5.5-8 A W18 × 71 is used as a beam with an unbraced length of 9 feet. Use $F_y = 65$ ksi and $C_b = 1$ and compute the nominal flexural strength. Compute everything with the equations in Chapter F of the AISC Specification.

$F_y = 65$ ksi ; $E = 29000$ ksi ;

$r_y = 1.70$ "

Solution

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

$$L_p = 1.76 (1.70) \sqrt{\frac{29000}{65}} = 63.20"$$

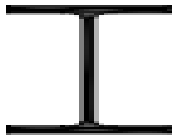
$$\gg / 12 \Rightarrow 5.266'$$

$$\gg 5.30'$$

$$\approx$$

W18x71

Table 1-1 (continued)
W-Shapes
Properties



Nominal Wt.	Compact Section Criteria		Axis X-X				Axis Y-Y				r_{ts}	h_o	Torsional Properties	
	$\frac{b_f}{2t_f}$	$\frac{h}{t_w}$	I	S	r	Z	I	S	r	Z			J	C_w
lb/ft			in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{3}	in.	in.^{3}	in.	in.	in.^{4}	in.^{6}
71	4.71	32.4	1170	127	7.50	146	60.3	15.8	1.70	24.7	2.05	17.7	3.49	4700
65	5.06	35.7	1070	117	7.49	133	54.8	14.4	1.69	22.5	2.03	17.7	2.73	4240
60	5.44	38.7	984	108	7.47	123	50.1	13.3	1.68	20.6	2.02	17.5	2.17	3850
55	5.98	41.1	890	98.3	7.41	112	44.9	11.9	1.67	18.5	2.00	17.5	1.66	3430
50	6.57	45.2	800	88.9	7.38	101	40.1	10.7	1.65	16.6	1.98	17.4	1.24	3040

ry	1.70
rts	2.05
Zx	146.0
Sx	127.0
Cw	4700
c	1
J	3.490
H0	17.700
lb	5.3
lr	16.3

16.33

$\frac{bf}{2tf} = 4.71$

$\frac{h}{tw} = 32.40$

W18 x 71

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

where

$$r_{ts}^2 = \frac{\sqrt{I_y C_w}}{S_x} \quad \text{(F2-7)}$$

$S_x = 127 \text{ inch}^3$

$h_o = 17.7''$

and the coefficient c is determined as follows:

(a) For doubly symmetric I-shapes: $c = 1$ (F2-8a)

(b) For channels: $c = \frac{h_o}{2}\sqrt{\frac{I_y}{C_w}}$ (F2-8b)

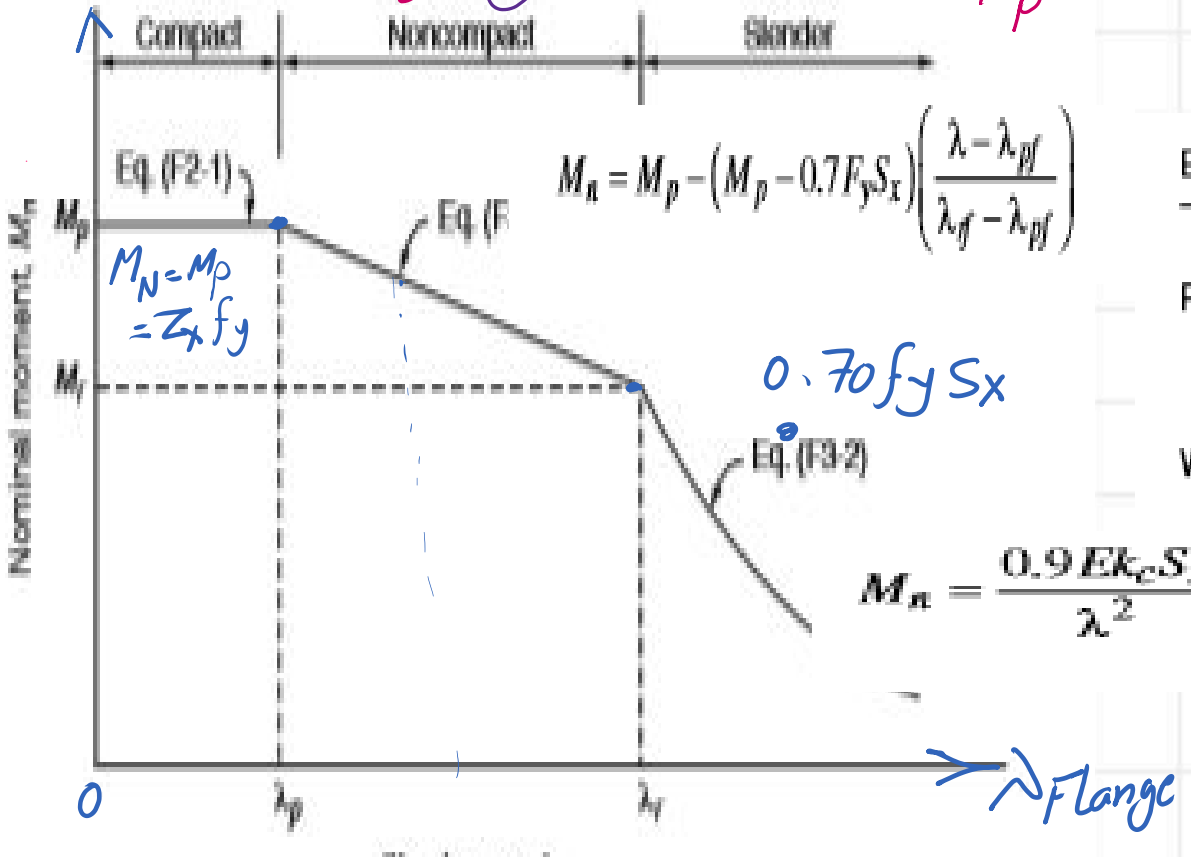
$$L_r = 1.95(2.05)\left(\frac{29000}{0.7(65)}\right)\sqrt{\frac{3.49(1)}{127(17.7)} + \sqrt{\left(\frac{3.49}{(127)(17.7)}\right)^2 + 6.76\left(\frac{0.7(65)}{29000}\right)^2}}$$

