A solved problem 9-9-6-given built section with Fy=65 ksi, we need to find the service Live load of the beam use LRFD design.

Remarks:

1-No tables to use.

2-We have to check Flange local buckling Get Mn.

3-We have to check web local buckling, Get Mn.

4-We need to check lateral Torsional buckling for the whole section, Get Mn.

Select the lowest value of the three cases.

In this post 25_steel beam

(a) which are the cases according to Table B4.1b For Flange, web ?

(b) Check Flange Compactness

(c) Estimate S_x and Z_x For the Built-up section

EXAMPLE 9.9.6

Given the welded I-shaped section of Fig. 9.9.6 used as a 45-ft simply supported beam laterally supported at the one-third points. Determine the service live load the beam may be permitted to carry if the dead load is 0.15 kip/ft including the beam weight. Use Load and Resistance Factor Design. The steel has $F_y = 65$ ksi.
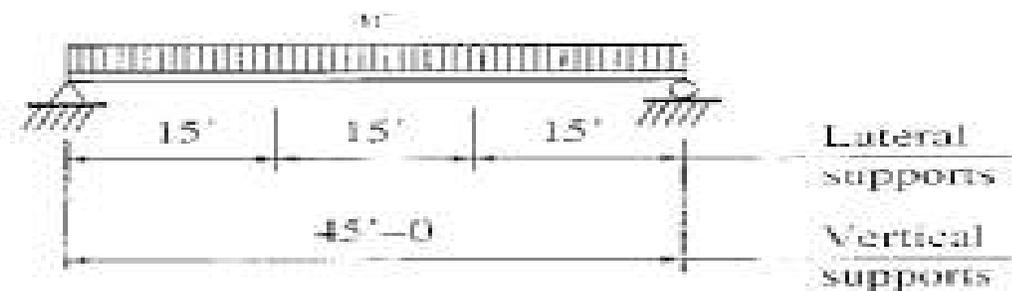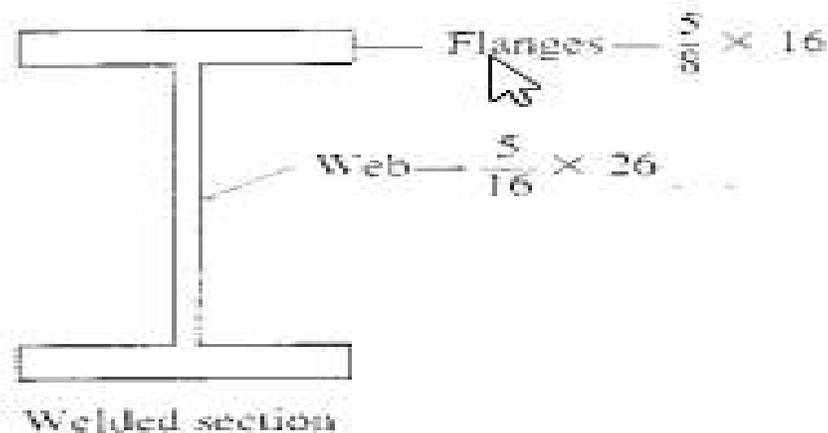

Flanges — $\frac{5}{8} \times 16$

Web — $\frac{5}{16} \times 26$

Welded section

15'   15'   15'

45'–0

Lateral supports

Vertical supports

Figure 9.9.6   Data for Example 9.9.6.

AISC-360-16
Pages 44,45

CHAPTER F

DESIGN OF MEMBERS FOR FLEXURE

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

The chapter is organized as follows:

- F1. General Provisions
- F2. Doubly Symmetric Compact I-Shaped Members and Channels Bent about Their Major Axis
- F3. Doubly Symmetric I-Shaped Members with Compact Webs and Noncompact or Slender Flanges Bent about Their Major Axis
- F4. Other I-Shaped Members with Compact or Noncompact Webs Bent about Their Major Axis
- F5. Doubly Symmetric and Singly Symmetric I-Shaped Members with Slender Webs Bent about Their Major Axis

