

LECTURE NOTES
ON
STRUCTURAL DESIGN-II

Diploma in Civil Engineering

By

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Govt. Polytechnic, Jajpur

FEBRUARY							2018						
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JANUARY

SD-II (Numericals)

MONDAY

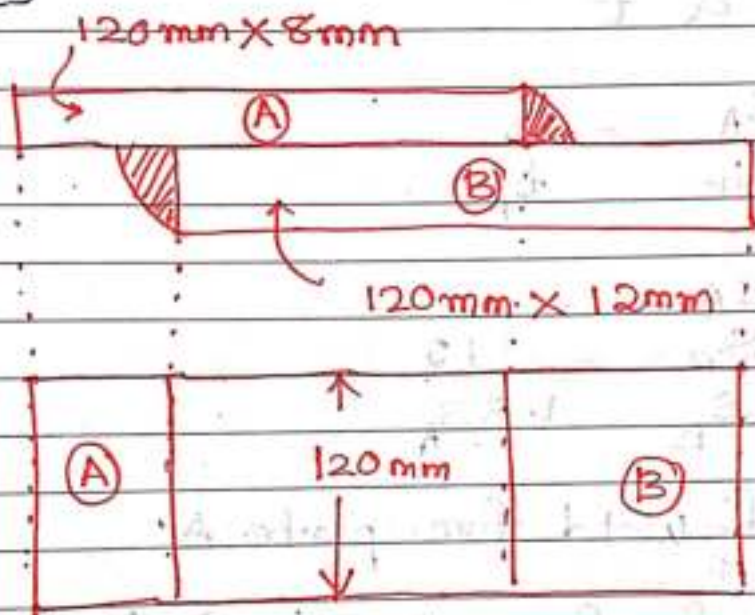
WEEK 01 • 001-364

01

① Design a welded lap joint for max^m efficiency shop welding, 8 Fe410 grade steel

$$\uparrow$$

$$\gamma = 1.25$$



→ 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

$$F_t = \frac{A_g f_y}{\gamma_{mo}}$$

$$= \frac{(120 \times 8) \times 250}{1.25}$$

$$= 218.18 \text{ kN} = \text{weld strength}$$

\therefore Width of plates same, the weld size S & thickness 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 for plate A

$$\text{Area of weld } A = (L_w)_A \cdot (t_w)_A, (t_w)_A = 0.7 S_A$$

$$\text{Stress} = \frac{f_u}{\sqrt{3}}$$

$$\text{Strength} = \text{Area} \times \frac{f_{mW}}{\sqrt{3}}$$

$$= (L_w)_A \cdot (t_w)_A \times \frac{f_u}{\sqrt{3}}$$

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

$$\begin{aligned} * (L_w)_A &= 120 - 2S_A \\ * (L_w)_B &= 120 - 2S_B \end{aligned}$$

- Strength of weld for plate B

$$\text{Area} = (L_w)_B (t_w)_B$$

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

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

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

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

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

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

$$= \left\{ (120 - 2S_A) S_A + (120 - 2S_B) S_B \right\}$$

$$\times \frac{0.7 \times 410}{1.25 \times \sqrt{3}}$$

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

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

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

U4

THURSDAY

004-361 • WEEK 01

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$$218.18 \times 10^3 = 132.56 (300S_A - 6.5S_A^2)$$

$$\Rightarrow 1645.9 = 300S_A - 6.5S_A^2$$

$$\Rightarrow 6.5S_A^2 - 300S_A + 1645.9 = 0$$

$$\Rightarrow S_A = \textcircled{39.8} \text{ \& } 6.4 \text{ mm}$$

← rejected

$$S_B = 1.5S_A = 9.6 \text{ mm}$$

check:

$$\text{Min}^3 = 3 \text{ mm}$$

For 8mm plate; Max^m weld size = $8 - 1.5 = 6.5 \text{ mm}$

For 12mm plate;

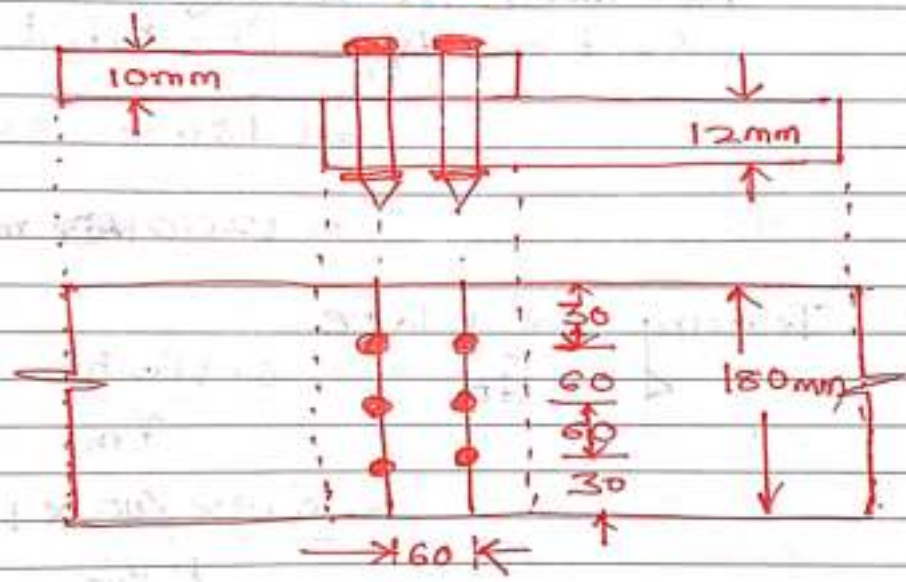
$$\text{Min}^m \text{ weld size} = 5 \text{ mm}$$

$$\text{Max}^m \text{ weld size} = 12 - 1.5 = 10.5 \text{ mm}$$

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

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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 -

$$d = 16 \text{ mm}$$

$$d_o = 16 + 2 = 18 \text{ mm}$$

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

$$\& f_{ub} = 400 \text{ MPa}$$

06

SATURDAY

006-359 • WEEK 01

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- Strength of plate :

Thickness of thinned plate = 10mm

width of plate $b = 180\text{mm}$

No. of bolts in critical section = 3

Net area at weakest

section $A_n = (b - nd_o)t$

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

$$= 1260 \text{ mm}^2$$

Tearing of plate

$$T_{dn} = \frac{0.9 f_u A_n}{\gamma_{mb}}$$

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

$$= 371.952 \text{ kN}$$

- Shearing strength of bolt :

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

$$\text{Total } n_n = 1 \times 6 = 6$$

$$n_s = 0 \times 6 = 0$$

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

FEBRUARY							2018					
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5	6	7	8	9	10	11						
12	13	14	15	16	17	18						
19	20	21	22	23	24	25						
26	27	28										

JANUARY

07

SUNDAY

WEEK 01 • 007-358

$$\text{Nominal shear strength } (V_{nsb}) = \frac{f_{ub}}{\sqrt{3}} [n_n A_{nb} + n_s A_{sb}]$$

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

$$= 217.268 \text{ kN}$$

$$\text{Design shear strength } (V_{ds})_b = \frac{(V_{nsb})}{\gamma_{mb}}$$

$$= \frac{217.268}{1.25}$$

$$= 173.8 \text{ kN}$$

- Bearing strength of bolt :

$$k_b = \text{min of } \left\{ \begin{array}{l} \text{(a) } \frac{e}{3d_o} = \frac{30}{3 \times 18} = 0.56 \\ \text{(b) } \frac{t}{3d_o} - 0.25 = 0.861 \\ \text{(c) } \frac{f_{ub}}{f_u} = \frac{400}{410} = 0.98 \\ \text{(d) } 1 \end{array} \right.$$

$$\Rightarrow k_b = 0.56$$

$$\text{Nominal bearing strength } V_{npb} = 2.5 k_b d t f_u$$

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

$$= 91.84 \text{ kN}$$

08

JANUARY

MONDAY

008-357 • WEEK 02

2017

DECEMBER

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$$\text{Design bearing strength } (V_{DP})_b = \frac{(V_{DP})_b}{\gamma_{ml}}$$

$$= \frac{91.84}{1.25}$$

$$= 73.5 \text{ kN}$$

$$\text{Force nos of bolts } (V_{DP})_b = 6 \times 73.5 = 440 \text{ kN}$$

$$\text{Strength of joint} = \min \left\{ \begin{array}{l} \text{(a) Shearing of bolt} = 173.8 \text{ kN} \\ \text{(b) Bearing of bolt} = 440 \text{ kN} \\ \text{(c) Tearing of plate} = 371.952 \text{ kN} \end{array} \right.$$

$$\text{(b) Bearing of bolt} = 440 \text{ kN}$$

$$\text{(c) Tearing of plate} = 371.952 \text{ kN}$$

\therefore Strength of joint = 173.8 kN is the max^m force that can be transmitted

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

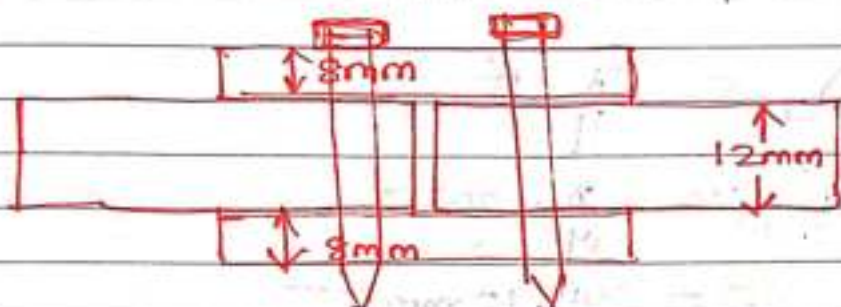
$$= \frac{173.8 \text{ kN}}{T_{sq} \cdot \gamma_{ml}} = \frac{173.8}{1800 \times 250} \times 100$$

$\leftarrow \frac{A_g \cdot f_y}{\gamma_{ml}}$
 gross area of thinner plate

$$= 42.5\%$$

2018

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



for 1 bolt } $n_n = 1$ (for double cover butt joint)
 $n_s = 1$

- Shearing of bolts :- 2 bolts

$d = 20\text{mm} \Rightarrow d_0 = 22\text{mm}$

4.6 grade $\Rightarrow f_{ub} = 400$ & $f_{yb} = 240\text{MPa}$

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

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

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

10

JANUARY

WEDNESDAY

010-355 • WEEK 02

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

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

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

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

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

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

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

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

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

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

$$= 206.61 \text{ kN}$$

- Bearing of bolts m

$$k_b = \min \left\{ \begin{array}{l} \text{(a)} \frac{e}{3d_0} = \frac{33}{3 \times 22} = 0.5 \\ \text{(b)} \frac{p}{3d_0} = 0.25 = 0.51 \\ \text{(c)} \frac{f_{yb}}{f_u} = 0.97 \\ \text{(d)} 1 \end{array} \right.$$

