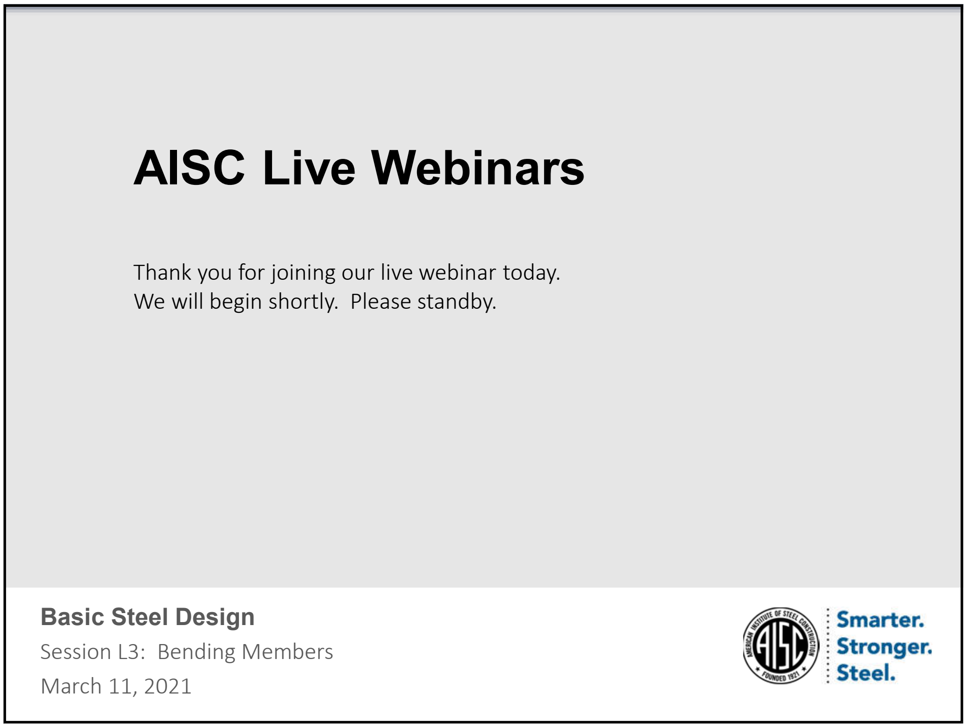

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Basic Steel Design
Session L3: Bending Members
March 11, 2021

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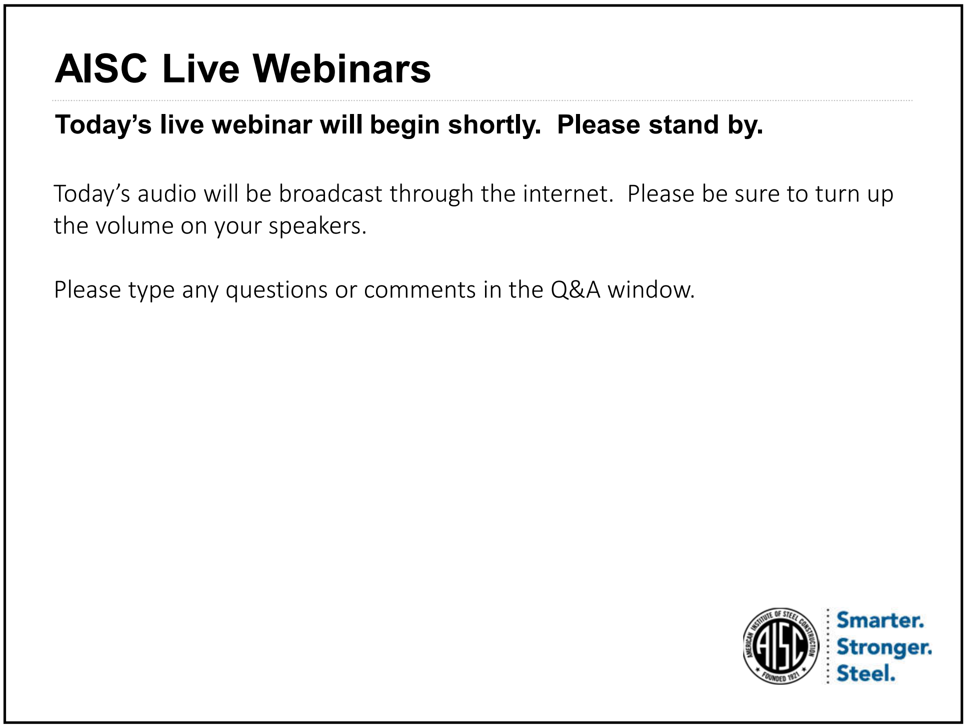

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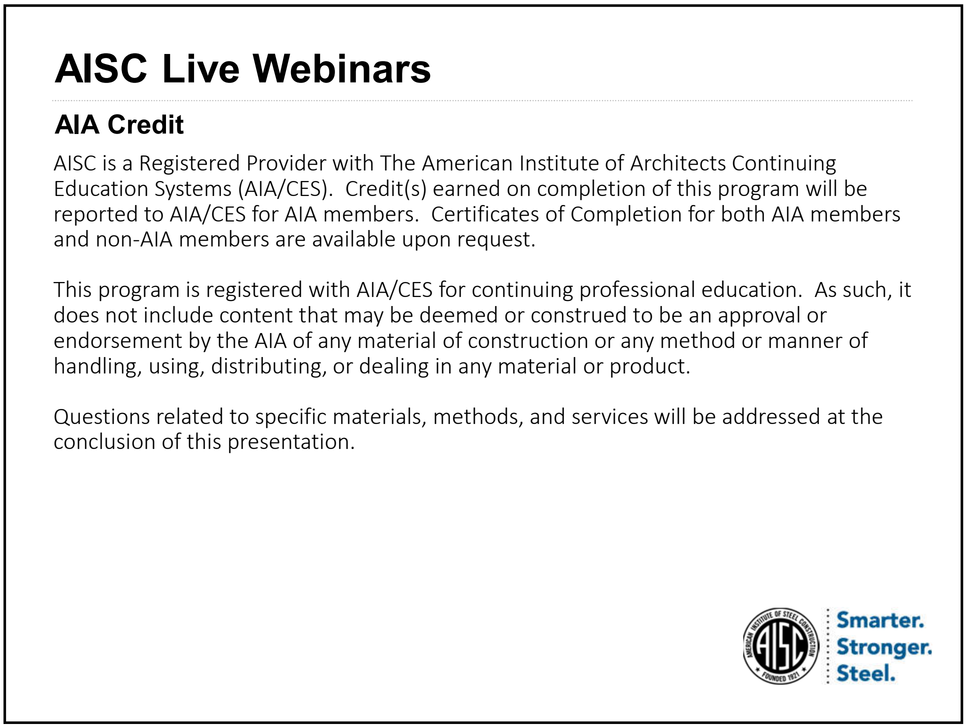

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Course Description

Bending Members

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.

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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.

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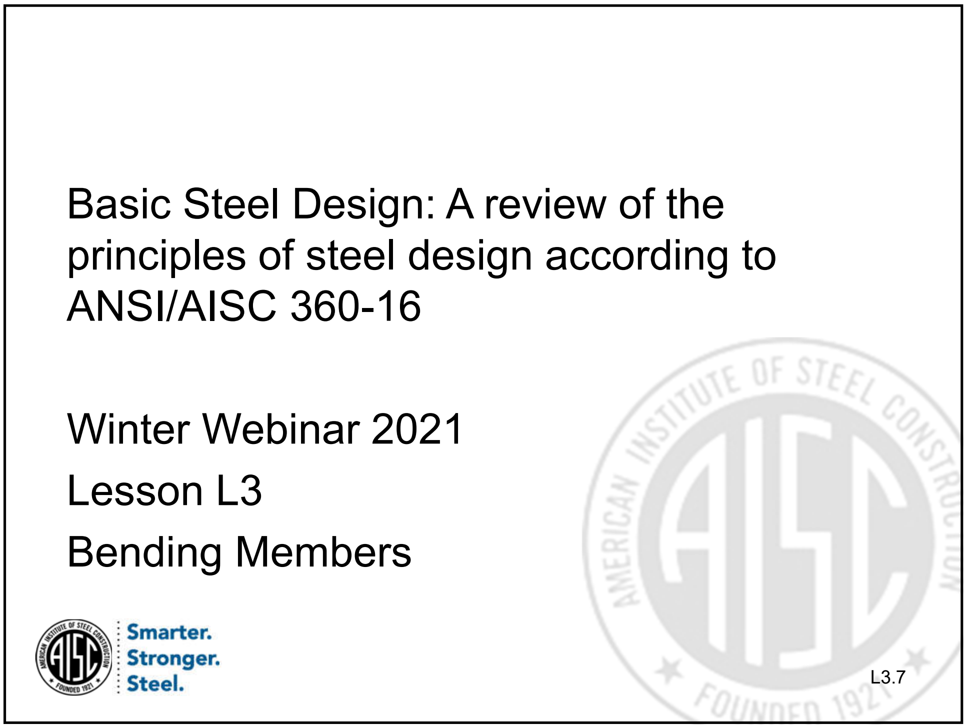

Basic Steel Design: A review of the principles of steel design according to ANSI/AISC 360-16

Winter Webinar 2021

Lesson L3

Bending Members

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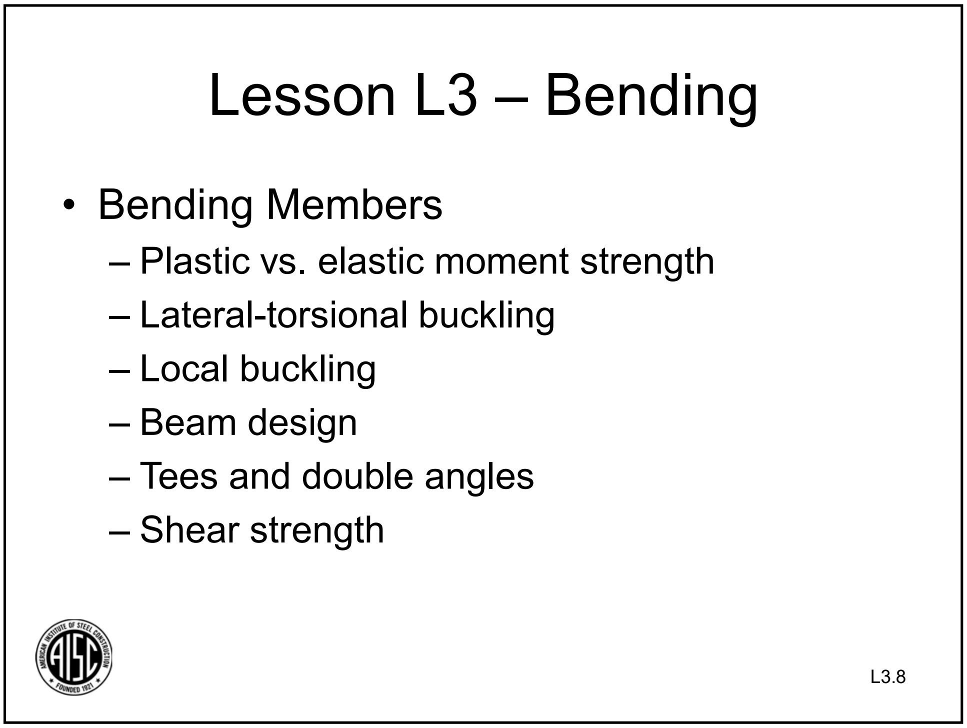
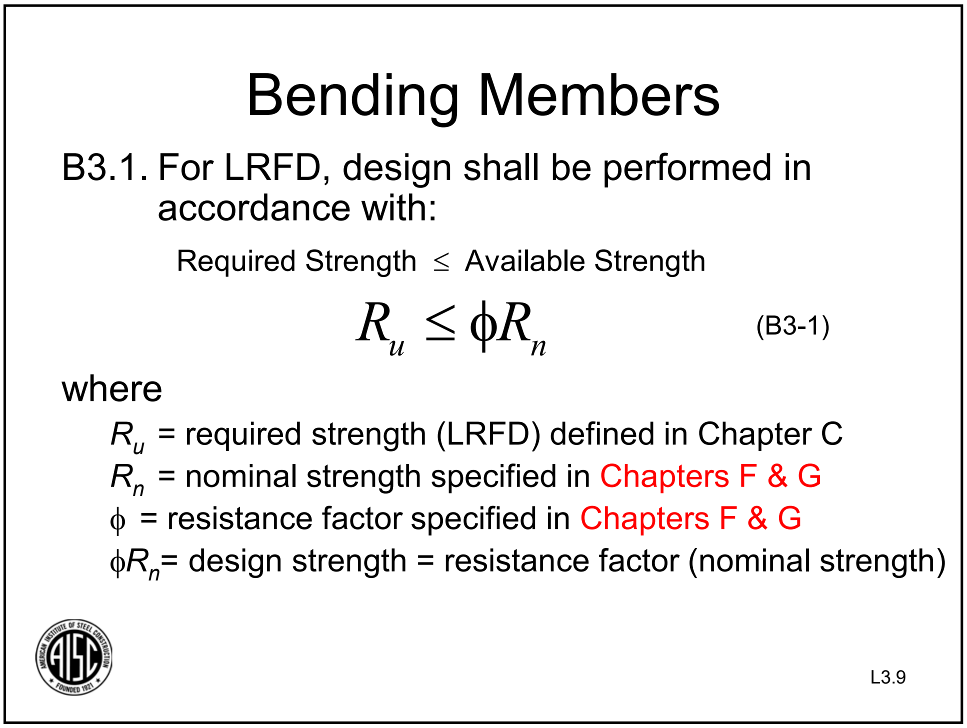