C: Compact   *NC: non Compact*

TABLE USER NOTE F1.1 Selection Table for the Application of Chapter F Sections				
Section in Chapter F	Cross Section	Flange Slenderness	Web Slenderness	Limit States
F2		C	C	Y, LTB
F3		NC, S	C	LTB, FLB
F4		C, NC, S	C, NC	CFY, LTB, FLB, TFY
F5		C, NC, S	S	CFY, LTB, FLB, TFY

From specification

Cases 11 and 15

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

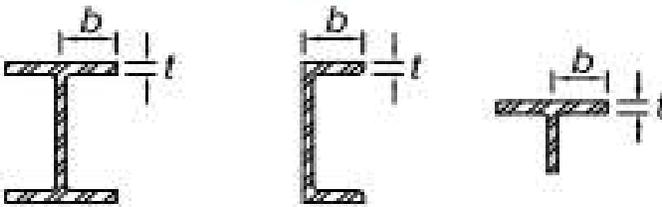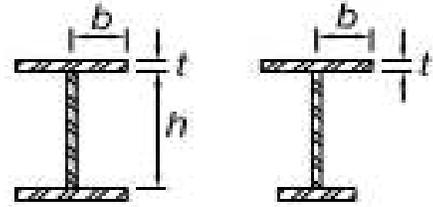| Case	Description of Element	Width-to-Thickness Ratio	Limiting Width-to-Thickness Ratio		Examples
			λ_p (compact/noncompact)	λ_r (noncompact/slender)	
10	Flanges of rolled I-shaped sections, channels, and tees	b/t	$0.38\sqrt{\dfrac{E}{F_y}}$	$1.0\sqrt{\dfrac{E}{F_y}}$	
11	Flanges of doubly and singly symmetric I-shaped built-up sections	b/t	$0.38\sqrt{\dfrac{E}{F_y}}$	[a] [b] $0.95\sqrt{\dfrac{k_c E}{F_L}}$	
12	Legs of single angles	b/t	$0.54\sqrt{\dfrac{E}{F_y}}$	$0.91\sqrt{\dfrac{E}{F_y}}$	
13	Flanges of all I-shaped sections and channels in flexure about the weak axis	b/t	$0.38\sqrt{\dfrac{E}{F_y}}$	$1.0\sqrt{\dfrac{E}{F_y}}$	
14	Stems of tees	d/t	$0.84\sqrt{\dfrac{E}{F_y}}$	$1.03\sqrt{\dfrac{E}{F_y}}$	
15	Webs of doubly-symmetric I-shaped sections and channels	h/t_w	$3.76\sqrt{\dfrac{E}{F_y}}$	$5.70\sqrt{\dfrac{E}{F_y}}$	
16	Webs of singly-symmetric I-shaped sections	h_c/t_w	$\dfrac{\frac{h_c}{h_p}\sqrt{\frac{E}{F_y}}}{\left(0.54\frac{M_p}{M_y}-0.09\right)^2} \le \lambda_r$ [c]	$5.70\sqrt{\dfrac{E}{F_y}}$	
17	Flanges of rectangular HSS and boxes of uniform thickness	b/t	$1.12\sqrt{\dfrac{E}{F_y}}$	$1.40\sqrt{\dfrac{E}{F_y}}$	
18	Flange cover plates and diaphragm plates between lines of fasteners or welds	b/t	$1.12\sqrt{\dfrac{E}{F_y}}$	$1.40\sqrt{\dfrac{E}{F_y}}$	
19	Webs of rectangular HSS and boxes	h/t	$2.42\sqrt{\dfrac{E}{F_y}}$	$5.70\sqrt{\dfrac{E}{F_y}}$	
20	Round HSS	D/t	$0.07\dfrac{E}{F_y}$	$0.31\dfrac{E}{F_y}$	

Unstiffened Elements (13–16) · *Stiffened Elements (17–20)* · *Stiffened Elements (10–12)*

