UNIT-I

① Advantages and Dis advantages of steel structures :-

1) steel members have high strength per unit weight therefore a steel member of a small section which has little weight is able to resist heavy loads.

2) The high strength of steel results in smaller sections to be used in buildings. The high strength to weight ratio is the most important property for construction of long span bridges, tall buildings & for buildings on soils with relatively low bearing capacities.

2) steel, being a ductile material, does not fail suddenly but gives visible evidence of impending failure by large deflection. also, The ductile nature of the usual structural steels enables them to yield locally, at the points of high stress concentrations. The ductility of steel is responsible for releiving the over-stressing in certain members by allowing redistibution of stresses due to yielding & thus, preventing pressure failures.

3) structural steel are tough i.e They have both strength & ductility. Thus steel members subjected to large deformation during fabrication & eraction will not fracture also the steel may be bends, hammered, sheared on a even a bolt holes may be punched

without any visible damage.

4) Being light, steel members can be convieniently handled & transported. for this reason, pre fabricated members can be frequently provided.

5) Properly maintained steel structures have a long life.

6) The properties of steel mostly do not change with time. This makes steel most suitable material for a structure when compared to R.C.C.

7) Additions & alternations can be made easily to steel structures.

8) They can be erected at a faster rates.

9) steel have the highest scrap value amongst all building materials, also, the steel can be reused after a structures is dismantled.

10) steel is ultimate recyclable material.

Dis advantages

1) Steel materials susceptable to corrosion.

2) maintanance cost is high.

3) steel structures can't bear fatique i.e some steel members some times subiected to compression & some times subiected to tension in the case of struts called as reversible stresses (fatique).

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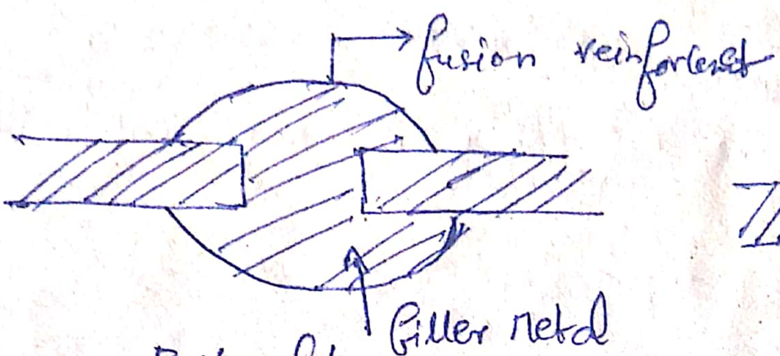
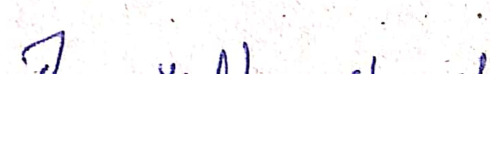
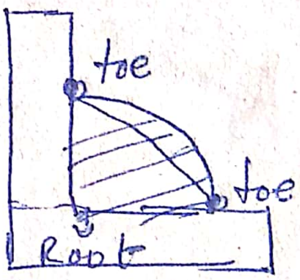


Butt weld

Single 'V' Butt weld.



Double ~~Single~~ 'V' Butt weld



Fillet weld.

(n)
18 Types of welded Joints :-

The various types of welded Joints are as follows

1. Butt ~~Joint~~ weld
2. Fillet weld.
3. slot weld
4. plug weld.
5. Spot weld
6. pipe weld
7. Steam weld

1. Butt weld :- Butt weld is also termed as Grooved weld. The Butt weld is used to Joint. The structural members Joining direct Compression or Tension

→ It is used to make Tee - Joint and Butt Joint.

→ Various types of butt Joints are as follows
There are named depending upon the slope of the groove Provided ~~Be~~ for welding

(i) Square Butt weld

(ii) single 'V' Butt weld

iii) Double - V Butt Weld

iv) Single - u Butt weld

v) Single Double v Butt Weld

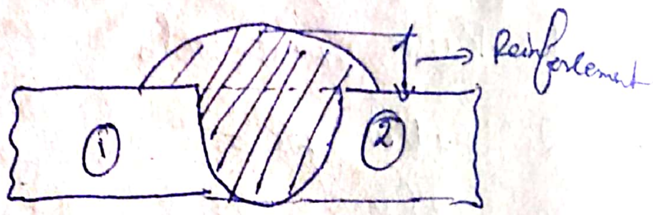
vi) Single J Butt Weld

vii) Double J " "

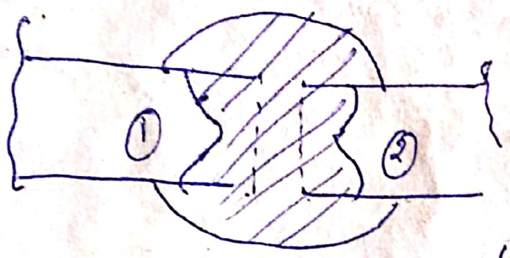
viii) Single bevel butt weld

ix) Double bevel butt weld

(i) Square butt weld :-



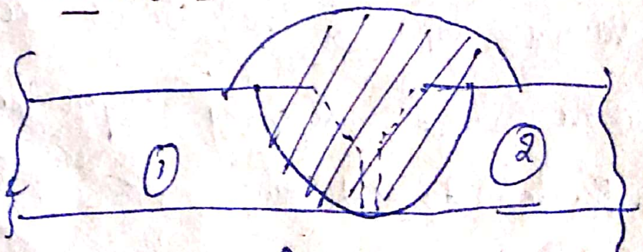
1 Side Square butt Weld



Both Side Square butt Weld

It is the weld preparation of each the fusion plates lies aproximately right angles to the Surfaces of the Components to be Joined and are Substaincially parellal to one another. Square butt weld is as shown in fig above.

(ii) Single V butt weld :-



Single v-butt weld

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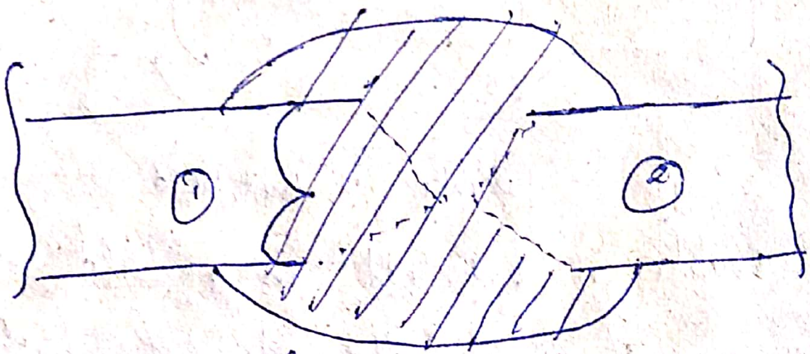
a single 'v' butt weld is a weld in the preparation of each the edges of both components are prepared so that in the c/s of the fusion phases from a 'v' shown in fig above

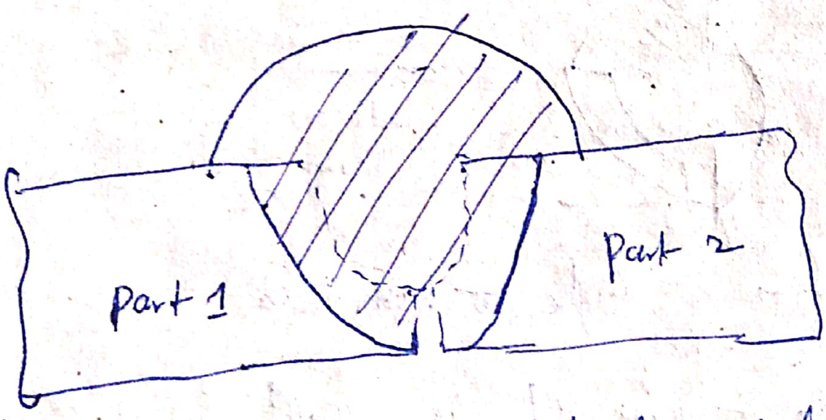
Double 'v' Butt weld :-



Double - v - butt weld

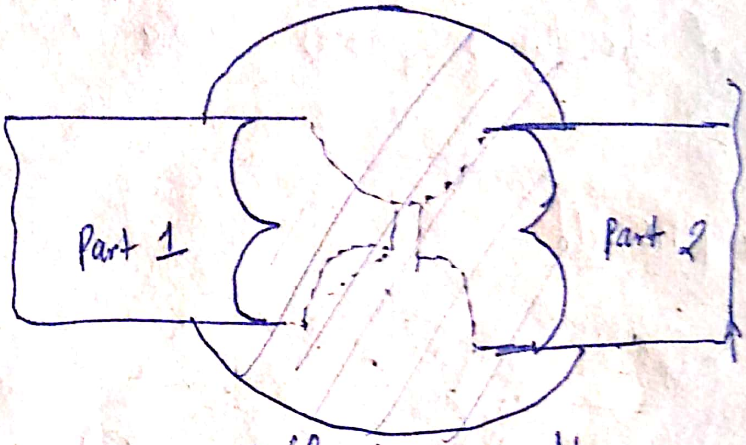
The Double 'v' butt weld is a weld in the preparation of each the edges of both components are double bevelled so that in c/s, The fusion phases from two opposite 'v's.

Single 'v' Butt weld



Single 'v' butt well is a weld in the preparation of each the edges of both components are prepared so that in c/s, The fusion faces from a ~~v~~ 'u' butt weld

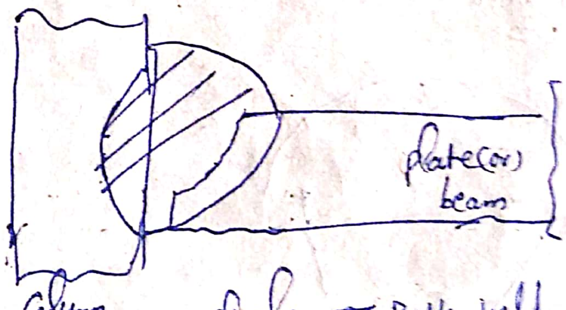
Double -U- Butt weld



Double 'U'. Butt weld

a Double. U butt weld ~~is~~ a weld ~~in which~~ the Preparation of which the edges of both Components ... prepared . So that in C/s the fusion from two phases opposing having a Common base as shown ...

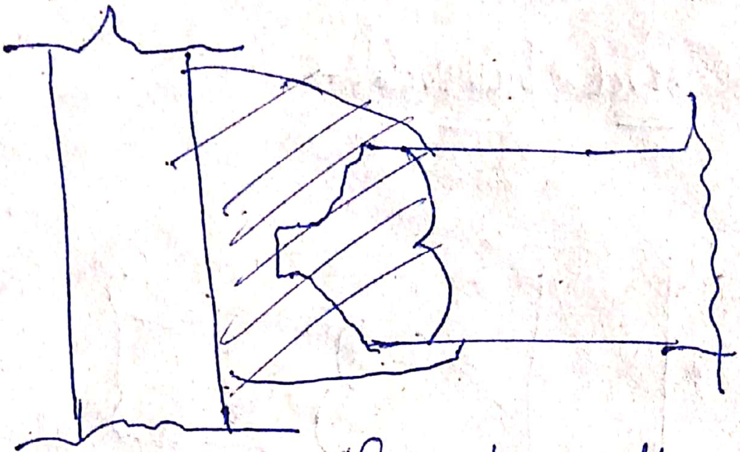
Single J Butt weld



Single J Butt weld

a single J Butt weld is a weld in the preparation of which the edges of one Component is prepared so that The fusion of faces forms in the form of J & The fusion phase of other Component (Column) is at right angles to the Surface of the 1st Component

7. <u>Double J Butt weld</u>:-

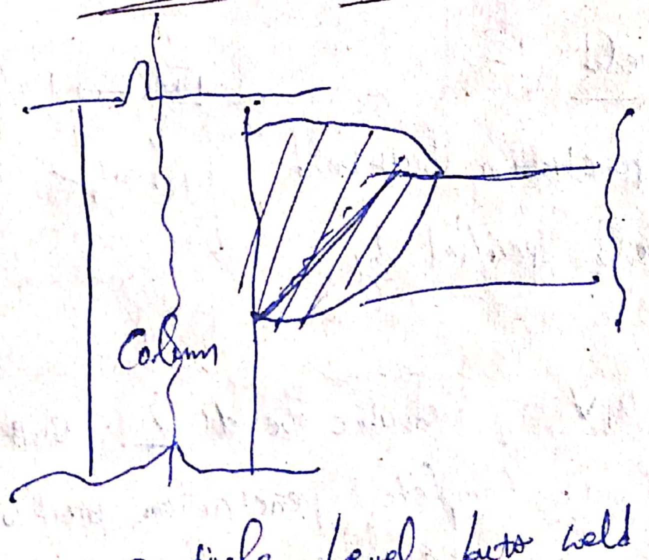


Double-J-butt weld

It is a weld in the preparation of which the edges of one component are prepared so that in c/s, The fusion phase is in the form of two opposing 'J's & The fusion phase of the other component is at right angles to the surfaces of first component as shown in fig.

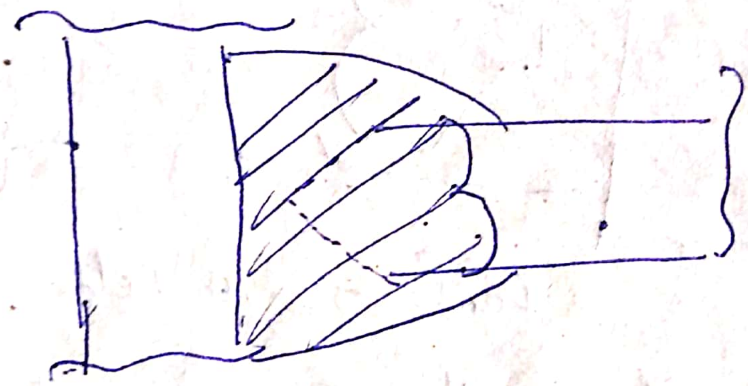
viii) <u>Single bevel butt weld</u>:-

V-v
V-bevelled



a single level butt weld is a weld in the preparation of each The edges of one Component are prepared so that ~~The edges of one Component are~~ a cross section, The fusion phase ~~in~~ is in the form of bevel and the fusion phase of other Component is at right angles to the surface of the 1st column

Double bevel butt weld:-



A Double bevel butt weld is a weld in the preparation of each the edges of one component are prepared so that in c/s, the fusion phase is in the form of a double bevel & the fusion phase of a other component is at right angles to the surface of the 1st component.

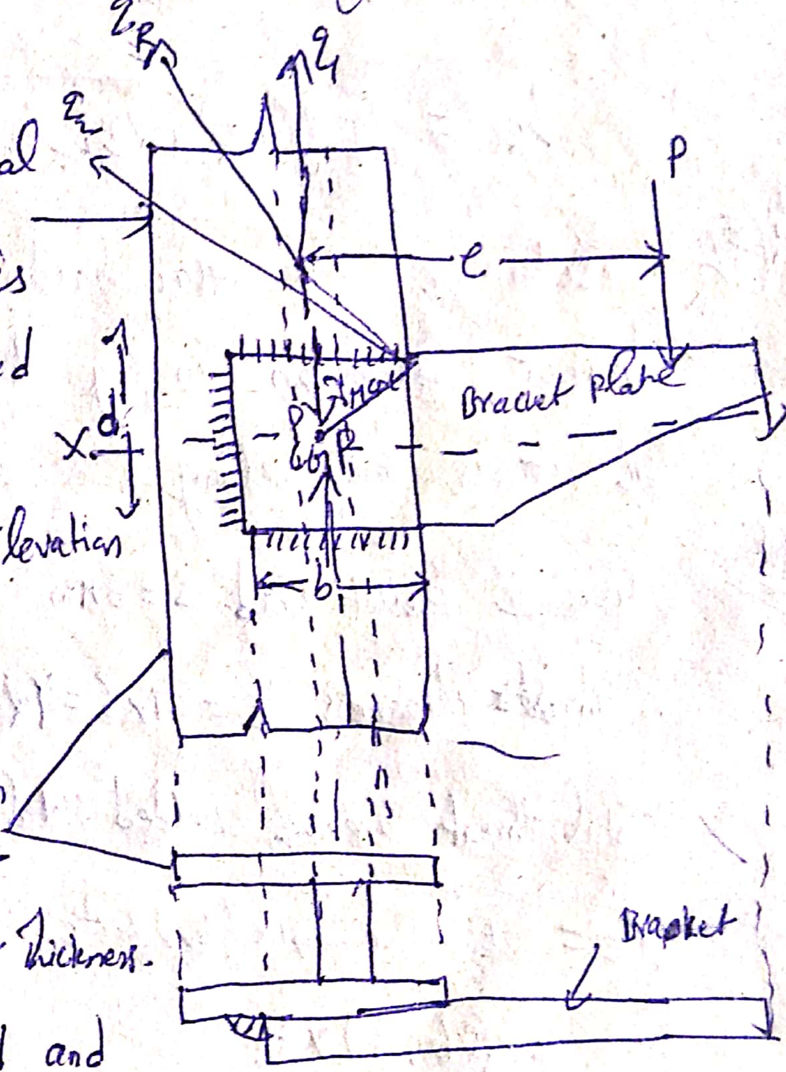
Eccentric Connections :-

Case: 1 plane of moments and plane of welds in the same plane : ~~tant~~

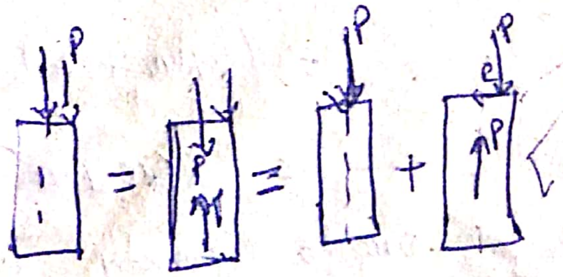
A fig shows a typical phase an eccentric load P is equivalent to i) a direct load 'P' at C.G

ii) a twisting moment

$$M = P \times e$$



let a weld of uniform size provided throughout and 't' be the effective throat thickness. if 'd' is the depth of weld and 'b' is width of weld as shown in fig.



The direct stress (shear stress) in the weld is $q_1 = \dfrac{P}{(2b+d) \times t_e}$

The stress in the weld due to twisting moment is the max. in the weld at the extream fiber distance from C.G of the group of weld and act in the direction $\perp^{lar}$ to the radius vector.

