Sec A Date: / / 20 ID # 7549 Sumer 2020

Q no 1

Eightest W-Shape Calum
A36 Steel
DL = 60K L.L = 110K
Pin Supported at top and bottem
$K_x l_x = 36ft$ $K_y l_y = 18ft$
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 $P = 248k$

Same possible Section are

$W_{14} \times 16$ $P = 364$ $r_x/r_y = 2.44$

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

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

$W_8 \times 58$ $P = 300k$ $r_x/r_y = 1.74$

Now

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

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Try $W12 \times 53$ $r_x / r_y = 2.11$

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

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

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

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

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

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

$$= \frac{87.09}{\pi} \sqrt{\frac{36}{29.000}}$$

$$= 0.097 < 1.5$$

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

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

$$F_{cr} = 24.28$$

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$P_n = A_g F_{cr}$

$= 15.6 \times 24.28$

$P_n = 378.78\,k$

$\phi P_n = 0.85 \times 378.78$

$= 321.96 > 248\,k$ OK

So

Use $W_{12} \times 53$

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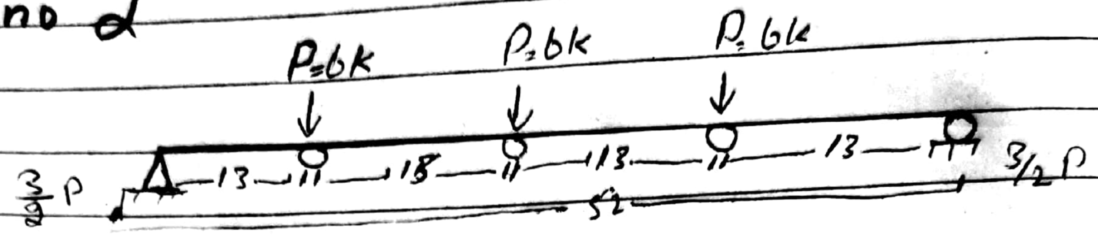
Q no 2



Lightest W - Section

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

(At each quater point)

→ Total length = 52'

→ Live load deflection = 1/360 of Span

Δ_{lim}

→ Fy = 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 ~~method~~ multiplied by the factor from table 5.4

$M = (3/2 \times 6 \times 26) - (6 \times 13) = 156$ k.ft

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

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$$I = \frac{5}{48} \cdot \frac{(158 \times 12)(52 \times 12)^2}{29{,}000\left(\frac{52}{360} \times 12\right)}$$

$$I = 1516.51 \ in^4$$

Try $W24 \times 62$ $\qquad I_x = 1550 \ in^4$

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

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

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

$L > L_c$ $\qquad$ from table 5.2

$C_b = 1.13$

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

$$\sqrt{\frac{510{,}000 C_b}{F_y}} = \sqrt{\frac{510{,}000 \times 1.13}{3.6}} = 127$$

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

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Condition

$$\sqrt{\frac{162{,}000\,Cb}{Fy}} \le \frac{L}{r_T} \le \sqrt{\frac{510{,}000\,Cb}{Fy}}$$

So

$$F_b = \left[\frac{2}{3} - \frac{F_y\,(L/r_T)^2}{1530\times10^3\times Cb}\right] F_y$$

$$= \left[\frac{2}{3} - \frac{36\,(91.21)^2}{1530\times10^3\times1.13}\right] 36$$

$F_b = 17.76$ ksi allowable

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

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

$M = 20.95\ k\,ft$

Total $M = 156 + 20.95$

$M = 176.95$

$$f_b = \frac{M}{S_x} \Rightarrow \frac{176.95\times12}{131} = 16.2\ \text{ksi}$$

$f_b < F_b$ OK

Use $W_{24\times62}$

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Q no 3

Given data:-

D.L = 50k
L.L = 150k
Bolts Dia = 3/4"
Length = 18 ft
Connection type = Bearing
ASD method

Required

Design A36 Steel double angle tension member

Sol Total load = D.L + L.L
= 50 + 150
= 200 or 100k/Angle

→ For yielding at the gross area allowed stress are

0.6fy = 0.6Fy x 36
= 22 ksi

→ For fracture at the net area allowed stress are

0.5 = 0.5 x 58
= 29 ksi

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→ Since the Connection is bolted so
$A_g \neq A_n$