Built up section

Case 11 → Unstiffened

15 → stiffened

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

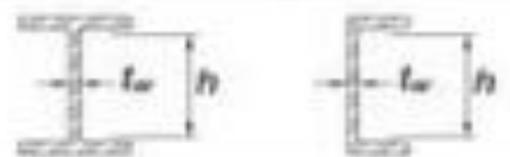| Case	Description of Element	Width-to-Thickness Ratio	Limiting Width-to-Thickness Ratio		Examples
			λ_p (compact/ noncompact)	λ_r (noncompact/ slender)	
10	Flanges of rolled I-shaped sections, channels, and tees	b/t	$0.38\sqrt{\dfrac{E}{F_y}}$	$1.0\sqrt{\dfrac{E}{F_y}}$	
11	Flanges of doubly and singly symmetric I-shaped built-up sections	b/t	$0.38\sqrt{\dfrac{E}{F_y}}$	[a] [b] $0.95\sqrt{\dfrac{k_c E}{F_L}}$	

Case # 11   For built up Sections

Kc

Prepared by Eng. Maged Kamel.

$F_L = 0.70\ F_y$ (Footnote)

Case #15 for Web Local buckling

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

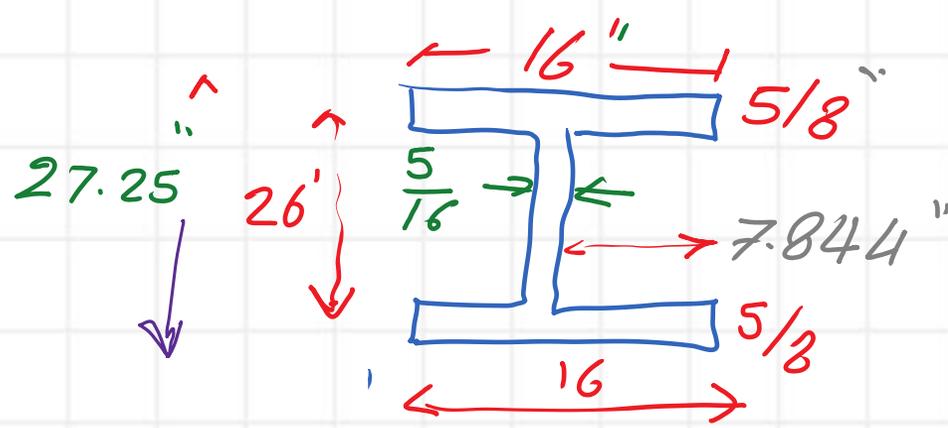| Case	Description of Element	Width-to-Thickness Ratio	Limiting Width-to-Thickness Ratio		Examples
			λ_p (compact/ noncompact)	λ_r (noncompact/ slender)	
15	Webs of doubly symmetric I-shaped sections and channels	h/t_w	$3.76\sqrt{\dfrac{E}{F_y}}$	$5.70\sqrt{\dfrac{E}{F_y}}$	
16	Webs of singly symmetric I-shaped sections	h_c/t_w	$\dfrac{\dfrac{h_c}{h_p}\sqrt{\dfrac{E}{F_y}}}{\left(0.54\dfrac{M_p}{M_y}-0.09\right)^2} \leq \lambda_r$ [c]	$5.70\sqrt{\dfrac{E}{F_y}}$	
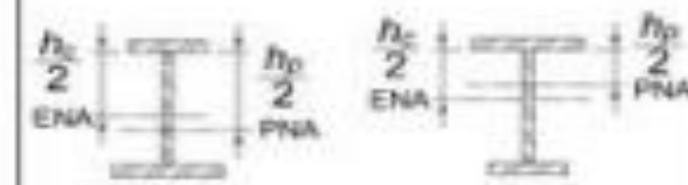
[l] $k_c = 4/\sqrt{h/t_w}$, shall not be taken less than 0.35 nor greater than 0.76 for calculation purposes.

[m] $F_L = 0.7F_y$ for slender web I-shaped members and major-axis bending of compact and noncompact web built-up I-shaped members with $S_{xt}/S_{xc} \geq 0.7$; $F_L = F_y S_{xt}/S_{xc} \geq 0.5F_y$ for major-axis bending of compact and noncompact web built-up I-shaped members with $S_{xt}/S_{xc} < 0.7$, where S_{xc}, S_{xt} = elastic section modulus referred to compression and tension flanges, respectively, in.³ (mm³).

