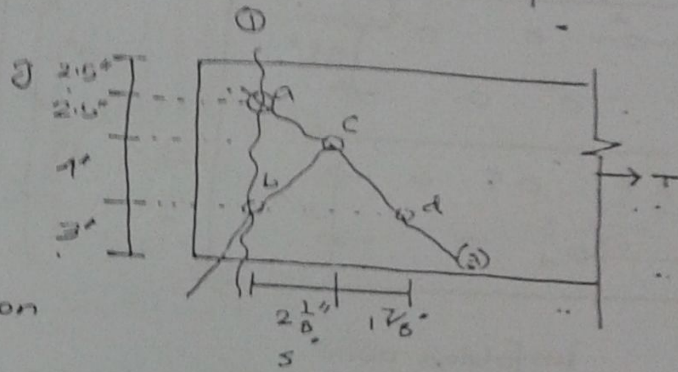
Given, $t = \frac{1}{4}$" bolt dia $= \frac{15}{16}$" Assuming A36 steel calculate critical net area and allowable strength. Assume. $A_e = 90\%$ of A_n.

Assume A36 steel...;

So, $F_y = 36$ Ksi, $F_u = 58$ Ksi

hole dia, $D = \left(\frac{15}{16} + \frac{1}{8}\right)'' = \frac{17}{16}$"



1. Nominal tensile strength based on yielding of gross are

$$R_n = T_n = F_y A_g = 36 * 12'' * \frac{1}{4}'' = 108K$$

So, Allowable strength $= \frac{R_n}{1.67} = \boxed{64.67K}$

2. Nominal tensile strength by effective area:

$$R_n = F_u A_e. = F_u * 0.9 A_n.$$

path ①: ab $\quad A_n = 12 \times \frac{1}{4} - 2 \times \frac{17}{16\times4} = \cancel{0.875 in^2} = 2.4688 in^2$

path ⑪: acb $\quad A_n = 12 * \frac{1}{4} - 3 \times \frac{17}{16\times4} + \left(\frac{2\frac{1}{8}^2}{4\times2.5} + \frac{2\frac{1}{8}^2}{4\times4}\right)\frac{1}{4} = 2.248 in^2$

path ⑪: acd $\quad A_n = 12 \times \frac{1}{4} - 3 \times \frac{17}{16} \times \frac{1}{4} + \left(\frac{2\frac{1}{8}^2}{4\times2.5} + \frac{(1\frac{7}{8})^2}{4\times4}\right)\frac{1}{4} = 2.28 in$

So, $A_n = 2.248 in^2$

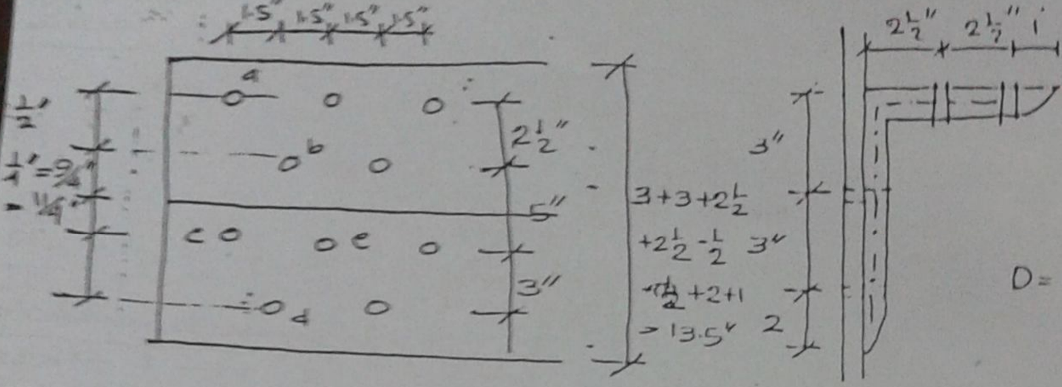
$\therefore R_n = (58 * 0.9 \times 2.248) = 117.3456k$

$\therefore$ allowable strength $= \frac{R_n}{2} = \boxed{58.6728K}$ governs.

Ans 58.6728k

Ham3 09/4/16

alculate :- the ultimate strength



L 8×6×½

$D = \frac{15}{16} + \frac{1}{8} = \frac{17}{8\times2}$

unfolded cond.

ere, unfolded length = $L + b - \frac{t_1}{2} - \frac{t_2}{2} = 8 + 6 - \frac{1/2}{2} - \frac{1/2}{2} = 13\frac{1}{2}''$

$A_g = 13\frac{1}{2} \times \frac{1}{2} = 6.75\ in^2$

Strength based on gross area

$R_n = F_y A_g = 36 \times 6.75\ in^2 = 243k$

allowable strength $= \frac{243}{1.67} = \boxed{145.5k}$

Strength based on fracture

alculation of A_n

Path (i) ac $\quad A_{n1} = 6.75 - 2 \times \frac{17}{8\times 2} \times \frac{1}{2} = 5.6875\ in^2$

Path (ii) abc $\quad A_{n2} = 6.75 - 3 \times \frac{17}{16} \times \frac{1}{2} + \left(\frac{(2\frac{1}{2})^2}{4\times 2\frac{1}{2}} + \frac{(1.5)^2}{4\times 5''}\right) \times \frac{1}{2} = 5.335\ in^2$

Path (iii) abd $\quad A_{n3} = 6.75 - 3\,\frac{17}{16} \times \frac{1}{2} + \frac{1.5^2}{4\times 2.5} \times \frac{1}{2} = 5.268\ in^2$

Path (iv) abcd

$A_{n4} = 6.75 - 4 \times \frac{17}{16} \times \frac{1}{2} + \left(\frac{1.5^2}{4\times 2.5} + \frac{1.5^2}{4\times 5} + \frac{1.5^2}{4\times 3}\right) \times \frac{1}{2} = 4.89\ in^2$

So, $\boxed{A_n = 4.89\ in^2}$

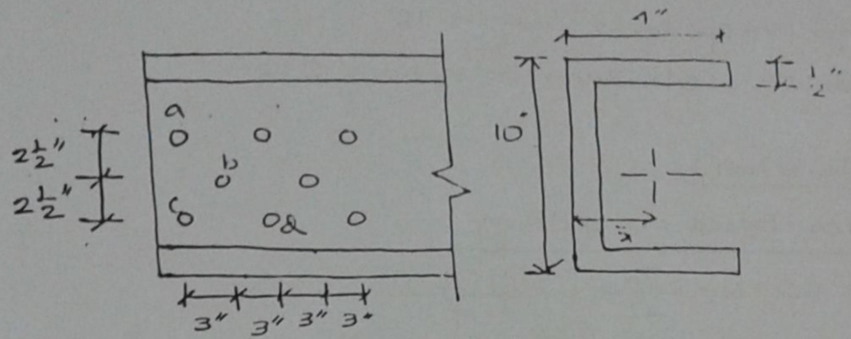
Given, $U = 1.0$

So, $R_n = F_u A_e = F_u U A_n = 58 \times 1 \times 4.89 = 283.62k$

So, allowable strength $= \boxed{141.81k}$ Jowons

Ans.

Calculate the ultimate strength



$D = \frac{15}{16} + \frac{1}{16} + \frac{1}{16}$

$= \frac{17}{16}''$

Give A 572 steel, $F_y = 50$ Ksi and $F_u = 65$ Ksi

– Allowable strength based on yielding of gross area

$A_g = 10 \times \frac{1}{2} + 3\frac{1}{2} \times \frac{1}{2} \times 2 = 8.5 \text{ in}^2$

So, allowable strength $= \frac{F_y A_g}{1.67} = \frac{8.5 \times 50}{1.67} = 254.5K$

– strength based on fracture

calculation of An

path ① ac : $A_{n1} = 8.5 - 2Dt = 8.5 - 2 \times \frac{17}{16} \times \frac{1}{2} = 7.4375 \text{ in}^2$

path ⑪ abc : $A_{n2} = 8.5 - 3 \times \frac{17}{16} \times \frac{1}{2} + \left(\frac{3^2}{4 \times 2.5} + \frac{3^2}{4 \times 2.5}\right)\frac{1}{2} = 7.50625 \text{ in}^2$

path ⑪ abd : $A_{n3} = 7.80625 \text{ in}^2$

So, $A_n = 7.4375 \text{ in}^2$

Calculation of u

$U = 1 - \frac{\bar{x}}{L}$. Here, $\bar{x} = \frac{9 \times 0.5 \times 0.25 + 2 \times 4 \times 0.5 \times 2}{9 \times 0.5 + 2 \times 4 \times 0.5} = 1.07$

So, $u = 1 - \frac{1.071}{12} \leq 0.9$

$= 0.91 \leq 0.9 \simeq 0.9$

allowable strength

$R_a = \frac{R_n}{2} = \frac{65 \times 0.9 \times 7.4375}{2} = \boxed{217.55K}$ Ans

* Design problem

DL = 60K, LL = 6K

Design a member for roof truss Length 12'

steel: A572 ; Grade 50 , slenderness ratio ≤ 240

Soln

Step: 01 (Area calculation)

- Required gross area based on yielding

$$\text{Load} = 60+6 = \frac{T_n}{\Omega} = \frac{F_y A_g}{1.67}$$

$$\text{So, } A_g = \frac{66 \times 1.67}{50} = 2.21 \text{ in}^2$$

- Required gross area based on fracture

Assume min^m 3 hole/gage line, $U = 0.85$

$$\text{So, } 66 = \frac{F_u U A_n}{2} \quad \text{or, } A_n = \frac{2 \times 66}{65 \times 0.85} = 2.39 \text{ in}^2$$

$$\text{So, } A_g = A_n + Dt = 2.39 + \left(\frac{7}{8}+\frac{1}{8}\right) \times t$$

$$\text{Select } A_g = A_n + Dt = 2.39 + \left(\frac{7}{8}+\frac{1}{8}\right)t = 2.39 + t$$

Table

Thickness t	Hole area Dt	Req. Gross area	Member size	A_g	r_{min}	L/r
5/16	0.313	2.7	L5x5x5/16	3.07	1.56	92.3
5/16	0.375	2.77	L5x3x3/8	2.86	1.23	117.07
			L4x4x3/8	2.86	1.23	117.07
7/16	0.438	2.83	L $3\frac{1}{2}$ x $3\frac{1}{2}$ x $\frac{7}{16}$	2.89	1.06	135.85

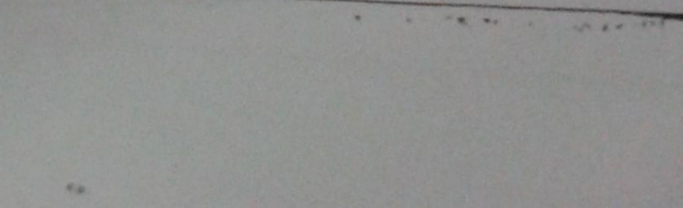
Selected member $\boxed{L\,4 \times 4 \times 3/8}$

check for block shear

$A_{gv} = 10 \times 3/8 = 3.75$

$A_{nv} = (10 - 2.5D) \times 3/8 = 2.8125$

$A_{nt} = (2 - 0.5D) \times 3/8 = 0.5625$



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Design a member for roof-truss Length 12'

Steel: A572 ; Grade 50 , slenderness ratio ≤ 240

Soln

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$$\text{So, } A_g = A_n + Dt = 2.39 + \left(\frac{7}{8} + \frac{1}{8}\right) \times t$$

$$\text{Select } A_g = A_n + Dt = 2.39 + \left(\frac{7}{8} + \frac{1}{8}\right) t = 2.39 + t$$

Table

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5/16	0.313	2.7	$L5 \times 5 \times 5/16$	3.07	1.56	92.3
6/16	0.375	2.77	$L5 \times 3 \times 3/8$	2.86	1.03	117.07
			$L4 \times 4 \times 3/8$	2.86	1.23	117.07
7/16	0.438	2.83	$L3\frac{1}{2} \times 3\frac{1}{2} \times \frac{7}{16}$	2.89	1.06	135.85

Selected member $\boxed{L4 \times 4 \times 3/8}$

check for block shear

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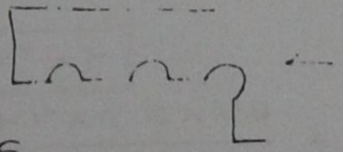

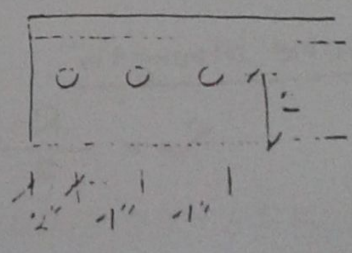

...ign problem

DL = 60K , LL = 6K

Design a member for roof-truss Length 12'
Steel : A572 ; Grade 50 , Slenderness ratio ≤ 240

Soln

Steps 01 (Area calculation)

– Required gross area based on yielding

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– Required gross area based on fracture

Assume min^m 3 hole/gage line, U = 0.85

$$\text{So, } 66 = \frac{F_u U A_n}{2} \quad \text{or, } A_n = \frac{2 \times 66}{65 \times 0.85} = 2.39\,in^2$$

$$\text{So, } A_g = A_n + Dt = 2.39 + \left(\frac{7}{8} + \frac{1}{8}\right) \times t$$

$$\text{Select } A_g = A_n + Dt = 2.39 + \left(\frac{7}{8} + \frac{1}{8}\right)t = 2.39 + t$$

Table

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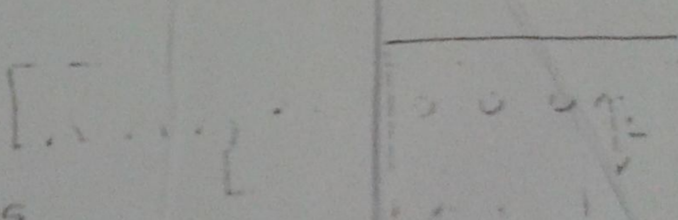
Selected member $\boxed{L\,4 \times 4 \times 3/8}$

check for block shear



$A_{gv} = 10 \times 3/8 = 3.75$

$A_{nv} = (10 - 2.5D) \times 3/8 = 2.8125$

$A_{nt} = (2 - 0.5D) \times 3/8 = 0.5625$

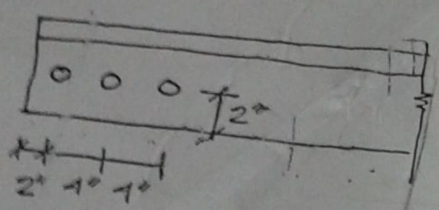
sume, $U_{bs} = 1.0$

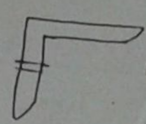
So, $\dfrac{0.6 F_u A_{nv} + U_{bs} F_u A_{nt}}{2} \le \dfrac{0.6 F_y A_{gv} + U_{bs} \times F_u \times A_{nt}}{1.67}$

$= \dfrac{0.6 \times 65 \times 2.8125 + 1.0 \times 65 \times 0.5625}{2.0} \le \dfrac{0.6 \times 50 \times 3.75 + 1.0 \times 65 \times 0.5625}{1.67}$

$= 73.125 \le 89.26 \simeq 73.125 < 66k \Rightarrow ok$

Plate $\Rightarrow$ L $4 \times 4 \times 3/8$





...tion 10-11 7(b)

plate :- 10″ × 3/8″ which is 8′ long

bolts : 3/4″ dia

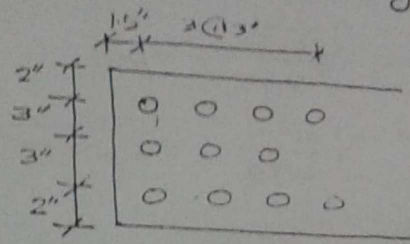
steel : A36, $U = 0.85$ and,

check slenderness ratio and determine tensile capacity

strength based on yielding

$$T = \frac{R_n}{\Omega} = \frac{A_g F_y}{1.67} = \frac{36 \times 10 \times 3/4}{1.67}$$

$$= \boxed{161.66K}$$



D: 3/4 + 1/8 = 7/8

strength based on fracture

$$T = \frac{F_u U A_n}{2}$$

- calculation of An

path ① $A_{n1} = A_g - 3Dt = 10 \times 3/4 - 3 \times 7/8 \times 3/4 = 5.53125\ in^2$

$A_n = 5.53125\ in^2$

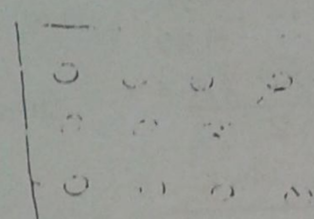
Now, $T = \frac{65 \times 0.85 \times 5.53125}{2}$

strength for block shear

$A_{gv} = 2 * 10.5'' * 3/8'' = 7.875''$

$A_{nv} = 2\left[10.5'' - 3.5 * 7/8\right] * 3/8 = 5.58'$

$A_{nt} = 2\left[3\sqrt{3} - 7/8\right] * 3/8 = 3.24\ in^2$



$$\text{so, } \frac{0.6 F_u A_{nv} + U_{bs} F_u A_{nt}}{2} \le \frac{0.6 F_y A_{gv} + U_{bs} F_u A_{nt}}{1.67}$$

$$= \frac{0.6 \times 58 \times 5.58 + 1 \times 58 \times 3.24}{2} \le \frac{0.6 \times 36 \times 7.875 + 1 \times 58 \times 3.24}{1.67}$$

$= \cancel{164.7} \le$

$\Rightarrow 191.05 \le 214.1$

$\simeq 191.05K$ so, $\boxed{\text{strength} = 153K}$

Check slenderness

$I_y = \frac{10^3 \times 3/8}{12} = 31.25$

$I_x = \frac{(3/8)^3 \times 10}{12} = 0.11$

$A = 10 \times 3/8 = 3.75$

So, $r_x = \sqrt{\frac{0.11}{3.75}} = 0.31$

$r_y = \sqrt{\frac{31.25}{3.75}} = 2.88$

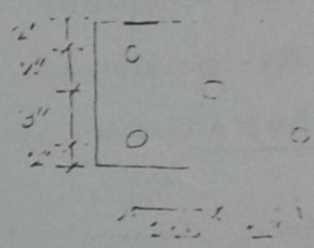
So, $\frac{KL}{r} = \frac{5 \times 12}{0.31} = 116 \le 300 = OK$

Ques 07-08

*(b) Determine: allowable tensile load, A36 steel and 7/8" bolt

Allowable strength based on yielding

$T = \frac{9 \times 0.5 \times 36}{1.67} = 95.3K$



Strength based on fracture

$T = \frac{F_u \times U \times A_n}{2}$

$A_n = 9 \times 0.5 - 2 \times 1 \times 0.5 = 3.5$

$U = 1 - \frac{\bar{x}}{L} = 1 - \frac{0.25}{4.25} = 0.94 = 0.9$

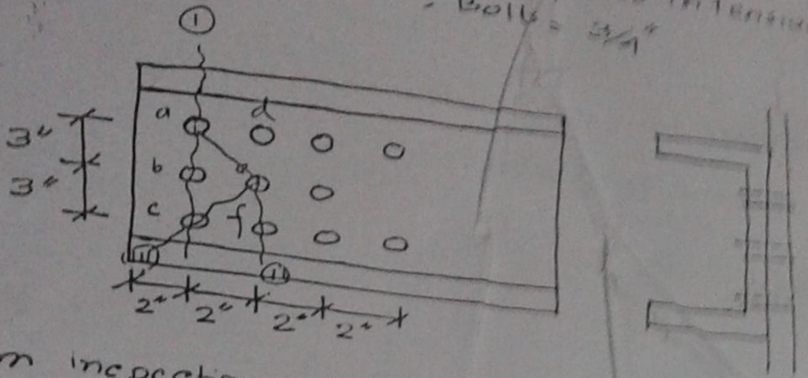
So, $T = \frac{58 \times 0.9 \times 3.5}{2} = 91.35$

Strength based on block shear

$A_{gv} =$

05-06

(a) Calculate the net section effective in tension channel C 12×30, Bolts = 3/4"



From inspection path (1) abc will be ...

