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Design of Built-up Plate Girder based on AISC 360-16

Lesson 1: Bending

July 22, 2021

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Course Description

Design of Built-up Plate Girders based on AISC 360-16 *Specification for Structural Steel Buildings*, Part 1 – Bending

There are many situations in flexural member design where the geometrical or loading conditions require the engineer to look beyond standard rolled wide-flange shapes. One solution for such cases is built-up plate girders, which introduce their own design challenges related to section slenderness that engineers rarely encounter when working with rolled shapes. The first session in this two-part webinar will focus on designing built-up plate girders for bending moments. We'll discuss plate girders with noncompact or slender webs and both doubly and singly symmetric cross-sections. The lesson will explore how the AISC *Specification* accounts for local buckling by using reduction factors to modify the strengths determined from commonly considered limit states of yielding and lateral-torsional buckling.

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Learning Objectives

- Demonstrate how web slenderness is assessed for singly symmetric I-shaped flexural members and compare this to those that are doubly symmetric.
- Explain how web local buckling affects the flexural limit states for built-up plate girders.
- Describe how the web plastification factor varies as a function of web slenderness.
- Compare the design procedures for plate girders with noncompact webs to those with slender webs.

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Design of Built-up Plate Girders based on AISC 360-16 *Specification for Structural Steel Buildings*

Summer Webinar 2021

Lesson 1

Bending

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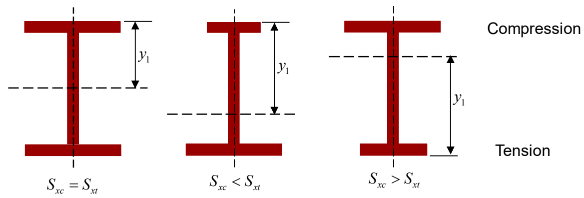
Plate Girders

- Plate girders as a term has not been used since the introduction of AISC 360-05.
- The previous ASD and LRFD Specifications each had a separate chapter, Chapter G, that dealt with them by that name.
- Currently the provisions are found in Chapter F for bending and Chapter G for shear.

8

Plate Girders

- A member made (built-up) from plates in the form of a singly or doubly symmetric I-shape is what we will be referring to as plate girders



9

Plate Girders

- For bending, plate girders are a part of
  - F2. Doubly symmetric and compact
  - F3. Doubly symmetric with compact web and noncompact or slender flanges
  - F4. Doubly symmetric or singly symmetric with compact or noncompact webs
  - F5. Doubly symmetric or singly symmetric with slender webs

10

Plate Girders

All rolled W-shapes

		Doubly Symmetric	Singly Symmetric
F2	Flange	compact	
	Web	compact	
F3	Flange	noncompact, slender	
	Web	compact	
F4	Flange	all	all
	Web	noncompact	compact, noncompact
F5	Flange	all	all
	Web	slender	slender

Plate girders could fall into any of these categories

11

Plate Girders

- For shear, plate girders are covered in
  - G2.1 Shear strength of webs without tension field action
  - G2.2 Shear strength of interior web panels with $a / h \leq 3$ considering tension field action

Rolled W-shapes do not benefit from stiffeners.

Plate girders may benefit from stiffeners and may benefit from tension field action.

12

Plate Girders

- F13. for proportioning of I-shaped members
  - Singly and doubly symmetric I-shaped members with slender webs

When $\frac{a}{h} \leq 1.5$

$$\left(\frac{h}{t_w}\right)_{max} = 12.0\sqrt{\frac{E}{F_y}} \quad \text{F13-3}$$

When $\frac{a}{h} > 1.5$

$$\left(\frac{h}{t_w}\right)_{max} = \frac{0.40E}{F_y} \quad \text{F13-4}$$

For unstiffened girders $h/t_w \leq 260$ and

ratio of web area to compression flange area ≤ 10

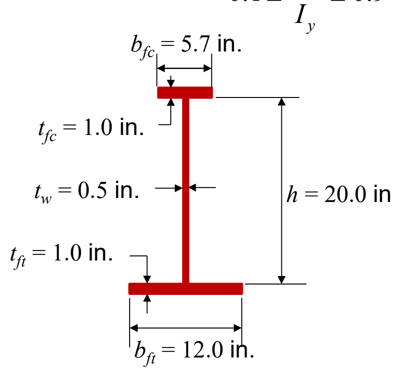
$a =$ clear distance between transverse stiffeners

13

Plate Girders

- F13. for proportioning of I-shaped members
  - Singly symmetric I-shaped members

$$0.1 \leq \frac{I_{yc}}{I_y} \leq 0.9 \quad \text{F13-2}$$



$$I_{yc} = \frac{1.0(5.7)^3}{12} = 15.4 \text{ in.}^4$$

$$I_{yw} = \frac{20(0.5)^3}{12} = 0.2 \text{ in.}^4$$

$$I_{yt} = \frac{1.0(12.0)^3}{12} = 144 \text{ in.}^4$$

$$\frac{I_{yc}}{I_y} = \frac{15.4}{15.4 + 0.2 + 144} = 0.096$$

Not acceptable

14

Plate Girders

- Bending Limit States
  - Compression Flange Yielding
    - Compact (may be influenced by web local buckling)
  - Compression Flange Local Buckling
    - Noncompact, slender (may be influenced by web local buckling)
  - Tension Flange Yielding
    - $S_{xt} < S_{xc}$ (may be influenced by web local buckling)
  - Lateral-Torsional Buckling
    - Unbraced length (may be influenced by web local buckling)
  - Web Local Buckling
    - Compact, noncompact, slender (this limit state is doing the influencing)

15

Plate Girders

- For our purposes, we will not address those plate girders that fall within the provisions that also cover W-shapes, that is F2 and F3.
- We will first look at bending of doubly symmetric plate girders.
- Then we will look at bending of singly symmetric plate girders.
- We will conclude in Lesson 2 by looking at the shear provisions.

16

Plate Girders

- F4. for doubly and singly symmetric I-shaped members with noncompact web
  - The limit state of *web local buckling* does not lead to a specific nominal strength
  - Rather, web local buckling modifies the strength determined for the other limit states; *yielding*, *flange local buckling* and *lateral-torsional buckling*, through the use of the web plastification factors, R_{pc} and R_{pt}.

17

Plate Girders

- F4.1 Compression Flange Yielding

$$M_n = R_{pc}M_{yc} = R_{pc}F_yS_{xc} \qquad \text{F4-1}$$

- F4.4 Tension Flange Yielding

$$M_n = R_{pt}M_{yt} = R_{pt}F_yS_{xt} \qquad \text{F4-15}$$

- F4.3 Compression Flange Local Buckling

Noncompact $M_n = R_{pc}M_{yc} - \left(R_{pc}M_{yc} - F_LS_{xc}\right)\left(\dfrac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right)$ F4-13

Slender $M_n = \dfrac{0.9Ek_cS_{xc}}{\lambda^2}$ F4-14 $\qquad \lambda = \dfrac{b_{fc}}{2t_{fc}}$

18

Plate Girders

- F4.1 Compression Flange Yielding

$$M_n = R_{pc}M_{yc} = R_{pc}F_yS_{xc} \quad \text{F4-1}$$

- F4.4 Tension Flange Yielding

$$M_n = R_{pt}M_{yt} = R_{pt}F_yS_{xt} \quad \text{F4-15}$$

- F4.3 Compression Flange Local Buckling

Noncompact $M_n = R_{pc}M_{yc} - \left(R_{pc}M_{yc} - F_LS_{xc}\right)\left(\frac{\lambda-\lambda_{pf}}{\lambda_{rf}-\lambda_{pf}}\right)$ F4-13

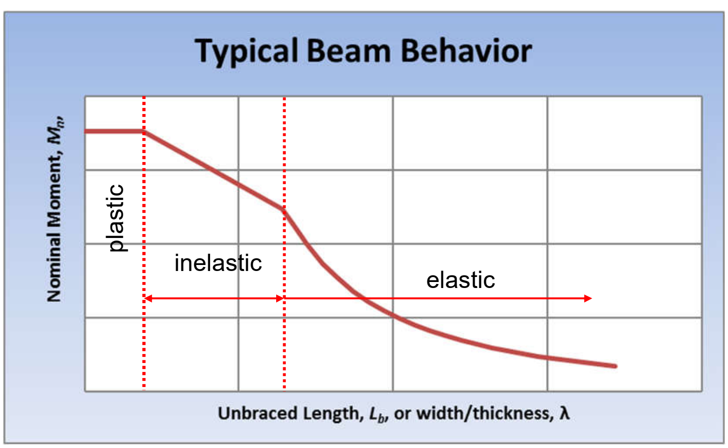
Slender $M_n = \frac{0.9Ek_cS_{xc}}{\lambda^2}$ F4-14

$$\lambda = \frac{b_{fc}}{2t_{fc}}$$

For a doubly symmetric shape, these are all equal since $S_{xc} = S_{xt}$

19

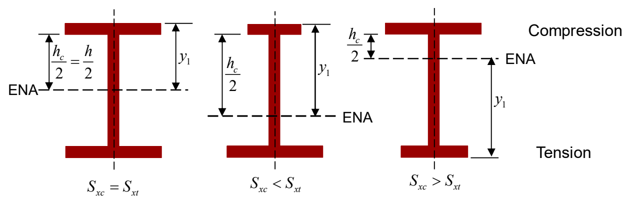
Plate Girders



20

Plate Girders

- The primary issue with plate girders is web slenderness. Since we treat both singly and doubly symmetric I-shapes, we need to have a way to look at the portion of the web that is in compression.



