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Section = B

Subject = Steel Structure

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Date = 29.9.2020

Problem#1

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Eightest W-Shape column
A36 steel
DL 60k   L.L = 110k
Pin Supported at top and bottom
$K_xL_x = 36ft$ $K_yL_y = 18ft$
AISC/LRFD Method

Sol:-

Required capacity $= (1.2 \times 60) + (1.6 \times 110)$
$= 248k$

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

Some possible Sections are.

W14x61	$P = 364$	$r_x/r_y = 2.44$
W10x49	$P = 301$	$r_x/r_y = 1.71$
W8x58	$P = 300k$	$r_x/r_y = 1.74$

Now

$$\frac{K_xL_x}{K_yL_y} = \frac{36}{18} = 2$$

Try W12x53 $r_x/r_y = 2.11$

$$r_x/r_y > \frac{K_xL_x}{K_yL_y}$$

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

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

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

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

$$\lambda_c = \frac{kl}{r\pi}\sqrt{\frac{F_y}{E}}$$

$$= \frac{87.09}{\pi}\sqrt{\frac{36}{29{,}000}}$$

$$= 0.97 < 1.5$$

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

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

$$F_{cr} = 24.28$$

$$P_n = A_g F_{cr}$$

$$= 15.6 \times 24.28$$

$$P_n = 378.78k$$

$$\phi P_n = 0.85 \times 378.78$$

$$= 321.96 > 248k$$

Ok

So

Use W12x53

———

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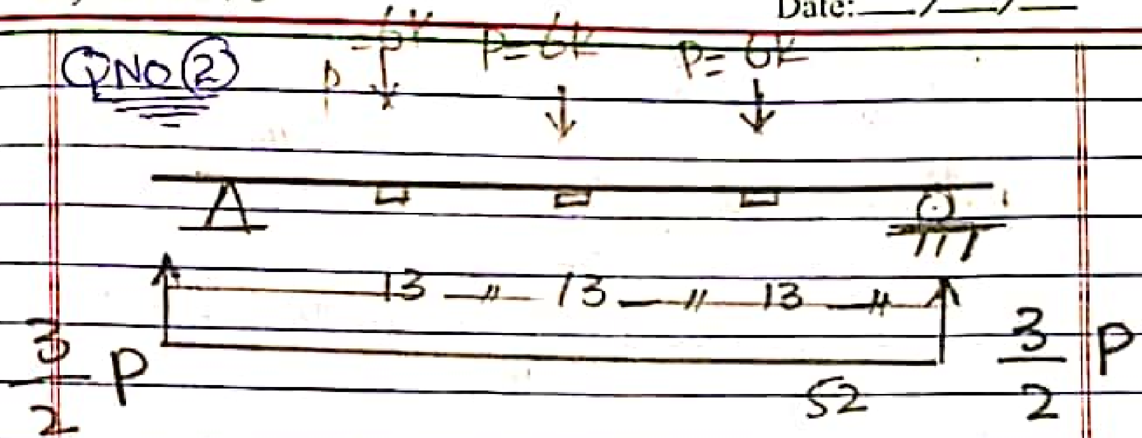
QNO 2



Lightest w- Section

D.L = 1.5k L.L = 4.5k

(At each quater point)

→ Total length = 52'

→ Live load deflection = $\frac{1}{360}$ of span

Δlim

→ F_y = 36ksi

AISC / ASD method

Sol:-

Design load = 4.5 + 1.5 6k

P = 6k

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

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

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

eq① → $I = \frac{5}{48} \times \frac{ML^2}{E\Delta} \times 0.95$

$$I = \frac{5}{48} \frac{(156 \times 12)(52 \times 12)^2}{29{,}000\left(\frac{52}{360} \times 12\right)}$$

$I = 1510.51 \text{ in}^4 \times 0.95$

$I = 1434.98 \text{ in}^4$

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Try W24x62 , $I_x = 1550 in^4$

$b_f = 7.04 in, d/A_f = 5.72$

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

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

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

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

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

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

Condition $\sqrt{\frac{102{,}000 C_b}{F_y}} \leq \frac{L}{r_T} \leq \sqrt{\frac{510{,}000 C_b}{F_y}}$

