

AISC Live Webinars

Thank you for joining our live webinar today.
We will begin shortly. Please standby.

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



AISC Live Webinars

Today's live webinar will begin shortly. Please stand by.

Today's audio will be broadcast through the internet. Please be sure to turn up the volume on your speakers.

Please type any questions or comments in the Q&A window.



AISC Live Webinars

AIA Credit

AISC is a Registered Provider with The American Institute of Architects Continuing Education Systems (AIA/CES). Credit(s) earned on completion of this program will be reported to AIA/CES for AIA members. Certificates of Completion for both AIA members and non-AIA members are available upon request.

This program is registered with AIA/CES for continuing professional education. As such, it does not include content that may be deemed or construed to be an approval or endorsement by the AIA of any material of construction or any method or manner of handling, using, distributing, or dealing in any material or product.

Questions related to specific materials, methods, and services will be addressed at the conclusion of this presentation.



AISC Live Webinars

Copyright Materials

This presentation is protected by US and International Copyright laws. Reproduction, distribution, display and use of the presentation without written permission of AISC is prohibited.

© The American Institute of Steel Construction 2021

The information presented herein is based on recognized engineering principles and is for general information only. While it is believed to be accurate, this information should not be applied to any specific application without competent professional examination and verification by a licensed professional engineer. Anyone making use of this information assumes all liability arising from such use.



AISC Live Webinars

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.



AISC Live Webinars

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

Winter Webinar 2021
Lesson L3
Bending Members



L3.7

Lesson L3 – Bending

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



L3.8

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)



L3.9

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



L3.10

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.



L3.11

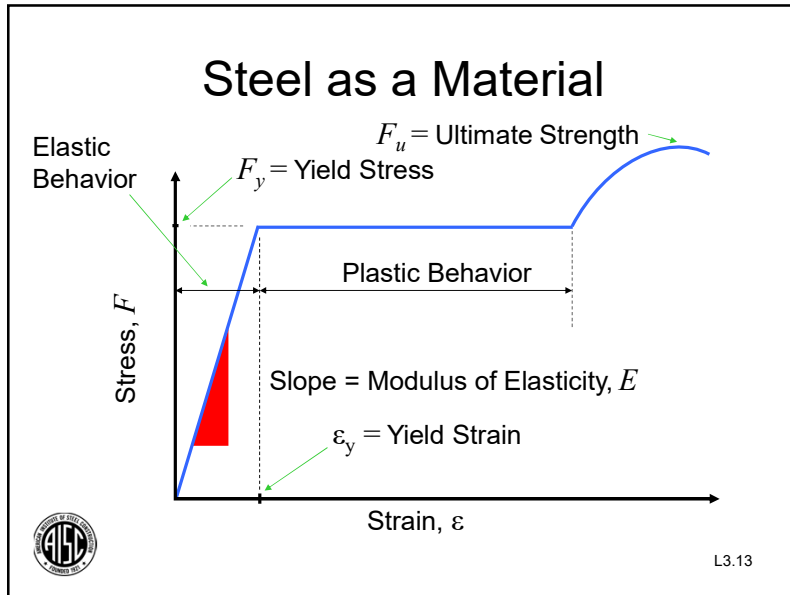
Bending Members

- 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.




L3.12



Bending Members


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



L3.14

Bending Members

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



L3.15

Bending Members

- Elastic response

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

$$E = \frac{f_a}{\epsilon_a} = \frac{F_y}{\epsilon_y}$$

Stress, f

Strain, ϵ

F_y

ϵ_y


Plastic Behavior

stress

strain

x

elastic neutral axis (ena)



L3.16

Bending Members

- Elastic response

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

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

L3.17

Bending Members

- Partial plastic response

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

L3.18

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

L3.19

Bending Members

- Determine the force in the flange and web

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

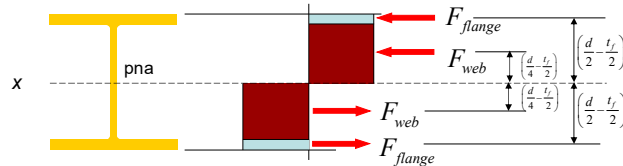
$$F_{web} = F_y \frac{A_{web}}{2} = F_y t_w \left(\frac{d}{2} - t_f \right)$$

L3.20



Bending Members

- Take moments about the plastic neutral axis



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



L3.21

Bending Members

- Simplifying

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

$$= F_y Z$$

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



L3.22

Bending Members

- How do we define the plastic neutral axis?

$$T = C$$

$$F_y A_T = F_y A_C$$

$$A_T = A_C$$

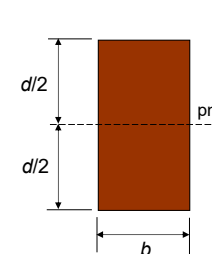
- Thus, the area above the pna must equal the area below the pna.



L3.23

Bending Members

- For a rectangle the pna is in the middle.



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

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

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

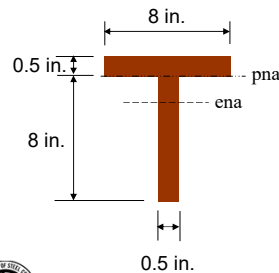
$$\beta = \frac{Z}{S} = \frac{\frac{bd^2}{4}}{\frac{bd^2}{6}} = 1.5 \quad \text{Shape Factor}$$



L3.24

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.

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

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

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



L3.25

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.



L3.26

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

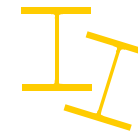
F2. User note: All current ASTM A6 W, S, M, C and MC shapes except W21x48, W14x99, W14x90, W12x65, W10x12, W8x31, W8x10, W6x15, W6x9, W6x8.5, and M4x6 have compact flanges for $F_y = 50$ ksi (345 MPa); all current ASTM A6 W, S, M, HP, C and MC shapes have compact webs at $F_y \leq 70$ ksi (485 MPa).