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

21

Plate Girders

- Look at the web plastification factor, F4.2(6)

When $I_{yc}/I_y > 0.23$

When $\frac{h_c}{t_w} \le \lambda_{pw}$

$$R_{pc} = \frac{M_p}{M_{yc}} = \frac{F_y Z}{F_y S_{xc}}$$

When $\frac{h_c}{t_w} > \lambda_{pw}$

$$R_{pc} = \left[\frac{M_p}{M_{yc}} - \left(\frac{M_p}{M_{yc}} - 1\right)\left(\frac{\lambda - \lambda_{pw}}{\lambda_{rw} - \lambda_{pw}}\right)\right] \le \frac{M_p}{M_{yc}}$$

$$M_p = F_y Z_x \le 1.6 F_y S_x$$

For doubly symmetric or singly symmetric, where S_{xc} is the smaller section modulus,

$\frac{Z}{S} =$ Shape Factor

22

Plate Girders

- Look at the web plastification factor, F4.2(6)

When $I_{yc}/I_y \leq 0.23$

$$R_{pc} = 1.0$$

$\frac{Z}{S} =$ Shape Factor

Rectangle $= 1.5$
W-shapes $= 1.1\text{-}1.3$

As you add thickness to the flanges, the shape could approach a rectangle, thus the shape factor will approach 1.5.

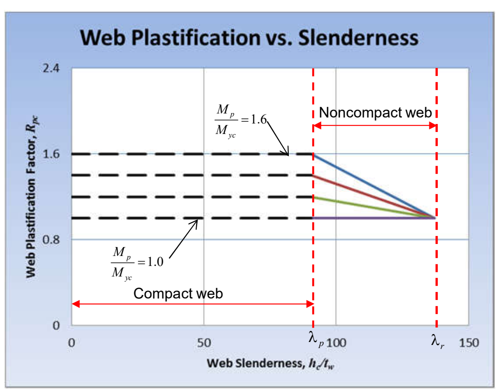
For doubly symmetric I-shapes we will not be concerned with this limit

$$M_p = F_y Z_x \leq 1.6 F_y S_x$$

23

Plate Girders

When $I_{yc}/I_y > 0.23$



I_{yc}/I_y range for W-shapes 0.49-0.50

If $I_{yc}/I_y \leq 0.23$ then $R_{pc} = 1.0$

For a compact web addressed through F4,

$$R_{pc} M_{yc} = \frac{M_p}{M_{yc}} M_{yc} = M_p \leq 1.6 F_y S$$

24

Plate Girders

- F4.2 Lateral-torsional buckling

When $L_p < L_b \le L_r$

$$M_n = C_b\left[R_{pc}M_{yc} - \left(R_{pc}M_{yc} - F_L S_{xc}\right)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \le R_{pc}M_{yc} \qquad \text{F4-2}$$

For rolled W-shapes, $F_L = 0.7F_y$

When $L_b > L_r$

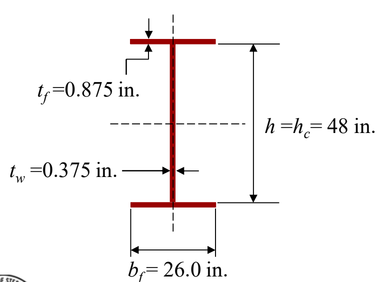
$$M_n = \frac{C_b \pi^2 E S_{xc}}{\left(\frac{L_b}{r_t}\right)^2}\sqrt{1 + 0.078\frac{J}{S_{xc}h_o}\left(\frac{L_b}{r_t}\right)^2} \le R_{pc}M_{yc} \qquad \text{F4-3, F4-5}$$

For rolled W-shapes, $r_t = r_{ts}$

25

Example 1

- Determine the nominal moment strength of the given doubly symmetric plate girder, A36 steel.



Section Properties

$I_x = 30,600$ in.^{4}

$I_y = 2560$ in.^{4}

$S_x = S_{xc} = S_{xt} = 1230$ in.^{3}

$Z_x = 1330$ in.^{3}

$d = 49.75$ in.

$b_f = 26.0$ in.

$t_w = 0.375$ in.

$t_f = 0.875$ in.

Without transverse stiffeners

26

Example 1

- Check flange slenderness, Table B4.1b Case 11

$$\frac{b_{fc}}{2t_{fc}} = \frac{26}{2(0.875)} = 14.9$$

$$\lambda_p = 0.38\sqrt{\frac{E}{F_y}} = 0.38\sqrt{\frac{29,000}{36}} = 10.8 \quad \lambda_r = 0.95\sqrt{\frac{k_c E}{F_L}} = 0.95\sqrt{\frac{k_c(29,000)}{F_L}} = ?$$

$$k_c = \frac{4}{\sqrt{h/t_w}} = \frac{4}{\sqrt{48/0.375}} = 0.354$$ (but no less than 0.35 nor more than 0.76)

$$\frac{S_{xt}}{S_{xc}} = 1.0 \ge 0.7 \text{ therefore } F_L = 0.7F_y$$ F4-6a

27

Example 1

- Check flange slenderness, Table B4.1b Case 11

$$\lambda_r = 0.95\sqrt{\frac{k_c E}{F_L}} = 0.95\sqrt{\frac{0.354(29,000)}{0.7(36)}} = 19.2$$

$$\lambda_p = 10.8 < \frac{b_{fc}}{2t_{fc}} = 14.9 < \lambda_r = 19.2$$ Flange - noncompact

- Check web slenderness, Table B4.1b Case 15

$$\lambda = \frac{h_c}{t_w} = \frac{h}{t_w} = \frac{48}{0.375} = 128 \le \left(\frac{h}{t}\right)_{max} = \frac{0.40E}{F_y} = 322$$ F13-4

$$\lambda_p = 3.76\sqrt{\frac{E}{F_y}} = 3.76\sqrt{\frac{29,000}{36}} = 107 \quad \lambda_r = 5.70\sqrt{\frac{E}{F_y}} = 5.70\sqrt{\frac{29,000}{36}} = 162$$

Web - noncompact

28

Example 1

- Since the web is noncompact, we must use Section F4.
- The web plastification factor impacts all limit states, so first determine R_{pc}.

$$\frac{I_{yc}}{I_y} = \frac{0.875(26.0)^3/12}{2560} = \frac{1282}{2560} = 0.50 > 0.23$$

$$\frac{M_p}{M_{yc}} = \frac{F_y Z}{F_y S_{xc}} = \frac{1330}{1230} = 1.08$$

For this shape, the web contributes 0.2 in.[4] to I_y

29

Example 1

- So R_{pc} becomes

This inequality will ALWAYS be satisfied

$$R_{pc} = \left[\frac{M_p}{M_{yc}} - \left(\frac{M_p}{M_{yc}} - 1\right)\left(\frac{\lambda - \lambda_{pw}}{\lambda_{rw} - \lambda_{pw}}\right)\right] \leq \frac{M_p}{M_{yc}}$$

F4-9b

$$= \left[1.08 - (1.08 - 1)\left(\frac{128 - 107}{162 - 107}\right)\right] = 1.05 \leq 1.08$$

30

Example 1

- F4.1 Compression flange yielding

$M_n = R_{pc} M_{yc} = 1.05(36)(1230) = 46{,}500$ in.-kips $= 3{,}880$ ft-kips   F4-1