So

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

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

$F_b = 17.76$ ksi allowable

The beam Self weight $= 62 \frac{lb}{ft} = 0.062 k/ft$

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

$M = 20.95$ kft

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Total $M = 156 + 20.95$

$M = 176.95$

$S_x = 131$

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

$f_b < F_b$

OK

Use W24x62

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Question # 03

Given

D.L = 50 K
L.L = 150 K

Bolts Dia = 3/4
lenght = 18 ft
Connection type = Bearing
ASD method

Required:

Design A36 Steel double angle tension member

Sol:

Total Load = D.L + L.L
= 50 + 150
= 200K or
100 K/Angle

→ For yielding at a grass area allowable Stresses are

$0.6 F_y = 0.6 \times 36$
22 ksi

→ For Fracture at the net area allowable stress are

$$0.5 Fu = 0.5 \times 58$$
$$= 29 \text{ Ksi}$$

→ Since the connection is loaded so

$$A_g \neq A_n$$

Now $A_e = 0.85 A_n$

For yielding

$$A_g \times 22 = 100$$
$$A_g = 100/22$$
$$A_g = 4.54 \text{ in}^2$$

For Fracture

$$29 \times A_e = 100$$
$$A_e = 3.44 \text{ in}^2$$

$$A_n = A_e/0.85 \Rightarrow \boxed{A_n = 4.04 \text{ in}^2}$$

⇒ Assume 15% deduction in grass area for holes

So,

$$A_g = \frac{A_n}{0.85} \Rightarrow A_g = \frac{4.04}{0.85}$$

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

For $L4 \times 4 \times 5/8$ $A_g = 4.61 \approx 4.76$ ok

$r_x = 1.20$ $r_y = 1.20$ with $3/8$ in Gusset plate

$$L/r_{min} = \frac{18 \times 12}{1.20} = 180 < 300\ \text{ok}$$

OK

Bolts Design

Using A325 bolts with threads included in shear plane as dia $= 3/4$

$$A_{req} = \frac{\pi}{4}(d)^2 = \frac{\pi}{4}(0.75)^2$$

$$\boxed{A = 0.441\ in^2}$$

$\Rightarrow$ Allowable bolt shear $= 21$ ksi

Since bolts are in

double shear fo allowable shear per bolt $= 2 \times 21 \times 0.44 = 18.5k$

Allowable bolt bearing stress $= 1.2F_u$
$= 1.2 \times 58$
$= 69.6\,ksi$

Allowable bearing on two $5/8$ = thick angle
long legs $= 69.6 \times 2 \times 5/8 \times 0.75$
$= 65.25 > 18.5$

So shear govans

No. of bolts $= 200/18.5 = 10.81$

use 10 bolts

Design of gusset plate

Bearing Stress $= 1.2F_y$
$1.2 \times 58 = 69.6\,ksi$

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

$t = 0.38$ in

use $3/4$'' G.P

Checking various limit states

yielding = $0.6 F_y A_g$

$= (0.6) F_y A_g$

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

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

$= 29.6K < 200K$

Not ok

Try $L\ 7 \times 4 \times \frac{1}{2}$ $A_g = 5.25$

$r_x = 2.25$ $r_y = 1.11$ with $\frac{3}{8}$" G.P

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

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 governs

Checking various limit states

yielding $= 0.6 F_y A_g$

$= 0.6 \times 36 \times (14 \times 0.75)$

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$= 226.8 > 200k$ Ok

$= 2\ 314 > 200\ k$
Ok

=> Check for tearing failure

$$L_e = \frac{2P}{F_{ct}}$$

$$1.25 = \frac{2P}{58 \times 0.5}$$

$$(1.25)(58 \times 0.5) = 2P$$

$$P = 18.125\ k$$

$$L = \frac{2P}{F_{ut}} + \frac{d_0}{2}$$

$$2 = \frac{2P}{58 \times 0.5} + \frac{3/4}{2}$$

$$2 \times (58 \times 0.5) = 2P + 0.375$$

$$116.1 - 0.375 = 2P$$

$$115.72 = 2P$$

$$P = 57.86k$$

Capacity since 10 bolt & five per bolts per row

Checked By.................. Parents.................. Excellent ☐ Good ☐ BABAR PAPER PRODUCTS

