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Subject # Steel Structure

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QUESTION # 01

Given Data:

Lightest W shape of A-36 steel column. of 36 ft long. Load = 60k , live compression load = 110k.

Y axis so that , K_xL = 36 ft

K_yL = 18 feet

Use AISC/LRFD Method.

Solution :-

Required capacity = (1.2 x 60) + (1.6 x 110)

= 248 k.

Enter design strength table of manual with KL = 18 FT and P = 248k.

= Some possible section are :-

W14x53	P = 364	r_x/r_y = 2.44
W12x53	P = 320	r_x/r_y = 2.11
W10x49	P = 301	r_x/r_y = 1.71
W8x58	P = 300k	r_x/r_y = 1.74

Now $\frac{K_x L_x}{K_y L_y} = \frac{36}{18} = 2$

Try W12x53 $r_x/r_y = 2.11$

$$r_x/r_y > \frac{K_x L_x}{K_y L_y}$$

$r_x = 5.23$ $r_y = 2.48$ $A = 15.6 in^2$

$$= \frac{K_x l_x}{r_x} = \frac{36 \times 12}{5.23} = 82.6$$

$$= \frac{K_y l_y}{r_y} = \frac{18 \times 12}{2.48} = 87.09$$

$$= \frac{KL}{r} = 87.09$$

$$= \lambda_c = \frac{KL}{r\pi} \sqrt{F_y / e}$$

$$= \frac{87.09}{\pi} \sqrt{\frac{36}{29000}}$$

$$= 0.97 < 1.5$$

$$F_{cr} = 0.658^{\lambda^2} \times F_y$$

$= 0.658^{(0.97)^2} \times 36$

$= F_{cr} = 24.28$

$= P_n = A_g F_{cR}$

$= 15.6 \times 24.28$

$= P_n = 378.78$ K

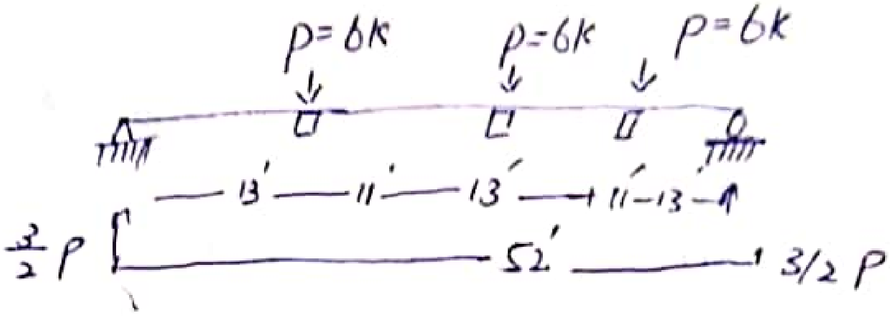
$= \phi P_n = 0.85 \times 378.78$

$= 321.96 > 248$ K

<u>OK</u>

So use W12x53

Q2



Lightest W- Section

Dead Load = 1.5K

Live load = 4.5K

(At each quator point)

-> Total length = 52'

-> Live load deflection = 1/360 of span.

-> F_y = 36 ksi

AISC/ASD Method.

Solution :-

Design load = 4.5 + 1.5 = 6k

$$\Delta = \frac{5}{48} \frac{ML^2}{EI} \rightarrow ①$$

Δ by This equation is multiplied by The factor from table 5.4.

$M = \left(\frac{3}{2} \times 6 \times 26\right) - \left(6 \times 13\right) = 156\ k.ft$

eq ① $\Rightarrow$ $I = \frac{5}{48} \times \frac{ML^2}{E\Delta} \times 0.95$

$I = \frac{5}{48} \frac{(156 \times 12)(52 \times 12)^2}{29{,}900\left(\frac{52}{360} \times 12\right)}$

$I = 1510.51\ in^4$

= Try W24 × 62, $I_x = 1550\ in^4$

$b_f = 7.04\ in$. $d/A_f = 5.72$

= $L_c = \frac{76\ b_f}{\sqrt{F_y}} \Rightarrow \frac{76 \times (7.04)}{\sqrt{36}} = 89'' = 7.41'$

$$L_c = \frac{20{,}000}{f_y \frac{d}{A_f}} \Rightarrow \frac{20{,}000}{36 \times 5.72} = 97.12'' = 8.09$$

$L > L_c$ from Table 5.2

$c_b = 1.13$

$$\sqrt{\frac{102{,}000 c_b}{F_y}} = \sqrt{\frac{102{,}000 \times 1.13}{36}} = 57$$

$$\sqrt{\frac{510{,}000 c_b}{F_y}} = \sqrt{\frac{510{,}000 \times 1.13}{36}} = 127 = 127$$

$$\frac{L}{r_T} = \frac{13 \times 12}{1.71} = 91.22$$

Condition.