- F4.3 Compression flange local buckling
  - We found that the flange was noncompact

$$M_n = R_{pc} M_{yc} - \left(R_{pc} M_{yc} - F_L S_{xc}\right)\left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right) \quad \text{F4-13}$$

$$= 46{,}500 - \left(46{,}500 - 0.7(36)(1230)\right)\left(\frac{14.9 - 10.8}{19.2 - 10.8}\right)$$

$$= 38{,}900 \text{ in.-kips} = 3{,}240 \text{ ft-kips}$$

31

Example 1

- F4.4 Tension flange yielding

  Since $S_{xt} \geq S_{xc}$ this limit state does not apply

- F4.2 Lateral-torsional buckling
  - Additional section properties

$$J = \sum \frac{bt^3}{3} = \frac{2(26)(0.875)^3 + 48(0.375)^3}{3} = 12.5 \text{ in.}^4$$ See Design Guide 9

$$r_t = \frac{b_{fc}}{\sqrt{12\left(1 + \frac{1}{6} a_w\right)}}$$ F4-11

r_t is the radius of gyration of the compression flange plus 1/6 the web

32

Example 1

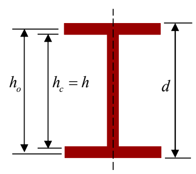
- F4.2 Lateral-torsional buckling
  - Additional section properties

$$a_w = \frac{h_c t_w}{b_{fc} t_{fc}} = \frac{48(0.375)}{26(0.875)} = 0.791$$

$$h = 48 \text{ in.}$$

$$h_o = 48 + 0.875 = 48.875 \text{ in.}$$

$$d = 48 + 2(0.875) = 49.75 \text{ in.}$$



$$r_t = \frac{b_{fc}}{\sqrt{12\left(1 + \frac{1}{6} a_w\right)}} \quad \text{F4-11}$$

$$= \frac{26}{\sqrt{12\left(1 + \frac{1}{6}(0.791)\right)}}$$

$$= 7.05 \text{ in.}$$

33

Example 1

- F4.2 Lateral-torsional buckling

$$L_p = 1.1 r_t \sqrt{\frac{E}{F_y}} = 1.1(7.05)\sqrt{\frac{29,000}{36}} = 220 \text{ in.} \Rightarrow 18.3 \text{ ft} \quad \text{F4-7}$$

$$L_r = 1.95 r_t \frac{E}{F_L} \sqrt{\frac{J}{S_{xc} h_o} + \sqrt{\left(\frac{J}{S_{xc} h_o}\right)^2 + 6.76\left(\frac{F_L}{E}\right)^2}} \quad \text{F4-8}$$

$$= 1.95(7.05)\left(\frac{29,000}{0.7(36)}\right)\sqrt{\frac{12.5}{1230(48.875)} + \sqrt{\left(\frac{12.5}{1230(48.875)}\right)^2 + 6.76\left(\frac{0.7(36)}{29,000}\right)^2}}$$

$$= 787 \text{ in.} \Rightarrow 65.6 \text{ ft}$$

34

Example 1

- F4.2 Lateral-torsional buckling

When $L_p < L_b \le L_r$

$$M_n = C_b\left[R_{pc}M_{yc} - \left(R_{pc}M_{yc} - F_L S_{xc}\right)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \le R_{pc}M_{yc} \quad \text{F4-2}$$

$$= 1.0\left[46{,}500 - \left(46{,}500 - 0.7(36)(1230)\right)\left(\frac{L_b - 18.3}{65.6 - 18.3}\right)\right]$$

$$= 46{,}500 - 328\left(L_b - 18.3\right) \quad \text{(in.-kips)}$$

35

Example 1

- F4.2 Lateral-torsional buckling

When $L_b > L_r$

$$M_n = \frac{C_b \pi^2 E S_{xc}}{\left(\frac{L_b}{r_t}\right)^2}\sqrt{1 + 0.078\frac{J}{S_{xc}h_o}\left(\frac{L_b}{r_t}\right)^2} \le R_{pc}M_{yc} \quad \text{F4-3, F4-5}$$

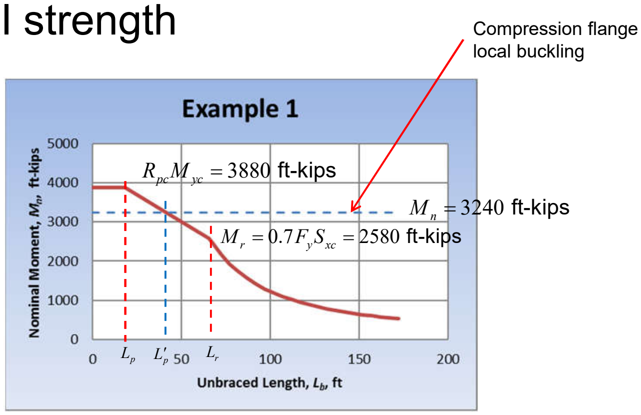
$$= \frac{1.0\pi^2(29{,}000)(1230)}{\left(\frac{(12\text{in./ft})L_b}{7.05}\right)^2}\sqrt{1 + 0.078\left(\frac{12.5}{1230(48.875)}\right)\left(\frac{12L_b}{7.05}\right)^2}$$

$$= \frac{1.22\times 10^8}{L_b^2}\sqrt{1 + 4.70\times 10^{-5}L_b^2} \quad \text{(in.-kips)}$$

36

Example 1

- Nominal strength

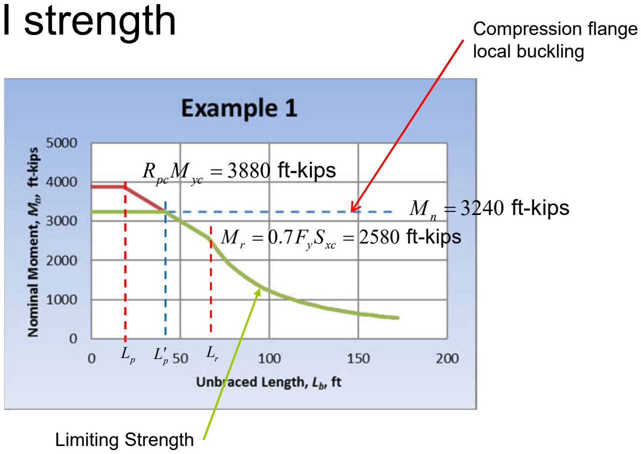


L_p = 18.3 ft
L'_p = 41.7 ft
L_r = 65.6 ft

37

Example 1

- Nominal strength



L_p = 18.3 ft
L'_p = 41.7 ft
L_r = 65.6 ft

38

Plate Girders

- For singly symmetric girders, nothing will change in our approach.
- However,

$$h_c \neq h$$

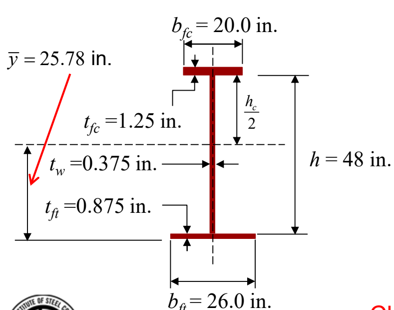
$$S_{xc} \neq S_{xt}$$

- So, we must be careful when h and h_c or S, S_{xc} and S_{xt} are called for.

39

Example 2

- Determine the nominal moment strength of the given singly symmetric plate girder, A36 steel.



Section Properties

$I_x = 32,200$ in.^{4}	$d = 50.125$ in.
$I_y = 2120$ in.^{4}	$b_{fc} = 20.0$ in.
$S_{xc} = 1320$ in.^{3}	$t_{fc} = 1.25$ in.
$S_{xt} = 1250$ in.^{3}	$b_{ft} = 26.0$ in.
$Z_x = 1380$ in.^{3}	$t_{ft} = 0.875$ in.
	$t_w = 0.375$ in.

Changed the size of the top flange.

$$S_{xt} < S_{xc}$$

40

Example 2

- Section F13 limitations

$$\frac{I_{yc}}{I_y} = \frac{1.25(20.0)^3/12}{2120} = \frac{833}{2120} = 0.39 \geq 0.1$$ F13-2

No stiffeners so $a/h > 1.5$ the same as for example 1

$$\frac{h}{t_w} = \frac{48.0}{0.375} = 128 \leq \left(\frac{h}{t_w}\right)_{max} = \frac{0.4E}{F_y} = 322$$ F13-4

41

Example 2

- Check flange slenderness (compression flange)

$$\frac{b_{fc}}{2t_{fc}} = \frac{20}{2(1.25)} = 8.0$$

$$\lambda_p = 0.38\sqrt{\frac{E}{F_y}} = 0.38\sqrt{\frac{29,000}{36}} = 10.8 \quad \lambda_r = 0.95\sqrt{\frac{k_c E}{F_L}} = 0.95\sqrt{\frac{k_c(29,000)}{F_L}} = ?$$

$$k_c = \frac{4}{\sqrt{h/t_w}} = \frac{4}{\sqrt{48/0.375}} = 0.354$$ (but no less than 0.35 nor more than 0.76)

$$\frac{S_{xt}}{S_{xc}} = \frac{1250}{1320} = 0.947 \geq 0.7$$ therefore $F_L = 0.7F_y$ F4-6a

42

Example 2

- Check flange slenderness, Table B4.1b, Case 11

$$\lambda_r = 0.95\sqrt{\frac{k_c E}{F_L}} = 0.95\sqrt{\frac{0.354(29{,}000)}{0.7(36)}} = 19.2$$

$$\frac{b_f}{2t_f} = 8.0 < \lambda_p = 10.8 < \lambda_r = 19.2$$ Flange - compact

- Check web slenderness, Table B4.1b, Case 16

$h_c = 2(48.0 + 0.875 - 25.78) = 46.2$ in. $\quad \frac{h_c}{t_w} = \frac{46.2}{0.375} = 123$ ← Less slender than Example 1

$$\lambda_p = \frac{\frac{h_c}{h_p}\sqrt{\frac{E}{F_y}}}{\left(0.54\frac{M_p}{M_y} - 0.09\right)^2} \le \lambda_r \qquad \lambda_r = 5.70\sqrt{\frac{E}{F_y}} = 5.70\sqrt{\frac{29{,}000}{36}} = 162$$

