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PAPER :- Steel structure

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Q No 1 :-

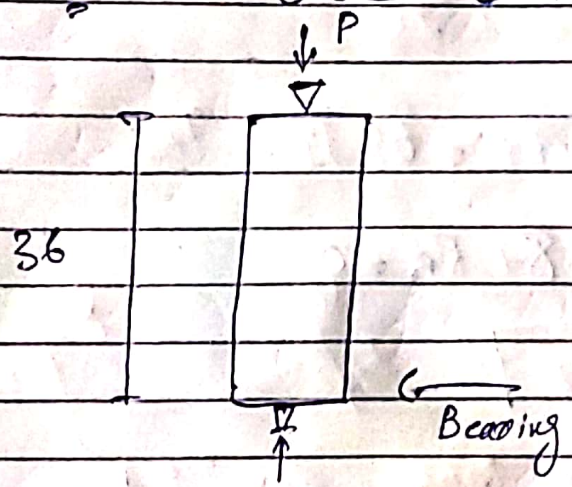
=> Given data.

DL = 60 kip
LL = 110 k

L = 36 ft.

Kx = 1
Ky = 1

Sol Use LRFD.



Assume hing ~~type~~ Top and Bottom x and y- axis bending.

Calculate the load (factor load).

$P_u = 1.2\,DL + 1.6\,L.L$

$\Rightarrow 1.2(60) + 1.6(110)$

$\boxed{= P_u = 248 \text{ kips.}}$

To find out the support reaction.

$(\overset{\curvearrowright}{-ve.}) \overset{\curvearrowleft}{+ve} \quad \Sigma M_B = 0$

$-15(R_A) + 1872(16) = 0$

$\boxed{R_A = 1996.8 \text{ lb}}$

$\downarrow \uparrow + \quad \Sigma F_y = 0$

$-1872 + R_A + R_B = 0$

$-1872 + 1996.8 + R_B = 0$

$\boxed{R_B = +124 \text{ lb}}$

$K_x = 1$

$K_y = 1$

Since The AISC column load Table are computed assuming $(KL/r)_y$ Controls enter these Table with effective length (KL/y)

Thus enter with $P_u = 248$

$W_{12} \times 45$ $\phi_c P_n = 257$ $r_x/r_y = 2.65$.

$W_8 \times 40$ $\phi P_n = 238$ kips $r_x/r_y = 1.73$.

$W_{10} \times 45$ $\phi_c P_n = 267$ kips $r_x/r_y = 2.65$.

Since $(KL)_x = 2(KL)_y$.

if $r_x/r_y = 2(KL)_y$.

if $r_x/r_y \geq 2$ Then weak axis control and tabular loads give the correct answer.

Since r_x/r_y for $W_8 \times 40$ is less Than 2.

Strong axis bending controls. The strength may be obtained from the table by entering with The equivalent $(KL)_y$ that corresponds to $(KL)_y = 36$ ft.

$$\frac{(KL)_x}{r_x} = \frac{(KL)_y}{r_y}.$$

Equivalent $(KL)_y = (KL)_x / r_x/r_y$

$= 36/1.73$

$\Rightarrow 20.80$ ft

with $P_u = 248$ k.

$W_{12} \times 36$ $\phi p_u = 247$ k.

and $r_x/r_y = 1.75$

for $(KL)/y$. 20 - 80 ft The

$W_{12} \times 36$ has a design strength

$\phi_c p_n$ at only 248 kip therefore

it is not accep table for $W_{10} \times 45$

with $A_g = 13.3\ in^2$.

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

$$= \frac{89.6}{\pi} \sqrt{\frac{36}{29,000}}$$

$\Rightarrow \lambda_c = 1.0049 < 1.75$

So for

$$F_{cr} = \left[(0.658)^{\lambda_c^2} \right] F_y$$

$\Rightarrow F_{cr} = 23.59$ ksi

$\phi F_{cr} = 0.85 \times 23.59$

$\phi F_{cr} = 20.1$ ksi

Now $\phi P_n = \phi_c F_{cr} A_g$

$= 20.1 (13.3)$

$= 267k > 248k$

$\phi_c P_n = 267 > 248k \rightarrow$ ok

$\boxed{\text{So Use } W10 \times 45}$

Q2:

GIVEN DATA:

D.L = 1.5 kips
L.L = 4.5 kips
L = 52 ft
$\Delta = \frac{l}{300}$
$F_y = 36$ ksi

Sol:

The moment of inertia required to satisfy the deflection limit can be concluded from eq the value of Δ by this equation should be multiplied by 0.95 in case of concentrated load Table (5-4)