FEBRUARY							2018						
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26	27	28											

JANUARY

THURSDAY
WEEK 02 • 011-354

11

* $e = 1.5 d_o$

$= 1.5 \times 22$

$= 33 \text{ mm}$

Assume pitch $p = 50 \text{ mm}$

$K_b = 0.5$

Nominal bearing strength $(V_{np})_b = 2.5 k_b d t f_u$

$= 2.5 \times 0.5 \times 20 \times 12 \times 410$
 $= 123 \text{ kN}$

Design bearing strength = $\frac{(V_{np})_b}{\gamma_{ml}}$

$= \frac{123}{1.25}$
 $= 98.4$
 $= 96 \text{ kN}$

For 2 bolts $(V_{dp})_b = 2 \times 96 \text{ kN}$
 $= 192 \text{ kN}$

Strength of bolt = min { (a) Shearing = 206.6 kN

(b) bearing = 196.8 kN
 $= 197 \text{ kN}$

$= 197 \text{ kN}$

12

JANUARY

FRIDAY

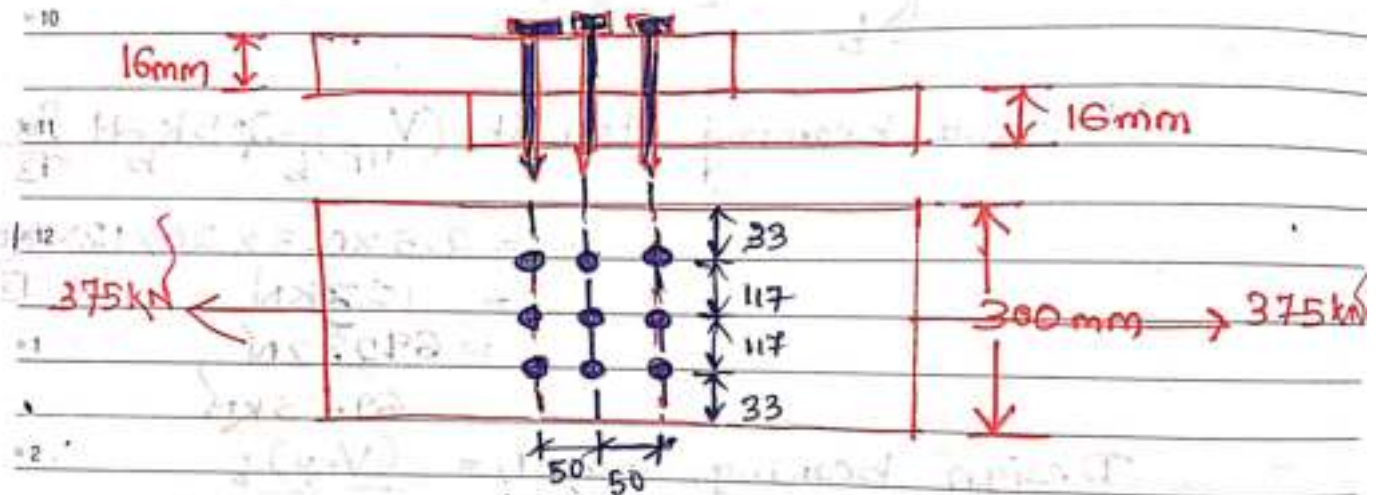
012-353 • WEEK 02

2017

DECEMBER

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- 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.



$$d = 20 \text{ mm}$$

$$d_o = 22 \text{ mm}$$

Shearing of bolts ;

$$n_n = 1, n_s = 0$$

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

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

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

$$= 56.58 \text{ kN}$$

FEBRUARY							2018							
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JANUARY

SATURDAY
WEEK 02 • 013-352

13

$$\text{Design } (V_{ds})_b = \frac{(V_{ns})_b}{d_{ml}}$$

$$= \frac{56.58}{1.25}$$

$$= 45.26 \text{ kN}$$

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

$$= 8.28$$

$$= 9 \text{ nos}$$

spacing of bolts

$$e = 1.5d_o = 1.5 \times 22 = 33 \text{ mm}$$

$$\text{Assume } p = 50 \text{ mm}$$

$$k_b = \min^m \text{ of } \left\{ \begin{array}{l} \text{(i) } \frac{e}{3d_o} = 0.5 \\ \text{(ii) } \frac{p}{3d_o} = 0.25 = 0.51 \\ \text{(iii) } \frac{f_{ub}}{f_u} = \frac{400}{410} = 0.9 \\ \text{(iv) } 1 \end{array} \right.$$

$$\text{Nominal bearing } (V_{np})_b = 2.5 k_b d \cdot t \cdot f_{ub}$$

$$= 2.5 \times 0.5 \times 20 \times 16 \times 410$$

$$= 164 \text{ kN}$$

2018

14

JANUARY

SUNDAY

014-351 • WEEK 02

2017

DECEMBER

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4	5	6	7	8	9	10
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design bearc $(V_{dp})_b = \frac{(V_{np})_b}{\gamma_{mb}}$

$87.2 = \frac{164 \text{ kN}}{1.25}$

$= 131.2 \text{ kN}$

Forc 9 no. of bolts = $1180 \text{ kN} > 45.26 \text{ kN}$

(Design is Safe)

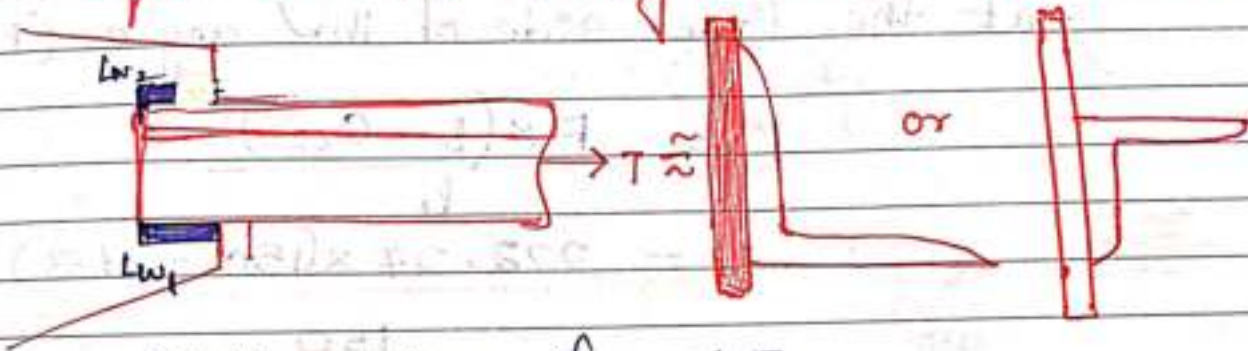
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19	20	21	22	23	24	25							
26	27	28											

JANUARY

MONDAY
WEEK 03 • 015-350

15

⑤ An angle sectⁿ 150 mm x 115 mm x 12 mm is to be connected to a 12 mm thick gusset plate at site. Design the weld to carry a load equal to the strength of the members.



Field welding $r_{mw} = 1.5$

From steel table, ISA 150 x 115 x 12

$$A_g = 978 \text{ mm}^2$$

$$C_{zx} = 27.3 \text{ mm}$$

Design strength of plate (thickness)

$$T_{dn} = \frac{A_g f_y}{r_{mo}}$$

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

$$= 222.272 \text{ kN}$$

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* Therefore, the weld will be designed to transmit a force equal to $222.27 \text{ kN} = F$

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

$$P_1 = \frac{F \times (L - C_{22})}{l}$$

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

$$= 181.81 \text{ kN}$$

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

$$P_2 = \frac{F \times C_{22}}{l}$$

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

$$= 40.45 \text{ kN}$$

FEBRUARY							2018						
M	T	W	T	F	S	S							
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5	6	7	8	9	10	11							
12	13	14	15	16	17	18							
19	20	21	22	23	24	25							
26	27	28											

JANUARY

WEDNESDAY
WEEK 03 • 017-348

17

Assume ; size of weld $s = 6\text{mm}$

$$\text{Throat thickness } (t_w) = 0.7s = 4.2\text{mm}$$

Length L_{w1} & L_{w2}

$$\text{Area} = (L_w)(t_w)$$

$$\text{Stress} = \frac{f_u}{\sqrt{3}}$$

$$P_{w1} = \text{Strength} = (L_{w1} t_w) \times \frac{f_u}{\sqrt{3}}$$

$$= L_{w1} \times 4.2 \times \frac{410}{1.5}$$

$$= 662.79 L_{w1} = P_1$$

$$P_{w2} = (L_{w2} t_w) \frac{f_u}{\sqrt{3}}$$

$$= L_{w2} \times 4.2 \times \frac{410}{1.5}$$

$$= 662.79 L_{w2} = P_2$$

$$\therefore 662.79 L_{w1} = P_1$$

$$\Rightarrow 662.79 \times L_{w1} = 181.81 \times 10^3$$

$$\Rightarrow L_{w1} = 274.3\text{mm} = 275\text{mm}$$

18

JANUARY

THURSDAY

018-347 • WEEK 03

2017

DECEMBER

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

$$662.79 Lw_2 = P_2$$

$$\Rightarrow 662.79 Lw_2 = 40.45 \times 10^3$$

$$\Rightarrow Lw_2 = 61 \text{ mm}$$

*10

*11

*12

*1

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*6

FEBRUARY							2018
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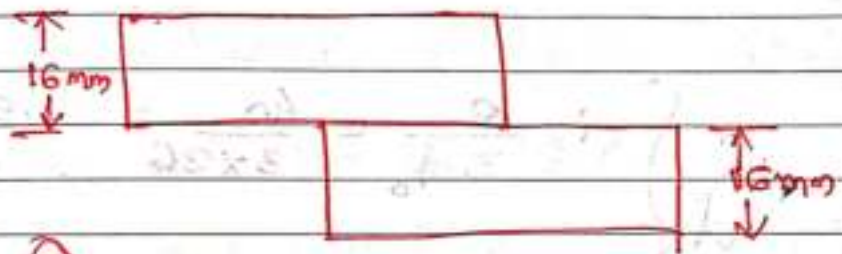
JANUARY