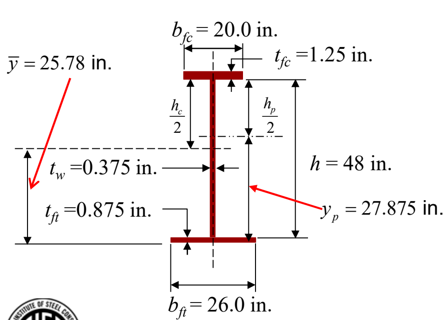
43

Example 2

$M_y = F_y S_x = 36(1250) = 45{,}000$ in.-kips

$M_p = F_y Z_x = 36(1380) = 49{,}680$ in.-kips

$h_p = 2(48.0 + 0.875 - 27.875) = 42.0$ in.



$$\lambda_p = \frac{\frac{h_c}{h_p}\sqrt{\frac{E}{F_y}}}{\left(0.54\frac{M_p}{M_y} - 0.09\right)^2} \le \lambda_r$$

$$= \frac{\frac{46.2}{42.0}\sqrt{\frac{29{,}000}{36.0}}}{\left(0.54\frac{49{,}680}{45{,}000} - 0.09\right)^2} = 122 \le \lambda_r = 162$$

44

Example 2

- Check web slenderness

$$\frac{h_c}{t_w} = \frac{46.2}{0.375} = 123$$

Less slender than Example 1

$$\lambda_p = \frac{\frac{h_c}{h_p}\sqrt{\frac{E}{F_y}}}{\left(0.54\frac{M_p}{M_y} - 0.09\right)^2} = 122 \qquad \lambda_r = 5.70\sqrt{\frac{E}{F_y}} = 5.70\sqrt{\frac{29{,}000}{36}} = 162$$

- Thus, the web is just barely noncompact, we again should use Section F4.

45

Example 2

- The web plastification factor impacts all limit states, so first determine R_{pc}.

$$\frac{I_{yc}}{I_y} = 0.39 > 0.23$$

$$\frac{M_p}{M_{yc}} = \frac{F_y Z}{F_y S_{xc}} = \frac{49{,}680}{36(1320)} = \frac{49{,}680}{47{,}520} = 1.05$$

$$M_p \le 1.6F_y S = 1.6(36)(1250) = 72{,}000 \text{ in.-kips}$$

46

Example 2

- So R_{pc} becomes

Remember from Example 1, this inequality will ALWAYS be satisfied

$$R_{pc} = \left[\frac{M_p}{M_{yc}} - \left(\frac{M_p}{M_{yc}} - 1\right)\left(\frac{\lambda - \lambda_{pw}}{\lambda_{rw} - \lambda_{pw}}\right)\right] \le \frac{M_p}{M_{yc}} \quad \text{F4-9b}$$

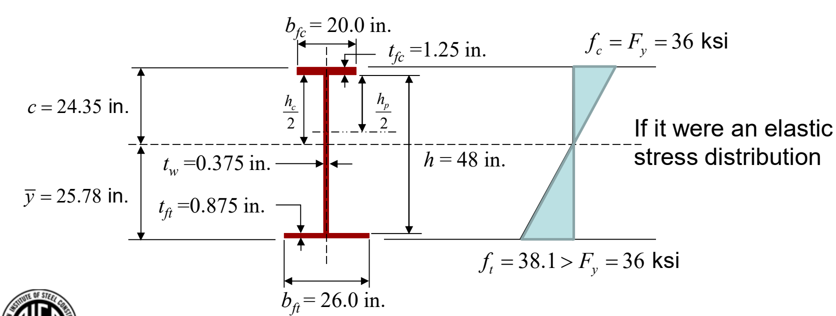
$$= \left[1.05 - (1.05 - 1)\left(\frac{123 - 122}{162 - 122}\right)\right] = 1.05 \le 1.05$$

47

Example 2

- F4.1 Compression flange yielding

$$M_n = R_{pc} M_{yc} = 1.05(36)(1320) = 49{,}900 \text{ in.-kips} \quad \text{F4-1}$$
$$= 4{,}160 \text{ ft-kips}$$



48

Example 2

- F4.3 Compression flange local buckling
  - We found that the flange was compact, so this limit state does not apply

$$\frac{b_f}{2t_f} = 8.0 < \lambda_p = 10.8 < \lambda_r = 19.2$$ Flange - compact

49

Example 2

- F4.4 Tension flange yielding

Since $S_{xt} < S_{xc}$ this limit state applies

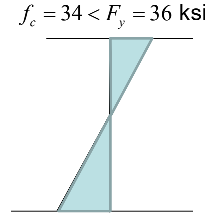
$$\frac{M_p}{M_{yt}} = \frac{F_y Z}{F_y S_{xt}} = \frac{49,680}{36(1250)} = \frac{49,680}{45,000} = 1.10$$

and

$$R_{pt} = \left[\frac{M_p}{M_{yt}} - \left(\frac{M_p}{M_{yt}} - 1\right)\left(\frac{\lambda - \lambda_{pw}}{\lambda_{rw} - \lambda_{pw}}\right)\right] \le \frac{M_p}{M_{yt}}$$

$$= \left[1.10 - (1.10 - 1)\left(\frac{123 - 122}{162 - 122}\right)\right] = 1.10 \le 1.10$$

$f_c = 34 < F_y = 36$ ksi



$f_t = F_y = 36$ ksi

F4-16b

Note that R_{pc} and R_{pt} are now different

50

Example 2

- F4.4 Tension flange yielding

$$M_n = R_{pt}F_yS_{xt} = 1.10(36)(1250) = 49,500 \text{ in.-kips} \quad \text{F4-15}$$
$$= 4,130 \text{ ft-kips}$$

But again, we do not actually have an elastic stress distribution

51

Example 2

- F4.2 Lateral-torsional buckling
  - Additional section properties

$$J = \sum \frac{bt^3}{3} = \frac{20(1.25)^3 + (26)(0.875)^3 + 48(0.375)^3}{3} = 19.7 \text{ in.}^4$$

$$a_w = \frac{h_c t_w}{b_{fc} t_{fc}} = \frac{46.2(0.375)}{20(1.25)} = 0.693$$

$$h = 48 \text{ in.}$$

$$h_o = 48 + 1.25/2 + 0.875/2 = 49.1 \text{ in.}$$

$$d = 48 + 1.25 + 0.875 = 50.1 \text{ in.}$$

$$r_t = \frac{b_{fc}}{\sqrt{12\left(1 + \frac{1}{6}a_w\right)}} \quad \text{F4-11}$$
$$= \frac{20}{\sqrt{12\left(1 + \frac{1}{6}(0.693)\right)}}$$
$$= 5.47$$

52

Example 2

- F4.2 Lateral-torsional buckling

$$L_p = 1.1 r_t \sqrt{\frac{E}{F_y}} = 1.1(5.47)\sqrt{\frac{29{,}000}{36}} = 171 \text{ in.} \Rightarrow 14.3 \text{ ft}$$ F4-7

$$L_r = 1.95 r_t \frac{E}{F_L}\sqrt{\frac{J}{S_{xc}h_o} + \sqrt{\left(\frac{J}{S_{xc}h_o}\right)^2 + 6.76\left(\frac{F_L}{E}\right)^2}}$$ F4-8

$$= 1.95(5.47)\left(\frac{29{,}000}{0.7(36)}\right)\sqrt{\frac{19.7}{1320(49.1)} + \sqrt{\left(\frac{19.7}{1320(49.1)}\right)^2 + 6.76\left(\frac{0.7(36)}{29{,}000}\right)^2}}$$

$$= 624 \text{ in.} \Rightarrow 52.0 \text{ ft}$$

53

Example 2

- F4.2 Lateral-torsional buckling

When $L_p < L_b \le L_r$

$$M_n = C_b\left[R_{pc}M_{yc} - \left(R_{pc}M_{yc} - F_L S_{xc}\right)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \le R_{pc}M_{yc}$$ F4-2

$$= 1.0\left[49{,}900 - \left(49{,}900 - 0.7(36)(1320)\right)\left(\frac{L_b - 14.3}{52.0 - 14.3}\right)\right]$$

$$= 49{,}900 - 441\left(L_b - 14.3\right) \quad \text{(in.-kips)}$$

54

Example 2

- F4.2 Lateral-torsional buckling

When $L_b > L_r$

$$M_n = \frac{C_b \pi^2 E S_{xc}}{\left(\frac{L_b}{r_t}\right)^2}\sqrt{1+0.078\frac{J}{S_{xc}h_o}\left(\frac{L_b}{r_t}\right)^2} \le R_{pc}M_{yc} \qquad \text{F4-3, F4-5}$$

$$= \frac{1.0\pi^2(29{,}000)(1320)}{\left(\frac{(12\text{in./ft})L_b}{5.47}\right)^2}\sqrt{1+0.078\left(\frac{19.7}{1320(49.1)}\right)\left(\frac{12L_b}{5.47}\right)^2}$$