Lesson L3 – Bending

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



Bending Members

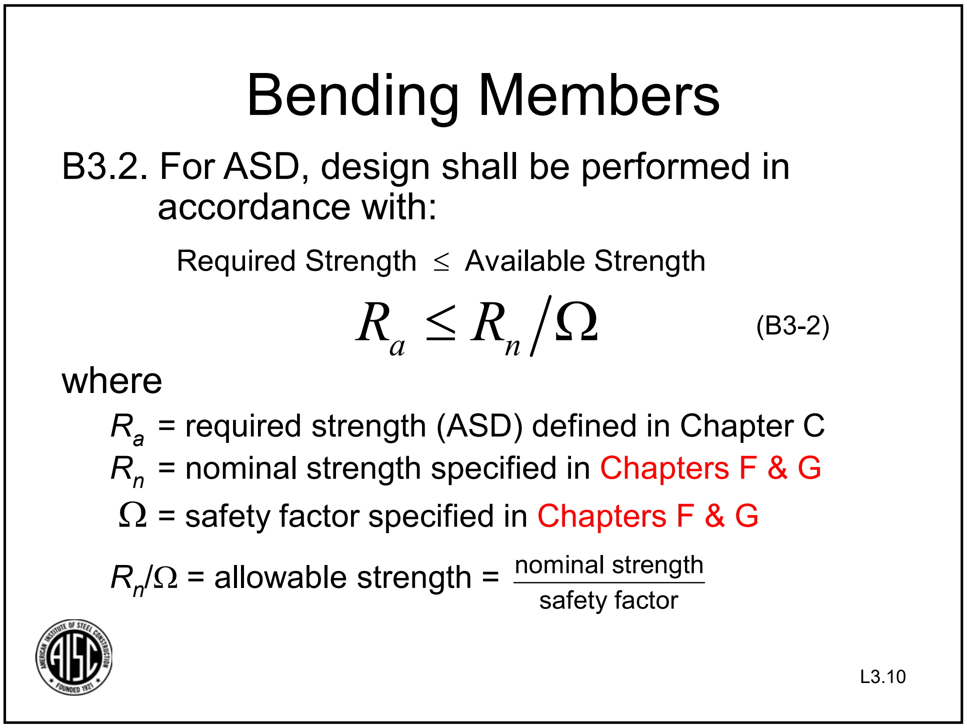
B3.1. For LRFD, design shall be performed in accordance with:

Required Strength $\le$ Available Strength

$$R_u \le \phi R_n \qquad \text{(B3-1)}$$

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)



Bending Members

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

Required Strength $\le$ Available Strength

$$R_a \le R_n / \Omega \qquad \text{(B3-2)}$$

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 = $\dfrac{\text{nominal strength}}{\text{safety factor}}$

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

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

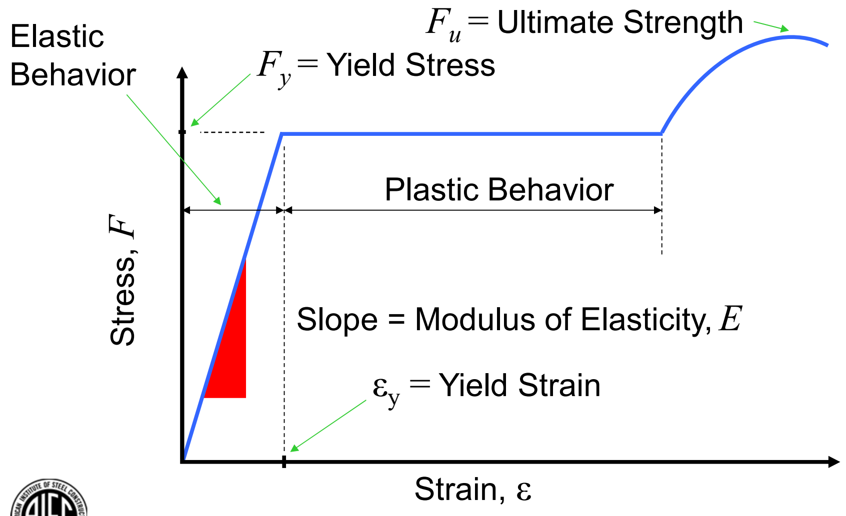
and the nominal flexural strength, M_n, shall be determined according to Sections F2 through F13.

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

Bending Members

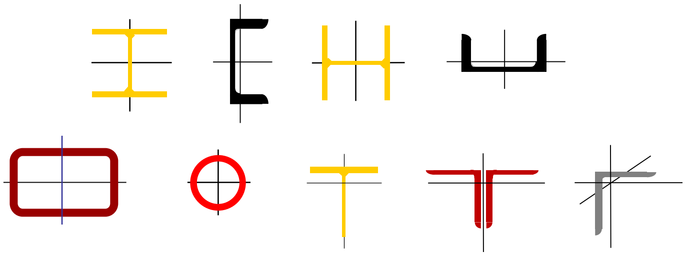
- Limit States
  - Yielding: as seen in the discussion of compression, yielding is the upper limit for all shapes.
  - Lateral-torsional buckling: a combination of lateral buckling and twist.
  - Local buckling: buckling of elements before they are able to reach yield.

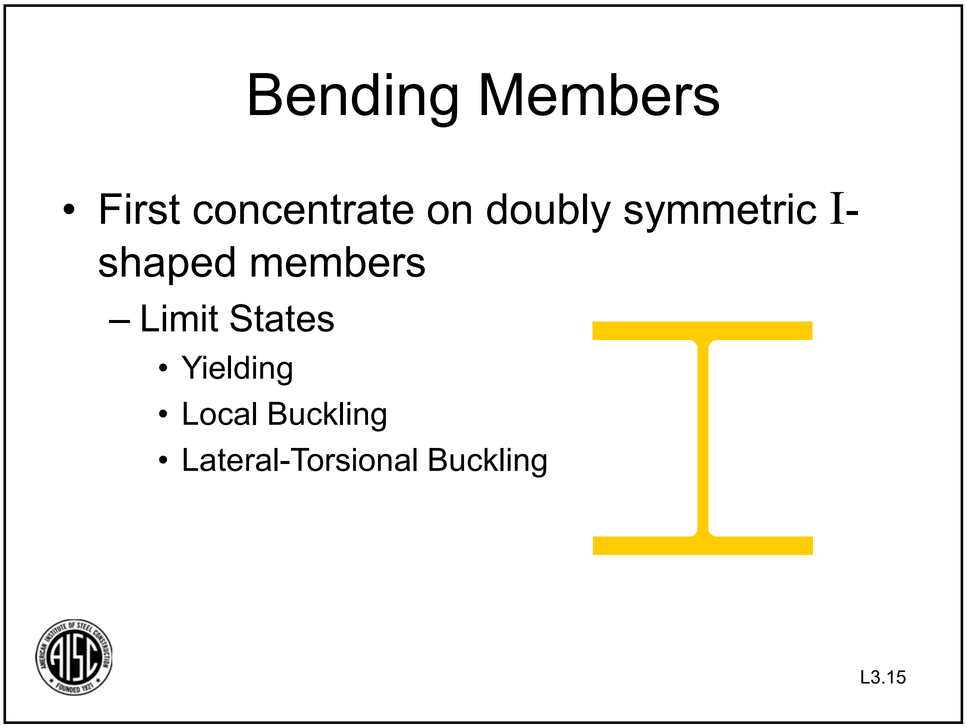
Steel as a Material



Bending Members

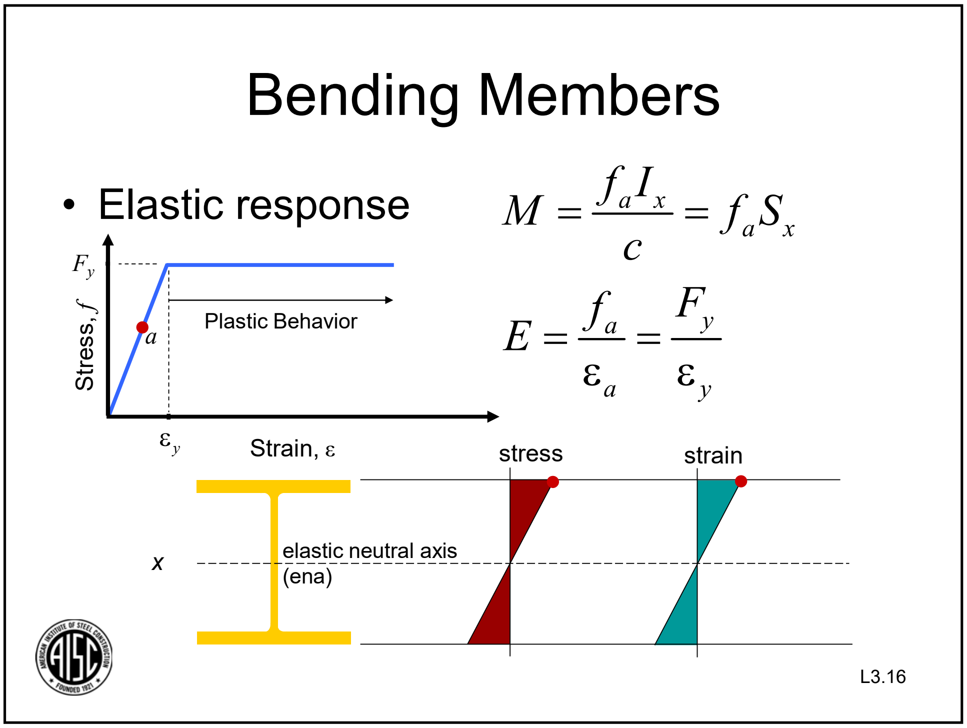
- Limit States
  - The buckling limit states influence strength differently depending on the shape





Bending Members

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

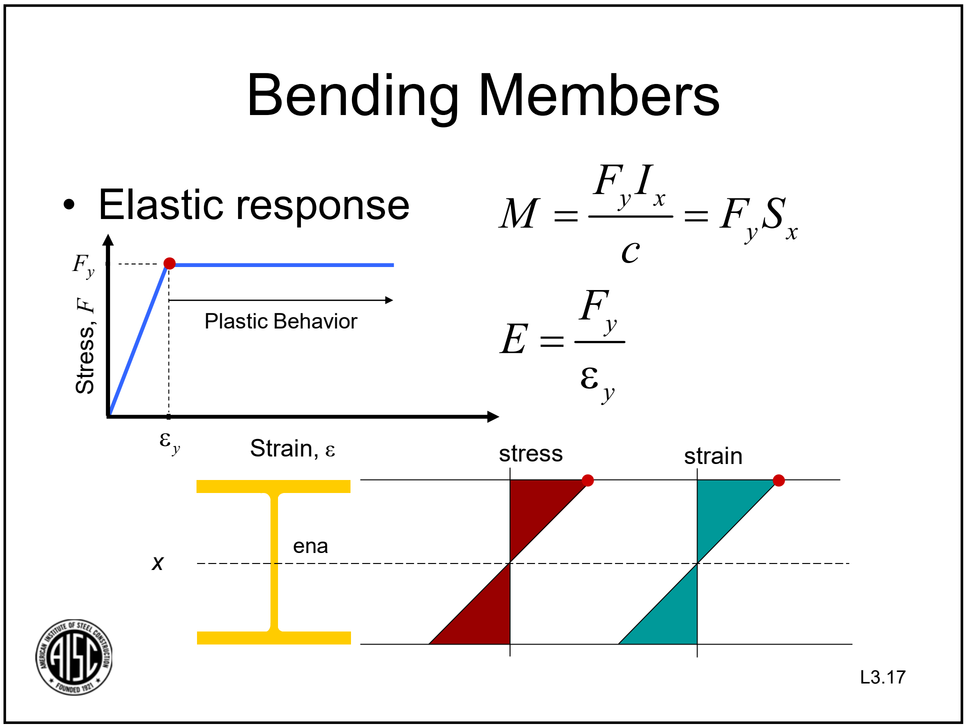


Bending Members

- Elastic response

$$M = \frac{f_a I_x}{c} = f_a S_x$$

$$E = \frac{f_a}{\varepsilon_a} = \frac{F_y}{\varepsilon_y}$$

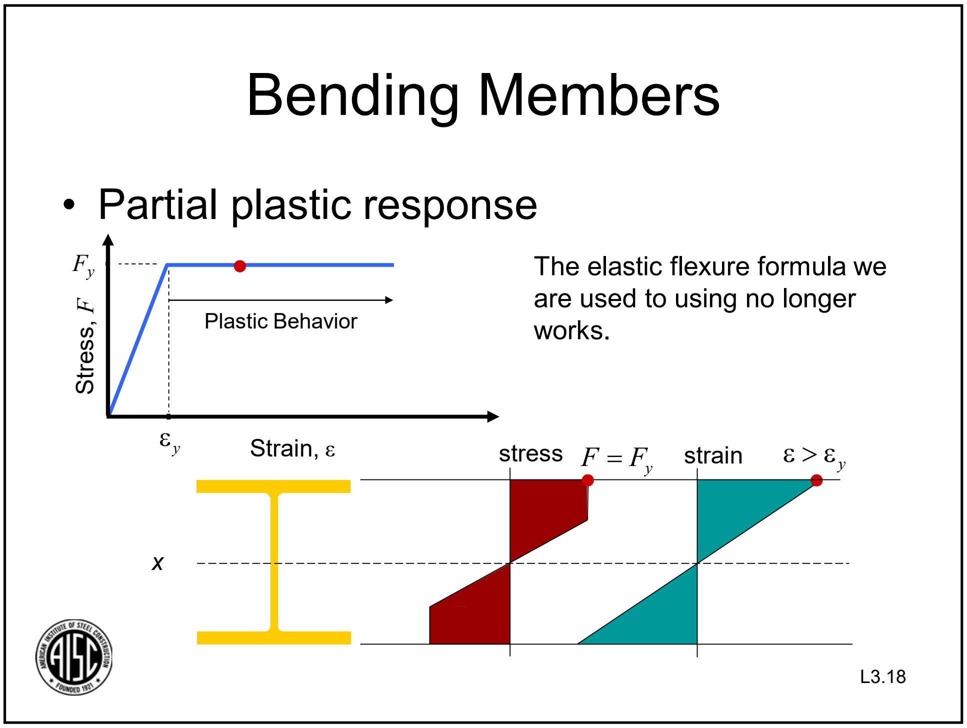


Bending Members

- Elastic response

$$M = \frac{F_y I_x}{c} = F_y S_x$$

$$E = \frac{F_y}{\varepsilon_y}$$



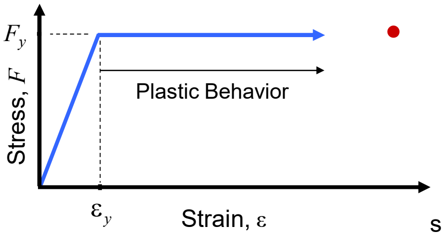
Bending Members

- Partial plastic response

The elastic flexure formula we are used to using no longer works.