19

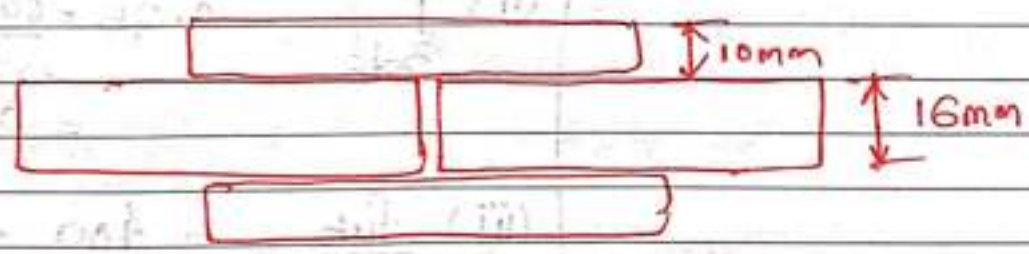
FRIDAY
WEEK 03 • 019-346

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 covers

(i)



(ii)



Given: $d = 24\text{mm}$ $p = 60\text{mm}$ $f_{ub} = 400$
 $d_o = 26\text{mm}$ $e = 40\text{mm}$ $f_u = 410$

Case-I

Shearing

$$n_n = 1, n_s = 0$$

$$A_n = 0.78 \frac{\pi}{4} d^2 = 353 \quad A_s = \frac{\pi}{4} d^2$$

$$\text{Nominal Shear} = \frac{f_{ub}}{\sqrt{3}} (n_n \cdot A_n + n_s \cdot A_s)$$

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

FEBRUARY
MARCH
APRIL
MAY
JUNE

20

JANUARY

SATURDAY

020-345 • WEEK 03

2017

DECEMBER

M	T	W	T	F	S	S
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4	5	6	7	8	9	10
11	12	13	14	15	16	17
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25	26	27	28	29	30	31

FEBRUARY

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

$$= \frac{81.521}{1.25}$$

$$= 65.21 \text{ kN}$$

Bearing

$$k_b = \min \left\{ \begin{array}{l} \text{(i) } \frac{e}{3d_o} = \frac{40}{3 \times 26} = 0.513 \\ \text{(ii) } \frac{P}{3d_o} - 0.25 = \frac{60}{3 \times 26} - 0.25 \\ \text{(iii) } \frac{f_{ub}}{f_u} = \frac{400}{410} \approx 0.9 \\ \text{(iv) } 1 \end{array} \right.$$

$$\text{(ii) } \frac{P}{3d_o} - 0.25 = \frac{60}{3 \times 26} - 0.25 = 0.52$$

$$\text{(iii) } \frac{f_{ub}}{f_u} = \frac{400}{410} \approx 0.9$$

$$\text{(iv) } 1$$

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

$$= 2.5 \times 0.513 \times 24 \times 16 \times 410$$

$$= 201.92 \text{ kN}$$

2018

FEBRUARY							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											

JANUARY

SUNDAY

WEEK 03 • 021-344

21

$$\begin{aligned} \text{Design shear } (V_{dp})_b &= \frac{(V_{dp})_b}{\gamma_{ml}} \\ &= \frac{201.92}{1.25} \\ &= 161.53 \text{ kN} \end{aligned}$$

∴ Strength of bolt = 65.21 kN

Case-II Double cover butt joint

For 1 bolt $\left\{ \begin{array}{l} n_n = 1 \text{ \& } n_s = 1 \\ A_n = 0.78 \frac{\pi d^2}{4}, \quad A_s = \frac{\pi d^2}{4} \end{array} \right.$

Min^m 2 bolts $\left\{ \begin{array}{l} = 353 \text{ mm}^2 \\ = 452 \text{ mm}^2 \end{array} \right.$

$$\begin{aligned} \text{Nominal Shear } (V_{ns})_b &= \frac{f_{ub}}{\sqrt{3}} (n_n A_n + n_s A_s) \\ &= \frac{400}{\sqrt{3}} (2 \times 353 + 2 \times 452) \\ &= 185.9 \text{ kN} \times 2 = 371.8 \end{aligned}$$

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

∴ per 1 bolt design shear = 148.7 kN

22

JANUARY

MONDAY

022-343 • WEEK 04

2017

DECEMBER

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

bearing will be same per bolt
 $= 161.53 \text{ kN}$

*8

*9

Strength of bolt = 148.7 kN

*10

*11

*12

*1

*2

*3

*4

*5

*6

FEBRUARY							2018
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JANUARY

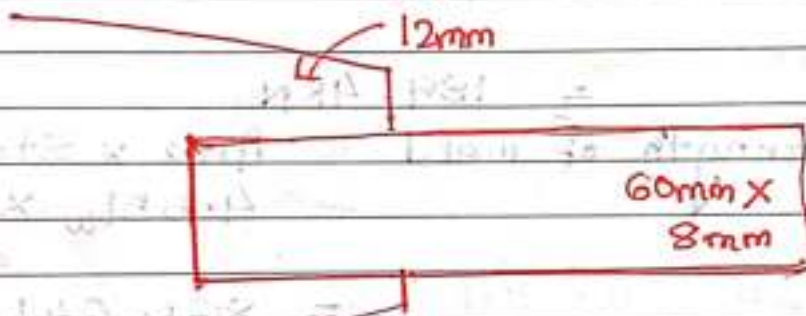
TUESDAY

WEEK 04 • 023-342

23

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 = $\frac{A_g f_y}{\gamma_{mo}} = \frac{(60 \times 8) \times 250}{1.1} = 109.1 \text{ kN}$

Case-I; Shop welding $\gamma_{mw} = 1.25$
plate thickness = 8mm

Min^m weld size = 3mm

Max^m weld size = $8 - 1.5 = 6.5 \text{ mm}$

[Adopt] size of weld s = 6.5

$$t_w = K \cdot S$$

$$= 0.7 \times 6.5$$

$$= 4.55$$

L_w

24

JANUARY

WEDNESDAY

024-341 • WEEK 04

2017

DECEMBER

M	T	W	T	F	S
				1	2
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$$\text{Area} = l_w \cdot t_w = 4.55 l_w$$

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

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

$$= 189.4 \text{ KN}$$

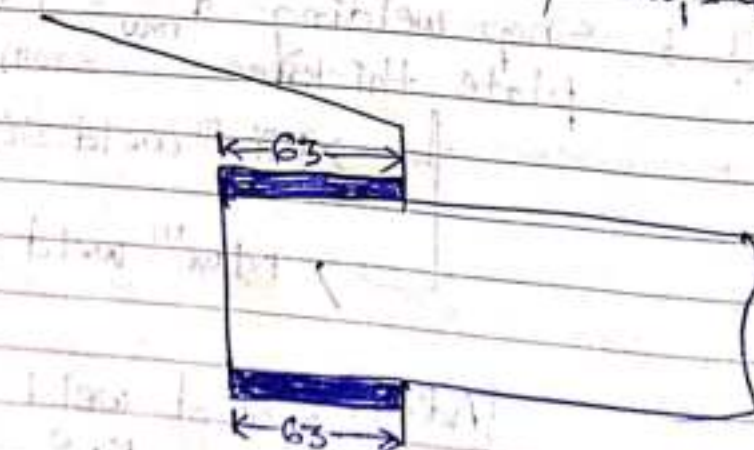
$$\text{Strength of weld} = \text{Area} \times \text{Stress}$$

$$= 4.55 l_w \times 189.4$$

$$= 861.64 l_w$$

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

$$\Rightarrow l_w = 126.6 \text{ mm} \quad l_{w1} = l_{w2} = 63 \text{ mm}$$



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12	13	14	15	16	17	18							
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JANUARY

THURSDAY

WEEK 04 • 025-340

25

Case - II Field welding $r_{mw} = 1.5$

Strength of weld = Area \times stress

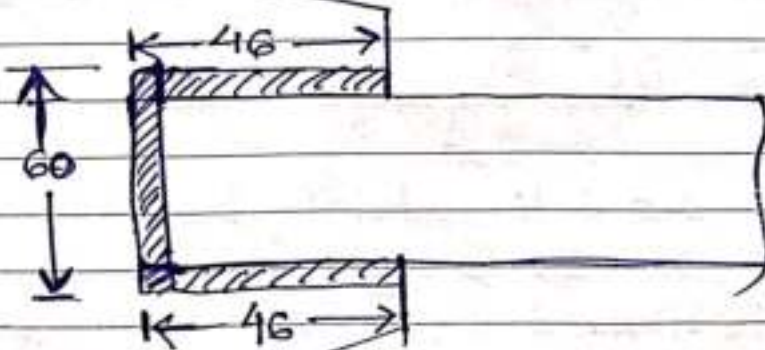
$$= L_w \times 4.55 \times \frac{f_u/\sqrt{3}}{1.5}$$

$$= 718 L_w$$

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

$$\Rightarrow L_w = 151.8 \text{ mm} \quad l_{w1} + 60 + l_{w2} = 152$$

$$\Rightarrow l_{w1} = l_{w2} = 46 \text{ mm}$$



FEBRUARY							2018						
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12	13	14	15	16	17	18							
19	20	21	22	23	24	25							
26	27	28											

JANUARY

SATURDAY
WEEK 04 • 027-338

27

① A tension member, 0.8m long, to resist a service load of 20kN & a service load of 50kN. Design a rectangular bar of standard structural grade Fe 410. 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 = 50kN

Partial Safety factor for load = 1.5
 $\gamma_{mb} = 1.25$

Fe 410 $f_u = 410, f_y = 250$
 $d = 16$

$d_0 = 16 + 2 = 18\text{mm}$

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

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

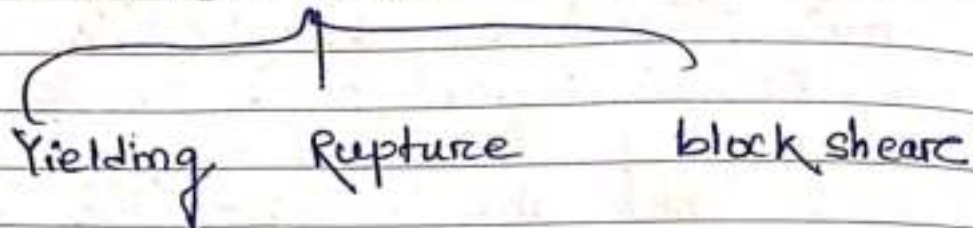
Design factored load = 1.5 DL + 1.5 LL

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

$$= 105\text{ kN}$$

2017							DECEMBER				
M	T	W	T	F	S						
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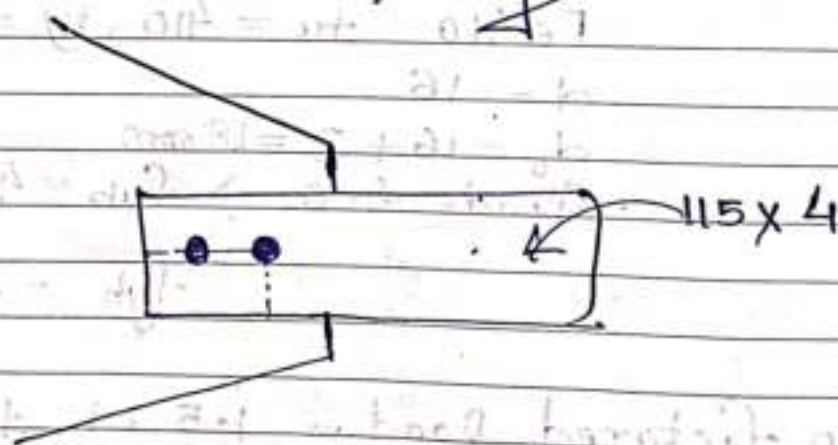
Tension member failure



