USN 1CR14CV051

10CV72

Seventh Semester B.E. Degree Examination, Dec.2018/Jan.2019

Design of Steel Structures

Time: 3 hrs. Max. Marks:100

Note: 1. *Answer any FIVE full questions, selecting at least TWO questions from each part.*
2. *Use of IS-800-2007 and steel tables permitted.*

PART – A

1 a. Describe briefly advantages and disadvantages of steel structures. 6 **(06 Marks)**
b. Explain the load combinations adopted in the design of steel structures. 8 **(06 Marks)**
c. Explain the four classes of section as per IS-800-2007. 6 **(08 Marks)**

2 a. Explain the various modes of failure of bolted connections with neat sketches. 3 **(06 Marks)**
b. Determine the strength and efficiency of lap joint consists of 10 mm and 8 mm thick plates. Use M18 grade 5.6 black bolts and Fe440 grade plate. 7

Fig.Q2(b) **(14 Marks)**

3 a. Describe briefly advantages and disadvantages of welded connections. 3 **(06 Marks)**
b. Determine the bracket load "P" the column can carry. Take size of weld as 8 mm.

8

Fig.Q3(b) **(14 Marks)**

4 a. Determine the shape factor of a 'T' section having a flange width 200 mm and 20 mm thick with a web of 10 mm thick and 180 mm depth. **(06 Marks)**
b. Calculate "plastic moment" for the beam as shown in Fig.Q4(b). Use load factor = 2.

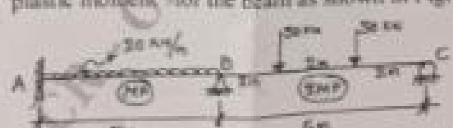

Fig.Q4(b) **(14 Marks)**

PART – B

5 Design a tie or tension member using double angle (equall legs) section connected on either side of gusset plate subjected to a force of 300 kN use suitable welded connection. If the length of member is 3m. Check for reversal of stresses. **(20 Marks)**

6 Design a compression member using double channel section "face to face" to carry a factored load of 1600 kN. The length of the column is 5m with one end is fixed and other end is hinged. Also design single lacing system. **(20 Marks)**

7 Design a column base (slab base) and concrete base for a column ISHB400 subjected to an axial load of 1000 kN. Use M20 grade concrete and safe bearing capacity of soil is 200 kN/m^2. Draw neat sketch. Use welded connection. **(20 Marks)**

8 The RCC floor of a classroom 6m × 12m is supported on beam kept at 3m c/c. The beams are simply supported at ends over a span of 6m and rest on 300 mm thick masonry wall. Assuming the thickness of slab is 125 mm. live load on slab is 4 kN/m^2. Design an interior beam using IS specifications. Apply all the necessary checks. **(20 Marks)**

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SOLUTIONS

1 a Advantages of steel structures

- Less self weight but able to resist heavy loads
- Gives sufficient warning before failure because of its ductile property
- Because of ductile property it enable to yield locally at the point of high stress concentration
- Prevents premature failure
- Being light, steel members are continuosly hanled and transported for this reason prefabricated members can be frequently provided.
- Has longer life span if it is maintained properly
- Properties of steel mostly do not change with time
- They can be erected at faster rate
- Steel has highest scrap value amongst all building material
- Steel is a recyclable material

Disadvantages of steel structures

- When placed in exposed condition are subjected to corrosion
- Steel structures need fire proof treatment which increases the cost
- Fatigue of steel is one of the major drawback
- Fatigue involves a reduction in strength when steel is subjected to large number of variations of tensile stress.
- At the place of stress concentrations in steel sections under certain conditions, steel may lose its ductility.

1 b

Load combinations for working stress design (section 11) of IS 800 -2007. As per limit state of serviceability Load combinations IS 800-2007 Table 4 along with 33% increase in permissible stress as per IS 800 -2007 clause 11.1.4.

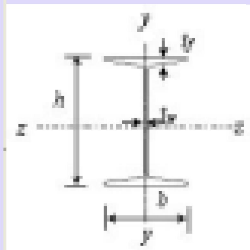
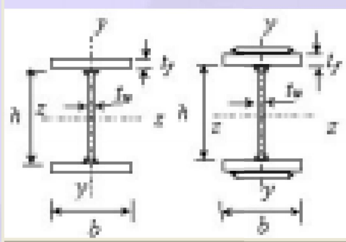
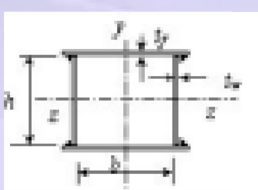
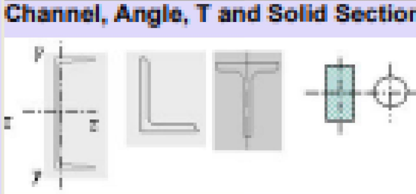
Dead load(DL) + Imposed load(IL) + wind load (WL) or Earth quake load (EL) = 1/1.33*(1.0DL + 0.8 IL + 0.8 WL or 0.8 EL) = 0.75DL + 0.6 IL + 0.6 WL or 0.6 EL

As per IS 875 Part 5 clause 8.1 along with 33% increase in permissible stress as per IS 800 -2007 clause 11.1.4.