So, $A_n = A_g - 3Dt$

For C 12×30 → $A_g = 8.82 \text{ in}^2$

$t = 0.51"$, $D = \frac{3}{4} + \frac{1}{8} = \frac{7}{8}"$

So, $A_n = 8.82 - 3 \times \frac{7}{8} \times 0.51 = \boxed{7.48 \text{ in}^2}$

Ans.

Column

column strength formula:

- For inelastic buckling : (from empirical equ^n obtained from eq^n)

when $\frac{KL}{r} \le 4.71\sqrt{E/F_y}$ (or, when $F_e \ge 0.44F_y$)

then, $F_{cr} = \left[0.658^{F_y/F_e}\right] F_y$

- For elastic buckling :

when, $\frac{KL}{r} > 4.71\sqrt{E/F_y}$ (or, when $F_e < 0.44F_y$)

$\therefore \boxed{F_{cr} = 0.877 F_e}$

- For column design max^m allowable slenderness $\frac{KL}{r} \le 200$

Nominal strength, $P_n = F_{cr} A_g$

Allowable strength $= \frac{P_n}{1.67}$

*: Problems related to find capacity

step : 1 — calculate slenderness ratio $\frac{K_x L_x}{r_x}$ and $\frac{K_y L_y}{r_y}$

step : 2 — Take which ever is greater as $\frac{KL}{r}$

step : 3 — check buckling type (elastic/inelastic)

- calculate → $4.71\sqrt{E/F_y}$
- if $\frac{KL}{r} \le 4.71\sqrt{E/F_y}$ => inelastic

otherwise elastic

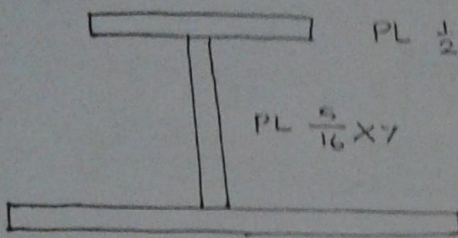
Step : 4 — Calculate F_{cr} based on elasticity

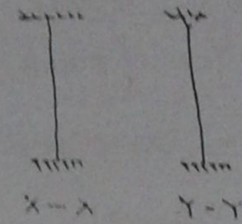
if inelastic → $F_{cr} = \left(0.658^{F_y/F_e}\right) F_y$

elastic → $F_{cr} = 0.877 F_e$ $\quad F_e = \frac{\pi^2 E}{(KL/r)^2}$

Step : 5 - calculate nominal and allowable strength

Find capacity of a column of [illegible] length
Fixed - fixed about strong axis and fixed - pinned about weak axis





- calculation of $\frac{KL}{r}$

Here, $\bar{y}_y = \frac{7/8 \times 14 \times 7/16 + \frac{5}{16} \times 7 \times 4.375 + [illegible]}{7/8 \times 14 + 5/16 \times 7 + [illegible]}$ [illegible]

$A_g = 16.434 \text{ in}^2$

$I_x = (7/8)^3 * 14 * \frac{1}{12} + 7/8 \times 14 \times (2.612 - 7/16)^2 + [illegible]$
$7 \times 5/16 * (2.612 - 4.375)^2 + ([illegible]) [illegible] \times (2.612 - [illegible])$

$= 196.05$

$I_y = \frac{14^3 \times 7/8}{12} + (\frac{5}{16}) [illegible]$

So, $r_x = \sqrt{\frac{I_x}{A}} = 3.26''$. and, $r_y = \sqrt{\frac{I_y}{A}} = [illegible]$

So, $K_x = 0.65$ and, $K_y = [illegible]$

$\frac{K_x L}{r_x} = 35.07$ $\frac{K_y L_y}{r_y} = [illegible]$

So, $\frac{K_y L_y}{r_y} [illegible]$

- inelastic/elastic:

$4.71 \sqrt{E/F_y} = 4.71 \sqrt{\frac{29000}{36}} = [illegible]$

So. $\frac{KL}{r} < 4.71 \sqrt{E/F_y}$ → inelastic buckling

Allowable strength

$$F_e = \frac{\pi^2 E}{(KL/r)^2} = \frac{\pi^2 \times 29000}{11.55^2} = 166 \text{ ksi}$$

$$F_{cr} = \left[0.658^{F_y/F_e}\right] F_y = \left[0.658^{36/166}\right] \times 36 = \ldots$$

So, $P_n = F_{cr} A_g = 32.22 \times 18.\ldots = 6\ldots k$

$$P_{allowable} = \frac{P_n}{1.67} = \frac{60\ldots}{1.67} = \boxed{36\ldots k}$$

Ans.

* Design problem (selecting lightest section)

- Design aid : - sectional properties table
  - colm load table

step : 01 — Compute required Area

Assume, $F_{cr} = 0.7 F_y$

So, $F_{allow} = \frac{F_{cr}}{\Omega} = \frac{0.7 F_{cr}}{1.67} \simeq 0.42 F_{cr}$

So, $A_g = P/F_{allow} = P/0.42 F_{cr} \simeq \frac{167}{70} P$

step : 02 select sections based on A_g

- Calculate capacities of colm (like capacity calculation)

if $P_{allow} \cong DL + LL$ then select the section

$=120^K$, $LL = 75K$ length $= 16'$, Braced frame, pin end

- calculation of Area

Assume, $F_{cr} = 0.7 F_y$

allowable stress, $F_a = \frac{0.7 \times 36}{1.67}$

$A_g = \frac{DL+LL}{15.1} = \frac{195}{15.1} \approx 13 in^2$

- selection of section

sec 01: W 12×45 $\left\{ \begin{array}{l} A_g = 13.2 in^2 \\ r_x = 5.15'' \\ r_y = 1.94'' \end{array} \right.$

Try with section 01

$\frac{K_x L_x}{r_x} = \frac{1 \times 16 \times 12}{5.15} = 37.3$

$\frac{K_y L_y}{r_y} = \frac{1 \times 16 \times 12}{1.94} = 99$

So, $\frac{KL}{r} = 99$

$4.71 \sqrt{E/F_y} = 133.68$

So, $\frac{KL}{r} < 133.68 \Rightarrow$ inelastic

$\therefore F_{cr} = \left[0.658^{F_y/F_e}\right] F_y$ $\left[F_e = \frac{\pi^2 E}{(KL/r)^2} = 29.2\right]$

$= \left[0.658^{36/29.2}\right] 36$

$= 21.5$ Ksi

So, $P_{allow} = \frac{21.5 \times 13.2}{1.67} = 170^k < 195^k$

not ok

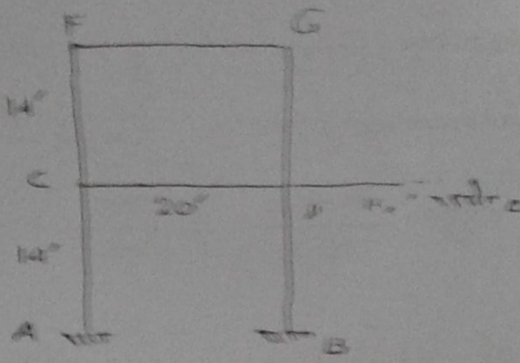
select section $\boxed{W8 \times 48}$

Ans

value of K in frame (CF)

$$G_c = \frac{\Sigma \text{colm stiffness meeting @ C}}{\Sigma \text{beam stiffness meeting @ C}}$$

$$= \frac{\Sigma (EI/L)_{col}}{\Sigma (EI/L)_{Beam}} = \frac{(EI/L)_{CF} + (EI/L)_{AC}}{(EI/L)_{CD}}$$



calculate → G_F

From alignment chart calculate K

Colm with fixed support, $G = 1.0$

" " Pinned " , $G = 10$

modification factor for beam:

Beam with fixed/pinned support modification of beam stiffness

modification factor (beam / Girder)

1. Unbraced → Far end fixed 2/3 ; → Far end pinned 1/2
2. Braced → Far end fixed (2) ; → " " pinned (3/2)

K value for BD

$$G_B = 1.0$$

$$G_D = \frac{(EI/L)_{BD} + (EI/L)_{DG}}{(EI/L)_{CD} + (EI/L)_{DE}} = \frac{425/14 + 425/14}{[illegible]/20 + 3/2 \times [illegible]}$$

- Calculate K for all colm.
- calculate load capacity of Colm BE

Here, $G_A = 1.0$

$G_B = 10.$

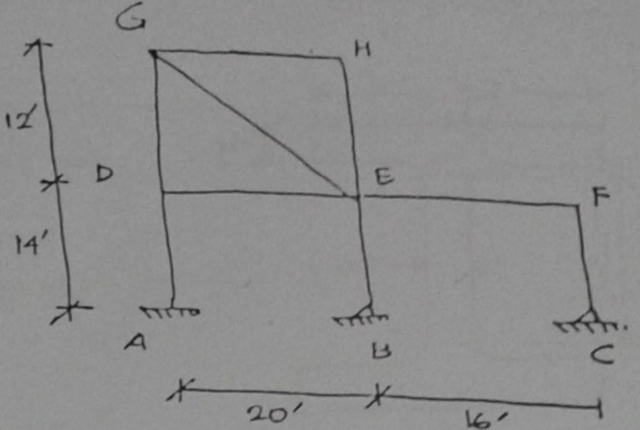
$G_C = 10$

$G_D = \dfrac{425(1/12 + 1/14)}{533/20} = 2.47.$

$G_E = \dfrac{425(1/12 + 1/14)}{533(\frac{1}{20} + \frac{1}{16})} = 1.1$

$G_F = \dfrac{425/14}{533/16} = 0.91$

$G_G = G_H = \dfrac{425/12}{533/20} = 1.33$



Value of K

Braced	Unbraced
$K_{DG} = 0.84$	$K_{AD} = 1.5$
$K_{HE} = 0.8$	$K_{BE} = 1.85$
	$K_{CF} = 1.84$

- Load capacity

$K_{BE} = 1.85$, $L = 14'$

for W12 X53 $\rightarrow$ $A_g = 15.6 in^2$, $r_x = 5.23''$, $r_y = 2.48''$

So, $\dfrac{KL}{r_{min}} = \dfrac{1.85 \times 14 \times 12}{2.48} = 125.3$

Now, $4.71\sqrt{E/F_y} = 133.68$

$\therefore \dfrac{KL}{r} < 4.71\sqrt{E/F_y} \Rightarrow$ inelastic

So, $F_{cr} = \left(0.658^{F_y/F_e}\right) F_y$ $\qquad F_e = \dfrac{\pi^2 E}{(KL/r)^2} = 18.23$

$= \left(0.658^{36/18.23}\right) 36$

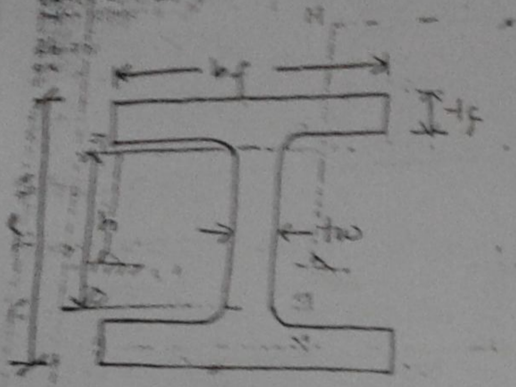
$= 15.75$

So, $P_{allow} = \dfrac{F_{cr} \times A_g}{1.67} = \boxed{147.13K}$

Ans.

Check for buckling (Local)

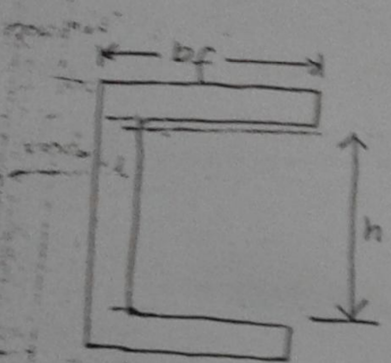
- Unstiffed element → end supported on one side (flange)
- stiffed element → both end supported

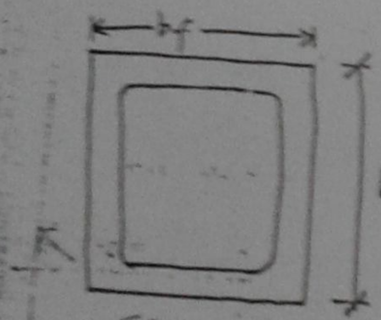


$h = d - 2t_f$

for flange $\frac{b_f}{2t_f} \le 0.56\sqrt{E/F_y}$

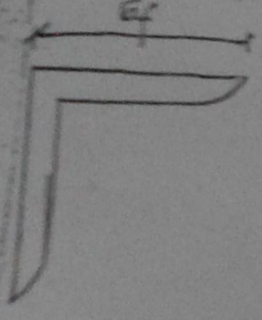
Web $\frac{h}{t} \le 1.49\sqrt{E/F_y}$





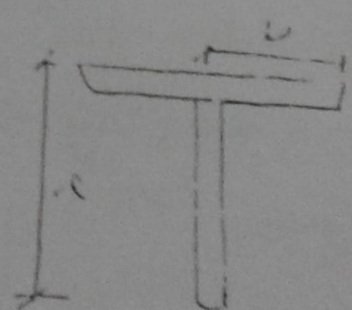
flange $b/t \le$ [illegible] $\sqrt{E/F_y}$

web $h/t \le$ [illegible] $\sqrt{E/F_y}$



For any leg

$b/t \le 0.45\sqrt{E/F_y}$

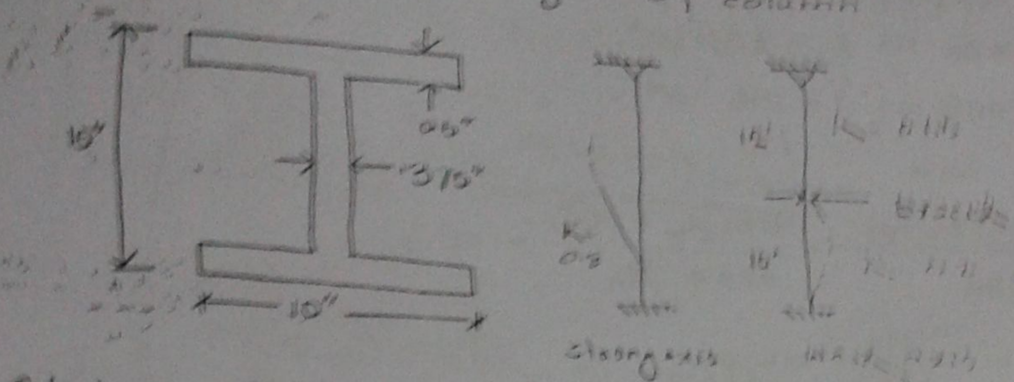


flange $b/t \le 0.56\sqrt{E/F_y}$

$d/t \le$ [illegible] $\sqrt{E/F_y}$

6-11

calculate ultimate strength of column



Sol^n Calculation of KL/r

$A_g = 10 \times 0.5 \times 2 + 14 \times .375 = 15.25 \text{ in}^2$

$\bar{y} = 7.5''$

$I_x = \left[\frac{0.5^3 \times 10}{12} + 0.5 \times 12 \times (7.5 - 0.25)^2\right] \times 2 + \frac{14^3 \times .375}{12} = 716.71 \text{ in}^4$

$I_y = \frac{10^3 \times 0.5}{12} \times 2 + \frac{.375^3 \times 14}{12} = 83.4 \text{ in}^4$

Now, $r_x = \sqrt{\frac{I_x}{A}} = 6.86''$ and, $r_y = 2.34''$

So, $\frac{K_x L_x}{r_x} = \frac{0.8 \times 22 \times 12}{6.86}$ and $\left(\frac{K_y L_y}{r_y}\right)_1 = \frac{0.65 \times 15 \times 12}{2.34} = 50$

$= 30.78$ $\qquad \left(\frac{K_y L_y}{r_y}\right)_2 = \frac{0.8 \times 15 \times 12}{2.34} = 41.03$

