LECTURE NOTES

on

STRUCTURAL DESIGN-II

For Diploma in Civil Engineering

By

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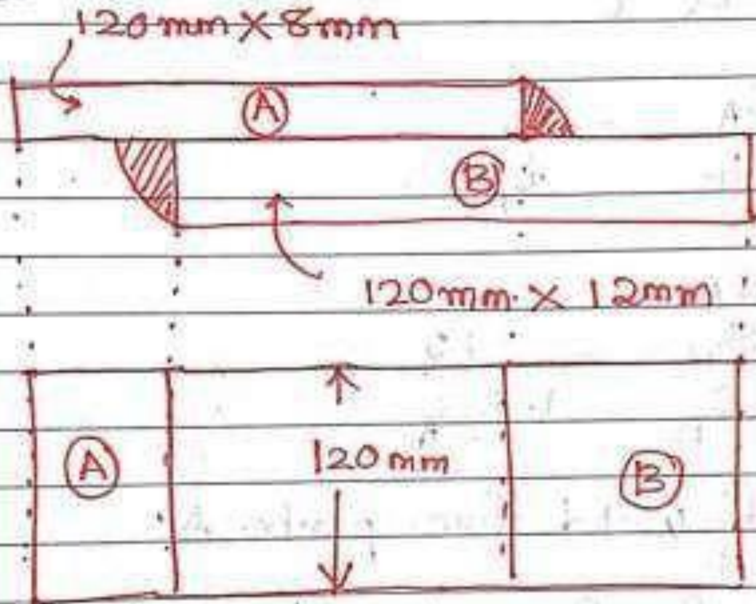
SD-II (Numericals)

(1) Design a welded lap joint for max^m efficiency shop welding & Fe410 grade steel

$\gamma = 1.25$



→ For max^m efficiency the joint will be designed for full strength of the thinner plate i.e 120 mm X 8 mm

- strength of thinner plate

$$F = \frac{A_g f_y}{\gamma_{m0}}$$

$$= \frac{(120 \times 8) \times 250}{1.1}$$

$= 218.18KN$ = weld strength

∵ width of plates same, the weld size $S \propto$ thickn of plates

$$S \propto t$$

$$\Rightarrow \frac{S_A}{S_B} = \frac{t_A}{t_B}$$

$$\Rightarrow \frac{S_A}{S_B} = \frac{8}{12}$$

$$\Rightarrow S_B = 1.5 S_A$$

- Strength of weld fore plate A

throat thickn

Area of weld A $= (L_w)_A (t_w)_A$, $(t_w)_A = 0.7 S_A$

Stress $= \frac{f_u/\sqrt{3}}{\gamma_{mw}}$

Strength = Area × Stress

$$= (L_w)_A (t_w)_A \times \frac{f_u/\sqrt{3}}{\gamma_{mw}}$$

$$= 0.7 (L_w)_A S_A \times \frac{f_u/\sqrt{3}}{\gamma_{mw}}$$

$$\boxed{\begin{array}{l} * \ (L_w)_A = 120 - 2S_A \\ * \ (L_w)_B = 120 - 2S_B \end{array}}$$

- Strength of weld for plate B

$$Area = (L_w)_B (t_w)_B$$

$$= (L_w)_B (0.7 S_B)$$

$$Stress = \frac{f_u/\sqrt{3}}{\gamma_{mw}}$$

$$Strength = Area \times Stress$$

$$= (L_w)_B \times 0.7 S_B \times \frac{f_u/\sqrt{3}}{\gamma_{mw}}$$

Weld strength = strength for plate A + strength for plate B

$$= \left\{ (L_w)_A \cdot S_A + (L_w)_B \cdot S_B \right\} \frac{0.7 f_u/\sqrt{3}}{\gamma_{mw}}$$

$$= \left\{ (120 - 2S_A) S_A + (120 - 2S_B) S_B \right\} \times \frac{0.7 \times 410}{1.25 \times \sqrt{3}}$$

$$= 132.56 \left(120 S_A - 2 S_A^2 + 120 S_B - 2 S_B^2 \right)$$

$$= 132.56 \left(120 S_A - 2 S_A^2 + 120 \times 1.5 S_A - 2 \times 1.5^2 S_A^2 \right)$$

$$= 132.56 \left(300 S_A - 6.5 S_A^2 \right)$$

$218.18 \times 10^3 = 132.56\,(360 S_A - 6.5 S_A^2)$

$\Rightarrow 1645.9 = 360 S_A - 6.5 S_A^2$

$\Rightarrow 6.5 S_A^2 - 360 S_A + 1645.9 = 0$

$\Rightarrow S_A = 39.8\ \&\ 6.4\ mm$

rejected

$S_B = 1.5 S_A = 9.6\,mm$

Check.

$Min^3 = 3\,mm$

For 8mm plate; Max^m weld size $= 8 - 1.5 = 6.5m$

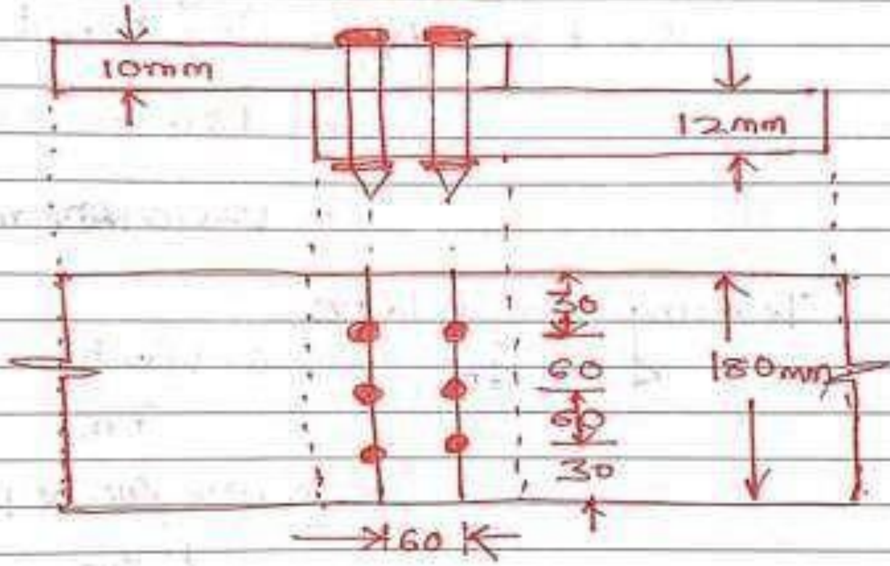
For 12mm plate;

Min^m weld size $= 5mm$

Max^m weld size $= 12 - 1.5 = 10.5mm$

Hence provide 9.5mm weld for plate A & 6.5mm weld for plate B.

(2) Find the max^m force that can be transmitted through a double bolted chain lap joint 6 bolts in 2 rows (M16, 4.6 grade), Fe410 plate. Thickness of plates connected are 10mm & 12mm



- M16 bolts & 4.6 grade bolt

$d = 16\ mm$

$d_0 = 16 + 2 = 18\ mm$

4.6 grade bolt $\Rightarrow$ $f_{yb} = 240\ MPa$

& $f_{ub} = 400\ MPa$

– Strength of plate :

Thickness of thinner plate = 10mm
width of plate $b = 180$ mm
No. of bolts in critical section = 3
Net area at weakest
section $A_n = (b - nd_o)t$

$= (180 - 3\times 18)\times 10$

$= 1260$ mm^2

Tearing of plate
$T_{dn} = \frac{0.9 f_u A_n}{\gamma_{ml}}$

$= \frac{0.9\times 410\times 1260}{1.25}$

$= 371.952$ kN

– Shearing strength of bolt :

For 1 bolt, $n_n = 1$ & $n_s = 0$ (for lap joint & single cover butt joint)

Total $n_n = 1\times 6 = 6$
$n_s = 0\times 6 = 0$

$(A_{nb}) = 0.78 \frac{\pi}{4} d^2 = 0.78\times \frac{\pi}{4}\times 16^2 = 156.8$ mm^2

Nominal sheare strength $(V_{ns})_b = \frac{f_{ub}}{\sqrt{3}}\left[n_n A_{nb} + n_s A_{sb}\right]$

$= \frac{400}{\sqrt{3}}\left[6 \times 156.8 + 0\right]$

$= 217.268 \text{KN}$

Design sheare streength $(V_{ds})_b = \frac{(V_{ns})_b}{\gamma_{mb}}$

$= \frac{217.268}{1.25}$

$= 173.8 \text{ KN}$

– Bearing strength of bolt :

k_b = Min^m of
(a) $\frac{e}{3d_o} = \frac{30}{3 \times 18} = 0.56$
(b) $\frac{p}{3d_o} - 0.25 = 0.861$
(c) $\frac{f_{ub}}{f_u} = \frac{400}{410} = 0.98$
(d) 1

$\Rightarrow k_b = 0.56$

Nominal beare strength $V_{npb} = 2.5 k_b d t f_u$

$= 2.5 \times 0.56 \times 16 \times 10 \times 410$

$= 91.84 \text{kN}$

Design beare strength $(V_{DP})_b = \frac{(V_{np})_b}{\gamma_{m2}}$

$= \frac{91.84}{1.25}$

$= 73.5\ kN$

For 6nos of bolt $(V_{DP})_b = 6 \times 73.5 = 440\ kN$

Strength of joint = min^n of
(a) Shearing of bolt = 173.8 kN
(b) Bearing of bolt = 440 kN
(c) Tearing of plate = 371.952 kN

∴ Strength of joint = 173.8 kN ie the max^m force that can be transmitted

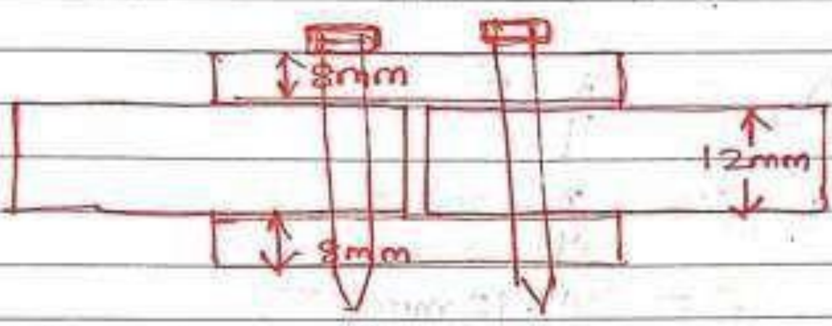
* Efficiency of joint $\eta = \frac{\text{Strength of joint}}{\text{Strength of solid plate}} \times 100$

$= \frac{173.8\ kN}{T_{dg}\ 409.09} = \frac{173.8}{\frac{A_g f_y}{\gamma_{m0}}}$ ← Gross area of thinner plate.