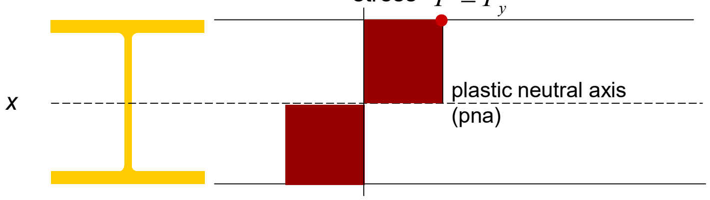
Bending Members

- Full plastic response – the yield limit state



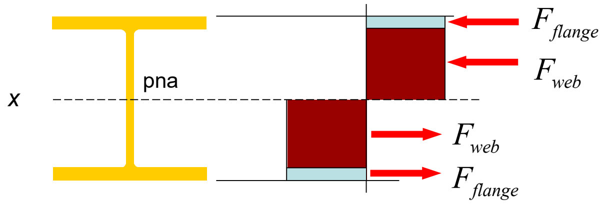
- Strain is assumed to be infinite

Thus, stress is at the yield level over the entire cross section



Bending Members

- Determine the force in the flange and web

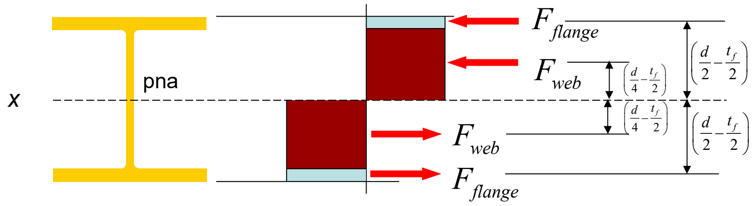


$$F_{flange} = F_y A_{flange} = F_y b_f t_f$$

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

Bending Members

- Take moments about the plastic neutral axis



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

Bending Members

- Simplifying

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

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

Bending Members

- How do we define the plastic neutral axis?

$$T = C$$

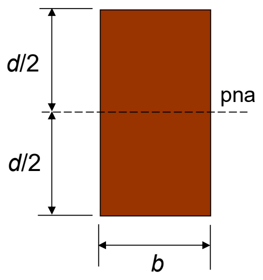
$$F_y A_T = F_y A_C$$

$$A_T = A_C$$

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

Bending Members

- For a rectangle the pna is in the middle.



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

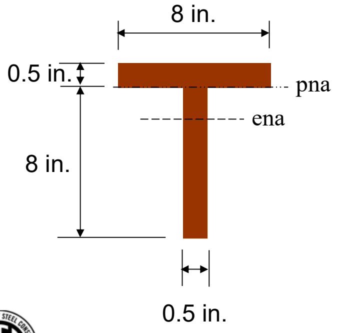
$$S = \frac{bd^2}{6}$$

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

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

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.

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

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

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

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.

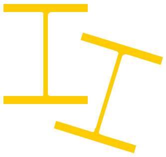
Bending Members

- 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).

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



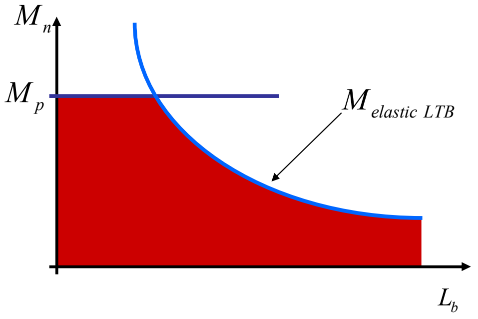
Bending Members

- 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.

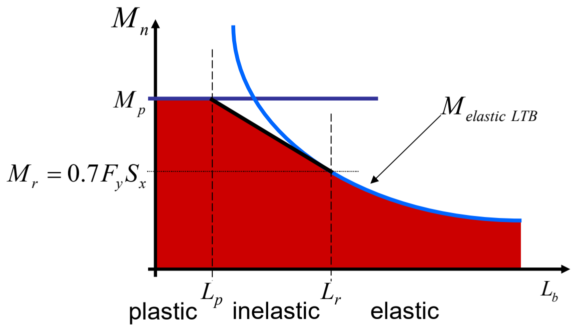
Bending Members

- If the member behaves either plastically or elastically



Bending Members

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



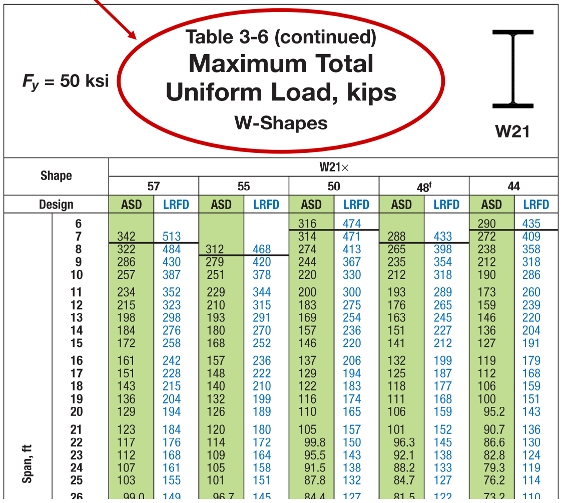
Lateral-Torsional Buckling

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

$$M_n = M_p = F_yZ_x \qquad \text{(F2-1)}$$

Design Aid

Based on an unbraced length less than or equal to L_p



F_y = 50 ksi

Table 3-6 (continued)

Maximum Total Uniform Load, kips

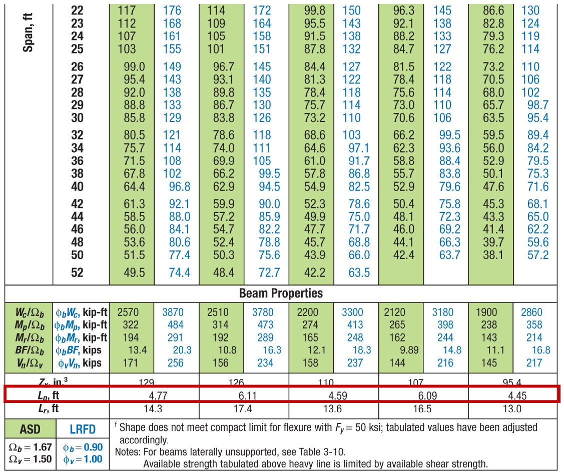
W-Shapes

W21

Shape		W21×									
		57		55		50		48^f		44	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft	6					316	474			290	435
	7	342	513			314	471	288	433	272	409
	8	322	484	312	468	274	413	265	398	238	358
	9	286	430	279	420	244	367	235	354	212	318
	10	257	387	251	378	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	183	118	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	158	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	81.5	122	73.2	110

Design Aid

Based on an unbraced length less than or equal to L_p



Span, ft	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	158	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	81.5	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	105	61.0	91.7	58.8	88.4	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	78.8	45.7	68.8	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_c/Ω_b	$\phi_b W_c$, kip-ft	2570	3870	2510	3780	2200	3300	2120	3180	1900	2860
M_p/Ω_b	$\phi_b M_p$, kip-ft	322	484	314	473	274	413	265	398	238	358
M_r/Ω_b	$\phi_b M_r$, kip-ft	194	291	192	289	165	248	162	244	143	214
BF/Ω_b	$\phi_b BF$, kips	13.4	20.3	10.8	16.3	12.1	18.3	9.89	14.8	11.1	16.8
V_n/Ω_v	$\phi_v V_n$, kips	171	256	156	234	158	237	144	216	145	217
Z_x, in.^{3}		129		126		110		107		95.4	
L_p, ft		4.77		6.11		4.59		6.09		4.45	
L_r, ft		14.3		17.4		13.6		16.5		13.0	

ASD	LRFD
$\Omega_b = 1.67$ $\Omega_v = 1.50$	$\phi_b = 0.90$ $\phi_v = 1.00$

[f] Shape does not meet compact limit for flexure with $F_y = 50$ ksi; tabulated values have been adjusted accordingly.

Notes: For beams laterally unsupported, see Table 3-10.
Available strength tabulated above heavy line is limited by available shear strength.

Lateral-Torsional Buckling

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

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

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

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.

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

Lateral-Torsional Buckling

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

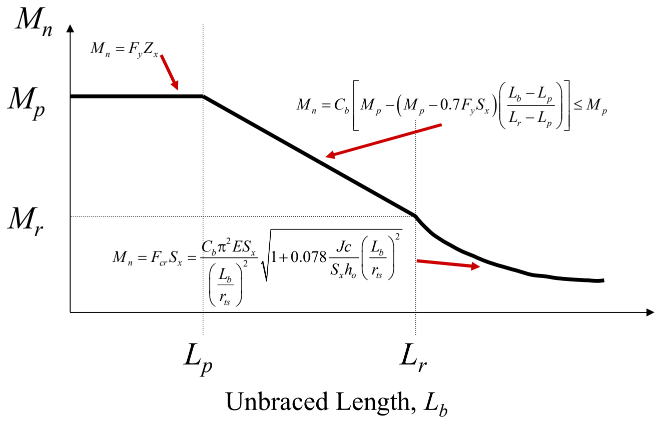
$$L_p = 1.76 r_y \sqrt{\frac{E}{F_y}} \qquad \text{(F2-5)}$$

Lateral-Torsional Buckling

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

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

Lateral-Torsional Buckling



Lateral-Torsional Buckling

Rewrite Eq. F2-2 to simplify

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

becomes

$$M_n = C_b\left[M_p - \boxed{\frac{\left(M_p - 0.7F_yS_x\right)}{\left(L_r - L_p\right)}}\left(L_b - L_p\right)\right] \le M_p$$

Taking

$$BF = \frac{\left(M_p - 0.7F_yS_x\right)}{\left(L_r - L_p\right)}$$

Lateral-Torsional Buckling

- Thus,

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

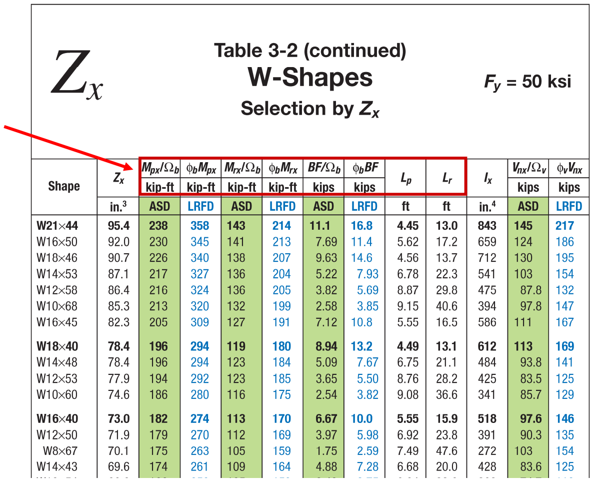
and the available strength is

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

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

Design Aid

Tabulate the terms in these equations



Z_x

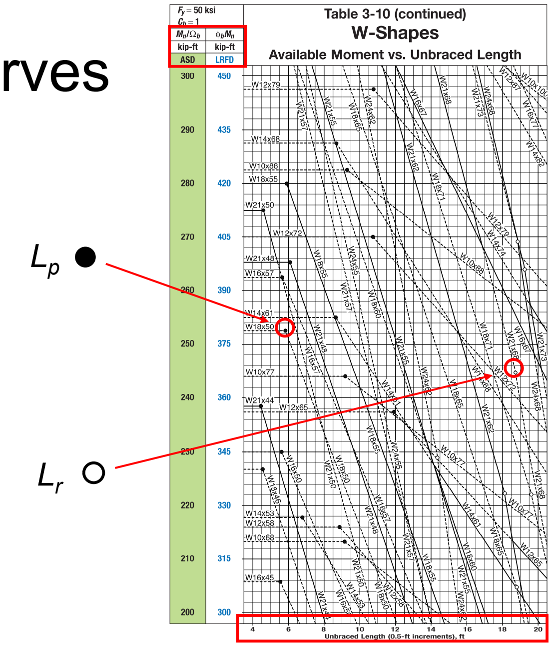
Table 3-2 (continued)
W-Shapes
Selection by Z_x