So, $\frac{KL}{r} = 41.03$

— elastic/inelastic buckling

$4.71 \sqrt{E/F_y} = 113.43$

So, $\frac{KL}{r} < 4.71 \sqrt{E/F_y} \Rightarrow$ inelastic buckling

$\therefore F_{cr} = \left(0.658^{F_y/F_e}\right) F_y$ $\qquad F_e = \frac{\pi^2 E}{(KL/r)^2} = 170$

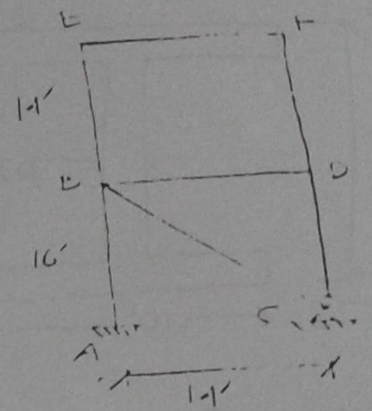
$= \left(0.658^{50/170}\right) 50 = 44.21$

$\therefore P_{allowable} = \frac{44.21 \times 15.25}{1.67} = \boxed{403.71 \text{K}}$ Ans

(a) Determine : Effective length factor

Beam : W12×96 → $I = 833\ in^4$

colm : W10×112 → $I = 716\ in^4$



here, $G_A = 1.0$ $G_C = 10.0$

$$G_B = \frac{(EI/L)_{AB} + (EI/L)_{BE}}{(EI/L)_{BD}} = \frac{716(1/16 + 1/14)}{833 \times 1/14} = 1.61$$

$$G_E = \frac{(EI/L)_{BE}}{(EI/L)_{EF}} = \frac{716/14}{833/14} = 0.86$$

$G_D = 1.61$

$G_F = 0.86$

Unbraced	Braced
$K_{BE} = 1.4$	$K_{AB} = 0.8$
$K_{DF} = 1.4$	$K_{CD} = 0.89$

Ans

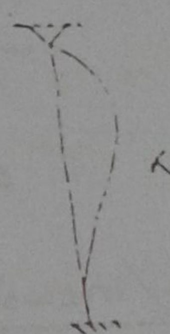
2(b) Select the lightest section

colm length — 20', Load = 180K, Assume fixed-pinned

Here, $F_{cr} = 0.7 F_y = 0.7 * 36 = 25.2$

$$\therefore P_{allow} = \frac{25.2 \times A_g}{1.67} = 180$$

So, $A_g = 11.93\ in^2$



Trail 01 Section :- W10×39

$$So,\ \frac{KL}{r} = \frac{0.8 \times 20 \times 12}{1.98} = 97 < 4.71\sqrt{E/F_y} = 133.69 \Rightarrow \text{inelastic}$$

$$So,\ F_{cr} = \left(0.658^{F_y/F_e}\right) F_y = 21.91$$

$F_e = \frac{\pi^2 E}{(KL/r)^2}$ = ...

$$So,\ P_{allowable} = \frac{21.91 \times 11.93}{1.67} = 156.7K < 180 \Rightarrow \text{not ok}$$

section : W12X40

$\frac{Kl}{r} = 99.46 < 4.7\sqrt{E/F_y} \Rightarrow$ inelastic buckling

So, $F_e = \frac{\pi^2 E}{(Kl/r)^2} = 28.92$

So, $P_{allowable} = \frac{\left(0.658^{36/28.92}\right) 36 \times 11.2}{1.67} = 151.07K \Rightarrow$ not ok

Try section : W10X45

$Kl/r = 96.52 < 4.7\sqrt{E/F_y} \Rightarrow$ inelastic

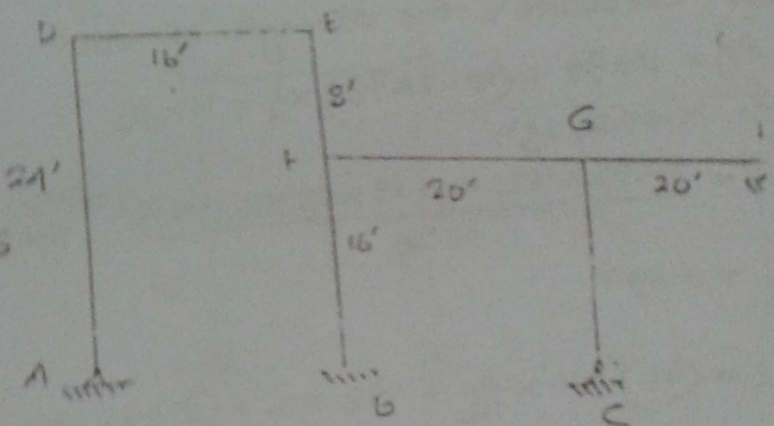
So, $F_e = \frac{\pi^2 E}{(Kl/r)^2} = 31.34$

$P_{allowable} = \frac{\left(0.658^{36/31.34}\right) \times 36 \times 13.3}{1.67} = 117.3K$ (within 2% of 120K)

Select section $\boxed{W10X45}$

Question 09-10

(c) calculate length factor K for BF, CG, EF



K_{BF} $G_B = 1.0$

$G_F = \frac{(EI/L)_{BF} + (EI/L)_{EF}}{(EI/L)_{FG}}$

$= \frac{983\,(1/16 + 1/8)}{983 \times 1/20} = 3.75$

So, $K_{BF} = 0.825$ (Non-sway)

K_{CG} $G_C = 1.0$, $G_G = \frac{1/16}{1/20 + 1/20} = 0.625$

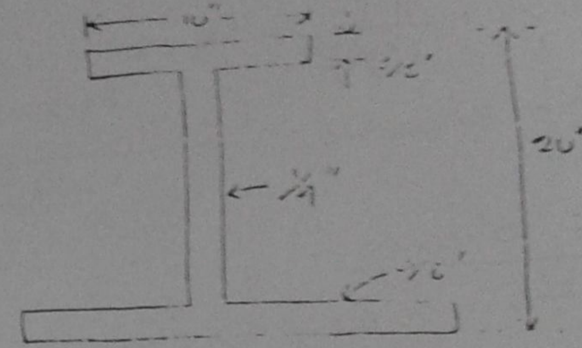
$K_{CG} =$

K_{EF} $G_E = \frac{1/8}{1/16} = 2$, $G_F = \frac{8^{-1} + 16^{-1}}{20^{-1}} = 3.75$

K_{EF}

2(a) Determine : whether beam shape will have local buckling

Assume A36 steel



For upper flange.

$$\frac{b_f}{2t_f} = \frac{10}{2\times 3/8} = \frac{10}{3} = 13.33$$

and, $0.56\sqrt{E/F_y} = 15.89$

So, $\frac{b_f}{2t_f} < 0.56\sqrt{E/F_y}$

Web:

$h = d - 3t_f = 20 - 3\times 3/8 = 18.875''$

$$h/t = \frac{18.875}{1/4} = 75.5$$

and, $1.49\sqrt{E/F_y} = 42.29$

So, $\frac{h}{t} > 1.49\sqrt{E/F_y}$

lower flange

$$\frac{b_f}{2t_f} = \frac{16}{2\times 3/8} = 21.33 > 0.56\sqrt{E/F_y}$$

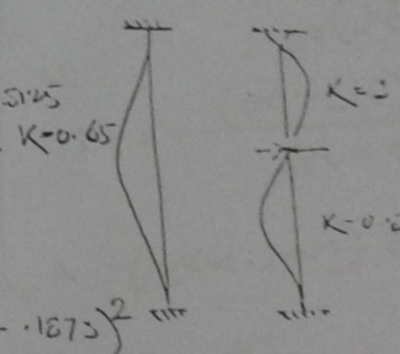
2(b) calculate : the strength of colm.

$Ag = 10\times 3/8 + (20 - 2\times 3/8)\times \frac{1}{4} + 16\times 3/8$

$= 14.5625^2$

$$\bar{y} = \frac{16\times 3/8 \times .1875 + 19.25\times .25\times 10 + 10\times 3/8 \times 19.8125}{14.56}$$

$= 8.49''$



$$I_x = (3/8)^3\times 16\times \frac{1}{12} + 3/8\times 16\times(8.49 - .1875)^2$$

$$+ \frac{19.25^3\times 0.25}{12} + 19.25\times 0.25\times(8.49-10)^2$$

$$+ (3/8)^3\times 10\times\frac{1}{12} + 3/8\times 10\times(8.49 - 19.8125)^2$$

$= 1054.02\ in^4$

$$I_x = 16^3\times 3/8\times\frac{1}{12} + (0.25)^3\times 19.25\times\frac{1}{12} + 10^3\times(3/8)\times\frac{1}{12} = 159.3\ in^4$$

$r_x = \sqrt{\frac{I_x}{A}} = 8.51$ $r_y = \sqrt{\frac{I_y}{A}} = 3.31$

So, $\frac{K_x L_x}{r_x} = \frac{0.65 \times 20 \times 12}{8.51}$ and $\left(\frac{L_y K_y}{r_y}\right)_1 = \frac{8 \times 12}{3.31} = 29$

$= 18.33$

$\left(\frac{K_y L_y}{r_y}\right)_2 = \frac{0.8 \times 12 \times 12}{3.31} = 34.8$

So, $\frac{KL}{r} = 34.8$

Now, $\frac{KL}{r} = 18.33 < 4.7\sqrt{E/F_y} = 133.7 \Rightarrow$ inelastic buckling

$F_e = \frac{\pi^2 E}{(KL/r)^2} = 236.37$

So, $P_{all} = \frac{\left(0.658^{F_y/F_e}\right) F_y \times 14.56}{1.67} = \boxed{294.5\text{ K}}$

Ans

Question 08-09

1(a) Determine: Effective length factor. $I = 233$

$G_A = 10$, $G_B = 1$.

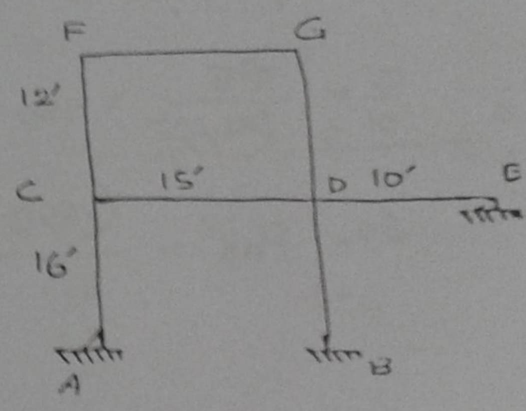
$G_C = \frac{1/16 + 1/12}{1/15} = 2.1875$

$G_D = \frac{1/16 + 1/12}{1/15 + 1/10 \times 3/2} = 0.673$

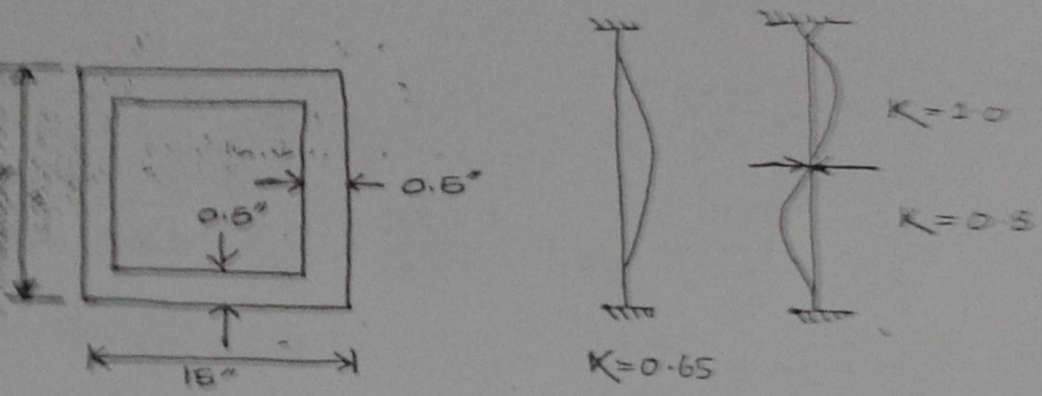
$G_F = \frac{1/12}{1/15} = 1.25$

$G_G = 1.25$

$K_{AC} =$



1(b) Calculate the strength having cross section



$$I_x = \frac{20^3 \times 15}{12} - \frac{19^3 \times 14}{12} = 1997.83\ in^4$$

$$I_y = \frac{15^3 \times 20}{12} - \frac{14^3 \times 19}{12} = 1280.33\ in^4$$

$$A = 20 \times 15 - 19 \times 14 = 34$$

So, $r_x = 7.67"$, $\frac{K_x L_x}{r_x} = \frac{0.65 \times 24 \times 12}{7.67} = 24.41"$

$r_y = 6.14"$, $\frac{K_y L_y}{r_y} = \frac{1 \times 12 \times 12}{6.14} = 23.45"$

So, $\frac{KL}{r} = 24.41 < 4.7\sqrt{E/F_y} = 133.3 \Rightarrow$ inelastic

$$F_e = \frac{\pi^2 E}{(KL/r)^2} = 480.4$$

So, $$P_{all} = \frac{\left(.658^{F_y/F_e}\right) F_y \times 34}{1.67} = \boxed{710.3\ K}$$

Ans

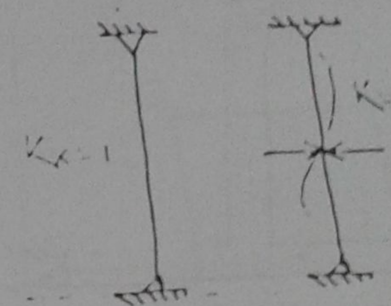
Select ; economical W12 section

Given, A572, Length of col^m = 20', load = 470K

let, $F_{cr} = 0.7 \times F_y = 35$

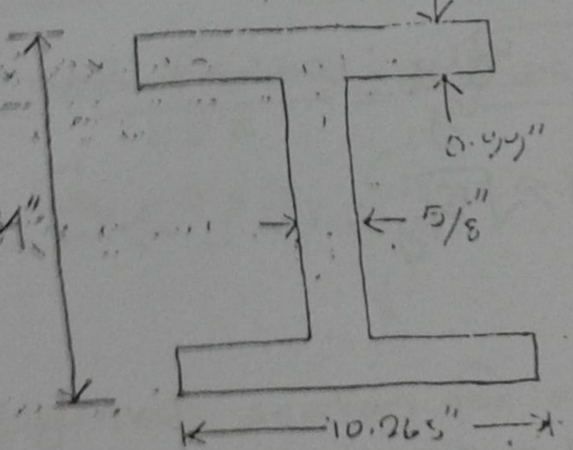
Now, $P_{all} = 470 = \dfrac{35 \times A_g}{1.67}$

$\therefore A_g = 22.42 \text{ in}^2$



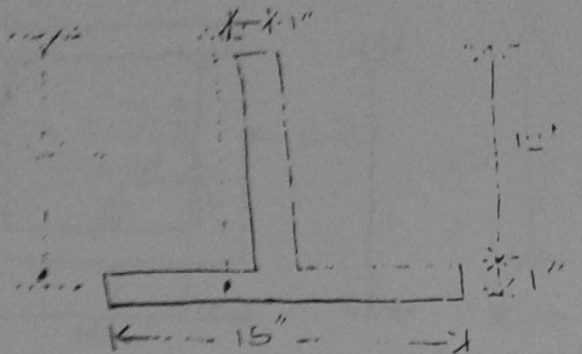
Section	A_g	r_x	r_y	$\frac{K_x L_x}{r_x}$	$\frac{K_y L_y}{r_y}$	KL/r	F_c	P_{all}
W10X77	22.6	4.49	2.6	80.18	69.23	80.12	14.32	422.9K
W10X88	25.9	4.51	2.63	79.3	66.1	79.3	15.51	489.63K

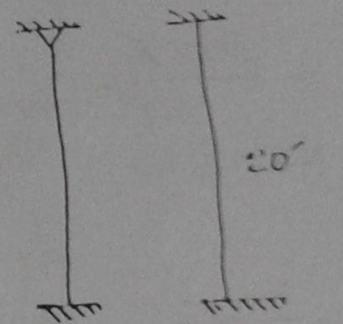
So, selected section $\boxed{\text{W10X88}}$



S 07-08

Determine the strength of the cross section





strong weak

$$= \frac{16\times1\times0.5 + 12\times1\times7}{15\times1 + 12\times1} = 3.39$$

$$I = \frac{1^3\times15}{12} + 1\times15\times((3.39)-0.5)^2 + \frac{12^3\times1}{12} + 12\times1\times(3.39-7)^2 = 426.92$$

$$I = \frac{15^3\times1}{12} + \frac{1^3\times12}{12} = 282.25\ in^4.$$

So, $r_x = \sqrt{I/A} = 3.98$ and, $r_y = \sqrt{I_y/A} = 3.23''$

Now, $\frac{K_xL_x}{r_x} = \frac{0.8\times20\times12}{3.98} = 48.24$, $\frac{K_yL_y}{r_y} = \frac{0.65\times20\times12}{3.23} = 48.3''$

So, $\frac{KL}{r} = 48.3 < 4.7\sqrt{E/F_y} = 113.2 \Rightarrow$ inelastic buckl

Now, $F_e = \frac{\pi^2E}{(KL/r)^2} = 122.69$

$$F_{cr} = \left(0.658^{F_y/F_e}\right)F_y = \cancel{44.22}\ 42.16$$

$\therefore$ $P_{allow} = \frac{44.22\times27}{1.67} = \cancel{[illegible]}\ 681K$