Concentrated load of = 1.5 + 4.5 = 6 kips

$M = 1\frac{1}{2} \times 6 \times 30 - 6 \times 15$

$M = 234$ ft-kips

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

$$= 0.95 \times \frac{5}{48} \frac{234 \times 12 (52 \times 12)^2}{29{,}000 \left(60 \times \frac{12}{300}\right)}$$

$$\Rightarrow I = \frac{519349788}{3340800}$$

$$\boxed{I = 1555 \text{ in}^4}$$

from manuals moment of inertia selection table we find the ligtest section to be the W24x62 for which $I = 1550 \text{ in}^4$

The factored design load at each of the load points is $1.2(1.4 + 0.062 \times 15) + 1.6 \times 5.4$

$= 11.4$ kips so

$$M = 1\tfrac{1}{2} \times 11.4 \times 26 - 11.4 \times 13$$

$$M = 296.4 - \text{kips}$$

$$L_P = \frac{300\, r_y}{\sqrt{f_y}} = \frac{300 \times 1.38}{\sqrt{36}}$$

$L_P = 69$ in

Since this is less then 15ft spacing of the lateral support Equ (5-26) applies. the following properties of the W24x62 are form the manual load-factor design selection table

$L_P = 5.8$ ft $\quad L_r = 17.2$ ft

$\phi M_P = 413$ ft-kips $\quad \phi M_r = 225.5$ ft kips

Since $\phi = 0.90$

$$M_p = \frac{413}{0.9}$$

$$M_p = 459 \text{ ft-kips}$$

$$M_r = \frac{225}{0.9}$$

$$M_r = 2.83 \text{ ft kips}$$

$C_b = 1.11$ then from equ (5-26)

$$M_n = 1.11\left(459 - 176 \frac{15 - 5.8}{17.2 - 5.8}\right)$$

$$M_n = \cancel{30} \; 352 \text{ ft-kips} < M_p = 459$$

design streaght is $\phi M_n = 0.9 \times 352 = 316 \text{ ft-kips}$

Since this is less than the required value of 296-kips.ft the w24.62 is adequation if the deflection chu is needed

The manual contains design-moment charts based on $C_b = 1$ which may be used if C_b is greater than unity the charts must be entered with a moment equal to required value divided by C_b

if $C_b = 1.11$, ϕ

$$\phi M_n = \frac{296.4}{1.11} = 267.03 \text{ ft-kips}$$

Ep an un braced lenght = 13 ft
we find the lightest section
to be the w24×62

used $\boxed{w24 \times 62}$

with Braced lenght $\boxed{= 13\,ft}$

Q3

Given data:

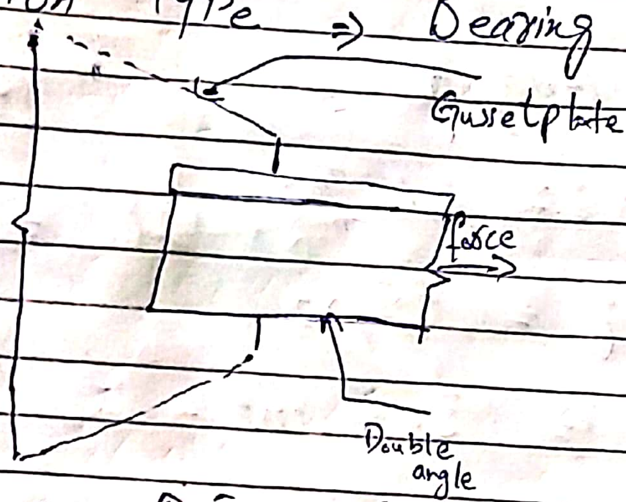
D.L = 50 K.

L.L = 150

B.dia = 3/4"

length = 18 Ft.

Connection type ⇒ Bearing Type.



Solution ⇒ Total load = D.L + Live load

⇒ 50 + 150

⇒ 200 kips. or

100 kip / Angle

For yielding at the gross area allowable stresses are 0.6 Fu = 0.6 × 36.

= 22 ksi

For Fracture at the net are allowable stresses are

$0.5 F_u = 0.5 \times 58$...

$\Rightarrow 29$ ksi

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

Now $A_e = 0.85 A_n$.

For yeilding

$A_g \times 22 = 75$

$A_g = 75/22$

$= 3.4 \text{ in}^2$

For Fracture

$29 \times A_e = 75$.

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

Assume 15% reduction in gross area for holes

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

Scanned with CamScanner

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

$A = 3.53 in \simeq 3.58 in^2$ ok

$r_x = 1.59 in$ & with $3/8 in$ G.P

$r_y = 1.47 in$

$$\frac{L}{r_{min}} = \frac{18 \times 12}{1.47} = \boxed{146.93 \leq 300 \quad \text{ok}}$$

Design of Bolts:-

Using A325 bolts with threads included in shear plane

$A = 0.44 in^2$ (dia $= 3/4''$).