$\left[\frac{M}{I} = \frac{F}{y}\right]$

$$\therefore \; q_2 = \frac{M}{I_{zz}} \times y_{max} = \frac{M}{I_{zz}} (r_{max}) \rightarrow (2)$$

I_{zz} = polar MoI $= I_{xx} + I_{yy}$

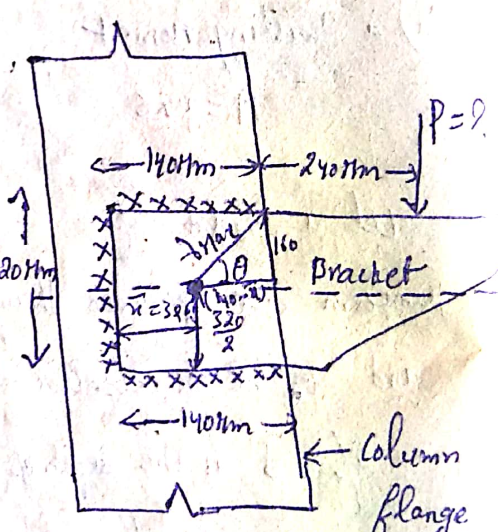
Y_{max} = Distance of the extream weld from C.G of weld groove

The resultant stress of direct stress and the bending stress

$$q_R = \sqrt{q_1^2 + q_2^2 + 2q_1q_2\cos\theta}$$

(n) Determine the max load that can be resisted by braket as shown in fig by fillet weld of size 6mm. if it is shop welding.

Sol:



size of Fillet weld. S = 6mm

$\therefore$ Throat Thickness $t_e = 0.7 \times 6 = 4.2$mm

Total throat area of welded metal

A_e = Effective length (l_w) $\times t_e$

$= (2b + d) \times t_e$

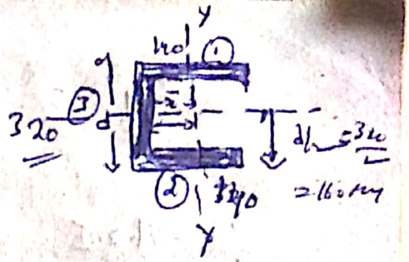
$= (2 \times 140 + 320) \times 4.2$

$= 2520 \text{ mm}^2$

Due to symmetry centroidal x-x axis is at the mid hight of vertical weld

Let Centroidal Y-Y axis be at a distance $\bar{x}$ from the vertical weld

Q) ∴ Let $\bar{x} = \dfrac{a_1\bar{x}_1 + a_2\bar{x}_2 + a_3\bar{x}_3}{a_1+a_2+a_3}$



$a_1 = 140 \times 4.2 = 588\ mm^2$

$a_2 = 140 \times 4.2 = 588\ mm^2$

$a_3 = 320 \times 4.2 = 1344\ mm^2$ [reference line = Y-axis]

$$\bar{x} = \frac{\left(588 \times \frac{140}{2}\right) + \left(588 \times \frac{140}{2}\right) + (1344 \times 0)}{588 + 588 + 1344}$$

$= 32.66\ mm$

∴ M.O.I about X-X axis: $I_{xx} = I_{xx_1} + I_{xx_2} + I_{xx_3}$

$I_{xx_1} = \dfrac{b_1 d_1^3}{12} + a_1 h_1^2$

$= \dfrac{140 \times (4.2)^3}{12} + 588 \times \left(\dfrac{320}{2}\right)^2 = 15.05 \times 10^6\ mm^4$

$I_{xx_1} = I_{xx_2} = 15.05 \times 10^6\ mm^4$

$I_{xx_3} = \dfrac{b_3 d_3^3}{12} = \dfrac{4.2 \times (320)^3}{12} = 11.46 \times 10^6\ mm^4$

$I_{xx} = 2(15.05 \times 10^6) + 11.46 \times 10^6 = 41.56 \times 10^6\ mm^4$

M.O.I about Y-Y axis (Minor axis)

$I_{yy} = I_{yy_1} + I_{yy_2} + I_{yy_3}$

$I_{yy_1} = \dfrac{b_1 d_1^3}{12} = \dfrac{4.2 \times (140)^3}{12} = 960.4 \times 10^3\ mm^4$

$I_{yy_1} = I_{yy_2} = 960.4 \times 10^3\ mm^4$

$I_{yy_3} = \dfrac{b_3 d_3^3}{12} + a_3 h_3^2$

$= \dfrac{320 \times 4.2^3}{12} + 1344 \times (\cancel{40}\ 32.66)^2 = 1.43 \times 10^6\ mm^4$

$I_{yy} = 2(960.4\times10^3) + 1.43\times10^6 = 3.35\times10^6\ mm^4$

$\therefore$ M.O.I about polar axis $I_{zz} = I_{xx} + I_{yy}$

$I_{zz} = 41.56\times10^6 + 3.35\times10^6 = 44.91\times10^6\ mm^4.$

r_{max} = distance of extream fibre weld from CG of weld frame

$r_{max} = \sqrt{(107.34)^2 + (160)^2} = 192.67\ mm$

let θ be the angle made by the radius vector with the horizontal

$\tan\theta = \frac{160}{107.34} \Rightarrow \theta = \tan^{-1}\left(\frac{160}{107.34}\right) = 56°8'$

Eccentricity e w.r.to minor axis $e_y = 240 + 107.34$

$e_y = 347.33\ mm.$

Let P be the load in k.N applied at a distance e from minor axis (Y-Y axis) on the braket plate

The direct stress $q_1 = \frac{\text{Direct load}}{\text{Throat area}} =$

$\Rightarrow q_1 = \frac{P}{(140\times2+320)4.2} = 3.96\times10^{-4}\ P$

Shear stress at extream edge due to torsional moment (M)

$q_2 = \frac{M}{I_{zz}}\times r_{max} = \frac{P\times e_y}{I_{zz}}\times r_{max}$

$\Rightarrow = \frac{P\times347.33}{44.99\times10^6}\times192.67$

$= 1.49\times10^{-3}\ P$

The resultant stress $q_R = \sqrt{q_1^2 + q_2^2 + 2 P q_1 q_2 \cos\theta}$

$$= \sqrt{(3.96\times10^{-4})^2 P^2 + (1.49\times10^{-3})^2 P^2 + 2(3.96\times10^{-4}) \times (1.49\times10^{-3}) P^2 \cos(56.51)}$$

$$= 1.74\times10^{-3} P \longrightarrow (1)$$

Note:- This resultant stress should not exceed permissable shear strength of the weld i.e $\frac{f_u}{\sqrt{3}\times\gamma_{mw}} = \frac{410}{\sqrt{3}\times1.25} = 189.37 \, N/mm^2 \rightarrow (2)$

if the joint in the safe condition, The resultant shear stress should be equal to ultimate shear strength of the welded metal. $\therefore$ equate eq^n (1) & (2)

$$1.74\times10^{-3} P = 189.37$$

$$\Rightarrow P = \frac{189.37}{1.74\times10^{-3}}$$

$$P = 108.83\times10^3 N = 108.8 \, kN.$$

(11) From the above problem The size of weld required if the eccentric load is 108.83 kN.

Sol Let Throat Thickness

$t_e = 0.7\times S$

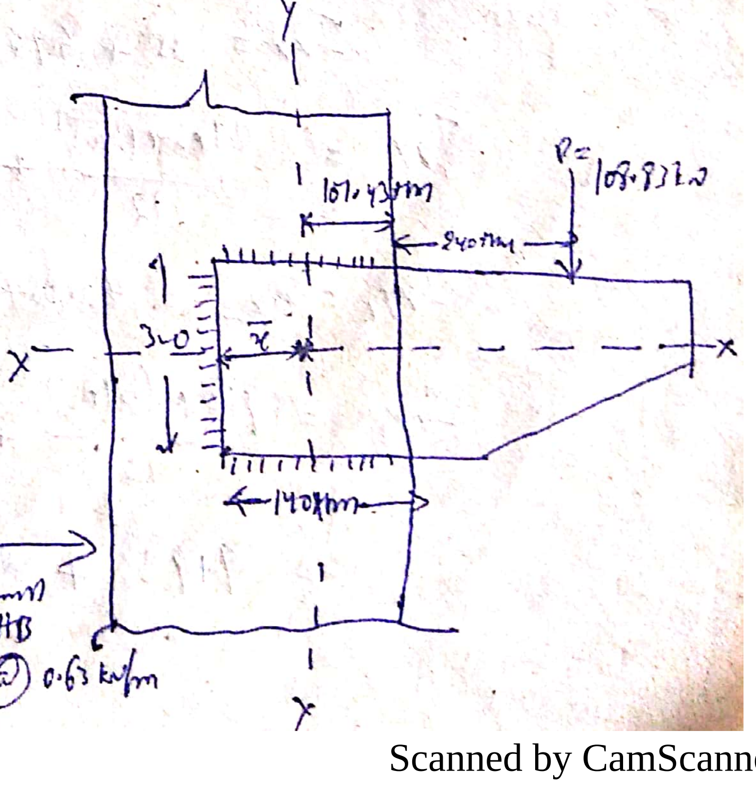
The total Area of weld

$= t_e \times L_w$

$= \cancel{0.7\times S \times (2\times140)}$ -

$= 0.7\times S\times(2(140)+320)$

$= 420 S$



~~Due to~~ Symmet From the previous problem we know

$\bar{y}$ = The position of x-x from the bottom weld

$= \frac{320}{2} = 160$ mm.

$\bar{x}$ = The position of y-y axis from left most weld

$= 32.66$ mm

$$I_{zz} = I_{xx} + I_{yy} = \cancel{\quad}10$$

$$I_{xx_1} = \frac{140 \times (0.7 \times s)^3}{12} + \cancel{\quad(160)^2} \; 140 \times 0.75 \times (160)^2$$

$$I_{xx_1} = I_{xx_2} = 4s^3 + 2.508 \times 10^6 s$$

$$\therefore \; I_{xx_3} = \frac{320^3 \times 0.7 \times s}{12} + (320 \times 0.7 \times s) \times (0)^2$$

$$= 1.91 \times 10^6 s \text{ mm}^4$$

$$\therefore \; I_{xx} = 2(4s^3 + 2.508 \times 10^6 s) + 1.91 \times 10^4$$

$$= 8s^3 + 5.01 \times 10^6 s + 1.91 \times 10^4 s$$

$$I_{xx} = 8s^3 + 6.92 \times 10^6 s$$

$$I_{yy_1} = \frac{0.7 \times s \times (140)^3}{12} + 0 =$$

$$I_{yy_1} = I_{yy_2} = 160.06 \times 10^3 s \text{ mm}^4$$

$$I_{yy_3} = \frac{320 \times (0.7 \times s)^3}{12} + (320 \times 0.7 \times s)(32.66)^2$$

$$= 9.14 s^3 + 238.93 \times 10^3 s.$$

$$I_{yy} = 2(160.06\times10^3 s) + 9.14 s^3 + 238.93\times10^3 s$$

$$= 9.14 s^3 + 559.05\times10^3 s$$

Direct stress $q_1 = \dfrac{108.83\times10^3}{(2\times140+220)\times0.7\times s} = \dfrac{259.11}{s}$

$$q_2 = \frac{M}{I_{zz}}(r_{max}) = \frac{P\times e}{I_{zz}} \times r_{max}$$

$$= \frac{108.83\times10^3\times347.33}{8s^3+6.92\times10^6 s+9.14s^3+559.05\times10^3 s}\times192.67$$

$$= \frac{37.83\times10^6}{17.14s^3+7.47\times10^6 s}\times192.67$$

$$= \frac{7.288\times10^9}{17.14s^3+7.47\times10^6 s}$$

Resultant stress

$$q_R = \sqrt{\left(\frac{259.11}{s}\right)^2 + \left(\frac{7.29\times10^9}{17.14s^3+7.47\times10^6 s}\right)^2 + 2\left(\frac{259.11}{s}\right)\left(\frac{7.29\times10^9}{17.14s^3+7.47\times10^6 s}\right)\cos(56^\circ 9')} \longrightarrow (1)$$

$$= \sqrt{\frac{\cancel{67.13\times10^3}}{s^2} + }$$

This resultant stress should not exceed the permissible shear stress of the welded metal i.e $189.37\ N/mm^2$

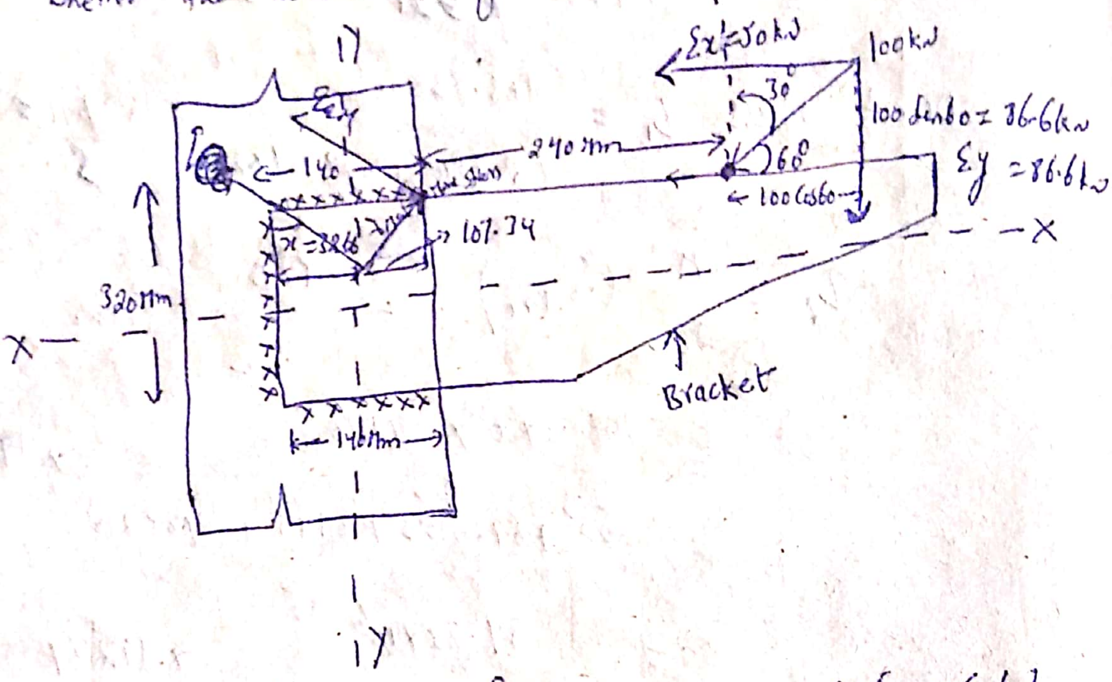
$$\left(\frac{259.11}{s}\right)^2 + \left(\frac{7.28\times10^9}{17.14s^3+7.47\times10^6 s}\right)^2 + 2\left(\frac{259.11}{s}\right)\left(\frac{7.28\times10^9}{17.14s^3+7.47\times10^6 s}\right)\cos(56^\circ 9')$$

$$= (189.37)^2$$

$$\Rightarrow \frac{(67.13\times10^3)}{s^2} + \left(\frac{5.299\times10^{19}}{17.14s^3+7.47\times10^6 s}\right)^2 + \frac{3.772\times10^{12}}{s(17.14s^3+7.47\times10^6 s)}\cos 56^\circ 9' = (189.37)^2$$

$\Rightarrow$ size of weld $s = 5.89 \simeq 6mm$

(8) Bracket plate is welded to the flange as shown in fig below. Check whether the weld is safe or not.



Sol:

The horizontal component load of 100 kN = 100 cos 60 = 50 kN

vertical " " Σy = 100 sin 60 = 86.6 kN.

Twisting moment on the joint about its C.G

$$= -\left(50 \times \frac{320}{2}\right) + \left(86.6 \times \left(240 + 107.34\right)\right)$$

$$= 22100 \times 10^3 \text{ kN-mm}$$

$$= 22.1 \text{ kN-m} = \text{unbalanced moment}$$

direct shear stress due to direct vertical component

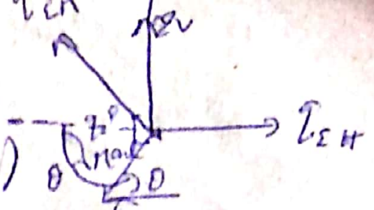
$$q_{\Sigma V} = \frac{\text{Load}}{\text{Throat Area}} = \frac{86.6 \times 10^3}{(320 + 2 \times 140) \times 0.7 \times 6}$$

$$= 34.36 \text{ N/mm}^2 \qquad (\text{assume } S = 6 \text{ mm})$$

direct shear stress due to horizontal component

$$q_{\Sigma H} = \frac{50 \times 10^3}{(320 + 2 \times 140) \times 0.7 \times 6} = 19.84 \text{ N/mm}^2$$

