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Subject # Steel Structure

Problem # 01

①

Given Data:

Lightest W-Shape column

A36 steel

$$DL = 60 \text{ k}$$

Pin supported at top and bottom

$$L.L = 110 \text{ k}$$

$$K_x / x = 36 \text{ ft}$$

$$K_y / y = 18 \text{ ft}$$

AISC / LRFD Method.

Solution:

$$\begin{aligned} \text{Required Capacity} &= (1.2 \times 60) + (1.6 \times 110) \\ &= 248 \text{ k} \end{aligned}$$

Enter design strength table of manual with $KL = 18 \text{ ft}$ and $P = 248 \text{ k}$

Some possible sections are

$$W_{14} \times 61 \quad P = 364 \quad r_x / r_y = 2.44$$

$$W_{12} \times 53 \quad P = 320 \quad r_x / r_y = 2.11$$

$$W_{10} \times 49 \quad P = 301 \quad r_x / r_y = 1.71$$

$$W_8 \times 58 \quad P = 300 \text{ k} \quad r_x / r_y = 1.74$$

$$\text{Now } \frac{K_x / x}{K_y / y} = \frac{36}{18} = 2$$

(2)

$$T_{xy} \quad W_{12} \times 53 \quad r_x / r_y = 2.11$$

$$r_x / r_y > \frac{k_x / r}{k_y / y}$$

$$r_x = 5.23, \quad r_y = 2.48, \quad A = 15.6 \text{ in}^2$$

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

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

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

$$\lambda_c = \frac{kL}{r_x} \sqrt{\frac{F_y}{\sigma}}$$

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

$$= 0.97 < 1.5$$

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

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

$$F_{cr} = 24.28$$

$$P_n = A_g F_{cr} = 15.6 \times 24.28$$

$$P_n = 378.78 \text{ k} \quad \phi P_n = 0.85 \times 378.78$$

$$321.96 > 248 \text{ k} \quad \text{OK}$$

So use W₁₂ × 53

✓ " "

Problem #2

(2)

Given Data

Light W-section

$$D.L = 1.5k$$

$$L.L = 4.5k$$

$$\text{Total length} = 52'$$

$$\text{Live Load deflection} = \frac{1}{360} \text{ of span}$$

$$F_y = 36ksi$$

Solution:

$$\text{Design Load} = 4.5 + 1.5$$
$$P = 6k$$

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

Δ by this equation is multiplied by the factor from table 5.4

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

$$M = 156kft$$

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

$$I = \frac{5}{48} \frac{(156 \times 12) (52 \times 12)^2}{29000 \left(\frac{52}{36} \times 12 \right)}$$

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$$I = 1510.51 \text{ in}^4$$

$$\text{Try } W_{24} \times 62, \quad I_x = 1550 \text{ in}^4$$

$$b_f = 7.04 \text{ in}, \quad d/A_f = 5.72$$

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

$$= 89'' = 7.41'$$

$$L_c = \frac{20000}{F_y \frac{d}{A_f}} \Rightarrow \frac{20000}{36 \times 5.72} = 97.12''$$

$$L_c = 8.09'$$

$L > L_c$ from table 5.2, $C_b = 1.13$

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

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

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

Condition

$$\sqrt{\frac{102,000 C_b}{F_y}} \leq \frac{L}{\delta T} \leq \sqrt{\frac{510,000 C_b}{F_y}}$$

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$$\begin{aligned}
 \text{So } F_b &= \left[\frac{2}{3} - \frac{f_y (L/r_T)^2}{1530 \times 10^3 \times c_b} \right] F_y \\
 &= \left[\frac{2}{3} - \frac{36(91.22)}{1530 \times 10^3 \times 1.13} \right] 36
 \end{aligned}$$

$$F_b = 17.76 \text{ ksi; allowable}$$

$$\text{The beam self weight} = 62 \frac{\text{lb}}{\text{ft}} = 0.062 \frac{\text{k}}{\text{ft}}$$

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

$$M = 20.95 \text{ kft}$$

$$\text{Total } M = 156 + 20.95$$

$$M = 176.95$$

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

$$f_b < F_b \quad \text{OK}$$

USE $W_{24} \times 62$

11' ————— 4

Q3:

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Given Data:

$$D.L = 50 \text{ Kips}$$

$$L.L = 150 \text{ Kips}$$

$$\text{Bolt dia} = \frac{3}{4}$$

$$\text{Length} = 18 \text{ ft}$$

Solutions:

$$\text{Total Load} = D.L + L.L$$

$$= 50 + 150$$

$$= 200 \text{ kips or } 100 \text{ kip/angle}$$

For yielding at the gross area allowable stresses are

$$0.6 F_y = 0.6 \times 36$$

$$= 22 \text{ ksi}$$

For fracture at the net area allowable

$$\text{Stresses are } 0.5 F_u = 0.5 \times 58$$

$$= 29 \text{ ksi}$$

Since the connection is bolted so $A_g \times A_n$

$$\text{Now } A_e = 0.85 A_n$$

For yielding

$$A_g \times 22 = 75$$

$$A_g = \frac{75}{22}$$

$$A_g = 3.41 \text{ in}^2$$

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For Fracture

$$29 \times A_e = 75$$

$$A_e = 2.59 \text{ in}^2$$

$$A_n = \frac{A_e}{0.85} = 3.04 \text{ in}^2$$

Assume 15% reduction in gross area for holes

$$\text{So } A_g = \frac{A_n}{0.85} = 3.58 \text{ in}^2$$

For $5'' \times 3\frac{1}{2}'' \times \frac{7}{16}''$

$$A = 3.53 \text{ in}^2 \approx 3.58 \text{ in}^2 \text{ ok}$$

$$x = 1.59 \text{ in} \text{ and with } \frac{3}{8} \text{ in GP}$$

$$y = 1.47 \text{ in}$$

$$\frac{L}{s_{\min}} = \frac{18 \times 12}{1.47} = 146 \leq 300 \text{ ok}$$

Design of Bolts: Using A325 bolts with threads included in shear plane

$$A = 0.44 \text{ in}^2 \text{ (dia} = \frac{3}{4} \text{'')}$$

Allowable bolts are shear = 21 ksi

(Table 2.11 Gray Load)

Since the bolts are in double

$$\text{Allowable Shear so Shear per bolts} = 2 \times 21 * 0.44 = 18.5 \text{ kips}$$

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Allowable bearing on Two

$\frac{7}{16}$ " thick angle

$$\text{Long} = 69.6 \times 2 \times \frac{7}{16} \times 0.75$$

$$= 45.68 \text{ kips} > 18.5 \text{ kips}$$

Allowable bolts bearing stress

$$= 1.2 F_u = 1.2 \times 58$$

$$= 69.6 \text{ ksi}$$

Now No of bolts = :

$$\frac{200}{18.5} = 10.81$$

or 10 Bolts

Design of Gusset plates

$$\text{Bearing stress} = 1.2 F_u = 69.6 \text{ ksi}$$

$$\text{So Allowable bearing} = 69.6 \times 10 \times 0.75 \times t$$

$$= 200$$

$$t = 0.38 \text{ in}$$

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use $\frac{3}{4}$ " Cap

Checking various Limit States

$$\begin{aligned}\text{Yielding} &= 0.6 f_y A_g \\ &= 0.6 \times 36 \times (10 \times 0.75) \\ &= 162 \text{ kip} > 150 \text{ kips}\end{aligned}$$

$$\text{Fracture} = 0.5 \times F_u \times A_e$$

$$\Rightarrow 0.5 \times 58 \times 0.85 \left(10 - \left(\frac{3}{4}\right) \times 2\right) \times \frac{3}{4}$$

$$\Rightarrow 157 \text{ kips} > 150 \text{ k}$$

Check for tearing failure

$$L_c = \frac{2P}{F_{ut}}$$

$$1.25 = \frac{2P}{58 \times 0.38}$$

$$P = 16.53 \text{ kips}$$

(10)

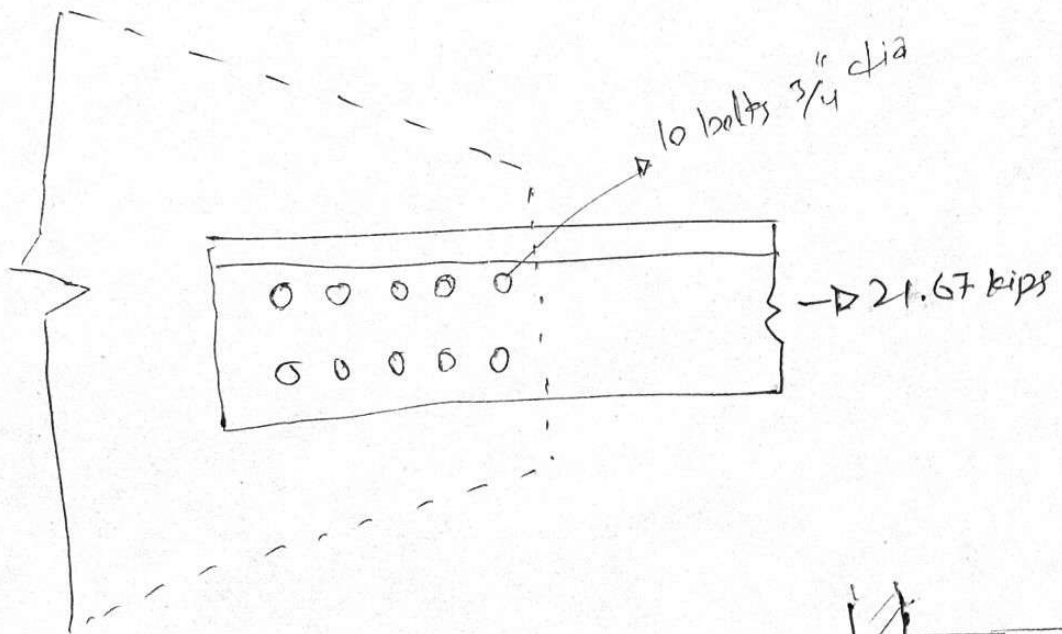
$$L = \frac{2P}{F_{ut}} + \frac{d_n}{2}$$

$$2 = \frac{2P}{58 \times 0.38} + \frac{3/4}{2} \Rightarrow 2 = \frac{2P}{22.04} + 0.37$$

$$2 \times 22.04 = 2P + 0.37$$

$$\frac{44.08}{2} = \frac{2P + 0.37}{2}$$

$$P \equiv 21.67 \text{ kips}$$



$$t = 0.38$$

use two 3/4"