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

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

$$\Rightarrow A_g = 462 \text{ mm}^2$$



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

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

$$\Rightarrow A_n = 355.69 \text{ mm}^2$$

FEBRUARY							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											

JANUARY

MONDAY

WEEK 05 • 029-336

29

Assume thickness of bar = 4mm

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

$$= \frac{462}{4}$$

$$= 115.5 \text{ mm}$$

$$= 115 \text{ mm}$$

check: $A_n = (b - n d_o) t$

$$= (115 - 1 \times 18) 4 \text{ mm}^2$$

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

Effective slenderness ratio

$$\lambda = \frac{l_{\text{eff}}}{r}$$

$A_{\text{net}} 180$
 $I_{\text{net}} 350$
 $W_{\text{net}} 400$
 Tensile Load

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

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

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

$$= 1.15 \text{ mm}$$

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

30

JANUARY

TUESDAY

030-335 • WEEK 05

2017

DECEMBER

M	T	W	T	F	S	S
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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

$$q = \frac{800}{1.15} = 695.65 \leftarrow A_{man} = 380$$

(Not safe)

Redesign

FEBRUARY							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							
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JANUARY

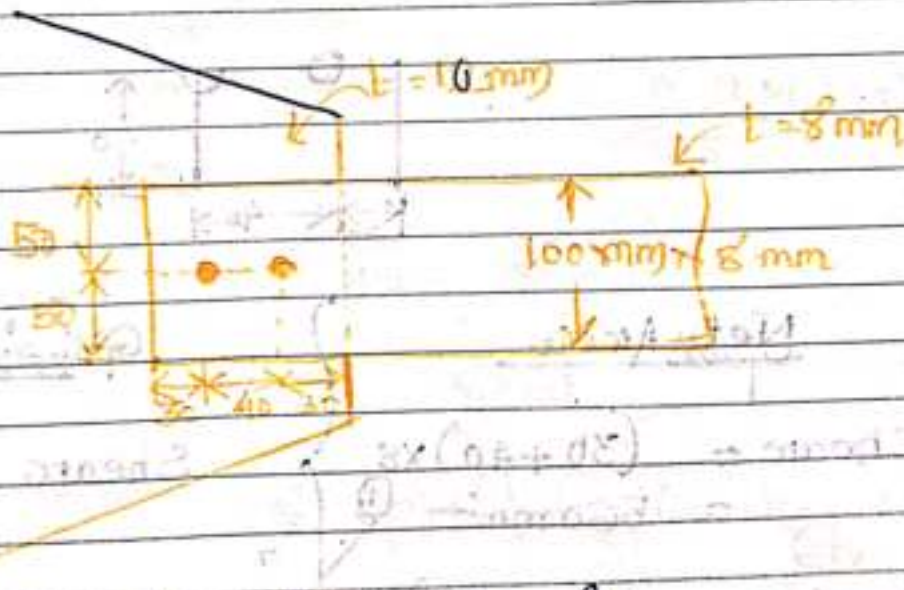
WEDNESDAY

WEEK 05 • 031-334

31

A tension member consists of a flat 100 mm x 8 m which is connected to a gusset plate of 10 mm 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 & Grades 4.6 bolts

Given



$$\text{Yielding Strength} = \frac{A_g f_y}{\gamma_{m0}}$$

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

1.1

$$= 181.81 \text{ KN}$$

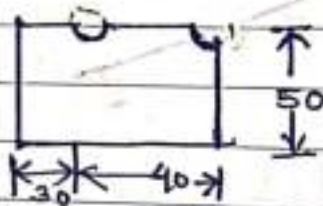
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29	30	31				

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

= $0.9 \times (100 - 1 \times 18)$

= 193.65 kN

Block Shear Strength



Gross Area

Net area

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

Shear = $(70 - 1.5 d_o) t$
 = $(70 - 1.5 \times 18) \times 8$
 = 344 mm^2

tension = 50×8 ②
 = 400 mm^2

Tension = $(50 - 0.5 \times 18) t$
 = 328 mm^2

(Block Strength) = $\frac{(Gross\ Shear) f_y}{\gamma_{mo}} + \frac{0.9 (Net\ Tension) f_u}{\gamma_{mb}}$

MARCH							2018						
M	T	W	T	F	S	S							
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12	13	14	15	16	17	18							
19	20	21	22	23	24	25							
26	27	28	29	30	31								

FEBRUARY

FRIDAY
WEEK 05 • 033-332

02

$$= 163218.29 \text{ N}$$

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

$$= 170.31 \text{ kN}$$

$$\text{(Block Strength)} = \frac{(400) \times 250}{\sqrt{3} \times 1.1} +$$

$$\frac{0.9 \times (344) \times 410}{\sqrt{3} \times 1.25}$$

$$= 149.54 \text{ kN}$$

Strength of tension members

- = min^m of
 - (i) Yielding 181.81 kN
 - (ii) rupture 193.65 kN
 - (iii) Block shear 149.54 kN

$$= 149.54 \text{ kN}$$

03

FEBRUARY

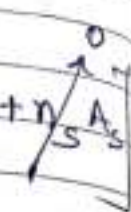
SATURDAY

034-331 • WEEK 05

2018

M	T	W	T	F	S	S
1	2	3	4	5	6	7
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15	16	17	18	19	20	21
22	23	24	25	26	27	28
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Strength of bolt

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


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

$$\text{Design shear per bolt} = 29 \text{ kN}$$

 K_b

$$\text{(i)} \frac{e}{3d_0} = \frac{30}{3 \times 18} = 0.555$$

$$\text{(ii)} \frac{p}{3d_0} = 0.25 = \frac{40}{3 \times 18} = 0.74$$

$$\text{(iii)} \frac{f_{ub}}{f_u} = \frac{400}{410} = 0.9$$

$$\text{(iv)} 1$$

$$K_b = 0.491$$

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

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

$$= 51.54 \text{ kN}$$

2018

MARCH							2018	
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19	20	21	22	23	24	25		
26	27	28	29	30	31			

FEBRUARY

SUNDAY

WEEK 05 • 035-330

04

$$\text{Bolt Strength} = \min \left\{ \begin{array}{l} \textcircled{1} \text{ Shear} = 29 \text{ kN} \\ \textcircled{2} \text{ Bear} = 51.54 \text{ kN} \end{array} \right.$$

$$= 29 \text{ kN}$$

$$2 \text{ Bolts} = 2 \times 29 = 58 \text{ kN}$$

$$\text{Max}^m \text{ load joint can carry} = \min \left\{ \begin{array}{l} \textcircled{1} \text{ strength of tension member} = 149.54 \text{ kN} \\ \textcircled{2} \text{ strength of bolt} = 58 \text{ kN} \end{array} \right.$$

$$= 58 \text{ kN}$$

MARCH							2018						
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26	27	28	29	30	31								

FEBRUARY

TUESDAY

WEEK 06 • 037-328

06

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

Channel Section

(i) Assume design compressive stress 80 MPa

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

(ii) Area Reqd

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

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

$$= 5000 \text{ mm}^2$$

Adopt

ISM C 350 @ 413 N/m

$$\text{Area} = 5366 \text{ mm}^2$$

07

FEBRUARY

WEDNESDAY

039-327 • WEEK 00

2018

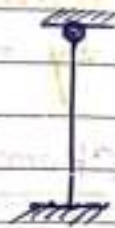
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15	16	17	18	19	20	21
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29	30	31				

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

$$r_{\min} = 28.3 \text{ mm}$$

(iii) For one end fixed & other end pinned

$$\begin{aligned} l_{\text{eff}} &= k \cdot L \\ &= 0.8L \\ &= 0.8 \times 4000 \\ &= 3200 \text{ mm} \end{aligned}$$



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

$$= \frac{3200}{28.3}$$

$$= 113.07 < \lambda_{\text{max}}^{\text{WT}} \quad (250)$$

(Design safe)

(iv) Channel Section \Rightarrow buckling class C

$$\text{For } \lambda = \frac{kl}{r} = 113.07$$

$$f_y = 250 \text{ MPa}$$

By Interpolation method:

λ	f_{red}
110	94.6
120	83.7

$\rightarrow 113.07$

2018

Interpolation in fx-991ES calculator

(i) Mode - 3 - 2 - Values - AC

(ii) (Write no) - shift + 1 - ~~FEBRUARY~~ - 5 - enter

or 5 - 5 - enter

THURSDAY

WEEK 06 • 039-326

08

MARCH							2018						
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5	6	7	8	9	10	11							
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19	20	21	22	23	24	25							
26	27	28	29	30	31								

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

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

$$= 5366 \times 91.25$$

$$= 489.65 \text{ kN} > 400 \text{ kN} \text{ (O.K.)}$$

* Check for limiting thickness *Explain*

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

From steel table $b_f = 100 \text{ mm}$

$$h = 350 \text{ mm}$$

$$t_f = 13.5 \text{ mm}$$

$$t_w = 8.1 \text{ mm}$$

$$R_1 = 14 \text{ mm}$$

Here,

$$b = 100 \text{ mm}$$

$$d = h - 2(t_f + R_1)$$

$$= 350 - 2(13.5 + 14)$$

$$= 295$$

For channel Section

$$\frac{b}{t_f} = \frac{100}{13.5} = 7.30 < 15.7 \epsilon = 15.7 \text{ } \left. \vphantom{\frac{b}{t_f}} \right\} \text{O.K.}$$

$$\frac{d}{t_w} = \frac{295}{8.1} = 36.42 < 42 \epsilon = 42$$

09

FEBRUARY

FRIDAY

040-325 • WEEK 00

2018

M	T	W	T	F	S	S
1	2	3	4	5	6	7
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22	23	24	25	26	27	28
29	30	31				

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 uncasted column in a single storey building.