F_y = 50 ksi

Shape	Z_x	M_{px}/Ω_b	$\phi_b M_{px}$	M_{rx}/Ω_b	$\phi_b M_{rx}$	BF/Ω_b	$\phi_b BF$	L_p	L_r	I_x	V_{nx}/Ω_v	$\phi_v V_{nx}$
		kip-ft	kip-ft	kip-ft	kip-ft	kips	kips				kips	kips
	in.^{3}	ASD	LRFD	ASD	LRFD	ASD	LRFD	ft	ft	in.^{4}	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
W12×58	86.4	216	324	136	205	3.82	5.69	8.87	29.8	475	87.8	132
W10×68	85.3	213	320	132	199	2.58	3.85	9.15	40.6	394	97.8	147
W16×45	82.3	205	309	127	191	7.12	10.8	5.55	16.5	586	111	167
W18×40	**78.4**	**196**	**294**	**119**	**180**	**8.94**	**13.2**	**4.49**	**13.1**	**612**	**113**	**169**
W14×48	78.4	196	294	123	184	5.09	7.67	6.75	21.1	484	93.8	141
W12×53	77.9	194	292	123	185	3.65	5.50	8.76	28.2	425	83.5	125
W10×60	74.6	186	280	116	175	2.54	3.82	9.08	36.6	341	85.7	129
W16×40	**73.0**	**182**	**274**	**113**	**170**	**6.67**	**10.0**	**5.55**	**15.9**	**518**	**97.6**	**146**
W12×50	71.9	179	270	112	169	3.97	5.98	6.92	23.8	391	90.3	135
W8×67	70.1	175	263	105	159	1.75	2.59	7.49	47.6	272	103	154
W14×43	69.6	174	261	109	164	4.88	7.28	6.68	20.0	428	83.6	125

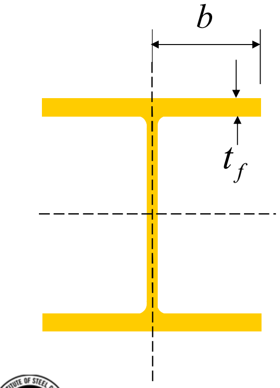
Beam Curves

Another design aid that will assist in design considering unbraced length



Slender Elements

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



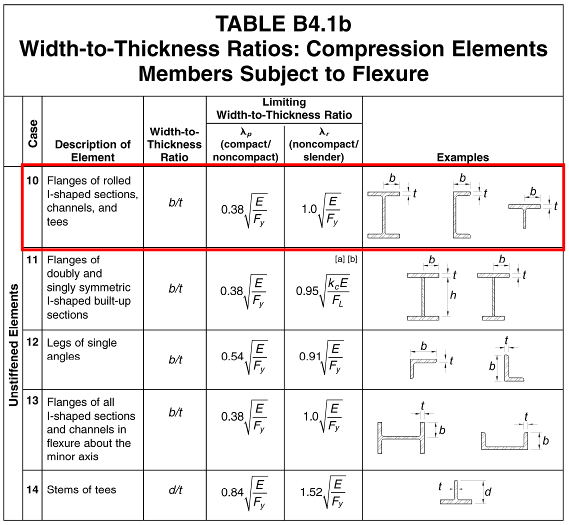
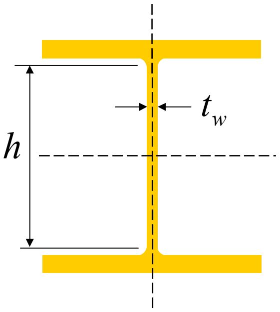


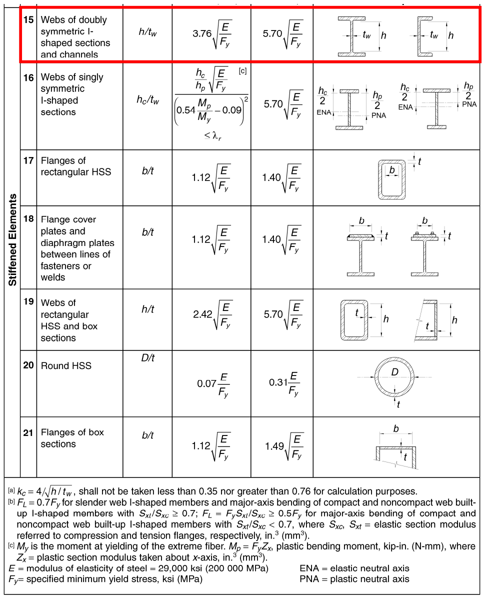
TABLE B4.1b
Width-to-Thickness Ratios: Compression Elements Members Subject to Flexure

	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	$0.38\sqrt{\frac{E}{F_y}}$	$1.0\sqrt{\frac{E}{F_y}}$	b t
	11	Flanges of doubly and singly symmetric I-shaped built-up sections	b/t	$0.38\sqrt{\frac{E}{F_y}}$	[a] [b] $0.95\sqrt{\frac{k_cE}{F_L}}$	b t h
	12	Legs of single angles	b/t	$0.54\sqrt{\frac{E}{F_y}}$	$0.91\sqrt{\frac{E}{F_y}}$	b t
	13	Flanges of all I-shaped sections and channels in flexure about the minor axis	b/t	$0.38\sqrt{\frac{E}{F_y}}$	$1.0\sqrt{\frac{E}{F_y}}$	t b
	14	Stems of tees	d/t	$0.84\sqrt{\frac{E}{F_y}}$	$1.52\sqrt{\frac{E}{F_y}}$	t d

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_c/t_w	$\frac{\frac{h_c}{h_p}\sqrt{\frac{E}{F_y}}}{\left(0.54\frac{M_p}{M_y}-0.09\right)^2} \le \lambda_r$ [c]	$5.70\sqrt{\frac{E}{F_y}}$	
	17	Flanges of rectangular HSS	b/t	$1.12\sqrt{\frac{E}{F_y}}$	$1.40\sqrt{\frac{E}{F_y}}$	
	18	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	$0.07\frac{E}{F_y}$	$0.31\frac{E}{F_y}$	
	21	Flanges of box sections	b/t	$1.12\sqrt{\frac{E}{F_y}}$	$1.49\sqrt{\frac{E}{F_y}}$	

[a] $k_c = 4/\sqrt{h/t_w}$, shall not be taken less than 0.35 nor greater than 0.76 for calculation purposes.

[b] $F_L = 0.7F_y$ for slender web I-shaped members and major-axis bending of compact and noncompact web built-up I-shaped members with $S_{xt}/S_{xc} \ge 0.7$; $F_L = F_yS_{xt}/S_{xc} \ge 0.5F_y$ for major-axis bending of compact and noncompact web built-up I-shaped members with $S_{xt}/S_{xc} < 0.7$, where S_{xc}, S_{xt} = elastic section modulus referred to compression and tension flanges, respectively, in.^{3} (mm^3).

[c] M_y is the moment at yielding of the extreme fiber. $M_p = F_yZ_x$, plastic bending moment, kip-in. (N-mm), where Z_x = plastic section modulus taken about x-axis, in.^{3} (mm^3).

E = modulus of elasticity of steel = 29,000 ksi (200 000 MPa)
F_y= specified minimum yield stress, ksi (MPa)
ENA = elastic neutral axis
PNA = plastic neutral axis

Flange Local Buckling

- Compact W-shape

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

- Noncompact W-shape

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

Flange Local Buckling

- Slender W-shape

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

Flange Local Buckling

Nominal Strength

- Compact $M_n = M_p = F_y Z_x$ (F2-1)
- Slender $M_n = \dfrac{0.9Ek_cS_x}{\lambda^2}$ (F3-2)

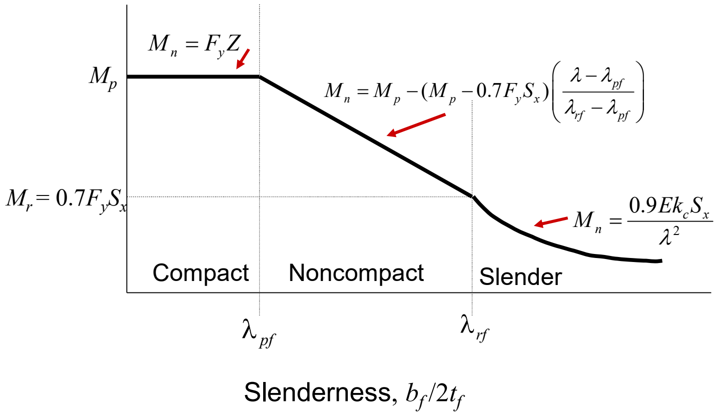
with $k_c = \dfrac{4}{\sqrt{h/t_w}}$ But shall not be taken less than 0.35 nor greater than 0.76 for calculation purposes

Flange Local Buckling

- Noncompact

$$M_n = M_p - (M_p - 0.7F_yS_x)\left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right) \qquad \text{(F3-1)}$$

Flange Local Buckling



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).

Flange Local Buckling

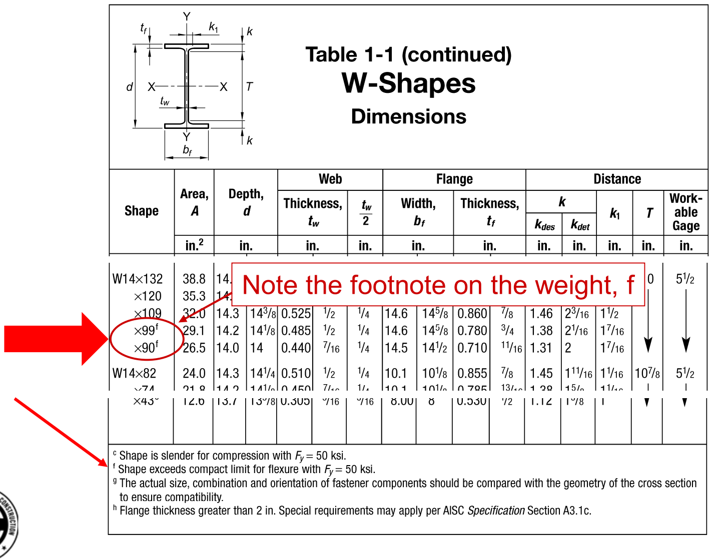


Table 1-1 (continued)
W-Shapes
Dimensions

Shape	Area, A	Depth, d		Web			Flange				Distance				
				Thickness, t_w		$\frac{t_w}{2}$	Width, b_f		Thickness, t_f		k_{des}	k_{det}	k_1	T	Workable Gage
	in.^{2}	in.		in.		in.	in.		in.		in.	in.	in.	in.	in.
W14×132	38.8	14.												0	5½
×120	35.3	14.													
×109	32.0	14.3	14⅜	0.525	½	¼	14.6	14⅝	0.860	⅞	1.46	2$\frac{3}{16}$	1½		
×99^f	29.1	14.2	14⅛	0.485	½	¼	14.6	14⅝	0.780	¾	1.38	2$\frac{1}{16}$	1$\frac{7}{16}$		
×90^f	26.5	14.0	14	0.440	$\frac{7}{16}$	¼	14.5	14½	0.710	$\frac{11}{16}$	1.31	2	1$\frac{7}{16}$		
W14×82	24.0	14.3	14¼	0.510	½	¼	10.1	10⅛	0.855	⅞	1.45	1$\frac{11}{16}$	1$\frac{1}{16}$	10⅞	5½
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]		
×43^c	12.6	13.7	[illegible]	0.305	[illegible]	[illegible]	8.00	8	0.530	½	1.12	[illegible]	1		

[c] Shape is slender for compression with $F_y = 50$ ksi.
[f] Shape exceeds 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 greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Flange Local Buckling

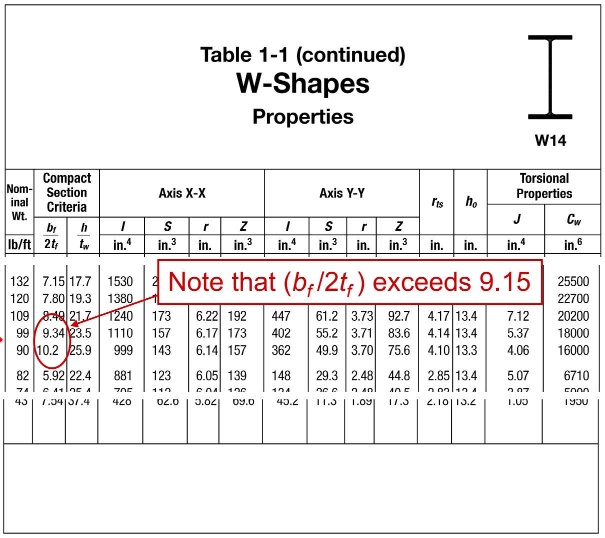


Table 1-1 (continued)
W-Shapes
Properties

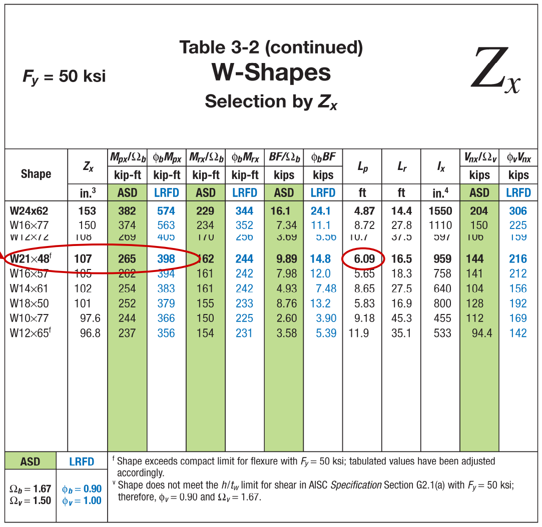
W14

Nom-inal 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}
132	7.15	17.7	1530	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	25500
120	7.80	19.3	1380	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	22700
109	8.49	21.7	1240	173	6.22	192	447	61.2	3.73	92.7	4.17	13.4	7.12	20200
99	9.34	23.5	1110	157	6.17	173	402	55.2	3.71	83.6	4.14	13.4	5.37	18000
90	10.2	25.9	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	123	6.05	139	148	29.3	2.48	44.8	2.85	13.4	5.07	6710
43	7.54	37.4	428	62.6	5.82	69.6	45.2	11.3	1.89	17.3	2.18	13.2	1.05	1950