L3.27

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



L3.28

Bending Members

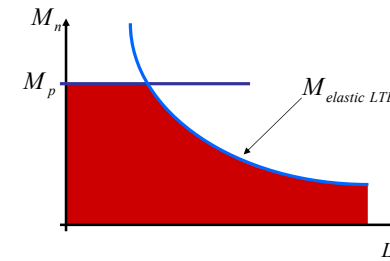
- To control lateral-torsional buckling the beam must be properly braced.
 - Supports must be restrained against twisting (a given)
 - Intermediate points along the span may have the compression flange braced against lateral translation.
 - Similar to column bracing but treated differently.
 - Intermediate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



L3.29

Bending Members

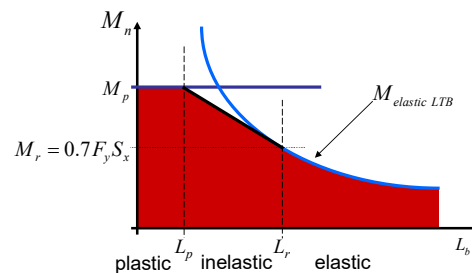
- If the member behaves either plastically or elastically



L3.30

Bending Members

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



L3.31

Lateral-Torsional Buckling

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

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



L3.32

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

W21

$F_y = 50$ ksi

| Design | W21x | | | | | | | | | |
|--------|------|------|------|------|------|------|------|------|------|------|
| | 57 | | 55 | | 50 | | 48 | | 44 | |
| | ASD | LRFD | ASD | LRFD | ASD | LRFD | ASD | LRFD | ASD | LRFD |
| 6 | | | | | 316 | 374 | | | 290 | 335 |
| 7 | 342 | 513 | | | 314 | 411 | 288 | 433 | 272 | 400 |
| 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 | 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 | 196 | 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 | 192 | 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 | 136 | 88.2 | 133 | 79.3 | 119 |
| 25 | 103 | 155 | 101 | 151 | 87.8 | 130 | 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 | 79.6 | 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 | 58.6 | 89.4 | 52.9 | 79.5 |
| 38 | 67.9 | 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 | 68.6 | 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

| M_x/I_x | $\phi_b M_n$ | $\phi_b M_p$ | M_x/I_x | $\phi_b M_n$ | $\phi_b M_p$ | M_x/I_x | $\phi_b M_n$ | $\phi_b M_p$ | M_x/I_x | $\phi_b M_n$ | $\phi_b M_p$ |
|-----------|--------------|--------------|-----------|--------------|--------------|-----------|--------------|--------------|-----------|--------------|--------------|
| 2570 | 3870 | 2510 | 3780 | 2200 | 3300 | 2120 | 3180 | 1900 | 2860 | | |
| 322 | 484 | 314 | 473 | 274 | 413 | 265 | 398 | 238 | 358 | | |
| 194 | 291 | 192 | 289 | 165 | 248 | 162 | 244 | 143 | 214 | | |
| 13.4 | 20.3 | 10.8 | 10.3 | 12.1 | 18.3 | 9.89 | 14.8 | 11.1 | 16.8 | | |
| 171 | 256 | 158 | 234 | 158 | 237 | 144 | 216 | 145 | 217 | | |

L_x, ft 129 126 110 107 94.4

L_r, ft 4.77 6.11 4.59 6.09 4.45

L_x, ft 14.3 17.4 13.6 16.5 13.0

ASD LRFD

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.

L3.33

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

W21

$F_y = 50$ ksi

| Design | W21x | | | | | | | | | |
|--------|------|------|------|------|------|------|------|------|------|------|
| | 57 | | 55 | | 50 | | 48 | | 44 | |
| | ASD | LRFD | ASD | LRFD | ASD | LRFD | ASD | LRFD | ASD | LRFD |
| 6 | | | | | 316 | 374 | | | 290 | 335 |
| 7 | 342 | 513 | | | 314 | 411 | 288 | 433 | 272 | 400 |
| 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 | 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 | 196 | 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 | 192 | 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 | 136 | 88.2 | 133 | 79.3 | 119 |
| 25 | 103 | 155 | 101 | 151 | 87.8 | 130 | 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 | 79.6 | 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 | 58.6 | 89.4 | 52.9 | 79.5 |
| 38 | 67.9 | 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 | 68.6 | 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

| M_x/I_x | $\phi_b M_n$ | $\phi_b M_p$ | M_x/I_x | $\phi_b M_n$ | $\phi_b M_p$ | M_x/I_x | $\phi_b M_n$ | $\phi_b M_p$ | M_x/I_x | $\phi_b M_n$ | $\phi_b M_p$ |
|-----------|--------------|--------------|-----------|--------------|--------------|-----------|--------------|--------------|-----------|--------------|--------------|
| 2570 | 3870 | 2510 | 3780 | 2200 | 3300 | 2120 | 3180 | 1900 | 2860 | | |
| 322 | 484 | 314 | 473 | 274 | 413 | 265 | 398 | 238 | 358 | | |
| 194 | 291 | 192 | 289 | 165 | 248 | 162 | 244 | 143 | 214 | | |
| 13.4 | 20.3 | 10.8 | 10.3 | 12.1 | 18.3 | 9.89 | 14.8 | 11.1 | 16.8 | | |
| 171 | 256 | 158 | 234 | 158 | 237 | 144 | 216 | 145 | 217 | | |

L_x, ft 129 126 110 107 94.4

L_r, ft 4.77 6.11 4.59 6.09 4.45

L_x, ft 14.3 17.4 13.6 16.5 13.0

ASD LRFD

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.

L3.34

Lateral-Torsional Buckling

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

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

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

L3.35

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.

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

L3.36

Lateral-Torsional Buckling

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

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



L3.37

Lateral-Torsional Buckling

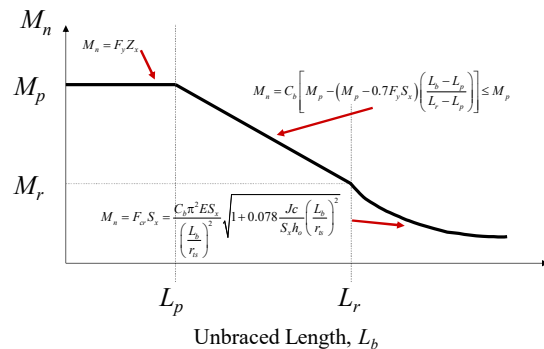
- The dividing line between inelastic and elastic lateral-torsional buckling is given by L_r .