Given: ISHB 250 @ 536.6 N/m

$$L = 4\text{m}$$

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

From steel Table:

$$h = 250\text{mm}$$

$$b_f = 250\text{mm}$$

$$t_f = 9.7$$

$$t_w = 8.8$$

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

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

$$t_f = 9.7\text{mm} \leq 100\text{mm}$$

\Rightarrow Buckling ^{clay} about Z-Z axis = b
Y-Y axis = C

Design Compressive Stress

About X-X axis

About Y-Y axis

Slenderness ratio

Slenderness ratio

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

$$\lambda_{y-y} = \frac{l_{eff}}{r_{yy}}$$

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

$$= 24.29$$

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

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

$$= 48.41$$

Forc, $f_y = 250 \text{ N/mm}^2$

For $f_y = 250 \text{ MPa}$
Buckling Class 'C'

Buckling class 'b'

λ_{x-x}	$f_{cd} \text{ N/mm}^2$
20	225
24.29	?
30	216

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

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

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

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

$$\Rightarrow y = 185.3 \text{ N/mm}^2$$

$$\Rightarrow y = 225 = (24.29 - 20) \frac{-9}{10} - 0.9$$

$$\Rightarrow y = 221.13 \text{ N/mm}^2$$

11

FEBRUARY

SUNDAY

042-323 • WEEK 06

2018						JANUARY					
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			

Design compressive load

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

$$= 6971 \times 221.13$$

$$= 1541.5 \text{ kN}$$

Design Compressive Load

$$P_d = A f_{cd}$$

$$= 6971 \times 185.3$$

$$= 1291.7 \text{ kN}$$

∴ Design Compressive load

= min of

①

1541.5 kN

②

1291.7 kN

= 1291.7 kN

= 1292 kN

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

MONDAY

WEEK 07 • 043-322

12

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 position & restrained against rotation at both the ends. Take $f_y = 250\text{MPa}$ & assume wind/earthquake actions.

Given: $P = 350\text{kN}$

$L = 3.5\text{m}$

Restrained at both ends $k_{eff} = 0.65$

$f_y = 250\text{MPa}$

Channel Section (buckling class 'c');

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

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

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

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

$= 4375\text{mm}^2$

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

13

FEBRUARY

TUESDAY

044-321 • WEEK 07

2018

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

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

$r_{\min} = 26.1 \text{ mm}$

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

$= \frac{0.65 \times 3.5 \times 10^3}{26.1}$

$= 87.165 < 250$

(Design safe)

λ

f_{cd}

80

136

87.165

90

121

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

\Rightarrow

$\gamma = 125.25 \text{ N/mm}^2$

MARCH							2018						
M	T	W	T	F	S	S							
			1	2	3	4							
5	6	7	8	9	10	11							
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26	27	28	29	30	31								

FEBRUARY

WEDNESDAY
WEEK 07 • 045-320

14

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

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

$$= 4564 \times 125.25$$

$$= 571.65 \text{ kN} > 350 \text{ kN}$$

(Design Safe)

* Check for limiting thickness = ?

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

$$h = 300 \text{ mm}$$

$$b_f = 90 \text{ mm}$$

$$t_f = 13.6 \text{ mm}$$

$$t_w = 7.6 \text{ mm}$$

$$R_1 = 13 \text{ mm}$$

Here, $b = 100 \text{ mm}$

$$d = h - 2(t_f + R_1)$$

$$= 300 - 2(7.6 + 13)$$

$$= 246.8 \text{ mm}$$

For channel sectⁿ $b = b_f$

$$\frac{b}{t_f} = 90/13.6 = 6.6 < 15.7\epsilon = 15.7 \quad (0. \text{K})$$

$$\frac{d}{t_w} = 246.8/7.6 = 32.47 < 42\epsilon = 42 \quad (0. \text{K})$$

MARCH							2018						
M	T	W	T	F	S	S	M	T	W	T	F	S	S
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26	27	28	29	30	31								

FEBRUARY

FRIDAY
WEEK 07 • 047-318

16

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 $\therefore D_f = 200 \text{ mm}$
Axial load = 400 kN

Assume

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

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

Step-1

Load = 400 kN

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

Base plate : $L_B = 350 \text{ mm}$

$B_B = 300 \text{ mm}$

$$A_{\text{provided}} = 350 \times 300 = 105000 \text{ mm}^2$$

Step-2

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

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

$$= 3.8 \text{ N/mm}^2 < \sigma_c \text{ 2018 (O.K.)}$$

17

FEBRUARY

SATURDAY

048-317 • WEEK 07

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			

Step-3 Thickness of base plate

$$T_B = \sqrt{\frac{3W}{6s} \left(\frac{a^2 - b^2}{4} \right)}$$

a = Greater projection beyond column face

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

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

$$= 75 \text{ mm}$$

b = Smaller projection beyond column face

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

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

$$= 50 \text{ mm}$$

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

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

Base plate = 350 mm x 300 mm x 40 mm

2018 Use 150 mm x 150 mm x 8 mm cleat angle

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

SUNDAY

WEEK 07 • 049-316

18

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
 $P = 1500 \text{ kN}$
 M30 concrete

Bearing strength of concrete = $0.45 f_{ck}$
 $= 0.45 \times 30$

$= 13.5 \text{ N/mm}^2$

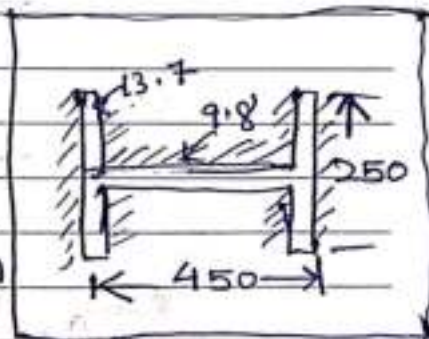
Properties;

$t_f = 13.7 \text{ mm}$

$t_w = 9.8 \text{ mm}$

$D_w = 450 \text{ mm}$

$b_f = 250 \text{ mm}$



Step-1

Reqd area for slab base $\leftarrow 500 \rightarrow$

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

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

$$= 111111.11 \text{ mm}^2$$

2018

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			

Let's provide square base plate

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

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

$$= 333.33 \text{ mm}$$

$$\approx 420 \text{ mm}$$

> 450 mm
> 25 mm

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

500 x 300 mm

Step II The bearing pressure of concrete

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

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

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

$$= 10.8 \text{ N/mm}^2 < 13.5 \text{ (O.K)}$$

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

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

$$b = \text{smaller projection} = \frac{B - b}{2}$$

$$= \frac{450 - 420}{2}$$

$$\frac{300 - 250}{2}$$

$$= 15 \text{ mm}$$

$$= 25$$

MARCH							2018	
M	T	W	T	F	S	S		
			1	2	3	4		
5	6	7	8	9	10	11		
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19	20	21	22	23	24	25		
26	27	28	29	30	31			

FEBRUARY

TUESDAY

WEEK 08 • 051-314

20

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

$$= \sqrt{2.5 \times \frac{8 \times 5}{10} (25^2 - 0.3 \times 25^2) \frac{1.1}{250}} > 13.7$$

$$= \text{~~13.7~~} 6.47 \text{ mm} < t_f$$

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

Slab base : 500 mm x 300 mm x 15 mm

Step - III

Connection of column to the base plate is

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

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

$$= 1525.6 \text{ mm}$$

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			

$$L_w = 1525.6 \text{ mm} \quad 1825.6 \text{ mm}$$

$$t_w = k.s$$

$$= 0.7s$$

$$\text{Area} = L_w \cdot t_w$$

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

$$\begin{aligned}
 &= \frac{1525.6 \times 0.7s}{1.25} \\
 &= 1525.6 \times 0.7s \times \frac{f_u/\sqrt{3}}{1.25} \\
 &= 202230.9s
 \end{aligned}$$

$$P = 202230.9s$$

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

$$\Rightarrow s = 7.4$$

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

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

$$= 13.5$$

Adopt Size of weld = 8mm

No. of total end returns = 12
Effective length of weld

$$\begin{aligned}
 (L_w)_{\text{eff}} &= 1825.6 \times 2 \times 8 \times 12 \\
 &= 1833.6 \text{ mm}
 \end{aligned}$$

MARCH							2018
M	T	W	T	F	S	S	
			1	2	3	4	
5	6	7	8	9	10	11	
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19	20	21	22	23	24	25	
26	27	28	29	30	31		

FEBRUARY

THURSDAY
WEEK 08 • 053-312

22

$$t_w = k \cdot s = 0.7 \times 8 = 5.6 \text{ mm}$$

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

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

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

$$\Rightarrow (L_w)_{\text{Required}} = 1414.46 \text{ mm}$$

~~$$(L_w)_{\text{eff}} = 1333.6$$

(Not O.K.)~~

Revision

weld size = 10 mm

$$(L_w)_{\text{eff}} = 1525.6 - (2 \times 10 \times 12) = 1285.6$$

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

$$L_w = 1131.56 \text{ mm} \leftarrow (L_w)_{\text{eff}} \text{ (O.K.)}$$

$$(L_w)_{\text{eff}} > L_w_{\text{Reqd}} \text{ (O.K.)}$$

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			

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
 $P = 1500 \text{ kN}$
 M25

$t_f = 11.6 \text{ mm}$
 $t_w = 10.1$
 $D = 350$
 $b_f = 250 \text{ mm}$

Step-I

Bearing strength of concrete

$$= 0.45 f_{ck}$$

$$= 0.45 \times 25$$

$$= 11.25 \text{ N/mm}^2$$

Area Reqd for slab base

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

$$= 133333.33 \text{ mm}^2$$

Provide base plate $420 \text{ mm} \times 320 \text{ mm}$

$$A = 134400 \text{ mm}^2$$

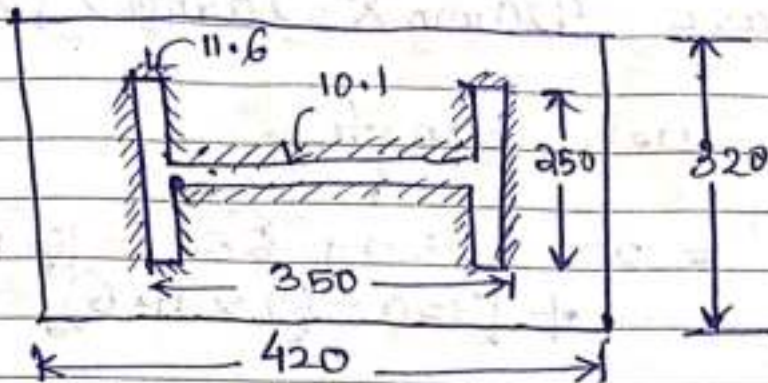
MARCH							2018
M	T	W	T	F	S	S	
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12	13	14	15	16	17	18	
19	20	21	22	23	24	25	
26	27	28	29	30	31		