Note that ($b_f/2t_f$) exceeds 9.15

Flange Local Buckling
Impact on Lateral-Torsional Buckling

Not compact



F_y = 50 ksi

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

Z_x

Shape	Z_x	M_{px}/Ω_b	$\phi_b M_{px}$	M_{rx}/Ω_b	$\phi_b M_{rx}$	BF/Ω_b	$\phi_b BF$	L_p	L_r	I_x	V_{nx}/Ω_v	$\phi_v V_{nx}$
		kip-ft	kip-ft	kip-ft	kip-ft	kips	kips				kips	kips
	in.^{3}	ASD	LRFD	ASD	LRFD	ASD	LRFD	ft	ft	in.^{4}	ASD	LRFD
W24x62	**153**	**382**	**574**	**229**	**344**	**16.1**	**24.1**	**4.87**	**14.4**	**1550**	**204**	**306**
W16×77	150	374	563	234	352	7.34	11.1	8.72	27.8	1110	150	225
W12×72	108	269	405	170	256	3.69	5.56	10.7	37.5	597	106	159
W21×48^f	**107**	**265**	**398**	**162**	**244**	**9.89**	**14.8**	**6.09**	**16.5**	**959**	**144**	**216**
W16×57	105	262	394	161	242	7.98	12.0	5.65	18.3	758	141	212
W14×61	102	254	383	161	242	4.93	7.48	8.65	27.5	640	104	156
W18×50	101	252	379	155	233	8.76	13.2	5.83	16.9	800	128	192
W10×77	97.6	244	366	150	225	2.60	3.90	9.18	45.3	455	112	169
W12×65^f	96.8	237	356	154	231	3.58	5.39	11.9	35.1	533	94.4	142

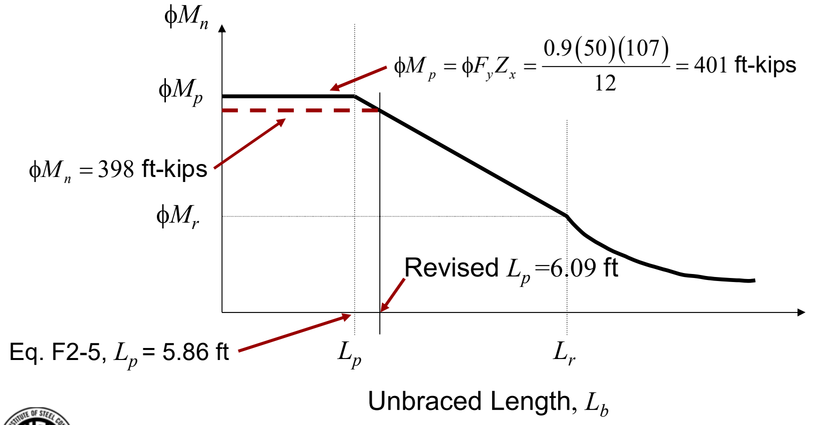
ASD	LRFD
$\Omega_b = 1.67$ $\Omega_v = 1.50$	$\phi_b = 0.90$ $\phi_v = 1.00$

[f] Shape exceeds compact limit for flexure with $F_y = 50$ ksi; tabulated values have been adjusted accordingly.

[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with $F_y = 50$ ksi; therefore, $\phi_v = 0.90$ and $\Omega_v = 1.67$.

Flange Local Buckling
Impact on Lateral-Torsional Buckling

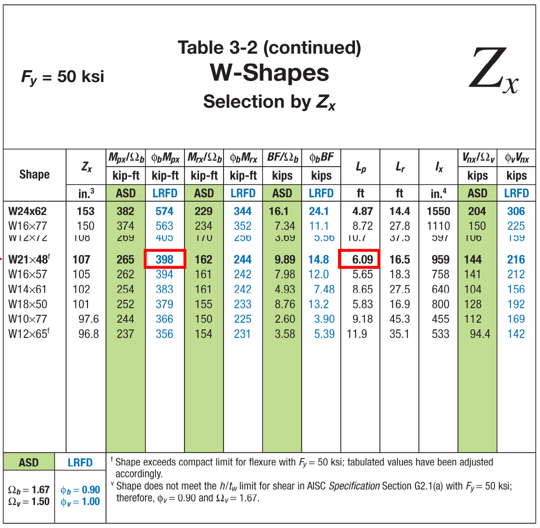
W21x48



Flange Local Buckling

Table 3-2 accounts for noncompact flange and unbraced length

ϕM_n = 398 ft-kips
L_p = 6.09 ft



F_y = 50 ksi

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

Z_x

Shape	Z_x	M_{px}/Ω_b	$\phi_b M_{px}$	M_{rx}/Ω_b	$\phi_b M_{rx}$	BF/Ω_b	$\phi_b BF$	L_p	L_r	I_x	V_{nx}/Ω_v	$\phi_v V_{nx}$
		kip-ft	kip-ft	kip-ft	kip-ft	kips	kips				kips	kips
	in.^{3}	ASD	LRFD	ASD	LRFD	ASD	LRFD	ft	ft	in.^{4}	ASD	LRFD
W24x62	**153**	**382**	**574**	**229**	**344**	**16.1**	**24.1**	**4.87**	**14.4**	**1550**	**204**	**306**
W16×77	150	374	563	234	352	7.34	11.1	8.72	27.8	1110	150	225
W12×72	108	269	405	170	256	3.69	5.56	10.7	37.5	597	106	159
W21×48^f	**107**	**265**	**398**	**162**	**244**	**9.89**	**14.8**	**6.09**	**16.5**	**959**	**144**	**216**
W16×57	105	262	394	161	242	7.98	12.0	5.65	18.3	758	141	212
W14×61	102	254	383	161	242	4.93	7.48	8.65	27.5	640	104	156
W18×50	101	252	379	155	233	8.76	13.2	5.83	16.9	800	128	192
W10×77	97.6	244	366	150	225	2.60	3.90	9.18	45.3	455	112	169
W12×65^f	96.8	237	356	154	231	3.58	5.39	11.9	35.1	533	94.4	142

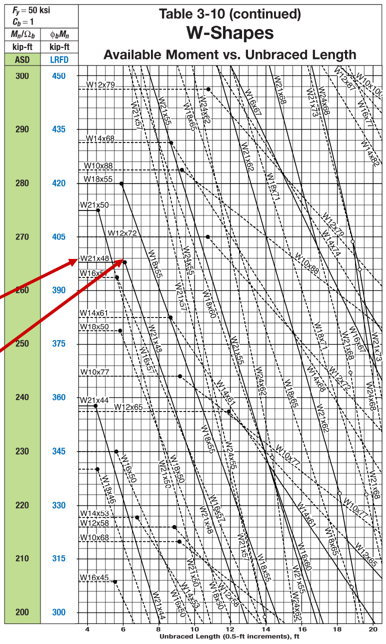
ASD	LRFD
$\Omega_b = 1.67$ $\Omega_v = 1.50$	$\phi_b = 0.90$ $\phi_v = 1.00$

[f] Shape exceeds compact limit for flexure with $F_y = 50$ ksi; tabulated values have been adjusted accordingly.

[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with $F_y = 50$ ksi; therefore, $\phi_v = 0.90$ and $\Omega_v = 1.67$.

Beam Curves

Unbraced length charts for beams



W21x48 ϕM_n = 398 ft-kips

L_p = 6.09 ft

This is a noncompact flange shape

Web Local Buckling

- Limits W-shape, A992

$$\lambda_p = 3.76\sqrt{\frac{E}{F_y}}$$

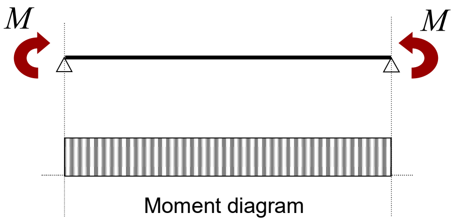
$$\lambda_p = 90.5$$

- All W-shapes have compact webs for A992
- Web local buckling then is only applicable to built-up members which we will not consider.

Lateral-Torsional Buckling

C_b in equations F2-2 and F2-4 accounts for nonuniform moment diagrams between bracing points.

For uniform moment along an unbraced segment, $C_b = 1.0$



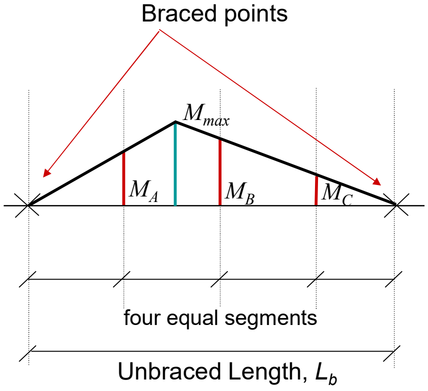
Lateral-Torsional Buckling

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

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

Lateral-Torsional Buckling

- Moment diagram over unbraced length



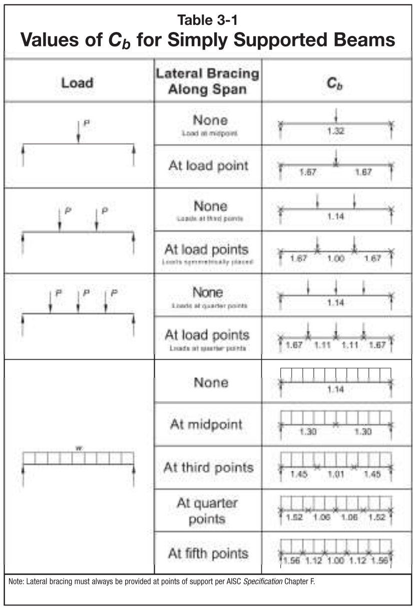
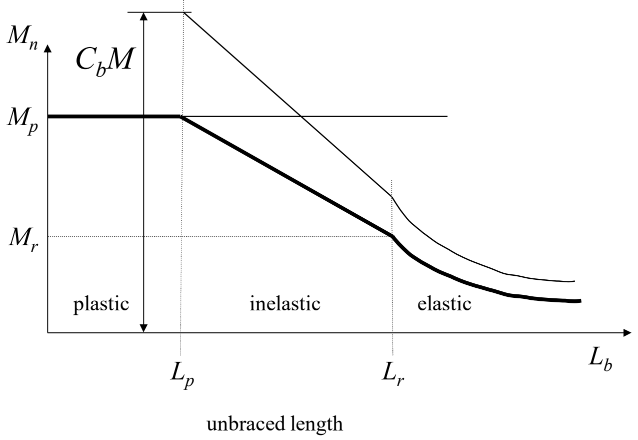


Table 3-1
Values of C_b for Simply Supported Beams

Load	Lateral Bracing Along Span	C_b
P	None Load at midpoint	1.32
	At load point	1.67 1.67
P P	None Loads at third points	1.14
	At load points Loads symmetrically placed	1.67 1.00 1.67
P P P	None Loads at quarter points	1.14
	At load points Loads at quarter points	1.67 1.11 1.11 1.67
w	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	1.56 1.12 1.00 1.12 1.56

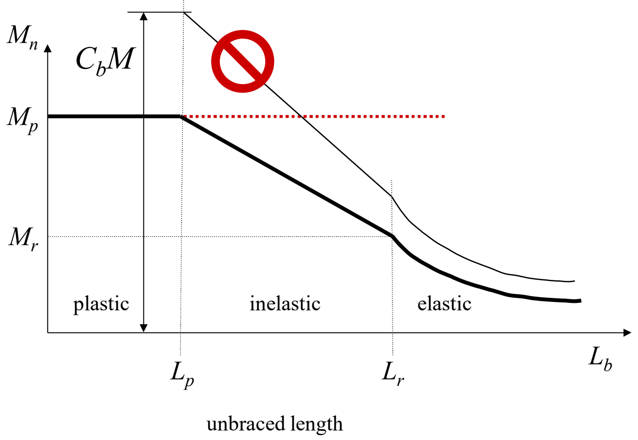
Note: Lateral bracing must always be provided at points of support per AISC *Specification* Chapter F.