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



L3.38

Lateral-Torsional Buckling



L3.39

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

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



L3.40

Lateral-Torsional Buckling

- Thus,

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

and the available strength is

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

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



L3.41

Design Aid

Tabulate the terms in these equations

Z_x

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

F_y = 50 ksi

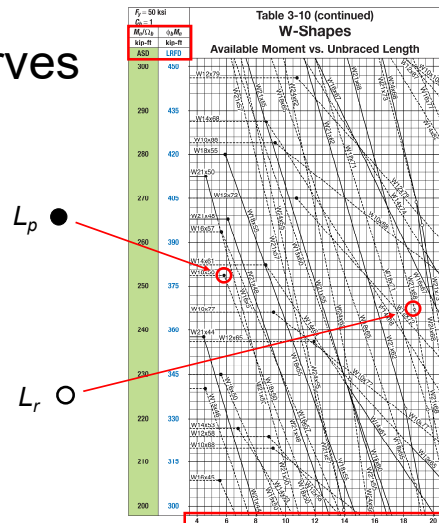
| Shape | Z _x in. ³ | M _p /L _b | | M _n /L _b | | BF/L _b | | L _p ft | L _r ft | I _x in. ⁴ | M _n /L _b | |
|--------|------------------------------------|--------------------------------|------|--------------------------------|------|-------------------|------|----------------------|----------------------|------------------------------------|--------------------------------|------|
| | | ASD | LRFD | ASD | LRFD | ASD | LRFD | | | | ASD | LRFD |
| W21-44 | 95.4 | 230 | 358 | 143 | 214 | 11.1 | 16.8 | 4.45 | 13.0 | 840 | 145 | 217 |
| W16-50 | 92.0 | 230 | 345 | 141 | 213 | 7.89 | 11.4 | 5.62 | 17.2 | 659 | 124 | 186 |
| W10-46 | 90.7 | 220 | 340 | 139 | 207 | 9.03 | 14.0 | 4.50 | 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 | 210 | 324 | 130 | 205 | 3.82 | 5.09 | 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 | 16.9 | 518 | 97.6 | 146 |
| W12-50 | 71.9 | 179 | 270 | 112 | 169 | 3.97 | 5.98 | 6.92 | 23.8 | 391 | 80.3 | 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 |



L3.42

Beam Curves

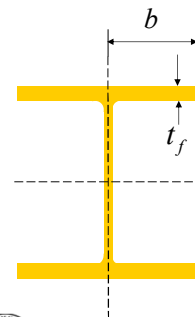
Another design aid that will assist in design considering unbraced length



L3.43

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**

| C Case | Description of Element | Width-to-Thickness Ratio | Limiting Width-to-Thickness Ratio | | Examples |
|-----------|---|--------------------------|-----------------------------------|---|----------|
| | | | λ_p (compact) | λ_r (noncompact/slender) | |
| 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 | $0.38 \sqrt{\frac{E}{F_y}}$ | $0.85 \sqrt{\frac{K_c E}{F_y}}$ (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 | $0.84 \sqrt{\frac{E}{F_y}}$ | $1.52 \sqrt{\frac{E}{F_y}}$ | |



L3.44



Slender Elements

| Slender Element | Symbol | Limit | Limit | Diagram | |
|-----------------|--|-----------|---|---------------------|--|
| 15 | Webs of doubly symmetric I-shaped sections and channels | h/t_w | $3.76 \sqrt{E/F_y}$ | $0.76 \sqrt{E/F_y}$ | |
| 16 | Webs of singly symmetric I-shaped sections | h_w/t_w | $\frac{h_w}{t_w} \sqrt{\frac{E}{F_y}} \leq \frac{h_w}{t_w} \sqrt{\frac{E}{F_y}} \left(\frac{0.64 M_p}{M_y} + 0.00 \right) < \lambda_c$ | $5.70 \sqrt{E/F_y}$ | |
| 17 | Flanges of rectangular HSS | b/t_f | $1.12 \sqrt{E/F_y}$ | $1.40 \sqrt{E/F_y}$ | |
| 18 | Flange cover plates and reinforcement plates between lines of fasteners or welds | b/t | $1.12 \sqrt{E/F_y}$ | $1.40 \sqrt{E/F_y}$ | |
| 19 | Webs of rectangular HSS and box sections | h/t | $2.42 \sqrt{E/F_y}$ | $0.76 \sqrt{E/F_y}$ | |
| 20 | Round HSS | D/t | $0.07 \frac{E}{F_y}$ | $0.3 \frac{E}{F_y}$ | |
| 21 | Flanges of box sections | b/t | $1.12 \sqrt{E/F_y}$ | $1.48 \sqrt{E/F_y}$ | |

$\lambda_c = 4.0/h/t_w$, shall not be taken less than 0.35 nor greater than 0.76 for calculation purposes.
 $F_y = 0.7 F_u$ for slender web I-shaped members and major-axis bending of compact and noncompact web built-up I-shaped members with $d_w/S_w \geq 0.7$, $F_u = F_y d_w/S_w \geq 0.6 F_y$ for major-axis bending of compact and noncompact web built-up I-shaped members with $d_w/S_w < 0.7$, where S_w , S_u = elastic section modulus referred to compression and tension flanges, respectively, in³ (mm³).
 M_p is the moment at yielding of the extreme fiber, $M_y = F_y Z_x$, plastic bending moment, kip-in. (N-mm), where Z_x = plastic section modulus taken about x-axis, in³ (mm³).
 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)

L3.45

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

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

L3.46

Flange Local Buckling

- Slender W-shape

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

L3.47

Flange Local Buckling

Nominal Strength

- Compact $M_n = M_p = F_y Z_x$ (F2-1)
- Slender $M_n = \frac{0.9 E k_c S_x}{\lambda^2}$ (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

L3.48

Flange Local Buckling

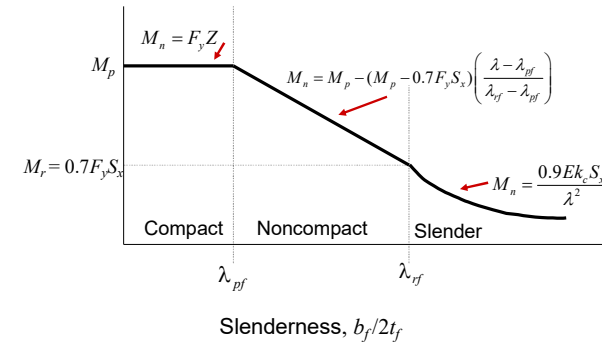
- Noncompact

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



L3.49

Flange Local Buckling



L3.50

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).