Dead load(DL) + Imposed load(IL) + wind load (WL) or Earth quake load (EL) = 1/1.33*(1.0DL + 1.0 IL + 1.0 WL or 1.0 EL) = 0.75DL + 0.75 IL + 0.75 WL or 0.75 EL

1c

Table5.3 Buckling class of cross sections
(Section 7.1.2.2)

Cross Section	Limits	Buckling about axis	Buckling Class
Rolled I-Sections 	$h/b > 1.2$: $t_f \leq 40$ mm	z-z	a
		y-y	b
	40 mm< $t_f \leq$ 100 mm	z-z	b
		y-y	c
	$h/b \leq 1.2$: $t_f \leq 100$ mm	z-z	b
		y-y	c
	t_f >100 mm	z-z	d
		y-y	d
Welded I-Section 	$t_f \leq 40$ mm	z-z	b
		y-y	c
	t_f >40 mm	z-z	c
		y-y	d
Hollow Section 	Hot rolled	Any	a
	Cold formed	Any	b
Welded Box Section 	Generally (Except as below)	Any	b
	Thick welds and $b/t_f < 30$ $h/t_w < 30$	z-z	c
		y-y	c
Channel, Angle, T and Solid Sections 		Any	c

2 a Modes of failures of Bolted connections

FAILURE OF BOLTED JOINTS

The bolted joint may fail in any of the following six ways, out of which some failures can be checked by adherence to the specifications of edge distance. Therefore, they are not of much importance, whereas the others require due consideration.

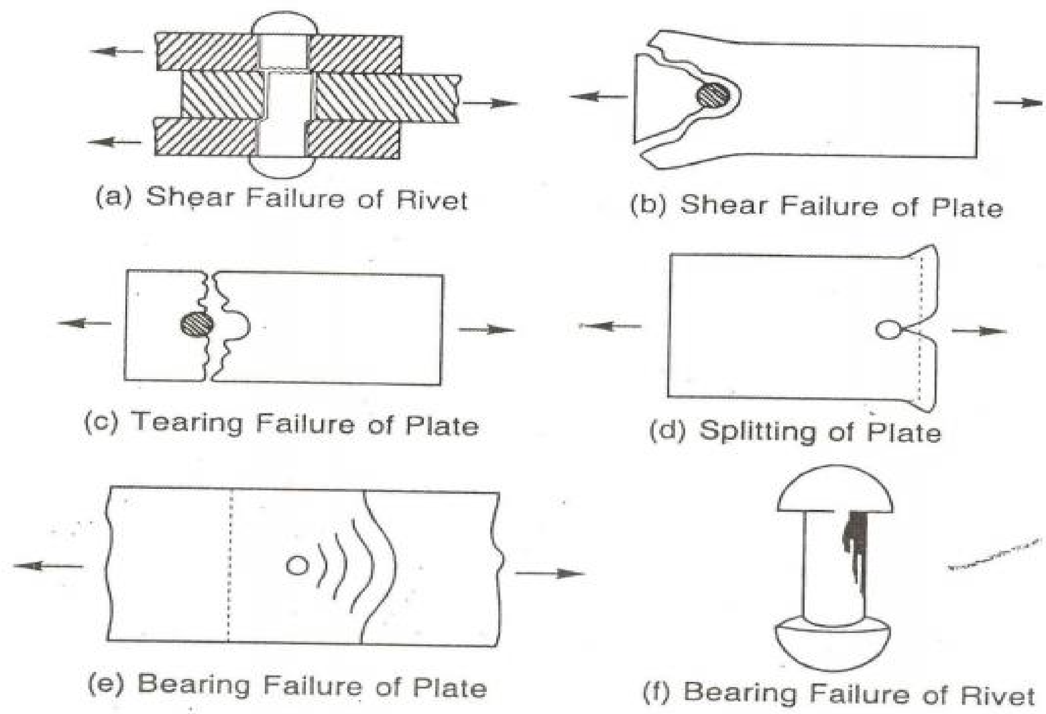
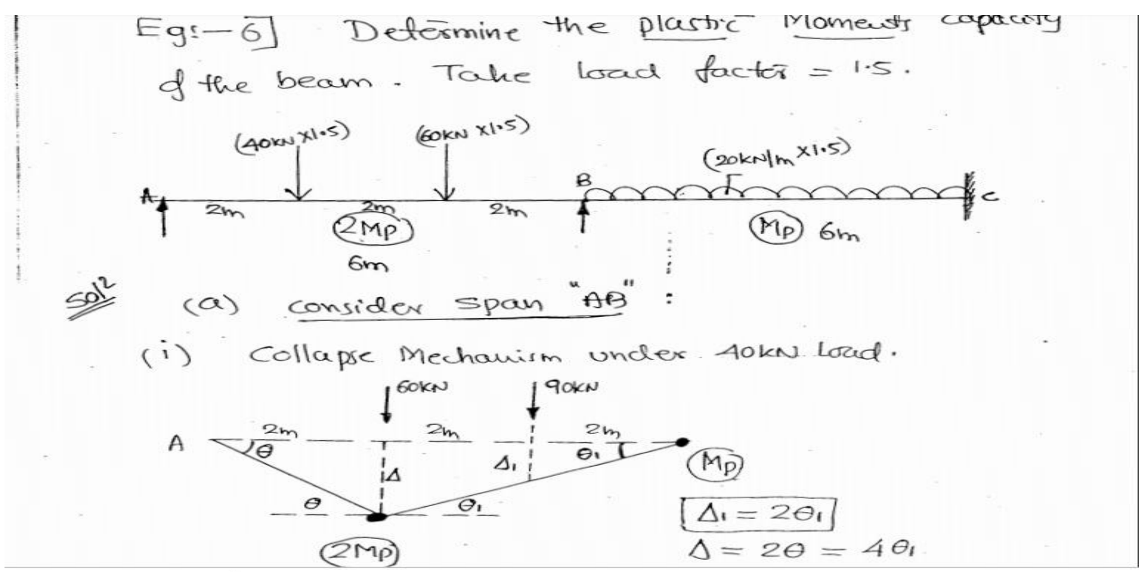