$L_r = 196.00'' \Rightarrow 1/12 \Rightarrow 16.33'$

$E = 29000$ ksi & $F_y = 65$ ksi

$\lambda_p = 0.38\sqrt{\frac{29000}{65}} = 8.03$

$\lambda_r = 1\sqrt{\frac{29000}{65}} = 21.12$



$M_N = M_p = Z_x f_y$

$0.70 f_y S_x$

$$M_n = \frac{0.9Ek_cS_x}{\lambda^2}$$

Element	λ	λ_p	λ_r	
Flange	$\frac{b_f}{2t_f}$	$0.38\sqrt{\frac{E}{F_y}}$	$1.0\sqrt{\frac{E}{F_y}}$	item No. 10
Web	$\frac{h}{t_w}$	$3.76\sqrt{\frac{E}{F_y}}$	$5.70\sqrt{\frac{E}{F_y}}$	item No. 15

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

$\lambda_{r_w} = 5.7\sqrt{\frac{29000}{65}} = 120.40$

where

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

h = distance as defined in Section B4.1b, in. (mm)

$\lambda = \frac{b_f}{2t_f}$

b_f = width of the flange, in. (mm)

t_f = thickness of the flange, in. (mm)

$\lambda_{pf} = \lambda_p$ is the limiting slenderness for a compact flange, defined in Table B4.1b

$\lambda_{rf} = \lambda_r$ is the limiting slenderness for a noncompact flange, defined in Table B4.1b

Check Local buckling For W18x71

$F_y = 65$ ksi

From Table 1-1

$\frac{b_f}{2t_f} = 4.71 \rightarrow \lambda_{fp} = 8.03$, $\lambda_{fr} = 21.12$ } $\frac{b_f}{2t_f} < \lambda_{fp}$ Compact Flange

$\frac{h}{t_w} = 32.40 \rightarrow \lambda_{wp} = 79.42$, $\lambda_{wr} = 120.4$ } $\frac{h}{t_w} < \lambda_{wp}$ Web is Compact