L3.51

Flange Local Buckling

Table 1-1 (continued)
W-Shapes
Dimensions

| Shape | Area, A | Depth, d | Web | | Flange | | Distance | | | | | |
|---------|------------------|----------|---------------|------|-----------|---------------|----------|-----|------|----------------|--------|---|
| | | | Thickness, tw | tw/2 | Width, bf | Thickness, tf | k1 | k2 | T | Workable Gauge | | |
| | in. ² | in. | in. | in. | in. | in. | in. | in. | in. | in. | in. | |
| W14x132 | 38.8 | 14.0 | 14.3 | 7.15 | 14.6 | 14.6 | 0.860 | 7/8 | 1.46 | 2 3/16 | 1 1/2 | 0 |
| W14x120 | 35.3 | 14.0 | 14.3 | 7.15 | 14.6 | 14.6 | 0.860 | 7/8 | 1.38 | 2 1/16 | 1 7/16 | 0 |
| W14x109 | 32.0 | 14.0 | 14.3 | 7.15 | 14.6 | 14.6 | 0.860 | 7/8 | 1.31 | 2 | 1 7/16 | 0 |
| W14x99 | 29.1 | 14.0 | 14.3 | 7.15 | 14.6 | 14.6 | 0.860 | 7/8 | 1.25 | 1 15/16 | 1 7/16 | 0 |
| W14x90 | 26.5 | 14.0 | 14.3 | 7.15 | 14.6 | 14.6 | 0.860 | 7/8 | 1.18 | 1 11/16 | 1 7/16 | 0 |
| W14x82 | 24.0 | 14.3 | 14.3 | 7.15 | 14.6 | 14.6 | 0.860 | 7/8 | 1.11 | 1 7/16 | 1 7/16 | 0 |
| W14x74 | 21.8 | 14.3 | 14.3 | 7.15 | 14.6 | 14.6 | 0.860 | 7/8 | 1.04 | 1 7/16 | 1 7/16 | 0 |
| W14x65 | 19.0 | 14.3 | 14.3 | 7.15 | 14.6 | 14.6 | 0.860 | 7/8 | 0.97 | 1 7/16 | 1 7/16 | 0 |

Note the footnote on the weight, f



L3.52



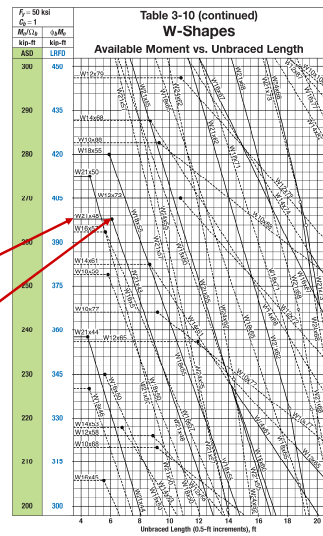
Beam Curves

Unbraced length charts for beams

W21x48 $\phi M_n = 398$ ft-kips

$L_p = 6.09$ ft

This is a noncompact flange shape



L3.57



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.



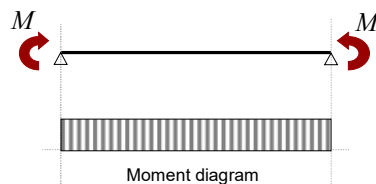
L3.58

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,

$C_b = 1.0$



L3.59



Lateral-Torsional Buckling

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

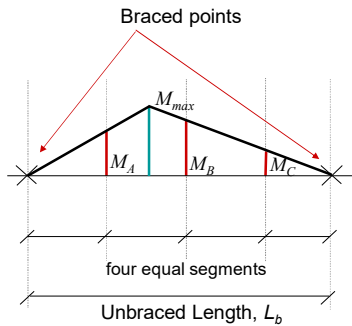
$$C_b = \frac{12.5M_{\max}}{2.5M_{\max} + 3M_A + 4M_B + 3M_C} \quad (F1-1)$$



L3.60

Lateral-Torsional Buckling

- Moment diagram over unbraced length



L3.61

Table 3-1
 Values of C_b for Simply Supported Beams

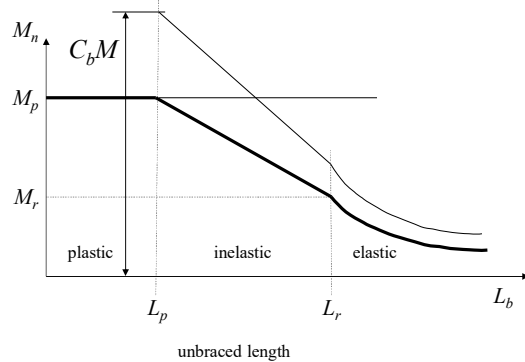
| Load | Lateral Bracing Along Span | C_b |
|------|----------------------------|--------------------------|
| | None | 1.33 |
| | At load point | 1.67 |
| | None | 1.14 |
| | At load points | 1.67 1.00 1.67 |
| | None | 1.14 |
| | At load points | 1.67 1.11 1.11 1.67 |
| | None | 1.14 |
| | At midpoint | 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 | 1.56 1.12 1.00 1.12 1.56 |

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



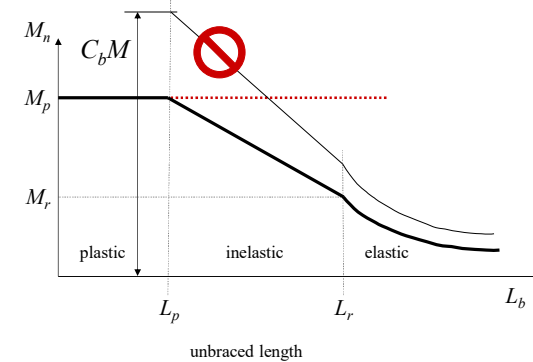
L3.62

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

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

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



L3.65

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

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



L3.66

Example 1(ASD)

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

| Shape | Z_x in. ³ | M_{px}/Ω_b | | $\phi_p M_{px}$ | | M_{rx}/Ω_b | | $\phi_p M_{rx}$ | | BF/Ω_b | $\phi_p BF$ | L_p | L_r | I_x | V_{nx}/Ω_v | | $\phi_p V_{nx}$ |
|---------------|---------------------------|-------------------|------------|-----------------|------------|-------------------|-------------|-----------------|-------------|---------------|-------------|------------|-------|-------|-------------------|------|-----------------|
| | | kip-ft | kip-ft | kip-ft | kip-ft | kip-ft | kip-ft | kip-ft | kip-ft | | | | | | 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 | | | | | |
| W10x24 | 66.0 | 160 | 240 | 103 | 151 | 6.48 | 9.73 | 3.04 | 33.0 | 303 | 74.7 | 112 | | | | | |
| W18x35 | 66.5 | 166 | 249 | 101 | 151 | 8.14 | 12.3 | 4.31 | 12.3 | 510 | 106 | 169 | | | | | |
| 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 | 64.0 | 160 | 240 | 95.4 | 143 | 5.37 | 8.20 | 5.47 | 16.2 | 385 | 87.4 | 131 | | | | | |
| W10x49 | 64.0 | 160 | 240 | 95.4 | 143 | 2.46 | 3.71 | 8.97 | 31.6 | 272 | 68.0 | 102 | | | | | |
| W8x58 | 64.0 | 160 | 240 | 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 | 84.0 | 128 | 5.01 | 7.55 | 5.40 | 15.6 | 340 | 78.8 | 120 | | | | | |