Max. Torsional stress due to twisting moment



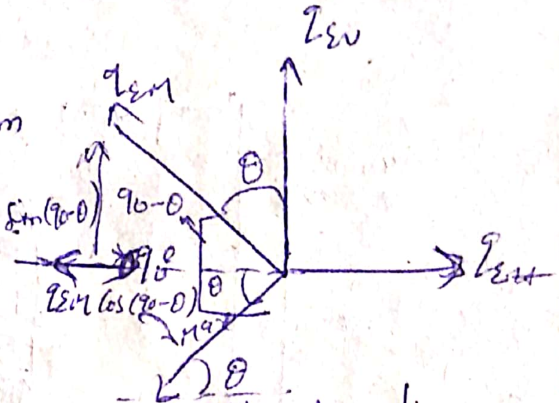
$$q_{Em} = \frac{T}{J} \times (R_{max}) \text{ (or) } \frac{M}{I_{zz}} (q_{max}) \qquad \left[\because \frac{T}{J} = \frac{\tau \text{ or } f_s}{R} \right]$$

$$= \frac{82.1 \times 10^6}{44.91 \times 10^6} \times 192.67$$

$J = I_{zz} = I_{xx} + I_{yy} = 44.91 \times 10^6$ [previous problem 1]

$$q_{Em} = 94.8 \text{ N/mm}^2$$

Let q_H = horizontal component from vertical stress diagram



vector stress diagram

$$q_h \overset{(\rightarrow)}{=} q_{EH} - \overset{(\rightarrow)}{q_{EM} \cos(90-\theta)}$$

$$= 19.84 - 94.8 \times \sin\theta$$

$$= 19.84 - 94.59 \times \sin(56°8')$$ [from previous problem]

$$= -58.7 = 58.7 \text{ N/mm}^2 \text{ (left} \leftarrow)$$

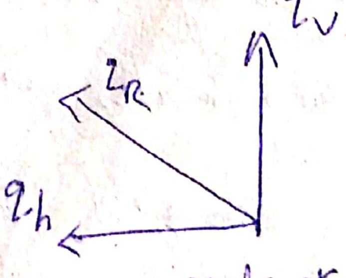
vertical component for vector stress diagram

$$q_v = q_{EV} + q_{EM} \sin(90-\theta)$$

$$= 34.36 + 94.8 \times \cos(56°8')$$

$$= \underline{\underline{87.68 \text{ N/mm}^2}} \text{ (upward)} \uparrow$$

q_R (resultant stress of q_H & q_v)



resultant st vector stress diagram

$$q_R = \sqrt{(q_H)^2 + (q_v)^2}$$

$$= \underline{\underline{105.44 \text{ N/mm}^2}}$$

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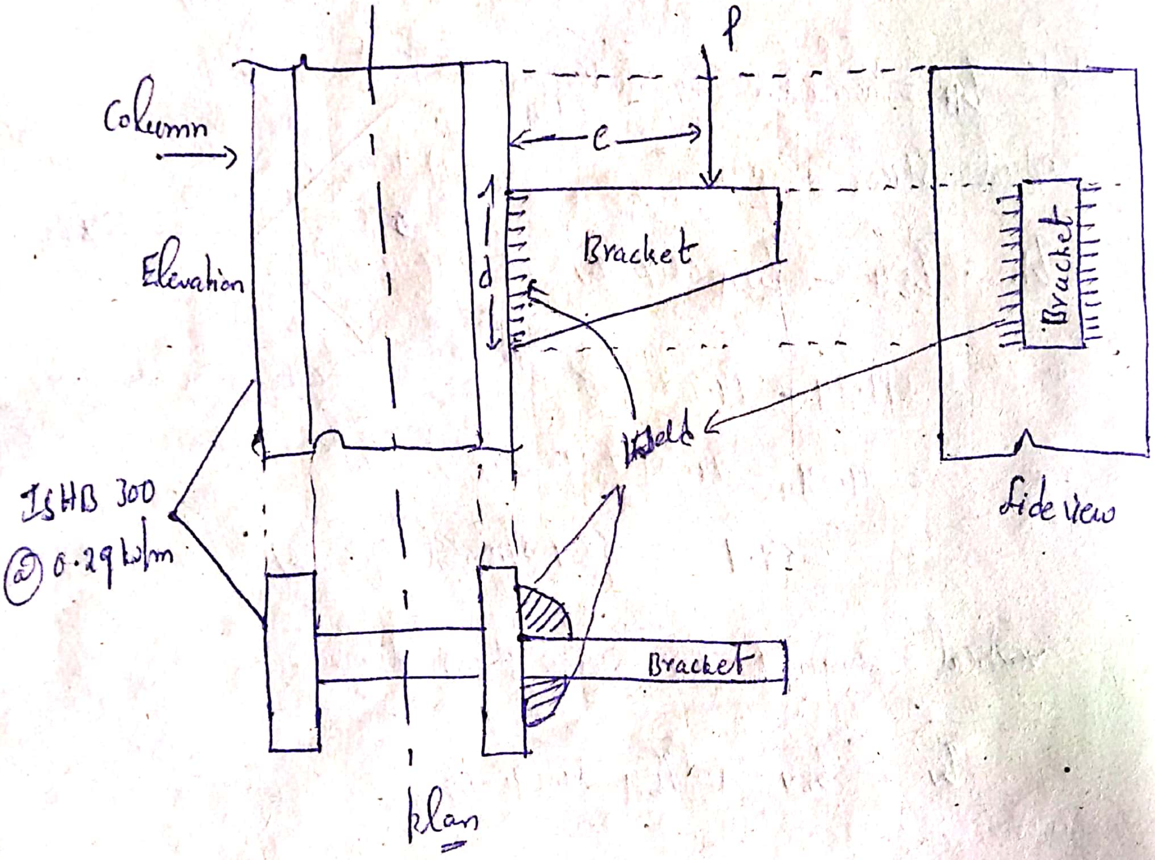
This resultant stress should not exceed the permissible shear stress of the welded metal

i.e $\frac{f_u}{\sqrt{3}\,\gamma_{mw}} = \frac{410}{\sqrt{3}\times 1.25} = 189.37\ N/mm^2$

$\therefore$ $105.03\ N/mm^2 < 189.37\ N/mm^2$, hence the weld is safe

Case: 2 - When the load does not lie in the plane of weld



This is a usual case, when plate bracket connect against the flang of stanchion (column), & connected by fillet weld on both sides by a plate bracket as shown in fig above.

The fillet weld is subjected to

1) vertical shear stress due to axial load (p).
2) Horizontal shear stress (f_b) due to B.M $M = p \times e$

Let the depth of bracket ~~plate~~ be equal to 'd' &
Fillet welds are applied to the both sides of Bracket Plate
for complete length equal to the depth of plate

∴ Length of the weld, $L = d + d = 2d$

Direct stress due to vertical load passing through the C.G from the analysis

$$f_a = \frac{\text{Direct load}}{\text{c/s Area of fillet weld}} = \frac{P}{2d \times t_e}$$

Similarly, Bending stress $f_b = \frac{M}{I} \times y$

$$f_b = \frac{P \times e}{I_{xx}} \times \left(\frac{d}{2}\right)$$

$$= \frac{P \times e}{\left(\frac{t_e \times d^3}{12}\right) \times 2} \times \left(\frac{d}{2}\right)$$

$$= \frac{3Pe}{t_e d^2}$$

For preliminary design depth 'd' [length of fillet weld on one side or full depth of the Bracket plate]

for preliminary design it is assumed that Bending stress F_b is taken as the resultant stress

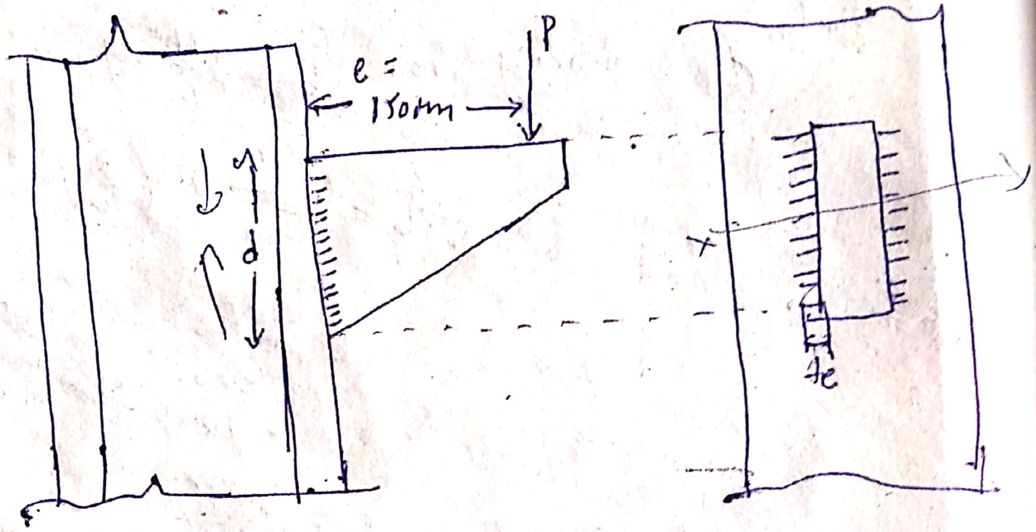
$$\therefore f_b = \text{Resultant stress} = \frac{3Pe}{t_e d^2}$$

$$\Rightarrow d = \sqrt{\frac{3Pe}{t_e f_b}}$$

This depth is based on the Bending stress
∴ it can be resisted the Bending stress only

To resist vertical shear also it is to be increased by 10% More [from codal provision

(*) Design a suitable fillet weld for the Bracket shown in fig below, if working load P = 100 kN and eccentricity e = 150mm Thickness of the bracket plate is 12mm and the Column used is ISHB-300 @ 618 N/m



Sol:

Load P = 100kN

∴ Factored load = 1.5 × 100 = 150 kN.

Thickness of flange of ISHB 300 @ 618 N/m is 10.6 mm (from steel table) (or) (63 kg/m) N/m → 9.81 × N/m → N/m

Page 3

∴ Min size of weld s = 5mm [From Table 21 In Code Page 78

Thickness of bracket plate = 12mm

∴ Consider size of the weld s = 8 mm [in b/w 5-10mm] provide on each side of bracket plate

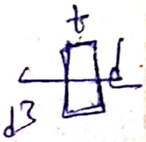
Throat thickness $t_e = 0.7 \times 8 = 5.6\,mm$

permissiable shear stress of the welded metal

$$f_{wd} = \frac{f_u}{\sqrt{3} \times \gamma_{mw}} = \frac{410}{\sqrt{3} \times 1.25} = 189.37\,N/mm^2$$

for this case to find the depth of the weld required based on the asumption is Bending stress = permissable shear stress of welded metal.

$$f_b = f_{wd}$$



$$\Rightarrow \frac{M}{I} \times y = f_{wd}$$

$$\frac{I}{y} = \frac{\frac{t \times d^3}{12}}{d/2} = \frac{td^3}{12} \times \frac{2}{d} = \frac{td^2}{6}$$

$$\Rightarrow \frac{P \times e}{\left(\frac{t_e \times d^3}{12}\right) \times 2} \times \frac{d}{2} = 189.37$$

$M = f z$

$$\Rightarrow \frac{3pe}{t_e d^2} = 189.37$$

$$\Rightarrow f = \frac{M}{2} = \frac{M}{\frac{td^2}{6}}$$

$$= \frac{6M}{2 \times td^2} = \frac{3M}{td^2}$$

$$\Rightarrow \frac{3 \times 150 \times 150 \times 10^3}{5.6 \times d^2} = 189.37$$

$$\Rightarrow d = 252.29\,mm$$

This depth is adequately capable to resist the Bending stress only.

$\therefore$ To accomudate for the vertical shear it depth is increased about 10%.

$\frac{10}{100}$ 0.1

Total depth $d' = h = 252.29 \times 1.1$ (or) $252.29 + 10\%$ of 252.29

$= 277.5\,mm \simeq 280\,mm$

<u>check for stresses</u>

direct shear stress $q = \frac{P}{Area} = \frac{P}{2 \times d' \times t_e}$

$= \frac{150 \times 10^3}{\quad} = 47.83\,N/mm^2$

Bending stress $f_b = \dfrac{3Pe}{t_e r d^2} = \dfrac{3\times150\times10^3\times150}{5.6\times(280)^2} = 153.74\,N/mm^2$

Equivalent stress from the code book page 81 clause 10.5.10

$$f_e = \sqrt{f_b^2 + 3.q_a^2} = \sqrt{(47.83)^2 + 3(153.74)^2}$$

$$= \sqrt{(153.74)^2 + 3(47.83)^2}$$

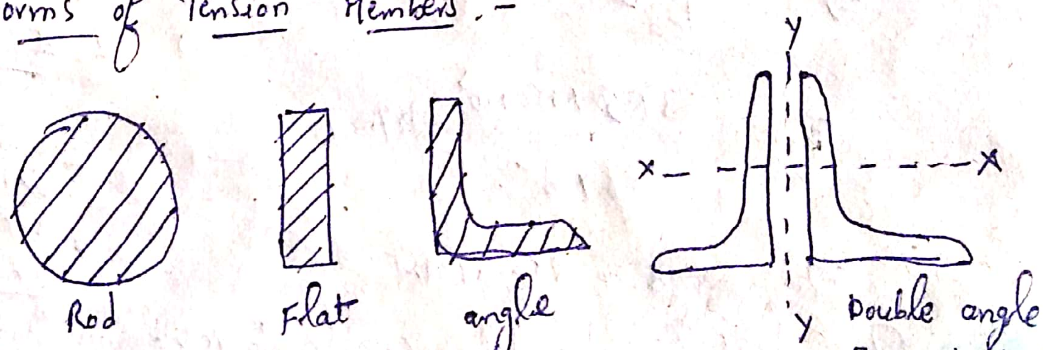
$$= 174.63 < 187.79$$

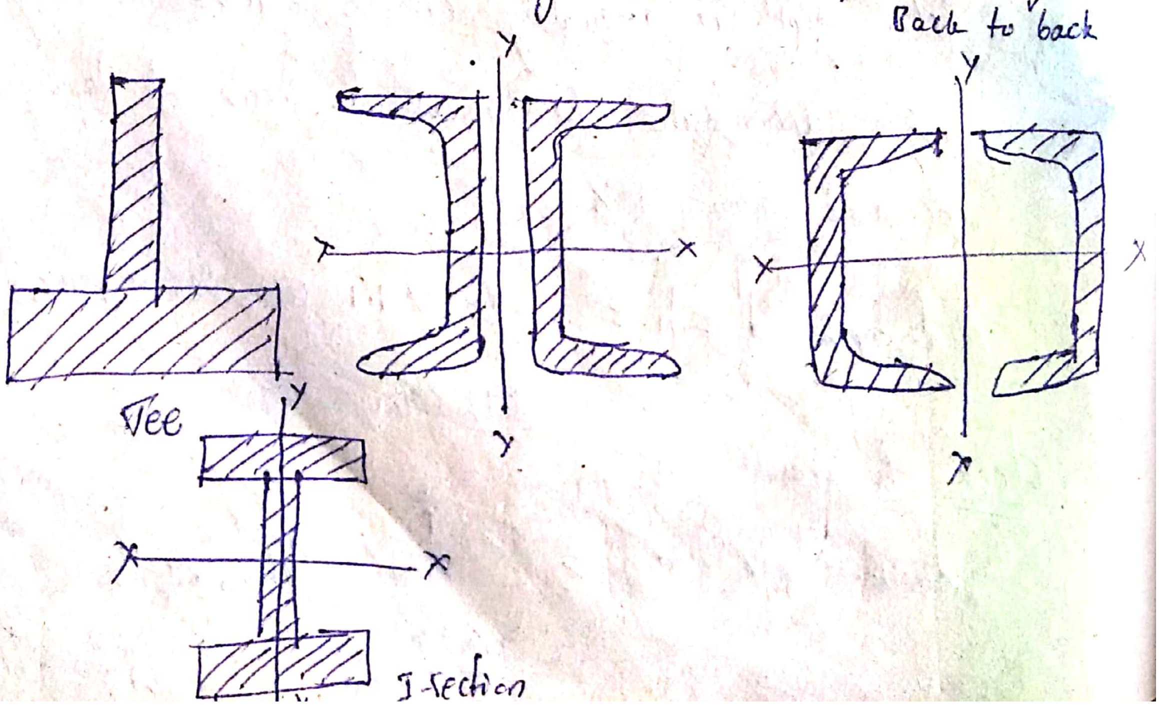
hence safe.

Design of Tension Members :-

If a number carries direct load then the member is called Tension Members.

Forms of Tension Members :-





Note:- The factored design tension (T) in the member should be less than the design strength of the member (i.e $T < T_d$)

1. The design strength of the member is the least of
   a) design strength due to yielding of gross c/s (T_{dg})
   b) design strength due to rupture of critical section (T_{dn})
   c) Design strength due to block shear (T_{db})

(7) Calculate the design strength due to yielding of Gross c/s for an angle I.S.A 100mm x 65mm x 10mm with yield stress of 250 Mpa.

Sol:

ISA 100mm x 65mm x 10mm

T_{dg} is govered by

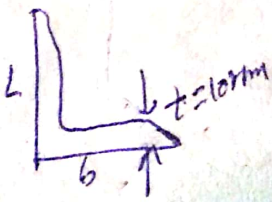
$$T_{dg} = \frac{A_g \cdot f_y}{\gamma_{m0}}$$ [From page 32 clause 6.2]

where f_y = yield strength = 250 mpa

A_g = Gross area of c/s

$= 15.51\ cm^2$ [From page 13 steel Table]

Calculation of A_g by Manually:



$$A_g = \left(L - \frac{t}{2}\right)t + \left(b - \frac{t}{2}\right)t$$

$= 15.5\ cm^2$

$$T_{dg} = \frac{250 \times 1550}{1.10} \quad [\because \gamma_{m0} = 1.0]$$

[Page no: 30, Table 5) Code book (i)

$$= 352.27 \times 10^3\ N$$

H.W

(*) Calculate the design strength due to yielding of Gross c... for an angle I.S.A $90 \times 90 \times 6$ with yield stress of 30...

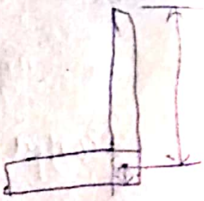
Sol:

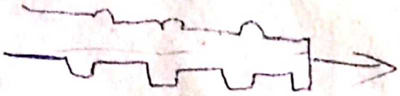
Design strength $T_{dg} = \frac{A_g \cdot f_y}{\gamma_{m0}}$

From page 49 steel book $A_g = 10.47\ cm^2 = 10.47\ (100)^2 = \ldots$

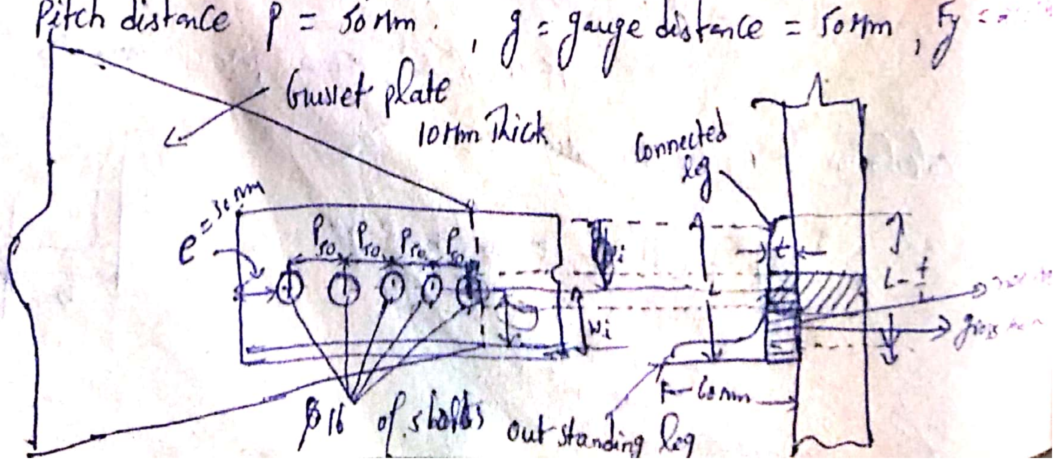
$$T_{dg} = \frac{104700\ mm \times 300}{1.1}$$

$$\boxed{T_{dg} = 28050.5 \times 10^3\ N = 285\ kN}$$





(*) A single unequal angle I.S.A $90 \times 60 \times 6$ is connected to a gusset plate of 10mm Thick with 5 no.of bolts of 16mm ... Determine the design strength. Take e = edge distance = ... Pitch distance p = 50mm, g = gauge distance = 50mm, f_y = ...