W18x46 is a compact section

$M_p = F_y Z_x = 65(146) = 9490$ inch. kips

$0.70 F_y S_x = 0.7(65)(127) = 5778.5$ inch. kips

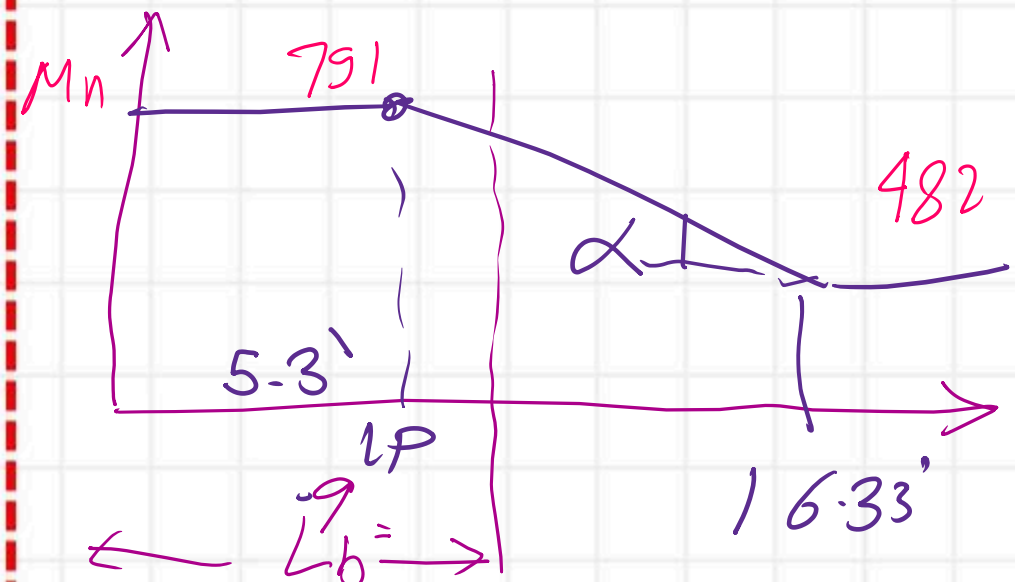
$M_p = 9490/12 = 791$ FT. kips $L_p = 5.3'$

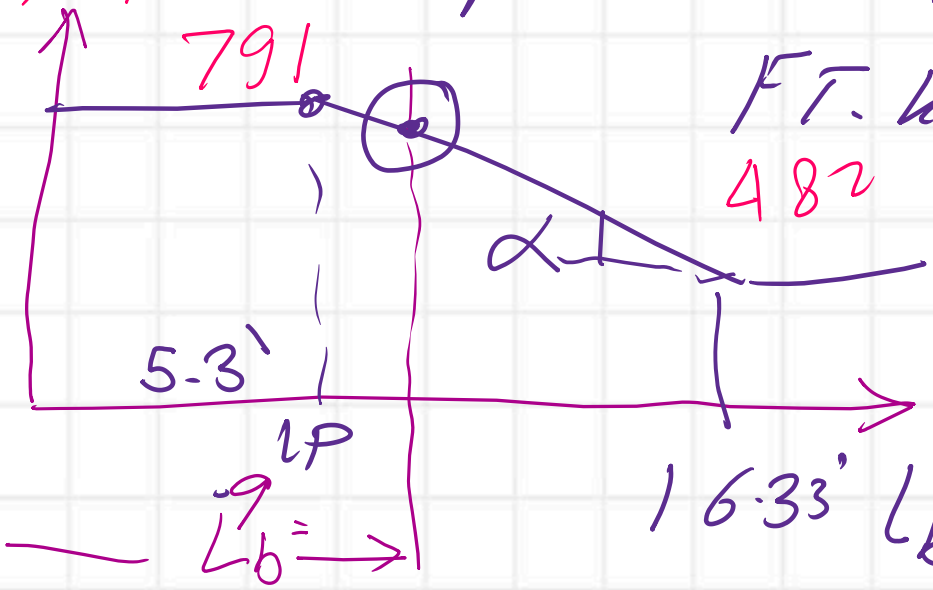
$0.70 F_y S_x = 5778.5/12 = 482$ FT. kips $L_r = 16.33'$

$BF = (791 - 482)/(16.33 - 5.3)$

$BF = 28.01$

$M_n = 791 - BF(L_b - 5.3) =$





W 18 x 71

FT. Kips

482

BF = 28.01

$M_n = 791 - BF(L_b - 5.3) =$

$L_b = 9'$

$L_b \quad L_p$

$M_n = 791 - 28.01 \ (9 - 5.3)$

$= 687.36$ FT. Kips $\Rightarrow$ 687 FT. Kips

W 18 x 71 $F_y = 65$ ksi

length of bracing in Ft	Mn-ft.kips-cb=1
0	791
1	791
2	791
3	791
4	791
5.3	791
6	771
7	743
8	715
9	687
10	659
11	631
14	547
15	519
16.33	482
16.83	462
20	361
25	269
30	214

Lb →