L3.67

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



L3.68

Example 1(LRFD)

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

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

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



L3.69

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

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



L3.70

Example 1(LRFD)

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

| Shape | Z_x in. ³ | M_{px}/Ω_b | | $\phi_p M_{px}$ | | M_{rx}/Ω_b | | $\phi_p M_{rx}$ | | BF/Ω_b | | $\phi_p BF$ | | L_p ft | L_r ft | I_x in. ⁴ | V_{ux}/Ω_v | | $\phi_p V_{ux}$ | |
|--------|---------------------------|-------------------|--------|-----------------|--------|-------------------|--------|-----------------|--------|---------------|--------|-------------|--------|-------------|-------------|---------------------------|-------------------|--------|-----------------|--------|
| | | kip-ft | kip-ft | kip-ft | kip-ft | kip-ft | kip-ft | kip-ft | kip-ft | kip-ft | kip-ft | kip-ft | kip-ft | | | | kip-ft | kip-ft | kip-ft | kip-ft |
| 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 | | | | | | | | |
| W10x24 | 66.5 | 166 | 249 | 101 | 151 | 8.14 | 12.3 | 4.31 | 12.3 | 510 | 106 | 169 | | | | | | | | |
| 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 | 61.4 | 153 | 228 | 95.4 | 143 | 5.37 | 8.20 | 5.47 | 16.2 | 385 | 87.4 | 131 | | | | | | | | |
| W10x49 | 61.4 | 151 | 227 | 95.4 | 143 | 2.46 | 3.71 | 8.97 | 31.6 | 272 | 68.0 | 102 | | | | | | | | |
| W8x58 | 61.4 | 149 | 224 | 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 | 78.8 | 120 | | | | | | | | |



L3.71

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

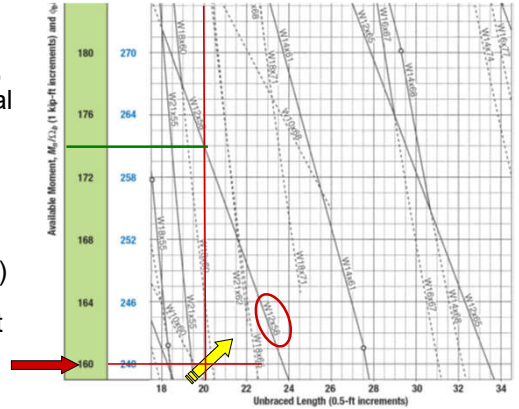


L3.72



Example 2 (ASD)

- For the beam of Example 1, assume lateral supports at ends only.
- $L_b = 20$ ft
- $C_b = 1.0$ (conservative)
- $M_a = 160$ kip-ft



L3.73

Example 2 (ASD)

Select W12x58.
 Check using Table 3-2

| Shape | Z_x in. ³ | M_p/Ω | | M_n/Ω | | $\phi_b M_n$ | | $\phi_b BF$ | L_p | L_r | I_x | S_x | |
|--------|---------------------------|--------------|------|--------------|------|--------------|------|-------------|-------|-------|-------|-------|------|
| | | ASD | LRFD | ASD | LRFD | ASD | LRFD | | | | | ASD | LRFD |
| W21x44 | 85.4 | 238 | 358 | 143 | 214 | 11.3 | 16.8 | 4.45 | 13.0 | 843 | 145 | 217 | |
| W16x60 | 220 | 330 | 500 | 141 | 213 | 7.60 | 11.4 | 5.62 | 17.2 | 650 | 124 | 186 | |
| W18x46 | 90.7 | 226 | 340 | 138 | 207 | 9.63 | 14.6 | 4.58 | 13.7 | 712 | 130 | 195 | |
| W14x69 | 171 | 317 | 470 | 139 | 204 | 5.92 | 8.85 | 6.78 | 22.3 | 541 | 153 | 224 | |
| W12x58 | 86.4 | 216 | 324 | 138 | 205 | 3.82 | 5.69 | 4.87 | 8.87 | 475 | 92.8 | 135 | |
| W10x88 | 85.3 | 213 | 320 | 132 | 199 | 2.58 | 3.85 | 3.15 | 40.6 | 394 | 97.8 | 147 | |
| W16x45 | 82.3 | 205 | 309 | 127 | 191 | 7.12 | 10.8 | 5.55 | 16.5 | 586 | 111 | 167 | |

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

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

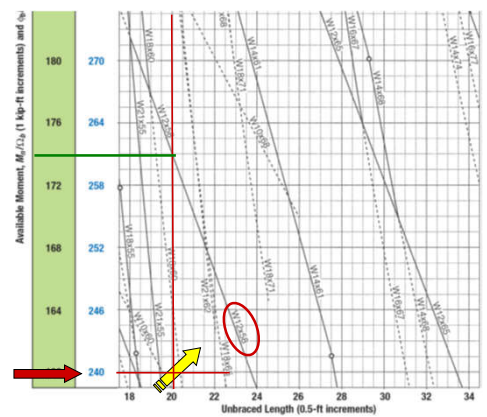
$$> M_a = 160 \text{ ft-kips}$$



L3.74

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



L3.75

Example 2 (LRFD)

Select W12x58.
 Check using Table 3-2

| Shape | Z_x in. ³ | M_p/ϕ | | M_n/ϕ | | $\phi_b M_n$ | | $\phi_b BF$ | L_p | L_r | I_x | S_x | |
|--------|---------------------------|------------|------|------------|------|--------------|------|-------------|-------|-------|-------|-------|------|
| | | ASD | LRFD | ASD | LRFD | ASD | LRFD | | | | | ASD | LRFD |
| W21x44 | 85.4 | 238 | 358 | 143 | 214 | 11.3 | 16.8 | 4.45 | 13.0 | 843 | 145 | 217 | |
| W16x60 | 220 | 330 | 500 | 141 | 213 | 7.60 | 11.4 | 5.62 | 17.2 | 650 | 124 | 186 | |
| W18x46 | 90.7 | 226 | 340 | 138 | 207 | 9.63 | 14.6 | 4.58 | 13.7 | 712 | 130 | 195 | |
| W14x69 | 171 | 317 | 470 | 139 | 204 | 5.92 | 8.85 | 6.78 | 22.3 | 541 | 153 | 224 | |
| W12x58 | 86.4 | 216 | 324 | 138 | 205 | 3.82 | 5.69 | 4.87 | 8.87 | 475 | 92.8 | 135 | |
| W10x88 | 85.3 | 213 | 320 | 132 | 199 | 2.58 | 3.85 | 3.15 | 40.6 | 394 | 97.8 | 147 | |
| W16x45 | 82.3 | 205 | 309 | 127 | 191 | 7.12 | 10.8 | 5.55 | 16.5 | 586 | 111 | 167 | |