Sol:

Given Data

ISA $90 \times 60 \times 6$

no. of bolts $= 5$

dia of bolt $= 16$ mm

Thickness of Gusset plate $t_g = 10$ mm

1) Design strength due to yielding of Gross Cross Section

$$T_{dg} = \frac{A_g \cdot f_y}{\gamma_{m0}}$$ [code book page 32]

where A_g = Gross Area of an angle $= 8.65\ cm^2$ [page 13 steel table]

$\gamma_{m0} = 1.10$ [Table 5 of page no: 30 code book]

$$\therefore T_{dg} = \frac{8.65 \times 10^2 \times 250}{1.1} = \cancel{1965 \times 10^6}\ N \quad 196.5 \times 10^3\ N$$

$= 196.5$ kN

2) Design strength due to rupture

$$T_{dn} = \frac{0.9 A_{nc} f_u}{\gamma_{m1}} + \frac{\beta \cdot A_{go} \cdot f_y}{\gamma_{m0}}$$ [page 33 clause 6.3.3]

where $\beta = 1.4 - 0.076 (w/t)(f_y/f_u)(b_s/L_c) \leq \left(\frac{f_u \gamma_{m0}}{f_y \cdot \gamma_{m1}}\right)$

≥ 0.7

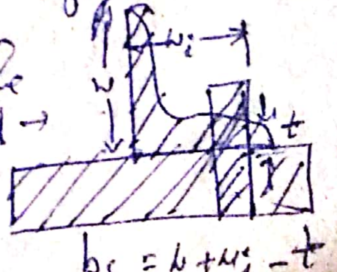
where w = width of unconnected length (or) out standing leg.

$= 60$ mm

t = Thickness of the angle or leg. $= 6$ mm

b_s = shear lag width as shown in fig.

Angle with single leg connected →



$b_s = w + w_1 - t$

w_j ~~$(L-g)$~~ $b_s = w + w_i - t = 60 + 50 - 6$
$= 104 \text{ mm}$

Note:- Shear lag :-

Consider a rectangular flat subjected to Tension along one edge of the flang. The normal stress distribution and deformation near the ends are the forces are applied is not uniform at ~~un~~loaded edge i.e more normal stress or more elongation at unconnected leg area and less normal stress or elongation at This phenomenon is due to the variation of stress and shear deform called shear lag.

→ This phenomenon has a practical significance in the design of steel structures such as angles, channels and Tee sections as there are connected to Gusset plate through 1 leg or web then used as Tension or Compression member

L_c = Length of end connection, i.e is the distance b/w the outer most bolts $= 4P$

$= 4 \times 50 = 200 \text{ mm}$
↓
spacing b/w bolts

$$\therefore \beta = 1.4 - 0.076 \left(\frac{w}{t}\right)\left(\frac{f_y}{f_u}\right)\left(\frac{b_s}{L_c}\right) \nless \frac{\left(\frac{f_u}{f_y}\right)}{\left(\frac{\gamma_{m1}}{\gamma_{m0}}\right)} \quad \geq 0.7$$

$$= 1.4 - 0.076\left(\frac{60}{6}\right)\left(\frac{250}{410}\right)\left(\frac{104}{200}\right)$$

$$= 1.15 \nless \left(\frac{\frac{410}{250}}{\frac{1.25}{1.10}}\right) = 1.15 \nless 1.44 > 0.7$$

$\therefore$ Considered $\beta = 1.15$

A_{nc} = net Area of the connected length (deduct bolt holes)

d_0 = Dia of bolt hole = Dia of bolt + allowance = 16+2 = 18 mm

$$A_{nc} = \left(L - \frac{t}{2} - d_0\right) t = \left(90 - \frac{6}{2} - 18\right) \times 6$$

$$= 414 \text{ mm}^2$$

A_{go} = gross area of outstanding leg

$$= \left(b - \frac{t}{2}\right) \times t = \left(60 - \frac{6}{2}\right) 6 = 342 \text{ mm}^2$$

$$T_{dn} = \frac{0.9 A_{nc} \cdot f_u}{\gamma_{m1}} + \frac{\beta \cdot A_{go} \cdot f_u}{\gamma_{mo}}$$

(γ_{m0}, γ_{m1}) [From Table 5 page 30

$$= \frac{0.9 \times 414 \times 410}{1.25} + \frac{1.15 \times 342 \times 410}{1.10}$$

$$= 211.58 \text{ kN}$$

3) Design strength due to block shear

$$T_{db_1} = \frac{A_{vg} \cdot f_y}{\sqrt{3} \cdot \gamma_{mo}} + \frac{0.9 A_{tn} f_u}{\gamma_{m1}}$$

[Page 33 code book]

(or)

$$T_{db_2} = \frac{0.9 A_{vn} f_u}{\sqrt{3} \cdot \gamma_{m1}} + \frac{A_{tg} \cdot f_y}{\gamma_{mo}}$$

where A_{vg} = gross Area in shear (min) along bolt line parellal to external force, respectively (1-2 and 3-4 as shown in fig. 7A and 1-2 as shown in fig 7B)

$$A_{vg} = (e + (n-1)p)\, t$$

$$= (30 + (5-1) \times 50)\, 6 = 1380 \text{ mm}^2$$

A_{vn} = Min net Area in shear along bolt line parallel to external force as shown in fig (1-2 as shown in fig)

$= [e + (n-1)p - (n - \frac{1}{2}) d_o] t$

$= [30 + (5-1) 50 - (5 - \frac{1}{2}) 18] 6$

$= 894 \text{ mm}^2$

A_{tg} = Min gross Area in Tension

$= g \times t = 50 \times 6 = 300 \text{ mm}^2$

A_{tn} = net Area in Tension (deduct bolt hole upto centre line of bolt)

$= (g - \frac{1}{2} d_o) t$

$= (50 - \frac{1}{2} \times 18) 6$

$= 246 \text{ mm}^2$

$$T_{db_1} = \frac{1380 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 246 \times 410}{1.25}$$

$= 253.69 \times 10^3 \text{ N} = 253.69 \text{ kN}$

$$T_{db_2} = \frac{0.9 \times 894 \times 410}{\sqrt{3} \times 1.25} + \frac{\cancel{0.9 \times 246} \; 300 \times 250}{1.1}$$

$= 220.54 \text{ kN}$

T_{db} = least of T_{db_1} & T_{db_2}

$= 220.54 \text{ kN}$

at this load the angle failed in block shear

The design strength of given member when it is connect to Gusset plate by adopting given edge, pitch, gauge distances is least of T_{dg}, T_{dn} and T_{db}

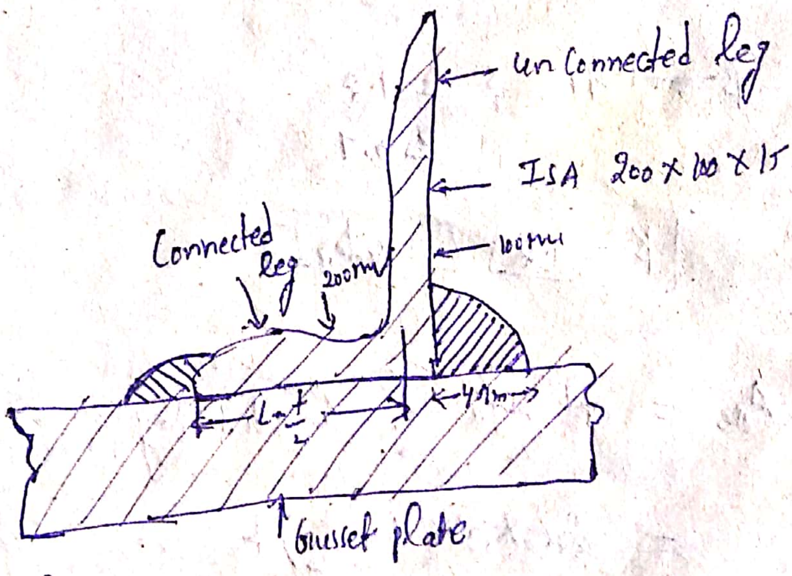
$\therefore$ The Design strength = 196.54 kN.

$\therefore$ Working load or safe load or permissable load

$$= \frac{T}{\text{Factor of safety}} = \frac{196.54}{1.5} = \underline{131\ kN}$$

(*) A single angle I.S.A 200×100×15 mm is connected to a gusset plate of thickness 12 mm by a fillet weld of 4 mm size. Determine the design strength. use Yield strength f_y = 300 Mpa, f_u = 410 Mpa, Length of weld = 240 mm.

Soln:-



Properties of ISA 200×100×15 :-

A_g = 42.78 cm^2 (page no: 13 steel Table)

$$\therefore T_{dg} = \frac{A_g \cdot f_y}{\gamma_{mo}}$$ [Page 32 Code IS 800]

$$T_{dg} = \frac{4278 \times 300}{1.1} = 1.166 \times 10^6 \, N = 1166 \, kN$$

Design strength due to rupture

$$T_{dn} = \frac{0.9 A_{nc} f_u}{\gamma_{m1}} + \frac{\beta \cdot A_{go} \cdot f_y}{\gamma_{m0}}$$

where

$$\beta = 1.4 - 0.076 \left(\frac{w}{t}\right)\left(\frac{f_y}{f_u}\right)\left(\frac{b_s}{L_c}\right) \leq \left(\frac{f_u \cdot \gamma_{m0}}{f_y \cdot \gamma_{m1}}\right)$$

$\boxed{b_s = w}$ page 33 fig 6 L_c = distance b/w outermost bolts]

≥ 0.7

$$\beta = 1.4 - 0.076 \left(\frac{100}{15}\right)\left(\frac{300}{410}\right)\left(\frac{100}{240}\right) \leq \left(\frac{410 \times 1.1}{300 \times 1.25}\right)$$

data

$= 1.245 \leq 1.2 \geq 0.7$

$\therefore$ ~~$\beta = 1.244$~~, $\beta \leq 1.2$

≥ 0.7

$\boxed{\beta = 1.2}$ [Considered]

A_{nc} = net Area of Connected leg [There is no bolts so, it = gross Area of Connected]

$= \cancel{200 \times 15} \, (L - \frac{t}{2})t$

$= \left(200 - \frac{15}{2}\right) \times 15 = 2887.5 \, mm^2$

A_{go} = gross Area of outstanding leg

$= \left(100 - \frac{15}{2}\right) 15 = 1387.5 \, mm^2$

$$T_{dn} = \frac{0.9 \times 2887.5 \times 410}{1.25} + \frac{1.2 \times 1387.5 \times 300}{1.1}$$

$$= 1.306 \times 10^6 \, N = 1306 \, kN$$

Design strength due to block shear

$$T_{db_1} = \frac{A_{vg} \cdot f_y}{\sqrt{3} \cdot \gamma_{m0}} + \frac{0.9 \, A_{tn} \cdot f_u}{\gamma_{m1}}$$

(or)

$$T_{db_2} = \frac{0.9 \, A_{vn} \cdot f_u}{\sqrt{3} \cdot \gamma_{m1}} + \frac{A_{tg} \cdot f_y}{\gamma_{m0}}$$

Where, $A_{vg} = A_{vn}$ = net Area in shear (or) gross Area in shear

= Total length of weld $\times 2.5$

$= 240 \times 2.5 = 240 \times 2 \times 4$

$= 1920 \, mm^2$

$A_{tg} = A_{tn}$ = gross Area in Tension (or) net Area in Tension.

= long leg length $\times 2.5$

$= 200 \times 2 \times 4$

$= 1600 \, mm^2$

$$T_{db_1} = \frac{1920 \times 300}{\sqrt{3} \times 1.1} + \frac{0.9 \times 1600 \times 410}{1.25}$$

$$= 774.64 \times 10^3 \, N = 774 \, kN$$

$$T_{db_2} = \frac{0.9 \times 1920 \times 410}{\sqrt{3} \times 1.25} + \frac{1600 \times 300}{1.1}$$

$$= 763.59 \times 10^3 \, N = 763 \, kN$$

$\therefore$ The design strength of an I.S.A $200\times100\times15$ mm
is the least of T_{dg}, T_{dn}, $\underbrace{T_{db_1} \text{ \& } T_{db_2}}_{\text{least}}$

$$\boxed{T = 763\,kN}$$

(*) Design procedure of Tension Member

1. Find the Tensile load or working load [P]
2. For limit state Factored load Pu find
3. Area required calculate as $A_{rq} = \dfrac{\text{Factored load}}{\text{Yield stress}} = \dfrac{P}{(f_y/\gamma_m)}$
4. Providing section (Choose suitable section which has more than area required). roughly 25-40% increment in Area and connect by using
   i) welding
   ii) Bolting
   iii) Rivetting

   and check

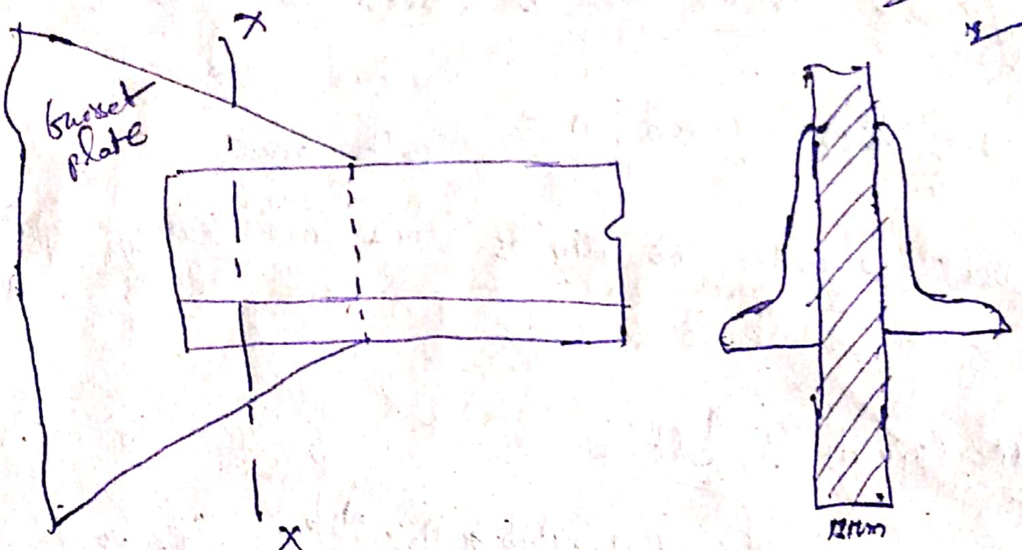
(*) (I) Design Double angle Tension member connected one each side of 12mm Thick Gusset plate to carry an axial load of 300kN. use 20mm bolts. $F_y = 250$ Mpa

sol: Given $P = 300\,kN \longrightarrow P_u = 1.5\times300 = 450\,kN$

$t_g = 12\,mm$, $f_y = 250\, N/mm^2$

d_0 = dia of bolt hole = dia of bolt + 2mm

$= 20 + 2 = 22\,mm$



c/s x-x

Area required = A_{req} = $\dfrac{\text{Factored load}}{\text{Yield stress}} = \dfrac{450\times10^{3}}{\left(\dfrac{250}{1.1}\right)} = \dfrac{P_u}{\left(\dfrac{f_y}{\gamma_{m0}}\right)}$

$A_{req} = 1980 \text{ mm}^2$

∴ For each angle area required for single L $= \dfrac{1980}{2}$

$= 990 \text{ mm}^2$

Provide an angle section which has more than the area calculated [Normally 25–40% Increase]

Increment Area = 35% $= 990\times\dfrac{35}{100} = 346.5 \text{ mm}^2$

∴ Total area for each angle $= 990 + A_{increment}$

$= 990 + 346.5$

$= 1336.5 \text{ mm}^2 \simeq 1337 \text{ mm}^2 > A_{req}$

hence ok.

Choose ISA 100×75×8 [Page 13 steel book]

∴ $A_g = 1336$ (from steel Table) $= A_{req}$

Design of Connection (use bolted Connection)

No. of bolts required, $n = \frac{\text{Factored load}}{\text{Bolt value}}$

Bolt value is the min of shear and bearing Capasity of bolt

Shear Capasity of bolt :-

$$V_{dsb} = \frac{\frac{f_u}{\sqrt{3}}(n_n \cdot A_{nb} + n_s \cdot A_{sb})}{\gamma_{mb}}$$ [Pag no: 75 Code book] [Clause 10.3.3]

Where f_u = ultimate Tensile strength of a bolt

Take $n_n = 1$ for double angles [n_n is the no. of shear planes with threads intercepting shear plane]

$n_s = 0.78$ for both single & double angles [n_s is the no. of shear planes without threads intersepting the shear plane]

$A_{sb} = A_{nb}$ = nominal plane shank area of the bolt (or) net shear area of the bolt at threads may be taken as the area corresponding to root dia of the thread.

$= \frac{\pi}{4} d^2 = \frac{\pi}{4} \times (20)^2 = 314.15 \; mm^2$

$\gamma_{mb} = 1.25$ [From Table 5 Page 30]

$P = 6 \times A$
$= f_u \times \frac{\pi}{4} d^2$

$f_u = 410$ Mpa

$$V_{dsb} = \frac{\frac{410}{\sqrt{3}}(1 \times 314.15 + 0.78 \times 314.15)}{1.25}$$

$= 105.89$ kN

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Bearing Capasity of bolt = $V_{dpb} = \frac{2.5\, k_b \cdot t \cdot d \cdot f_u}{\gamma_{mb}}$ [Page 75] 10.3.4

where V_{dpb} = The design bearing strength of a bolt on any plate

k_b is the factor as follows and Consider least of $\frac{e}{3d_0}$, $\frac{p}{3d_0} - 0.25$, $\frac{f_{ub}}{f_u}$, 1.0

$\frac{e}{3d_0} = \frac{30}{3 \times 22}$ [where e, p = end & pitch distances of the fastioner along bearing direction] [$e = 30mm$, $P = 50mm$]

$= 0.45$

$\frac{p}{3d_0} - 0.25 = \frac{50}{3 \times 22} - 0.25 = 0.5$ [

$\frac{f_{ub}}{f_u} = 1$ [f_{ub}, f_u = ultimate Tensile stress of the bolt & the ultimate Tensile stress of the plate res]

(assume $f_{ub} = f_u$ = made of Same material for plate & bolt, so Tensile strength will be same) ~~Same~~

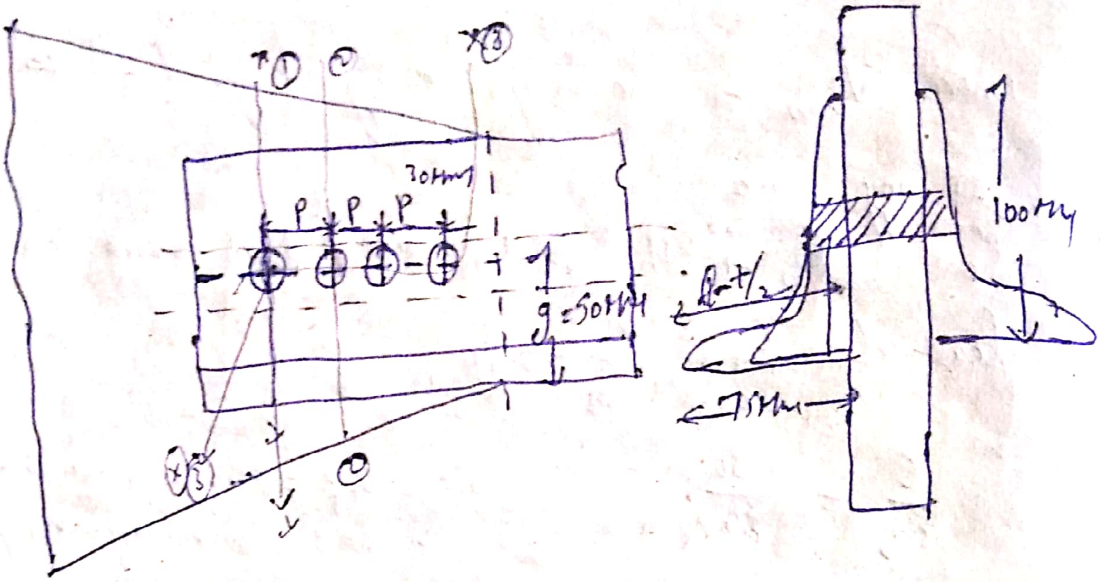
$k_b = 0.45$ [Thicness t = 100×75×8]

$\therefore V_{dpb} = \frac{2.5 \times 0.45 \times 20 \times 8 \times 410}{1.25} = 59.04kN$

∴ Bolt value = min of shear strength of bolt or bearing capasity of bolt. = 59.04 kN.

