Q1) Problem # 1:-

Lightest W-shape column A-36 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 = $(1.2 \times 60) + (1.6 \times 110)$

= 248 k

Enter design strength table of manual with

KL = 18ft and P = 248k

Some possible Sections are

$W_{14 \times 61}$	P = 364	r_x/r_y = 2.44
$W_{12 \times 53}$	P = 320	r_x/r_y = 2.11
$W_{10 \times 49}$	P = 301	r_x/r_y = 1.71
$W_{8 \times 58}$	P = 300	r_x/r_y = 1.74

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

Try $W_{12\times53}$ $r_x/r_y = 2.11$

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

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

$\frac{K_x l_x}{r_x} = \frac{36\times12}{5.23} = 82.6$

$\frac{K_y l_y}{r_y} = \frac{18\times12}{2.48} = 87.09$

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

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

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

$= 0.97 < 1.5$

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

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

$$F_{cr} = 24.28$$

$$P_n = A_g F_{cr}$$

$$= 15.6 \times 24.28$$

$$P_n = 378.78\,k$$

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

$$= 321.96 > 248\,k$$

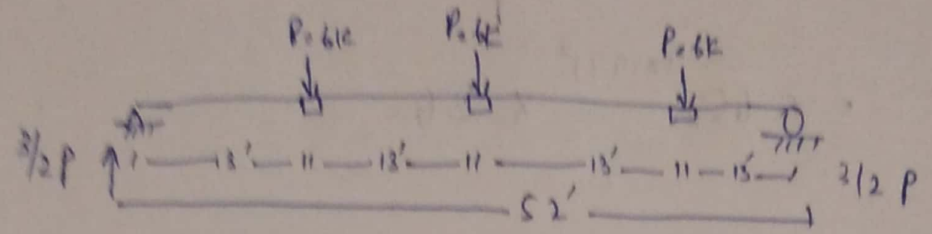
ok!

So

Use $W_{12 \times 53}$

x ——— x ——— x ——— x ——— x ——— x

Q2) Problem #2 :-



lightest w-Section

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

(At each quarter point)

⇒ Total length = 52'

live load deflection $= \frac{1}{360}$ of Span
δ_{lim}

⇒ $f_y = 360$ ksi

AISC/ASD Method.

*) Solution :-

Design load = 4.5 + 1.5 = 6k

P = 6k

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

(∴) Δ by this eq is multiplied by the factor from table 5.4

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

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

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

$I = 1510.51\ in^4$

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

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

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

$L_c > l_c$ from table 5.2

$C_b = 1.13$

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

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

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

Condition

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

So,

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

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

$\Rightarrow$ $F_b = 17.76$ ksi (allowable)

The beam of self weight = 62 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 $M = 156 + 20.95$

$$M = 176.95$$

$S_x = 131$

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

$f_b < F_b$ OK!

Use W24x62

Q3) Problem # 3:-

*) :: Given:-

D.L = 50 k

L.L = 150 k

Bolts dia = 3/4"

length = 18 ft

connection type = Bearing ASD Method

*) :: Required:-

Design A-36 Steel double angle tension member

*) :: Solution:-

Total load = D.L + L.L

= 50 + 150

(= 200 k or 100 k / 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 geilding

$$A_g \times 22 = 100$$

$$A_g = \frac{100}{22}$$

$$\left(A_g = 4.54 \text{ in}^2\right)$$

for fracture

$$29 \times A_e = 100$$

$$A_e = 3.44 \text{ in}^2$$

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

$$\left(A_n = 4.04 \text{ in}^2\right)$$

⇒ Assume 15% deduction in gross Area for holes

So,

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

$$\left(A_g = 4.76 \text{ in}^2\right)$$

for $L_4 \times 4 \times 5/8$ $A_g = 4.61 \simeq 4.76$ ok!

$r_x = 1.20$, $r_y = 1.20$ with $\frac{3}{8}$ in Gusset plate

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

ok!

*) :: Bolts design:-

using A_{325} bolts with threads included in shear plate as dia $= 3/4"$

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

$$(A = 0.441\ in^2)$$

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 bolt bearing stress $= 1.2\ F_u = 1.2 \times 58$

$= 69.6$ ksi

Allowable stress on two $5/8"$ 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 gusset plate :-

Bearing Stress $= 1.2 F_u$

$= 1.2 \times 58 = 69.6$ 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

yeilding $= 0.6 F_y A_g$

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

$= 129.6\,k < 200\,k$

Not ok!

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

$r_x = 2.25$, $r_y = 1.11$ with $3/8$" G.P

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

OK!

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

$52.2 > 18.5$

So shear governs.

Checking various units states.

yeilding = $0.6 F_y A_g$

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

$= 226.8 > 200k$

Ok!

Fracture = $0.5 \times F_u \times A_e$

$= 0.5 \times 58 \times 0.85 \left[14 - \left(1\frac{3}{4}\right) \times 2\right] \times \frac{3}{4}$

$= 231\ k > 200k$

ok!

499.13 k > 200 k

ok!