Ans

2(a) Calculate : Effective length factor

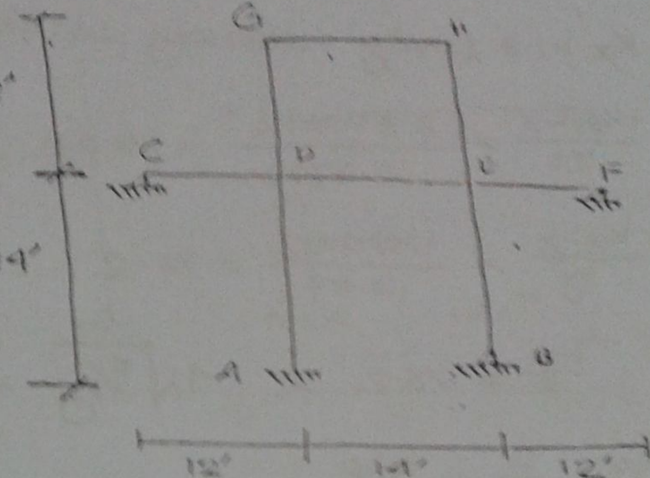
Column : W12X53 $I_x = 425$

Beam : W12 X65 $I_x = 533$

Here, $G_A = 1.0$, $G_B = 10$

$$G_D = \frac{425\left(\frac{1}{12}+\frac{1}{14}\right)}{533\left(\frac{1}{12}\times\frac{2}{2}+\frac{1}{14}\right)} = 0.63 = G_E$$

$$G_G = \frac{425\left(\frac{1}{12}\right)}{533\left(14^{-1}\right)} = 0.93 = G_H$$



K_{AD} $G_A = 1.0$ and $G_D = 0.63$

$\boxed{K_{AD} = 0.74}$

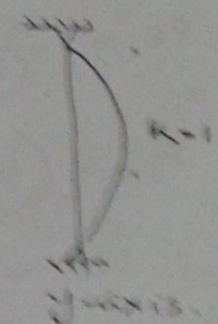
K_{BE} $G_B = 10$ and $G_E = 0.63$, $\boxed{K_{BE} = 0.885}$ (non-sway)

K_{DG} (sway) $G_D = 0.63$, $G_G = 0.93$, $\boxed{K_{DG} = 1.25}$

2(b) Select lightest section for exterior colm. Assume the colm is braced at top and bottom in the plane perpendi to the plane of frame. (anx 1)

Soln Here DL + LL = 140 + 310 = 450K

$I_{beam} = 882 \text{ in}^4$



Here, $K_y = 1.0$, $L_y = 20'$ So, KL = 20

From Annexure -1 KL = 20 and P = 450K

So, section W12×87

P = 496K, A = 25.6, $I_x = 740$, $I_y = 241$, $r_y = 3.09''$, $r_x = 1.75\times 3.09 = 5.37$

so, $G_{lower} = 1.0$

$$G_{upper} = \frac{740\,(1/20)}{882\,(1/22)} = 0.923$$

$K_x = 1.3$ (sway – sideway uninhibited)

So, $$\frac{K_xL_x}{r_x} = \frac{1.3 \times 20 \times 12}{5.3725} = 58.07$$

$$\frac{K_yL_y}{r_y} = \frac{1 \times 20 \times 12}{3.07} = 78.2$$

$$\frac{KL}{r} = 78.2 < 4.71\sqrt{E/F_y} = 113.9$$

Here, $$F_e = \frac{\pi^2 E}{(KL/r)^2} = 46.8$$

$$F_{cr} = \left(0.658^{50/46.8}\right) 50 = 31.97$$

$$P_{all} = \frac{31.97 \times 25.6}{1.67} = 490^k < 450$$

select section $\boxed{W\,12 \times 87}$

Ans

6(a) check adequacy of W14×82 section. Load = 300 kip

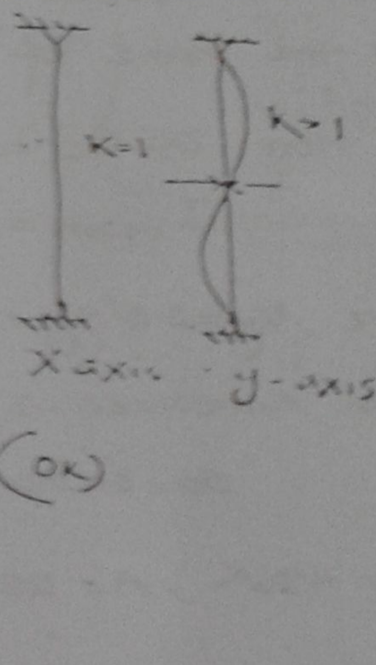
Here, $\frac{K_xL_x}{r_x} = 51.57$ and $\frac{K_yL_y}{r_y} = 62.9$

So $KL/r = 62.9 < 4.71\sqrt{E/F_y}$

$$F_e = \frac{\pi^2 E}{(KL/r)^2} = 72.34$$

$$F_{cr} = \left(0.658^{36/72.34}\right) 36 = 29.23$$

$$\therefore P_{all} = \frac{29.23 \times 24.1}{1.67} = 421^k < 300^k \text{ (OK)}$$



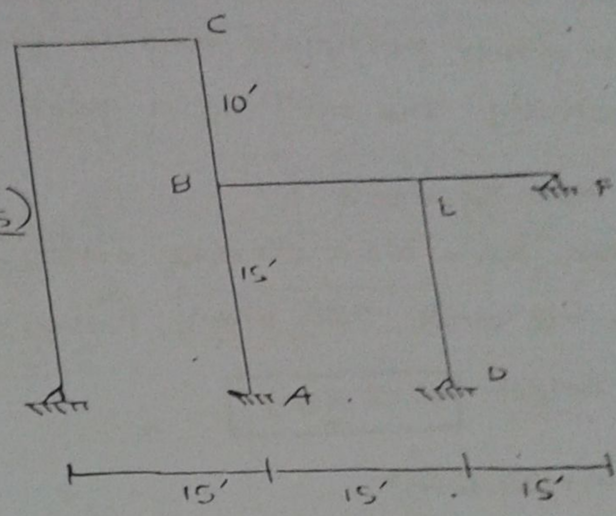
The section is adequate

es. 2006-07

1(a). claculate: Eff. length factor of colm AB, BC, DE

Colm : W21 X57, I = 1170

Beam: W24 X62, I = 1550



$\underline{\underline{K_{AB}}}$ (No sideway).

$G_A = 1.0$ and, $G_B = \dfrac{1170\left(\frac{1}{10}+\frac{1}{15}\right)}{1550/15}$

$= 1.89$

so, $\boxed{K_{AB} = 0.81}$

$\underline{\underline{K_{BC}}}$ $G_B = 1.89$, $G_C = \dfrac{1170/10}{1550/15} = 1.132$

so, $\boxed{K_{BC} = 1.45}$ (sideway)

$\underline{K_{DE}}$ $K_D = 10$, $G_E = \dfrac{1170/15}{\frac{1550}{15} + \frac{1550}{15} \times 3/2} = 0.302$

$\boxed{K_{DE} = 0.775}$ (no-side way)

s 06-07

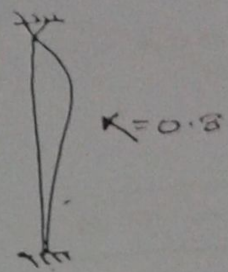
select lightest W12 section of column 20′ long. Given, load = 400K. Assume top and bottom for bending about either principle axis is pinned/hinged and fixed respectively. The col^m is a part of braced frame.

Here, $K_x = K_y = 0.8$

So, $KL = 0.8 \times 20 \times 12 = 192/12 = 16$

For $KL = 16'$ and A36 steel, $P = 400K$

Select $\boxed{W12 \times 87}$



Where, $A = 25.6\ in^2$

$I_x = 740$

$I_y = 241$

$r_y = 3.07''$

$r_x = 1.75 \times 3.07 = 5.373''$

Now, $\frac{KL}{r} = \frac{0.8 \times 20 \times 12}{3.07} = 62.54 < 4.7\sqrt{E/F_y} = 133$

So, $F_e = \frac{\pi^2 E}{(KL/r)^2} = 73.17$

$\therefore F_{cr} = \left[0.658^{F_y/F_e}\right] F_y = 29.3$

$P_{allow} = \frac{29.3 * 25.6}{1.67} = 449K < 400K$ (OK)

So, selected section $\boxed{W12 \times 87}$

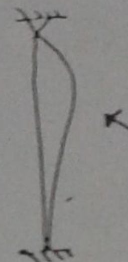
Ans.

s 06-07

Select lightest W12 section of column 20' long. Given, load = 400K. Assume top and bottom for bending about either principle axis is pinned/hinged and fixed respectively. The col^m is a part of braced frame.

Here, $K_x = K_y = 0.8$

So, $KL = 0.8 * 20 * 12 = 192/12 = 16$



K=0.8

For KL = 16' and A36 steel, P = 400K

Select $\boxed{W12 \times 87}$

Where, $A = 25.6 \text{ in}^2$

$I_x = 740$

$I_y = 241$

$r_y = 3.07''$

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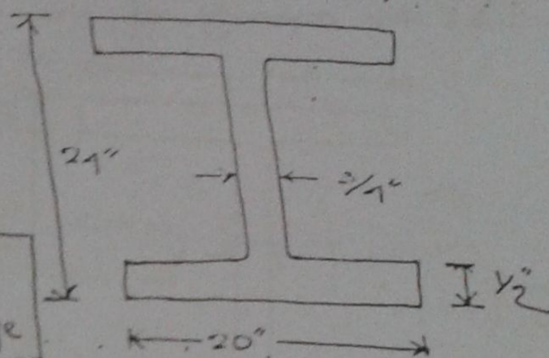
$P_{allow} = \frac{29.3 * 25.6}{1.67} = 449K < 400K \text{ (ok)}$

So, selected section $\boxed{W12 \times 87}$

Ans.

2(a) Determine whether col^{m} shape will have local buckling ...

<u>Sol^n</u>



flange: $\frac{b_f}{2t_f} = \frac{20}{2 \times \frac{1}{2}} = 20$

$0.56\sqrt{E/F_y} = 13.48$

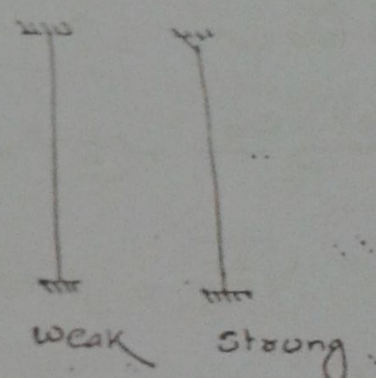
So, $\frac{b_f}{2t_f} > 0.56\sqrt{E/F_y} \Rightarrow$ $\boxed{\text{local buckling occure @ flange}}$

Web: $\frac{d - 3t_f}{t_w} = \frac{24 - 3 \times \frac{1}{2}}{3/4} = 30$

and, $1.49\sqrt{E/F_y} = 35.89$

So, $\frac{h}{t_w} < 1.49\sqrt{E/F_y} \Rightarrow$ $\boxed{\text{no local buckling @ web}}$

2(b) : Find : the ultimate compression capacity of W 21×157.
L = 20' use A 572 steel



For W12×157

$A = 16.7\ in^2$, $r_x = 8.36"$, $r_y = 1.35"$

So, $\frac{K_x L_x}{r_x} = \frac{0.8 \times 20 \times 12}{8.36} = 22.97$

$\frac{K_y L_y}{r_y} = \frac{0.65 \times 20 \times 12}{1.35} = 115.56$

So, $\frac{KL}{r} = 115.56 > 4.7\sqrt{E/F_y} = 113.2$

So, $F_e = \frac{\pi^2 E}{(KL/r)^2} = 21.43$

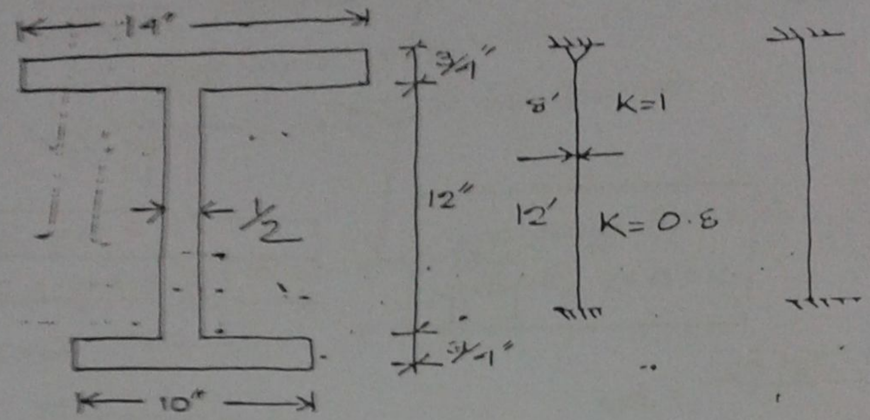
$\therefore F_{cr} = 0.877 F_e = 18.8$

Now, $P_{allowable} = \frac{18.8 \times 16.7}{1.67}$

$= \boxed{188K}$ Ans

Ques 05-06

(c) Find the capacity of the colm. Use A36 steel



$A = 10\times 3/4 + 12\times \frac{1}{2} + 14\times 3/4 = 24$

$\bar{y} = \dfrac{10\times 3/4\times 3/8 + 12\times \frac{1}{2}\times 6.75 + 14\times 3/4\times 13.125}{24} = 7.547''$

$I_x = (3/4)^3\times 10\times \frac{1}{12} + 3/4\times 10\times (7.547 - 3/8)^2 + 12^3\times \frac{1}{2}\times \frac{1}{12} +$
$12\times \frac{1}{2}\times (7.547 - 6.75)^2 + (3/4)^3\times 14\times \frac{1}{12} + 3/4\times 14\times (13.125 - 7.547)^2$
$= 789.135\ in^4.$

$I_y = \dfrac{10^3\times 3/4}{12} + \left(\frac{1}{2}\right)^3\times 12\times \frac{1}{12} + 14^3\times 3/4\times \frac{1}{12} = 234.125$

So, $r_x = \sqrt{\dfrac{I_x}{A}} = 5.734''$ and, $r_y = \sqrt{\dfrac{I_y}{A}} = 3.123''$

$\therefore \left(\dfrac{K_x L_x}{r_x}\right)_1 = \dfrac{1\times 8\times 12}{5.734} = 16.742$

$\left(\dfrac{K_x L_x}{r_x}\right)_2 = \dfrac{0.8\times 12\times 12}{5.734} = 20.091$

$\dfrac{K_y L_y}{r_y} = \dfrac{0.65\times 20\times 12}{3.123} = 45.95$

$\therefore \dfrac{KL}{r} = 45.95 < 4.7\sqrt{E/F_y}$

$\Rightarrow$ inelastic buckli[ng]

So, $F_e = \frac{\pi^2 E}{(KL/r)^2} = 114.71$

Now, $P_{all} = \frac{\left(0.658^{36/114.71}\right) \times 36 \times 24}{1.67} = \boxed{453.68K}$

Ans.

4(a) Using load table select the lightest section for int col^m. Assume col^m is braced at top and bottom in plane perpendicular to the plane of frame. Use A36 steel

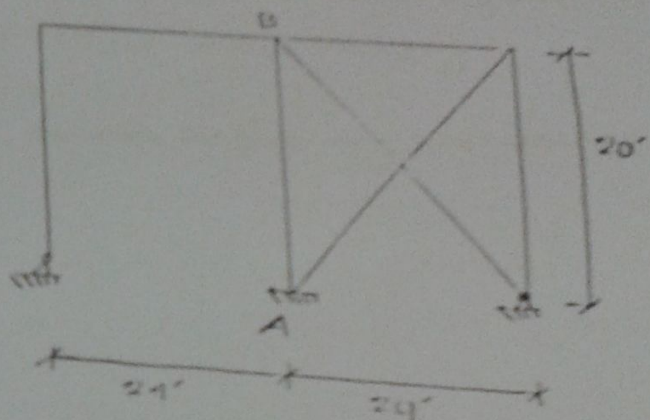
Given, DL = 100K, Live load = 250K, $I_{beam} = 882\ in^4$

As top and bottom of col^m is braced so, $K_y = 1$.

Now, $K_y L_y = 1 \times 20 = 20$

For, KL = 20, P = 350K and $F_y = 36$

Select W 12X79



Here, $A = 23.2\ in^2$

$I_x = 662\ in^4$, $I_y = 216$

$r_y = 3.05"$, $r_x = 1.75 \times 3.05 = 5.335"$

Now, $G_A = 1.0$, $G_B = \frac{662 \times 1/20}{882 \times (1/24 + 1/24)} = 0.45$

col^m AB is non-sway or sideway inhibited so, $K_{AB} = 0.72$

$\therefore \frac{K_y L_y}{r_y} = \frac{1 \times 20 \times 12}{3.05} = 78.69$ and $\frac{K_x L_x}{r_x} = \frac{0.72 \times 20 \times 12}{5.335} = 32$

So, $KL/r = 78.69K$

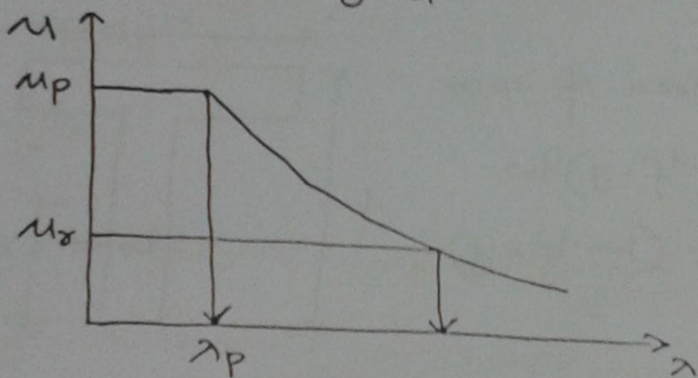
$\therefore F_e = \frac{\pi^2 E}{(KL/r)^2} = 46.22$

$P_{allowable} = \frac{\left(0.658^{36/46.22}\right) \times 36 \times 23.2}{1.67} = 361K > 350K$

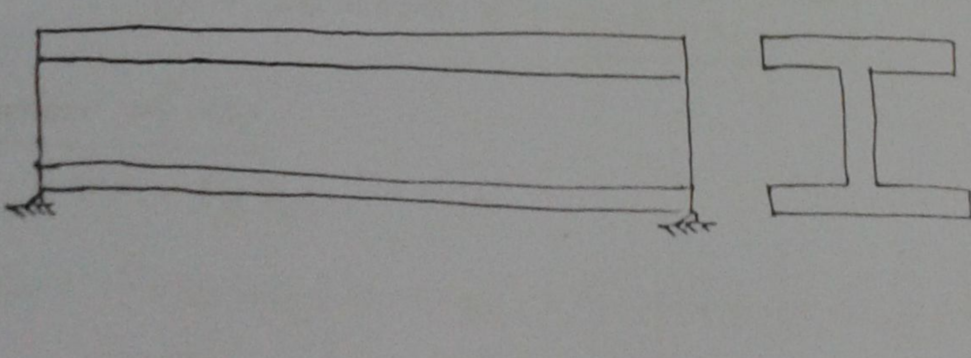
OK

Beam

* Yield moment, $M_y = F_y S_x$
  where, S_x = section modulus.
- plastic moment, $M_p = F_y Z_x$
  Z_x = plastic modulus.
- section types: three types-
  (i) compact section $\lambda \leq \lambda_p$: section that can develop fully plastic stress distribution before local buckling
  (ii) Non compact $\lambda_p < \lambda < \lambda_r$: some part of section reaches yield stress but not all before buckling
  (iii) slender section $\lambda > \lambda_r$: buckling occurs before any part reaches to yield.