Limit on use of C_b



L3.63

Limit on use of C_b



L3.64

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

Example 1(ASD)

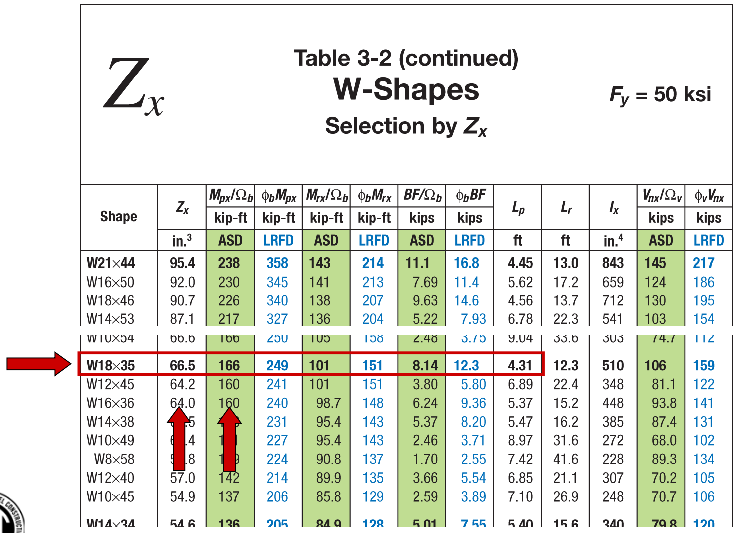
$$P_a = 8.0 + 24.0 = 32.0 \text{ kips}$$

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

for a compact, fully braced section

$$Z_{req} = \frac{160(12)}{(50/1.67)} = 64.0 \text{ in}^3$$

Example 1(ASD)



Z_x

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

F_y = 50 ksi

Shape	Z_x	M_{px}/Ω_b	$\phi_b M_{px}$	M_{rx}/Ω_b	$\phi_b M_{rx}$	BF/Ω_b	$\phi_b BF$	L_p	L_r	I_x	V_{nx}/Ω_v	$\phi_v V_{nx}$
		kip-ft	kip-ft	kip-ft	kip-ft	kips	kips				kips	kips
	in.^{3}	ASD	LRFD	ASD	LRFD	ASD	LRFD	ft	ft	in.^{4}	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
W10×54	66.6	166	250	105	158	2.48	3.75	9.04	33.6	303	74.7	112
W18×35	**66.5**	**166**	**249**	**101**	**151**	**8.14**	**12.3**	**4.31**	**12.3**	**510**	**106**	**159**
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	[illegible]	[illegible]	231	95.4	143	5.37	8.20	5.47	16.2	385	87.4	131
W10×49	[illegible]	[illegible]	227	95.4	143	2.46	3.71	8.97	31.6	272	68.0	102
W8×58	[illegible]	[illegible]	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**	**84.9**	**128**	**5.01**	**7.55**	**5.40**	**15.6**	**340**	**79.8**	**120**

Example 1(ASD)

Select W18x35

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

$$M_n / \Omega_b = 166 \text{ ft-kips} > 160 \text{ ft-kips}$$

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

Example 1(LRFD)

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

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

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

Example 1(LRFD)

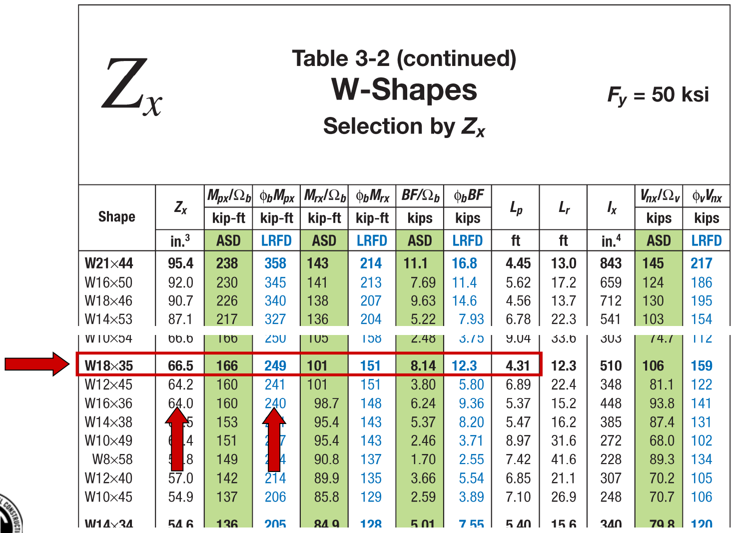
$$P_u = 1.2(8.0) + 1.6(24.0) = 48.0 \text{ kips}$$

$$M_u = \frac{48.0(20)}{4} = 240 \text{ ft-kips}$$

for a compact, fully braced section

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

Example 1(LRFD)



Z_x

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

F_y = 50 ksi

Shape	Z_x	M_{px}/Ω_b	$\phi_b M_{px}$	M_{rx}/Ω_b	$\phi_b M_{rx}$	BF/Ω_b	$\phi_b BF$	L_p	L_r	I_x	V_{nx}/Ω_v	$\phi_v V_{nx}$
		kip-ft	kip-ft	kip-ft	kip-ft	kips	kips				kips	kips
	in.^{3}	ASD	LRFD	ASD	LRFD	ASD	LRFD	ft	ft	in.^{4}	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
W10×54	66.6	166	250	105	158	2.48	3.75	9.04	33.6	303	74.7	112
W18×35	**66.5**	**166**	**249**	**101**	**151**	**8.14**	**12.3**	**4.31**	**12.3**	**510**	**106**	**159**
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	[illegible]	153	[illegible]	95.4	143	5.37	8.20	5.47	16.2	385	87.4	131
W10×49	[illegible]	151	[illegible]	95.4	143	2.46	3.71	8.97	31.6	272	68.0	102
W8×58	[illegible]	149	[illegible]	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**	**84.9**	**128**	**5.01**	**7.55**	**5.40**	**15.6**	**340**	**79.8**	**120**

Example 1(LRFD)

Select W18x35

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

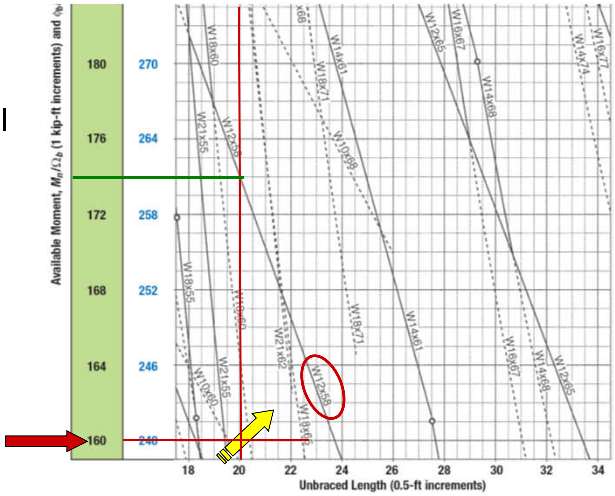
$$\phi_b M_p = 249 \text{ ft-kips} > 240 \text{ ft-kips}$$

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

Example 2 (ASD)

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

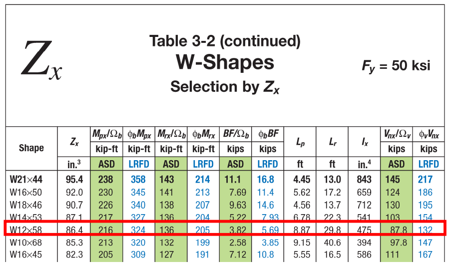
$$M_a = 160 \text{ kip-ft}$$



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 ksi

Shape	Z_x	M_{px}/Ω_b	$\phi_b M_{px}$	M_{rx}/Ω_b	$\phi_b M_{rx}$	BF/Ω_b	$\phi_b BF$	L_p	L_r	I_x	V_{nx}/Ω_v	$\phi_v V_{nx}$
		kip-ft	kip-ft	kip-ft	kip-ft	kips	kips				kips	kips
	in.^{3}	ASD	LRFD	ASD	LRFD	ASD	LRFD	ft	ft	in.^{4}	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
W12×58	86.4	216	324	136	205	3.82	5.69	8.87	29.8	475	87.8	132
W10×68	85.3	213	320	132	199	2.58	3.85	9.15	40.6	394	97.8	147
W16×45	82.3	205	309	127	191	7.12	10.8	5.55	16.5	586	111	167

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

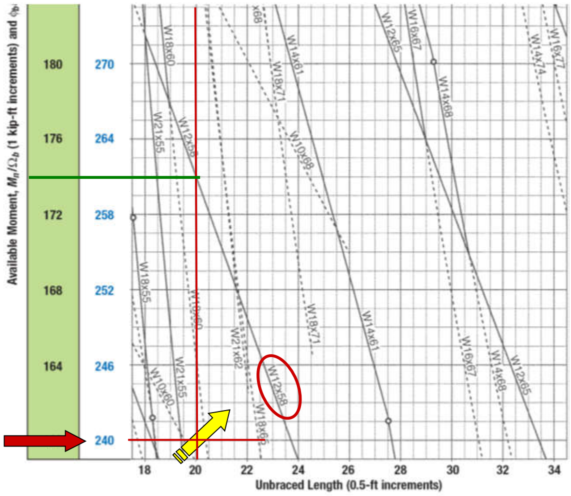
$$\frac{M_n}{\Omega_b} = 1.0\left[216 - 3.82\left(20 - 8.87\right)\right] = 173 \text{ ft-kips}$$

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

Example 2 (LRFD)

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

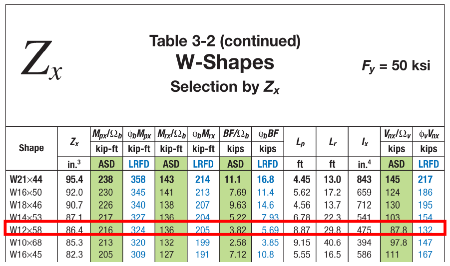
$M_u = 240$ kip-ft



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	M_{px}/Ω_b	$\phi_b M_{px}$	M_{rx}/Ω_b	$\phi_b M_{rx}$	BF/Ω_b	$\phi_b BF$	L_p	L_r	I_x	V_{nx}/Ω_v	$\phi_v V_{nx}$
		kip-ft	kip-ft	kip-ft	kip-ft	kips	kips				kips	kips
	in.^{3}	ASD	LRFD	ASD	LRFD	ASD	LRFD	ft	ft	in.^{4}	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
W12×58	86.4	216	324	136	205	3.82	5.69	8.87	29.8	475	87.8	132
W10×68	85.3	213	320	132	199	2.58	3.85	9.15	40.6	394	97.8	147
W16×45	82.3	205	309	127	191	7.12	10.8	5.55	16.5	586	111	167

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

$$\phi M_n = 1.0\left[324 - 5.69\left(20 - 8.87\right)\right] = 261 \text{ ft-kips}$$

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

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

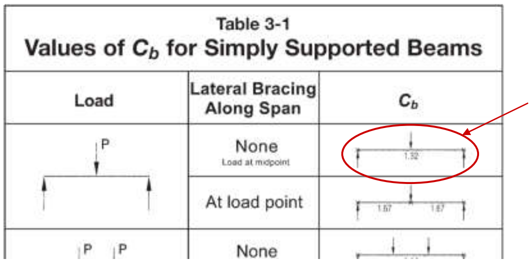


Table 3-1
Values of C_b for Simply Supported Beams

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

$C_b = 1.32$

Example 3 (ASD)

- Using C_b, determine a modified required moment strength

$$C_b = 1.32$$

$$M_a = 160/1.32 = 121 \text{ ft-kips}$$

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

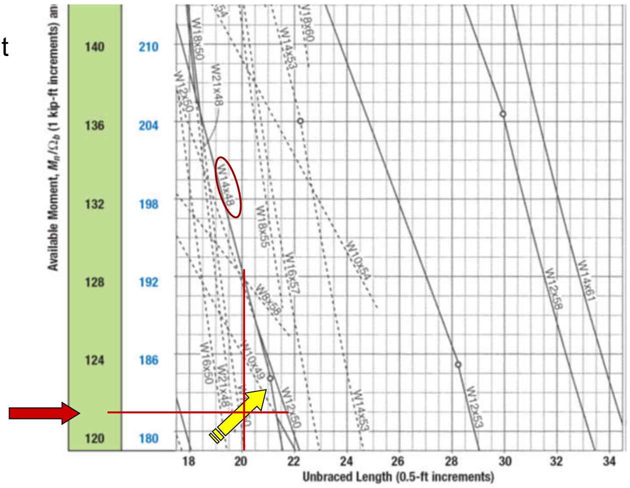
Example 3 (ASD)

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

$$\frac{M_p}{\Omega_b} = 196$$

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

$$\therefore \text{ ok}$$



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

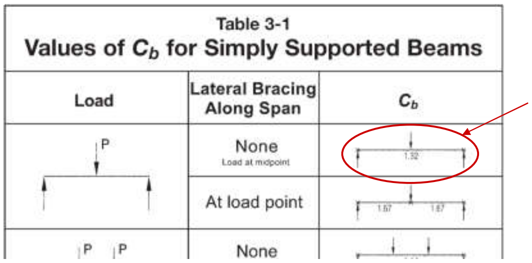


Table 3-1
Values of C_b for Simply Supported Beams

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

$C_b = 1.32$

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

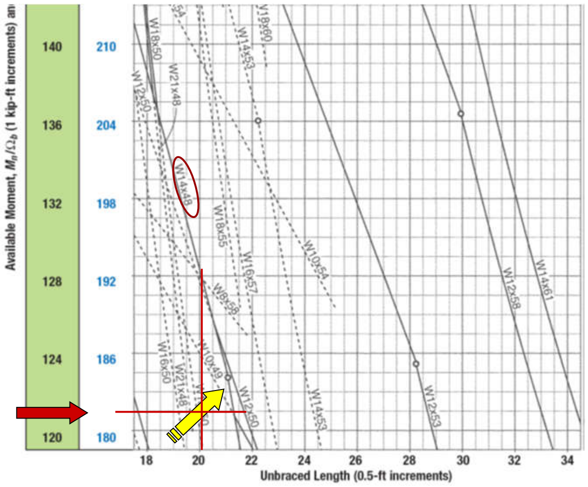
Example 3 (LRFD)