FEBRUARY

SATURDAY

WEEK 08 • 055-310

24



Step - II

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

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

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

Step - III

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

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

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

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

$$= 10.26 < 11.6$$

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

2018

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			

Slab base $420\text{mm} \times 320\text{mm} \times 12\text{mm}$

Available weld length

$$= 250 + 250 + (250 - 10 \cdot 1) + (250 - 10 \cdot 1) + [350 - (2 \times 11.6)]$$

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

$$= 1633.4\text{mm}$$

Plate 12mm , Min^m weld size = 5mm

Max^m " " " = $12 - 1.5 = 10$

Adopt^y weld size = 6mm

$$t_w = k_s s$$

$$= 0.7 \times 6$$

$$= 4.2$$

$$(L_w)_{\text{eff}} = 1633.4 - (2 \times 6)$$

$$= 1489.4$$

P = weld strength

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

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

$$\Rightarrow L_w = 1885 \quad (\text{not o.k.})$$

Revision:

$$s = 8\text{mm}$$

$$t_w = 0.7s = 5.6$$

$$(L_w)_{\text{eff}} = 1633.4 - (2 \times 8 \times 6)$$

$$= 1441.4$$

MARCH							2018			
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12	13	14	15	16	17	18				
19	20	21	22	23	24	25				
26	27	28	29	30	31					

FEBRUARY

MONDAY

WEEK 09 • 057-308

26

$P = \text{weld strength}$

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

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

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

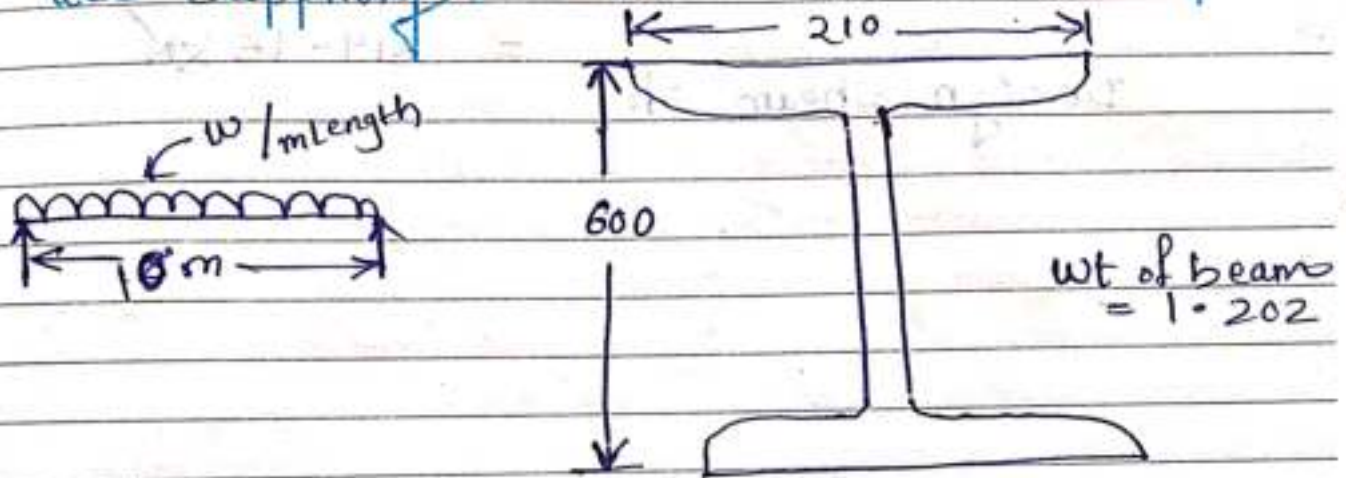
MARCH							2018	
M	T	W	T	F	S	S		
			1	2	3	4		
5	6	7	8	9	10	11		
12	13	14	15	16	17	18		
19	20	21	22	23	24	25		
26	27	28	29	30	31			

FEBRUARY

WEDNESDAY
WEEK 09 • 059-305

28

A laterally supported beam ISMB 600@1202.71 is placed between two supports. Determine the safe uniformly distributed load the beam can carry for an effective span of 10m. $f_y = 250 \text{ N/mm}^2$. Neglect web buckling & web crippling.



ISMB 600@1202.71 N/m

Step I: (Section Properties)

For ISMB 600@1202.71 N/m
Plastic section modulus $Z_p = 3510630 \text{ mm}^3$

Depth of section $h = 600 \text{ mm}$

Width of flange $b_f = 210 \text{ mm}$ $r_1 = 20 \text{ mm}$

Sectional Area $A = 15621 \text{ mm}^2$

Thickness of flange $t_f = 20.8 \text{ mm}$

Thickness of web $t_w = 12 \text{ mm}$

Depth of web $d = h - 2(t_f + r_1)$
 $= 600 - 2(20.8 + 20) = 518.4 \text{ mm}$

Moment of inertia about z axis
 $I_{zz} = 91813 \times 10^4 \text{ mm}^4$

Elastic section modulus

$Z_{zz} \text{ or } Z_e = 3060.4 \times 10^3 \text{ mm}^3$ 2018

03

SATURDAY

062-303 • WEEK 09

Step-II Sectⁿ Classification

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

Here,

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

For rolled section,

out stand of compression flange

$$\frac{b}{t_f} = \frac{105}{20.8} = 5.05 < 9.48$$

 \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)

APRIL						
M	T	W	T	F	S	S
						1
30	31	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				

Step-III Design bending strength

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

Assume, $V \leq 0.6V_d$

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

$\beta_b = 1$ for plastic section
 $\gamma_{mo} = 1.1$

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

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

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

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

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

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

$$M_d = 797.87 \text{ kN-m}$$

05

MARCH

MONDAY

064-301 • WEEK 10

2018

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

Step-IV Design Moment & load carrying Capacity

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

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

$$= 12.5 w_u$$

$$12.5 w' = 797.87$$

$$\Rightarrow w' = 63.83 \text{ kN/m}$$

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

$$= 42.55 \text{ kN/m}$$

Safe working load the beam can carry = w - self wt of beam

$$= 42.55 - 1.202$$

$$= 41.35 \text{ kN/m}$$

Step-V check for Shear

$$\text{Factored shear } V = \frac{w'l}{2}$$

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

$$= 319.15 \text{ kN}$$

APRIL 2018

M	T	W	T	F	S	S
						1
30	31	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				

MARCH

TUESDAY

WEEK 10 • 065-300

06

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

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

$= 944.75 \text{ kN} > V \text{ (O.K.)}$

$0.6 V_d = 0.6 \times 944.75$

$= 566.85 \text{ kN} > V \text{ (O.K.)}$

Step-VI Load carrying capacity from deflection criteria:-

For UDL:-

$\delta_{max} = \frac{5 W' L^4}{384 E I}$

$\delta_{permissible} = \frac{\text{Span}}{240}$
elastic cladding

For point load

$\delta_{max} = \frac{W L^3}{48 E I}$

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

$\delta_{max} = \frac{5 \times W' \times (10,000)^4}{38 \times (2 \times 10^5) \times I_{zz}}$

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

$= 41.67 \text{ mm}$

$= \frac{5 W' \times (10,000)^4}{38 \times 2 \times 10^5 \times 91813 \times 10^4}$

$= 0.709 W'$

07

MARCH

WEDNESDAY

066-299 • WEEK 10

2018

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

$$0.709w' = 41.67$$

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

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

$$= 39.18 \text{ KN/m}$$

$$\text{Safe working load} = w - \text{self wt. of struct}^n$$

$$= 39.18 - 1.202$$

$$= 37.9 \approx 38 \text{ KN/m}$$

APRIL						
M	T	W	T	F	S	S
						1
30	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

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-1

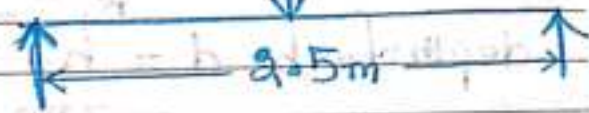
Max^m bending Moment

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

$$= \frac{300 \times 10^3 \times 2.5 \times 10^3}{4} \text{ N-mm}$$

$$= 187.5 \times 10^6 \text{ N-mm}$$

300kN



Max^m Shear

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

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

$$= 150 \text{ kN or } 150 \times 10^3 \text{ N}$$

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

$$= \frac{187.5 \times 10^6 \times 1.1}{250}$$

$$= 825 \times 10^3 \text{ mm}^3$$

09

MARCH

FRIDAY

068-297 • WEEK 10

2018

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

Step-II

Adopt

ISLB 350 @ 0.486 kN/m

$$b_f = 165 \text{ mm}$$

$$h_f = 350 \text{ mm}$$

$$t_f = 11.4 \text{ mm}$$

$$f_{tw} = 7.4 \text{ mm}$$

$$R_1 = 16 \text{ mm}$$

$$\text{depth of web } d = h - 2(t_f + R_1)$$

$$= 350 - 2(11.4 + 16)$$

$$= 295.2 \text{ mm}$$

Fore Plastic Sectⁿ:

$$\text{Flange } \frac{b_f}{t_f} = \frac{b_f/2}{t_f} = \frac{165/2}{11.4} = 7.2 < \frac{\epsilon}{\gamma} = \frac{\sqrt{\frac{250}{f_y}}}{\gamma} = \frac{\sqrt{\frac{250}{250}}}{1} = 1$$

$$\text{Web } \frac{d}{t_w} = \frac{295.2}{7.4} = 40 < 84 \epsilon = 84 \times 1 = 84$$

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

$$\text{Sect}^n \text{ modulus } Z_{zz} \text{ or } Z_e = 751.9 \times 10^3 \text{ mm}^3$$

$$Z_p = 851.11 \times 10^3 \text{ mm}^3$$

APRIL							2018	
M	T	W	T	F	S	S		
						1		
30	31							
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		