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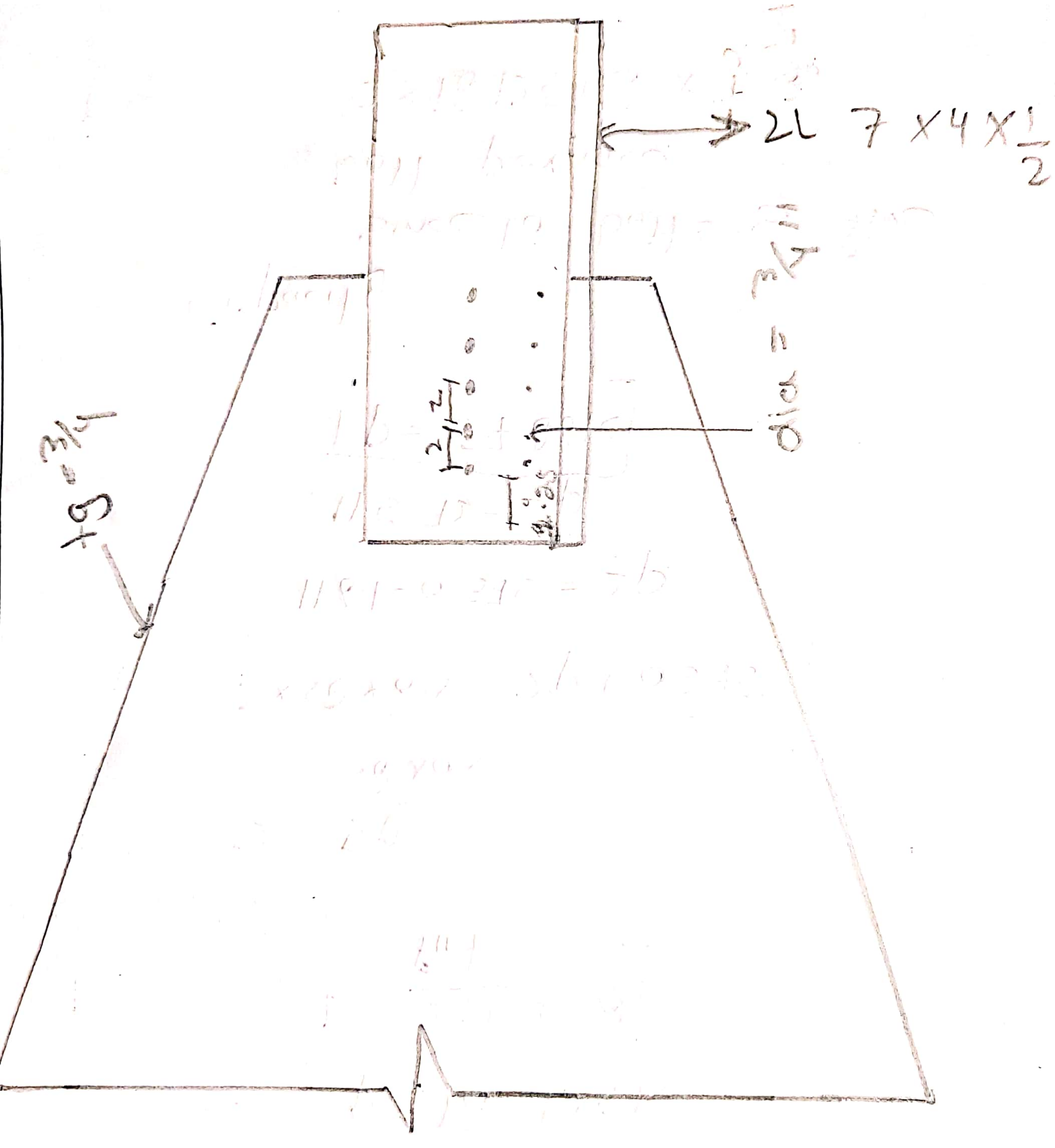
$2 \times 18.125 \neq 8157.86$

$499.13k > 200 \Omega$

OK



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