[n] M_y is the moment at yielding of the extreme fiber. $M_p = F_y Z_x$, plastic bending moment, kip-in. (N-mm), where Z_x = plastic section modulus taken about x-axis, in.³ (mm³).

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

F_y = specified minimum yield stress, ksi (MPa)

ENA = elastic neutral axis

PNA = plastic neutral axis

$$Kc = \frac{4}{\sqrt{\frac{hw}{tw}}} = \frac{4}{\sqrt{26/(5/16)}} = 0.4385 \longrightarrow KC$$

Case 7) Unstiffened

$$\lambda r = 0.95 \sqrt{\frac{KcE}{FL}} \qquad Kc = \frac{4}{\sqrt{\frac{h}{tw}}}$$

$$FL = 0.7(Fy) = 0.7(65) = 45.5 \, ksi$$

Dimensions: 16", 5/8", 27.25", 26', 5/16, 7.844", 16, 5/8, 16

Check F.L.B (Flange Local Buckling)

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

$$\lambda_r = 0.95\sqrt{\frac{k_c E}{F_L}}$$

$$k_c = \frac{4}{\sqrt{\frac{h_w}{t_w}}} = \frac{4}{\sqrt{\frac{26}{\frac{5}{16}}}} = 0.4385$$

$$0.35 \le k_c \le 0.76$$

$$F_L = 0.70\, F_y$$

$$F_y = 65\ ksi \longrightarrow F_L = 0.70(65) = 45.50\ ksi$$

$$\lambda_{P_F} = 0.38\sqrt{\frac{29\,000}{65}} = 8.026$$

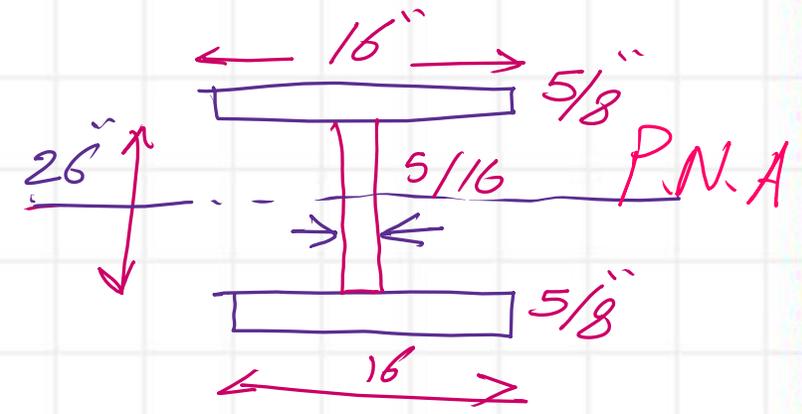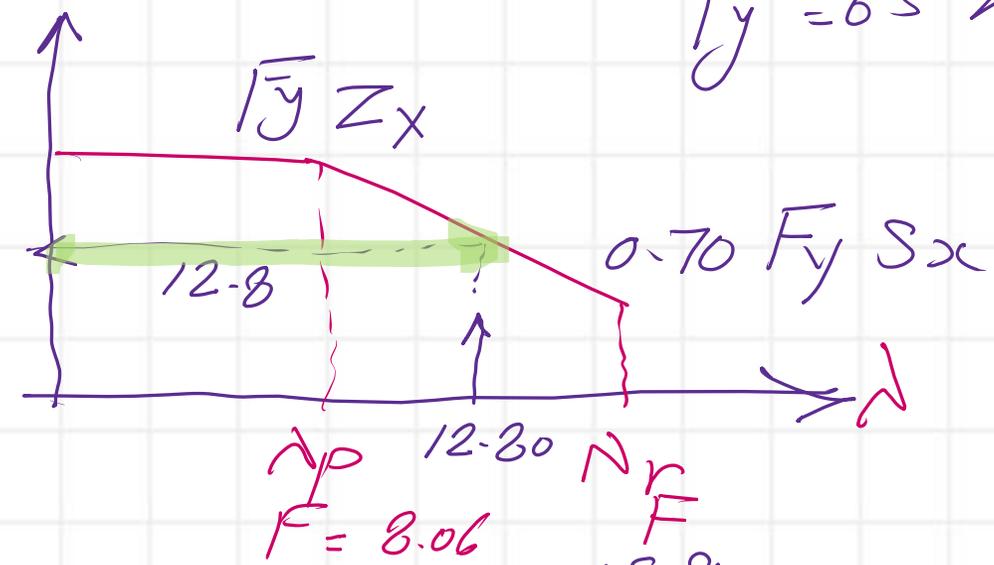$$\lambda_{r_F} = 0.95\sqrt{\frac{0.4385(29000)}{45.5}} = 15.88 \approx 15.90$$