MARCH

SATURDAY
WEEK 10 • 069-296

10

Step-III Check for Section

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

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

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

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

Step-IV Check for sheare

$$V = 150 \text{ kN}$$

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

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

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

$$0.6 V_d = 0.6 \times 340$$

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

11

MARCH

SUNDAY

070-295 • WEEK 10

2018

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

Step-V Check for design capacity of section

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

$$\Leftrightarrow M_d = \phi_b Z_p \frac{f_y}{\gamma_m} < 1.2 Z_e f_y$$

$$\Leftrightarrow M_d = 1 \times 851.11 \times 10^3 \times 250 < 1.2 \times 951.11 \times 10^3$$

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

Step-VI Check for deflection

S.S. beam
point load

$$\therefore \delta_{\max} = \frac{WL^3}{48EI}$$

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

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

$$= 3.71 \text{ mm}$$

APRIL							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	

MARCH

MONDAY

WEEK 11 • 071-294

12

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

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

$$= 8.33$$

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

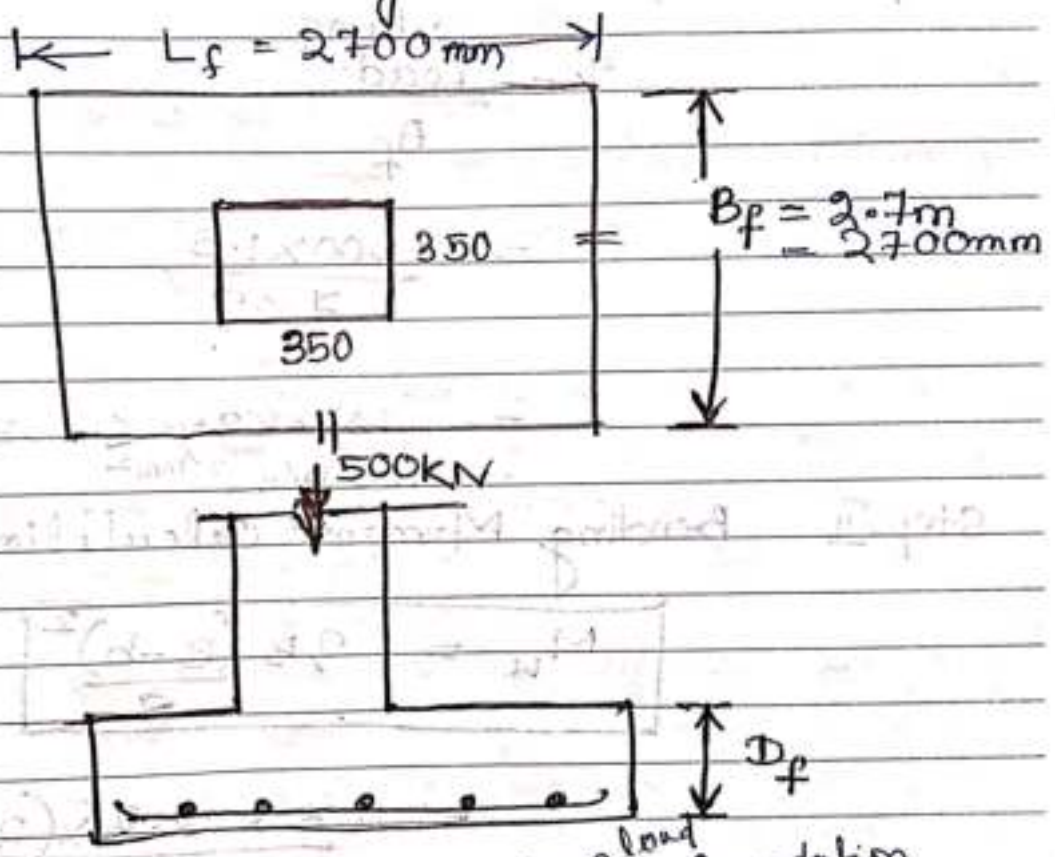
APRIL							2018
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30						1	
2	3	4	5	6	7	8	
9	10	11	12	13	14	15	
16	17	18	19	20	21	22	
23	24	25	26	27	28	29	

MARCH

WEDNESDAY
WEEK 11 • 073-292

14

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



Step-I

Column load = 500kN → column load
 Total load = 500kN + 10% of 500kN → foundation load
 = 500kN + 50kN
 = 550kN

Ultimate load = 1.5 × 550
 = 825 kN

$A_{reqd} = \frac{\text{ultimate load}}{\text{SBC of soil}} = \frac{825}{120} \text{ m}^2$
 = 6.875 m²

15

MARCH

THURSDAY

074-291 • WEEK 11

2018

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

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

Upward Pressure (q)

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

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

$$= 102.88 \text{ N/mm}^2 < \text{SBC. of Soil} = 120$$

$$\approx 103 \text{ N/mm}^2 \text{ (O.K.)}$$

Step-II Bending Moment calculation

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

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

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

Step-III Depth Calculation

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

2018 $x_{u,max}$ for M20 & Fe 415 = 0.48d

2018

APRIL						
M	T	W	T	F	S	S
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16	17	18	19	20	21	22
23	24	25	26	27	28	29

MARCH

16

FRIDAY
WEEK 11 • 075-290

$$\Rightarrow 191.97 \times 10^6 = 0.36 \times 20 \times 0.48 \left[1 - 0.42 \times 0.48 \right] \times 2700 d^2$$

$$\Rightarrow d = 160 \text{ mm}$$

Twice Take, $d = 350 \text{ mm}$

effective cover = 50 mm (for footing)

$$D = 350 + 50$$

$$f = 400 \text{ mm}$$

Step - IV Steel Calculation

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

$$\Rightarrow 191.97 \times 10^6 = 0.87 \times 415 \times A_{st} \times 350 \times \left(1 - \frac{A_{st} \times 415}{2700 \times 350 \times 26} \right)$$

$$\Rightarrow 1519.14 = A_{st} \left(1 - 21.9 \times 10^{-6} A_{st} \right)$$

$$\Rightarrow 1519.14 = A_{st} - 21.9 \times 10^{-6} A_{st}^2$$

$$\Rightarrow 21.9 \times 10^{-6} A_{st}^2 - A_{st} + 1519.14 = 0$$

$$\Rightarrow A_{st} = 1573.35 \text{ mm}^2 \quad \& \quad \sqrt{\frac{44088}{4157}} \text{ mm}^2$$

(max^m)
2018

17

MARCH

SATURDAY

076-289 • WEEK 11

2018

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Main Reinforcement

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

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

$$= \therefore \therefore 30.06 \text{ mm}$$

25mm ϕ bars @ 50mm c/c

Longitudinal Reinforcement

$$\text{Area} = 0.12\% \text{ bD}$$

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

$$= 1296 \text{ mm}^2$$

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

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

$$= 235 \text{ mm}$$

12mm ϕ bars @ 250mm c/c

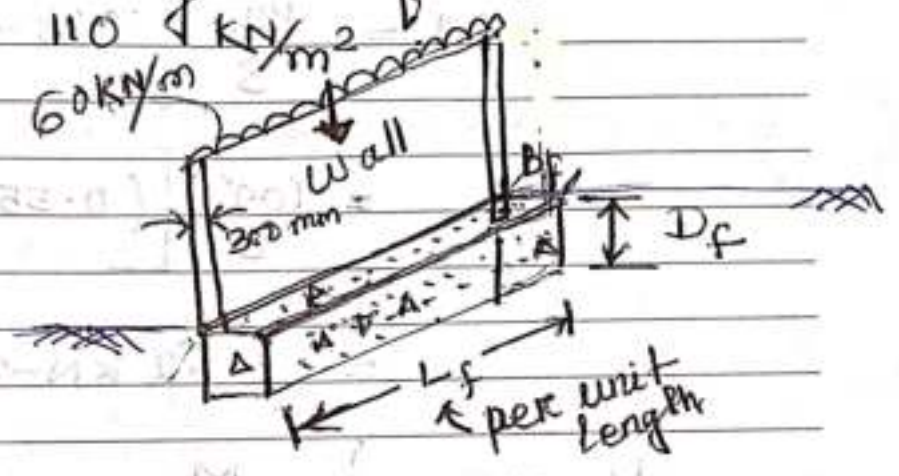
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MARCH

18

SUNDAY
WEEK 11 • 077-289

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



Wall thickness = 300 mm = 0.3 m

Load = 60 kN/m

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

Width of footing $B_f = \frac{60 \text{ kN/m}}{110 \text{ kN/m}^2} = 0.55 \text{ m}$
 $B_f = 550 \text{ mm}; L_f = 1 \text{ m}$

Net upward pressure

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

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

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

19

MARCH

MONDAY

078-287 • WEEK 12

2018

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Step-II

Max Bending Moment occurs at sectⁿ xx 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

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

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

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

Adopt $d = 350 \text{ mm}$

$$D = 350 + 50 = 400$$

Adopt

$$d = 600 \text{ mm}$$

effective cover = 50 mm (foundation)

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

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Step-IV Steel area calculation for foundation

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

$$\Rightarrow 1.62 \times 10^6 = 0.87 \times 415 A_{st} \times 600 \left(1 - \frac{A_{st} \times 415}{\frac{1670 \times 600 \times 26}{550 \times 350}} \right)$$

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

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

9359.43
9266

$$\Rightarrow A_{st} = 28978 \text{ mm}^2 \text{ or } \cancel{4.5 \text{ mm}^2}$$

Main reinforcement

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

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

10mm ϕ bar @ 50mm c/c

$$= 15 \text{ mm} \approx 50 \text{ mm}$$

Adopt: 22 ϕ bar @ 50mm c/c

Longitudinal reinforcement

$$\text{Area} = 0.12\% bD$$

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

8mm ϕ bar @ 150mm c/c

$$= 780 \text{ mm}^2$$

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

21

MARCH

WEDNESDAY

080-285 • WEEK 12

2018

FEBRUARY

M	T	W	T	F	S
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$$\text{Spacing} = \frac{1000 \times \frac{\pi}{4} \times 12^2}{780}$$

$$= 144 \text{ mm}$$

Adopt: 12 ϕ bars @ 144 mm c/c

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23	24	25	26	27	28	29

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 (15.41 kg/m)

$$\Rightarrow \text{If length of bar} = 4.2 \text{ m}$$

$$\text{Mild steel round bar weight}$$

$$= 4.2 \times 15.41$$

$$= 64.722 \text{ kg}$$

23

MARCH

FRIDAY

082-283 • WEEK 12

2018

FEBRUARY

M	T	W	T	F	S	S
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7.2 Tube columns & compression members crinkling