Fig. 2.3 Failure of Riveted Joints

2 b



$$EWD = 60 \times \Delta + 90 \times \Delta_1 = 60(4\theta_1) + 90(2\theta_1)$$

$$= (420)\theta_1 \rightarrow$$

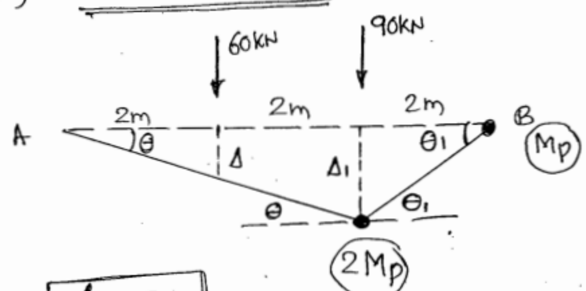
$$IWD = 2M_p(\theta + \theta_1) + M_p\theta_1$$

$$= 2M_p\theta + 3M_p\theta_1 = 2M_p(2\theta_1) + 3M_p\theta_1$$

$$= 7M_p\theta_1 \rightarrow$$

Equating, $7M_p\cancel{\theta_1} = 420\cancel{\theta_1}$ $\boxed{M_p = 60 \text{ kN-m}}$

(II) Under 60kN Load : (90kN)



$\boxed{\Delta = 2\theta}$

$\Delta_1 = 2\theta_1 = 4\theta$ $\therefore \boxed{\theta_1 = 2\theta}$

$$EWD = 60\,\Delta + 90\,\Delta_1 = 60(2\theta) + 90(4\theta) = (480)\theta \rightarrow$$

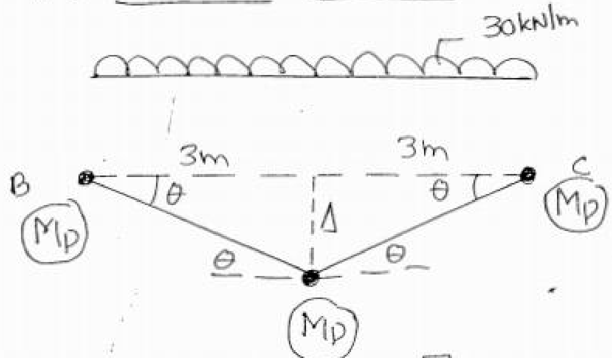
$$IWD = 2M_p(\theta + \theta_1) + M_p(\theta) = 3M_p\theta + 2M_p\theta_1$$

$$= 3M_p\theta + 2M_p(2\theta) = 7M_p\theta \rightarrow$$

Equating, $7M_p\cancel{\theta} = 480\cancel{\theta}$

$\boxed{M_p = 68.57\ kN\text{-}m}$

(b) Consider Span "BC" :



$\Delta = 3\theta$

$EWD = 30\left[\frac{1}{2} \times 6 \times \Delta\right] = 90(3\theta) = 270\,\theta \rightarrow$

$IWD = 4M_p\theta \rightarrow$

Equate $4M_p\cancel{\theta} = 270\cancel{\theta}$ $\boxed{M_p = 67.5\ kN\text{-}m}$

$f_u = 410$ Mpa.

$f_{ub} = 400$ Mpa.

$d = 20mm$

$d_o = 20+2 = 22mm.$

$A_{nb} = 245mm$

$\gamma_{mb} = 1.25$

The bolt will be in single shear.

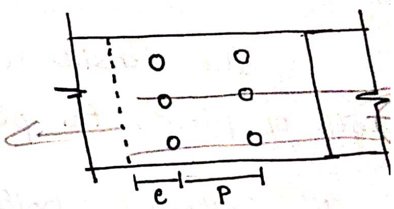
Design shear strength : Vdsb

$$V_{dsb} = \frac{V_{npb}}{\gamma_{mb}} = \frac{\frac{f_{ub}}{\sqrt{3}}(n_n A_n + n_s \overset{0}{\cancel{A_{ns}}})}{1.25} = \frac{\frac{400}{\sqrt{3}}(1 \times 0.78 \times \pi/4 \times 20^2)}{1.25}$$

$$\boxed{V_{dsb} = 45.26\ KN}$$

∴ for this shear force

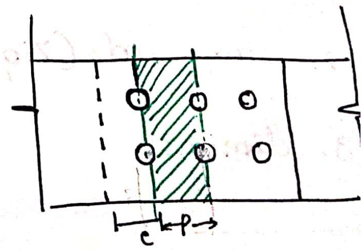
no. of bolt $= \frac{250}{45.26} = 5.52 \approx 6$ no.



∴ Arrange the bolts in three lines as

Now, Strength of joint per pitch length on the basis of shear or

$= 2 \times 45.26 = 90.52$ KN.



Equating this strength to the net Tensile strength of plate / pitch length.

$$T_{dn} = 0.9 \frac{f_u}{\gamma_{m1}}(p - d_o)t$$

$$90.52 \times 10^3 = 0.9 \times \frac{410}{1.25}(p - 22)8$$

$$p = 60.32mm, \approx 65mm \geq \overset{2.5 \times d}{2.5 \times 20} \text{ (i.e 50 mm)}$$

Now, edge distance available $= \frac{1}{2}(210 - 2 \times 65) = 40mm$

$\therefore K_b = \frac{e}{3d_0}, \frac{p}{3d_0} - 0.25, \frac{f_{ub}}{f_u}, 1$

$= \frac{40}{3 \times 22}, \frac{65}{3 \times 22} - 0.25, \frac{400}{410}, 1$

$= 0.60, 0.75, 0.975, 1$

Hence $K_b = 0.60$.

$\therefore$ Strength of bolt in bearing.

$V_{dpb} = 2.5 K_b d t \frac{f_u}{\gamma_{mb}}$

$= 2.5 \times 0.6 \times 20 \times 8 \times \frac{410}{1.25} \times 10^{-3}$

$= 78.72$ KN. $>$ Design Shear Strength $= 45.26$ KN.

Hence, the design needs no revision.