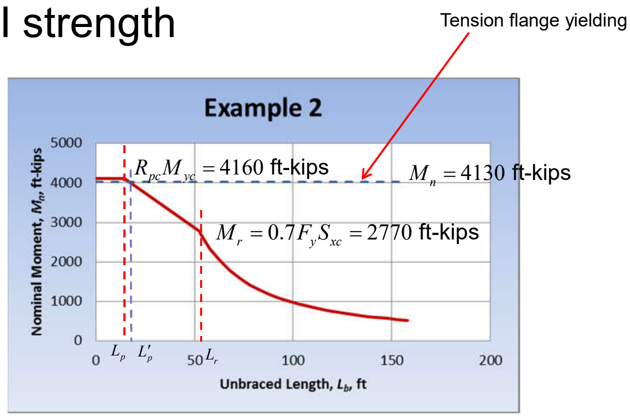
$$= \frac{7.85\times10^7}{L_b^2}\sqrt{1+1.14\times10^{-4}L_b^2} \quad \text{(in.-kips)}$$

55

Example 2

- Nominal strength

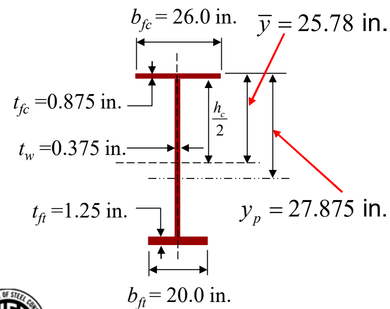
L_p = 14.3 ft
L'_p = 15.2 ft
L_r = 52.0 ft



56

Example 3

- Reverse the flanges of the shape in Example 2 so that the smaller width flange is in tension.



Section Properties

$I_x = 32,200$ in.^{4}

$I_y = 2120$ in.^{4}

$S_{xc} = 1250$ in.^{3}

$S_{xt} = 1320$ in.^{3}

$Z_x = 1380$ in.^{3}

$d = 50.125$ in.

$b_{fc} = 26.0$ in.

$t_{fc} = 0.875$ in.

$b_{ft} = 20.0$ in.

$t_{ft} = 1.25$ in.

$t_w = 0.375$ in.

57

Example 3

- Section F13 limitations

$$\frac{I_{yc}}{I_y} = \frac{0.875(26.0)^3/12}{2120} = \frac{1280}{2120} = 0.60 \le 0.9$$ F13-2

No stiffeners so $a/h > 1.5$ the same as for examples 1 and 2

$$\frac{h}{t_w} = \frac{48.0}{0.375} = 128 \le \left(\frac{h}{t_w}\right)_{max} = \frac{0.4E}{F_y} = 322$$ F13-4

58

Example 3

- Check flange slenderness

Note change from Example 2

$$\frac{b_{fc}}{2t_{fc}} = \frac{26}{2(0.875)} = 14.9$$

$$\lambda_p = 0.38\sqrt{\frac{E}{F_y}} = 0.38\sqrt{\frac{29{,}000}{36}} = 10.8 \qquad \lambda_r = 0.95\sqrt{\frac{k_c E}{F_L}} = 0.95\sqrt{\frac{k_c(29{,}000)}{F_L}} = ?$$

$$k_c = \frac{4}{\sqrt{h/t_w}} = \frac{4}{\sqrt{48/0.375}} = 0.354$$

(but no less than 0.35 nor more than 0.76)

$$\frac{S_{xt}}{S_{xc}} = \frac{1320}{1250} = 1.06 \geq 0.7 \text{ therefore } F_L = 0.7F_y$$ F4-6a

Note change from Example 2

59

Example 3

- Check flange slenderness, Table B4.1b Case 11

$$\lambda_r = 0.95\sqrt{\frac{k_c E}{F_L}} = 0.95\sqrt{\frac{0.354(29{,}000)}{0.7(36)}} = 19.2$$

$$\lambda_p = 10.8 < \frac{b_{fc}}{2t_{fc}} = 14.9 < \lambda_r = 19.2$$ Flange - noncompact

- Check web slenderness, Table B4.1b Case 16

$h_c = 2(25.78 - 0.875) = 49.8$ in. $\qquad \frac{h_c}{t_w} = \frac{49.8}{0.375} = 133$ Change from Example 2

$$\lambda_p = \frac{\frac{h_c}{h_p}\sqrt{\frac{E}{F_y}}}{\left(0.54\frac{M_p}{M_y} - 0.09\right)^2} \leq \lambda_r \qquad \lambda_r = 5.70\sqrt{\frac{E}{F_y}} = 5.70\sqrt{\frac{29{,}000}{36}} = 162$$

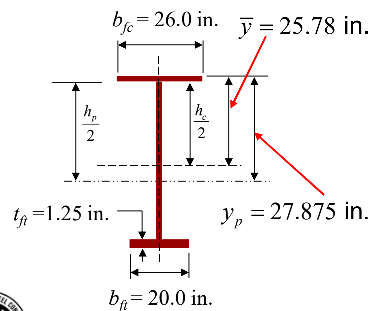
60

Example 3

$$M_y = F_y S_x = 36(1250) = 45{,}000 \text{ in.-kips}$$

$$M_p = F_y Z_x = 36(1380) = 49{,}680 \text{ in.-kips}$$

$$h_p = 2(27.875 - 0.875) = 54.0 \text{ in.}$$



$$\lambda_p = \frac{\frac{h_c}{h_p}\sqrt{\frac{E}{F_y}}}{\left(0.54\frac{M_p}{M_y} - 0.09\right)^2} \le \lambda_r$$

$$= \frac{\frac{49.8}{54.0}\sqrt{\frac{29{,}000}{36.0}}}{\left(0.54\frac{49{,}680}{45{,}000} - 0.09\right)^2} = 102 \le \lambda_r = 162$$

61

Example 3

- Check web slenderness

$$\frac{h_c}{t_w} = \frac{49.8}{0.375} = 133$$

$$\lambda_p = \frac{\frac{h_c}{h_p}\sqrt{\frac{E}{F_y}}}{\left(0.54\frac{M_p}{M_y} - 0.09\right)^2} = 102 \qquad \lambda_r = 5.70\sqrt{\frac{E}{F_y}} = 5.70\sqrt{\frac{29{,}000}{36}} = 162$$

- Thus, the web is noncompact and we again will use Section F4.

62

Example 3

- The web plastification factor impacts all limit states, so first determine R_{pc}.

$$\frac{I_{yc}}{I_y} = 0.60 > 0.23$$

$$\frac{M_p}{M_{yc}} = \frac{F_y Z}{F_y S_{xc}} = \frac{49,680}{36(1250)} = \frac{49,680}{45,000} = 1.10$$

63

Example 3

- So R_{pc} becomes

Remember this inequality will ALWAYS be satisfied

$$R_{pc} = \left[\frac{M_p}{M_{yc}} - \left(\frac{M_p}{M_{yc}} - 1\right)\left(\frac{\lambda - \lambda_{pw}}{\lambda_{rw} - \lambda_{pw}}\right)\right] \le \frac{M_p}{M_{yc}} \qquad \text{F4-9b}$$

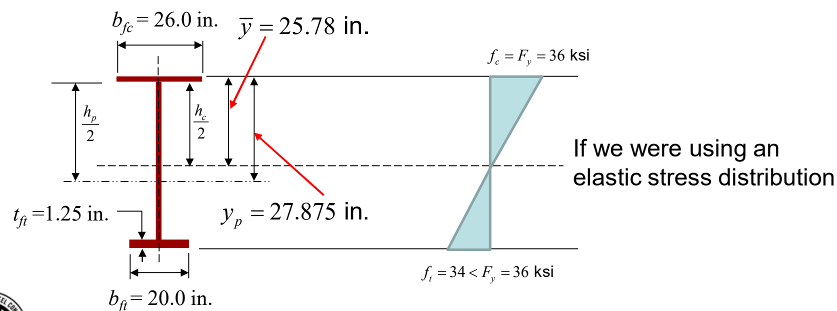
$$= \left[1.10 - (1.10 - 1)\left(\frac{133 - 102}{162 - 102}\right)\right] = 1.05 \le 1.10$$

64

Example 3

- F4.1 Compression flange yielding

$$M_n = R_{pc}M_{yc} = 1.05(36)(1250) = 47,300 \text{ in.-kips} = 3,940 \text{ ft-kips}$$ F4-1



65

Example 3

- F4.3 Compression flange local buckling
  - We found that the flange was noncompact

$$M_n = R_{pc}M_{yc} - \left(R_{pc}M_{yc} - F_L S_{xc}\right)\left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right)$$ F4-13

$$= 47,300 - \left(47,300 - 0.7(36)(1250)\right)\left(\frac{14.9 - 10.8}{19.2 - 10.8}\right) = 39,600 \text{ in.-kips}$$

$$= 3,300 \text{ ft-kips}$$

66

Example 3

- F4.4 Tension flange yielding

  Since $S_{xt} \geq S_{xc}$ this limit state does not apply

- F4.2 Lateral-torsional buckling
  - Additional section properties

$$J = \sum \frac{bt^3}{3} = \frac{20(1.25)^3 + (26)(0.875)^3 + 48(0.375)^3}{3} = 19.7 \text{ in.}^4$$