$$\sqrt{\frac{102{,}000 c_b}{F_y}} < \frac{L}{r_T} < \sqrt{\frac{510{,}000 c_b}{F_y}}$$

So $F_b = \left[\frac{2}{3} - \frac{F_y (L/r_T)^2}{1530 \times 10^3 \times C_b}\right] F_y$

$= \left[\frac{2}{3} - \frac{36(91.22)^2}{1530 \times 10^3 \times 1.13}\right] 36$

$\Rightarrow F_b = \boxed{17.76 \text{ ksi}}$ allowable

The beam self weight $= 62 \frac{lb}{ft} = 0.062 \, K/ft$

$= M = \frac{wL^2}{8} = \frac{1}{8}(0.062)(52)^2$

$M = 20.95 \text{ K ft}$

Total $M = 156 + 20.95 = \boxed{176.95}$

$f_b = \frac{M}{S_x} \Rightarrow \frac{176.95 \times 12}{131} \times 16.2$ ksi

$= f_b < F_b$

OK

use W24x62

Q3

Given Data

Dead load = 50 K

Live load = 150K

Bolts Dia = 3/4"

Length = 18 FT

Connection Type = Bearing

ASD Method

Required:-

Design A36 steel double angle tension angle

solution:

Total Load = D.L + Live load

= 50 + 150

= 200K or 100K/angle

=> For yielding at the gross area allowable

$0.6F_y = 0.6 \times 36$

$= 22$ ksi

=> For fracture at the net area allowable stresses are

$0.5F_u = 0.5 \times 58$

$= 29$ ksi

=> Since the connection is bolted so $A_g \neq A_n$

Now $A_e = 0.85 A_n$

For yielding

$A_g \times 22 = 100$

$Ae = 3.44\ in^2$

$An = Ae/0.85 \Rightarrow 3.44/0.85$

$An = \boxed{4.04\ in^2}$

So $Ag = An/0.85 \Rightarrow Ag = \frac{4.04}{0.85}$

$\boxed{Ag = 4.76\ in^2}$

= FOR $L4 \times 4 \times \frac{5}{8}$ $Ag = 4.61 = 4.76$ ok

$r_n = 1.20$ $r_y = 1.20$ with $\frac{3}{8}$ in Gusset plate.

$= \frac{L}{r_{min}} = \frac{18 \times 12}{1.20} = 180 \leq 300$ OK

OK

Bolt Design :-

using A325 bolts with threads are included in shear plate

as dia = 3/4"

Area = $\pi/4 (d)^2 \Rightarrow \pi/4 (0.75)^2$

$A = 0.441 \ in^2$

Allowable bolts shear = 21 ksi

Since bolts are in double shear so allowable shear per bolt = $2 \times 21 \times 0.44 = 18.5k$

Allowable bolts per bolts = $1.2 F_u = 1.2 \times 58$
$= 69.6$ ksi

Allowable bearing on two = $\frac{5}{8}$" thick angle

Long legs = $69.6 \times 2 \times \frac{5}{8} \times 0.75 = 65.25 > 18.5$

so shear governs.

Number of bolts = $\frac{200}{18.5} = 10.81$

use 10 bolts.

Design of guest plate

Bearing stress = 1.2 Fu

= $1.2 \times 58 = 69.6$ ksi

So

Allowable bearing = $69.6 \times 10 \times 0.75 \times t = 200$

$t = 0.38$ in

use $\frac{3}{4}''$ G.P

Checking various limit states

$= (0.6) F_y A_g$

$= 0.6 \times 36 \times (8 \times 0.75)$

$= 129.6K < 200K$

= NOT OK

Try $L_7 \times 4 \times \frac{1}{2}$ $A_g = 5.25$

$r_x = 2.25$ $r_y = 1.11$ with $\frac{3}{8}$" GP

$\frac{L}{r_{min}} = \frac{18 \times 12}{1.11}$ $194.59 \leq 300$ K OK

Allowable bearing on two $\frac{1}{2}$" thick angle long legs $= 69.6 \times 2 \times \frac{1}{2} \times 0.75$

$52.2 > 18.5$

so shear govern

checking various limit state

Fracture $= 0.5 \times F_u \times A_e$

$= 0.5 \times 58 \times 0.85 \left[14 - \left(\frac{3}{4}\right) \times 2\right] \times \frac{3}{4}$

$= 231 K > 200 K$

OK