$V_a = \frac{32.0}{2} = 16.0 \text{ kips}$$

4.115

Example 4(ASD)

$V_a = 16.0 <$
 $V_n = 106 \text{ kips}$
 $\Omega =$

Z_x

Table 3-2 (continued)
W-Shapes
Selection by Z_x

F_y = 50 ksi

Shape	Z _x in. ³	M _x /Ω _b		M _x /φ _b		M _y /Ω _b		M _y /φ _b		BF/Ω _b kips	BF/φ _b kips	L _p ft	L _r ft	I _x in. ⁴	S _x /Ω _b		S _x /φ _b	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD									
W21×44	95.4	238	358	143	214	11.1	16.8	4.45	13.0	843	145	217						
W16×50	92.0	230	345	141	213	7.69	11.4	5.62	17.2	659	124	186						
W18×46	90.7	226	340	138	207	9.63	14.6	4.56	13.7	712	130	195						
W14×53	87.1	217	327	136	204	5.22	7.93	6.78	22.3	541	103	154						
W10×34	66.6	166	249	101	151	8.14	12.3	4.31	12.3	610	106	159						
W12×45	64.2	160	241	101	151	3.80	5.80	6.89	22.4	348	81.1	122						
W10×36	64.0	160	240	98.7	148	6.24	9.36	5.37	15.2	448	93.8	141						
W14×38	61.5	153	231	95.4	143	5.37	8.20	5.47	16.2	385	87.4	131						
W10×49	60.4	151	227	95.4	143	2.46	3.71	8.97	31.6	272	68.0	102						
W8×58	59.8	149	224	90.8	137	1.70	2.55	7.42	41.6	228	89.3	134						
W12×40	57.0	142	214	89.9	135	3.66	5.54	6.85	21.1	307	70.2	105						
W10×45	54.9	137	206	85.8	129	2.59	3.89	7.10	26.9	248	70.7	106						
W14×34	54.6	136	205	84.8	128	5.01	7.55	5.40	15.6	340	70.8	106						

Available strength tabulated above herein (in kips) is limited by available shear strength.

4.116

Example 4(LRFD)

- For the beam designed in Example 1, confirm that it has sufficient strength in shear.

$$P_u = 1.2(8.0) + 1.6(24.0) = 48.0 \text{ kips}$$

$$V_u = \frac{48.0}{2} = 24.0 \text{ kips}$$



4.117

Example 4(LRFD)

$$V_u = 24.0 <$$

$$\phi V_n = 159 \text{ kips}$$

Z_x

Table 3-2 (continued)
W-Shapes
Selection by Z_x F_y = 50 ksi

Shape	Z _x in. ³	M _{px} /Ω _b		φ _b M _{px}		M _{px} /Ω _b		φ _b M _{px}		BF/Ω _b		φ _b BF		L _p ft	L _r ft	I _x in. ⁴	W _{pl} /Ω _y		φ _y W _{pl}	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	kips	kips	ASD	LRFD	kips	kips							
W21×44	95.4	238	358	143	214	11.1	16.8	4.45	13.0	843	145	217								
W16×50	92.0	230	345	141	213	7.69	11.4	5.62	17.2	659	124	186								
W18×46	90.7	226	340	138	207	9.63	14.6	4.56	13.7	712	130	195								
W14×53	87.1	217	327	136	204	5.22	7.93	6.78	22.3	541	103	154								
W10×34	66.0	166	249	101	151	2.46	3.73	3.04	33.0	303	74.7	112								
W18×35	66.5	166	249	101	151	8.14	12.3	4.31	12.3	610	106	169								
W12×45	64.2	160	241	101	151	3.80	5.80	6.89	22.4	348	81.1	122								
W16×36	64.0	160	240	98.7	148	6.24	9.36	5.37	15.2	448	93.8	141								
W14×38	61.5	153	231	95.4	143	5.37	8.20	5.47	16.2	385	87.4	131								
W10×49	60.4	151	227	95.4	143	2.46	3.71	8.97	31.6	272	68.0	102								
W8×58	59.8	149	224	90.8	137	1.70	2.55	7.42	41.6	228	89.3	134								
W12×40	57.0	142	214	89.9	135	3.66	5.54	6.85	21.1	307	70.2	105								
W10×45	54.9	137	206	85.8	129	2.59	3.89	7.10	28.9	248	70.7	108								
W14×34	54.6	136	205	84.6	128	4.01	7.05	5.40	15.6	340	70.8	120								



4.118

Summary

- Looked at elastic and plastic behavior of beams
- Addressed the limit state of lateral-torsional buckling
- Investigated the influence of local buckling on beam strength
- Illustrated the use of several design aids
- Reviewed how beam shear is addressed



4.119

Lesson 5

- The next lesson will consider the principles of interaction of compression and bending.
- The Specification equations will be addressed.
- We concentrate on Chapter H of the Specification and Part 6 of the Manual



4.120



Thank You

American Institute of Steel Construction
130 East Randolph St., Suite 2000
Chicago, IL 60601



3.121

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Night School Resources for 8-session package Registrants

Go to www.aisc.org and sign in.

The screenshot shows the MyAISC user interface. On the left is a sidebar titled 'IN THIS SECTION' with a list of menu items: Edit Profile, My Downloads, My Pending Quizzes, My Events, Order History, Course History, and Course Resources. The 'Course Resources' item is circled in red. The main content area is titled 'MyAISC' and contains three sections: 'MY PROFILE' with an 'EDIT PROFILE' button, 'MY PURCHASED DOWNLOADS' with a 'VIEW DOWNLOADS' button, and 'MY COURSE RESOURCES' with a 'VIEW RESOURCES' button. The 'MY COURSE RESOURCES' section is also circled in red.

Please fill out our brief survey at the conclusion of the webinar. We greatly appreciate your feedback.

AISC | Thank you