67

Example 3

- F4.2 Lateral-torsional buckling
  - Additional section properties

$$a_w = \frac{h_c t_w}{b_{fc} t_{fc}} = \frac{49.8(0.375)}{26(0.875)} = 0.821$$

$$h = 48 \text{ in.}$$

$$h_o = 48 + 1.25/2 + 0.875/2 = 49.1 \text{ in.}$$

$$d = 48 + 1.25 + 0.875 = 50.1 \text{ in.}$$

$$r_t = \frac{b_{fc}}{\sqrt{12\left(1 + \frac{1}{6} a_w\right)}} \quad \text{F4-11}$$

$$= \frac{26}{\sqrt{12\left(1 + \frac{1}{6}(0.821)\right)}}$$

$$= 7.04 \text{ in.}$$

68

Example 3

- F4.2 Lateral-torsional buckling

$$L_p = 1.1 r_t \sqrt{\frac{E}{F_y}} = 1.1(7.04)\sqrt{\frac{29,000}{36}} = 220 \text{ in.} \Rightarrow 18.3 \text{ ft}$$ F4-7

$$L_r = 1.95 r_t \frac{E}{F_L}\sqrt{\frac{J}{S_{xc}h_o} + \sqrt{\left(\frac{J}{S_{xc}h_o}\right)^2 + 6.76\left(\frac{F_L}{E}\right)^2}}$$ F4-8

$$= 1.95(7.04)\left(\frac{29,000}{0.7(36)}\right)\sqrt{\frac{19.7}{1250(49.1)} + \sqrt{\left(\frac{19.7}{1250(49.1)}\right)^2 + 6.76\left(\frac{0.7(36)}{29,000}\right)^2}}$$

$$= 806 \text{ in.} \Rightarrow 67.2 \text{ ft}$$

69

Example 3

- F4.2 Lateral-torsional buckling

When $L_p < L_b \le L_r$

$$M_n = C_b\left[R_{pc}M_{yc} - \left(R_{pc}M_{yc} - F_L S_{xc}\right)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \le R_{pc}M_{yc}$$ F4-2

$$= 1.0\left[47,300 - \left(47,300 - 0.7(36)(1250)\right)\left(\frac{L_b - 18.3}{67.2 - 18.3}\right)\right]$$

$$= 47,300 - 323\left(L_b - 18.3\right) \quad \text{(in.-kips)}$$

70

Example 3

- F4.2 Lateral-torsional buckling

When $L_b > L_r$

$$M_n = \frac{C_b \pi^2 E S_{xc}}{\left(\frac{L_b}{r_t}\right)^2} \sqrt{1 + 0.078 \frac{J}{S_{xc} h_o} \left(\frac{L_b}{r_t}\right)^2} \leq R_{pc} M_{yc} \qquad \text{F4-3, F4-5}$$

$$= \frac{1.0\pi^2 (29{,}000)(1250)}{\left(\frac{(12\text{in./ft}) L_b}{7.04}\right)^2} \sqrt{1 + 0.078 \left(\frac{19.7}{1250(49.1)}\right)\left(\frac{12 L_b}{7.04}\right)^2}$$

$$= \frac{1.23 \times 10^8}{L_b^2} \sqrt{1 + 7.27 \times 10^{-5} L_b^2} \quad \text{(in.-kips)}$$

71

Example 3

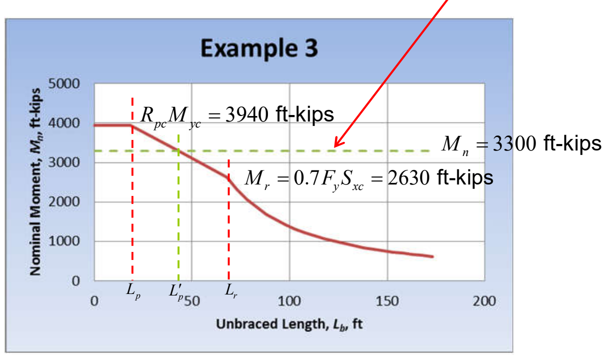
- Nominal strength

Compression flange local buckling

L_p = 18.3 ft

L'_p = 42.4 ft

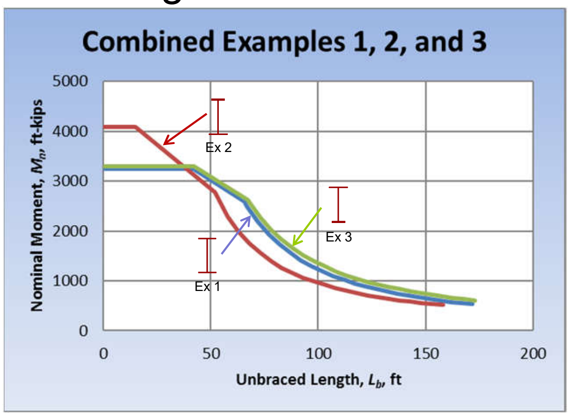
L_r = 67.2 ft



72

Example 3

- Nominal strength



73

Plate Girders

- F5. for doubly and singly symmetric I-shaped members with slender web
  - As with F4, the limit state of *web local buckling* does not lead to a specific nominal strength
  - Rather, web local buckling modifies the strength determined for the other limit states; *yielding*, *flange local buckling* and *lateral-torsional buckling*, through the use of the bending strength reduction factor, R_{pg}.

74

Plate Girders

- F5.1 Compression flange yielding

$$M_n = R_{pg} F_y S_{xc} \quad \text{F5-1}$$

- F5.4 Tension flange yielding, $S_{xt} < S_{xc}$

$$M_n = F_y S_{xt} \quad \text{F5-10}$$

- F5.3 Compression flange local buckling

$$M_n = R_{pg} F_{cr} S_{xc} \quad \text{F5-7}$$

Noncompact $F_{cr} = F_y - \left(0.3F_y\right)\left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right)$ F5-8

Slender $F_{cr} = \frac{0.9Ek_c}{\left(\frac{b_f}{2t_f}\right)^2}$ F5-9

75

Plate Girders

- Look at the bending strength reduction factor, R_{pg}

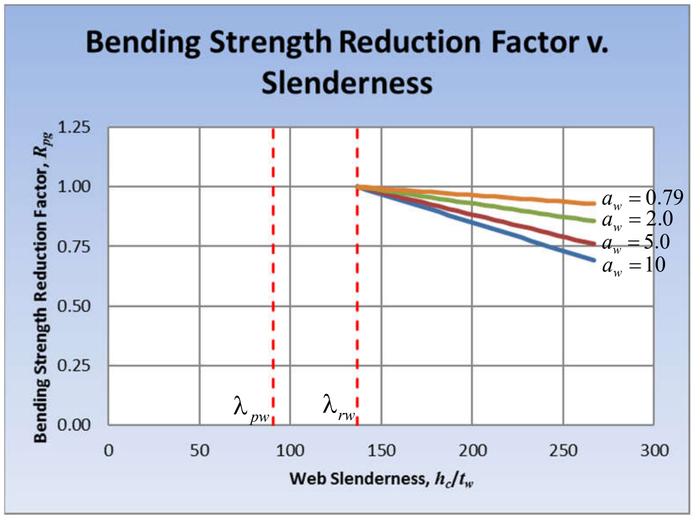
$$R_{pg} = 1 - \frac{a_w}{1,200 + 300a_w}\left(\frac{h_c}{t_w} - 5.7\sqrt{\frac{E}{F_y}}\right) \leq 1.0 \quad \text{F5-6}$$

$$a_w = \frac{h_c t_w}{b_{fc} t_{fc}} \leq 10.0$$

F4-12 plus the limit to 10

76

Plate Girders



77

Plate Girders

- F5.2 Lateral-torsional buckling

$$M_n = R_{pg} F_{cr} S_{xc} \qquad \text{F5-2}$$

When $L_p < L_b \le L_r$

$$F_{cr} = C_b \left[F_y - \left(0.3F_y\right) \left(\frac{L_b - L_p}{L_r - L_p} \right) \right] \le F_y \qquad \text{F5-3}$$

When $L_b > L_r$

$$F_{cr} = \frac{C_b \pi^2 E}{\left(\frac{L_b}{r_t} \right)^2} \qquad \text{F5-4}$$

78

Plate Girders

- F5.2 Lateral-torsional buckling

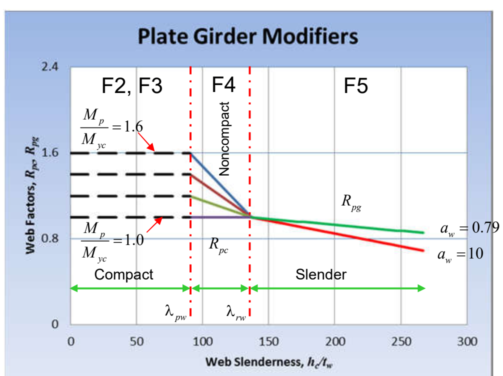
$$L_p = 1.1 r_t \sqrt{\frac{E}{F_y}}$$ F4-7

$$r_t = \frac{b_{fc}}{\sqrt{12\left(1+\frac{1}{6}a_w\right)}}$$ F4-11

$$L_r = \pi r_t \sqrt{\frac{E}{0.7F_y}}$$ F5-5

79

Plate Girders

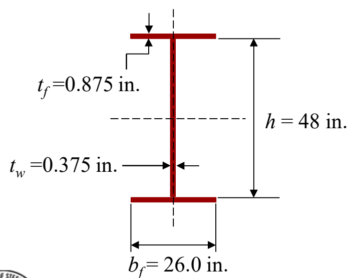


A user note in Section F4 says that Section F5 may conservatively be used for shapes that fall under the provisions of Section F4

80

Example 4

- Reconsider the plate girder from Example 1 using Section F5



Section Properties

$I_x = 30,600$ in.^{4}

$I_y = 2560$ in.^{4}

$S_x = 1230$ in.^{3}

$Z_x = 1330$ in.^{3}

$d = 49.75$ in.