Compress³ 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 at collapse load

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

radius $\quad \quad \quad \frac{1}{m} = \text{poisson's ratio}$

7.3 Tube tension members & tubular roof trusses

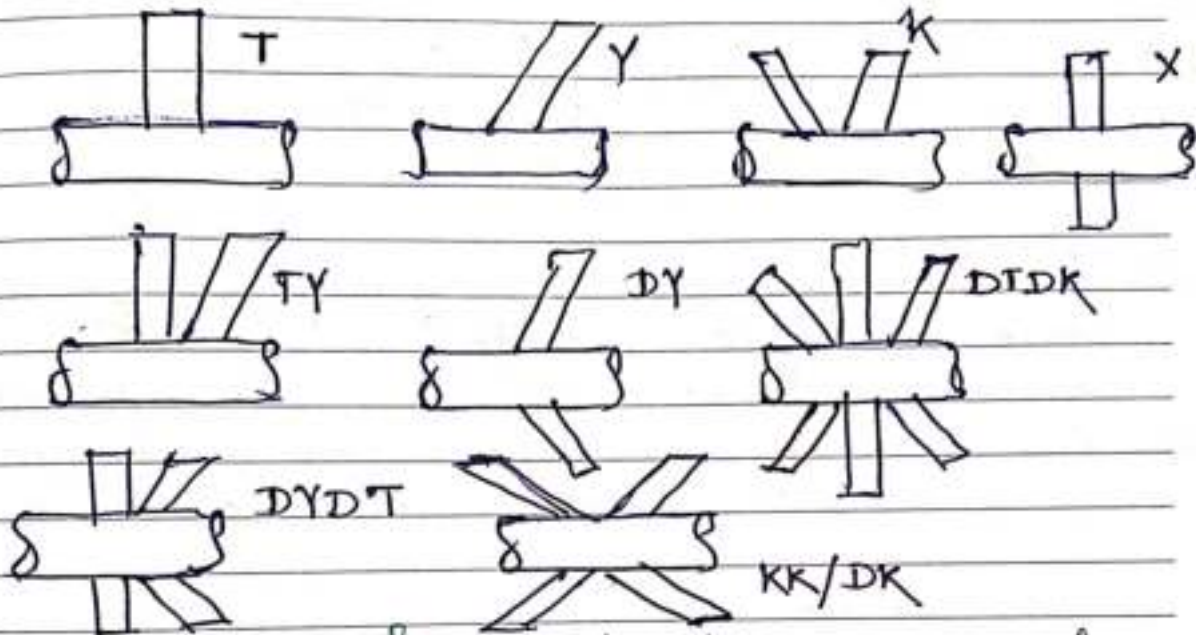
- The effectiveness of a member subjected to axial tension depends on c/s area.

- So rolled steel section / tubular section has same value of tensile resistant.

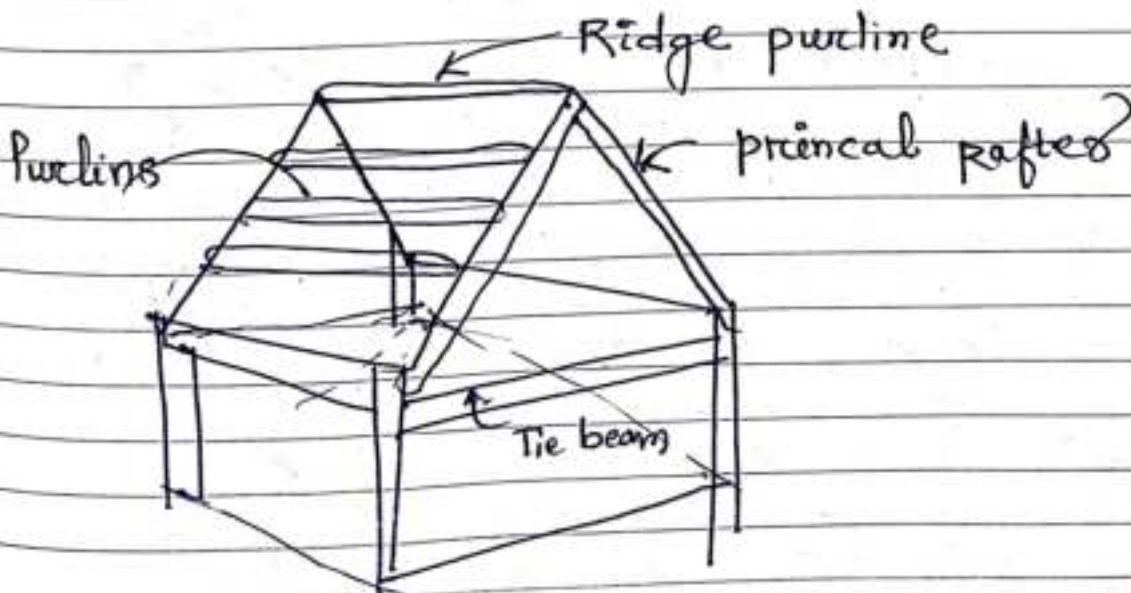
- Roof trusses are subjected to both axial compressive & tension force only

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7.4 Joints in tubular trusses



7.5 Design of tubular beams & purlins.



APRIL
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JUNE

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27	28	29				

Ch-8 Design of Timber Structure

- The wood which is used as structural material is called timber.

- Use Code IS 883 : 1974

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) 2.8m (iii) 4m. Assume inside location & standard grade.

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

$$f_{cp} = 10.6 \text{ MPa}$$

$$E = 12670 \text{ MPa (IS-code Tabled)}$$

(i) Unsupported length of column $S = 1.5\text{m} = 1500\text{mm}$
Least lateral dimension of the column
 $d = 150\text{mm}$

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

⇒ short column

* 3 conditions

short column

$$\frac{S}{d} < 11$$

intermediate column

$$11 < \frac{S}{d} < K_8$$

Long column

$$\frac{S}{d} > K_8 = 0.584 \sqrt{\frac{E}{f_c}}$$

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19	20	21	22	23	24
26	27	28			

MARCH

27

TUESDAY

066-279 • WEEK 13

For short column,

$$\text{Allowable stress } f_c = \frac{f}{C_p}$$

$$= 10.6 \text{ MPa}$$

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

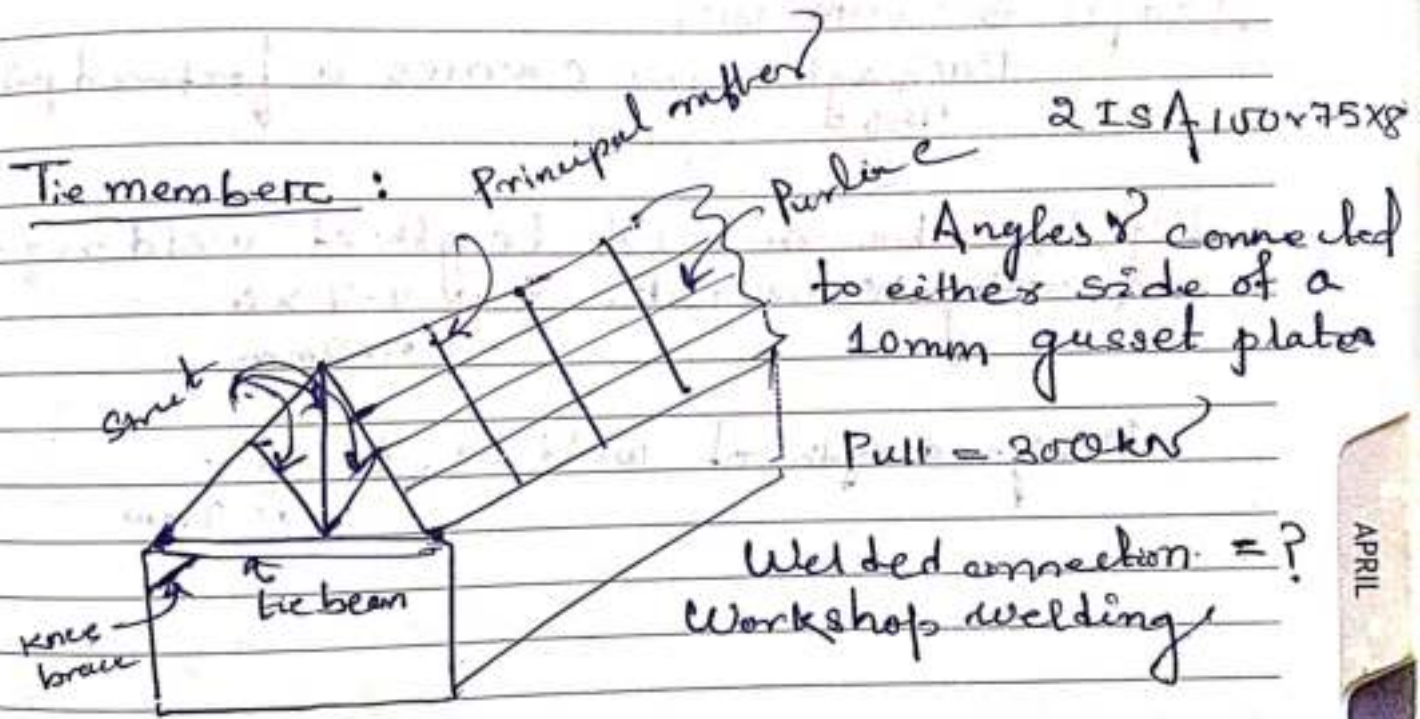
$$= (150 \times 200) \times 10.6$$

$$= 318000 \text{ N}$$

$$= 318 \text{ kN}$$

(ii)

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16	17	18	19	20	21	22
23	24	25	26	27	28	29



Working load = 300kN
Factored load = 450kN = 1.5 x 300kN

Thickness of weld :

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

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

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

APRIL
MAY
JUNE

29

MARCH

THURSDAY

088-277 • WEEK 13

2018

M	T	W	T	F	S	S
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5	6	7	8	9		
12	13	14	15	16		
19	20	21	22	23		
26	27	28				

(ii) At top, the thickness should not exceed

$$a = t - 1.5$$

$$= 8 - 1.5$$

$$= 6.5 \text{ mm}$$

Adopt $\phi = 6 \text{ mm}$ weld

Each angle carries carries a factored pull

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

Let, $L_w =$ be the total length of weld reqd

Assuming normal weld $t = 0.7 \times \phi$

$$= 4.2 \text{ mm}$$

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