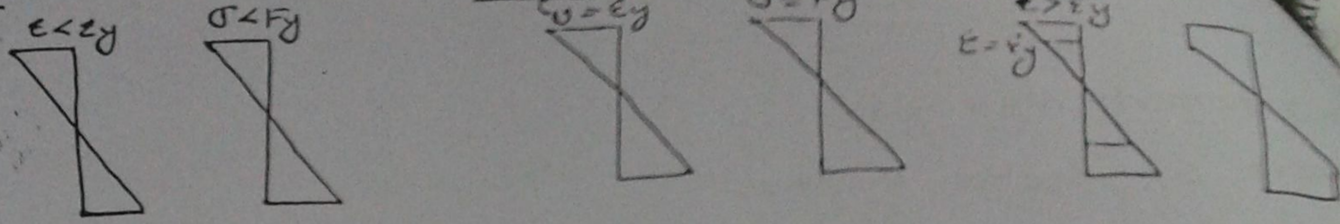
stress - strain condition



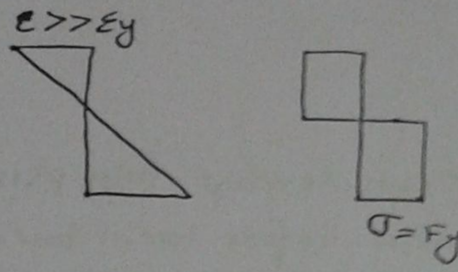
Case : I

Case II

Case III



Case 4



all fibers yield

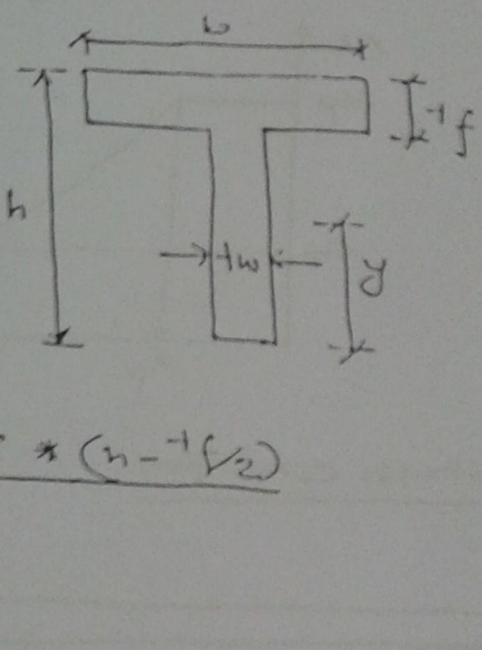
⊛ Find: NA of yield moment and plastic moment
Calculate: yield and plastic moment and shape factor

NA of ~~yield~~ M_p

Area of tension = Area of com

So, $y t_w = b t_f + (h - t_f - y) t_w$

$\therefore y = b(t_f/t_w) + (h - t_f - y)$



N.A of M_y:

$$\bar{y} = \frac{(h-t_f)t_w * (h-t_f)/2 + b t_f * (h - t_f/2)}{(h-t_f)t_w + b t_f}$$

...ulate : NA of M_y and M_p, M_y and M_p.

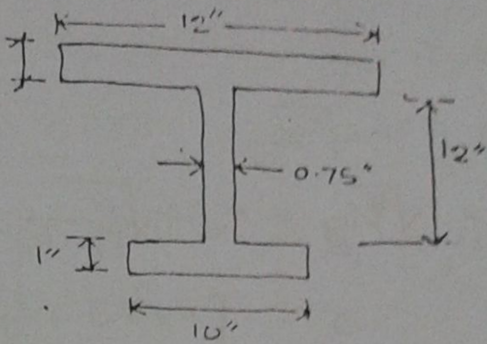
Soln

NA for M_p :

$1\times10 + (y-1)\times0.75 = 12\times1 + (13-y)\times0.75''$

or, $10 + 0.75y - 0.75 = 12 + 9.75 - 0.75y$

or, $1.5y = 12.5''$ So, $\boxed{y_{M_p} = 8.33''}$ Ans.



(ii) NA of M_y

$$\bar{y} = \frac{10\times1\times0.5 + 12\times0.75\times7 + 12\times1\times13.5}{10\times1 + 12\times0.75 + 12\times1} = \boxed{7.42''}$$

Ans.

(iii) Calculation of M_y

$c = \frac{F_y}{2}$ [12 ×1

Now, $M_y = F_y S_x$

$S_x = \frac{I_x}{c}$

$$I_x = \frac{1^3\times10}{12} + 1\times10\times(7.42-0.5)^2 + \frac{12^3\times0.75}{12} + 12\times0.75\times(7.42-7)^2$$
$$+ \frac{1^3\times12}{12} + 1\times12\times(13.5-7.42)^2 = 1033.88$$

So, $S_x = \frac{1033.88}{7.42} = 139.34$

$\therefore M_y = 36\times139.31 = \boxed{5016\text{ K-in}}$

(iv) Calculation of M_p

$M_p = F_y Z_x$

$$Z_x = 1\times12\times(13.5-7.42) + 5.58\times0.75\times(2.79) + 6.42\times0.75\times3.21$$
$$+ 10\times1\,(7.42-0.5) = 169.3$$

So, $\boxed{M_p = 6094.5\text{ K-in}}$ Ans

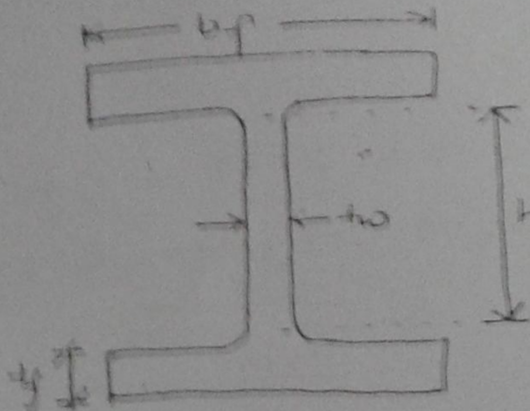
* λ_p and λ_r for I-section :

– $\lambda_{flange} = \dfrac{b_f}{2t_f}$

$\lambda_p = 0.38\sqrt{E/F_y}$

$\lambda_r = \sqrt{E/F_y} = 0.83\sqrt{\dfrac{E}{F_y - F_r}}$

$[F_r = 0.3F_y]$



$\lambda_{web} = h/t_w$

$\lambda_p = 3.76\sqrt{E/F_y}$ and $\lambda_r = 5.7\sqrt{E/F_y}$

* identify the section : Compact/Non compact/Slender

– For flange

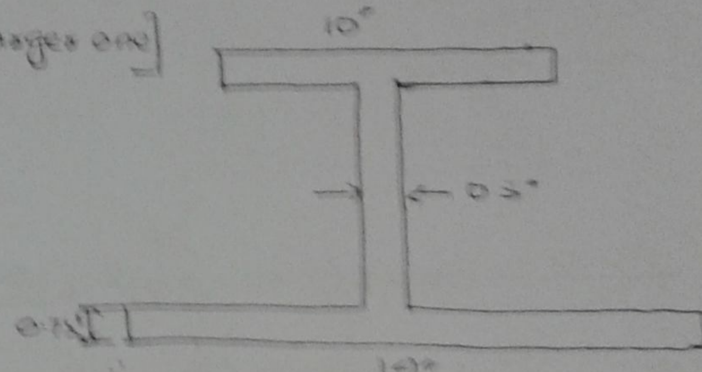
$\lambda = \dfrac{b_f}{2t_f} = \dfrac{14}{2\times 0.75}$ [bf larger one]

$= 9.33$

$\lambda_p = 0.38\sqrt{E/F_y} = 10.78$

$\lambda_r = \sqrt{E/F_y} = 28.38$

so, $\lambda < \lambda_p \Rightarrow$ compact



– For web:

$h = d - 3t_f = 16 - 3\times 0.75 = 13.75$

So, $\lambda = h/t_w = \dfrac{13.75}{0.5} = 27.5$

$\lambda_p = 3.76\sqrt{E/F_y} = 106.72$

$\lambda_r = 5.7\sqrt{E/F_y} = 161.78$

so, $\lambda < \lambda_p \Rightarrow$ compact

...rally supported beams:

max^m unbraced length = distance b/n two adjacent supports

$$L_b < L_p = 1.76\, r_y \sqrt{E/F_y} \quad [\text{I and C section}]$$

- lateral support provided @ supports and point of concentrate

nominal strength that governed by FLB and WLB only

For compact section ($\lambda \le \lambda_p$); $M_n = M_p = F_y Z_x \le 1.5 M_y$

For non compact section ($\lambda_p < \lambda < \lambda_r$),

$$M_n = M_p - (M_p - M_r) \times \frac{\lambda - \lambda_p}{\lambda_r - \lambda_p} \le M_p$$

$$M_r = 0.7 F_y S_x$$

when WLB controls → compact $M_n = M_p = F_y Z_x$

→ Non compact, same as before

$$M_r = F_y S_x$$

* section W 14X90 → $\lambda_{flange} = 10.2$

$\lambda_{web} = 25.9$

$Z_x = 157 \ in^3$

$S_x = 143 \ in^3$

DL = 0.9 K/' LL = 1 K/' A992 steel, $F_y = 50$ ksi

Soln

① compactness of section

$\lambda_{flange} = 10.2$

$\lambda_p = 0.38 \sqrt{E/F_y} = 9.15$

$\lambda_r = \sqrt{E/F_y} = 24.08$

so, $\lambda_p < \lambda_{flange} < \lambda_r$

⇒ non compact (governs)

$\lambda_{web} = 25.9$

$\lambda_p = 3.76 \sqrt{E/F_y} = 90.55$

$\lambda_r = 5.7 \sqrt{E/F_y} = 137.3$

so, $\lambda_w < \lambda_p$ ⇒ compact

So the section will be non compact

$$M_n = M_p - (M_p - M_r)\frac{\lambda - \lambda_p}{\lambda_p - \lambda_r} \le 1.5 F_y S_x$$

$M_p = F_y Z_x = 50 \times 157 = 655 K'$

$M_r = 0.7 F_y S_x = 0.7 \times 50 \times 143 = 417.1 K$

So, $M_n = 655 - (655 - 417)\dfrac{10.2 - 9.15}{-9.15 + 22.35} \le 1.5 F_y S_x$

$= 636.26$

So, $M_u = \dfrac{636.26}{1.67} = 380.88 K'$

Now, $M_{max} = \dfrac{wL^2}{8} = \dfrac{1.4 \times 40^2}{8} = 280 K' < M_u$

* $DL = 0.6 K/'$, $LL = 1 K/'$ Assume no LTB, W 12 X 87
- Adequate or not
- distance between lateral supports

Soln $w = 0.2 + 0.8 = 1 K/'$

$M_u = \dfrac{wL^2}{8} = 50 K\text{-}ft$

For flange: $\lambda_{flange} = 7.5$

$\lambda_p = 0.38\sqrt{E/F_y} = 9.15$

$\lambda_r = \sqrt{E/F_y} = 21.083$

So, $\lambda_f < \lambda_p \Rightarrow$ compact

$\lambda_w = 24.3$

$\lambda_p = 3.76\sqrt{E/F_y} = 90.55$

$\lambda_w < \lambda_p \Rightarrow$ compact

So the section is compact

$M_n = F_y Z_x \le 1.5 F_y S_x$

$M_n = 50 \times 132 \le 1.5 \times 50 \times 118$

$= 6600 \le 8850$

So, $M_n = 6600$

$$So,\ M_u = \frac{6600}{1.67} = 3952.1K$$

$$M_{max} = \frac{wL^2}{8} = \frac{1.6\times36^2}{8} = 259.2K' < M_u\ (OK)$$

section is adequate

* select lightest section assuming A992 steel. The beam is fully supported against LTB, $w = DL + LL = (0.2 + 0.8) = 1K/'$, $L = 20'$

Soln

$w = 0.2 + 0.8 = 1K/'$ $\quad M_u = \frac{wL^2}{8} = 50\ K\text{-}ft$

Assume a compact section

Moment capacity, $M_n = M_p = F_y Z_x$

Now, $\frac{F_y Z_x}{\Omega} = M_u$

So, $Z_x = \frac{50\times12\times1.67}{50} = 20.04\ in^3$

select W12X16 section, $Z_x = 20.1\ in^3$

checking compactness:

$\lambda = b/t = \frac{b_f}{2t_f} = 7.53 < \lambda_p = 0.38\sqrt{E/F_y} = 9.15$

FLB section compact

So, $M_n = F_y Z_x = 50\times20.1 = 1005\ K.in = 83.75K'$

$M_u = \frac{M_n}{1.67} = 50.15 > 50K'$ (OK).

- max^m unbraced length at which the nominal bending strength equals the plastic moment capacity

$$L_p = 1.76\, r_y \sqrt{E/F_y}$$

- The unbraced length at which LTB occurs:-

$$L_r = 1.95\, r_{ts} \frac{E}{0.7F_y} \sqrt{\frac{Jc}{S_x h_o}} \sqrt{1 + \sqrt{1 + 6.76\left(\frac{0.7F_y}{E}\, \frac{S_x h_o}{Jc}\right)^2}}$$

Here, $r_{ts} = \left(\frac{\sqrt{I_y C_w}}{S_x}\right)^{\frac{1}{2}}$

$c = \frac{h_o}{2}\sqrt{\frac{I_y}{C_w}}$ (For channel)

$c = 1.0$ (I-shapes)

F_y = yield strength

E = modulus of elasticity

J = Torsional constant

S_x = section modulus (x-axis)

I_y = moment of inertia (y-axis)

C_w = warping constant

h_o = distance between flange centroids.

- if $L_b \le L_p$ = failure mode flexural yielding

$$M_n = M_p = F_y Z_x$$

- for compact I or C-shapes $L_p < L_b < L_r$

$$M_n = C_b \left[M_p - (M_p - 0.7F_y S_x)\frac{L_b - L_p}{L_r - L_p}\right] \le M_p$$

- if $L_b > L_r$

$$M_n = F_{cr} S_x \le M_p$$

$$F_{cr} = \frac{C_b \pi^2 E}{\left(\frac{L_b}{r_{ts}}\right)^2} \sqrt{1 + 0.078 \frac{Jc}{S_x h_o}\left(\frac{L_b}{r_{ts}}\right)^2}$$

- maxm unbraced length at which the nominal bending strength equals the plastic moment capacity

$$L_p = 1.76\, r_y \sqrt{E/F_y}$$

- The unbraced Length at which LTB occurs:-

$$L_r = 1.95\, r_{ts} \frac{E}{0.7F_y} \sqrt{\frac{Jc}{S_x h_o}} \sqrt{1+\sqrt{1+6.76\left(\frac{0.7F_y}{E}\frac{S_x h_o}{Jc}\right)^2}}$$

Here, $r_{ts} = \left(\frac{\sqrt{I_y C_w}}{S_x}\right)^{\frac{1}{2}}$

$c = \frac{h_o}{2}\sqrt{\frac{I_y}{C_w}}$ (For channel)

$c = 1.0$ (I-shapes)

F_y = yield strength

E = modulus of elasticity

J = Torsional constant

S_x = section modulus (x-axis)

I_y = moment of inertia (y-axis)

C_w = warping constant

h_o = distance between flange centroids

- if $L_b \le L_p$: failure mode flexural yielding

$$M_n = M_p = F_y Z_x$$

- for compact I or C-shapes $L_p < L_b < L_r$

$$M_n = C_b\left[M_p - (M_p - 0.7F_y S_x)\frac{L_b - L_p}{L_r - L_p}\right] \le M_p$$

- if $L_b > L_r$

$$M_n = F_{cr} S_x \le M_p$$

$$F_{cr} = \frac{C_b \pi^2 E}{\left(\frac{L_b}{r_{ts}}\right)^2}\sqrt{1+0.078\frac{Jc}{S_x h_o}\left(\frac{L_b}{r_{ts}}\right)^2}$$

C_b = moment gradient

$$= \frac{12.5 M_{max}}{2.5 M_{max} + 3M_A + 4M_B + 3M_C} \quad R_m \le 3.0$$

M_{max} = Absolute value of max^n moment in the unbraced seg

M_A = " " of moment @ 1/4 point of " "

M_B = " " of " @ CL. " " "

M_C = " " " " @ 3/4 " " "

R_m = section symmetry factor

= 1.0 for doubly symmetric members (I-shapes)...