Now $A_e = 0.85 A_n$

For yielding

$A_g \times 22 = 100$

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

$A_g = 4.54 \, in^2$

For fracture

$29 \times A_e = 100$

$A_e = 3.44 \, in^2$

$A_n = A_e/0.85 \Rightarrow 3.44/0.85 \quad A_n = 4.04 \, in^2$

Assume 15% deduction in gross area for holes

So

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

$A_g = 4.76 \, in^2$

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For $L4 \times 4 \times \frac{5}{8}$ $A_g = 4.61 < 4.76$ ok

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

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

Bolts Design

Using A325 bolts with threads included in Area plan as dia 3/4"

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

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

Allowable bolt shear $\Rightarrow$ 21 ksi

Since bolt are double shear so Allowable shear per bolt $= 2 \times 21 \times 0.44 = 18.5k$ Allowable bolt bearing stress $= 1.2 F_u = 1.2 \times 58 = 69.6k$

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

legs legs $= 69.6 \times 2 \times \frac{5}{8} \times 0.75 = 65.25 > 18.5$

So shear govern

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No 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 $= 69.6 \times 10 \times 0.75 \times = 200$

$t = 0.38$ in
use $= \frac{3}{4}$ B.P.

Checking various limit states
yield $= 0.6\,F_y A_g$
$= 0.6\,F_y A_g$
$= 0.6 \times 36 \times (8 \times 0.75)$
$= 129.6k < 200k$ (Not ok)

Try $L_7 \times 4 \times \frac{1}{2}$ $A_g = 5.25$

$r_x = 2.25$ $r_y = 1.11$ with $3/8''$ B.P

$\frac{L}{r_{min}} = \frac{18 \times 12}{1.11} = 194.59 < 300$ (ok)

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Allowable bearing on two $\frac{1}{2}$ thick Angles long legs

$= 69.6 \times 2 \times 1/2 \times 0.75$

$52.2 > 18.5$

So shear governs

Check various limit state

Yielding $= 0.6\, F_y\, A_g$

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

$226.8 > 200$ ok

Fracture $= 0.5 \times F_u \times A_e$

$= 0.5 \times 58 \times 0.85 \left[14 - \left(\frac{13}{4}\right) \times 2\right] \times \frac{3}{4}$

$231\,k > 200\,k$ [ok]

Check for bearing also

$l_e = \frac{2P}{F_u t}$

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

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

$P = 18.125\,k$

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

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

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

$$116.1 - 0.375 = 2P$$

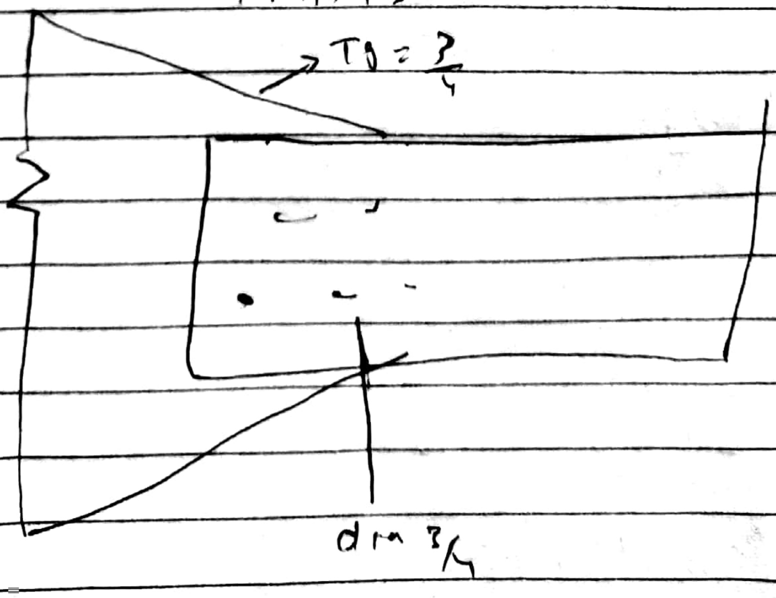
$$115.72 = 2P$$

$$P = 57.86 k$$

Capacity since 10 bolt & Gone bolts per row

$$2 \times 18.121 + 8 \times 57.86$$

$$499.13 k > 200 \text{ (OK)}$$



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