3a Advantages of Welding

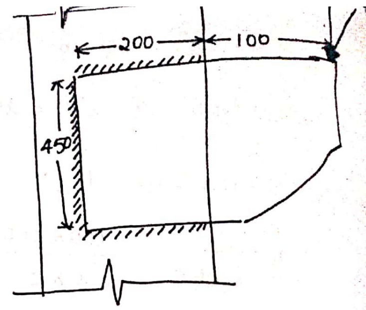
1. No hole is required for welding
2. Reduced overall weight of the structure.
3. Less material is required.
4. It is more than that of the riveted joint.
5. The speed of fabrication is faster in comparison

Disadvantages of Welding Joints

1. Welded joints are more brittle and therefore their fatigue strength is less than the members joined.
2. Due to uneven heating & cooling of the members during the welding, the members may distort resulting in additional stresses.
3. Skilled labor and electricity are required for welding.

4. No provision for expansion and contraction is kept in welded connection & therefore, there is possibility of racks.

3b



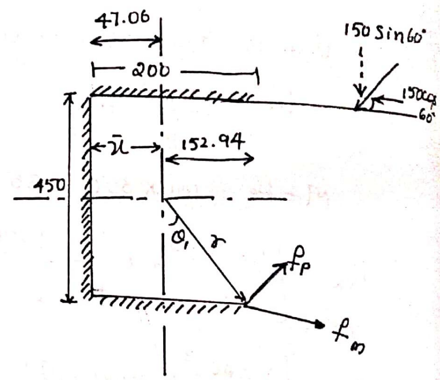
$$\bar{x} = \frac{(2 \times 200 \times t \times 100) + 450 \times t \times t/2}{(2 \times 200 \times t) + (450 \times t)}$$

$\bar{x} = 47.06\,mm.$

$e_x = 100 + (200 - 47.06) = 252.94\,mm$

$e_y = \frac{450}{2} = 225\,mm$

Total length of weld $= 200 \times 2 + 450 = 850\,mm$



* Direct Shear Stress (f_p):

$$f_p = \frac{P}{L_w \times t} = \frac{150 \times 1000}{850 \times t} = \frac{176.47}{t}\ N/mm^2$$

* Stress due to torsional moment : $M = Pe$.

$M = 150 \sin 60° \times e_x - 150 \cos 60° \times e_y$

$= 15.98\ kN\text{-}mm \times 10^6$

$$I_{xx} = \left[\frac{t \times 450^3}{12}\right] + 2\left[\frac{200 \times t^3}{12} + (200 \times t) \times 225^2\right] = 27.84 \times 10^6 t\ mm^4$$

$$I_{yy} = 2\left[\frac{t \times 200^3}{12} + 200 \times t\left(152.94 - \frac{200}{2}\right)^2\right] + \left[\frac{t^3 \times 450}{12} + (450 \times t) \times 47.05^2\right]$$

$= 3.45 \times 10^6\,mm^4 \cdot \times t \cdot$

$I_p = I_{xx} + I_{yy}$

$= 27.84 \times 10^6 t + 3.45 \times 10^6 t$

$= 31.29 \times 10^6\ t\ mm^4$

225

$\theta_1 = 34.20°$

$r = \sqrt{152.94^2 + 225^2} = 272.06\ mm$

θ = Angle between f_p & f_m

$\theta = 60 - 34.2$

$\theta = 25.8°$.

* Stress due to torsional moment :

$$f_m = \frac{Mr}{I_p} = \frac{15.98 \times 10^6 \times 272.06}{31.29 \times 10^6\ t} = \frac{138.04}{t}\ N/mm^2$$

$$R = \sqrt{f_p^2 + f_m^2 + 2 f_p f_m \cos\theta}$$

$$R = \sqrt{\left(\frac{176.4}{t}\right)^2 + \left(\frac{138.94}{t}\right)^2 + 2 \times \left(\frac{176.41}{t}\right) \times \left(\frac{138.94}{t}\right) \times \cos 25.8}$$

$$R = \frac{307.32}{t}\ N/mm^2$$

Combined stress in the fillet field weld should not be exceed

$$\frac{f_u}{\sqrt{3} \times \gamma_{mw}} = \frac{410}{\sqrt{3} \times 1.5} = 157.81\ N/mm^2$$

$$157.81 = \frac{307.32}{t}$$

$t = 1.95mm$.

$\therefore$ Size of weld required $= \frac{1.95}{0.707} = 2.76mm \simeq 6mm$.

$\therefore$ provide 6mm fillet weld.

P

4a Shape Factor of T section

Z_{ez} = 54.6 cm^3 (From hand book; Table VI pp - 18,19)
The section is unsymmetrical about the NA and hence the EAA has to be located.
Total area of the c/s = 29.08 cm^2 = 2908 mm^2 (From hand book; Table VI, pp - 18,19)
$A_1 = A_2 = A / 2 = 1454\ mm^2$
If EAA is at distance y from top, we have
150 * y = 1454
y = 9.69 mm

The plastic section modulus is calculated by taking moments of the area above and below the EAA about EAA.

Z_{pz} = 150 * 9.69 * 9.69 / 2 + 150 * 0.31 * 0.31 / 2 + 10 * 140 * (140 / 2 + 0.31)
= 105.48 x 10^3 mm^3
(0.31 = 10 - 9.69 ; 140 = 150 - 10)

$S = Z_{pz} / Z_{ez} = 1.93$

4 b

Max size of weld = $3/4 \times 10 = 7.5\,mm \simeq 6\,mm$

$\therefore$ Strength of weld @ bottom $(P_2) = 0.707 \times D \times l_2 \times \frac{f_u}{\sqrt{3}\,\lambda_{mw}}$

$$= 0.707 \times 6 \times \frac{410}{\sqrt{3} \times 1.5}$$

$$= 670\, l_2 \ N/mm$$

$\therefore$ Strength of weld @ top $(P_1) = 0.707 \times D \times l_1 \times \frac{f_u}{\sqrt{3} \times 1.5}$

$$= 670\, l_1 \ N/mm$$

$P_1 + P_2 = P'$

Each angle carries a load of $(P') = \frac{250}{2} = 125\ kN$.

Distributing weld in such a way that C.G of the weld coincides with that of angle section. Taking moment about top edge of weld.