∴ No. of bolts required = $\frac{P_u}{\text{Bolt value}} = \frac{450 \times 8}{59.04} = \underline{\underline{8}}$

$= \underline{\underline{4}}$



Check:-

i.e for the choosen section I.S.A 100×75×8 mm with the Connection of Gusset plate by means of 4 bolts, $e = 30$ mm, $P = 50$ mm and $g = 50$ mm. find T_{dg}, T_{dn}, T_{db} & ensure least of those that should be > factored load Carried by each angle i.e $\frac{P_u}{2} = \frac{450}{2} = 225$ kN

This strength should be more than the given load

1) $\underline{T_{dg}}$

$$T_{dg} = \frac{A_g \cdot f_y}{\gamma_{m0}} = \frac{1336 \times 250}{1.1} = 303.6 \text{ kN}.$$

$$T_{dg} > \frac{P_u}{2}$$

2) $\underline{T_{dn}}$:-

$$T_{dn} = \frac{0.9\, A_{nc}\, f_u}{\gamma_{m1}} + \frac{\beta \cdot A_{go} \cdot f_y}{\gamma_{m0}}$$

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$$\beta = 1.4 - 0.076\left(\frac{w}{t}\right)\left(\frac{f_y}{f_u}\right)\left(\frac{b_s}{L_c}\right) \le \left(\frac{f_u \gamma_{m0}}{f_y \gamma_{m1}}\right) \ge 0.7$$

$b_s = w + w_1 - t = 75 + 50 - 8 = 117\,mm$

$$= 1.4 - 0.076\left(\frac{75}{8}\right)\left(\frac{250}{410}\right)\left(\frac{117}{3\times 50}\right)$$

$$A_{nc} = \left[\left(l - \frac{t}{2}\right)t - t\times d_0\right] \text{ (or) } \left(l - \frac{t}{2} - d_0\right)t$$

$$A_{nc} = \left(100 - \frac{8}{2} - 22\right)8 = 592\,mm^2$$

$$A_{go} = \left(b - \frac{t}{2}\right)t = \left(75 - \frac{8}{2}\right)8 = 568\,mm^2$$

$$\beta = 1.06 \le (1.44) \ge 0.7 \quad \therefore \boxed{\beta = 1.06}$$

$$T_{dn} = \frac{0.9 \times 592 \times 410}{1.25} + \frac{1.06 \times 568 \times 250}{1.1}$$

$$= 311.59\,kN > \frac{P_u}{2} = 250\,kN$$

T_{db} :- Need not found block shear because it $> T_{dg}, T_{dn}$

$\therefore$ The design strength is considered as the least of T_{dg}, T_{dn}, T_{db} i.e 303.6 kN and it is $> \frac{P_u}{2} = 250\,kN$

$\therefore$ hence design is safe.

Date
14-2-2017

UNIT- II

Design of Compression members

Compression Member:- An element or a member subjected to primary compression is called a compression member. These are 2 types.

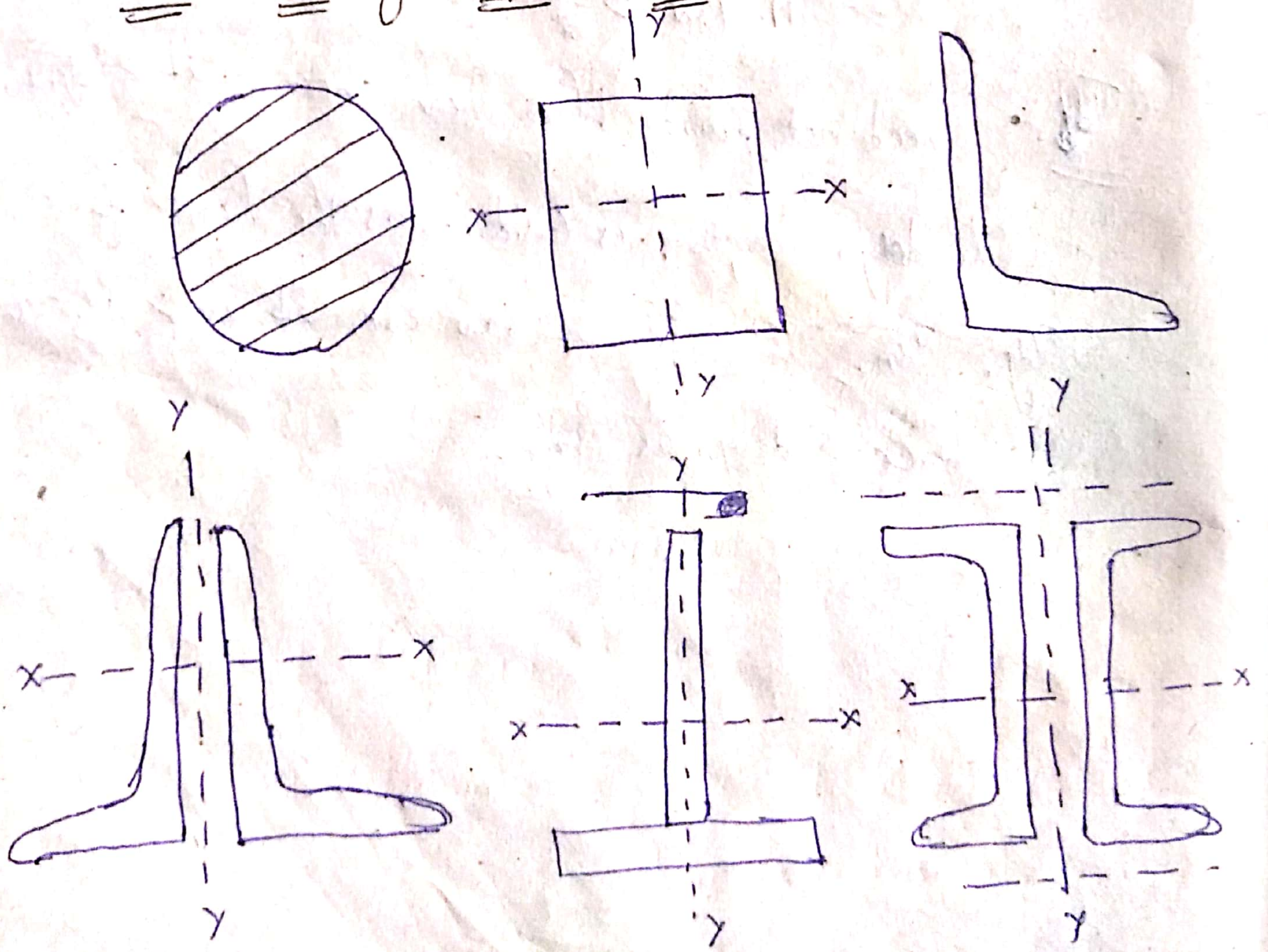
1. Columns
2. strut

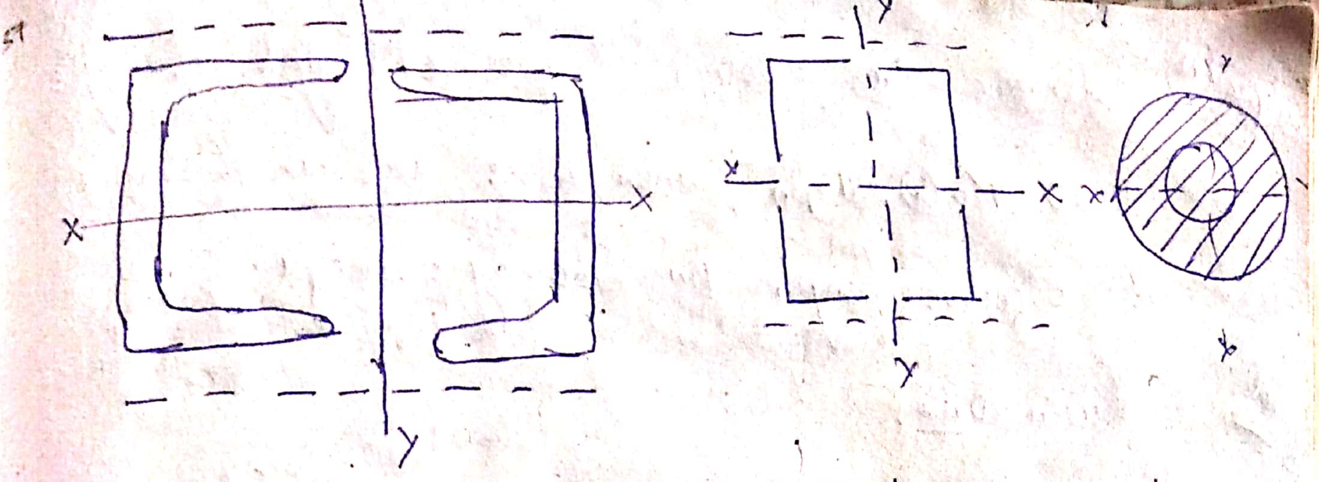
→ The vertical compression member in a building frame is called column or post or stanchion

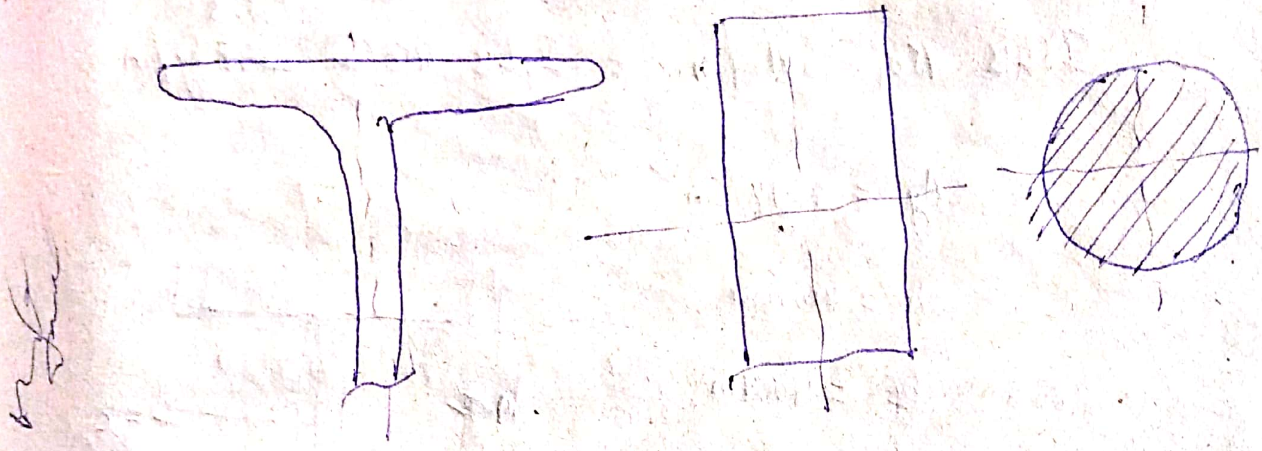
→ Compression Member in a truss is called strut.

V.V.Imp The compression member of a crane is called "boom".

Different forms of Compression member







Design strength of Compression member :- (P_d)

The design strength of Compression member is given by

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

Where A_e = effective Sectional area

f_{cd} = The design Compressive stress $= \left(\frac{f_y}{\gamma_{mo}}\right) \chi$

Where χ = stress-reduction factor

Note :- Design strength of Compressive member shall be > than the action on the member

$$\therefore P_d > P$$

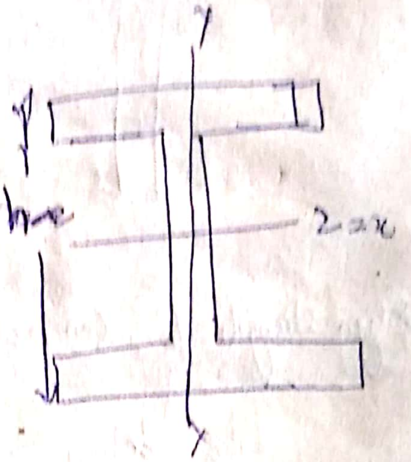
P = load applied on the member.

(11) Determine the design compressive strength of single ISLB 450 @ 641 N/m, when it is used as follows column of effective length 4m. The f_y = 250 MPa.

Sol: Given Data

ISLB 450 @ 641 N/m = ISLB 450 @ $\cancel{64.88}$ 65.3 kg/m

A_g = 83.14 cm^2



h = 450 mm

b_f = 170 mm

t_f = 13.4 mm

$r_{zz} = r_{xx}$ = 18.20 cm = 182 mm

r_{yy} = 3.2 cm = 32 mm

Classification of section

The classification of section based on the depth - breadth ratio

i.e $\frac{h}{b_f}$ and Thickness of the flange

(i) $\frac{h}{b_f} = \frac{450}{170} = 2.647 > 1.2$

ii) t_f = 13.4 mm < 40 mm

(Page 44)

refer Table No: 10 of IS 800-2007 for buckling class of section

The given section about Z-Z axis under the category of class (a)

about Y-Y axis - under the buckling class b

Effective slenderness ratio :-

about z-z axis, $\frac{le}{r_{zz}} = \frac{4000}{182} = 21.97$

about y-y axis $\frac{le}{r_{yy}} = \frac{4000}{32} = 125$

Design Compressive stress :-

To find f_{cd} :-

(i) Based on the slenderness ratio = 21.97
(or) (about z-z axis)

from the Table 9a of IS 800-2007 (Page 40)

$\frac{le}{r_{zz}}$	$f_y = 300$ Mpa
x_1 20	270 y_1
x_2 21.97	$y_2 = y = ?$
x_2 30	262 y_2

By Enterpolation

$270 + \frac{262-270}{30-20}(21.97-20)$

$= 268.42$ Mpa

$\therefore$ $\boxed{f_{cd} = 268.42 \text{ Mpa}}$

$\frac{y_2 - y_1}{x_2 - x_1} = \frac{262-270}{30-20} = -0.8$,

$\Rightarrow \frac{y - 270}{21.97-20} = -0.8 \Rightarrow \boxed{y = 268.48 \text{ Mpa}}$

$\therefore$ The design compressive stress about z-z axis (buckling class a)

$f_{cd} = 268.48$ Mpa

(ii) Based on the slenderness ratio = 125 (about y-y axis buckling class b)

Page 41 (Table 9b)

$f_{cd} = \frac{95.4 + 83.7}{2} = 89.55$ Mpa (or) Interpolation also same

∴ The design compressive stress under the minor axis (Y-Y axis) belong to buckling class b $f_{cd} = 89.55$ Mpa

∴ Consider lesser value of f_{cd}

∴ $\boxed{f_{cd} = 89.55 \text{ Mpa}}$

∴ Design strength of compression member $P_d = A_e \cdot f_{cd}$

⇒ $P_d = 8314 \times 89.55 = 744.51$ kN

∴ $\boxed{P_d = 744.51 \text{ kN}}$ [P_d applied it is bending about Y-Y axis]

Design procedure

1. Find design stress in compression (f_{cd})
2. Design stress in compression for single rolled steel sections may be assumed as 135 Mpa $= f_{cd}$
3. For angle struts f_{cd} may be assumed as 90 Mpa
4. For compression member carrying large loads, f_{cd} may be assumed as 200 Mpa
5. Find the sectional area required

$$A_{e\,req} = \frac{P}{f_{cd}}$$

6. select a section to given effective area required and calculate min radius of gyration.

7. Knowing the end conditions or support conditions and find effective length of the member based on Table 11, 9 page 45 of IS-800-2007.

8. Find the slenderness ratio & hence design stress f_{cd} and load carrying capacity P_d calculate.

9. If $P_d >$ given load, Then the design is safe. if not redesign by choosing another section.

(A) Design a steel column using single rolled I section to carry an axial load of 800kN. both ends of the columns are restrained against translation and rotation. (i.e both ends fixed.) The actual length column b/w intersections is 8m. The $f_y = 280$ Mpa.

Sol:

Given $P = 800$ kN

$\Rightarrow P_u = 1.5 \times 800 = 1200$ kN.

Assume, buckling class b about z-z axis and also assume slenderness ratio 100. i.e $\frac{le}{r_{min}} = 100$

From the table 9b of IS 800-2007, $f_y = 280$ Mpa, [Page 41]

The design compressive stress $f_{cd} = 123$ Mpa

assume $f_{cd} = 125$ Mpa

$\therefore$ sectional area required $A_{e\,req} = \frac{P}{f_{cd}}$

$$= \frac{1200 \times 10^3}{123} = 9600 \text{ mm}^2$$

$$= 9756 \text{ mm}^2$$

From steel Table, page (?). choose ISHB 400 @ 77.4 kg/m

Properties

$h = 400\ mm$

$b_f = 250\ mm$

$t_f = 12.7\ mm$

$r_{xx} = r_{zz} = 16.87\ cm = 168.7\ mm$

$r_{yy} = 5.26\ cm = 52.6\ mm$

$A_g = 98.66\ cm^2 = 9866\ mm^2 > A_{e\,req}$.