$= \frac{173.8\ kN}{\frac{1800 \times 250}{1.1}}$

$= 42.5\%$

③ Calculate the strength of a 20mm diameter bolt of greade 4.6 for double cover butt joint. Cover plates 8mm thick & main plates are 12mm thick.



for 1 bolt $\begin{cases} n_n = 1 \\ n_s = 1 \end{cases}$ (for double cover butt joint)

- Shearing of bolts :— 2 bolts

$d = 20mm \therefore d_0 = 22mm$

4.6 grade ⇒ $f_{ub} = 400$ & $f_{yb} = 240$ MPa

Nominal Shear strength $= \dfrac{f_{ub}}{\sqrt{3}}(n_n \cdot A_{nb} + n_s A_{sb})$

$$= \frac{400}{\sqrt{3}}(2 \times A_{nb} + 2A_{sb})$$

$$= \frac{800}{\sqrt{3}}(A_{nb} + A_{sb})$$

$$A_{nb} = 0.78 \frac{\pi}{4} d^2$$

$$= 0.78 \times \frac{\pi}{4} \times 20^2$$

$$= 245 \text{ mm}^2$$

$$A_{sb} = \frac{\pi}{4} d^2$$

$$= \frac{\pi}{4} \times 20^2$$

$$= 314.16 \text{ mm}^2$$

Nominal shear $= \frac{800}{\sqrt{3}} (245 + 314.16)$

$$(V_{ns})_b = 258.26 \text{ kN}$$

Design shear $= (V_{ds})_b = \frac{(V_{ns})_b}{\gamma_{mL}}$

$$= \frac{258.26}{1.25}$$

$$= 206.61 \text{ kN}$$

- Bearing of bolts m

$k_b =$ min of
(a) $\frac{e}{3d_0} = \frac{33}{3 \times 22} = 0.5$
(b) $\frac{p}{3d_0} - 0.25 = 0.51$
(c) $\frac{f_{ub}}{f_u} = 0.97$
(d) 1

* $\boxed{e = 1.5 d_o}$

$= 1.5 \times 22$

$= 33mm$

Assume pitch $p = 50mm$

$K_b = 0.5$

Nominal bearing strength $(V_{nP})_b = 2.5 K_b d t f_u$

$= 2.5 \times 0.5 \times 20 \times 12 \times 410$

$= 123 KN$

Design bearing strength $= \frac{(V_{nP})_b}{\gamma_{mL}}$

$= \frac{123}{1.25}$

$= \cancel{96} KN$ 98.4

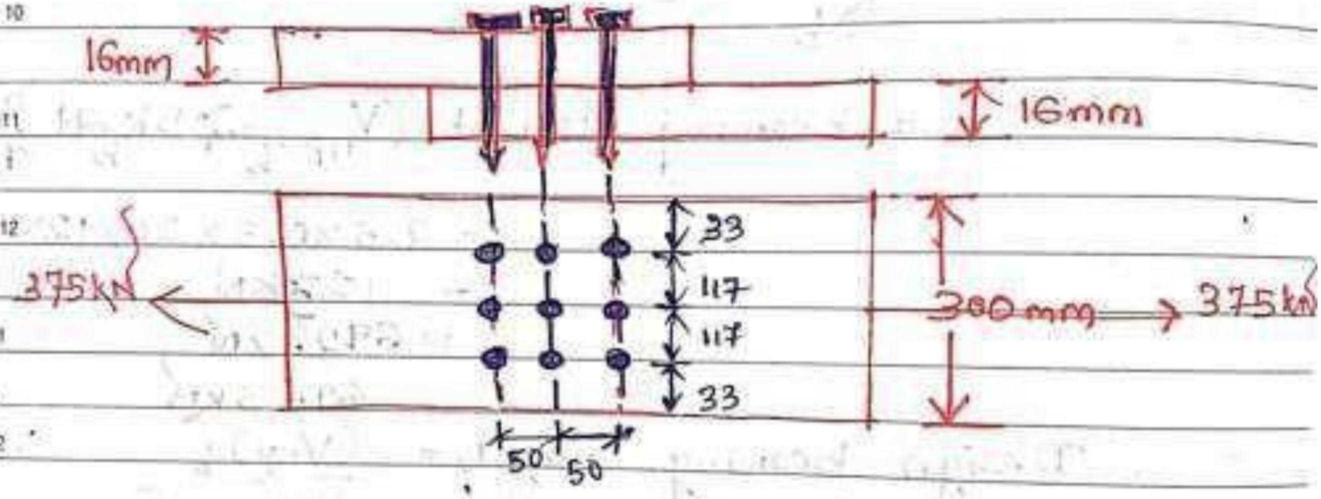
For 2 bolts $(V_{dP})_b = 2 \times 96 KN$

$= \cancel{192} kN$ 196.8

Strength of bolt $= \min \begin{cases} \text{(a) Shearing} = 206.6 KN \\ \text{(b) bearing} = 196.8 KN = 197 kN \end{cases}$

$= 197 kN$

④ Design a lap joint to connect two plates 300mm wide & 16mm thick using 20mm dia bolts of grade 4.6. The applied service Load is 375kN.



$d = 20mm$

$d_o = 22mm$

Shearing of bolts ; $n_n = 1, \; n_s = 0$

$$\text{Nominal } (V_{ns})_b = \frac{f_{ub}}{\sqrt{3}}\left(n_n A_{nb} + \cancel{n_s A_{sb}}^{\,0}\right)$$

$$= \frac{f_{ub}}{\sqrt{3}}\left(0.78\frac{\pi}{4} \times 20^2 \times 1\right)$$

$$= \frac{400 \times 245}{\sqrt{3}}$$

$$= 56.58 \text{ kN}$$

Design $(V_{ds})_b = \dfrac{(V_{ns})_b}{\gamma_{mb}}$

$= \dfrac{56.58}{1.25}$

$= 45.26$ KN

No. of bolts required $= \dfrac{375 kN}{45.26 kN}$

$= 8.28$

$= 9$ nos

~~[illegible]~~

$e = 1.5 d_o = 1.5 \times 22 = 33 mm$

Assume, $p = 50 mm$

$$K_b = \min^m \text{ of } \begin{cases} \text{(i) } \dfrac{e}{3d_o} = 0.5 \\ \text{(ii) } \dfrac{p}{3d_o} - 0.25 = 0.51 \\ \text{(iii) } \dfrac{f_{ub}}{f_u} = \dfrac{400}{410} = 0.9 \\ \text{(iv) } 1 \end{cases}$$

$K_b = 0.5$

Nominal bear $(V_{np})_b = 2.5 K_b \, d . t . f_{ub}$

$= 2.5 \times 0.5 \times 20 \times 16 \times 41$

$= 164$ kN

design bear $(V_{dp})_b = \dfrac{(V_{np})_b}{\gamma_{mb}}$

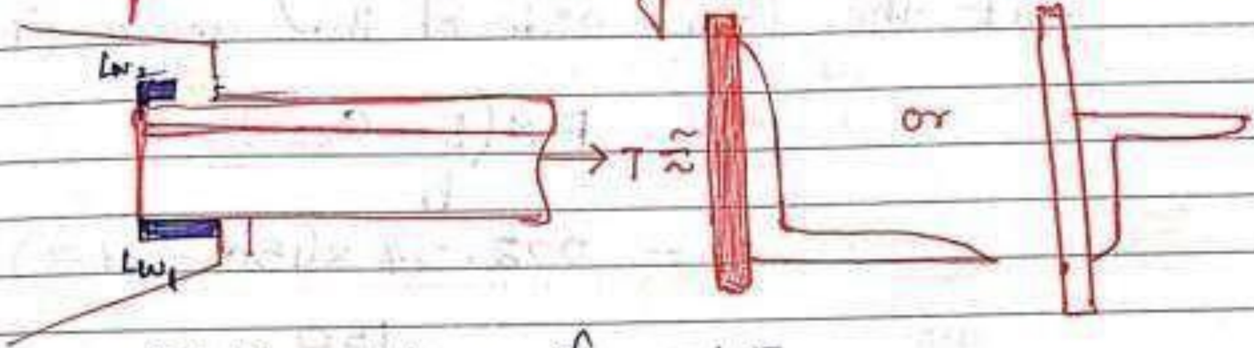
$= \dfrac{164 KN}{1.25}$

$= 131.2 KN$

For 9 no. of bolts = 1180 KN > 45.26 KN

(Design is Safe)

(5) An angle sect^n 150 mm × 115 mm × 12 mm is to be connected to a 12mm thick gusset plate at site. Design the weld to carry a load equal to the strength of the member.



Field welding $\gamma_{mw} = 1.5$

From steel table, ISA 150 × 115 × 12

$\Leftrightarrow$ $A_g = 978 mm^2$

$C_{zz} = 27.3 mm$

Design strength of plate (thinner)

$$T_{dn} = \frac{A_g f_y}{\gamma_{m0}}$$

$$= \frac{978 \times 250}{1.1}$$

$$= 222.272 \text{ kN}$$

				1	2	
4	5	6	7	8	9	
11	12	13	14	15	16	
18	19	20	21	22	23	
25	26	27	28	29	30	31

*) Therefore, the weld will be designed to transmit a force equal to 222.27kN = F

The force to be resisted by the weld at the lower side of the angle (P_1)

$$P_1 = \frac{F \times (l - C_{zz})}{l}$$

$$= \frac{222.27 \times (150 - 27.3)}{150}$$

$$= 181.81 kN$$

The force to be resisted by the weld at upper side of the angle P_2

$$P_2 = \frac{F \times C_{zz}}{l}$$

$$= \frac{222.27 \times 27.3}{150}$$

$$= 40.45 kN$$

Assume ; size of weld $S = 6mm$

Throat thickness $(t_w) = 0.7S$

$= 4.2mm$

Length Lw_1 & Lw_2

$$Area = (L_w)(t_w)$$

$$Stress = \frac{f_u/\sqrt{3}}{\gamma_{mw}}$$

$$P_{w_1} = Strength = (L_w)_1(t_w) \times \frac{f_u/\sqrt{3}}{\gamma_{mw}}$$

$$= Lw_1 \times 4.2 \times \frac{410/\sqrt{3}}{1.5}$$

$$= 662.79\, Lw_1 = P_1$$

$$P_{w_2} = (Lw)_2 t_w \frac{f_u/\sqrt{3}}{\gamma_{mw}}$$

$$= Lw_2 \times 4.2 \times \frac{410/\sqrt{3}}{1.5}$$

$$= 662.79\, Lw_1 = P_2$$

$\therefore$ $662.79\, Lw_1 = P_1$

$\Rightarrow 662.79 \times Lw_1 = 181.81 \times 10^3$

$\Rightarrow Lw_1 = 274.3\ mm = 275mm$

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M	T	W	T	F	S	S
				1	2	3
4	5	6	7	8	9	10
11	12	13	14	15	16	17
18	19	20	21	22	23	24
25	26	27	28	29	30	31

$$662.79\, L_{w_2} = P_2$$

8 $$\Rightarrow 662.79\, L_{w_2} = 40.45 \times 10^3$$

9 $$\Rightarrow L_{w_2} = 61\,mm$$

10

11

12

1

2

3

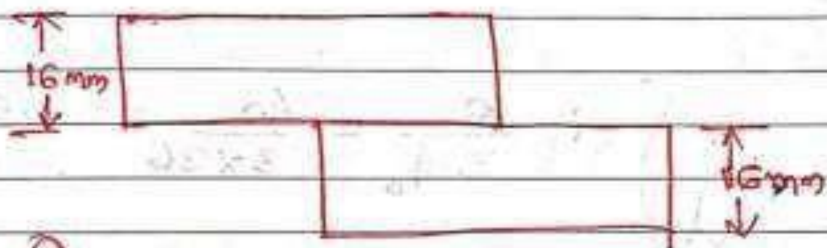
4

5

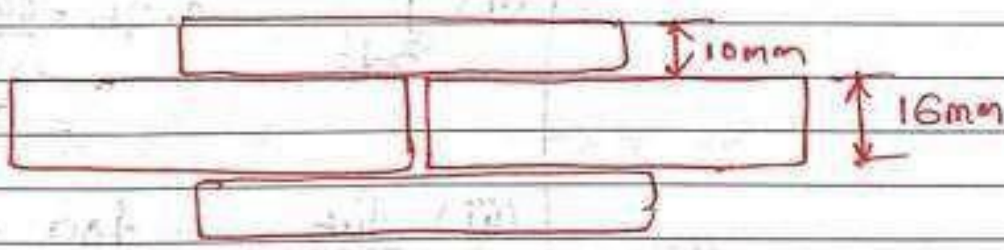
6

Two steel plates of Fe 410 grade 16mm thick are to be joined by 24mm dia bolts of 4.6 grade. Assume a pitch of 60mm & edge distance of 40mm. Calculate strength of bolt (i) Lap joint
(ii) double cover butt joint with 10mm thick cover