$$\frac{B_F}{2t_F} = \frac{16}{2\left(\frac{1.25}{2}\right)} = 12.80$$

$$\frac{B_F}{2t_F} > \lambda_{P_F}$$
$$< \lambda_{r_F}$$

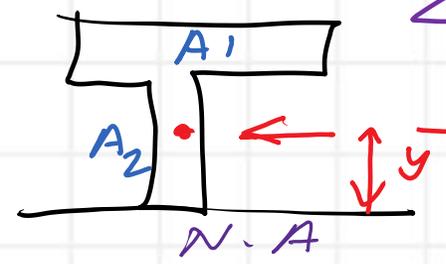$\overline{F_y} = 65 \text{ ksi}$

M_n graph: $\overline{F_y} Z_x$ ... $0.70 F_y S_x$

$12-8$

λ_p ... $12-30$ λ_{rF}

$F = 8.06$

$15-90$

Flange is
Non Compact
$\lambda_F > \lambda_{PF}$
$< \lambda_{rF}$

Top right section:
$16''$ — $5/8$ — P.N.A
$26''$ — $5/16$
$5/8$
16

Requirement

Z_x
S_x

I-section diagram:
16, $5/8$, $5/16$, $26''$, $16''$

$A_1 y_1 \leftarrow C_1$, C $\ominus$
$A_2 y_2 \leftarrow C_2$
$\rightarrow T_1 \rightarrow T$
$\rightarrow T_2$