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

$$\phi M_p = 294$$

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

$$\therefore \text{ ok}$$



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

Tees and Double Angles

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

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

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

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

Tees

- F9.1. Yielding

$$M_n = M_p \qquad \text{(F9-1)}$$

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

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

(b) For tee stems in compression ⊥

$$M_p = M_y \qquad \text{(F9-4)}$$

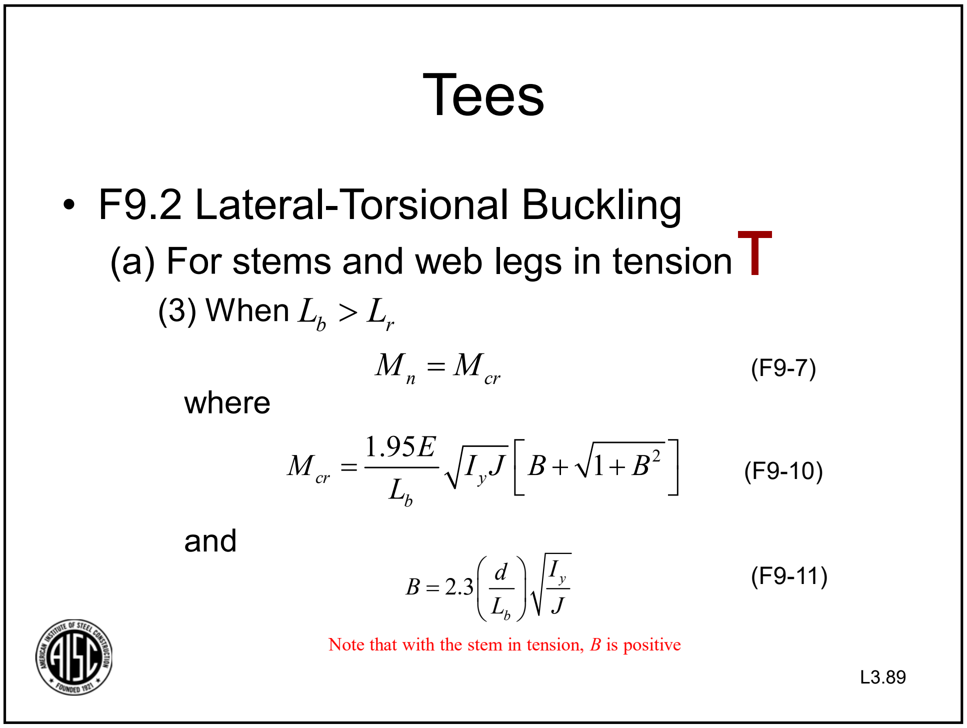
Tees

- 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 $L_p < L_b \leq L_r$

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



Tees

- F9.2 Lateral-Torsional Buckling
  - (a) For stems and web legs in tension T
    - (3) When $L_b > L_r$

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

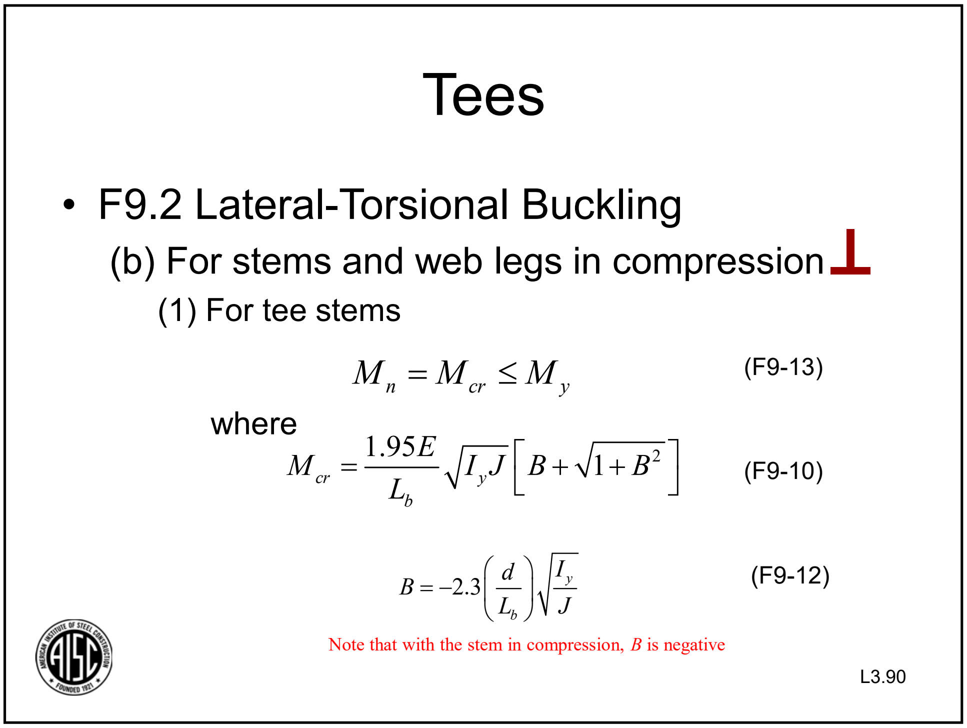
where

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

and

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

Note that with the stem in tension, B is positive



Tees

- F9.2 Lateral-Torsional Buckling
  - (b) For stems and web legs in compression ⊥
    - (1) For tee stems

$$M_n = M_{cr} \le M_y \qquad \text{(F9-13)}$$

where

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

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

Note that with the stem in compression, B is negative

Tees

- F9.3. Flange Local Buckling T
  - (a) For tee flanges
    - (1) Compact flange

      FLB does not apply
    - (2) Noncompact flange

$$M_n = M_p - \left(M_p - 0.7F_yS_{xc}\right)\left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right) \le 1.6M_y \qquad \text{(F9-14)}$$

    - (3)) Slender flange

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

Tees

- F9.4. Stem Local Buckling ⊥
  - (a) For tee stems

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

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

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

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

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

Tees

- F9.4. Stem Local Buckling ⊥

  (a) For tee stems

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

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

Example 4

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

$A_g = 17.4$ in.^{2} $\qquad d = 16.4$ in.

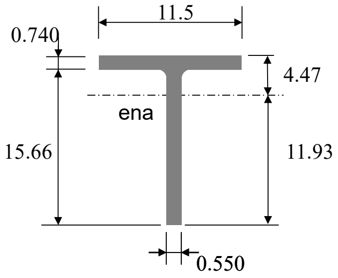
$S_x = 39.2$ in.^{3} $\qquad t_w = 0.550$ in.

$Z_x = 70.8$ in.^{3} $\qquad J = 2.64$ in.^{4}

$I_y = 93.5$ in.^{4} $\qquad r_y = 2.32$ in.

Example 4

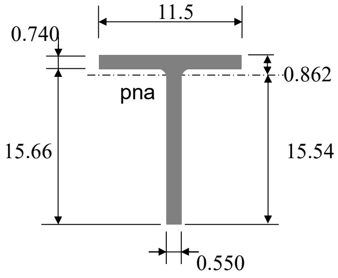
- Elastic Neutral Axis (ena)
  - Moment of Area above ena = Moment of Area below ena



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

Example 4

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



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

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

Example 4

- F9.1 Yielding

  (a) Tee stem in tension T

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

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

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

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

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

thus,

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

Example 4

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

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

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

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

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

Example 4

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

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

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

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

Example 4

- Flange local buckling T

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

  - Therefore it is compact
- Stem local buckling T
  - The stem is not in compression, therefore this limit state is not applicable

Example 4

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

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.

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

Shear

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

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

Shear

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

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

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

This means the web will yield in shear.

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),

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

For others, there are only 8 W-shapes,

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

Shear

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

  (i) when

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

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

This means the web will yield in shear.

Shear

- G2.1.(b)(1)

  (ii) when

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

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

This means the web will buckle inelastically.

Shear

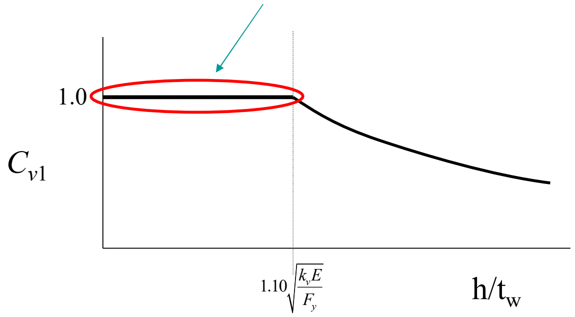
- Web plate shear buckling coefficient
  - For webs without transverse stiffeners

$$k_v = 5.34$$

- **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.

Shear

All W-shapes will fall in this range



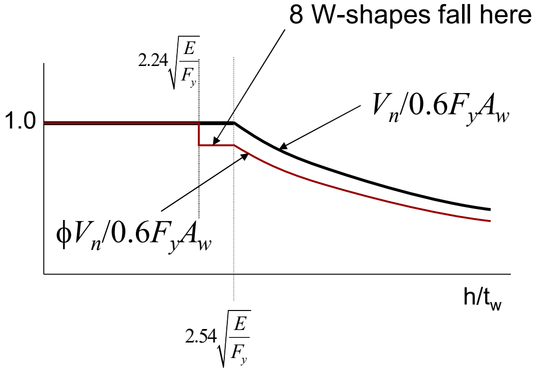
Shear

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

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

$\phi_v = 1.00$ (LRFD) $\Omega_v = 1.50$ (ASD)

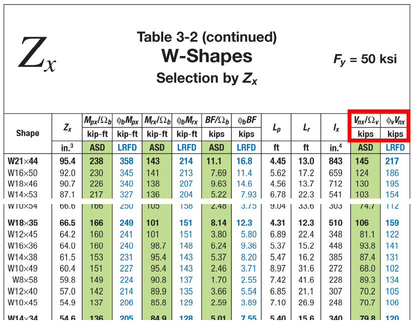
Shear



For W-shapes, k_v = 5.34

Manual

Z_x tables and uniform load tables show shear strength

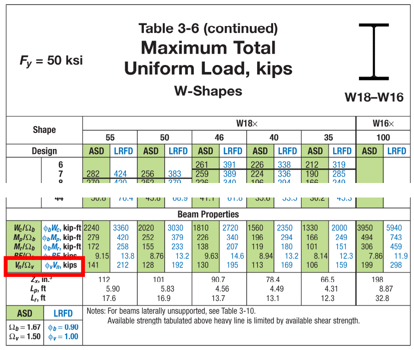


Z_x

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

F_y = 50 ksi

Shape	Z_x	M_{px}/Ω_b	$\phi_b M_{px}$	M_{rx}/Ω_b	$\phi_b M_{rx}$	BF/Ω_b	$\phi_b BF$	L_p	L_r	I_x	V_{nx}/Ω_v	$\phi_v V_{nx}$
		kip-ft	kip-ft	kip-ft	kip-ft	kips	kips				kips	kips
	in.^{3}	ASD	LRFD	ASD	LRFD	ASD	LRFD	ft	ft	in.^{4}	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
W10×54	66.6	166	250	105	158	2.48	3.75	9.04	33.6	303	74.7	112
W18×35	**66.5**	**166**	**249**	**101**	**151**	**8.14**	**12.3**	**4.31**	**12.3**	**510**	**106**	**159**
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	26.9	248	70.7	106
W14×34	**54.6**	**136**	**205**	**84.9**	**128**	**5.01**	**7.55**	**5.40**	**15.6**	**340**	**79.8**	**120**



F_y = 50 ksi

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

W18–W16

Shape		W18×										W16×	
		55		50		46		40		35		100	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
	6					261	391	226	338	212	319		
	7	282	424	256	383	259	389	224	336	190	285		
	8	279	420	252	379	226	340	196	294	166	249		
	44	50.8	76.4	45.8	68.9	41.1	61.8	35.6	53.5	30.2	45.3		
Beam Properties													
W_c/Ω_b	$\phi_b W_c$, kip-ft	2240	3360	2020	3030	1810	2720	1560	2350	1330	2000	3950	5940
M_p/Ω_b	$\phi_b M_p$, kip-ft	279	420	252	379	226	340	196	294	166	249	494	743
M_r/Ω_b	$\phi_b M_r$, kip-ft	172	258	155	233	138	207	119	180	101	151	306	459
BF/Ω_b	$\phi_b BF$, kips	9.15	13.8	8.76	13.2	9.63	14.6	8.94	13.2	8.14	12.3	7.86	11.9
V_n/Ω_v	$\phi_v V_n$, kips	141	212	128	192	130	195	113	169	106	159	199	298
Z_x, in.^{3}		112		101		90.7		78.4		66.5		198	
L_p, ft		5.90		5.83		4.56		4.49		4.31		8.87	
L_r, ft		17.6		16.9		13.7		13.1		12.3		32.8	

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.50$	$\phi_v = 1.00$

Notes: For beams laterally unsupported, see Table 3-10.
Available strength tabulated above heavy line is limited by available shear strength.