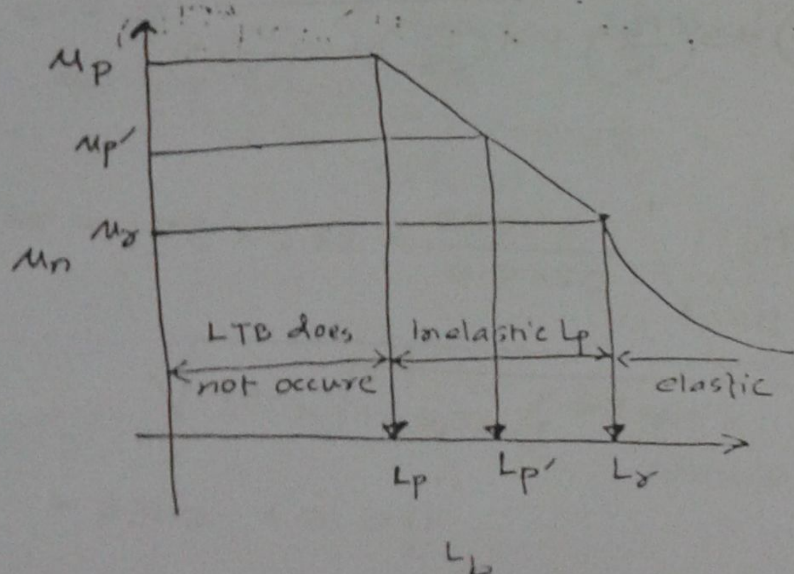
= 1.0 for singly " shapes in single curvature bending

$= 0.5 + 2\left(\frac{I_{yc}}{I_y}\right)^2$ for singly symmetric shapes subjected to reverse curvature bending

I_{yc} = moment of inertia of comp flange about y-axis

= for doubley symmetric shapes $= \frac{I_y}{2}$

– For reverse curvature moment of inertia of smaller.



C_b = moment gradient

$$= \frac{12.5 M_{max}}{2.5 M_{max} + 3M_A + 4M_B + 3M_C} R_m \le 3.0$$

M_{max} = Absolute value of max^m moment in the unbraced seg

M_A = " " of moment @ 1/4 point of " "

M_B = " " of " @ CL " "

M_C = " " " " @ 3/4 " "

R_m = section symmetry factor

= 1.0 for doubly symmetric members (I-shapes)

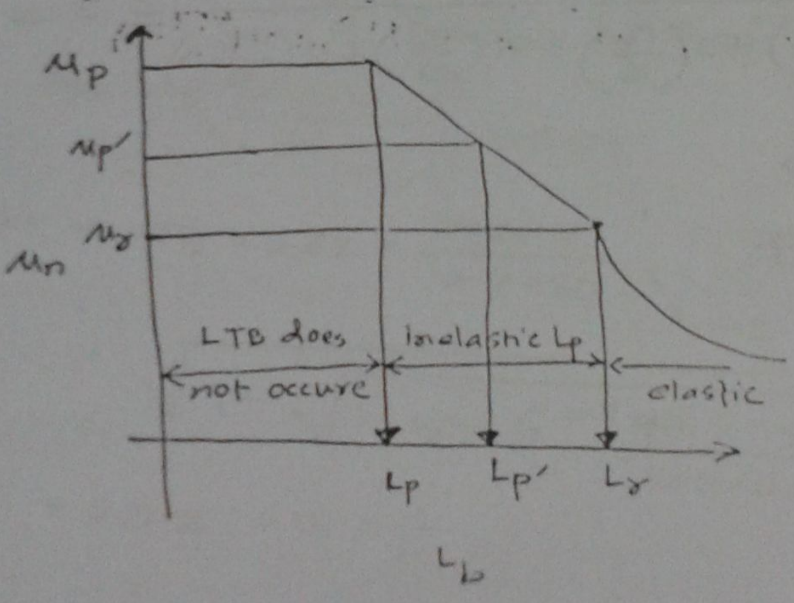
= 1.0 for singly " shapes in single curvature bending

$= 0.5 + 2\left(\frac{I_{yc}}{I_y}\right)^2$ for singly symmetric shapes subjected to reverse curvature bending

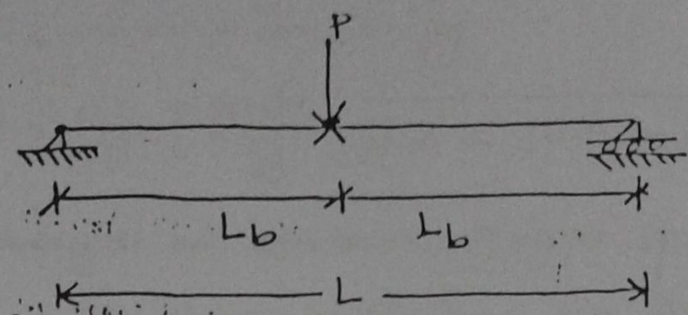
I_{yc} = moment of inertia of comp flange about y-axis

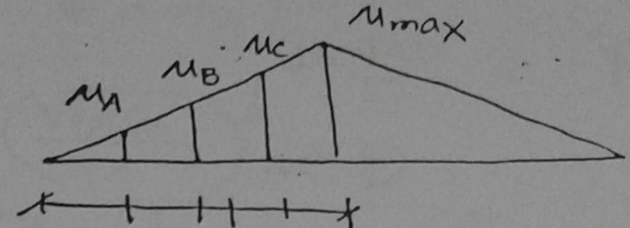
= for doubley symmetric shapes $= \frac{I_y}{2}$

– For reverse curvature moment of inertia of smaller.



8.2 Determine the moment gradient factor





Here $M_{max} = \frac{PL}{4}$

$$M_A = \frac{PL/4}{L/2} \times L_b = \left(\frac{PL}{4} \times \frac{2}{L}\right) * L/8 = \frac{PL}{16}$$

$$M_B = \frac{P}{2} * L/4 = \frac{PL}{8}$$

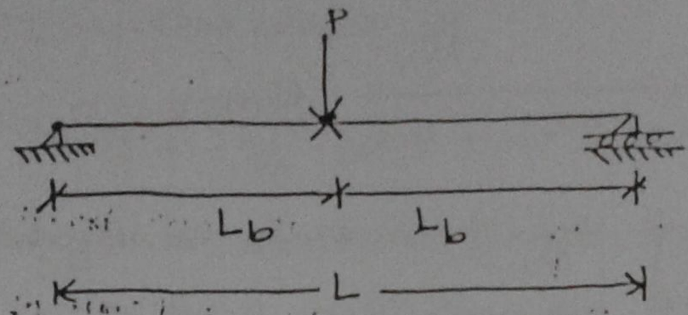
$$M_C = \frac{P}{2} * 3/4 \times \frac{L}{2} = \frac{3PL}{16}$$

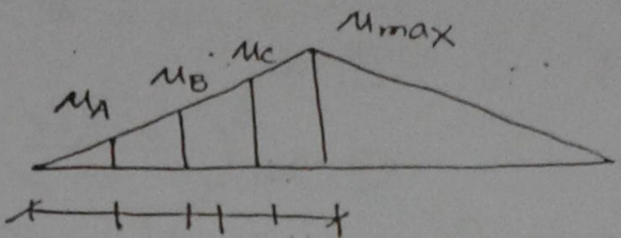
So, $$C_b = \frac{12.5\left(PL/4\right)}{2.5\left(\frac{PL}{4}\right) + 3\left(\frac{PL}{16}\right) + 4\left(\frac{PL}{8}\right) + 3\left(\frac{3PL}{16}\right)} \leq 3.0$$

$= 1.67 < 3.0$

$\therefore C_b = 1.67$

8.2 Determine the moment gradient factor





Here $M_{max} = \frac{PL}{4}$

$$M_A = \frac{PL/4}{L/2} \times L_b = \left(\frac{PL}{4} \times \frac{2}{L}\right) \times L/8 = \frac{PL}{16}$$

$$M_B = \frac{P}{2} \times L/4 = \frac{PL}{8}$$

$$M_C = \frac{P}{2} \times 3/4 \times \frac{L}{2} = \frac{3PL}{16}$$

$$\text{So, } C_b = \frac{12.5\,(PL/4)}{2.5\left(\frac{PL}{4}\right) + 3\left(\frac{PL}{16}\right) + 4\left(\frac{PL}{8}\right) + 3\left(\frac{3PL}{16}\right)} \leq 3.0$$

$$= 1.67 < 3.0$$

$$\therefore C_b = 1.67$$

Determine the design bending strength ϕM_n for a W14×70, $F_y = 50$ Ksi assuming:- (i) Continuous lateral support
(ii) Unbraced length 15 ft, $C_b = 1.0$
(iii) Unbraced length = 15', $C_b = 1.3$

compactness:

flange: $\lambda_f = \frac{b_f}{2t_f} = 6.41$

$\lambda_{pf} = 0.38\sqrt{E/F_y} = 9.15$

So, $\lambda_f < \lambda_{pf} \Rightarrow$ compact

Web: $\lambda_w = h/t_w = 26.4$

$\lambda_{pw} = 3.76\sqrt{E/F_y} = 90.5$

$\lambda_w < \lambda_{pw} \rightarrow$ compact

i) continuous lateral support $L_b = 0$

$M_n = M_p = F_y Z_x = 50 \times 126 = 6300$ K-in $= 525$ K'

$\phi M_n = 0.9 \times 525 = 473$ K'

ii) $L_b = 15'$ and $C_b = 1.0$

$L_p = 1.76 r_y \sqrt{E/F_y} = 1.76(2.48)\sqrt{\frac{29,000}{50}} = 106.1" = 8.76'$

$L_r = 1.95 r_{ts} \frac{E}{0.7F_y}\sqrt{\frac{Jc}{S_x h_o}}\sqrt{1+\sqrt{1+6.76\left(\frac{0.7F_y S_x h_o}{E\,Jc}\right)^2}}$

Here, $r_{ts} = \left(\frac{\sqrt{I_y C_w}}{S_x}\right)^{1/2} = 2.82$

So, $L_r = 1.95 \times 2.82 \times \frac{29000}{0.7 \times 50}\sqrt{\frac{(3.87)(1.0)}{112 \times 13.4}} \times$

$\sqrt{1+\sqrt{1+6.76\left(\frac{0.7 \times 50 \times 112 \times 13.4}{29000 \times 3.87 \times 1}\right)^2}}$

$= 371.2 = 31.0$ ft

So, $L_p < L_b < L_r$

$$M_n = C_b\left[M_p - (M_p - 0.7F_yS_x)\frac{L_b - L_p}{L_r - L_p}\right] \le M_p$$

$$= 1.0\left[6300 - \left(6300 - 0.7\times 50\times 112\right)\left(\frac{15 - 8.76}{31.0 - 8.76}\right)\right] \le 6300$$

$$= 5632 \text{ Kin} = 469 K'$$

$$\phi M_n = 0.9\times 469 = 422 K'$$

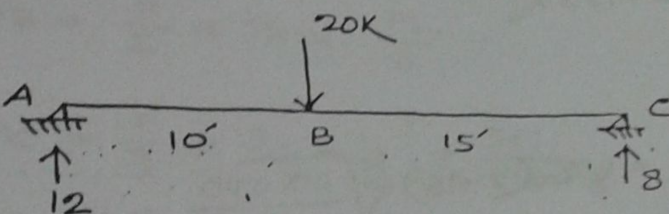
or $C_b = 1.3$

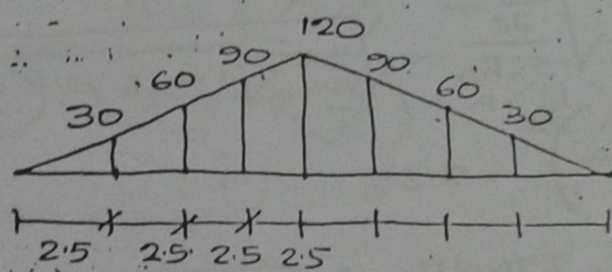
$$M_n = 1.3\times 5632 > 6300$$

$$= 6300 \text{ Kin} = 525 K'$$

$$\phi M_n = 473 K'$$

Calculate: C_b

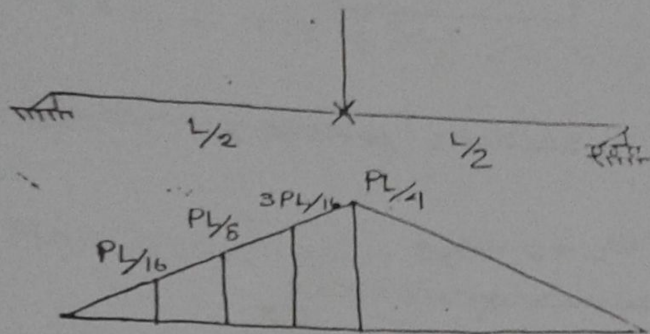




$$\text{So, } (C_b)_{AB} = \frac{12.5\times 120}{2.5\times 120 + 3\times 30 + 3\times 90 + 4\times 60} = 1.67$$

$$(C_b)_{BC} = 1.67$$

etermine : P



Calculation of $C_b = \dfrac{12.5\,(PL/4)}{2.5\,(PL/4) + 3\,\frac{PL}{16} + 4\,\frac{PL}{8} + 3\,\frac{3PL}{16}} = 1.67$

So, $L_p = 1.76\, r_y \sqrt{E/F_y} = 1.76 \times 2.4 \sqrt{\frac{29000}{50}} = 106.4'' = 8.86'$

$L_r = 1.95\, r_{ts} \dfrac{E}{0.7F_y} \sqrt{\dfrac{Jc}{S_x h_o}} \sqrt{1 + \sqrt{1 + 6.76\left(\dfrac{0.7F_y}{E}\dfrac{S_x h_o}{Jc}\right)^2}}$

Here, $r_{ts} = 2.81$; $c = 1.0$

$L_r = 1.95 \times 2.81\, \dfrac{29000}{0.7 \times 50} \sqrt{\dfrac{2.1 \times 1}{78 \times 11.6}} \sqrt{1 + \sqrt{1 + 6.76\left(\dfrac{0.7 \times 50 \times 78 \times 11.6}{29000 \times 2.1 \times 1}\right)^2}}$

$= 358'' = 29.87'$

So, $L_p < L_b < L_r \Rightarrow$ non compact

$M_n = C_b \left[M_p - (M_p - 0.7 F_y S_x) \dfrac{L_b - L_p}{L_r - L_p} \right] \le M_p$

$M_p = F_y Z_x = 50 \times 86.4 = 4320\, k'' = 360 k'$

$M_r = 0.7 F_y S_x = 0.7 \times 50 \times 78 = 2730 k'' = 227.5 k'$

$M_n = 1.67 \left[360 - (360 - 227.5) \dfrac{15 - 8.86}{29.87 - 8.86} \right] \le 360$

$= 536 \le 360$

$\simeq 360 k'$

$M_{allowable} = \dfrac{PL}{4} = \dfrac{P \times 15}{4}$

So, $M_u = \underline{0.9 \times 360 = 324 k'}$

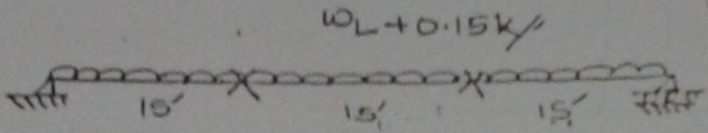
$0.75 \times 360 = 270 k'$

So, $\dfrac{15P}{4} \le 324$ ∴ $P \le 43$

Now, $M_{allow} = 270 k' = \frac{P \times 30}{4}$

So, $\boxed{P \leq 36 K}$

* Determine : Live load , section: w12X53 A992 steel



$\underline{\underline{Sol^n}}$ $L_p = 1.76 r_y \sqrt{E/F_y} = 105'' = 8.75'$

$L_r = 1.95 r_{ts} \frac{E}{0.7 F_y} \sqrt{\frac{Jc}{S_x h_o}} \sqrt{1 + \sqrt{1 + 6.76\left(\frac{0.7 F_y S_x h_o}{E J c}\right)^2}}$

$= 1.95 \times 2.79 \times \frac{29000}{0.7 \times 50} \sqrt{\frac{1.58 \times 1}{70.6 \times 11.5}} \times \sqrt{1 + \sqrt{1 + 6.76\left(\frac{0.7 \times 50 \times 70.6 \times 11.5}{29000 \times 1.58}\right)^2}}$

$= 338'' = 28.2'$

So, $L_p < L_b < L_r$ => non compact

$M_n = C_b \left[M_p - (M_p - 0.7 F_y S_x) \frac{L_b - L_p}{L_r - L_p} \right] \leq M_p$

$C_b = 1.0$ [since max^m moment occure between end moment]

$M_p = F_y Z_x = 3895 K'' = 324 K'$

$0.7 F_y S_x = 2471 K'' = 205.4 K'$

So, $M_n = 1.0 \left[324 - (324 - 205.4) \frac{15 - 8.75}{28.2 - 8.75} \right] \leq 324$

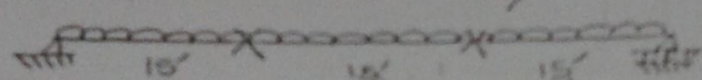
$= 286 K'$

Now, $M_{allow} = 270\,k' = \frac{P \times 30}{4}$

So, $\boxed{P \le 36K}$

* Determine : Live load , section: W12X53 A992 steel

$w_L + 0.15 k/'$



Sol^n $L_p = 1.76\, r_y \sqrt{E/F_y} = 105'' = 8.75'$

$L_r = 1.95\, r_{ts} \frac{E}{0.7F_y} \sqrt{\frac{Jc}{S_x h_o}} \sqrt{1+\sqrt{1+6.76\left(\frac{0.7F_y}{E}\frac{S_x h_o}{Jc}\right)^2}}$

$= 1.95 \times 2.79 \times \frac{29000}{0.7 \times 50} \sqrt{\frac{1.58 \times 1}{70.6 \times 11.5}} \times \sqrt{1+\sqrt{1+6.76\left(\frac{0.7 \times 50 \times 70.6 \times 11.5}{29000 \times 1.58}\right)^2}}$

$= 338'' = 28.2'$

So, $L_p < L_b < L_r \Rightarrow$ non compact

$M_n = C_b\left[M_p - (M_p - 0.7F_y S_x)\frac{L_b - L_p}{L_r - L_p}\right] \le M_p$

$C_b = 1.0$ [since max^m moment occure between end moment]

$M_p = F_y Z_x = 3895\,K'' = 324\,K'$

$0.7 F_y S_x = 2471\,K'' = 205.9\,K'$

So, $M_n = 1.0\left[324 - (324 - 205.9)\frac{15 - 8.75}{28.2 - 8.75}\right] \le 324$

$= 286\,K'$

compact moment for FLB

$$\frac{b_f}{2t_f} = 8.67 < 0.38\sqrt{E/F_y} = 9.15 \Rightarrow \text{compact section}$$

So, $M_n = M_p = 324K'$

$M_n = 286K'$ governs

$$M_{allow} = \frac{286}{1.67} = \frac{\omega L^2}{8}$$

or, $\omega \le 0.68$

So, Live load $= (0.68 - 0.15)\, k/'$

$= \boxed{0.53\, k/'}$

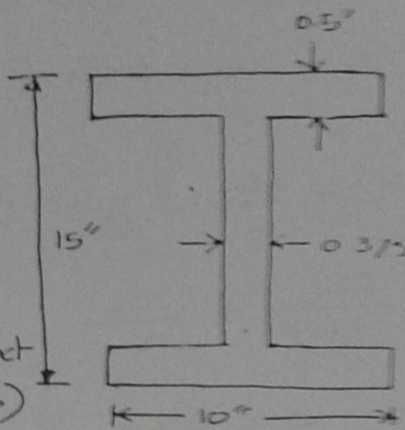
Ans.