Allowable bolts are shear $= 21 ksi$ (Table 2.11 Gay load)

Since the bolts are in double shear so

Allowable shear per bolts

$= 2 \times 21 \times 0.44 = 18.5$ Kips.

Allowable bearing on Two 7/16" thick angle long ~~angle~~ leg = 69.6 x 2 x 7/16 x 0.75

= 45.68 kips > 18.5 kips

Allowable bolts bearing stress = 1.2 Fu = 1.2 x x58 => 69.6 ksi

Now No of bolts =

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

$\boxed{\text{or } 10 \text{ Bolts}}$

=> Design of Gusset plates

Bearing Stress = 1.2 Fu = 69.6 ksi

So

Allowable bearing = 69.6 x 10 x 0.75 x t = 200

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

Use 3/4" G.P.

⇒ Checking various limit States

Yeilding = 0.6 Fy Ag.

$= 0.6 \times 36 \times (10 \times 0.75)$

⇒ 162 kip > 150 kips.

Fracture = $0.5 \times Fu \times Ae$

$\Rightarrow 0.5 \times 58 \times 0.85 [10 - (3/4) \times 2] \times 3/4$

⇒ 157 kips > 150 k.

Check for Tearing failure

$Lc = \frac{2P}{Fut}$ ⇒

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

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

$$L = \frac{2P}{F_u t} + \frac{d_n}{2}$$

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

$$2 = \frac{2P}{22.04} + 3/4 / 2$$

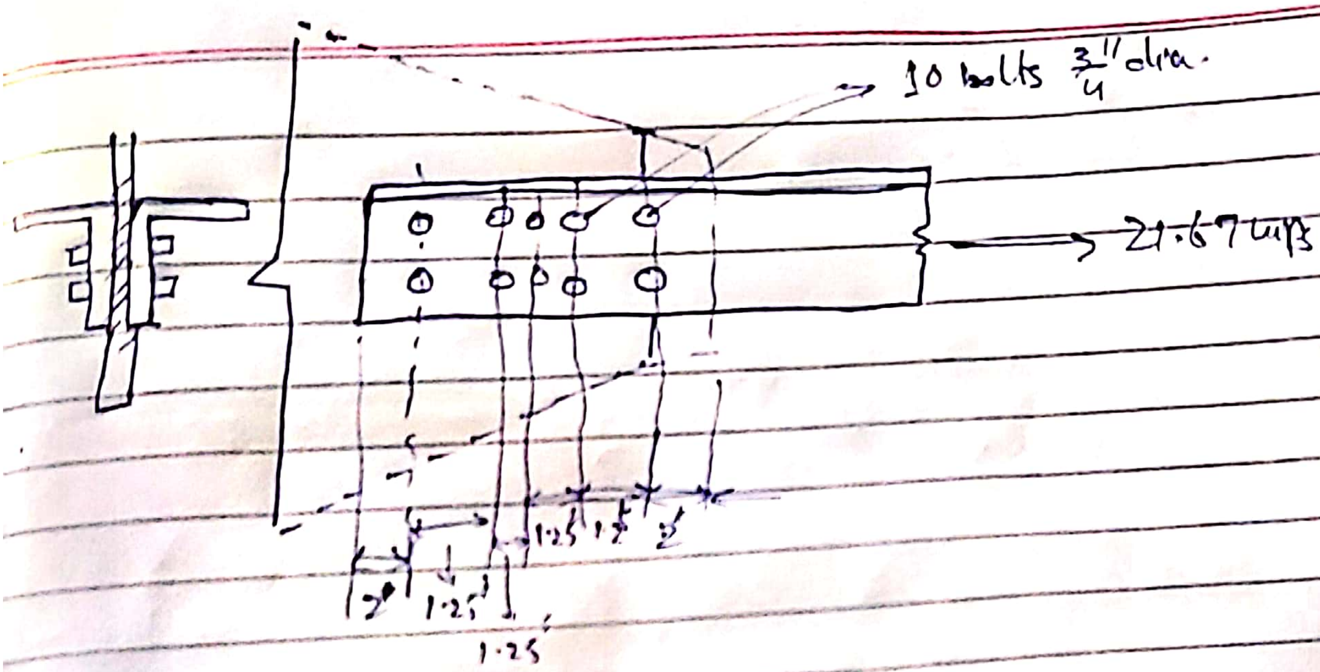
$$2 = \frac{2P}{22.04} + 0.37$$

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

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

$$22.04 = P + 0.37$$

$$\boxed{P = 21.67 \text{ KIP}}$$



$t = 0.38$

use two $\frac{3}{4}$ Cn.P.