Example 4(ASD)

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

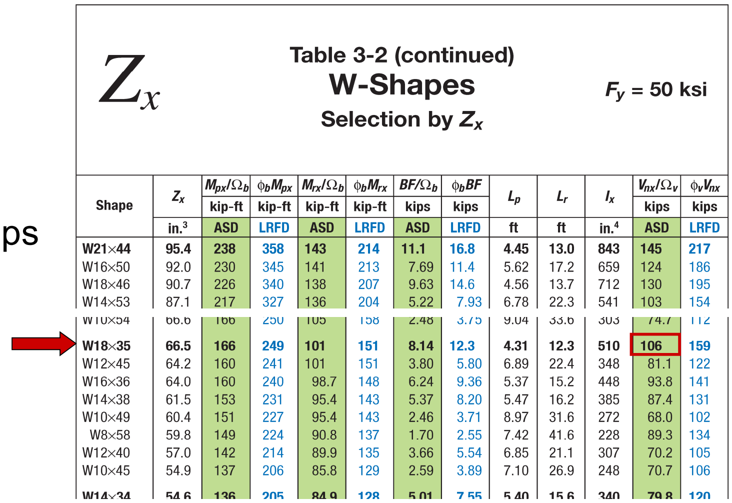
$$P_a = 8.0 + 24.0 = 32.0 \text{ kips}$$

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

Example 4(ASD)

$$V_a = 16.0 <$$

$$\frac{V_n}{\Omega} = 106 \text{ kips}$$



Z_x

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

F_y = 50 ksi

Shape	Z_x	M_{px}/Ω_b	$\phi_b M_{px}$	M_{rx}/Ω_b	$\phi_b M_{rx}$	BF/Ω_b	$\phi_b BF$	L_p	L_r	I_x	V_{nx}/Ω_v	$\phi_v V_{nx}$
		kip-ft	kip-ft	kip-ft	kip-ft	kips	kips				kips	kips
	in.^{3}	ASD	LRFD	ASD	LRFD	ASD	LRFD	ft	ft	in.^{4}	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
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W10×54	66.6	166	250	105	158	2.48	3.75	9.04	33.6	303	74.7	112
W18×35	**66.5**	**166**	**249**	**101**	**151**	**8.14**	**12.3**	**4.31**	**12.3**	**510**	**106**	**159**
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
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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**	**84.9**	**128**	**5.01**	**7.55**	**5.40**	**15.6**	**340**	**79.8**	**120**

Example 4(LRFD)

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

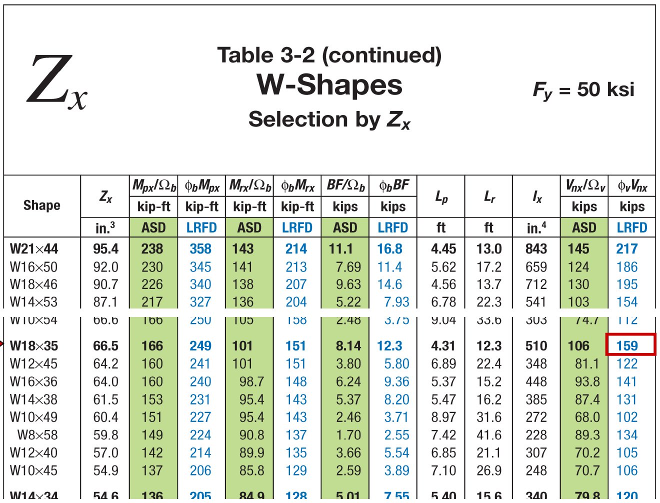
$$P_u = 1.2(8.0) + 1.6(24.0) = 48.0 \text{ kips}$$

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

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	M_{px}/Ω_b	$\phi_b M_{px}$	M_{rx}/Ω_b	$\phi_b M_{rx}$	BF/Ω_b	$\phi_b BF$	L_p	L_r	I_x	V_{nx}/Ω_v	$\phi_v V_{nx}$
		kip-ft	kip-ft	kip-ft	kip-ft	kips	kips				kips	kips
	in.^{3}	ASD	LRFD	ASD	LRFD	ASD	LRFD	ft	ft	in.^{4}	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
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W10×54	66.6	166	250	105	158	2.48	3.75	9.04	33.6	303	74.7	112
W18×35	**66.5**	**166**	**249**	**101**	**151**	**8.14**	**12.3**	**4.31**	**12.3**	**510**	**106**	**159**
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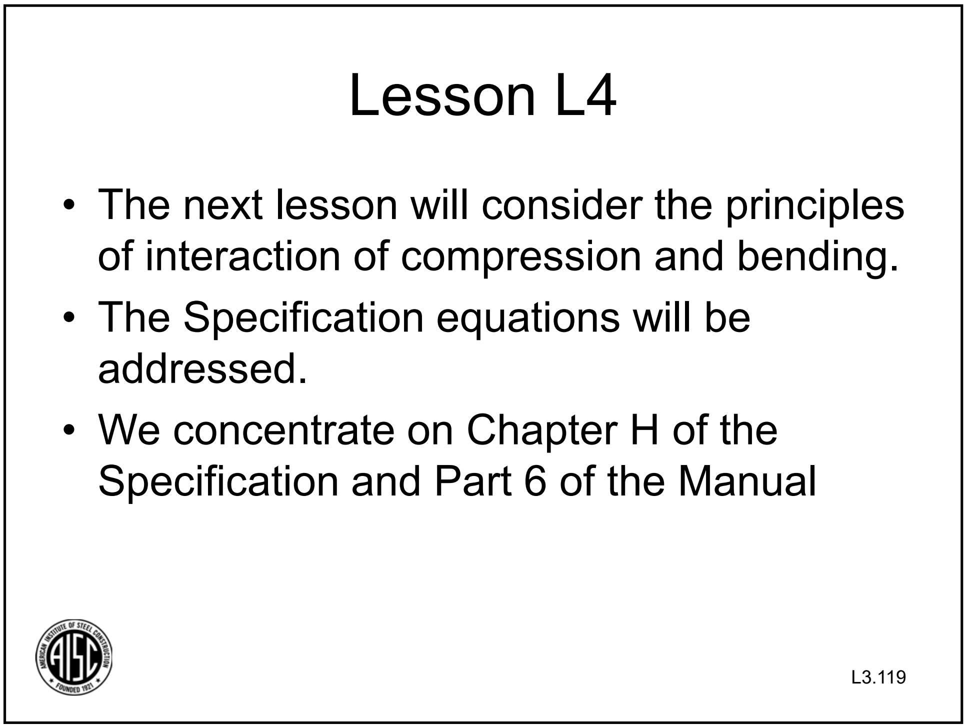
W14×34	**54.6**	**136**	**205**	**84.9**	**128**	**5.01**	**7.55**	**5.40**	**15.6**	**340**	**79.8**	**120**

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



Lesson L4

- 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

Thank You

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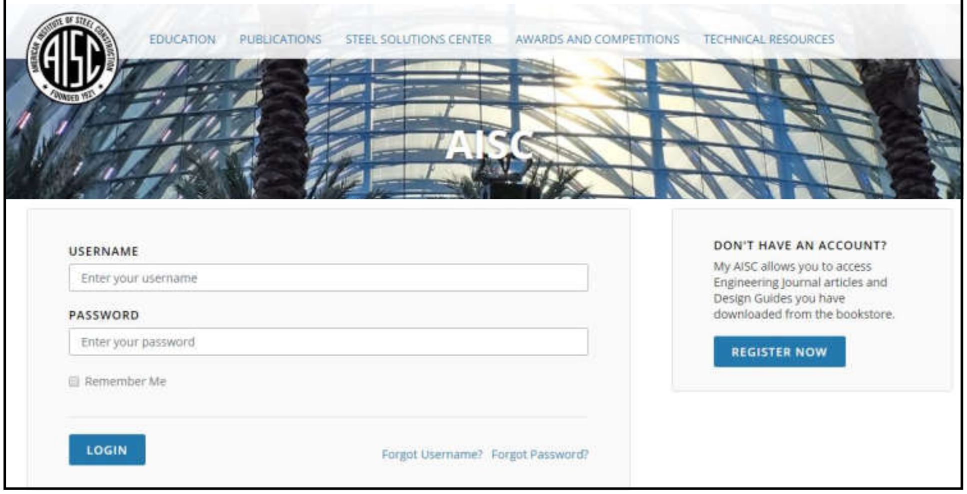

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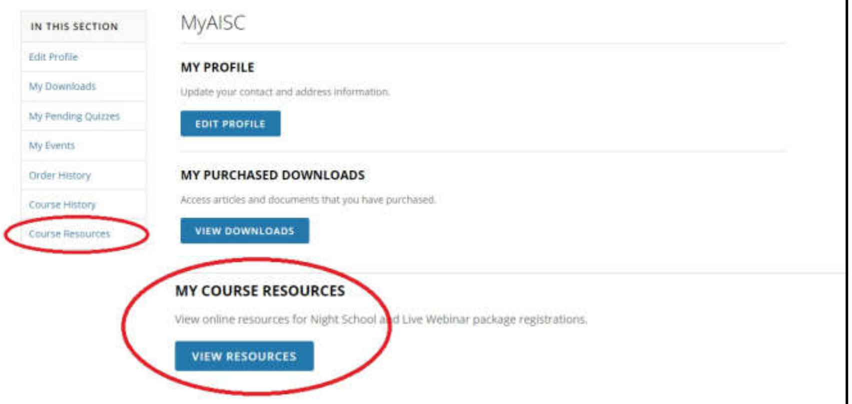

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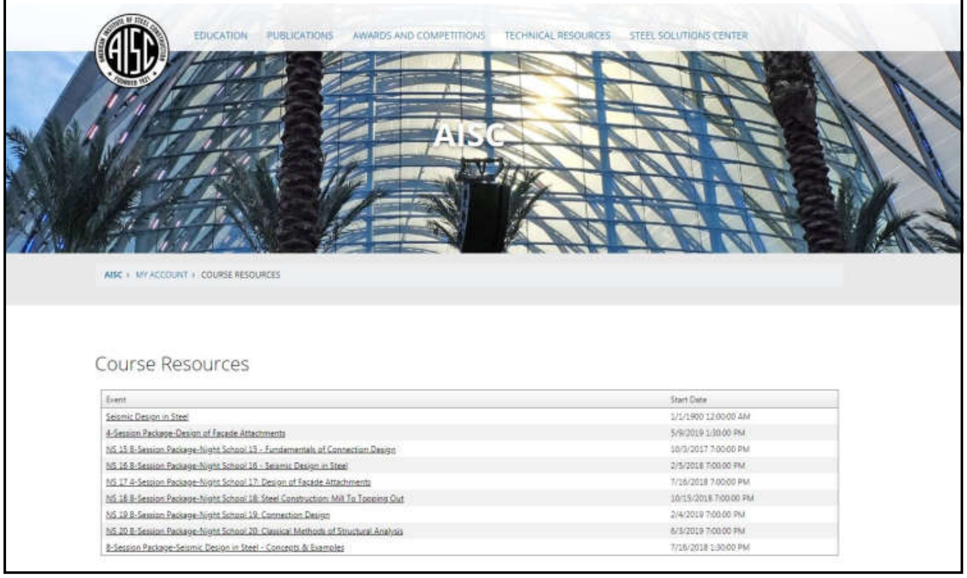
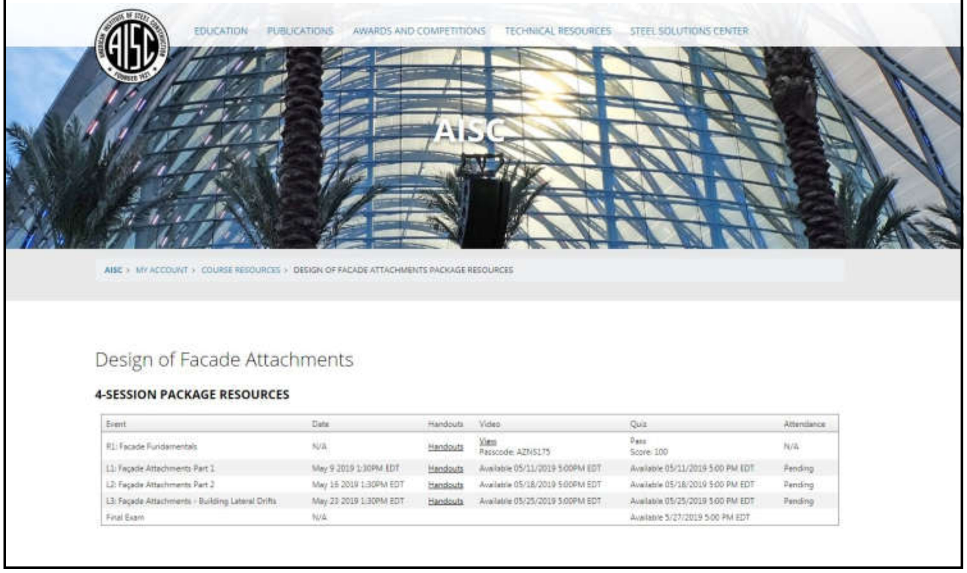


Course Resources

Event	Start Date
Seismic Design in Steel	1/1/1900 12:00:00 AM
4-Session Package-Design of Facade Attachments	5/9/2019 1:30:00 PM
NS 15 8-Session Package-Night School 15 - Fundamentals of Connection Design	10/3/2017 7:00:00 PM
NS 16 8-Session Package-Night School 16 - Seismic Design in Steel	2/5/2018 7:00:00 PM
NS 17 4-Session Package-Night School 17: Design of Facade Attachments	7/16/2018 7:00:00 PM
NS 18 8-Session Package-Night School 18: Steel Construction: Mill To Topping Out	10/15/2018 7:00:00 PM
NS 19 8-Session Package-Night School 19: Connection Design	2/4/2019 7:00:00 PM
NS 20 8-Session Package-Night School 20: Classical Methods of Structural Analysis	6/3/2019 7:00:00 PM
8-Session Package-Seismic Design in Steel - Concepts & Examples	7/16/2018 1:30:00 PM

Course Package Registrants

Course Resources



Design of Facade Attachments

4-SESSION PACKAGE RESOURCES

Event	Date	Handouts	Video	Quiz	Attendance
R1: Facade Fundamentals	N/A	Handouts	View Passcode: AZNS175	Pass Score: 100	N/A
L1: Façade Attachments Part 1	May 9 2019 1:30PM EDT	Handouts	Available 05/11/2019 5:00PM EDT	Available 05/11/2019 5:00 PM EDT	Pending
L2: Façade Attachments Part 2	May 16 2019 1:30PM EDT	Handouts	Available 05/18/2019 5:00PM EDT	Available 05/18/2019 5:00 PM EDT	Pending
L3: Façade Attachments - Building Lateral Drifts	May 23 2019 1:30PM EDT	Handouts	Available 05/25/2019 5:00PM EDT	Available 05/25/2019 5:00 PM EDT	Pending
Final Exam	N/A			Available 5/27/2019 5:00 PM EDT	

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