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

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

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



L3.76



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 | 1.50 1.07 |
| | None | 1.10 1.07 |

$C_b = 1.32$



L3.77

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



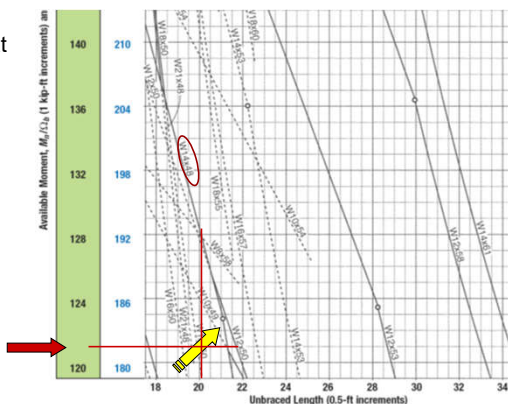
L3.78

Example 3 (ASD)

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

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

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



L3.79

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 | 1.50 1.07 |
| | None | 1.10 1.07 |

$C_b = 1.32$



L3.80

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



L3.81

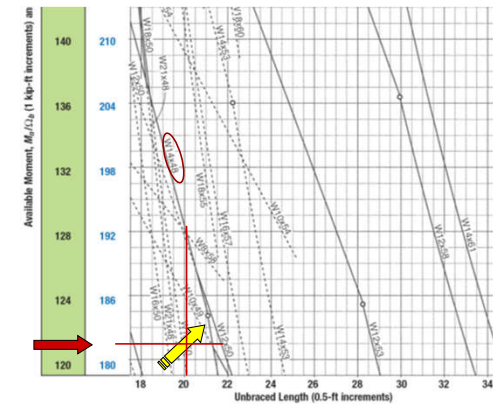
Example 3 (LRFD)

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

$$\phi M_p = 294$$

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

$$\therefore \text{ok}$$



L3.82

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



L3.83

Tees and Double Angles

- Limit States
 - Yielding **L T**
 - Lateral-Torsional Buckling **L T**
 - Flange Local Buckling **T**
 - Stem Local Buckling **L**
- For this presentation assume a simple beam, thus compression is on top



L3.84

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



L3.85

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



L3.86

Tees

- F9.1. Yielding

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

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

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

- (b) For tee stems in compression **L**

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



L3.87

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 $L_p < L_b \leq L_r$

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



L3.88

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



L3.89

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



L3.90

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

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



L3.91

Tees

- F9.4. Stem Local Buckling **L**

(a) For tee stems

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

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

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

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

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



L3.92

Tees

- F9.4. Stem Local Buckling \perp

(a) For tee stems

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

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



L3.93

Example 4

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

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

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

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

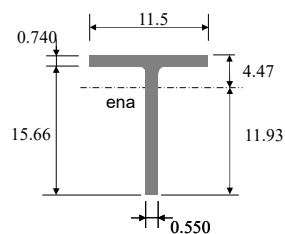
$$I_y = 93.5 \text{ in.}^4 \quad r_y = 2.32 \text{ in.}$$



L3.94

Example 4

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



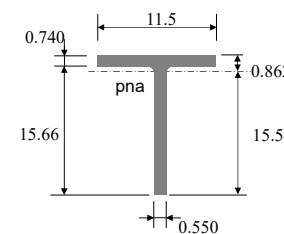
$$S_{bottom} = 39.2 \text{ in.}^3$$



L3.95

Example 4

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



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

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



L3.96

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,

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



L3.97

Example 4

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

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

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

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

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



L3.98

Example 4

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

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

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

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



L3.99

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



L3.100

Example 4

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



L3.101

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.



L3.102

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



L3.103

Shear

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

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



L3.104

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.



L3.105

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,

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



L3.106

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.



L3.107

Shear

- G2.1.(b)(1)
 (ii) when

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

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

This means the web will buckle inelastically.



L3.108

Shear

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

$$k_v = 5.34$$

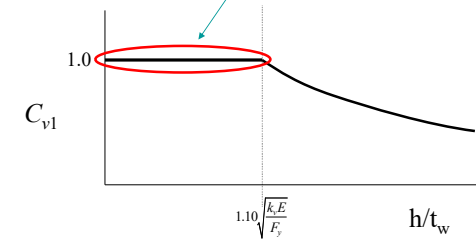
- **User Note:** For all ASTM A6 W, S, M and HP shapes except M12.5x12.4, M12.5x11.6, M12x11.8, M12x10.8, M12x10, M10x8 and M10x7.5, when $F_y = 50$ ksi (345 MPa), $C_{v1} = 1.0$.



L3.109

Shear

All W-shapes will fall in this range



L3.110

Shear

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

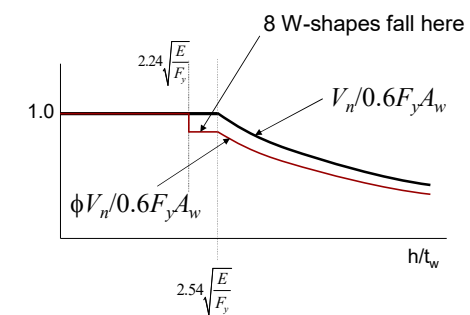
$$\frac{h}{t_w} \leq 2.24 \sqrt{\frac{E}{F_y}}$$