$b_f = 26.0$ in.

$t_w = 0.375$ in.

$t_f = 0.875$ in.

81

Example 4

- Flange and web slenderness are the same as already calculated in Example 1

$$\lambda_p = 10.8 \leq \frac{b_{fc}}{2t_{fc}} = 14.9 < \lambda_r = 19.2$$

$$\lambda_p = 107 \leq \frac{h}{t_w} = 128 < \lambda_r = 162$$

- Since the web is noncompact, we could use Section F4 as we did in Example 1 but we are permitted to <u>conservatively</u> use Section F5

82

Example 4

- The bending strength reduction factor impacts other limit states, so first determine R_{pg}. From Example 1, $a_w = 0.791$

$$R_{pg} = 1 - \frac{a_w}{1,200 + 300a_w}\left(\frac{h_c}{t_w} - 5.7\sqrt{\frac{E}{F_y}}\right) \leq 1.0 \quad \text{F5-6}$$

$$= 1 - \frac{0.791}{1,200 + 300(0.791)}\left(\frac{48.0}{0.375} - 5.7\sqrt{\frac{29,000}{36}}\right)$$

$= 1.02 > 1.0$ therefore $R_{pg} = 1$

When using F5 in place of F4, R_{pg} will always be 1.0

83

Example 4

- F5.1 Compression flange yielding

$$M_n = R_{pg}M_{yc} = 1.0(36)(1230) = 44,300 \text{ in.-kips} = 3,690 \text{ ft-kips} \quad \text{F5-1}$$

- F5.3 Compression flange local buckling
  - We found that the flange was noncompact

$$F_{cr} = F_y - (0.3F_y)\left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right) \quad \text{F5-8}$$

$$= 36 - 0.3(36)\left(\frac{14.9 - 10.8}{19.2 - 10.8}\right)$$

$= 30.7$ ksi

$$M_n = R_{pg}F_{cr}S_{xc} \quad \text{F5-7}$$

$= 1.0(30.7)(1230)$

$= 37,800$ in.-kips

$= 3,150$ ft-kips

Note that both of these are elastic

84

Example 4

- F5.4 Tension flange yielding

  Since $S_{xt} \geq S_{xc}$ this limit state does not apply

- F5.2 Lateral-torsional buckling

$r_t = 7.05$ F4-11

$$L_p = 1.1 r_t \sqrt{\frac{E}{F_y}} = 1.1(7.05)\sqrt{\frac{29,000}{36}} = 220 \text{ in.} \Rightarrow 18.3 \text{ ft}$$ F4-7

$$L_r = \pi r_t \sqrt{\frac{E}{0.7F_y}} = \pi(7.05)\sqrt{\frac{29,000}{0.7(36)}} = 751 \text{ in.} \Rightarrow 62.6 \text{ ft}$$ F5-5

85

Example 4

- F5.2 Lateral-torsional buckling

$$M_n = R_{pg} F_{cr} S_{xc}$$ F5-2

When $L_p < L_b \leq L_r$

$$F_{cr} = C_b \left[F_y - (0.3F_y)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \leq F_y$$ F5-3

$$= 1.0\left[36 - (0.3(36))\left(\frac{L_b - 18.3}{62.6 - 18.3}\right)\right]$$

$$= 36 - 0.244(L_b - 18.3) \quad \text{(ksi)}$$

86

Example 4

- F5.2 Lateral-torsional buckling

When $L_b > L_r$

$$F_{cr} = \frac{C_b \pi^2 E}{\left(\frac{L_b}{r_t}\right)^2} \leq F_y \qquad \text{F5-4}$$

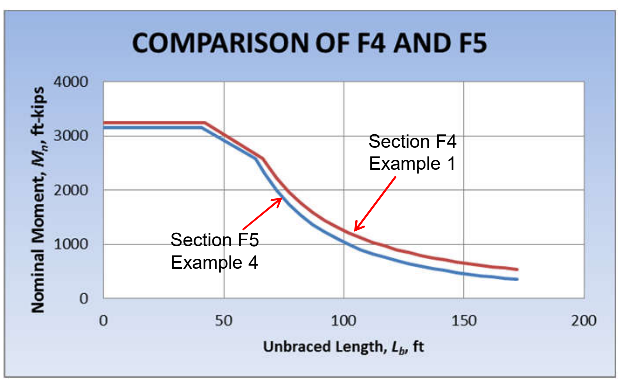
$$= \frac{1.0\pi^2 (29{,}000)}{\left(\frac{(12\text{in./ft})L_b}{7.05}\right)^2}$$

$$= \frac{9.88 \times 10^4}{L_b^2} \quad \text{(ksi)}$$

87

Example 4

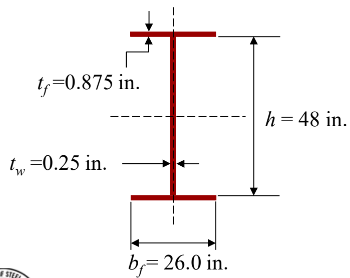
From this example, it does appear that using F5 in place of F4 is a bit conservative. Is the simplicity worth it?



88

Example 5

- Determine the nominal strength of a plate girder with a slender web. This is Example 1 with a reduced web



Section Properties

$I_x = 29{,}500$ in.^{4}

$I_y = 2560$ in.^{4}

$S_x = 1190$ in.^{3}

$Z_x = 1260$ in.^{3}

$d = 49.75$ in.

$b_f = 26.0$ in.

$t_w = 0.250$ in.

$t_f = 0.875$ in.

89

Example 5

- Check flange slenderness

$$\frac{b_{fc}}{2t_{fc}} = \frac{26}{2(0.875)} = 14.9$$

$$\lambda_p = 0.38\sqrt{\frac{E}{F_y}} = 0.38\sqrt{\frac{29{,}000}{36}} = 10.8 \quad \lambda_r = 0.95\sqrt{\frac{k_c E}{F_L}} = 0.95\sqrt{\frac{k_c(29{,}000)}{F_L}} = ?$$

$$k_c = \frac{4}{\sqrt{h/t_w}} = \frac{4}{\sqrt{48/0.25}} = 0.289$$

A change from Example 1

(but no less than 0.35 nor more than 0.76)

$$k_c = 0.35$$

90

Example 5

- Check flange slenderness

$$\lambda_r = 0.95\sqrt{\frac{k_c E}{F_L}} = 0.95\sqrt{\frac{0.35(29,000)}{0.7(36)}} = 19.1$$

$$\lambda_p = 10.8 < \frac{b_{fc}}{2t_{fc}} = 14.9 < \lambda_r = 19.1$$

Flange - noncompact

- Check web slenderness (limits from Example 1)

$$\lambda_p = 107 \le \frac{h}{t_w} = \frac{48}{0.25} = 192 > \lambda_r = 162$$

$$\left(\frac{h}{t}\right)_{max} = \frac{0.40E}{F_y} = 322 \quad \text{F13-4}$$

Therefore, the web is slender, and we must use Section F5

91

Example 5

- The bending strength reduction factor, R_{pg}.

$$a_w = \frac{h_c t_w}{b_{fc} t_{fc}} = \frac{48(0.25)}{26(0.875)} = 0.527 < 10 \quad \text{F4-12}$$

$$R_{pg} = 1 - \frac{a_w}{1,200 + 300a_w}\left(\frac{h_c}{t_w} - 5.7\sqrt{\frac{E}{F_y}}\right) \le 1.0 \quad \text{F5-6}$$

$$= 1 - \frac{0.527}{1,200 + 300(0.527)}\left(\frac{48.0}{0.250} - 5.7\sqrt{\frac{29,000}{36}}\right)$$

$$= 0.988 < 1.0 \text{ therefore } R_{pg} = 0.988$$

92

Example 5

- F5.1 Compression flange yielding

$M_n = R_{pg}M_{yc} = 0.988(36)(1190) = 42,300$ in.-kips $= 3,530$ ft-kips F5-1

- F5.3 Compression flange local buckling
  - We found that the flange was noncompact

$F_{cr} = F_y - (0.3F_y)\left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right)$ F5-8

$= 36 - 0.3(36)\left(\frac{14.9 - 10.8}{19.1 - 10.8}\right)$

$= 30.7$ ksi

$M_n = R_{pg}F_{cr}S_{xc}$ F5-7

$= 0.988(30.7)(1190)$

$= 36,100$ in.-kips

$= 3,010$ ft-kips

93

Example 5

- F5.4 Tension flange yielding

  Since $S_{xt} \geq S_{xc}$ this limit state does not apply

- F5.2 Lateral-torsional buckling
  - Additional section properties

$a_w = 0.527$

$h = 48$ in.