Procedure:-

radius of gyration. $r_{zz} = 168.7\ mm$

$r_{yy} = 52.6\ mm$

Total length $= 8m = 8 \times 10^3\ mm$

Effective length (both ends fixed) $= \frac{1}{\sqrt{2}} \times L = 0.65L$ [Page 45 Table 11]

$l_{eff} = 0.65 \times 8000 = 5200\ mm$

Slenderness ratio:-

$$\frac{l_{eff}}{r_{zz}} = \frac{5200}{168.7} = 30.82$$

$$\frac{l_{eff}}{r_{yy}} = \frac{5200}{52.6} = 98.85$$

To find f_{cd}:-

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40

(i) Based on slenderness ratio 168.7 mm (about zz axis class b)

$\frac{l_e}{r_{zz}}$		f_{cd} (Mpa)
x_1	30 ~~160~~	~~[illegible]~~ y_1 240
x_2	30.82 ~~[illegible]~~	y =
x_2	40 ~~170~~	~~[illegible]~~ 228 y_2

$$240 + \frac{228-240}{40-30}(30.82-30)$$

$= 239.01$ Mpa

$$\frac{y_2-y_1}{x_2-x_1} = \frac{228-240}{40-30} = -1.2$$

$$\Rightarrow \frac{y-240}{30.82-30} = -1.2 \Rightarrow y-240 = -0.984$$

$$\Rightarrow \boxed{y = 239.01} \text{ Mpa} \Rightarrow f_{cd} = 239 \text{ Mpa}$$

ii) Based on slenderness ratio 98.85 (about y-y axis)

$\frac{l_e}{r_{yy}}$	f_{cd}
90	141
98.85	
100	123

$$141 + \frac{123-141}{100-90}(98.85-90)$$

$= ~~141~~$ 125.07

$=$

$\therefore f_{cd} = 125.07$

Design strength P_d :-

$$P_d = A_e . f_{cd}$$

$P_d = A_e (f_{cd})_{xx}$

$= 9756 \times 239$

$= 233~~1~~$ kN

$P_d = A_e . (f_{cd})_{yy}$

$= 9756 \times 125.07$

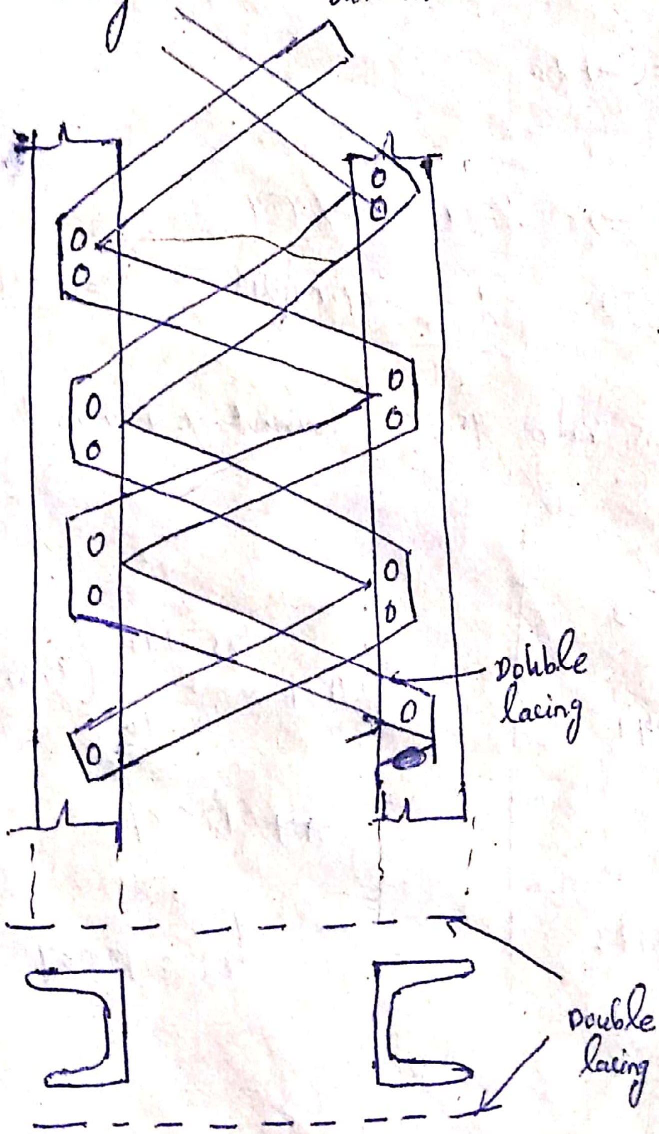
$= 122~~01~~$ kN

∴ Consider least P_d = 1220 kN > P_{cu} = 1200 kN

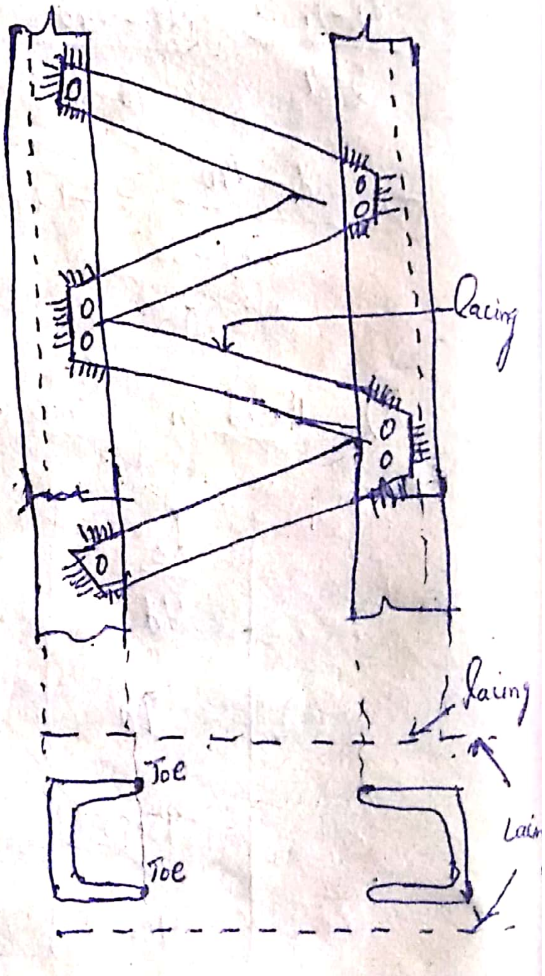
hence safe (or) The design is ok.

2) **Built-up Compression Members**

The built up compression members are used to ~~bare~~ bear the heavy loads which includes columns associated with lacings and battens



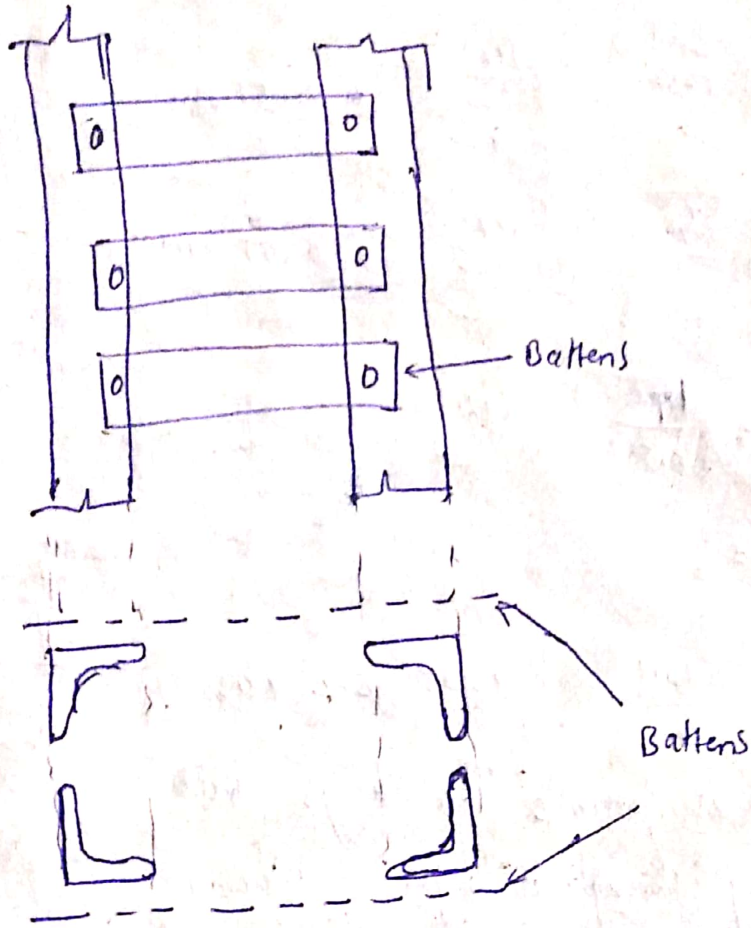
Double lacing



Single lacing

lacings

41)



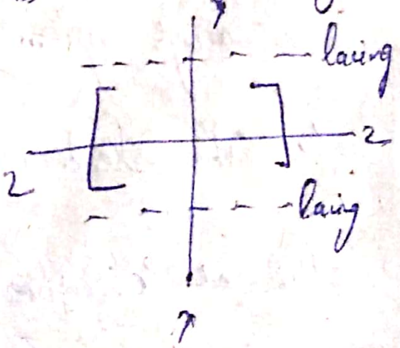
Battens

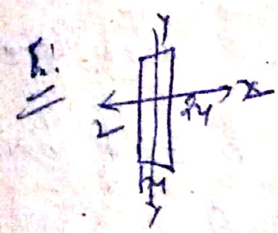
Design of Laced Columns

The lacing should satisfy the following requirements as far as possible.

A compression member having 2 main components laced and tied should where practicable have a radius of gyration about axis ⊥lar to the plane of lacing not < the radius of gyration about axis in the plane of lacing

$$\boxed{r_{yy} \nless r_{zz}}$$



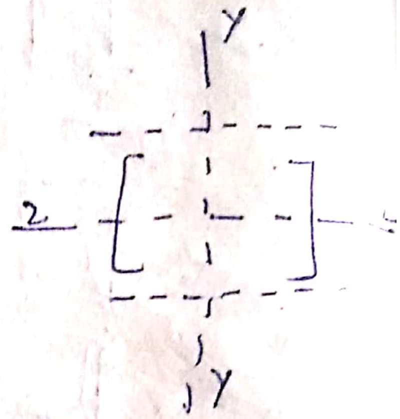


$$r_{zz} = \sqrt{\frac{I_{zz}}{A}} = \sqrt{\frac{\frac{1 r_z^3}{12}}{1 r_z}} = 0.577\ m$$

$$r_{yy} = \sqrt{\frac{I_{yy}}{A}} = \sqrt{\frac{\frac{l r_y^3}{12}}{1 r_z}} = 0.289\ m$$

$$\frac{l_e}{r_{zz}} = \frac{140}{0.577} = 242.63$$

$$\frac{l_e}{r_{yy}} =$$



i.e it will bend about 2-2 axis

ii) as far as possible The lacings shall not vary throughout length of column

iii) single laced system on opposite sides of the components shall preferably be in the same direction so that one be the shadow of the other. instead of bare mutually opposed in direction.

iv) The lacing should be designed to resist a total transverse shear = 2.5 % of load on the column and This shear should be equaly divided among all Transverse lacings system in parallal planes

v) The slenderness ratio of lacing part shall not exceed 145

vi) The effective length of lacing bar in riveted connection should be the length b/w inner end rivets of bars for single lacing & 0.7 times for double lacigs.

For welded connection; The effective length to be taken as

0.7 times the distance b/w inner ends of the weld.

vii) The angle of Inclination of lacing bar with the axis of member should not be less than 45° and should not be more than 70°.

viii) width of lacing

In bolted, rivetted connection the min width of lacing bar shall be 3 times the nominal dia of the end bolt or rivet

Nominal dia of Rivet	16	18	20	22
Min width of lacing bar	50	55	60	65

The Thickness of lacing bar should not be less than $\frac{1}{40}$ of the length b/w inner end rivets or welds for single lacing and $\frac{1}{60}$ of it's length for double lacing.

→ For spacing of lacing bar should be such that the max slenderness ratio of the member in b/w the consecutive connections should be not greater than 50. or 0.7 times the slenderness ratio of member as a whole whichever is less.

→ The connection b/w the main member and the lacing bar should be sufficient to transmit the force in the lacing bar.

→ If the column is subjected to bending, also transverse shear equal to bending shear + 2.5% of column force.

(*) → The effective slenderness ratio of laced column shall be taken as 1.05 times the actual max slenderness ratio, in order to account for shear deformation effects

⑳ A column 300 mm × 300 mm consists of 4 angles ISA 80×80×10 mm, The column is 6m long and hinged at both ends. Find the max allowable load the column can carried. and design a suitable lacing. Assume steel of grade 250 Mpa

Sol:

Since the ends are hinged ∴ (Page 45)

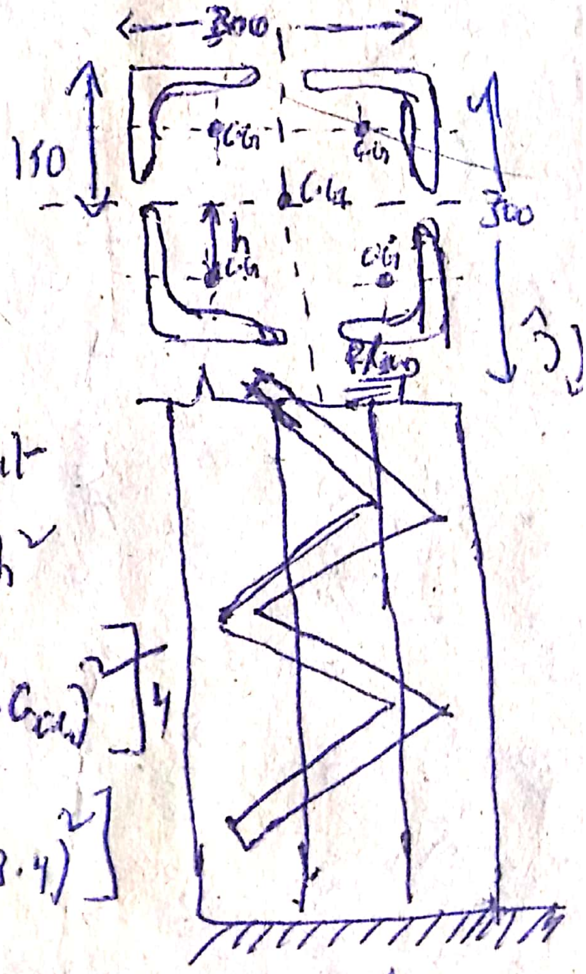
∴ effective length $l_e = 1 \times L = 1 \times 6 = 6m = 6000mm$

Properties of ISA 80×80×10

For each angle

$A = 15.05 cm^2 = 1505 mm^2$

$I_{xx} = I_{yy} = 87.7 cm^4 = 87.7 \times 10^4 mm^4$



I_{xx} = M.O.I built up column about xx = M.O.I of each angle + ah^2

$I_{xx} = [87.7 \times 10^4 + 1505 \times (150 - C_{xx})^2] 4$

$= 4[87.7 \times 10^4 + 1505(150 - 23.4)^2]$

$I_{xx} = 99.99 \times 10^6 mm^4$

$I_{xx} = I_{yy}$ because built up section is symmetric about both axis.

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radius of gyration of the total built up column

$$r_{zz} = r_{yy} = \sqrt{\frac{I_{xx}}{A}} = \sqrt{\frac{99.99 \times 10^6}{4 \times 1505}} = 128.87 \text{ mm}$$

slenderness ratio

about 2-2 axis $\frac{l_e}{r_{zz} \text{ (or) } r_{xx}} = \frac{6000}{128.77} = 46.55.$

$\frac{l_e}{r_{zz}}$	f_{cd}
40	198
46.55	$y = ?$
50	183

From Table 9c of IS 800: 2007 (page 42)

$$y = 198 + \frac{183 - 198}{50 - 40}(46.55 - 40)$$

$$\boxed{f_{cd} = 188.175 \text{ Mpa}}$$

∴ Design strength of built up column (P_d):

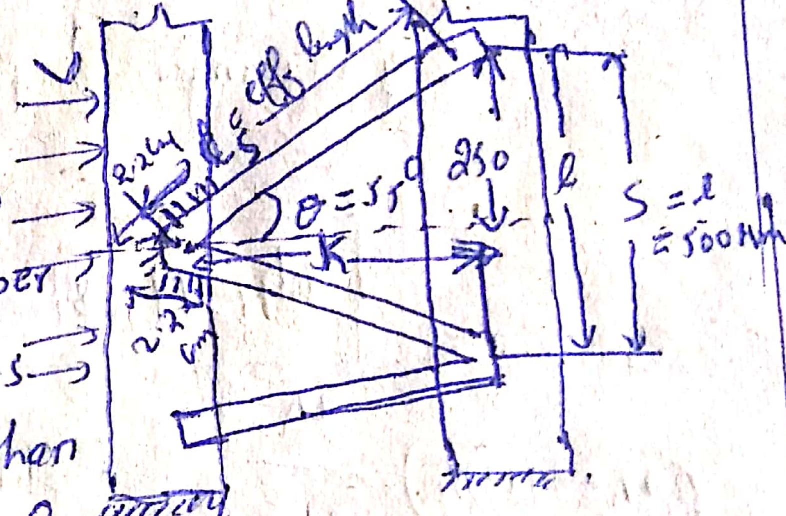
$$P_d = A_e \cdot f_{cd} = 4 \times 1505 \times 188.175$$

$$P_d = 1132 \times 10^3 N = \underline{1132 \text{ kN}}$$

∴ Working (or) safe load $= \frac{P_d}{1.5} = 754.66 \text{ kN}$

Design of lacings

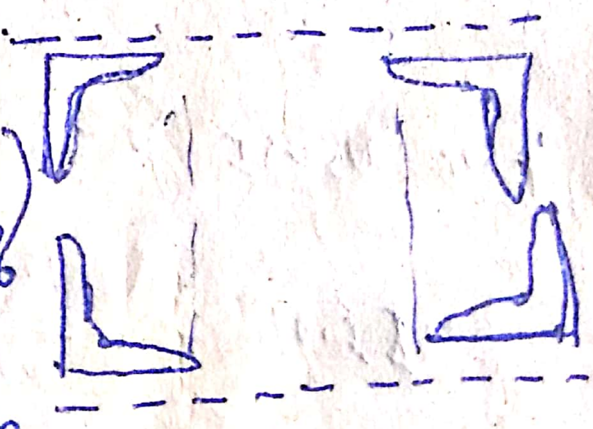
Single lacing system consists of M.S (Mild steel) flats will be provided. The inclination of lacing bar is assumed as 55° with the vertical.