$P_2 \times 75 = P' \times 17.6$

$670 \times l_2 \times 75 = 125 \times 10^3 \times 17.6.$

$l_2 = 43.78\,mm \simeq 50\,mm$

$P_2 = 670 \times 50 = 33.5 \times 10^3\ N.$

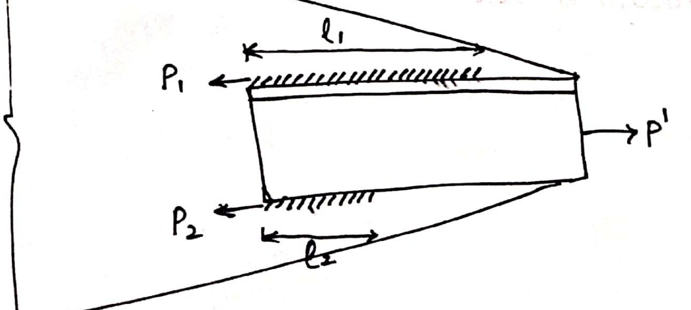
$P_1 = P' - P_2$

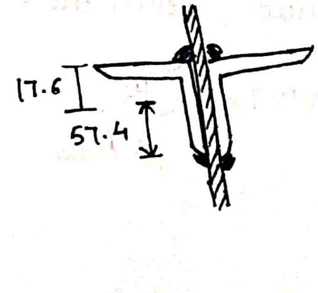
$= 125 \times 10^3 - 33.5 \times 10^3$

$P_1 = 91.5 \times 10^3\ N$

$P_1 = 670\, l_1$

i,e $l_1 = \frac{91.5 \times 10^3}{670} = 136.57\,mm \simeq 140\,mm.$





6A

Design a compression member using double channel section "face to face" to carry a factored load of 1600 kN. The length of column is 5m with one end fixed and one end hinged. Also design " single lacing system.

Design of column $l_c = KL$

$= 0.8 \times 5000 = 4000 mm$

Assume $f_{cd} = 200 N/mm^2$

$Area = \frac{Load}{\text{Stress}} = \frac{1600 \times 10^3}{200}$

$= 8000 mm^2 = 80.0 cm^2$

Area of one channel $= \frac{80}{2} = 40 cm^2$

From steel table, try 2ISLC - 300 @ 33.1 kg/m

Properties of one channel

area $4211 mm^2$

$I_{xx} = 6047.9 \times 10^4 mm^4$

$I_{yy} = 346. \times 10^4 mm^4$

$C_{yy} = 25.5$

$b = 100 mm$

To make any section (structure) safe

$I_{xx} = I_{yy}$

$I_{xx} = I_{xx} + Ah^2$

$= 2[6047.9 \times 10^4 + 4211 \times 0^2]$

$I_{xx} = 120.958 \times 10^6 mm^4$

$\cancel{I_{yy} = 2[346. \times 10^4 + 4211 (C b - C_{yy}) + \frac{S}{2})^2]}$

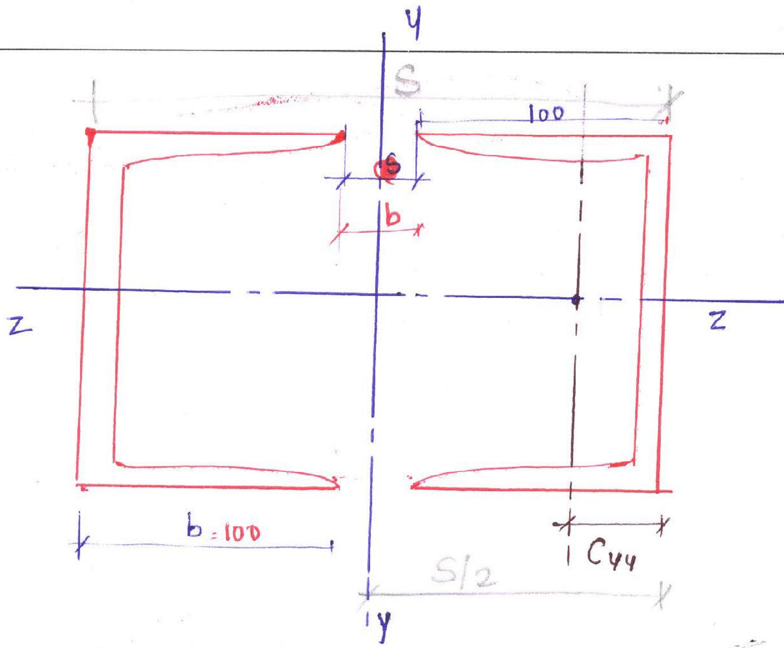
$\cancel{= 2[346. \times 10^4 + 4211 ((100 - 25.5) + \frac{S}{2})^2]}$

On equating $I_{xx} = I_{yy}$

$\cancel{\frac{S^2}{4} + 74.55 + 8.04 \times 10^3}$

$\cancel{(120.958 \times 10^6) = 2[346.0 \times 10^4 + 4211 (74.5 + \frac{S}{2})^2]}$