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



L3.111

Shear



For W-shapes, $k_v = 5.34$



L3.112

Manual

Z_x tables and uniform load tables show shear strength

| Shape | Z _x in. ² | M _p /Ω _b | | M _r /Ω _b | | M _p /Ω _b | | B _f /Ω _b | | L _p | L _r | I _x | S _x | |
|--------|------------------------------------|--------------------------------|------|--------------------------------|------|--------------------------------|------|--------------------------------|------|----------------|----------------|----------------|----------------|------|
| | | 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 | 12.2 | 859 | 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 | 227 | 327 | 136 | 204 | 5.22 | 7.93 | 6.78 | 22.3 | 541 | 103 | 154 | | |
| W10×24 | 66.5 | 166 | 249 | 101 | 151 | 8.14 | 12.3 | 4.31 | 12.3 | 610 | 106 | 169 | | |
| W18×26 | 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 | 26.9 | 248 | 70.7 | 106 | | |
| W14×24 | 54.6 | 136 | 205 | 84.9 | 128 | 6.01 | 7.55 | 5.40 | 15.6 | 240 | 70.8 | 106 | | |

| Shape | W18 | | | | W16 | | | |
|--------|-----|------|-----|------|-----|------|-----|------|
| | ASD | LRFD | ASD | LRFD | ASD | LRFD | ASD | LRFD |
| Design | 6 | 7 | 6 | 7 | 6 | 7 | 6 | 7 |
| W18×46 | 282 | 424 | 226 | 361 | 281 | 421 | 225 | 358 |
| W18×50 | 280 | 420 | 224 | 356 | 279 | 419 | 223 | 354 |
| W18×44 | 278 | 418 | 222 | 352 | 277 | 417 | 221 | 350 |
| W18×42 | 276 | 416 | 220 | 348 | 275 | 415 | 219 | 346 |
| W18×40 | 274 | 414 | 218 | 344 | 273 | 413 | 217 | 342 |
| W18×38 | 272 | 412 | 216 | 340 | 271 | 411 | 215 | 338 |
| W18×36 | 270 | 410 | 214 | 336 | 269 | 409 | 213 | 334 |
| W18×34 | 268 | 408 | 212 | 332 | 267 | 407 | 211 | 330 |
| W18×32 | 266 | 406 | 210 | 328 | 265 | 405 | 209 | 326 |
| W18×30 | 264 | 404 | 208 | 324 | 263 | 403 | 207 | 322 |
| W18×28 | 262 | 402 | 206 | 320 | 261 | 401 | 205 | 318 |
| W18×26 | 260 | 400 | 204 | 316 | 259 | 399 | 203 | 314 |
| W18×24 | 258 | 398 | 202 | 312 | 257 | 397 | 201 | 310 |
| W18×22 | 256 | 396 | 200 | 308 | 255 | 395 | 199 | 306 |
| W18×20 | 254 | 394 | 198 | 304 | 253 | 393 | 197 | 302 |
| W18×18 | 252 | 392 | 196 | 300 | 251 | 391 | 195 | 298 |
| W18×16 | 250 | 390 | 194 | 296 | 249 | 389 | 193 | 294 |
| W18×14 | 248 | 388 | 192 | 292 | 247 | 387 | 191 | 290 |
| W18×12 | 246 | 386 | 190 | 288 | 245 | 385 | 189 | 286 |
| W18×10 | 244 | 384 | 188 | 284 | 243 | 383 | 187 | 282 |
| W18×8 | 242 | 382 | 186 | 280 | 241 | 381 | 185 | 278 |
| W18×6 | 240 | 380 | 184 | 276 | 239 | 379 | 183 | 274 |
| W18×4 | 238 | 378 | 182 | 272 | 237 | 377 | 181 | 270 |
| W18×2 | 236 | 376 | 180 | 268 | 235 | 375 | 179 | 266 |



L3.113

Example 4(ASD)

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

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

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



L3.114

Example 4(ASD)

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



| Shape | Z _x in. ² | M _p /Ω _b | | M _r /Ω _b | | M _p /Ω _b | | B _f /Ω _b | | L _p | L _r | I _x | S _x | |
|--------|------------------------------------|--------------------------------|------|--------------------------------|------|--------------------------------|------|--------------------------------|------|----------------|----------------|----------------|----------------|------|
| | | 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 | 12.2 | 859 | 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 | | |
| W10×24 | 66.5 | 166 | 249 | 101 | 151 | 8.14 | 12.3 | 4.31 | 12.3 | 610 | 106 | 169 | | |
| W18×26 | 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 | 26.9 | 248 | 70.7 | 106 | | |
| W14×24 | 54.6 | 136 | 205 | 84.9 | 128 | 6.01 | 7.55 | 5.40 | 15.6 | 240 | 70.8 | 106 | | |



L3.115

Example 4(LRFD)

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

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

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



L3.116



Example 4(LRFD)

$$V_u = 24.0 <$$

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

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

| Shape | Z_x in. ³ | M_{px}/Ω_b | | $\phi_b M_{px}$ | | M_{rx}/Ω_b | | $\phi_b M_{rx}$ | | BF/Ω_b | | $\phi_b BF$ | | L_p ft | L_r ft | I_x in. ⁴ | V_n/Ω_v | | $\phi_v V_n$ | |
|--------|---------------------------|-------------------|------|-----------------|------|-------------------|------|-----------------|------|---------------|------|-------------|------|-------------|-------------|---------------------------|----------------|--|--------------|--|
| | | ASD | LRFD | ASD | LRFD | 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 | | | | | | | | |
| W10x29 | 66.0 | 160 | 230 | 105 | 136 | 2.46 | 3.73 | 9.04 | 33.6 | 303 | 74.7 | 112 | | | | | | | | |
| W18x36 | 66.5 | 166 | 249 | 101 | 151 | 8.14 | 12.3 | 4.31 | 12.3 | 610 | 106 | 160 | | | | | | | | |
| 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 | 61.5 | 153 | 231 | 95.4 | 143 | 5.37 | 8.20 | 5.47 | 16.2 | 385 | 87.4 | 131 | | | | | | | | |
| W10x49 | 60.4 | 151 | 227 | 95.4 | 143 | 2.46 | 3.71 | 8.97 | 31.6 | 272 | 68.0 | 102 | | | | | | | | |
| W8x58 | 59.8 | 149 | 224 | 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 | 28.9 | 248 | 70.7 | 106 | | | | | | | | |
| W14x34 | 54.6 | 136 | 205 | 84.9 | 128 | 5.01 | 7.55 | 5.40 | 15.6 | 340 | 79.8 | 120 | | | | | | | | |



L3.117

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



L3.118

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



L3.119



Thank You

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



L3.120

Single-Session Registrants

CEU / PDH Certificates

- You will receive an email on how to report attendance from: registration@aisc.org.
- Be on the lookout: Check your spam filter! Check your junk folder!
- Completely fill out online form. Don't forget to check the boxes next to each attendee's name!