Note:- The lacing should be such that min slenderness ratio of the components of the member b/w the consecutive connections of the member is not greater than 50 or 0.7 × slenderness ratio of whole built up member.

i.e $\dfrac{L}{r_{min}}$ = 0.7 × slenderness ratio of whole built up column (or)

$\ngtr 50$



$= 0.7 \times 46.55$

$= 32.58 \}$ less

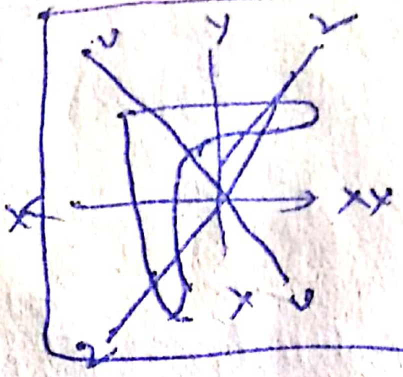
50

(Length of member = size of lacing)

$\therefore\ l = s = 32.58 \times r_{min}$ $= \cancel{32.58 \times 125.87}$

(for single component)

$s = 32.58 \times (1.55 \times 10)$ [From Steel Table page 9 $r_{vv} = 1.55\,cm$]



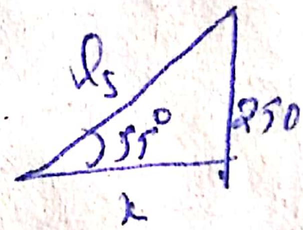
$s = 504.99\,mm$

Provide, adopt $s = 500\,mm$

$\frac{l}{2} = \frac{s}{2} = 250\,mm$

effective length of lacing bar (l_s)

The effective length of lacing bar l_s =



$\sin 55^0 = \frac{250}{l_s}$

$\Rightarrow l_s = \frac{250}{\sin 55^0}$

$l_s = 305.2\,mm$

but from the Codal provision, the effective length of lacing bar $l_{es} = 0.7 \times l_s$

$= 0.7 \times 305.2$

$= 213.6\,mm$.

Thickness of lacing bar.

Thickness of lacing bar $= t = \frac{1}{40} \times l_{es}$

$= \frac{1}{40} \times 213.6$

$t = 5.34 \simeq 6\,mm$

Width of lacing bar (b)

M.O.I of lacing bar $I_{yy} =$

r_{min} = Min. radius of gyration of lacing bar

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

$r_{min} = \sqrt{\frac{\frac{bt^3}{12}}{bt}} = \sqrt{\frac{bt^2}{12}} = \frac{t}{\sqrt{12}}$

$r_{min} = \frac{6}{\sqrt{12}} = 1.732$

Slenderness ratio of lacing bar $= \frac{l_{es}}{r_{min}}$

$= \frac{213.6}{1.732} = 123.32 < 145$

hence ok.

From the Table no: 9c page 42 of IS 800 : 2007.

$\frac{l_e}{r_{min}}$	f_{cd}
120	83.7
123.32	y = 80.51
130	74.3

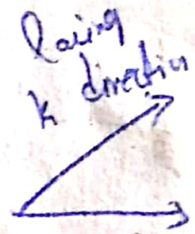
$$f_{cd} = 83.7 + \frac{74.3 - 83.7}{130 - 120}(123.32 - 120)$$

$$= 80.57 \text{ MPa}.$$

Transverse shear on lacing bar (V) :-

$$V = 754 \times 10^3 \times 2.5\,\% = 18850\,N$$

Transverse shear, sheared by 2 lacing



$\therefore$ V for each lacing $= \frac{V}{2} = 9425\,N$

$\therefore$ The Transverse shear component along the lacing bar (k)

$$\cos 55^\circ = \frac{V/2}{k} \Rightarrow k = \frac{V}{2\cos 55^\circ}$$

$$\Rightarrow k = 16431.99\,N.$$

Compressive stress on lacing bar

$$\sigma_{c_L} = \frac{16431.98}{t \times b}$$

$$\sigma_{c_L} = \frac{16431.98}{6b} \longrightarrow (1) \quad [\because t = 6mm]$$

Equate σ_{c_L} to f_{cd} (permissable)

$$\therefore \frac{16431.98}{6b} = 80.57$$

$$\Rightarrow 6b = \frac{16431.98}{80.57}$$

$$\Rightarrow b = 33.99 \Rightarrow b = 34mm \simeq 40mm$$

$\therefore$ provide $\boxed{b = 40mm}$

$\therefore$ Finally The lacing bar has length $= 213.6mm$

width $= 40mm$ and $t = 6mm$

Design of End Connection

Length of the weld required to bear $k = 16431.98N$

$$L_w \times t_e \times \frac{f_u}{\sqrt{3} \times \gamma_{mw}} = k = 16431.98$$

$$\Rightarrow L_w = \frac{16431.98 \times \sqrt{3} \times \gamma_{mw}}{t_e \times f_u}$$

$$= \frac{16431.98 \times \sqrt{3} \times 1.25}{0.7 \times 6 \times 410}$$

$$L_w = 20.65mm \simeq 22mm$$

$$\boxed{L_w = 22mm = 2.2cm}$$

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Q) Design a slab base for a column consisting of ISHB 300 @ 588 kN/m. The column carries an axial load of 800 kN. ~~also design~~ also design concrete pedestal for the column. The surfaces of the base plate and the column end are faced for full of bearing. The permissable bearing stress in concrete is $4 N/mm^2$ and S.B.C of soil is $180 kN/m^2$. 1600/wy.

Sol: Design

Given Data:

ISHB 300 @ 588 N/m

$P = 800 kN$

factored load $P_u = 1.5 \times 800 = 1200 kN$.

Bearing strength of concrete $= 4 N/mm^2$

S.B.C of soil $= 180 kN/m^2$.

1. Area of the base plate required

$$A_{req} = \frac{P_u}{0.45 f_{ck}} = \frac{1200 \times 10^3}{4} = 300 \times 10^3 mm^2$$

2. Side of square base plate $= \sqrt{3 \times 10^5} = 547.723 mm$
$\simeq 550 mm$

$\therefore$ Provide $550 mm \times 550 mm$ base plate.

Check:-

upward pressure under the base plate $w = \frac{P_u}{A_{req}}$

$$= \frac{1200 \times 10^3}{550 \times 550}$$

$$= 3.96 N/mm^2 < 4 N/mm^2$$

hence ok.

Thickness of base plate required $t_s = \sqrt{\frac{2.5w(a^2-0.3b^2)\gamma_{m0}}{f_y}}$

b = ~~greater~~ lesser projection $= \frac{550-250}{2} = 150$ mm

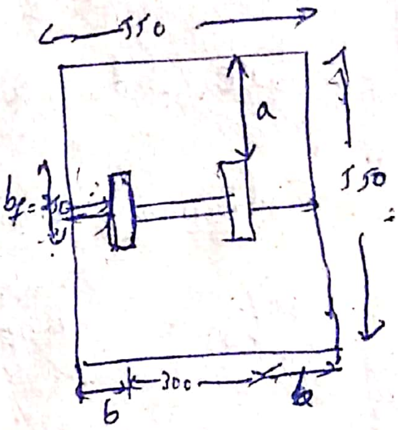
~~lesser projection~~ =

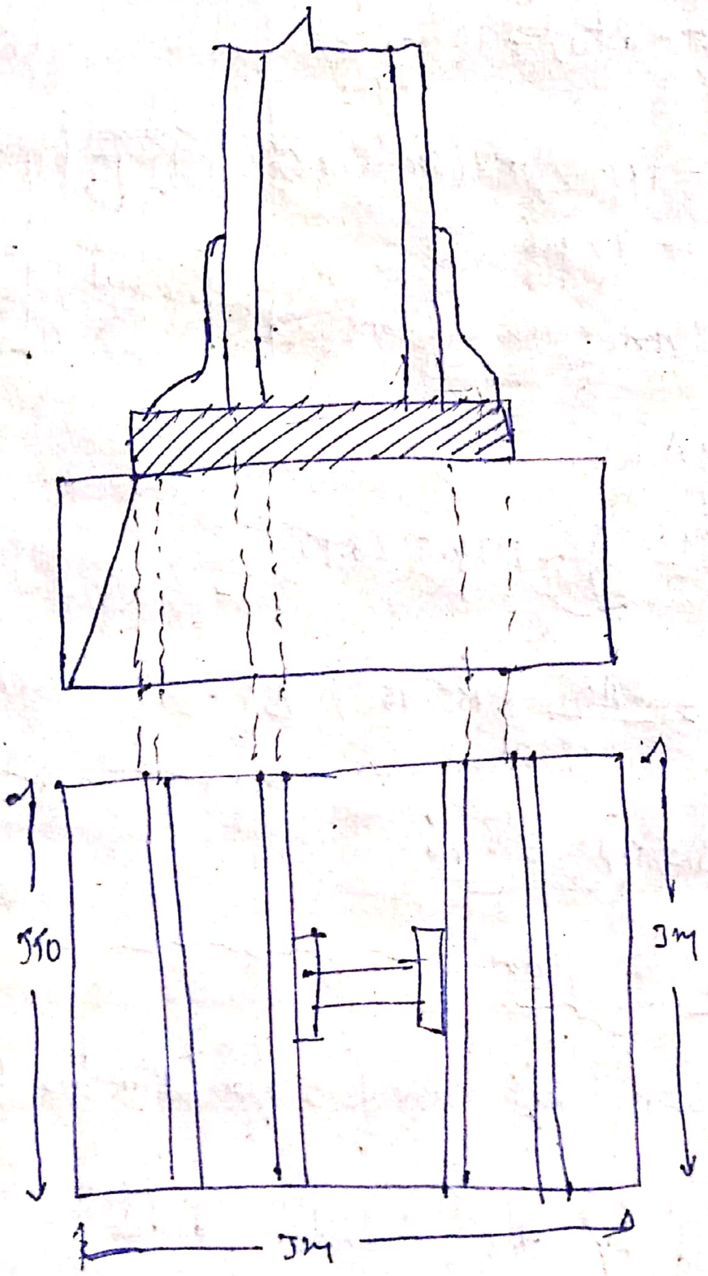
a = greater projection $= \frac{550-300}{2} = 125$ mm

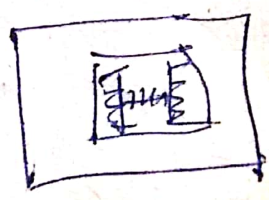
$t_s = 21.61 \simeq 22$ mm (provide)

$> 10.6\ (t_f)$

hence ok.





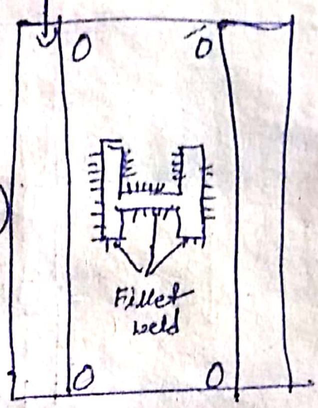


Connection:-

Provide 2 no. of I.S.A 60 x 60 x 6 mm nominal cleate angles to be connected by 2 number 18 mm dia bolts for the flanges of the column.

The cleate angles may connected to the base plate using 2 no. of 25 mm and anchor bolts.

Alternative Connection:- Concrete pedastal



$$L_w = (4 \cdot b_f - 2t_w) + 2(h - 2t_f) + 4t_f$$

$$= (4 \times 250 - 2 \times 7.6) + 2(300 - 2 \times 10.6) + 4 \times 10.6$$

$$= 1584.8 \text{ mm}$$

$$P = 6\theta \times A$$

$$= \left(\frac{f_u}{\sqrt{3}\,\gamma_{mw}}\right) \times 1584.8 \times 0.7S$$

$$1200 \times 10^3 = \frac{410}{1.25} \times 158.48 \times 0.7S$$

$$S = 5.7 \approx 6 \text{ mm}$$

Design of Concrete pedastal:-

Assume self wt of foundation has 15% of column load.

Total load by the Concrete pedastal

$$W_1 = \text{Column load} + 15\% \text{ of column load}$$

$$= 1200 + \frac{15 \times 1200}{100} = 1318 \text{ kN}$$

Area of the concrete pedastal $= \frac{L_1}{S.B.C \text{ of soil}} = \frac{1319}{180} = 7.6$

$\therefore$ length of the concrete pedastal

$= \frac{\text{Area of the concrete pedastal}}{\text{width of the concrete pedastal}}$

$= \frac{7.6}{550} = 13818 \text{ mm}$

$\therefore$ Projection of concrete pedastal beyond the base plate along the length $= \frac{13818 - 550}{2} = 6634 \text{ mm}$

$\therefore$ provide concrete pedastal of length $6634 \text{ mm} \times 550 \text{ mm}$

$\therefore$ thickness of concrete pedastal

From $\triangle^{le}$ ABC

$\tan 45° = \frac{AC}{AB}$

$AC = AB \tan 45°$

$AC = 6634 \text{ mm}$

Provide $6634 \text{ mm} \times 550 \text{ mm}$ concrete pedastal.

Check :-

wt of concrete pedastal = volume × unit wt

$= l \times b \times d \times \text{unit wt}$

$= 6634 \times 550 \times 6634 \times 24$

$= 5.80 \times 10^{11} \text{ kN/mm}$.

$\therefore$ upward soil pressure on the concrete pedastal

$= \frac{\text{Total load on the soil}}{\text{Area of concrete pedastal}}$

Scanned by CamScanner

$$= \frac{1200\times10^3 + 24\times10^3\,(6.634\times6.634\times0.55)}{6.634\times6.634\times0.55}$$

$= 489\ kN/m^2 > 180\ kN/m^2$

unsafe

To overcome this increase area of the concrete pedestal

length $l = 3m$, $b = 3m$.

$\therefore$ Thickness of the concrete pedestal $= \frac{3-0.55}{2} =$

$$= \frac{1200\times10^3 + (3\times3\times1.20)\,24\times10^3}{3\times3\times0.55}$$

$= 160\ kN/m^2 < 180\ kN/m^2$, hence ok.

Design of Gusseted base :-

Procedure :-

1. Area of base plate $A = \frac{\text{Factored load}}{0.45\,f_{ck}}$

   where f_{ck} = characteristic compressive strength of concrete block.

2. Assume various members of Gusset base
   i) Thickness of gusset plate is assumed as 16mm
   ii) Size of gusset angle is assumed as such that it's vertical leg can accomodate 2 bolts in one vertical line. Corresponding to this leg the other leg is assumed in which one bolt can be provided.
   iii) Thickness of angle is kept approximately equal to the thickness of gusset plate.

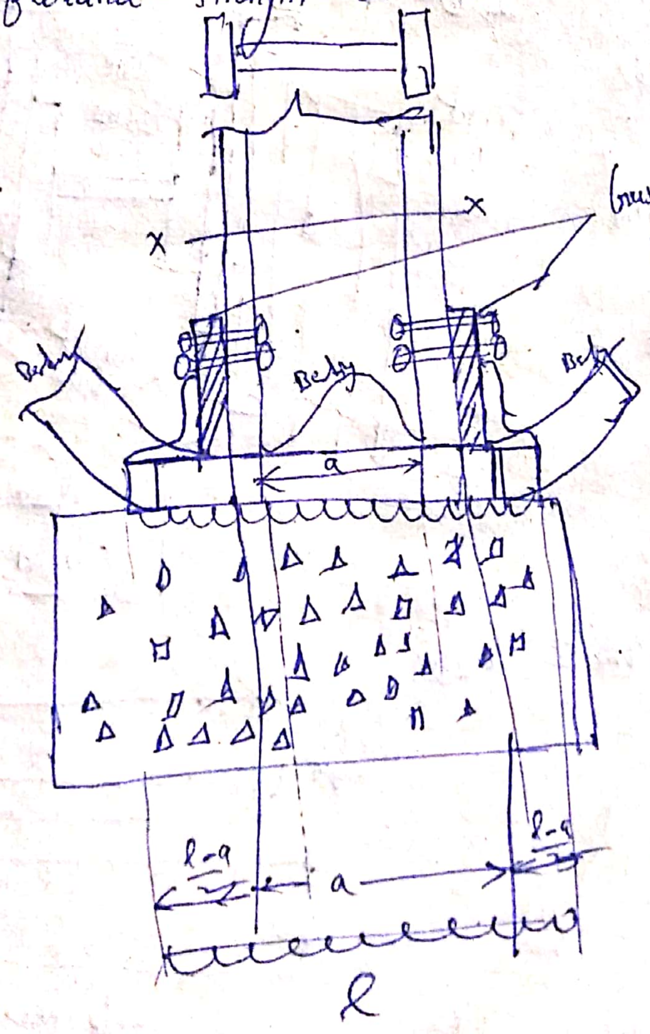
iv) Width of Gusset plate is kept such that it will just project outside the Gusset angle and hence, ~~length~~

$$\text{length} = \frac{\text{Area of plate}}{\text{width}}$$

3) When the end of the column is machined for complete bearing on the base plate, 50% of the load is assumed to be transferred by the bearing & 50% by ~~the~~ fastenings (bolts, nuts).

4) When the ends of the column shaft & Gusset plate are not faced for complete bearing, the fashionings connecting them to the base plate shall be designed to transmit all the forces to which the base is subjected.

5) The Thickness of base plate is computed by flexural strength at the critical section.

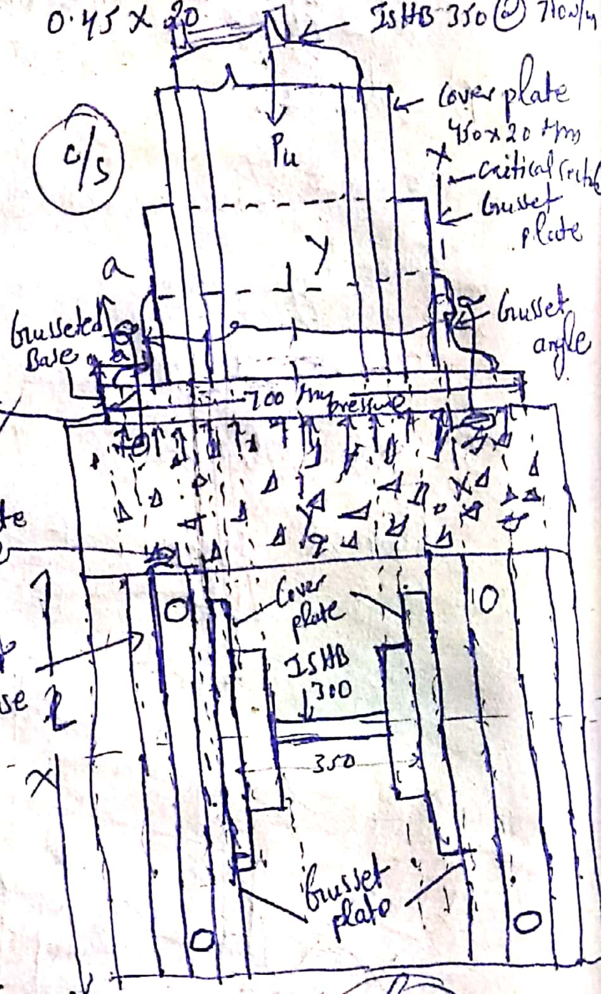


(4) Design a Gusseted base for a Column ISHB 350 @ 710 N/m with two plates 450 mm × 20 mm carrying a factored load of 3600 kN. The column is to be supported on concrete pedastal to be built with M20 concrete.