$\cancel{= 2[346.0 \times 10^4 + 4211 (5.5 \times 10^3 + 74.55 +} \frac{S^2}{4})$



$$120.95\times10^{6} = 346.0\times10^{4} + 4211\left(74.5+\frac{S}{2}\right)^{2}\Big]$$

$$-27.9\times10^{3} = \left(74.5+\frac{S}{2}\right)^{2}$$

$$\frac{S^{2}}{4} + 149s + 5.5\times10^{3} + 27.9\times10^{3}$$

$$\frac{S^{2}}{4} + 149s + 33.45\times10^{3}$$

$$2\left(120.95\times10^{6}\right) = 2\left[346.0\times10^{4} + 4211\left(\frac{S}{2}-25.5\right)^{2}\right]$$

$$120.95\times10^{6} = 2\Big[346\times10^{4} + 4211\left(\frac{S^{2}}{4} - 25.5 + 650.25\right)$$

$$\frac{S^{2}}{4} - 25.5s + 650.25 - 27.9\times10^{3}$$

$$\frac{S^{2}}{4} - 25.5s - 27.25\times10^{3}$$

$$S_1 = 385\,mm$$

$$S_2 = -283\,mm$$

~~b = 885 - 200~~

$\frac{S^2}{4} - 25.5 + 650.25 - 13.53 \times 10^3$

$\frac{S^2}{4} - 25.5s - 12.87 \times 10^3$

$S_1 = 283.5mm$

$S_2 = -181.5mm$

$b = 283.5 - 200 = 83.5mm \approx 80 mm$

$I_{min} = 120.9 \times 10^4$

$A = 4211 \ mm^2$

$r_{min} = \sqrt{\frac{120.9 \times 10^4}{2 \times 4211}}$, $r_{min} = 119.84 mm$

From Table 9(c) = Buckling class (c)

$\lambda = \frac{4000}{119.84} = 33.37$

$f_{cd} = 206.61 N/mm^2$

Compression load $= P_d = A_e f_{cd}$

$= 2 \times 4211 \times 206.61$

$= 1740 \times 10^3 N > 1600 \times 10^3 N$

Hence safe

Design of lacing - Pg No. 48, 49, 50

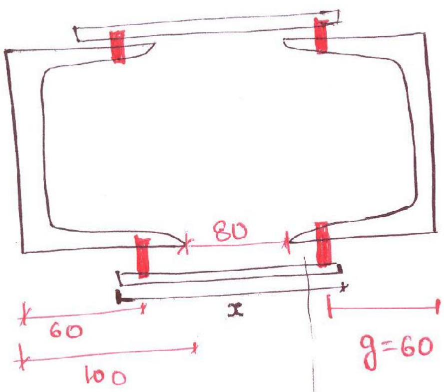
Transverse shear = 2.5% column load

$$V_t = \frac{2.5}{100} \times 1600 = 40\,kN$$

7.6.4 Angle of Inclination.

Not less than 40°, or more than 70°. Page 50.

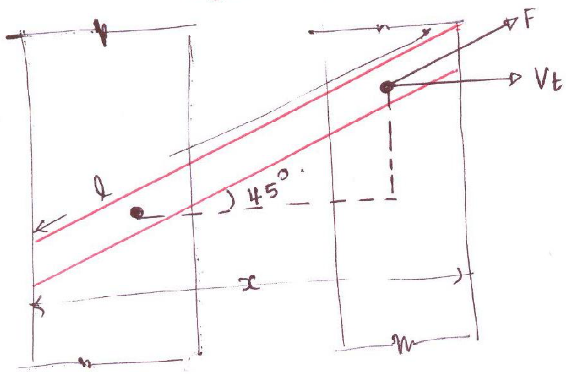
$\theta = 45°$.



g=60 givenin SP6.

$x = 2(b-g) + 80$

$2(100-60) + 80 = 160\,mm$



$$\cos 45 = \frac{x}{l} = \frac{160}{l}$$

$l = 226.27\,mm$

7.6.6.3

For single lacing, $l_e = l =$

double lacing $l_e = 0.7l = 0.7 \times 226.27$

length of lacing $= 226.27$ mm

7.6.2

Lacing dimensions

width = 3 x dia of end bolt / rivet

$= 3 \times 18$ (assume 18mm dia bolt)

7.6.3

$= 55$ mm

Thickness

For single lacings

$= \frac{1}{40} \times l_e$

$= \frac{226.27}{40} = 5.66 \approx 6$ mm

Lacing dimensions = $b \times t = 55 \times 6$ mm.

~~Spacing~~

7.6.6.3 check for slenderness ratio

$\lambda = \frac{l_e \sqrt{12}}{t} \not> 145$

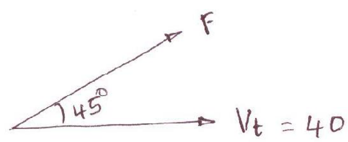
$= \frac{226.27 \times \sqrt{12}}{6}$

$= 130 \not> 145$

Taking $\lambda = 130$, find f_{cd} from table 9(c)

$f_{cd} = 71.3$ N/mm^2

Forces in lacing



$$F = \frac{V_t}{n \cdot \cos 45}$$

$$= \frac{40 \times 10^3}{2 \times \cos 45}$$

$F = 28.28\,kN$

$$\cos 45 = \frac{V_t}{F}$$

'F' force in lacing

Check for strength = Area × Stress

$b \times t \times f_{cd}$

$= 55 \times 6 \times 71.3$

$= 23.5\,kN$

Tensile strength of lacing / flat $= 0.9(b - d_0)\,t \times f_{cd}$

$= (55 - 20) \times 6 \times 0.9 \times 71.3$

$= 14.97\,kN$

Provide single bolt at each end.

Solution

Axial load = 700 kN.

Factored load = 1.5 x 700 = 1050 kN.

Bearing strength of concrete = $0.45 f_{ck} = 0.45 \times 20 = 9\ N/mm^2$

$$\text{Area of base plate} = \frac{\text{Factored load}}{\text{Bearing strength of concrete}} = \frac{1050 \times 10^3}{9} = 116.67 \times 10^3 m$$

Properties of ISHB 225 @ 46.8 kg/m. (IS 808:1989, Pg No: 5)

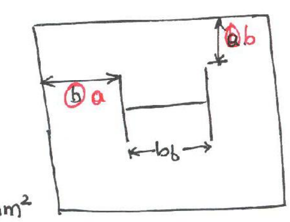
$D = h = 225mm$, $b_f = 225mm$, $t_f = 9.1mm$

∵ Overall dimension of the column is square, let us design a square base plate.

Side of square plate = $\sqrt{116.67 \times 10^3} = 341.56mm \simeq 350mm$.

