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Subject STEEL Structure

Submitted to: Engr AMJAD ISLam

Section : "A"

Q#01

Lightest W-Shape Column

A-36 Steel

D.L = 60K   L.L = 110K

Pin Supported at top & 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 & P = 248 K

Some Possible Sections are;

$W_{14\times61}$	P = 364	$r_x/r_y = 2.44$
$W_{12}\times53$	P = 320	$r_x/r_y = 2.11$
$W_{10}\times49$	P = 301	$r_x/r_y = 1.71$
$W_8\times58$	P = 300K	$r_x/r_y = 1.74$

Now $\frac{K_x l_x}{K_y l_y} = \frac{36}{18} = 2$

Try $W_{12}\times 53$ $\qquad r_x / r_y = 2.11$

$r_x / r_y > \frac{K_x l_x}{K_y l_y}$

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

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

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

$KL/r = 87.09$

$\lambda c = \frac{KL}{r\pi}\sqrt{\frac{F_y}{E}}$

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

$= 0.97 < 1.5$

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

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

$F_{cr} = 24.28$

$P_n = A_g F_{cr}$

$= 15.6 \times 24.28$

$P_n = 378.78K$

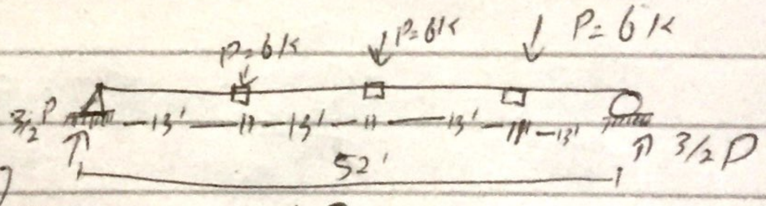
$\phi P_n = 0.85 \times 378.78$

$= 321.96 > 248K$

OK

So $\boxed{\text{Use } W_{12} \times 53}$

Q#2



→ lightest W-Section

→ D.L = 1.5K L.L = 4.5K
(At each quarter point)

→ Total length = 52'

→ Live load deflection = 1/360 of span
Δ_{lim}

→ $F_y = 36$ ksi

By AISC/ASD method

Sol:- Design Load $= 4.5 + 1.5 = 6K$

$P = 6K$

$$\Delta = \frac{5}{48} \frac{ML^2}{EI} \quad \text{--- (1)}$$

Δ by this equation is multiplied by the factor ~~[illegible]~~ from table 5.4

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

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

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

$$I = 1510.51\ in^4$$

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

$$I = 1510.51\ in^4 \times 0.95$$

$$I = 1434.98\ in^4$$

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

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

$$L_c = \frac{76 b_f}{\sqrt{F_y}} \Rightarrow \frac{76 \times (7.04)}{\sqrt{36}} = 89'' \cancel{[illegible]}$$

$$= 7.41'$$

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

$L < L_c$ 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

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

So:

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

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

$F_b = 17.76$ Ksi allowable

The beam self weight

$= \frac{62\,lb}{ft} = 0.062\,K/ft$

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

$M = 20.95$ Kft

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 $\boxed{W_{24} \times 62}$

Q#3

Given:

D.L = 50K
L.L = 150K
Bolt dia = 3/4"
length = 18ft
Connection type = Bearing

"ASD method"

Required:

Design A36 steel double angle tension member.

Sol:

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

→ for yielding at the gross area allowable stresses are;

$0.6\,F_y = 0.6 \times 36$
$= 22$ Ksi

→ for fracture at the net area allowable stresses are

$0.5\,F_u = 0.5 \times 58$
$= 29$ Ksi

Since the Connection is Bolted So,

$A_g \neq A_n$

Now $A_e = 0.85 A_n$

for yielding

$A_g \times 22 = 100$

$A_g = 100/22$

$A_g = 4.54 in^2$

for fracture

$29 \times A_e = 100$

$A_e = 3.44 in^2$

$A_n = A_e/0.85 \Rightarrow 3.44/0.85$

$\Rightarrow A_n = 4.04 in^2$

Assume 15% deduction in gross area for holes

So,

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

$A_g = 4.76 in^2$

For $L_{4\times4\times5/8}$ $A_g = 4.61 \simeq 4.76$ ok

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

$$\frac{L}{r_{min}} = \frac{18\times12}{1.20} = 180 \leq 300\text{ ok}$$

OK

Bolt Design:

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

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

$$A = 0.441\,in^2$$

Allowable Bolts Shear = 21ksi

Since bolts are in double shear so
allowable shear per bolt = $2\times21\times0.44 = 18.5K$
Allowable bolts bearing stress = $1.2F_u = 1.2\times58$
$= 69.6K$

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

long legs = $69.6\times2\times5/8\times0.75 = 65.25 > 18.5$

So shear governs.