(i)



(ii)



Given: $d = 24mm$ $\quad p = 60mm$ $\quad f_{ub} = 400$

$d_0 = 26mm$ $\quad e = 40mm$ $\quad f_u = 410$

Case-I

Shearing:

$$n_n = 1, \quad n_s = 0$$

$$A_n = 0.78 \frac{\pi}{4} d^2 \qquad A_s = \frac{\pi}{4} d^2$$

$$= 353$$

$$\text{Nominal sheare} = \frac{f_{ub}}{\sqrt{3}} \left(n_n \cdot A_n + \underbrace{\cancel{n_s} \cdot A_s}_{0} \right)$$

$$= \frac{400}{\sqrt{3}} (1 \times 353) = 81.521 kN$$

M	T	W	T	F	S	S
				1	2	3
4	5	6	7	8	9	10
11	12	13	14	15	16	17
18	19	20	21	22	23	24
25	26	27	28	29	30	31

design sheare $= \dfrac{\text{Nominal Sheare}}{1.25}$

$= \dfrac{81.521}{1.25}$

$= 65.21$ KN

Bearing

$k_b = \min^n$ of
- (i) $\dfrac{e}{3d_o} = \dfrac{40}{3\times26} = 0.513$
- (ii) $\dfrac{p}{3d_o} - 0.25 = \dfrac{60}{3\times26} - 0.25 = 0.52$
- (iii) $\dfrac{f_{ub}}{f_u} = \dfrac{400}{410} \simeq 0.9$
- (iv) 1

nominal bear $(V_{np})_b = 2.5 k_b d \cdot t \cdot f_u$

$= 2.5\times0.513\times24 \times 16\times 410$

$= 201.92$ KN

Design beare $(V_{dp})_b = \dfrac{(V_{dp})_b}{\gamma_{ml}}$

$= \dfrac{201.92}{1.25}$

$= 161.53$ kN

$\therefore$ Strength of bolt $= 65.21$ kN

Case - II) Double cover butt joint

For 1 bolt $\begin{cases} n_n = 1 \ \& \ n_s = 1 \\ A_n = 0.78 \frac{\pi}{4} d^2, \ A_s = \frac{\pi}{4} d^2 \end{cases}$

Min^m 2 bolts

$= 353 \text{ mm}^2 \qquad = 452 \text{ mm}^2$

Nominal Shear $(V_{ns})_b = \dfrac{f_{ub}}{\sqrt{3}} (n_n A_n + n_s A_s)$

$= \dfrac{400}{\sqrt{3}} (2 \times 353 + 2 \times 452)$

$= 185.9$ kN $\times 2 = 371.8$

Design shear $(V_{ds})_b = \dfrac{(V_{ns})_b}{1.25} = 297$ kN

Per 1 bolt design shear $=$ 148.7 kN

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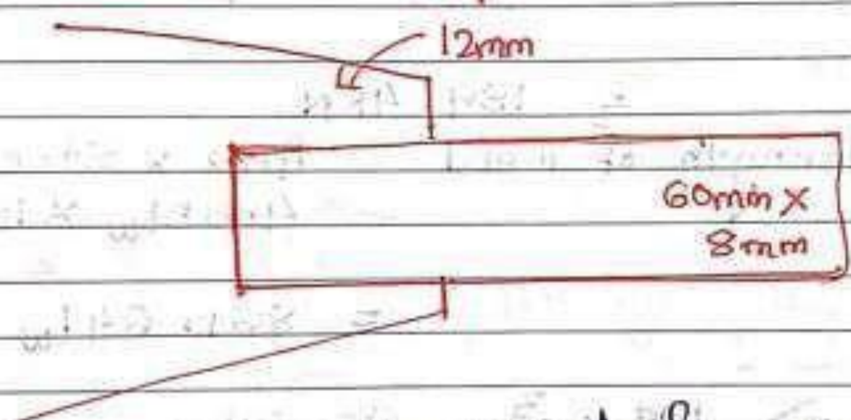
bearing will be same per bolt
= 161.53 kN

Strength of bolt = 148.7 kN

Design a suitable fillet weld to connect a tie bar 60mm x 8mm to a 12mm thick gusset plate so as to develop maximum force of
(i) Shop welding is done on 2 sides
(ii) field welding is done on 3 sides.



Given: Strength of weld $= \frac{A_g f_y}{\gamma_{mo}} = \frac{(60 \times 8) \times 250}{1.1} = 109\text{ kN}$

Case - I ; Shop welding $\gamma_{mw} = 1.25$

plate thickness $= 8mm$

→ Min^m weld size $= 3mm$

→ Max^m weld size $= 8 - 1.5 = 6.5mm$

$\boxed{\text{Adopt}}$ size of weld $S = 6.5$

$t_w = K \cdot S$
$= 0.7 \times 6.5$
$= 4.55$

L_w

8
9
10
11
12
1
2
3
4
5
6

Area $= l_w \cdot t_w = 4.55\, l_w$

Stress $= \dfrac{f_u/\sqrt{3}}{\gamma_{mw}}$

$= \dfrac{410/\sqrt{3}}{1.25}$

$= 189.4\,KN$

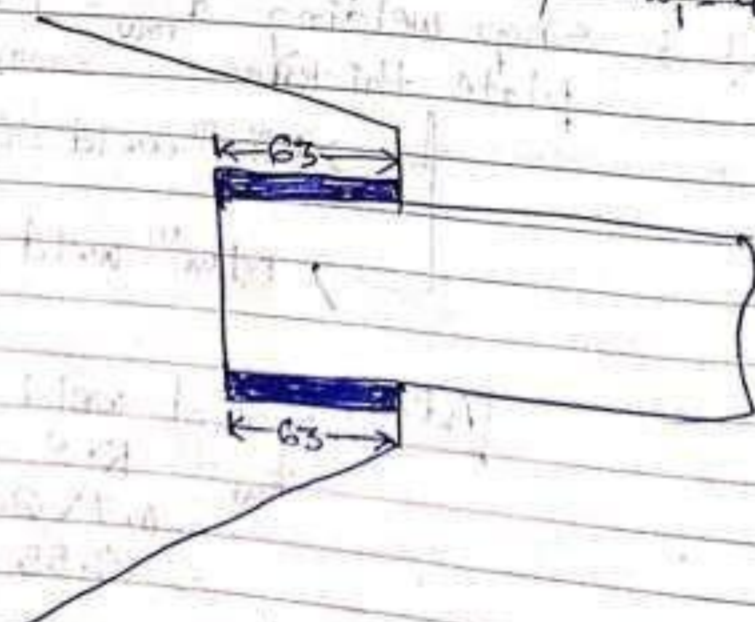
Strength of weld = Area × Stress

$= 4.55 l_w \times 189.4$

$= 861.64\, l_w$

$\Rightarrow 109 \times 10^3 = 861.64 l_w$

$\Rightarrow l_w = 126.6\,mm \quad l_{w_1} = l_{w_2} = 63\,mm$



Case - II field welding $\gamma_{mw} = 1.5$

Strength of weld = Area × stress

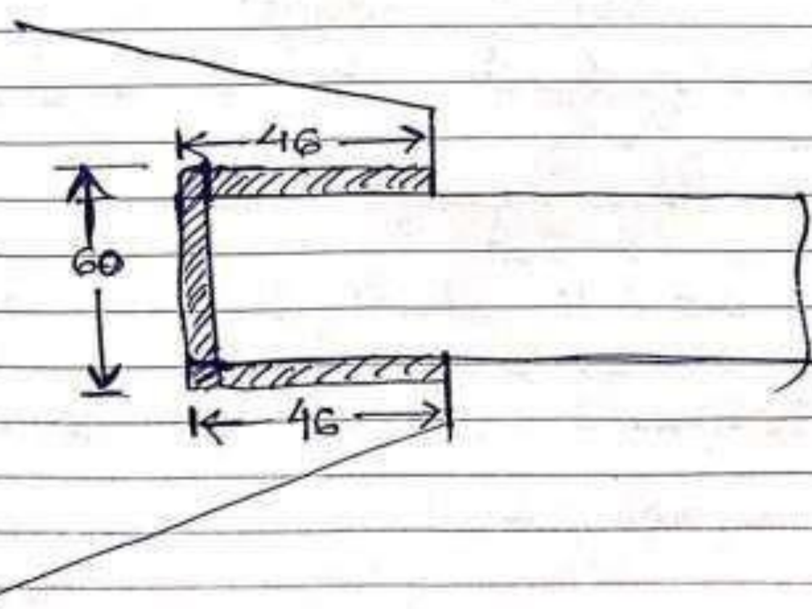
$= L_w \times 4.55 \times \frac{f_u/\sqrt{3}}{1.5}$

$= 718 L_w$

$\Rightarrow 109 \times 10^3 = 718 L_w$

$\Rightarrow L_w = 151.8 \text{ mm}$ $L_{w_1} + 60 + L_{w_2} = 152$

$\Rightarrow L_{w_1} = L_{w_2} = 46 \text{mm}$



(1) A tension member belongs to resist a service D load of 20kN & a service live load of 50kN, Design a rectangular bar of standard stru steel of grade Fe410. Assume that the member is connected by one line of 16mm dia bolts of grade 4.6.

Given : Service load = 20KN D.L

L.L = 50 KN

Partial Safety factor for load = 1.5

$\gamma_{mb} = 1.25$

Fe410 $f_u = 410$, $f_y = 250$

$d = 16$

$d_0 = 16 + 2 = 18\,mm$

Grade 4.6 $\Rightarrow f_{ub} = 400\,N/mm^2$

$f_{yb} = 240\,N/mm^2$

Design factored Load = 1.5 DL + 1.5LL

$= 1.5 \times 20 + 1.5 \times 50$

$= 105\,kN$

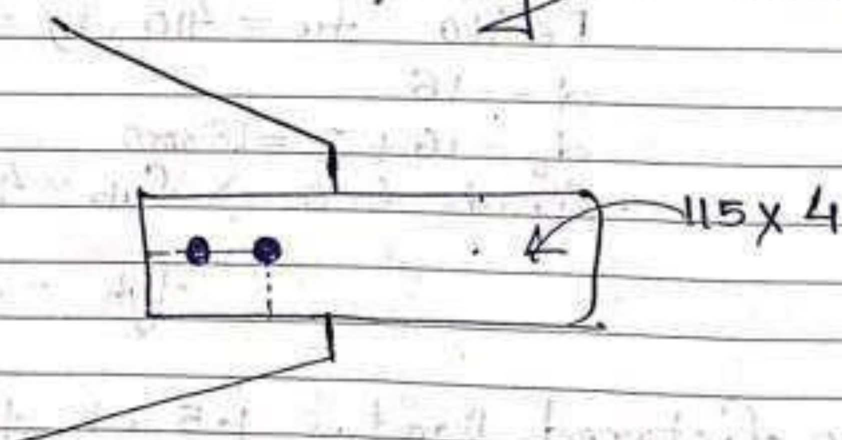
Tension member failure

Yielding Rupture block shear

Yielding strength $T_{dg} = \frac{A_g \times f_y}{\gamma_{mo}}$

$\Rightarrow 105 \times 10^3 = A_g \times \frac{250}{1.1}$

$\Rightarrow A_g \Rightarrow 462 mm^2$



Rupture strength $T_{dn} = 0.9 A_n \frac{f_u}{\gamma_{m1}}$

$\Rightarrow 105 \times 10^3 = 0.9 A_n \frac{410}{1.25}$

$\Rightarrow A_n = 355.69 mm^2$

Assume; thickness of bar = 4mm

$$\text{width of bar} = \frac{A_g}{4\,mm} = \frac{462}{4} = 115\,mm$$

Check: $A_n = (b - nd_o)t$

$$= (115 - 1 \times 18)\, 4\,mm^2$$

$$= 388\,mm^2 > 355.69\,mm^2 \text{ (Safe)}$$

Effective slenderness ratio $\boxed{\lambda = \frac{l_{eff}}{r}}$ λ_{max} 180 DL+LL, 350 WL+EL, 400 Tensile Load