Single-Session Registrants

CEU / PDH Certificates

- Reporting site (URL will be provided in the forthcoming email).
- Username: Same as AISC website username.
- Password: Same as AISC website password.



Course Package Registrants

CEU / PDH Certificates

One certificate will be issued at the conclusion of the course.



Course Package Registrants

Attendance and PDH Certificates

- You have two options to receive credit for a given session.
 - Option 1: Watch the live session. Credit for live attendance will be displayed on the Course Resources table within two days of the session.
 - Option 2: Watch the recording and pass the associated quiz.

Videos and Quizzes

- For each session, find access within two business days after the live air date. (An email will be sent from webinars@aisc.org.)
- Quiz scores are displayed in the Course Resources table.

Distribution of Certificates

All certificates will be issued after the course is completed. Only the registrant will receive a certificate for the course.



Course Package Registrants

Course Resources

Find all your handouts, quizzes and quiz scores, recording access, and attendance information in one place!

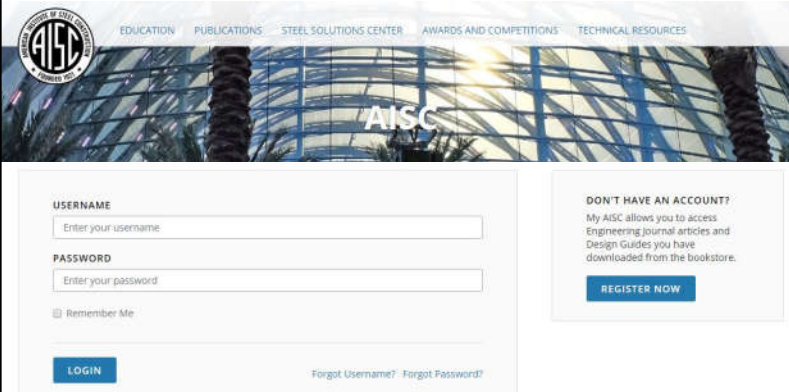


**Smarter.
Stronger.
Steel.**

Course Package Registrants

Course Resources

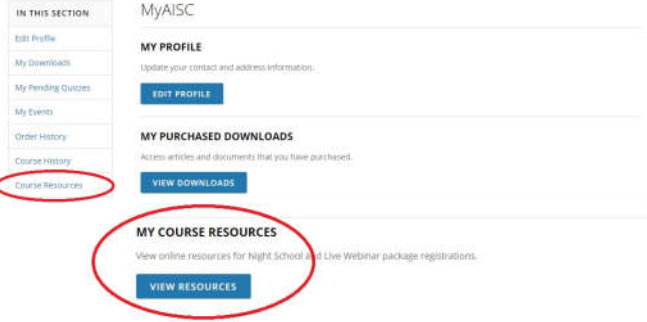
Go to www.aisc.org and sign in.



Course Package Registrants

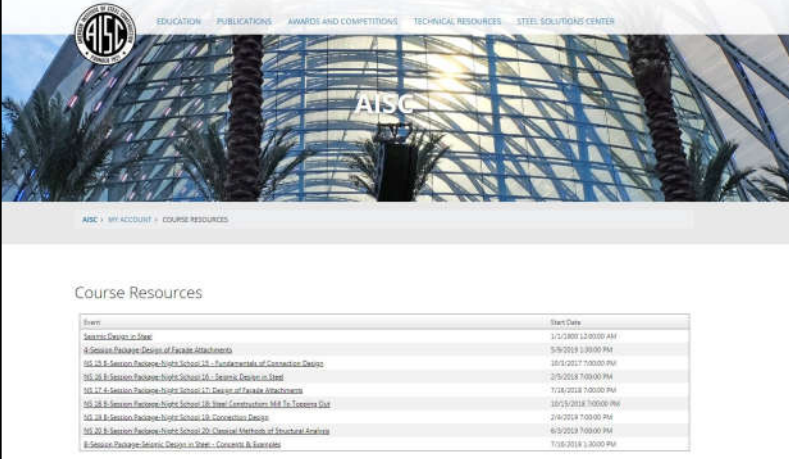
Course Resources

Go to www.aisc.org and sign in.



Course Package Registrants

Course Resources




| Event | Start Date |
|---|-----------------------|
| Session Design in Steel | 3/11/2021 12:00:00 AM |
| A Session Package Design of Fabric Attachments | 5/9/2018 1:00:00 PM |
| MS 13.8 Session Package Steel School 13... Fundamentals of Connection Design | 10/11/2017 7:00:00 PM |
| MS 16.8 Session Package Steel School 16... Statics Review on Steel | 3/9/2018 7:00:00 PM |
| MS 17.8 Session Package Steel School 17... Design of Fabric Attachments | 7/23/2018 7:00:00 PM |
| MS 18.8 Session Package Steel School 18... Steel Construction: MS & Trainers Day | 10/11/2018 7:00:00 PM |
| MS 19.8 Session Package Steel School 19... Connections Design | 2/4/2019 7:00:00 PM |
| MS 20.8 Session Package Steel School 20... Classical Methods of Structural Analysis | 6/29/2019 7:00:00 PM |
| 8 Session Package Statics Design in Steel... Concepts & Examples | 7/30/2018 1:00:00 PM |




Course Package Registrants

Course Resources



EDUCATION | PUBLICATIONS | AWARDS AND COMPETITIONS | TECHNICAL RESOURCES | STEEL SOLUTIONS CENTER



HOME > MY ACCOUNT > COURSE RESOURCES > DESIGN OF FACADE ATTACHMENTS PACKAGE RESOURCES

Design of Facade Attachments

4-SESSION PACKAGE RESOURCES

| Event | Date | Handouts | Video | Quiz | Attendance |
|---|------------------------|--------------------------|---|--|------------|
| K2 Facade Fundamentals | N/A | Download | View Passcode: A276175 | Pass Score: 100 | None |
| L1 Facade Attachments Part 1 | May 9 2019 1:00PM EDT | Download | Available 05/11/2019 5:00PM EDT | Available 05/11/2019 5:00 PM EDT | Pending |
| L2 Facade Attachments Part 2 | May 14 2019 1:00PM EDT | Download | Available 05/16/2019 5:00PM EDT | Available 05/16/2019 5:00 PM EDT | Pending |
| L3 Facade Attachments - Building Lateral Drifts | May 23 2019 1:00PM EDT | Download | 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 | |



Smarter.
Stronger.
Steel.

AISC | Thank you.