Note: Keeping in view the cleat angle to be provided, fix the size of base plate. Size of base plate = 360mm x 360mm.

Projection beyond the column $a = b = \frac{360 - 255}{2} = 67.5mm$ 62.5mm



$$\text{Net upward pressure } (w) = \frac{P_u}{\text{Area of base plate}}$$

$$= \frac{1050 \times 10^3}{360 \times 360} = 8.10\ N/mm^2 \quad 8.5 < 9\ N/mm^2$$

★ Thickness of base plate: (t_s) [P-41, Cl. No - 7.4.3.1]

$$t_s = \sqrt{\frac{2.5w(a^2 - 0.3b^2)\gamma_{mo}}{f_y}} \nless t_f.$$

$$t_s = \sqrt{\frac{2.5 \times 8.1 (67.5^2 - 0.3 \times 67.5)^2 \times 1.1}{250}} = 16.86 \nless t_f\ (9.1mm) \quad \text{Say } 18mm.$$

15.98

∴ provide base plate of thickness 18mm i.e 360 x 360 x 18mm.

Design of bolt:

$$\text{No. of bolt} = \frac{\text{Force}}{\text{Bolt Value}}$$

Force / Side = 25% of column load = 0.25 × 1050 = 262.5 KN.

let us assume 22mm dia bolt

∴ bolt hole dia = 22 + 2 = 24 mm

Bolt value : least of following.

1. Strength of one bolt in shear.
2. Strength of one bolt in bearing.

1. Strength of one bolt in single shear (P-75, cl: 10.3.3)

$$V_{dsb} = \left(\frac{f_u}{\sqrt{3}}\right) \times \left(\frac{n_n A_{nb} + n_s A_{sb}}{\lambda_{mb}}\right)$$

Assuming shank is interfering the shear plane.

$n_n = 0$, $n_s = 1$, $\lambda_{mb} = 1.25$

$$A_{sb} = \frac{\pi}{4} d^2 = \frac{\pi}{4} \times 22^2 = 379.94 \text{mm}^2$$

$$V_{dsb} = \frac{400}{\sqrt{3}} \times \left(\frac{0 + 1 \times 379.94}{1.25}\right)$$

$V_{dsb} = 70.23$ KN.

2. Strength of bolt in bearing $V_{dpb} = \dfrac{2.5 \times K_b \times d \times t \times f_u}{\lambda_{mb}}$

K_b = least of following.

$$\frac{e}{3d_0} = \frac{45}{3 \times 24} = 0.625.$$

$e = 1.7 \times 24 = 40.8$mm. ≃ 45mm

$P = 2.5 \times 22 = 55$mm. ≃ 60mm

$$\frac{P}{3d_0} - 0.25 = 0.58$$

$$\frac{f_{ub}}{f_u} = \frac{400}{410} = 0.98$$

1

∴ $K_b = 0.58$.

t = The minimum of 1) Thickness of gusset plate (assumed) = 8mm

2) Thickness of ~~angle~~ base bol = ~~6mm~~. 9.1mm

Bolt value = 70.23 KN.

$$\text{No. of bolts} = \frac{262.5}{70.23} = 3.73 \simeq 4 \text{ No's}$$

Cleat angle :

Provide cleat angle ISA 100×75×10 mm to secure the column with the base plate by 4 bolts of 22 mm dia.

Concrete bed block:

Axial load = 700 KN.

Self wt of concrete = 10% of 700 i,e 0.1×700 = 70.

Total load = 770 KN.

$$\text{Area} = \frac{\text{Total load}}{\text{SBC of Soil}} = \frac{770}{180} = 4.28 \text{ m}^2$$

Side of Square block = $\sqrt{4.28}$ = 2.06m Say 2.1m

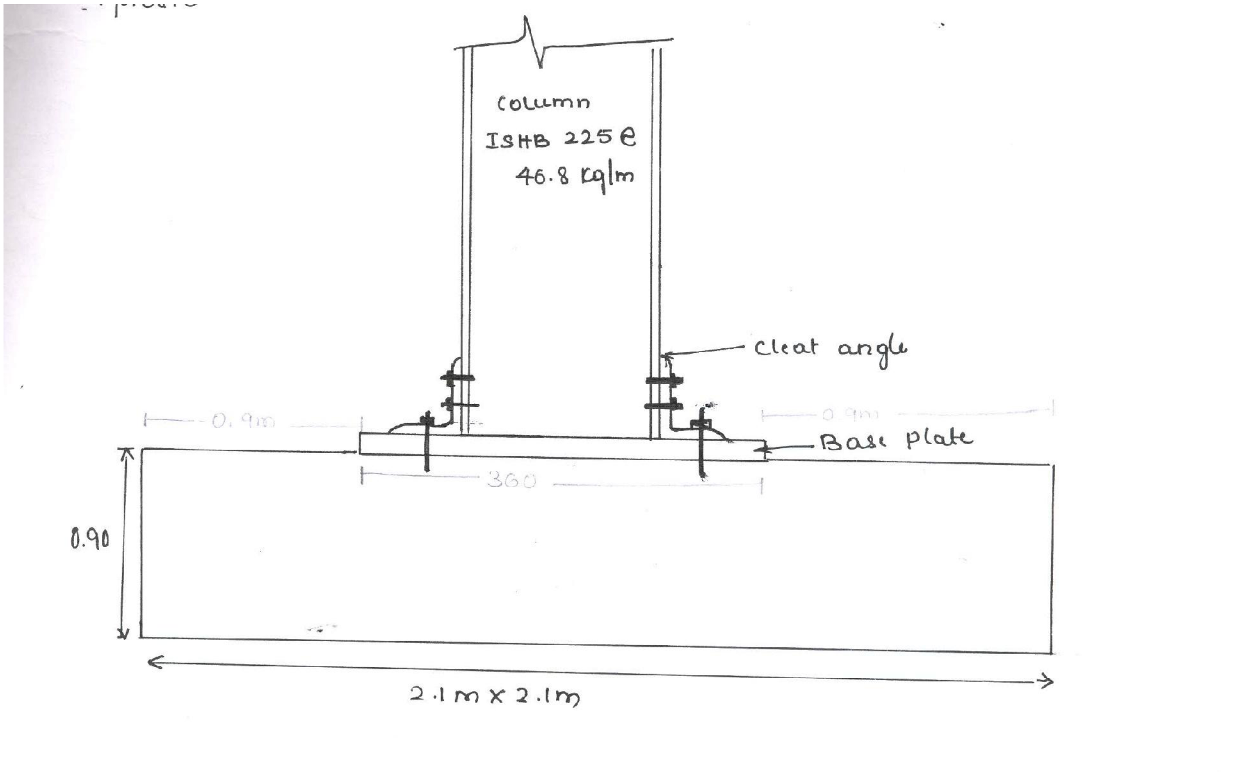
Thickness of block :