$$r = \sqrt{\frac{I}{A}}$$

$$= \sqrt{\frac{bt^3/12}{bt}}$$

$$= \sqrt{\frac{(115 \times 4^3)/12}{115 \times 4}}$$

$$= 1.15\,mm$$

$$l_{eff} = 0.8\,m = 800\,mm$$

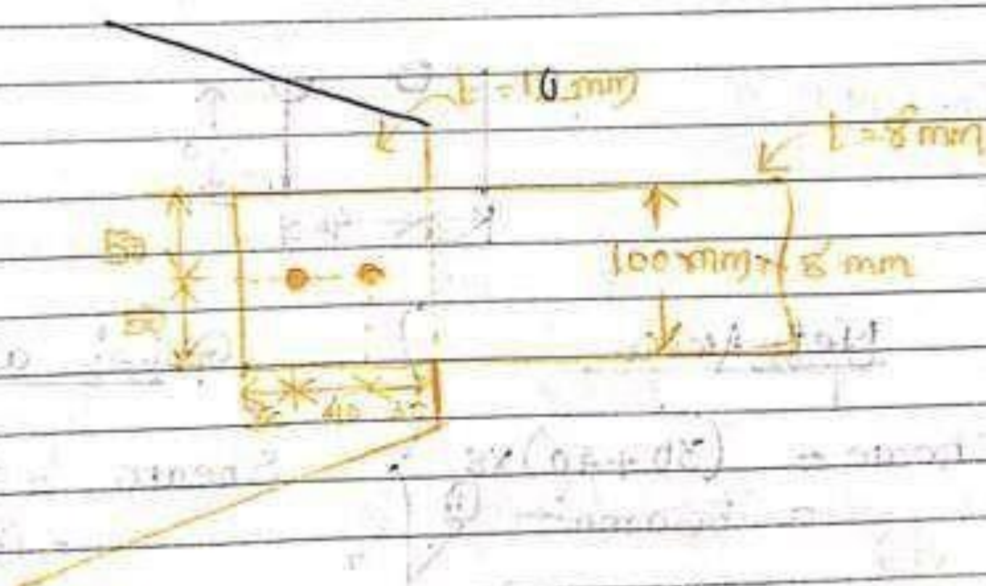
$$\lambda = \frac{800}{1.15} = 695.65 < \lambda_{max} = 180$$

(Not safe)

$\therefore$ Redesign

A tension member consists of a flat 100 mm x 8 mm which is connected to a gusset plate of 10mm thick by 2 nos of 16 mm dia bolts as shown in fig. Determine the strength of flat. Also determine the max^m load the joint can carry safely. Fe410 grade steel & Grade 4.6 bolts.

Given



Yielding strength $= \dfrac{A_g f_y}{\gamma_{m0}}$

$= \dfrac{(100 \times 8) \times 250}{1.1}$

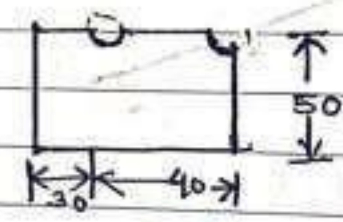
$= 181.81$ KN

Rupture strength $= \frac{0.9 A_n f_u}{\gamma_{m1}}$, $A_n = (b - n$

$= 0.9 \times (100 - 1 \times 18)$ 8×410 / 1.25

$= 193.65$ kN

Block Shear Strength



Gross Area

Shear $= (30 + 40) \times 8$
$= 560 \text{ mm}^2$ ①

tension $= 50 \times 8$ ②
$= 400 \text{ mm}^2$

Net area

Shear $= (70 - 1.5 \times d_o)$
$= (70 - 1.5 \times 18)\, 8$
$= 344 \text{ mm}^2$

Tension $= (50 - 0.5 \times 18)$
$= 328 \text{ mm}^2$

$(\text{Block Strength})_1 = \frac{(\text{Gross Shear}) f_y}{\sqrt{3}\, \gamma_{m0}} + \frac{0.9 (\text{Net tension}) f_u}{\gamma_{m1}}$

$=$

$= 163218.29\ N$

$$= \frac{(560) \times 250}{\sqrt{3} \times 1.1} + 0.9 \times (328) \times \frac{410}{1.25}$$

$= 170.31\ KN$

$$(\text{Block Strength})_2 = \frac{(400) \times 250}{1.1} + \frac{0.9 \times (344) \times 410}{\sqrt{3} \quad 1.25}$$

$= 149.54\ KN$

Strength of tension member

$= \text{min}^m$ of
(i) Yielding 181.81 KN
(ii) rupture 193.65 KN
(iii) Block shear 149.54

$= 149.54\ KN$

Strength of bolt

$$\text{Design shear} = \frac{f_{ub}}{\sqrt{3}\times 1.25}\left[n_n A_n + n_s A_s\right]$$

$$= \frac{400}{\sqrt{3}\times 1.25}\left[2\cdot \frac{0.78\pi}{4}\times d^2\right]$$

$$= \quad 58$$

Design shear per bolt = 29 KN

$$K_b = \begin{cases} \text{(i)}\ \dfrac{e}{3d_0} = \dfrac{30}{3\times 18} = 0.555 \\ \text{(ii)}\ \dfrac{p}{3d_0} - 0.25 = \dfrac{40}{3\times 18} - 0.25 = 0.491 \\ \text{(iii)}\ \dfrac{f_{ub}}{f_u} = \dfrac{400}{410} = 0.9 \\ \text{(iv)}\ 1 \end{cases}$$

$K_b = 0.491$

$$\text{Design beare strength} = 2.5 K_b\, d \cdot t \cdot \frac{f_u}{1.25}$$

$$= 2.5\times 0.491\times 16\times 8\times \frac{410}{1.25}$$

$$= 51.54\ kN$$

Bolt Strength = $\min^m$ of $\begin{cases} ① \text{ Shear} = 29\,kN \\ ② \text{ Bear} = 51.54\,kN \end{cases}$

$= 29\,KN$

2 Bolts $= 2 \times 29 = 58\,KN$

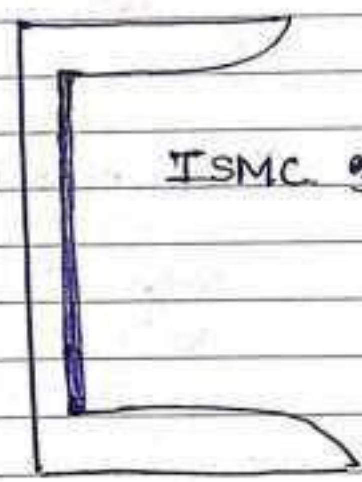
Max^m load joint can carry

$= \min^m \begin{cases} (i) \text{ strength of tension member} = 149.54\,kN \\ (ii) \text{ strength of bolt} = 58\,kN \end{cases}$

$= 58\,KN$

Design a steel column section using channel section only to carry a factored axial load of 400kN. The column is 4m long & is effectively held in position at both ends but restrained against rotation at one end only. Consider $f_y = 250MPa$ & assume wind/earthquake act?

Channel Section



ISMC 350 @ 413 N/m

(i) Assume design compressive stress 80MPa

$f_{cd} = 80MPa$

(ii) Area Reqd

$$A_{reqd} = \frac{P}{f_{cd}}$$

$$= \frac{400 \times 10^3}{80}$$

$$= 5000\ mm^2$$

$\boxed{\text{Adopt}}$ ISMC 350 @ 413 N/m

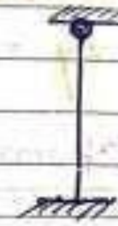
Area $= 5366\ mm^2$

M	T	W	T	F	S
1	2	3	4	5	6
8	9	10	11	12	13
15	16	17	18	19	20
22	23	24	25	26	27
29	30	31			

Column; $r_{zz} = 136.6\,mm$, $r_{yy} = 28.3\,mm$

$r_{min} = 28.3\,mm$

(iii) For one end fixed & other end pin



$l_{eff} = K \cdot L$

$= 0.8L$

$= 0.8 \times 4000$

$= 3200\,mm$

Slenderness ratio $\lambda = \dfrac{l_{eff}}{r_{min}}$

$= \dfrac{3200}{28.3}$

$= 113.07 < 250$ (λ_{max})

(Design Safe).

(iv) Channel Section $\Rightarrow$ buckling class C

For $\lambda = \dfrac{KL}{r} = 113.07$

$f_y = 250\,MPa$

By Interpolation method :

	λ_{max}	f_{cd}
$\rightarrow 113.07$	110	94.6
	120	83.7

Interpolation in fx-991ES Calculator
(i) Mode − 3 − 2 − Values − AC
(ii) (Write no) − shift + 1 − (5) − 5 :) − enter
or 5 − 5 ↵

MARCH 2018

M	T	W	T	F	S	S
			1	2	3	4
5	6	7	8	9	10	11
12	13	14	15	16	17	18
19	20	21	22	23	24	25
26	27	28	29	30	31	

FEBRUARY
08
THURSDAY
WEEK 06 • 039-326

$f_{cd} = 91.25 N/mm^2$

Design Strength $P_d = A \cdot f_{cd}$

$= 5366 \times 91.25$

$= 489.65 kN > 400 kN$ (O.K.)

* Check for limiting thickness Explain ?

$\varepsilon = \sqrt{\frac{250}{f_y}} = \sqrt{\frac{250}{250}} = 1 \Rightarrow \varepsilon = 1$

From steel table $b_f = 100 mm$
$h = 350 mm$
$t_f = 13.5 mm$
$t_w = 8.1 mm$
$R_1 = 14 mm$

Here,
$b = 100 mm$
$d = h - 2(t_f + R_1)$
$= 350 - 2(13.5 + 14)$
$= 295$

For channel Section

$b/t_f = \frac{100}{13.5} = \cancel{12.35}\ 7.30 < 15.7\varepsilon = 15.7$

$d/t_w = 295/8.1 = 36.42 < 42\varepsilon = 42$

O.K.

2018

Calculate the design compressive load for an ISHB 250 @ 536.6 N/m, 4m high. The column is restrained in erection only at both the ends. It is to be used as an uncased column in a single storey building?