$h_o = 48 + 0.875 = 48.875$ in.

$d = 48 + 2(0.875) = 49.75$ in.

$r_t = \frac{b_{fc}}{\sqrt{12\left(1 + \frac{1}{6}a_w\right)}}$ F4-11

$= \frac{26}{\sqrt{12\left(1 + \frac{1}{6}(0.527)\right)}}$

$= 7.20$ in.

94

Example 5

- F5.2 Lateral-torsional buckling

$$L_p = 1.1 r_t \sqrt{\frac{E}{F_y}} = 1.1(7.20)\sqrt{\frac{29{,}000}{36}} = 225 \text{ in.} \Rightarrow 18.8 \text{ ft}$$ F4-7

$$L_r = \pi r_t \sqrt{\frac{E}{0.7F_y}} = \pi(7.20)\sqrt{\frac{29{,}000}{0.7(36)}} = 767 \text{ in.} \Rightarrow 63.9 \text{ ft}$$ F5-5

95

Example 5

- F5.2 Lateral-torsional buckling

$$M_n = R_{pg} F_{cr} S_{xc} = 0.988 F_{cr} S_{xc}$$ F5-2

When $L_p < L_b \le L_r$

$$F_{cr} = C_b \left[F_y - (0.3F_y)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \le F_y$$ F5-3

$$= 1.0\left[36 - (0.3(36))\left(\frac{L_b - 18.8}{63.9 - 18.8}\right)\right]$$

$$= 36 - 0.239(L_b - 18.8) \quad \text{(ksi)}$$

96

Example 5

- F5.2 Lateral-torsional buckling

When $L_b > L_r$

$$F_{cr} = \frac{C_b \pi^2 E}{\left(\frac{L_b}{r_t}\right)^2} \leq F_y \qquad \text{F5-4}$$

$$= \frac{1.0\pi^2 (29{,}000)}{\left(\frac{(12\text{in./ft})L_b}{7.20}\right)^2}$$

$$= \frac{1.03 \times 10^5}{L_b^2} \quad \text{(ksi)}$$

97

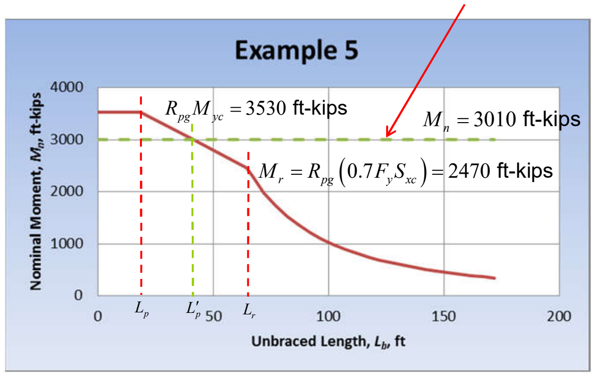
Example 5

Flange local buckling

L_p = 18.8 ft

L'_p = 41.3 ft

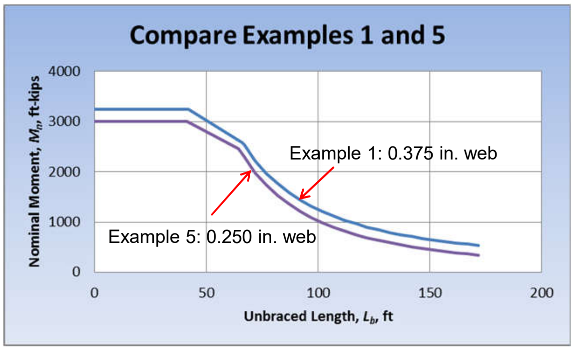
L_r = 63.9 ft



98

Example 5

- Look at the impact of reducing the web thickness



99

Compare F4 and F5

- Compression flange yielding

$$M_n = R_{pc} F_y S_{xc} \quad \text{F4-1} \qquad R_{pc} = 1 \text{ to } M_p/M_{yc}$$

$$M_n = R_{pg} F_y S_{xc} \quad \text{F5-1} \qquad R_{pg} \le 1.0$$

See Slide 80 (next)

- Compression flange local buckling

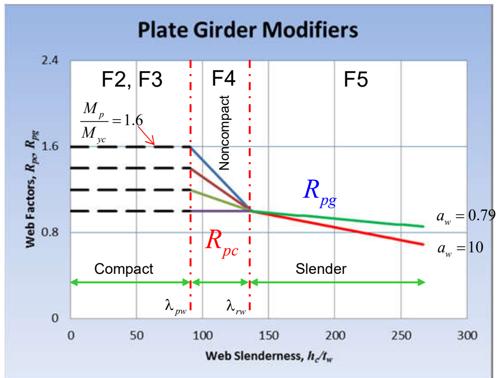
Noncompact

$$M_n = R_{pc} M_{yc} - \left(R_{pc} M_{yc} - F_L S_{xc}\right)\left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right) \quad \text{F4-13}$$

$$M_n = R_{pg} M_{yc} - \left(R_{pg} M_{yc} - R_{pg}\left(0.7 F_y S_{xc}\right)\right)\left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right) \quad \text{F5-8 modified}$$

100

Compare F4 and F5



101

Compare F4 and F5

- F_L, nominal compressive stress above which the inelastic buckling limit states apply.

$$\frac{S_{xt}}{S_{xc}} \geq 0.7 \qquad F_L = 0.7F_y \qquad \text{F4-6a}$$

$$\frac{S_{xt}}{S_{xc}} < 0.7 \qquad F_L = F_y \frac{S_{xt}}{S_{xc}} \geq 0.5F_y \qquad \text{F4-6b}$$

In Section F5, F_L is not used but it is essentially $0.7F_y$

102

Compare F4 and F5

- Compression flange local buckling

Slender

$$M_n = \frac{0.9Ek_cS_{xc}}{\left(\frac{b_f}{2t_f}\right)^2} \qquad \text{F4-14}$$

$$M_n = R_{pg}\frac{0.9Ek_cS_{xc}}{\left(\frac{b_f}{2t_f}\right)^2} \qquad \text{F5-9 modified}$$

Compare F4 and F5

- Tension flange yielding

$$M_n = R_{pt}F_yS_{xt} \qquad \text{F4-15} \qquad R_{pt} = 1 \text{ to } M_p/M_{yt}$$

$$M_n = F_yS_{xt} \qquad \text{F5-10}$$

- Lateral-torsional buckling

When $L_p < L_b \le L_r$

$$M_n = C_b\left[R_{pc}M_{yc} - \left(R_{pc}M_{yc} - F_LS_{xc}\right)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \le R_{pc}M_{yc} \qquad \text{F4-2}$$

$$M_n = C_b\left[R_{pg}M_{yc} - \left(R_{pg}M_{yc} - R_{pg}\left(0.7F_yS_{xc}\right)\right)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \le R_{pg}M_{yc} \qquad \text{F5-3 modified}$$

Compare F4 and F5

- Lateral-torsional buckling

When $L_b > L_r$

$$M_n = \frac{C_b \pi^2 E S_{xc}}{\left(\frac{L_b}{r_t}\right)^2} \sqrt{1 + 0.078 \frac{J}{S_{xc} h_o} \left(\frac{L_b}{r_t}\right)^2} \leq R_{pc} M_{yc}$$ F4-3, F4-5

$$M_n = \frac{R_{pg} C_b \pi^2 E S_{xc}}{\left(\frac{L_b}{r_t}\right)^2} \leq R_{pg} M_{yc}$$ F5-2, F5-4

105

Summary

- We have determined the flexural strength of doubly and singly symmetric girders with noncompact webs.
- We have also looked at how girder strength changed as we altered the flange size.
- We have treated doubly symmetric girders with slender webs and noted how singly symmetric girders would be treated.
- We compared the Section F4 and F5 equations.

106

Lesson 2

- We will next address shear in plate girders.
- We will consider shear strength in I-shaped members with post buckling strength modeled by the rotated stress field theory.
- Then we will look at post buckling strength through tension field action.

107

Thank You

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108

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- Once attendance has been reported, you will be receiving certificates via email from registration@aisc.org.

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