Ques 010-11

3(b) Determine: compactness of the section (i) $F_y = 50$ (ii) $F_y = 65$

Soln

(i) $F_y = 50$



$\lambda_{flange} = \frac{b_f}{2t_f} = 10$

$\lambda_p = 0.38\sqrt{E/F_y} = 9.15$

$\lambda_r = \sqrt{E/F_y} = 24.08$

So, $\lambda_p < \lambda < \lambda_r \Rightarrow$ non compact (governs)

$\lambda_w = \frac{15 - 2\times0.5}{0.375} = 37.33$

$\lambda_p = 3.76\sqrt{E/F_y} = 90.54$

$\therefore \lambda_w < \lambda_p \Rightarrow$ compact

The section is non compact @ $F_y = 50$ Ksi

(ii) $F_y = 65$ Ksi

– $\lambda_p = 8.02$, $\lambda_r = 21.12$

So, $\lambda_p < \lambda < \lambda_r \Rightarrow$ non compact (Governs)

– $\lambda_w = 37.33$, $\lambda_p = 79.42$, $\lambda_r = 120.4$

$\therefore \lambda_w < \lambda_p \Rightarrow$ non compact

So, the section will be non compact

(a) compute : M_y, M_p, S assume $F_y = 50$ Ksi

Here, $M_y = F_y S_x$

$S_x = \frac{I_x}{c} = \frac{716.71}{7.5}$ [I_x calculated from 1(a)]

$= 95.56$ in^3.

$\therefore M_y = 50 \times 95.56 = 4778$ K-in $= \boxed{398.2\ K'}$

- M_p calculation

$M_p = F_y Z_x$

$Z_x = (0.5 \times 10 * 7.25 + 7 \times 0.375 \times 3.5) * 2 = 90.88$

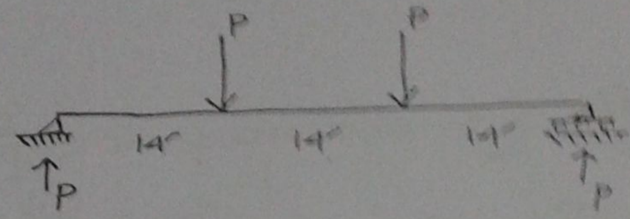
$\therefore M_p = 50 \times 90.88 = 4543.75 = 378.6K'$

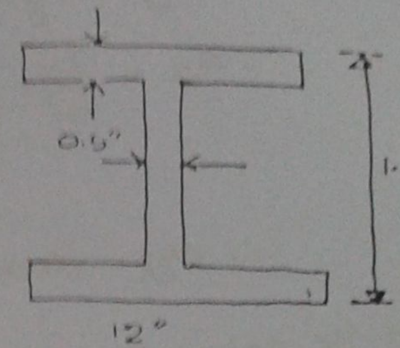
- Shape factor : $= \frac{M_p}{M_y}$

A(b) Compute : C_b

Ques. 09-10.

3(b) calculate = P' Assume, A 572 steel





Soln $L_b = 14'$

$L_p = 1.76\, r_y \sqrt{E/F_y}$

and, $L_r = 1.95\, r_{ts} \dfrac{E}{0.7F_y} \sqrt{\dfrac{Jc}{S_x h_o}} \sqrt{1 + \sqrt{1 + 6.76\left(\dfrac{0.7F_y S_x h_o}{E Jc}\right)^2}}$

Here, $I_y = 2\times\dfrac{12^3 \times 0.5}{12} + \dfrac{0.5^3 \times 13}{12} = 144.135$, $A = 12\times0.5\times2 + 13\times0.5$
$= 18.5$

so, $r_y = \sqrt{I_y/A} = 2.79''$

$\therefore L_p = 1.76 \times 2.79 \sqrt{\dfrac{29000}{50}} = 118.3'' = 9.85'$

and, $r_{ts} = \left(\dfrac{\sqrt{I_y C_w}}{S_x}\right)^{\frac{1}{2}}$ here, $I_x = 638.54\,in^4$

$= \left(\dfrac{\sqrt{144.135 \times 13.5}}{91.22}\right)^{\frac{1}{2}}$ $S_x = \dfrac{I_x}{c} = \dfrac{638.54}{7} = 91.22$

$= 0.695$ $C_w = 14 - 0.25 - 0.25 = 13.5$

$c = 1.0$ [rolled section]

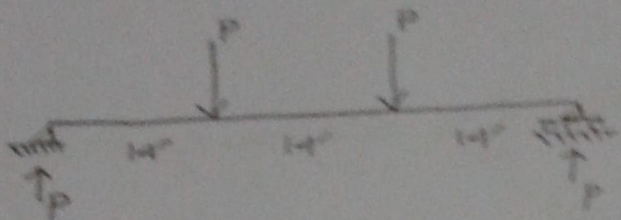
For, $t_f = t_w = t$ $J = \dfrac{2b t_f^3 + h_o t_w^3}{3} = \dfrac{2\times12\times0.5^3 + 13.5\times0.5^3}{3} = 1.562$

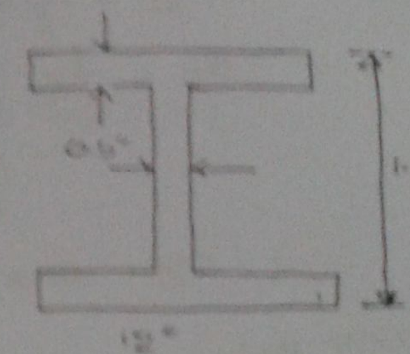
and, $C_w = \dfrac{t_f h_o^2 b^3}{24} = \dfrac{0.5\times13.5^2\times12^3}{24} = 6561$

so, $r_{ts} = \left(\dfrac{\sqrt{I_y C_w}}{I_x/c}\right)^{\frac{1}{2}} = \left(\dfrac{\sqrt{144.135\times6561}}{638.54/7}\right)^{\frac{1}{2}} = 3.27''$

Ques: 08-10

3(b) calculate 'P' Assume, A 572 steel





Soln $L_b = 14'$

$L_p = 1.76\, r_y \sqrt{E/F_y}$

and, $L_r = 1.95\, r_{ts} \dfrac{E}{0.7F_y} \sqrt{\dfrac{Jc}{S_x h_o}} \sqrt{1+\sqrt{1+6.76\left(\dfrac{0.7F_y S_x h_o}{E\, Jc}\right)^2}}$

Here, $I_y = 2\times\dfrac{12^3\times 0.5}{12} + \dfrac{0.5^3\times 13}{12} = 144.135$, $A = 12\times 0.5\times 2 + 13\times 0.5 = 18.5$

So, $r_y = \sqrt{I_y/A} = 2.79"$

$\therefore L_p = 1.76\times 2.79\sqrt{\dfrac{29000}{50}} = 118.3" = 9.86'$

and, $r_{ts} = \left(\dfrac{\sqrt{I_y C_w}}{S_x}\right)^{\frac{1}{2}}$ here, $I_y = 638.54\, in^4$

$= \left(\dfrac{\sqrt{144.135\times 13.5}}{91.22}\right)^{\frac{1}{2}}$ $S_x = \dfrac{I_x}{c} = \dfrac{638.54}{7} = 91.22$

$= 0.695$ $C_w = 14 - 0.25 - 0.25 = 13.5$

$c = 1.0$ [rolled section]

For, $t_f = t_w = t$ $J = \dfrac{2bt_f^3 + h_o t_w^3}{3} = \dfrac{2\times 12\times 0.5^3 + 13.5\times 0.5^3}{3} = 1.5625$

and, $C_w = \dfrac{t_f h_o^2 b^3}{24} = \dfrac{0.5\times 13.5^2\times 12^3}{24} = 6561$

So, $r_{ts} = \left(\dfrac{\sqrt{I_y C_w}}{I_x/c}\right)^{\frac{1}{2}} = \left(\dfrac{\sqrt{144.135\times 6561}}{638.54/7}\right)^{\frac{1}{2}} = 3.27"$

$$L_r = 1.95 \times 3.27 \frac{29000}{0.7 \times 50} \sqrt{\frac{1.5625 \times 1}{91.22 \times 13.5}} \sqrt{1 + \sqrt{1 + 6.76\left(\frac{43101.5}{45312.5}\right)^2}}$$

$= 360.42''$

$= 30'$

So, $L_p < L_b < L_r \Rightarrow$ non compact

Now, $M_n = C_b\left[M_p - (M_p - 0.7 F_y S_x)\frac{L_b - L_p}{L_r - L_p}\right] \leq M_p$

$C_b = 1.0 \quad M_p = F_y Z_x$

$Z_x = (0.5 \times 12 \times 6.75 + 6.5 \times 0.5 \times 3.25) \times 2 = 102.125$

So, $M_p = 50 \times 102.125 = 5106$ K-in $= 425.5$K'

$0.7 F_y S_x = 0.7 \times 50 \times 91.2 = 3192.7$K'' $= 266$K'

So, $M_n = \left[-(425.5 - 266)\frac{14 - 9.85}{30 - 9.85} + 425.5\right] \leq 425.5$

$= 392.65 < 425.5$K'

$= 392.65$K'

$M_{allowable} = \frac{392.65}{1.67} = 235.12$K'

Now, $14P \leq 235.12$

So, $\boxed{P \leq 16.8K}$

$L_r = 1.95 \times 3.27 \dfrac{29000}{0.7 \times 50} \sqrt{\dfrac{1.5625 \times 1}{91.22 \times 13.5}} \sqrt{1 + \sqrt{1 + 6.76\left(\dfrac{43101.5}{45312.5}\right)^2}}$

$= 360.42''$

$= 30'$

So, $L_p < L_b < L_r \Rightarrow$ non compact

Now, $M_n = C_b\left[M_p - (M_p - 0.7 F_y S_x)\dfrac{L_b - L_p}{L_r - L_p}\right] \le M_p$

$C_b = 1.0 \quad M_p = F_y Z_x$

$Z_x = (0.5 \times 12 \times 6.75 + 6.5 \times 0.5 \times 3.25) \times 2 = 102.125$

So, $M_p = 50 \times 102.125 = 5106$ K-in $= 425.5K'$

$0.7 F_y S_x = 0.7 \times 50 \times 91.2 = 3192.7K'' = 266K'$

So, $M_n = \left[-(425.5 - 266)\dfrac{14 - 9.85}{30 - 9.85} + 425.5\right] \le 425.5$

$= 392.65 < 425.5K'$

$= 392.65K'$

$M_{allowable} = \dfrac{392.65}{1.67} = 235.12K'$

Now, $14P \le 235.12$

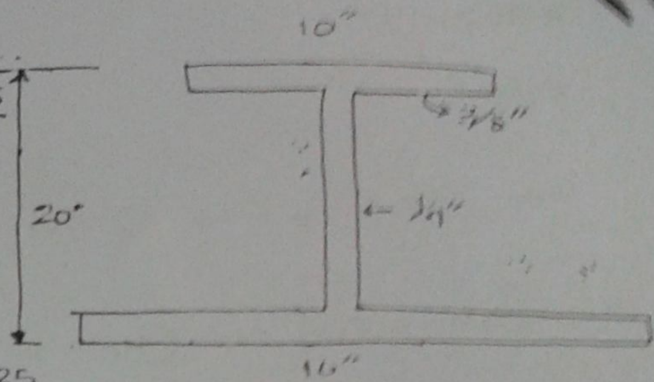
So, $\boxed{P \le 16.8K}$

1(a) Find: (i) elastic modulas, Z_x (ii) S_x (iii) M_y (iv) M_p, (v) ... A & G ...

Z_x

$$\bar{y} = \frac{16\times 3/8 * 3/16 + 1/4 \times 19.25 \times 10 + 10 \times 3/8 \times 19.8125}{16\times 3/8 + 1/4 \times 19.25 + 10\times 3/8}$$

$$= 8.48''$$



So, $Z_x = 10\times 3/8 \times (11.52 - 3/16) + 11.145 \times 0.25 \times 5.5725 + 8.105 \times 0.25 \times 4.0525 + 16 \times 3/8 \times 8.3 = 116.03$

S_x $= \dfrac{I_x}{c}$

$$I_x = (3/8)^3 \times 16 \times \frac{1}{12} + 3/8 \times 16 \times \left(8.48 - 3/16\right)^2 + \frac{19.25^3 \times 1/4}{12} + 19.25 \times \frac{1}{4} \times (10 - 8.48)^2$$
$$+ (3/8)^3 \times 10 \times \frac{1}{12} + 3/8 \times 10 \left(19.8125 - 8.48\right)^2 = 1054.03$$

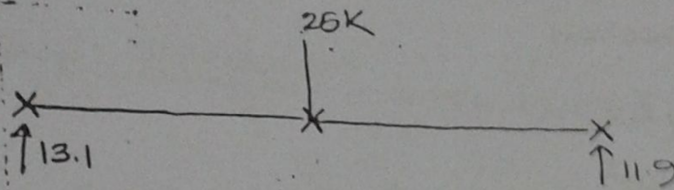
So, $S_x = \dfrac{1054.3}{(20 - 8.48)} = 91.5$

M_y : $M_y = F_y S_x = 36 \times 91.5 = 3293.8\ K'' = 274.5K'$

M_p : $M_p = F_y Z_x = 4177.08\ K'' = 348K'$

Shape factor : $\dfrac{M_p}{M_y} = \dfrac{348}{274.5} = 1.27$

select lightest sex from Annexure-02 Use A572 steel



let the section as a compact section

So, $M_n = M_p = F_y Z_x$

Here, $M_n = 262 \text{ K-ft} = 50 \times Z_x$

So, $Z_x = 62.88$

Select W16X16.

compactness test

$\lambda_f = 8.1$ and, $\lambda_w = 53.8$

and, $L_p = 1.76\, r_y \sqrt{E/F_y} = 27.12'' = 2.26'$

$$L_r = 1.95\, r_{ts} \frac{E}{0.7F_y} \sqrt{\frac{Jc}{S_x h_o}} \sqrt{1 + \sqrt{1 + 6.76\left(\frac{0.7F_y S_x h_o}{E J c}\right)^2}}$$

$= 1.95$

Ques 08-09

3(c) select: lightest section of W10,
span 20' and $w = 2.5$ K/ft. A 572 grade 65 steel

Assume a compact section

So, $M_n = M_p = F_y Z_x$

Here, $M_u = \dfrac{2.5 \times 20^2}{8} = 125$ k-ft

So, $\dfrac{F_y Z_x}{1.67} = 125$ ∴ $Z_x = 3.21 \times 12 = 38.5''$

Select W 10×33

Compactness test

$\lambda_f = 9.1$ and $\lambda_w = 33.6$

Now, for flange

$\lambda_p = 0.38\sqrt{\dfrac{29000}{65}}$

$= 8.03$

$\lambda_r = \sqrt{\dfrac{29000}{65}}$

$= 21.12$

for web:

$3.76\sqrt{E/F_y} = 79.42$

$\lambda < \lambda_p \Rightarrow$ compact

So, $\lambda_p < \lambda < \lambda_r \Rightarrow$ non compact (governs)

$M_n = M_p - (M_p - 0.7 F_y S_x)\dfrac{\lambda - \lambda_p}{\lambda_r - \lambda_p} \leq M_p$

$M_p = F_y Z_x = 65 \times 38.8 = 210$ K'

$0.7 F_y S_x = 0.7 \times 65 \times 35 = 132$ K'

So, $M_n = 210 - (210 - 132)\dfrac{9.1 - 8.03}{21.12 - 8.03} \leq 210$

$= 203.6$ K'

So, $M_{all} = \dfrac{203.62}{1.67} = 121.93K'$

Here, $w = (2.5 + \underset{SW}{0.033})$ k/ft

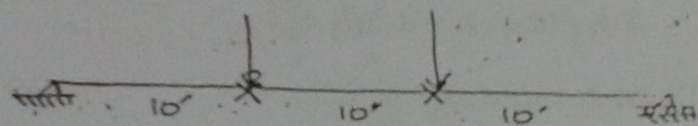
$M_{ult} = \dfrac{2.533 \times 20^2}{8} = 126.65K'$ (within 2~4%)

Select section W10X33

Ans.

4(b) Calculate : P

Span: 30', Section : W10X112, Steel : A36



Soln : Here, $L_b = 10'$

$L_p = 1.76\, r_y \sqrt{E/F_y} = 1.76 \times 2.68 \times \sqrt{\dfrac{29000}{36}} = 133.87 = 11.15$

$L_r = 1.95\, r_{ts} \dfrac{E}{0.7F_y} \sqrt{\dfrac{JC}{S_x h_o}} \sqrt{1 + \sqrt{1 + 6.76\left(\dfrac{0.7F_y}{E}\dfrac{S_x h_o}{JC}\right)^2}}$

$r_{ts} = \left(\dfrac{\sqrt{I_y C_w}}{S_x}\right)^{\frac{1}{2}} = \left(\dfrac{\sqrt{236 \times 6022.88}}{126}\right)^{1/2}$

$= 3.076''$

$C_w = \dfrac{t_f h_o^2 b^3}{24}$

$= \dfrac{1.25 \times (11.36 - 1.25)^2 \times 1}{24}$

$= 6022.88$

$J = 15.1\ in^4$, $h_o = 10.11$

So, $L_r = 88.96'$.