Given: ISHB 250 @ 536.6 N/m

$L = 4m$

column is restrained at both the ends $L_{eff} = 0.65L$

From steel Table:

$h = 250m$

$b_f = 250mm$

$t_f = 9.7$

$t_w = 8.8$

$\text{Area} = A = 6971 mm^2$

$\frac{h}{b_f} = \frac{250}{250} = 1 \leq 1.2$

$t_f = 9.7mm \leq 100mm$

$\Leftrightarrow$ Buckling class about Z-Z axis = b

Y-Y axis = C

Design compressive stress

About X-X axis

Slenderness ratio

$$\lambda_{x-x} = \frac{l_{eff}}{r_{min}} = \frac{KL}{r_{zz}}$$

$$= \frac{0.65 \times 4 \times 10^3}{107}$$

$$= 24.29$$

For, $f_y = 250 \, N/mm^2$

Buckling class 'b'

λ_{z-z} (x)	f_{cd} N/mm^2 (Y)
20	225
24.29	?
30	216

$$\frac{y - y_1}{x - x_1} = \frac{y_2 - y_1}{x_2 - x_1}$$

$$\Rightarrow y - y_1 = (x - x_1)\frac{-9}{10}$$

b $\Rightarrow y - 225 = (24.29 - 20) - 0.9$

$\Rightarrow y = 221.13 \, N/mm^2$

About Y-Y axis

Slenderness ratio

$$\lambda_{Y-Y} = \frac{l_{eff}}{r_{min}}$$

$$= \frac{0.65L}{r_{yy}}$$

$$= \frac{0.65 \times 4 \times 10^3}{53.7}$$

$$= 48.41$$

For $f_y = 250 MPa$

Buckling Class 'c'

λ_{y-y}	f_{cd} N/mm^2
40	198
48.41	?
50	183

$$\frac{Y - 198}{48.41 - 40} = \frac{183 - 198}{50 - 40}$$

$$\Rightarrow y = 185.3 \, N/mm^2$$

11 FEBRUARY
SUNDAY
042-323 • WEEK 06

Design compressive load	Design compressive load
$P_d = A \cdot f_{cd}$	$P_d = A\, f_{cd}$
$= 6971 \times 221.13$	$= 6971 \times 185.3$
$= 1541.5$ KN	$= 1291.7$ kN

$\therefore$ Design Compressive load

$$= \min^m \text{ of } \begin{cases} \text{(i)} \quad 1541.5 \text{ kN} \\ \text{(ii)} \quad 1291.7 \text{ kN} \end{cases}$$

$$= 1291.7 \text{ kN}$$

$$\simeq 1292 \text{ kN}$$

2018

Design a steel column using channel section to carry a factored axial load of 350kN. The column is 3.5m long & is effectively held in posit° & restrained against rotation at both the ends. Take $f_y = 250$MPa & assume wind/ earthquake actions.

Given : $P = 350$KN
$L = 3.5$m
Restrained at both ends $L_{eff} = 0.65L$
$f_y = 250$MPa

Channel Section (buckling class 'c') ;

(i) Assume design compressive stress = 80MPa for Chanel

$$f_{cd} = 80 \text{ MPa}$$

(ii) $$A_{reqd} = \frac{P}{f_{cd}}$$

$$= \frac{350 \times 10^3}{80}$$

$$= 4375 \text{ mm}^2$$

$\boxed{\text{Adopt}}$ ISMC 300 @ 351.2 N/m having area $A = 4564 \text{ mm}^2$

Column ; $r_{zz} = 118.1mm$ $r_{yy} = 26.1mm$

$r_{min} = 26.1mm$.

Slenderness ratio $\lambda = \frac{L_{eff}}{r_{min}}$

$= \frac{0.65 \times 3.5 \times 10}{26.1}$

$= 87.165 < 250$

(Design safe)

λ	f_{cd}
80	136
87.165	?
90	121

$$\frac{Y - 136}{87.165 - 80} = \frac{121 - 136}{90 - 80}$$

$\Rightarrow Y = 125.25 N/mm^2$

$$f_{cd} = 125.25\,N/mm^2$$

$$P_d = A \cdot f_{cd}$$

$$= 4564 \times 125.25$$

$$= 571.65\ KN > 350\ KN$$

(Design Safe)

* Check for limiting thickness = ?

$$\varepsilon = \sqrt{\frac{250}{f_y}} = \sqrt{\frac{250}{250}} = 1, \Rightarrow \varepsilon = 1$$

$h = 300\,mm$
$b_f = 90\,mm$
$t_f = 13.6\,mm$
$t_w = 7.6\,mm$
$R_1 = 13\,mm$

Here, $b = 100\,mm$
$d = h - 2(t_f + R_1)$
$= 300 - 2(7.6 + 13)$
$= 246.8\,mm$

For channel sect^n $b = b_f$

$$b/t_f = 90/13.6 = 6.60 < 15.7\varepsilon = 15.7\ (O.K.)$$

$$d/t_w = 246.8/7.6 = 32.47 < 42\varepsilon = 42\ (O.K.)$$

Design a slab base for column ISHB 200 @ 365.9 N/m subjected to a factored load of 400 kN. M20 concrete is used for foundation. Provide welded connection betⁿ column & base plate.

Given

Column : ISHB 200 @ 365.9 N/m $\therefore D_f = 200$ mm

Axial load = 400 kN

Assume

Allowable pr. intensity $\sigma_c = 4 \ N/mm^2$

bearing stress $\sigma_s = 185 \ N/mm^2$

Step-1

Load = 400 kN

$$A_{reqd} = \frac{400 \times 10^3}{4} = 100000 \ mm^2$$

Base plate : $L_B = 350$ mm

$B_B = 300$ mm

$$A_{provided} = 350 \times 300 = 105\,000 \ mm^2$$

Step-2

$$w = \text{Upward pr.} = \frac{\text{Load}}{A_{provided}}$$

$$= \frac{400 \times 10^3}{105000}$$

$$= 3.8 \ N/mm^2 < \sigma_c \ \text{(O.k.)}$$

Step-3 Thickness of base plate

$$T_B = \sqrt{\frac{3W}{\sigma_s}\left(a^2 - \frac{b^2}{4}\right)}$$

a = Greater projection beyond column face

$$= \frac{L_B - D_f}{2}$$

$$= \frac{350 - 200}{2}$$

$$= 75mm$$

b = Smaller project^n beyond column face

$$= \frac{B_B - D_f}{2}$$

$$= \frac{300 - 200}{2}$$

$$= 50mm$$

$$T_B = \sqrt{\frac{3 \times 3.8}{185}\left(75^2 - \frac{50^2}{4}\right)}$$

$$= 17.55 \text{ mm} \approx 40mm$$

Base plate = 350mm x 300 mm x 40mm

 Use 150 mm x 150mm x 8mm cleat angle

Design a slab base for a column ISHB 450 @ 855.4 N/m to carry an axial factored load of 1500 kN. M-30 concrete is used for the foundation. Provide welded connection between column & base plate.

Given : ISHB 450 @ 855.4 N/m

$P = 1500\,kN$

M30 concrete

Bearing strength of concrete $= 0.45 f_{ck}$

$= 0.45 \times 30$

$= 13.5\ N/mm^2$

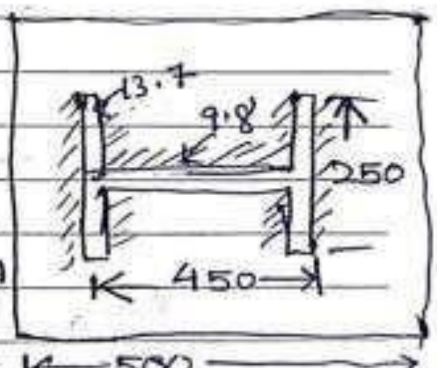
Properties ;

$t_f = 13.7\,mm$

$t_w = 9.8\,mm$

$D = 450\,mm$

$b_f = 250\,mm$



Step - 1

Reqd area for slab base

$$A_{reqd} = \frac{P}{f_c}$$

$$= \frac{1500 \times 10^3}{13.5}$$

$$= 111111.11\ mm^2$$

Let's provide square base plate

$$L = B = \sqrt{Area}$$

$$= \sqrt{111{,}111.1}$$

$$= 333.33\,mm, \quad > 450\,mm, \quad > 25\,mm$$

~~420 mm~~

Provide a base plate 420 mm x 420 mm in
500 x 300 mm
size.

Step-II The bearing pressure of concrete

$$w = \frac{P}{A}$$

$$= \frac{1500 \times 10^3}{420 \times 420} \qquad \frac{1500 \times 10^3}{500 \times 300}$$

$$= 10\ N/mm^2 < 13.5\ (O.K)$$

a = greater projection = $\frac{L - h}{2}$

$$= \frac{500 - 450}{2} = 25$$

b = Smaller projection = $\frac{B - b}{2}$

$$= \frac{450 - 420}{2} \qquad \frac{300 - 250}{2}$$

$$= 15\,mm \qquad = 25$$

$$t_{slab\,base} = \sqrt{2.5\,w\left(a^2 - 0.3 b^2\right)\frac{\gamma_{mo}}{f_y}} > t_f$$

$$= \sqrt{2.5 \times \frac{8.5}{10}\left(25^2 - 0.3 \times 25^2\right)\frac{1.1}{250}} > 13.7$$

$$= 6.47\,mm < t_f$$

Adopt $t_{sb} = 15\,mm$

Slab base : 500mm × 300 mm × 15mm

Step - III

Connection of column to the baseplate ;

By fillet weld

Shop welding, $\gamma_{mw} = 1.25$

Total welding length

$$= 2\left[(250 + 250 - 9.8) + (450 + (2 \times 13.7))\right]$$

or

$$= 250 + 250 + (250 - 9.8) + (250 - 9.8) + \{450 - (2 \times 13.7)\} + \{450 - (2 \times 13.7)\}$$

$$= 1525.6\,mm$$

$L_w = \cancel{1525.6\,mm}$ $1825.6\,mm$

$t_w = k.S$

$= 0.7S$

Area $= L_w \cdot t_w$

$$\text{Stress} = \frac{f_u/\sqrt{3}}{\gamma_{mw}} = \frac{f_u/\sqrt{3}}{1.25}$$

$$\cancel{\text{Strength} = \text{Area} \times \text{Stress}}$$

$$\cancel{= (1525.6 \times 0.7 \times S) \times \frac{f_u/\sqrt{3}}{1.25}}$$

$$\cancel{= 1525.6 \times 0.7 \times S \times \frac{410}{\sqrt{3} \times 1.25}}$$

$$\cancel{= 202230.9S}$$

$P = 202230.9S$

$\Rightarrow 1500 \times 10^3 = 202230.9S$

$\Rightarrow S = 7.41$

Min^m weld size for 15mm plate = 5mm

Max^m ,, ,, ,, $= 15 - 1.5$

$= 13.5$

$\boxed{\text{Adopt}}$ Size of weld = 8mm

No. of total end returns = 12

Effective length of weld

$(L_w)_{eff} = 1825.6 - 2 \times 8 \times 12$

$= 1633.6\,mm$

$t_w = K \cdot 8 = 0.7 \times 8 = 5.6\,mm$

$$\text{Weld strength} = \underbrace{(L_w \cdot t_w)}_{\text{Area}} \times \underbrace{\frac{f_u/\sqrt{3}}{\gamma_{mw}}}_{\text{Stress}}$$

$\Rightarrow P = \text{weld strength}$

$$\Rightarrow 1500 \times 10^3 = L_w \times 5.6 \times \frac{410}{\sqrt{3} \times 1.25}$$

$\Rightarrow (L_w) = 1414.46\,mm$ Required ~~$< (L_w)_{eff} = 1333.6$ (Not o.k.)~~

Revision

weld size = 10 mm

$(L_w)_{eff} = 1525.6 - (2 \times 10 \times 12)$

$= 1285.6$

$$1500 \times 10^3 = L_w \times (0.7 \times 10) \times \frac{410}{\sqrt{3} \times 1.25}$$

$L_w = 1131.56\,mm < (L_w)_{eff}$ (o.k)

$(L_w)_{eff} > L_{w\,Reqd}$ (o.k.)

Design a slab base for a column ISHB 350 @ 710.2 N/m subjected to a factored load of 1500 kN. M25 concrete used for the foundation. Provide welded connection between column & base plate.