Assuming the dispersion angle of load as 45°

Depth of concrete bed = projection of concrete beyond base plate

$$= \frac{2.1 - 0.36}{2} = 0.87\text{m} \quad \text{Say } 0.90\text{m}.$$

∴ provide a concrete bed block = 2.1×2.1×0.9 m.


Column
ISHB 225 @
46.8 kg/m
Cleat angle
0.9m
0.9m
Base plate
360
0.90
2.1m x 2.1m

8A

along ... The beam subjected to an UDL 10kN/m. The thickness of wall is 230mm. is laterally supported.

(a) load calculation.

UDL on beam = 10 kN/m.

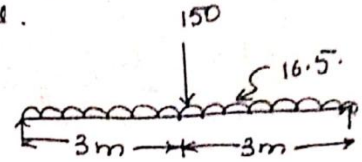
Assume self wt. = 1 kN/m.

11 kN/m.

ultimate load = UDL = 11 × 1.5 = 16.5 kN/m.

& ultimate point load = 1.5×100 = 150 kN.

$$V_u = \frac{16.5 \times 6}{2} + \frac{150}{2} = 124.5 \text{ kN}.$$



$$M_u = \frac{16.5 \times 6^2}{8} + \frac{150 \times 6}{4} = 299.2 \text{ kN-m}.$$

Plastic Modulus required $= Z_p = \frac{M_d \gamma_{mo}}{\beta \times f_y}$ mm². → pg 53.

$$Z_p = \frac{299.2 \times 10^6 \times 1.10}{1 \times 250} = 1316.5 \times 10^3 \text{ mm}^3 \simeq 1316.5 \text{ cm}^3$$

let us increase value by 20% approximately.

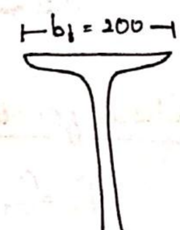
$= 1.20 \times 1316.5 = 1580 \text{ cm}^3$.

from IS-800 page 138 Try. ISWB -450 @ 79.4 kg/m.

$Z_p = 1760.59 \text{ cm}^3$

$Z_e = 1558.1 \text{ cm}^3 = Z_{xx}$.

$I_{xx} = 35057.6 \text{ cm}^4$ → SP 6 steel table.



(b) Check for deflection.

... span 6000 ... 24mm.

(c) Check for Shear. → 59 pg NO

$$V_d = 0.6\left[\frac{f_y}{\sqrt{3} \times \lambda_{mo}} \times A_v\right] > V_u.$$

$$= 0.6\left[\frac{250}{\sqrt{3} \times 1.1} \times 450 \times 9.2\right] = 325.94 \text{ KN} > V_u \text{ Safe.}$$

(d) Check for Moment of Resistance:

Section classification based on table 2 @ page' 18.

$$\left(\frac{b}{t_f}\right) = \frac{200/2}{15.4} = 6.49 < 9.4.$$

$$\left(\frac{d}{t_w}\right) = \frac{(h - 2t_f)}{t_w} = \frac{450 - 2 \times 15.4}{9.2} = 45.56 < 84.$$

Hence the Section is Plastic $\therefore \beta = 1$

$$M_d = \frac{\beta_b Z_p f_y}{\lambda_{mo}} > M_u.$$

$$= \frac{1 \times 1760.59 \times 10^3 \times 250}{1.1}$$

$$= 400.13 \text{ KN-m} > M_u = 299.2 \quad \text{SAFE}$$

But $M_d < 1.2 \dfrac{Z_e f_y}{\lambda_{mo}}$.

$$= 1.2 \times 1558.1 \times 10^3 \times \frac{250}{1.1}$$

$M_d \leq 424.93$ KN-m SAFE.

) Check for ~~deflection~~ web crippling:

$$F_w = (b_1 + n_2) t_w \frac{f_{yw}}{\lambda_{mo}}$$

$$= (\cancel{200}\,230 + 70.5) 9.2 \times \frac{250}{1.1}$$

F_w $> V_u$

Hence safe.

Assume $b_1 = \cancel{200}$ 230mm

$$n_2 = 2.5\,(t_f + r_1)$$

$$= 2.5\,(14.2 + 14)$$

$$= 70.5 \text{ mm}$$

(f) check for web buckling.

$F_{wb} = (b_1 + n_1)\, t_w\, f_c$

=

So find f_{cd} from table 9c for $\lambda =$

$\therefore f_{cd} =$

$F_{wd} = (200 + \quad\quad) 9.2 \times$

$= \quad\quad > V_u$

Hence Safe.

$\lambda = 2.5 \dfrac{d}{t_w}$

$= 2.5 \times \dfrac{(h - 2t_f)}{t_w}$

$= 2.5 \dfrac{(450 - 2 \times 15.4)}{9.2}$

=

$n_1 = h/2 = 450/2 =$

$\therefore$ The Section Decided is ISWB 450 @ 79.4 kg/m. is Safe to take loads coming on 6m Span Beam.