So, $L_b < L_p \Rightarrow$ compact

So, $M_n = M_p = F_y Z_x = 36 \times 147 = 5292 = 441K'$

$M_{ult} = \dfrac{441}{1.67} = 264K'$

Now, $10P = 264K'$ So, $\boxed{P = 26.4K}$ Ans ues-07-08

(c) Select : Lightest W12 section
$w = 0.75 k/ft$ and $LL = 1 k/'$, span = 25'.
compression flange fully supported against lateral movement
A36 steel

Soln $M_u = \frac{(1+0.75) \times 25^2}{8} = 136.72 K' =$

assume a compact section

so, $M_n = M_p = F_y Z_x$

$\therefore \frac{M_p}{\Omega} = \frac{F_y Z_x}{1.67} = 136.72K'$

so, $Z_x = 6.34 \times 12 = 76.11''$

Select W12X53 section

Here,

For flange,

$\lambda_f = 8.7$

$\lambda_p = 0.38 \left(\frac{E}{F_y}\right)^{\frac{1}{2}} = 10.78$

So, $\lambda_f < \lambda_p \Rightarrow$ compact

For web,

$\lambda_w = 35$

$\lambda_p = 3.76\sqrt{E/F_y} = 106.72$

So, $\lambda_w < \lambda_p \Rightarrow$ compact

Now, $M_n = M_p = F_y Z_x \leq 1.5 F_y S_x$

Here, $F_y Z_x = 36 \times 77.9 < 1.5 \times 36 \times 70.5$

$= 2804.1 < 3807$

$= 233.7 K'$

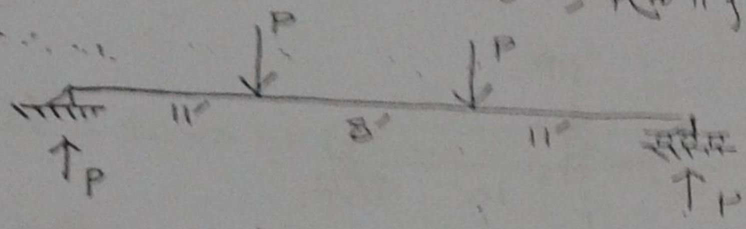
So, $M_u = \frac{233.7}{1.67} = 139.91 K' > 136.72 K'$

(ok)

Select the section W12X53

Calculate : P

Span = 30' , section W12X120 , K@ 11 ft from each suppor



Now, $L_p = 1.76\, r_y \sqrt{E/F_y} = 1.76 \times 3.13 \sqrt{\frac{29000}{36}} = 156.35'' = 13.0'$

So, $L_b < L_p$ => compact

$\therefore M_n = M_p = F_y Z_x = 36 \times 185 = 6660\ K'' = 555K'$

Now, $11P = \frac{555}{1.67}$

$\therefore \boxed{P = 30.2K}$ Ans.

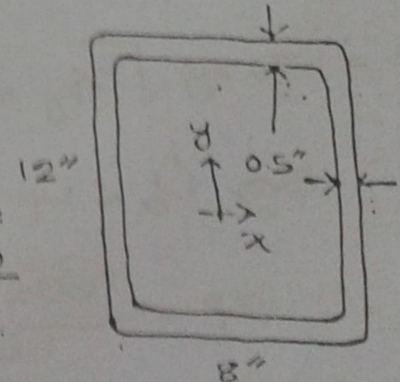
7(c) Calculate: shape factor:

$I_x = \frac{12^3 \times 8}{12} - \frac{11^3 \times 7}{12} = 375.583\ in^4$

$S_x = \frac{I_x}{c} = \frac{375.583}{6} = 62.6$

and, $Z_x = [7 \times 0.5 \times 5.75 + 2 \times 6 \times 0.5 \times 3]2$

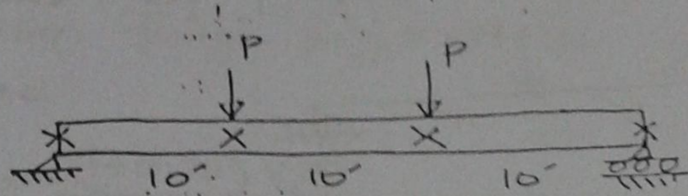
$= 76.25$



shape factor $= \frac{Z_x}{S_x} = \boxed{1.2181}$ Ans es 06-07

) Calculate : P

Span = 30', section W27×178, A572 Grade 36 steel



Soln

$L_b = 10'$

$L_p = 1.76\, r_y \sqrt{E/F_y} = 1.76 \times 3.26 \sqrt{\frac{29000}{50}} = 162.85 = 13.6'$

So, $L_b < L_p \Rightarrow$ compact section

$\therefore M_n = M_p = F_y Z_x = 50 \times 567 = 28350\, k'' = 2362.5\, k'$

Now, $10P = \frac{2362.5}{1.67}$

$\therefore \boxed{P = 141.5k}$ Ans

(b) calculate P, span 30', section W27×178, A572 Grade 50

$\therefore L_p = 1.76\, r_y \sqrt{E/F_y} = 1.76 \times 3.26 \sqrt{\frac{29000}{50}} = 138.2'' = 11.51$

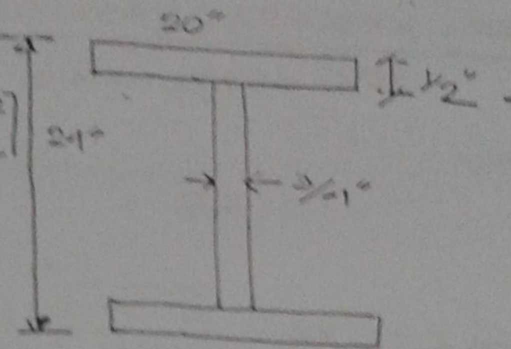
So, $L_b < L_p \Rightarrow$ compact section

$\therefore 10P = \frac{50 \times 567}{1.67}$

$\therefore P = 141.5K$ Ans

4(b) Determine : shape factor

Here, $S_x = \frac{I_x}{c}$



$I_x = 2\left[\frac{20 \times (0.5)^3}{12} + 20 \times 0.5 \times (12-0.25)^2\right]$

$+ \frac{23^3 \times 3/4}{12} = 3522.1$

So, $S_x = \frac{3522.1}{12} = 293.51$

$Z_x = \left[20 \times \frac{1}{2} \times (12-0.25) + 11.5 \times \frac{3}{4} \times \frac{11.5}{2}\right] 2 = 334.1875''$

Shape factor $= \frac{334.1875''}{293.51} = \boxed{1.139}$

Ans

4(c) check :- adequacy of the section W21×44 , $F_y = 50$ Ksi

$w = LL + DL = (2+0.5) k/' = 2.5 k/'$ (let including self wt)

Span = 21'

Here, $M_u = \frac{2.5 \times 21^2}{8} = 180 K'$

Here, $\lambda_f = 7.2$

For flange,

$\lambda_p = 0.38\sqrt{E/F_y} = 9.15$

So, $\lambda_f < \lambda_p \Rightarrow$ compact

$\lambda_w = h/t_w = \frac{19.31}{0.35} = 55.17$

For web,

$\lambda_p = 3.76\sqrt{E/F_y} = 90.55$

$\therefore \lambda_w < \lambda_p \Rightarrow$ compact

$\therefore M_n = M_p = F_y Z_x = 50 \times 95.4 = 4770 K'' = 397.5 K'$

$M_u = \frac{397.5}{1.67} = 238 K' > 180 K'$ (ok)

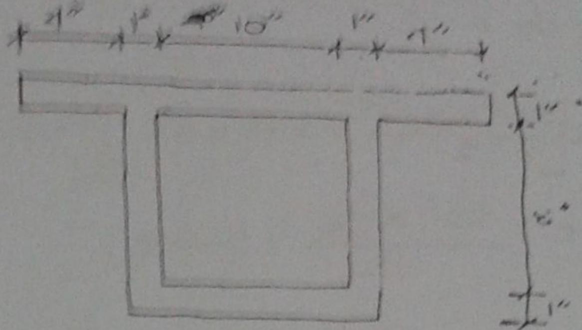
Section is adequate

Ques 08-06

5(b) Determine : shape factor

$$\bar{y} = \frac{20\times1\times9.5 + 9\times12\times4.5 - 8\times10\times5}{20\times1 + 9\times12 - 8\times10}$$

$= 5.75''$



$$I_x = \frac{1^3\times20}{12} + 20\times1\times(9.5-5.75)^2 + \frac{9^3\times12}{12} + 9\times12\times(5.75-4.5)^2$$
$$- \frac{8^3\times10}{12} - 8\times10\,(5-5.75)^2 = 709$$

$$\therefore S_x = \frac{I_x}{c} = \frac{709}{5.75} = 123.3$$

and, $Z_x = 20\times1\times3.75 + 3.25\times12\times3.25\times0.5 -$
$3.25\times10\times3.25\times0.5 + 5.75\times12\times5.75\times0.5 -$
$4.75\times10\times4.75\times0.5 = 171.125$

So, shape factor $= \dfrac{Z_x}{S_x} = 1.388$

Ans

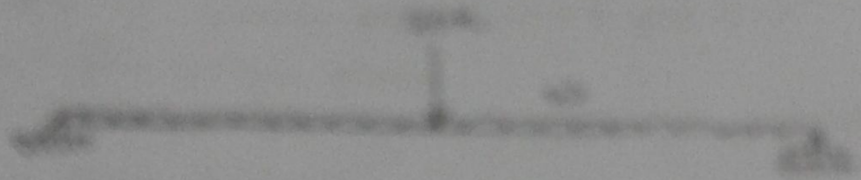
5(c) Prove : shape factor of rectangle 1.5

$$S_x = \frac{I_x}{c} = \frac{bh^3}{12\times h/2} = \frac{bh^2}{6}$$

$$Z_x = \left(b\times\frac{h}{2}\times h/4\right)2 = \frac{bh^2}{4}$$

$$S = \frac{bh^2/4}{bh^2/6}$$

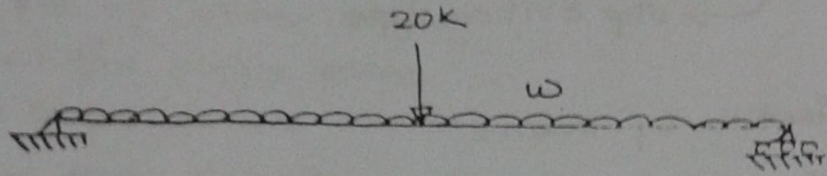
$= 6/4 = 3/2 = 1.5$ (Proved)



calculate : w

Span = 40′ section W 18×106 , P = 20k @ mid span

A36 steel.



Here, $L_p = 1.76\, r_y \sqrt{E/F_y} = 1.76 \times 2.66 \sqrt{\frac{29000}{36}} = 132.8 = 11.07'$

$L_r = 1.96\, r_{ts} \frac{E}{0.7F_y} \sqrt{\frac{Jc}{S_x h_o}} \sqrt{1 + \sqrt{1 + 6.76\left(\frac{0.7F_y S_x h_o}{Jc\,E}\right)^2}}$

$r_{ts} = \left(\frac{\sqrt{I_y C_w}}{S_x}\right)^{\frac{1}{2}}$

$C_w = \frac{t_f h_o^2 b^3}{24}$

Simple connection in tension member:

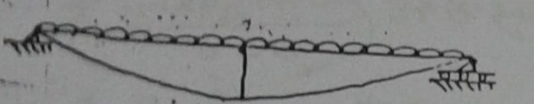
— Bolt connection → Bearing type
— Bolt connection → Slip critical type

— Max^m allowable deflection of Beam:

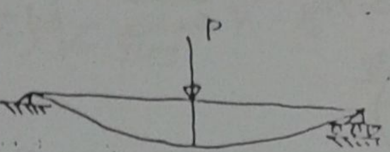
$\Delta_a = L/360$ for live load only

$\Delta_a = L/240$ for live and dead load

L = span of the beam.



$\Delta_{max} = \frac{5}{384}\frac{\omega L^4}{EI}$



$\Delta = \frac{PL^3}{48EI}$

if $\Delta_{max} > \Delta_a$ → increase I

Revise I → $\Delta_{max} = \Delta_a$

$\therefore I_{min} = ?$

— For design of the Beam

1. FLB
2. WLB
3. LTB
4. Deflection
5. Shear Check

Shear check:

Nominal shear $V_n = 0.6 F_y A_w$ [A_w = web area considering full dept

when $h/t_w \leq 2.24\sqrt{E/F_y}$

Ω for $V_n = 1.5$

No flange conderation. shear is zero @ top and bottom. max^m @ N g type connection

Allowable shear strength, $\frac{R_n}{\Omega} = \frac{F_{nv}\, m\, A_b}{\Omega}$

where, F_{nv} = nominal shear strength of bolt

m = no. of shear plane

= 1 for single plane

= 2 for double shear

A_b = Area of bolt

A 325 bolt → shear strength when threads are included = 48

shear " " " " excluded = 60 Ksi

A490 bolt → " " " " included = 60 "

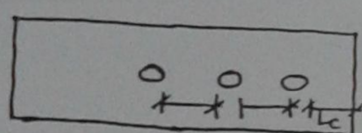
" " " " excluded = 75 Ksi

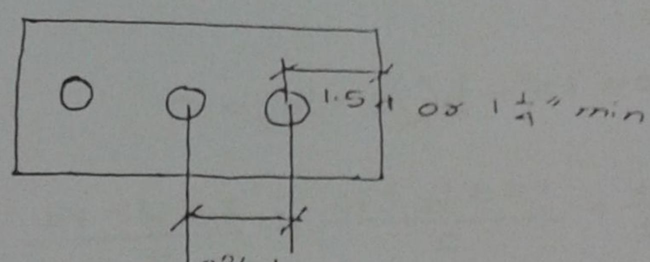
Allowable bearing strength, $\frac{R_n}{\Omega} = \frac{1.2 L_c t F_u}{2} \le \frac{2.4 d t F_u}{\Omega}$

where, t = Plate thickness

F_u = ultimate strength

L_c = clear distance btn the edge one hole and adjacent





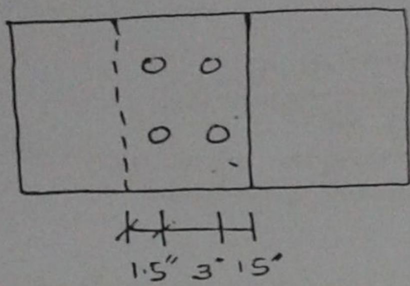
2⅔d bul-

3d permeable

dia of bolt

Example:

plate $\frac{5}{8} \times 6$



$\frac{7}{8}$" A325 bolt in std. hole
Bearing type connection with threads excluded from shear
plate A572 Grade 50

– investigate adequacy of the connection if the total applied load 75k

Soln

Bolt shear check

$$\frac{R_n}{\Omega} = \frac{F_{nv}\, m\, A_b}{\Omega} = \frac{60 \times 1 * \pi/4\,(7/8)^2}{2} = 18^k/\text{bolt} \quad \left[\begin{array}{l} F_{nv} = 60\,\text{ksi} \\ m = 1 \end{array} \right.$$

For 4 bolts, shear strength = $4 \times 18 = 72K < 75K$ (diff 4%)
not acceptable

Bearing strength of plate

equal thickness → take any plate
unequal " → take thinner one

Edge hole:

$$l_c = 1.5'' - \frac{1}{2}\left(\frac{7}{8} + \frac{1}{8}\right) = 1''$$

Now, $$\frac{R_n}{\Omega} = \frac{1.2 * L_c\, t\, F_u}{2} \le \frac{2.4\, d\, t\, F_u}{2}$$

$$= \frac{1.2 \times 1 \times 5/8 \times 65}{2} \le \frac{2.4 \times 7/8 \times 5/8 \times 65}{2}$$

$$= 24.37\,K \le 42.65K$$

$= 24.37K$/bolt or hole

...r hole:

$$L_c = 3 - \left(\tfrac{7}{8} + \tfrac{1}{8}\right) = 2''$$

So, $\dfrac{R_n}{\Omega} = \dfrac{1.2 \times 2 \times 5/8 \times 65}{2} \leq \dfrac{2.4 \times 7/8 \times 5/8 \times 65}{2}$

$= 48.74K \leq 42.65K$

$\simeq 42.65K/\text{bolt}$

So, Total bearing $2 \times 24.37 + 2 \times 42.65 = 134K > 75K$ (OK)

3. check the adequacy of member

Based on yielding $\dfrac{F_y A_g}{\Omega} = \dfrac{50 \times 5/8 \times 6}{1.67} = 112K > 75K$

Based on fracture, $\dfrac{F_u A_e}{\Omega} = \dfrac{65 \times 1 \times (5/8 \times 6 - 2 \times 1 \times 7/8)}{2} = 81 > 75$

*** correction**

...r hole:

$L_c = 3 - \left(\frac{7}{8} + \frac{1}{8}\right) = 2''$

So, $\frac{R_n}{\Omega} = \frac{1.2 \times 2 \times 5/8 \times 65}{2} \leq \frac{2.4 \times 7/8 \times 5/8 \times 65}{2}$

$= 48.74K \leq 42.65K$

$\simeq 42.65K/\text{bolt}$

So, Total bearing $2 \times 24.37 + 2 \times 42.65 = 134K > 75K$ (OK)

3. Check the adequacy of member

Based on yielding $\frac{F_y A_g}{\Omega} = \frac{50 \times 5/8 \times 6}{1.67} = 112K > 75K$

Based on fracture, $\frac{F_{cr} A_e}{\Omega} = \frac{65 \times 1 \times (5/8 \times 6 - 2 \times 1 \times 5/8)}{2} = 81 > 75$

*** Correction**