Given ISHB 350 @ 710.2 N/m — $t_f = 11.6$ mm, $t_w = 10.1$, $D = 350$, $b_f = 250$ mm

$P = 1500$ kN

M25

Step-I

Bearing strength of concrete

$= 0.45 f_{ck}$

$= 0.45 \times 25$

$= 11.25$ N/mm^2

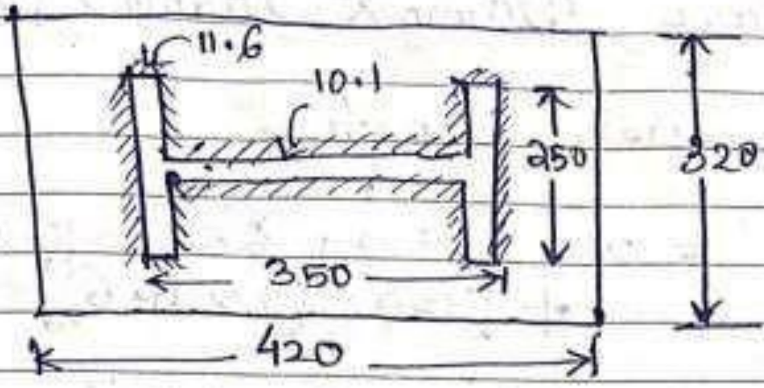
∴ Area Reqd for slab base

$= \dfrac{1500 \times 10^3}{11.25}$

$= 133333.33$ mm^2

Provide base plate 420 mm × 320 mm

$A = 134400$ mm^2



Step - II

$$w = \frac{P}{A}$$

$$= \frac{1500 \times 10^3}{134400}$$

$$= 11.16 \text{ N/mm}^2$$

Step - III

$$a = \frac{420 - 350}{2} = 35$$

$$b = \frac{320 - 250}{2} = 35$$

$$t_{sb} = \sqrt{2.5w\left(a^2 - 0.3b^2\right)\frac{\gamma_{mo}}{f_y}} > t_f$$

$$= \sqrt{2.5 \times 11.16 \times \left(35^2 - 0.3 \times 35^2\right)\frac{1.1}{250}}$$

$$= 10.26 < 11.6 .$$

$$t_{sb} \approx 12\text{mm}$$

Slab base 420mm X 320 mm X 12mm

Available weld Length

$= 250 + 250 + (250 - 10.1) + (250-10.$
$+ [350 - (2 \times 11.6)]$

$+ [350 - (2 \times 11.6)]$

$= 1633.4$ mm

Plate 12mm, Min^m weld size $= 5$mm
Max^m " " $= 12 - 1.5 = 10$
Adopt; weld size $= 6$mm
$t_w = K.S$ $(L_w)_{eff} = 1633.4 - (2 \times 6$
$= 0.7 \times 6$ $= 1489.4$
$= 4.2$

P = weld strength

$\Rightarrow 1500 \times 10^3 = L_w \, t_w \, \frac{f_u/\sqrt{3}}{\gamma_{mw}}$

$\Rightarrow 1500 \times 10^3 = L_w \times 4.2 \times \frac{410}{\sqrt{3} \times 1.25}$

$\Rightarrow L_w = 1885$ (not. o.k.)

Revision :

$S = 8$mm
$t_w = 0.7S = 5.6$ $(L_w)_{eff} = 1633.4 - (2 \times 8 \times 1$
$= 1441.4$

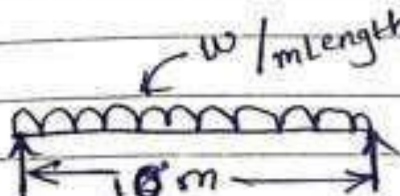
P = weld strength

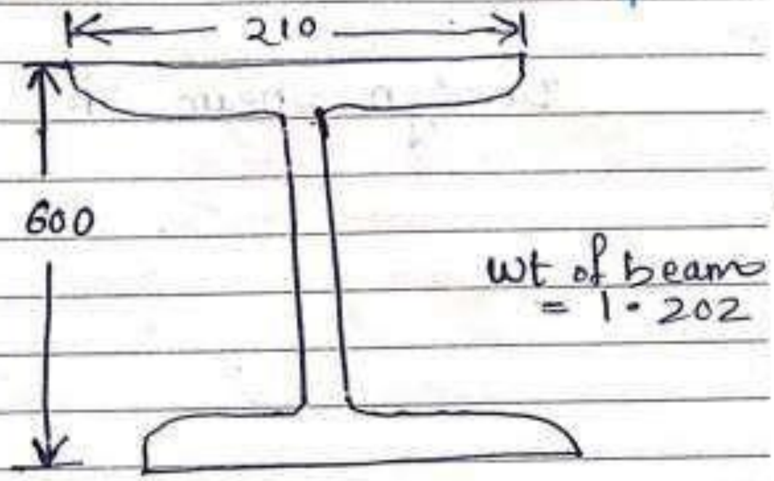
$$\Rightarrow 1500 \times 10^3 = L_w \times t_w \times \frac{f_u/\sqrt{3}}{\gamma_{mw}}$$

$$\Rightarrow 1500 \times 10^3 = L_w \times 5.6 \times \frac{410}{\sqrt{3} \times 1.25}$$

$$\Rightarrow L_w = 1414.5\,mm < L_{w_{eff}} \text{ (O.K.)}$$

A laterally supported beam ISMB 600@1202.7N/m is placed between two supports. Determine the safe uniformly distributed load the beam can carry for an effective span of 10m. Take $f_y = 250 N/mm^2$. Neglect web buckling & web crippling.





ISMB 600 @1202.71N/m

Step - I : (Section Properties)

For ISMB 600@ 1202.71N/m

Plastic section modulus $Z_p = 3510630\ mm^3$

Depth of section $h = 600mm$

Width of flange $b_f = 210mm$ $\quad r_1 = 20mm$

Sectional Area $A = 15621mm^2$

Thickness of flange $t_f = 20.8mm$

Thickness of web $t_w = 12mm$

Depth of web $d = h - 2(t_f + r_1)$

$= 600 - 2(20.8 + 20) = 518.4 m$

Moment of inertia about x axis

$I_{zz} = 91813 \times 10^4 mm^4$

Elastic section modulus

Z_{zz} or $Z_e = 3060.4 \times 10^3 mm^3$

Step-II Sect^n Classification.

$$\varepsilon = \sqrt{\frac{250}{f_y}} = \sqrt{\frac{250}{250}} = 1 \quad \Rightarrow \varepsilon = 1$$

Here,

$$b = b_f/2 = \frac{210}{2} = 105\,mm$$

For rolled section,

out stand of compression flange

$$\frac{b}{t_f} = \frac{105}{20.8} = 5.05 < 9.4\varepsilon$$

$\Rightarrow$ Flange is plastic section

Web with N.A. at mid depth

$$\frac{d}{t_w} = \frac{518.4}{12} = 43.2 < 84$$

$\Rightarrow$ Web is plastic section.

Hence Overall Section is Plastic (Class-I)

Step - III Design bending strength

$$\frac{d}{t_w} = 43.2 < 67\varepsilon$$

Assume, $V \leq 0.6 V_d$

IS:800

$$\boxed{M_d = \beta_b Z_p f_y \frac{1}{\gamma_{mo}} < 1.2 Z_e \frac{f_y}{\gamma_{mo}}}$$

$\beta_b = 1$ for plastic section

$\gamma_{mo} = 1.1$

$$M_d = \frac{1 \times 3510630 \times 250}{1.1}$$

$$= 797.87 \text{ KN-m}$$

$$1.2 Z_e \frac{f_y}{\gamma_{mo}}$$

$$= 1.2 \times 3060.4 \times 10^3 \times \frac{250}{1.1}$$

$$= 834.65 \text{ kN-m}$$

$M_d < 1.2 Z_e f_y / \gamma_{mo}$ (O.K.)

$- M_d = 797.87$ KN-m

Step-IV Design Moment & load carrying Capacity

Design moment $= \frac{w'L^2}{8}$

$= \frac{w' \times 10^2}{8}$

$= 12.5 w_u$

$12.5w' = 797.87$

$\Rightarrow w' = 63.83$ KN/m

Working load $w = \frac{w'}{1.5} = \frac{63.83}{1.5}$

$= 42.55$ kN/m

Safe working load the Beam can carry $= w -$ self wt of beam → 1202.7 N/m

$= 42.55 - 1.202$

$= 41.35$ kN/m

Step-V Check for Shear

Factored Shear $V = \frac{w'L}{2}$

$= \frac{63.83 \times 10}{2}$

$= 319.15$ kN

Design shear strength $V_d = \frac{f_y/\sqrt{3}}{\gamma_{m0}} (h \cdot t_w)$

$= \frac{250/\sqrt{3}}{1.1} (600 \times 12)$

$= 944.75 kN > V$ (O.K.)

$0.6 V_d = 0.6 \times 944.75$
$= 566.85 kN > V$ (O.K.)

Step-VI Load carrying capacity from deflection criteria:-

For S.S. beam UdL:- $\delta_{max} = \frac{5 W' L^4}{384 E I_{zz}}$ ($E = 2 \times 10^5$) → $\delta_{permissible} = \frac{Span}{240}$ elastic cladding

For pont Load $\delta_{max} = \frac{W L^3}{48 E I}$ → $\delta_{permissible} = \frac{Span}{300}$

$\delta_{max} = \frac{5 \times W' \times (10{,}000)^4}{38 \times (2 \times 10^5) \times (I_{zz})}$ not try beam

$= \frac{5 W' \times (10000)^4}{38 \times 2 \times 10^5 \times 91813 \times 10^4}$

$= 0.709 W'$

$\delta_{permissible} = \frac{10{,}000}{240}$

$= 41.67 mm$

$0.709 w' = 41.67$

$\Rightarrow w' = 58.77$ N/mm or 58.77 KN/m

Working load $w = \dfrac{w'}{1.5}$

$= 39.18$ KN/m

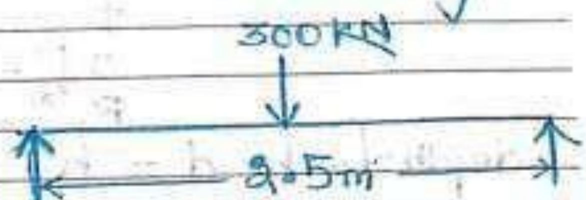
Safe working load = w − selfwt. of struct^n

$= 39.18 - 1.202$

$= 37.9 \approx 38$ KN/m.

8 9 10 11 12 1 2 3 4 5 6

Design a simply supported beam of effective span 2.5m carrying a factored concentrated load of 300KN at mid span point. Assuming it to be laterally supported.



Step-I

Max^m bending Moment

$$M = \frac{WL}{4}$$

$$= \frac{300\times10^3 \times 2.5\times10^3}{4} \text{ N-mm}$$

$$= 187.5\times10^6 \text{ N-mm}$$

Max^m Shear

$$V = \frac{W}{2}$$

$$= \frac{300}{2}$$

$$= 150\text{KN or } 150\times10^3\text{ N}$$

$$\boxed{(Z_p)_{reqd} = \frac{M\gamma_{mo}}{f_y}}$$

$$= \frac{187.5\times10^6\times1.1}{250}$$

$$= 825\times10^3 \text{ mm}^3$$

Step-II **Adopt** ISLB 350 @ 0.486 kN/m

$b_f = 165mm$
$h = 350mm$
$t_f = 11.4mm$
$t_w = 7.4mm$
$R_1 = 16mm$

depth of web $d = h - 2(t_f + R_1)$
$= 350 - 2(11.4+16)$
$= 295.2mm$

For Plastic Sectn :-

flange $b/t_f = \frac{b_f/2}{t_f} = \frac{165/2}{11.4} = 7.2 < 9.4\varepsilon = 9.4$ $\quad \varepsilon = \sqrt{\frac{250}{f_y}} = \sqrt{\frac{250}{250}} = 1$

Web $d/t_w = \frac{295.2}{7.4} = 40 < 84\varepsilon = 84$

$I_{zz} = 13158.3 \times 10^4 \ mm^4$

Sectn modulus Z_{zz} or $Z_e = 751.9 \times 10^3 \ mm^3$
$Z_p = 851.11 \times 10^3 \ mm^3$

Step-III Check for Section

Bending moment $M = 187.5 kN\text{-}m$
$\approx 190 kN\text{-}m$

$$\boxed{(Z_p)_{reqd} = \frac{M \gamma_{mo}}{f_y}}$$

$$= \frac{190 \times 10^6 \times 1.1}{250}$$

$$= 836 \times 10^3 \ mm^3 < Z_p = 851 \times 10^3$$
(O.K.)