T-section:
A_1
A_2
$\overline{y}$
N.A

$Z_x = \dfrac{A_T}{2}(2\overline{y}) = A_T \cdot \overline{y}$

$\dfrac{A_T}{2} = 16\left(\dfrac{5}{8}\right) + 13\left(\dfrac{5}{16}\right)$
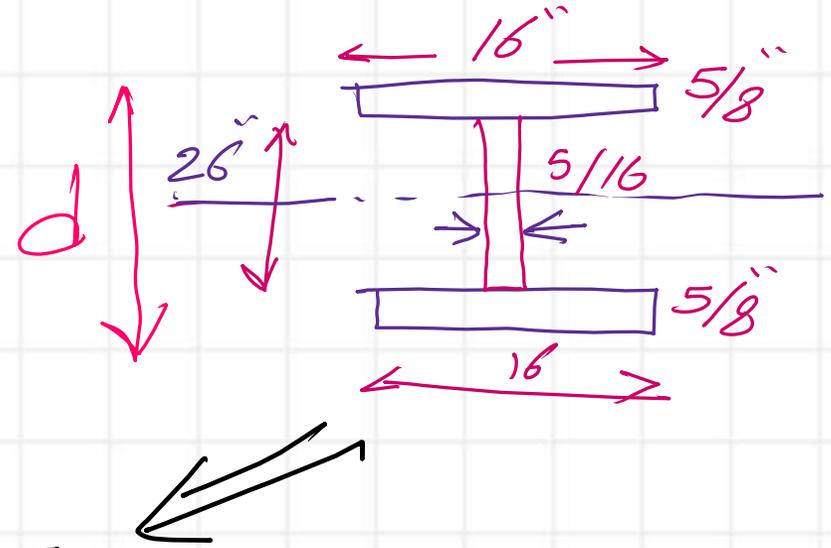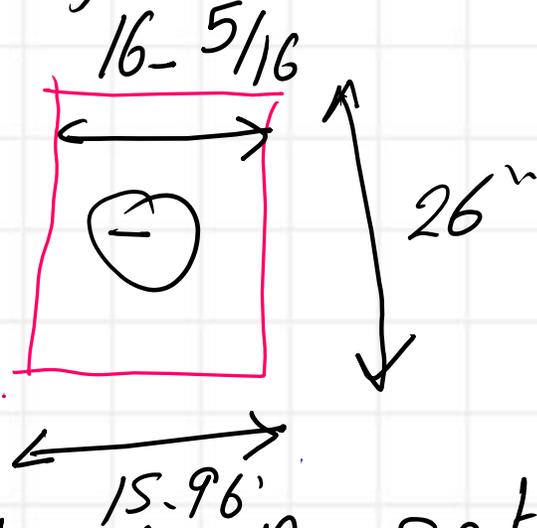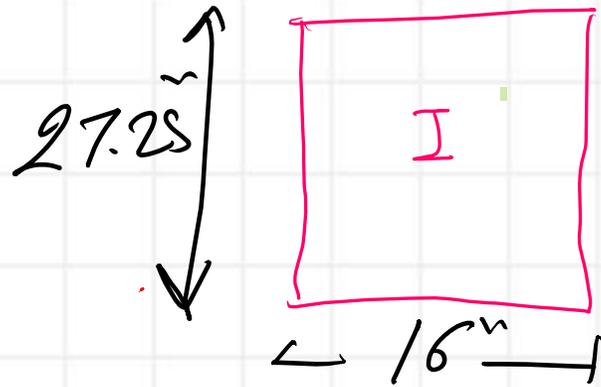$\qquad = 14.0628 \text{ inch}^2$

$y_1 = 13 + \dfrac{5}{16} \Rightarrow A_1 = 10 \text{ inch}^2$

$y_2 = \dfrac{13}{2} \qquad \& \quad A_2 = 4.0625 \text{ inch}^2$

$\overline{y} = \dfrac{A_1 y_1 + A_2 y_2}{A_1 + A_2} = \dfrac{10\left(13\frac{5}{16}\right) + 4.0625)\left(\frac{13}{2}\right)}{14.0625} = 11.344''$

$Z_x = 14.0625 (2)(11.344) = 319.06 \text{ inch}^3$

Estimation of I_x, S_x

$27.25''$

I

$\leftarrow 16'' \rightarrow$

$16-5/16$

$26''$

$15.96''$

$\leftarrow 16'' \rightarrow$ $5/8''$

d $26''$ $5/16$

$5/8''$

$\leftarrow 16 \rightarrow$

$=$

Consider built up Section represented by a rectangle of $16'' \times 27.25''$, we deduct other rectangle of width $= bf - tw$, height $= d - 2tf$
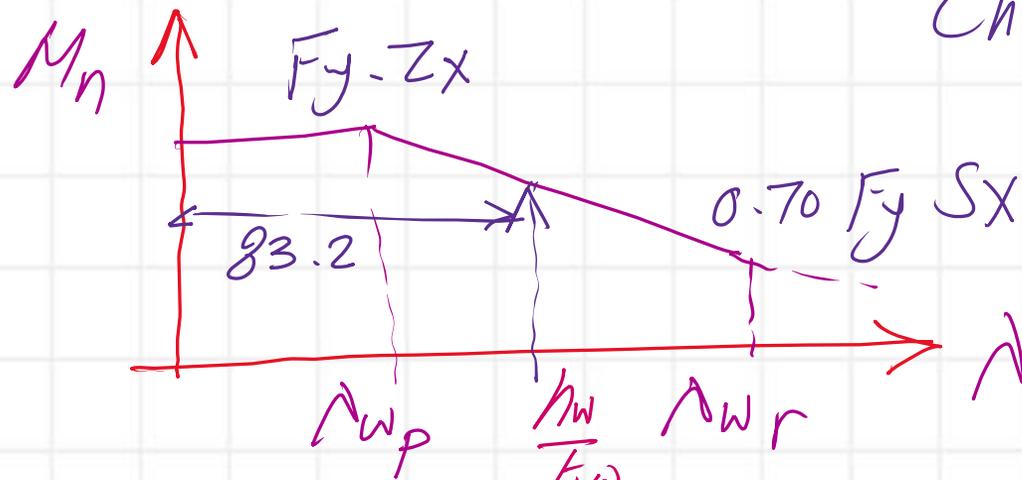
$$I_x = \frac{16\,(27.25)^3}{12} - \frac{(15.96)\,(26)^3}{12} = 4002.813 \text{ inch}^4$$