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

Use 10 bolts

Design of gusset plate:

So Bearing stress $= 1.2 F_u = 69.6$ ksi

Allowable bearing $=$

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

$t = 0.38$ in

Use $\frac{3}{4}$" GP

Checking various limit states

Yielding $= 0.6 F_y A_g$

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

$= 129.6k < 200k$

Not ok

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

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

yield = $0.6 F_y A_g$

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

$= 226.8 > 200k$ ok

Fracture = $0.5 \times F_u \times A_e$

$= 231k > 200^k$ ok

CHECK for failure

$L_e = 2P/F_u t$

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

$P = 18.125^k$

$L = 2P/F_u t + d_h/2$

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

$115.72 = 2P$

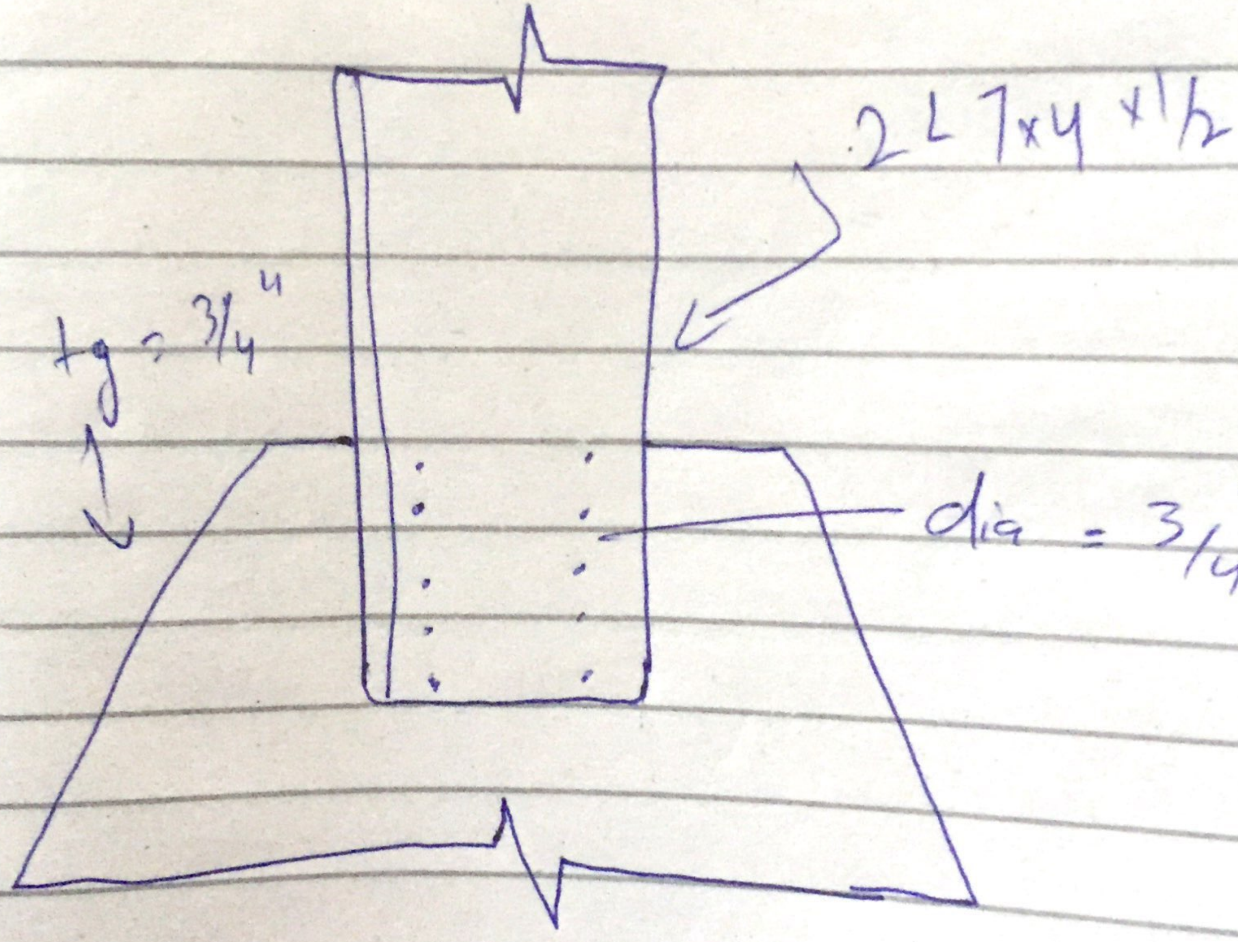
$P = 57.86k$

Capacity: Since 10 bolts & five bolts per row

$2 \times 18.125 + 8 \times 57.86$

$499.13k > 200^k$

ok


2L 7x4 x1/2
tg = 3/4"
dia = 3/4"