Step-IV Check for shear

$V = 150 kN$

$$\boxed{V_d = \frac{f_y/\sqrt{3}}{\gamma_{mo}} (h t_w)}$$

$$= \frac{250/\sqrt{3}}{1.1} (350 \times 7.4)$$

$$= 340 kN > V \ (O.K)$$

$$0.6 V_d = 0.6 \times 340$$

$$= 204 kN > V \ (O.K.)$$

Step-V Check for design capacity of section

$$d/t_w = 40 < 67\varepsilon = 67$$

$$\Leftrightarrow \boxed{M_d = \beta_b Z_p \frac{f_y}{\gamma_{m0}}} < \frac{1.2 Z_e f_y}{\gamma_{m0}}$$

$$\Leftrightarrow M_d = \frac{1 \times 851.11 \times 10^3 \times 250}{1.1} < \frac{1.2 \times 751 \ldots}{1.1}$$

$$= 193.43 \text{ kN-m} < 205 \text{ kN-m}$$

Step-VI Check for deflection

S.S. beam & point load :- $\delta_{max} = \frac{WL^3}{48 E I_{zz}}$

$$= \frac{(300 \times 10^3) \times (2.5 \times 10^3)^3}{48 \times 2 \times 10^5 \times 13158.3 \times 10^4}$$

$$= \frac{300 \times 2.5^3 \times 10^{12}}{48 \times 2 \times 13158.3 \times 10^9}$$

$$= 3.71 \text{ mm}$$

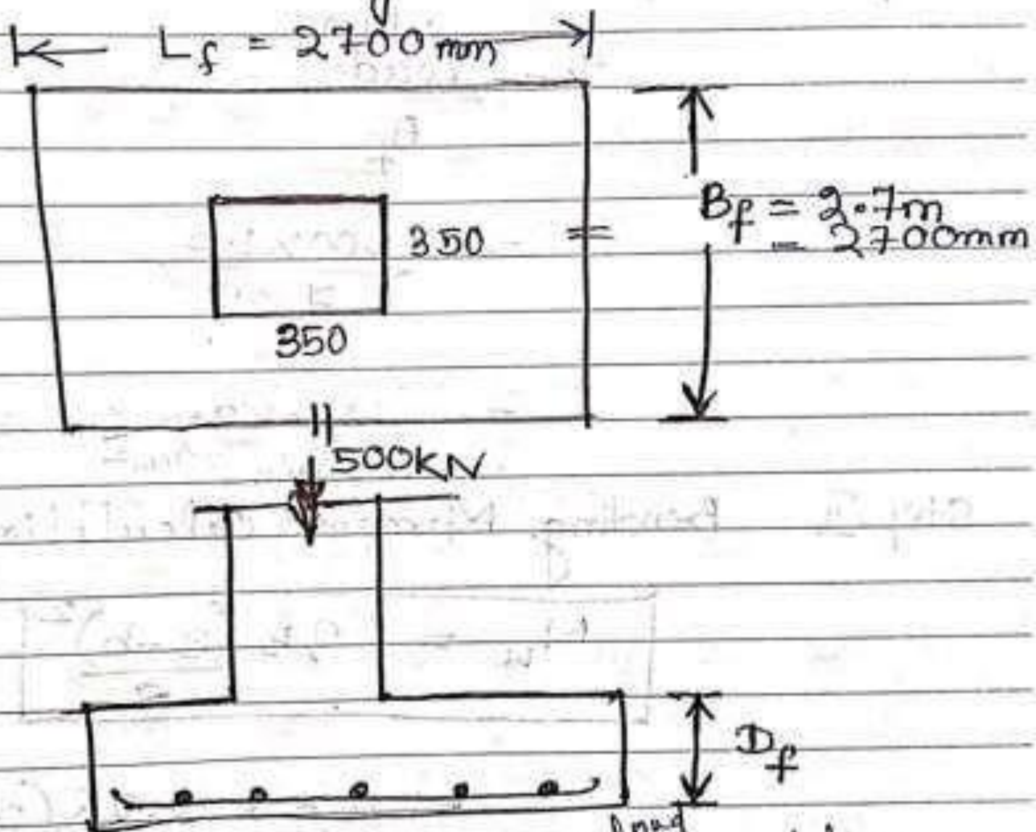
$$\delta_{permissible} = \frac{Span}{300}$$

$$= \frac{2.5 \times 10^3}{300}$$

$$= 8.33$$

$\delta_{max} < \delta_{permissible} \Rightarrow$ (Design O.K.)

Design a square footing for a RCC column of 350mm x 350mm carrying a load of 500KN. The SBC of soil is 120 KN/m². The materials are M20 concrete & HYSD reinforcement of grade Fe415.



Step-I

Column load = 500KN

Total load = 500KN + 10% of 500KN (column load, foundation load)

$= 500KN + 50KN$

$= 550KN$

Ultimate load $= 1.5 \times 550$

$= 825\ KN$

$$A_{reqd} = \frac{\text{ultimate Load}}{\text{SBC of Soil}} = \frac{825}{120}\ m^2$$

$= 6.875 m^2$

$\boxed{\text{Adopt}}$ $B_f = 2.7\ m$
$L_f = 2.7\ m$
area $A_f = 7.29 m^2$

Upward Pressure (q)

$$q = \frac{\text{column Load}}{A_f}$$

$$= \frac{500 \times 1.5}{7.29}$$

$$= 102.88\ N/mm^2 < \text{SBC. of Soil} = 120$$
$$\approx 103\ N/mm^2 \quad (O.K.)$$

Step-II Bending Moment calculation

$$\boxed{M_u = qB\frac{(B-b)^2}{8}}$$

$$= \frac{103 \times 2.7 \times (2.7 - 0.35)^2}{8}$$

$$= 191.97\ KN\text{-}m$$

Step-III Depth Calculation

$$\boxed{M_u = 0.36 f_{ck} \frac{x_{u,max}}{d}\left[1 - 0.42\frac{x_{u,max}}{d}\right] bd^2}$$

$x_{u,max}$ for M20 & Fe415 = 0.48d

$$\Rightarrow 191.97\times10^{6} = 0.36\times20\times0.48\left[1-0.42\times0.48\right]\times 2700\, d^2$$

$\Rightarrow$ $d = 160\,mm$

Twice of it Take, $d = 350\,mm$

effective cover $= 50\,mm$ (for footing)

$D_f = 350+50$

$= 400\,mm$

Step – IV Steel Calculation

$$\boxed{M_u = 0.87 f_y A_{st} d\left(1-\frac{A_{st}}{bd}\frac{f_y}{f_{ck}}\right)}$$

$$\Rightarrow 191.97\times10^{6} = 0.87\times415\times A_{st}\times350\times\left(1-\frac{A_{st}\times415}{2700\times350\times20}\right)$$

$$\Rightarrow 1519.14 = A_{st}\left(1-21.9\times10^{-6}A_{st}\right)$$

$$\Rightarrow 1519.14 = A_{st}-21.9\times10^{-6}A_{st}^2$$

$$\Rightarrow 21.9\times10^{-6}A_{st}^2 - A_{st} + 1519.14 = 0$$

$$\Rightarrow A_{st} = 1573.35\,mm^2 \;\&\; 44088\,mm^2$$

4157 (max^m)

Main Reinforcement

Provide, 25mm ϕ bar with area $= \frac{\pi}{4} \times 25^2$

$$spacing = \frac{2700 mm \times \frac{\pi}{4} \times 25^2 mm^2}{44088\ mm^2}$$

$= 30.06\ mm$

25mm ϕ bar @ 50mm c/c

Longitudinal Reinforcement

$$Area = 0.12\%\ bD$$

$$= \frac{0.12}{100} \times 2700 \times 400$$

$$= 1296\ mm^2$$

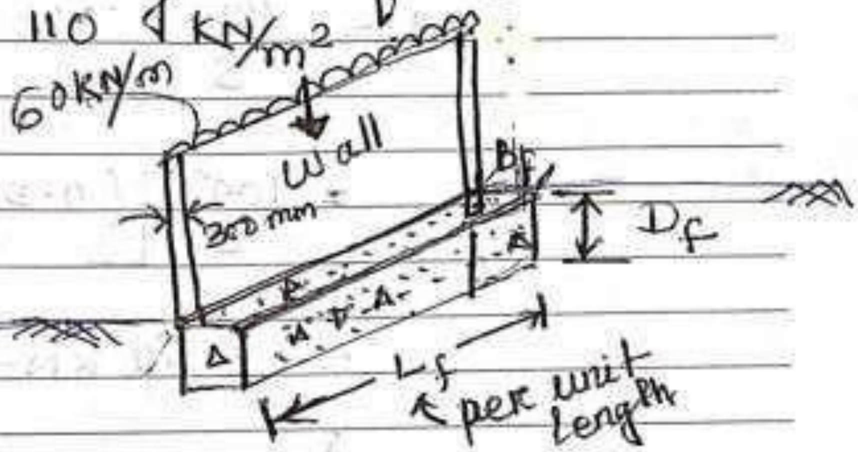
Provide, 12mm ϕ bars with area $= \frac{\pi}{4} \times 12^2$
$= 113 m$

$$Spacing = 2700 \times \frac{113}{1296}$$

$= 235\ mm$

12mm ϕ bar @ 250mm c/c

Design a RCC footing for a masonry wall of 300mm thick, subjected to a load of 60 KN/m including self wt. The SBC of soil is 110 KN/m²



Wall thickness = 300mm = 0.3m
Load = 60 KN/m

For M20 & Fe415, $x_{u\,max} = 0.48d$

Width of footing $B_f = \dfrac{60\ \text{KN/m}}{110\ \text{KN/m}^2} = 0.55\text{m}$

$(B_f = 550\ \text{mm} ;\ L_f = 1\text{m})$

Net upward pressure

$$q = \frac{\text{wall load}}{\text{foundat}^n\ \text{area}}$$

$$= \frac{60\ \text{KN/m}}{0.55\ \text{m} \times 1}$$

$$= 109\ \text{N/mm}^2 < \text{SBC of soil}$$ (O.K.)