$$S_x = \frac{I_x}{d/2} = \frac{4002.813}{0.5\,(27.25)} = 293.78 \text{ inch}^3$$
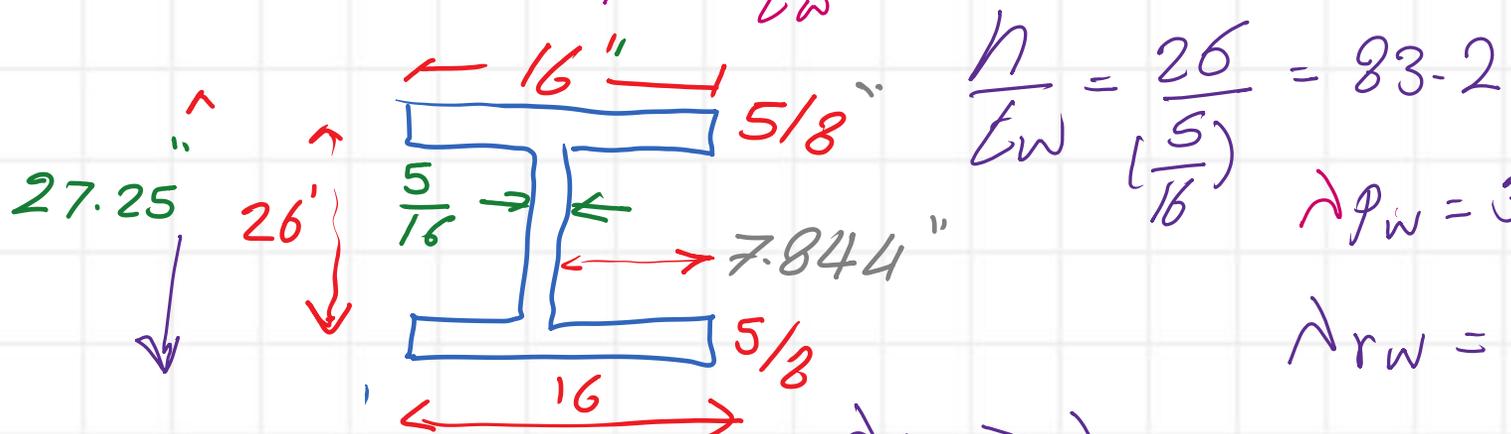
Prepared by Eng.Maged Kamel.

What is the Condition for web ? $\quad F_y = 65\,Ksi,\ E = 29000\,ksi$

Check W.L.B (web local Buckling)

$$\lambda_{Pw} = 3.76\sqrt{\dfrac{E}{F_y}}$$

$$\lambda_w \quad \lambda_{rw} = 5.70\sqrt{\dfrac{E}{F_y}} \quad \Big\} \ Case-15$$

$F_y \cdot Z_x$

$0.70\,F_y\,S_x$

83.2

$\lambda_{wp} \quad \dfrac{h_w}{t_w} \quad \lambda_{wr}$

$$\dfrac{h}{t_w} = \dfrac{26}{\left(\dfrac{5}{16}\right)} = 83.2$$

$$\lambda_{pw} = 3.76\sqrt{\dfrac{29000}{65}} = 79.42$$

$$\lambda_{rw} = 5.70\sqrt{\dfrac{29000}{65}} = 120.397$$

16″

5/8

27.25″   26′ $\dfrac{5}{16}$ 7.844″

5/8

16

$$\lambda_w > \lambda_{pw}$$
$$\quad < \lambda_{rw}$$

Web is Non-Compact