Step-II

Max Bending Moment occurs at sect^n x-x distance b/4 from the centre of wall.

$$M = \frac{q}{8}\left[(B-b)\left(B-\frac{b}{4}\right)\right]$$

$$= \frac{109}{8}\left[(0.55-0.3)\left(0.55-\frac{0.3}{4}\right)\right]$$

$$= 1.62 \text{ KN-m}$$

Step-III Depth Calculation of foundation

$$M_u = 0.36 f_{ck} \frac{x_{umax}}{d}\left(1 - 0.42\frac{x_{umax}}{d}\right)bd^2$$

$$\Rightarrow 1.62 \times 10^6 = 0.36 \times 20 \times 0.48 (1 - 0.42 \times 0.48) \cancel{1000}\, d^2$$

550

$$\Rightarrow d^2 = 587.1 \text{ mm}$$

Adopt d = 350mm

D = 350 + 50
= 400

Adopt d = 600 mm

effective cover = 50 mm (foundation)

$$D_f = 600 + 50 = 650 \text{ mm}$$

Step-IV Steel area Calculation for foundation

$$M_u = 0.87 f_y A_{st} d \left(1 - \frac{A_{st} f_y}{b d f_{ck}}\right)$$

350

$$\Rightarrow 1.62 \times 10^6 = 0.87 \times 415 A_{st} \times 600 \left(1 - \frac{A_{st} \times 415}{\cancel{1000} \times 600 \times 20}\right)$$

550 350

$$\Rightarrow 7.5 = A_{st} - 34.5 \times 10^{-6} A_{st}^2$$

9359.43
9266

$$\Rightarrow 34.5 \times 10^{-6} A_{st}^2 - A_{st} + 7.5 = 0$$

$$\Rightarrow A_{st} = 28978\,mm^2 \text{ or } \cancel{7.5\,mm^2}$$

Main reinforcement

Bar 22ϕ, $A = \frac{\pi}{4} \times 22^2$

$$\text{Spacing} = 1000 \times \frac{\frac{\pi}{4} \times 22^2}{28978}$$

16mm ϕ bar @ 50mm c/c

$= 15mm \simeq 50mm$

Adopt : 22ϕ bar @ 50mm c/c

Longitudinal reinforcement

Area $= 0.12\%\ bD$,

$= \frac{0.12}{100} \times 1000 \times 650$

8mm ϕ bar @ 150 mm c/c

$= 780mm^2$

bar $= 12\phi$, $A = \frac{\pi}{4}(12^2)$

$$\text{Spacing} = \frac{1000 \times \frac{\pi}{4} \times 12^2}{780}$$

$$= 144\ mm$$

Adopt : 12 ϕ bars @ 144 mm c/c

Ch-7 Design of tubular steel structures

- Tubular steel Structures are made to resist Lateral Loads (wind, seismic, impact) in high rise building

- It can be used for office, apartment of over 40 Storey.

7.1 Round tubular sections, permissible stresses

Mild steel round bar
Dia: 6mm - 305mm available
Length: 6.1m - 6.4m

Example: 50mm dia mild steel round bar (15.41kg/m)

⇒ If length of bar = 4.2m
Mild steel round bar weight
= 4.2×15.41
= 64.722kg

M	T	W	T	F	S	S
			1	2	3	4
5	6	7	8	9	10	11
12	13	14	15	16	17	18
19	20	21	22	23	24	25
26	27	28				

7.2 Tube columns & compression members crinkling

Compress^n member : Tube column — Circular / Square / Rectangular

- When a steel tubular member is subjected to compression, then, the tube may crinkle (i.e the walls of the tube may cave in & form folds after the manner of concertina.
- crinkling of collapse load

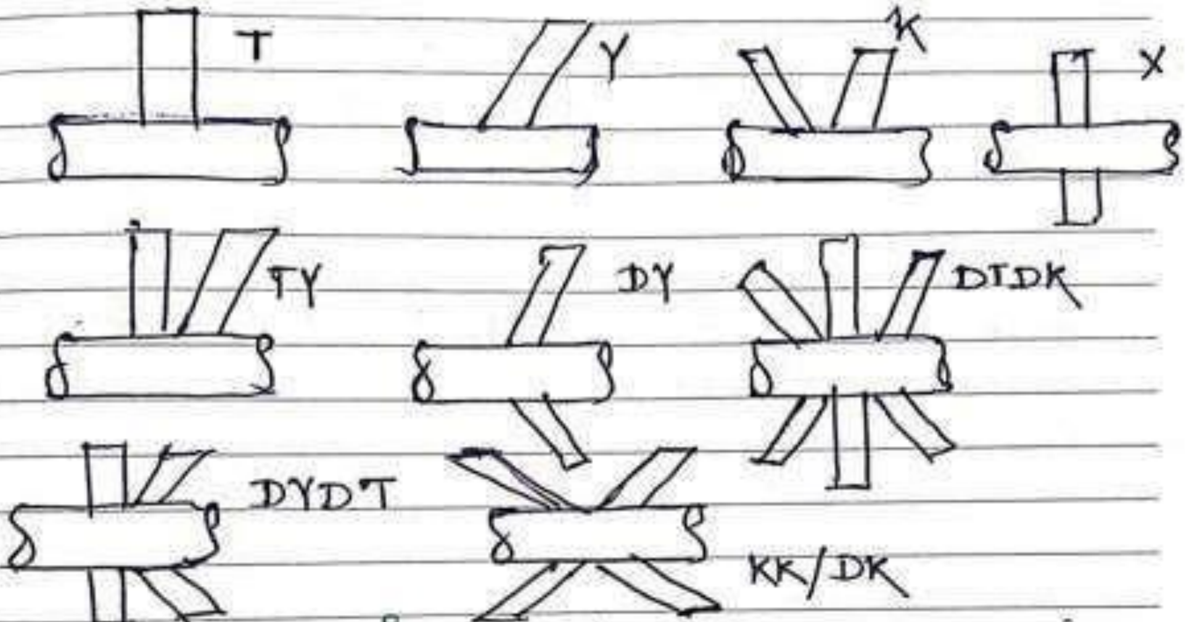
$$\text{collapse stress } p = E \frac{t}{R} \left[\frac{m^2}{3(m^2-1)} \right]^{1/2}$$

t = thickness, R = radius, $\frac{1}{m}$ = poisson's ratio

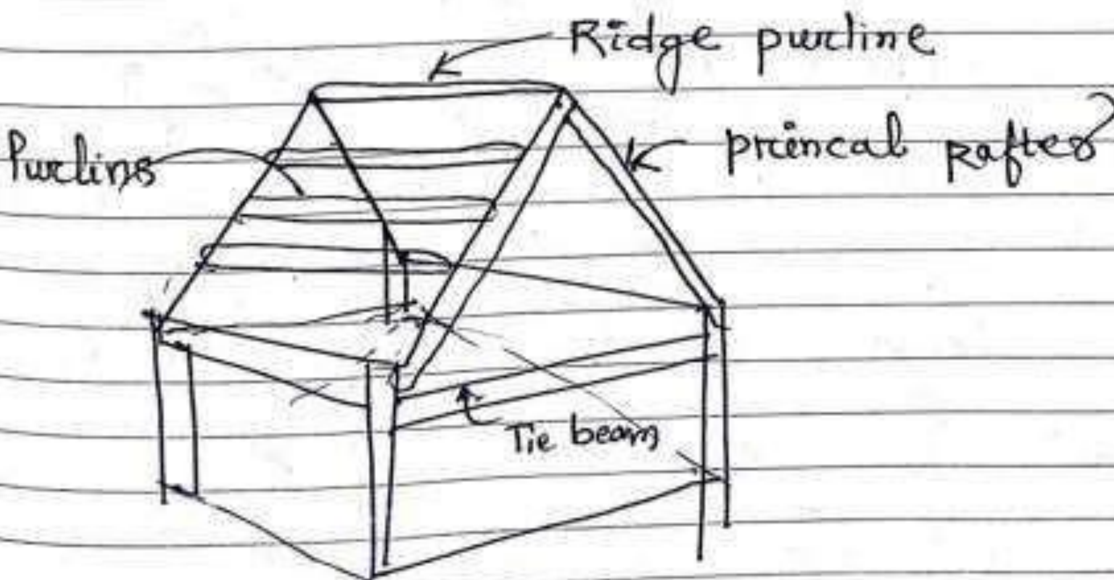
7.3 Tube tension members & tubular roof trusses

- The effectiveness of a member subjected to axial tension depends on c/s area.
- So rolled steel section / tubular section has same value of tensile resistant.
- Roof trusses are subjected to both axial compressive & tension force only

7.4 Joints in tubular trusses



7.5 Design of tubular beams & purlins.



Ch-8 Design of Timber Structure

- The wood which is used as structural material is called timber
- Use Code IS883 : 1994

Problem - 1 A Sal wood (M.P.) Column is 150mm x 200mm. Determine the safe axial Load on the column, if the unsupported Length of the column is (i) 1.5m (ii) 2.8 m (iii) 4m. Assume inside Location & standard grade.

For sal wood & inside Location ;
& permissible compressive stress parallel to the grain

stress
f_{cp} = 10.6 MPa   E = 12670 MPa (IS-Code Table-1)

(i) Unsupported Length of column S = 1.5m = 1500mm
Least Lateral dimension of the column
d = 150mm

Max^m slenderness ratio $\frac{S}{d} = \frac{1500}{150} = 10 < 11$

$\Rightarrow$ Short column

* 3conditions

Short column — $\frac{S}{d} < 11$

Intermediate column — $11 < \frac{S}{d} < K_8$

Long column — $\frac{S}{d} > K_8$ = 0.584

For short column,

$$\text{Allowable stress } f_c = f_{cp}$$

$$= 10.6 MPa$$

$$\text{Safe axial load} = \text{Area} \times \text{Stress}$$

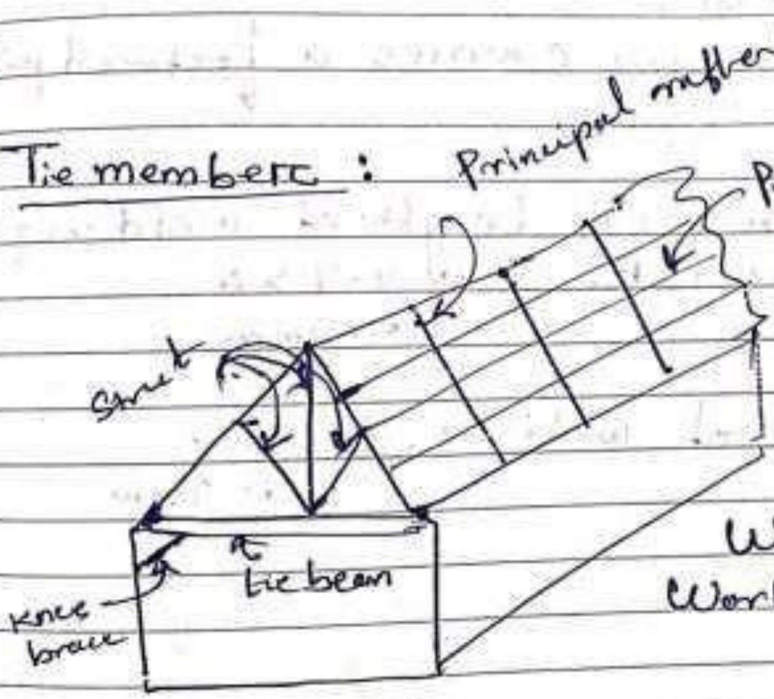
$$= (150 \times 200) \times 10.6$$

$$= 318000 N$$

$$= 318 kN$$

(ii)

Tie member :



2 ISA 150×75×8

Angles connected to either side of a 10mm gusset plate

Pull = 300kN

Welded connection = ?
Workshop welding

Working load = 300KN
Factored load = 450KN = 1.5×300N

Thickness of weld :

(i) At the rounded toe of the angle section, size of the weld shouldn't exceed

$= \frac{3}{4} \times$ thickness of weld

$S = \frac{3}{4} \times 8mm = 6mm$

(ii) A top, the thickness should not exceed

$s = t - 1.5$

$= 8 - 1.5$

$= 6.5 mm$

$\boxed{\text{Adopt}}$ $s = 6mm$ weld

Each angle carries a factored pull

$\frac{450}{2} = 225 kN$

Let, L_w = be the total length of weld required

Assuming normal weld $t = 0.7 \times 6$

$= 4.2mm$

Design strength of weld $= L_w t \times \frac{f_u}{\sqrt{3}\, \gamma_{mw}}$