Sol:-

Area of the base plate

$$A = \frac{P_u}{0.45 f_{ck}} = \frac{3600 \times 10^3}{0.45 \times 20} = 400 \times 10^3 \ mm^2$$



Selecting ISA 150 × 115 × 15 mm and Gusset plate thickness assume 16 mm.

Consider

Min width of the Gusset plate base required

$b = 350 + (2 \times 20) + 2(16) + 2(115)$

$b = 652 \ mm \simeq 700 \ mm$

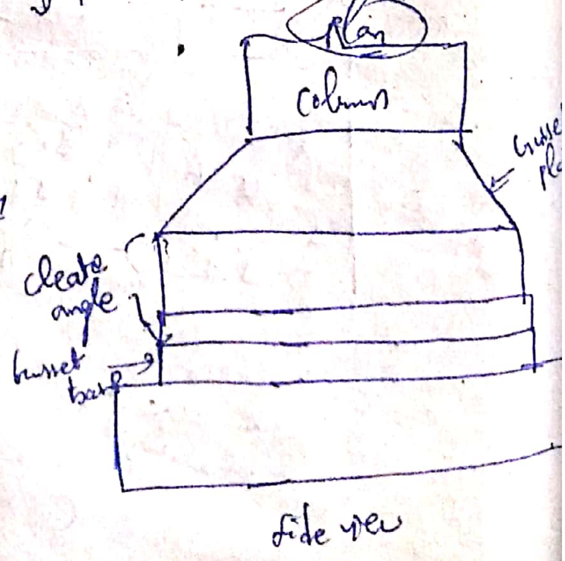
Provide 700 mm.

Length of Gusseted base plate 'L'

$$l = \frac{Area}{Width} = \frac{400 \times 10^3}{700}$$

$= 571.42 \ mm \simeq 600 \ mm$

∴ provide 600 mm



∴ provide 700 mm × 600 mm plate

Pressure under base plate = $\frac{\text{Factored load}}{\text{Area of base plate provided}}$

$= \frac{3600 \times 10^3}{700 \times 600} = 8.57\ N/mm^2$

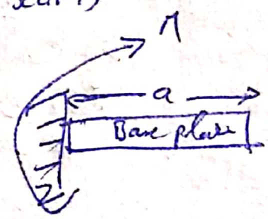
$0.45 \times 20 = 9\ N/mm^2$

∴ hence ok.

Projection $a = \frac{700 - (350 + 20 \times 2 + 2 \times 16 + 2 \times 15)}{2}$

$a = 124\ mm$

∴ Max B.M at section x-x (Critical section)



$M = wa \times \frac{a}{2} = \left(\frac{wa^2}{2}\right) \times$ limit width (mm)

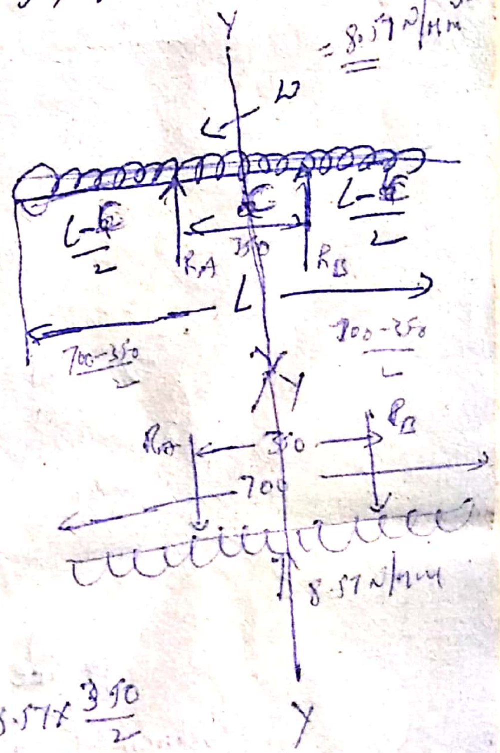
$= \frac{8.57 \times (124)^2}{2}$

$= 65.89 \times 10^3\ N\text{-}mm$

B.M at section (critical section) Y-Y

Total load $= 8.57 \times 700$

$= 5999\ N$



$R_A = R_B = \frac{5999}{2} = 2999.5\ N$

$= 2.9\ kN$

(From right ward)

$M_{yy} = -\left(R_B \times \frac{350}{2}\right) + \left(8.57 \times 350 \times \frac{350}{2}\right)$

$= -2999 \times \frac{350}{2} + 350 \times 8.57 \times \frac{350}{2}$

$= 87.5\ N\text{-}mm$

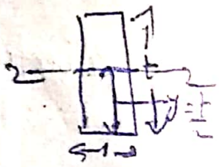
∴ Consider Max. Moment i.e M at x-x axis $= 65.89 \times 10^3$ N-mm.

Finding the Thickness of the Gusseted base plate

from the bending eq^n,

$$\frac{M}{I} = \frac{f}{y} = \frac{E}{R}$$

$$\Rightarrow \frac{M}{I} = \frac{f}{y}$$



$$I = \frac{bd^3}{12} = \frac{1 \times t^3}{12} = \frac{t^3}{12}$$

assume $f = 250\ N/mm^2$

y = Top most extreme fibre distance

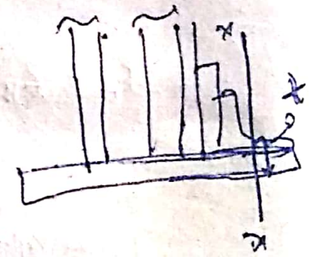
$= \frac{t}{2}$

$$M = \frac{f}{y} \times I$$

$$= f\left(\frac{t^3/12}{t/2}\right)$$

$$= f\left(\frac{t^3}{\cancel{12}_6} \times \frac{\cancel{2}}{t}\right)$$

$$M = \frac{ft^2}{6}$$



$$\Rightarrow 65.89 \times 10^3 = \frac{250 \times t^2}{6}$$

$$\Rightarrow t^2 = 1581.36$$

$$t = 39.7 \cong 40\ mm$$

$\therefore$ provide $\boxed{t = 40\ mm}$

$\therefore$ Thickness of Gusseted base = t − angle Thickness $= 40 - 15$
$= \underline{25\ mm}$

Design of Connection:-

Total axial load $= P_u = 3600\ kN$

The load shared by Gusset plates, let it be

assume 50% (from code)

$\therefore = 3600 \times \frac{50}{100} = \underline{1800\ kN}$

$\therefore$ force resisted by each gusset plate $= \frac{1800}{2} = 900\ kN$

Design of bolted Connections :-

assume size of the bolt $= 16\ mm$

$\therefore$ dia of bolt hole $= 16 + 2 = \underline{18\ mm}$

bearing strength of the bolt $= \frac{2.5\, k_b\, d t f_u}{\gamma_{mb}}$ (page 75)

$k_b = \frac{e}{3 d_0} = \frac{30}{3 \times 18} = 0.55$

$= \frac{p}{3 d_0} - 0.25 = \frac{50}{3 \times 18} - 0.25 = 0.67$

$= \frac{f_{ub}}{f_u} = \frac{410}{410} = 1$

$= 1$

assume $e = 30\ mm$, $p = 50\ mm$

lesser

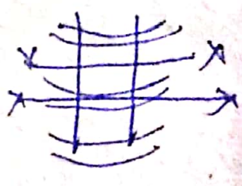
$k_b = 0.55$

$V_{dpb} = \frac{2.5 \times 0.55 \times 16 \times 16 \times (410)}{1.25}$

$= \underline{115.45\ kN}$

shear strength of the bolt:

$V_{dsb} = \frac{f_u}{\sqrt{3} \cdot \gamma_{mb}} (n_n \cdot A_{nb} + n_s \cdot A_{sb})$ (Page 75) code book



where n_n = no. of shear planes with threads intercepting the shear plane

$= 1$ (for double shear

n_s = no. of shear planes without threads intercepting shear plane = 1 (for double shear)

$\boxed{\text{for single shear } n_s = 0}$

A_{nb} = strength of the bolt (shank dia is considered avg of thread, root) $0.78 \times \frac{\pi}{4} \times d^2$

A_{sb} = area of the shank = $\frac{\pi}{4} \times d^2$

$$V_{dsb} = \frac{410}{\sqrt{3} \times 1.1} \left(1 \times 0.78 \frac{\pi}{4} d^2 + 1 \times \frac{\pi}{4} \times d^2\right)$$

$$= \frac{410}{\sqrt{3} \times 1.1} \frac{\pi}{4} d^2 (0.78 + 1)$$

$$= \frac{410}{\sqrt{3} \times 1.1} \times \frac{\pi}{4} \times 16^2 (1.78)$$

$$= 77.01 \text{ kN}$$

∴ bolt value = least of shear, bearing strength

$$= 77.01 \text{ kN}$$

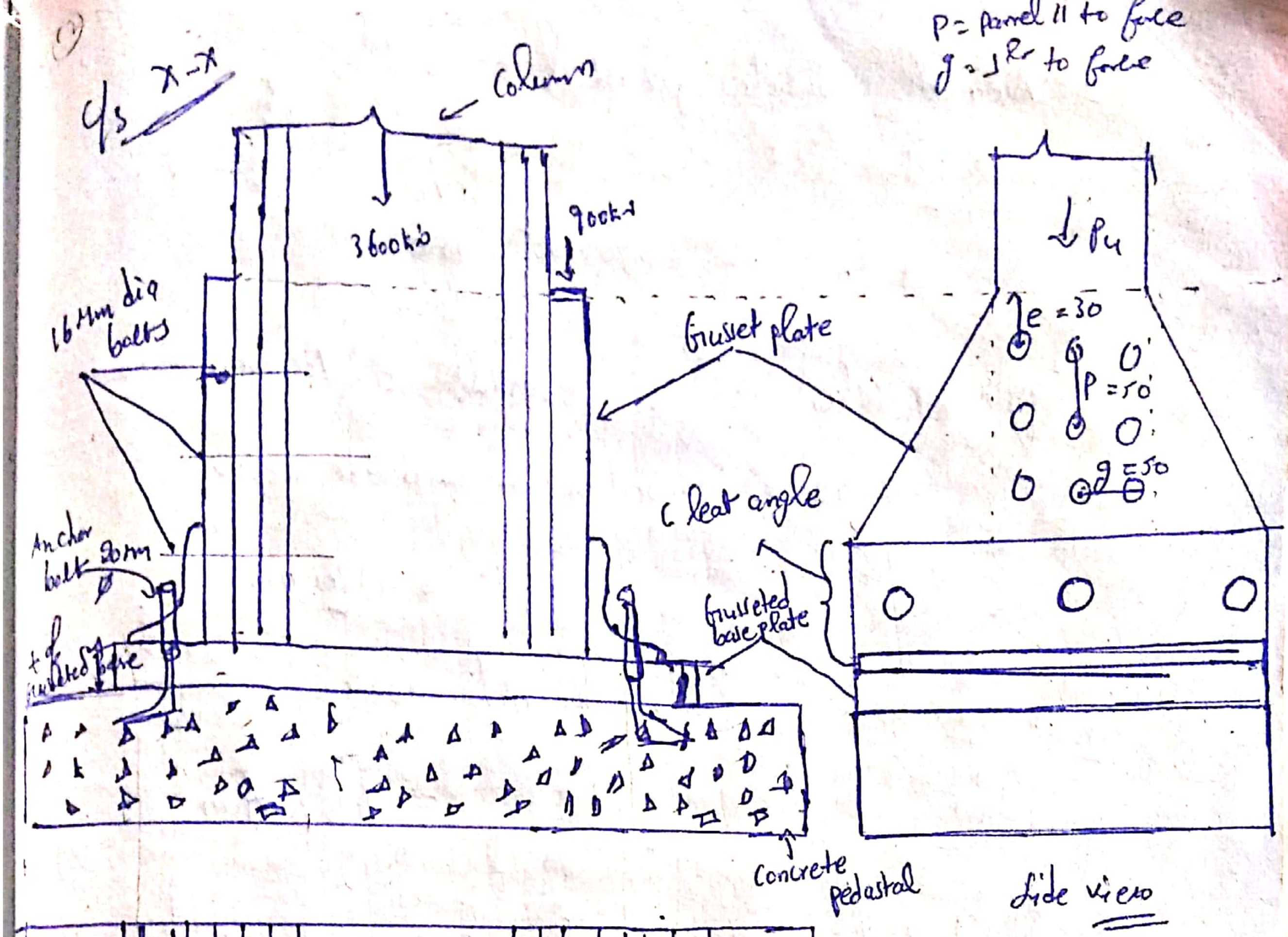
$$\text{no. of bolts required} = \frac{\text{load}}{\text{bolt value}}$$

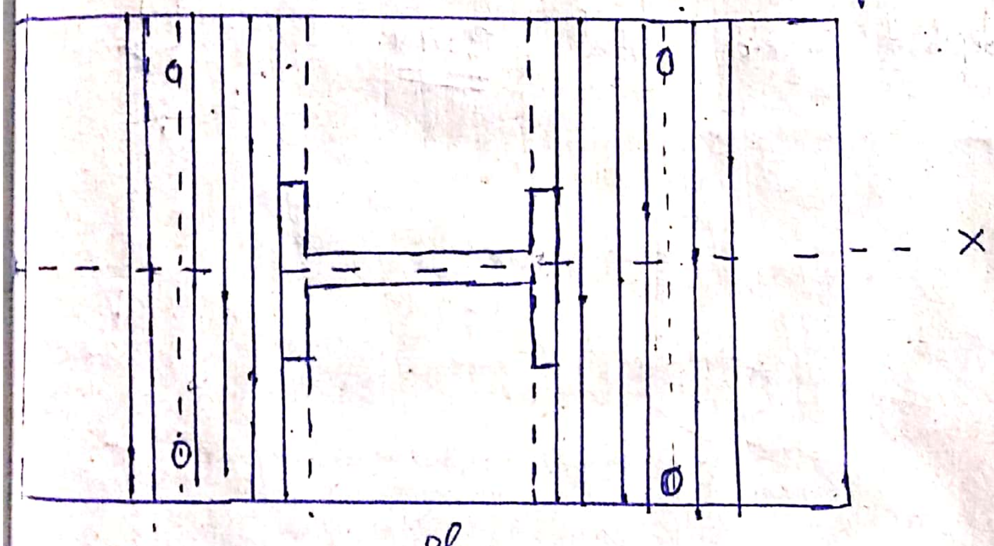
$$= \frac{900 \text{ kN}}{77.01 \text{ kN}}$$

$$= 11.6 \cong 12$$

$$= 12$$

∴ provide 12 no. of bolts





Plan

Design of Concrete pedastal

assume S.B.C of soil = 180kN/m²

∴ Area of the Concrete pedastal = $\frac{3600}{\ }$?

Assume self wt of concrete pedastal = 15% of axial load

$= \frac{15}{100} \times 3600 = 540\,kN$

(13) ∴ Area of Concrete pedastal = $\frac{\text{load}}{\text{S.B.C of soil}}$

$= \frac{540}{180}$

$= 3\,m^2$

length of the Concrete pedastal = 700 mm + 500 + 500

assume projection on each side = 500

= 1700 mm

∴ width of concrete pedastal required = $\frac{3 \times 10^6}{1700}$

= 1.764 ~~mm~~ m

depth of Concrete pedastal = 500 mm

$\tan 45° = \frac{d}{500}$

d = 500×1

= 500 mm

(7) Design of Battens :-

(*) A Built-up column consists of 2 columns ISLC 225 placed back to back at a distance of 180mm apart. The eff length of the column 5.5m, and it carries an axial compressive load of 600kN. Design suitable batten system for the column

Sol:

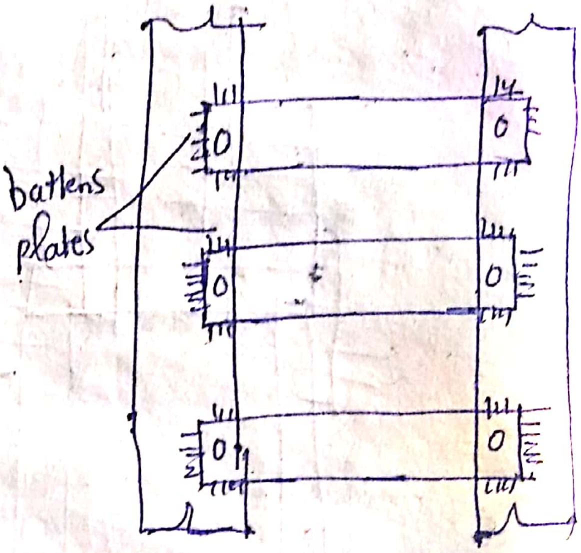
Given:

ISLC 225

$l_e = 5.5m$

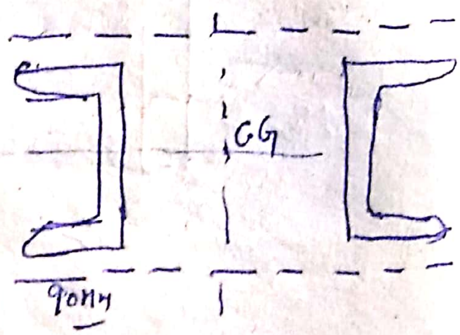
$P = 600kN$

Ms flats (mild steel)



for ISLC 225

Properties



(2) I_{xx} of Compound section $= 2 \cdot I_{xx}$

$= 50.95 \times 10^6 \ mm^4$

I_{yy} of Compound section $= [I_{yy}$ of each channel $+ Ah^2]$

$$= \left[209.5 \times 10^4 + 3053 \times (90 + C_{yy})^2\right] \times 2$$

radius of gyration for the Compound section,

$$r_{xx} = \sqrt{\frac{I_{xx}}{A}} = \sqrt{\frac{50.95 \times 10^6}{2 \times 3053}} = 91.34 \ mm$$

$$r_{yy} = \sqrt{\frac{I_{yy}}{A}} = \sqrt{\frac{84.38 \times 10^6}{2 \times 3053}} = 117.55 \ mm$$

Slenderness ratio

$$\frac{l_e}{r_{xx}} = \frac{5.5 \times 10^3}{91.34} = 60.21 \checkmark$$

$$\frac{l_e}{r_{yy}} = \frac{5.5 \times 10^3}{117.55} = 46.79$$

$\therefore$ Min radius of gyration $r_{min} = \cancel{46.79}\ 91.34 \ mm$

from Table 9(c)

$$\frac{y_2 - y_1}{x_2 - x_1} = \frac{152 - 168}{70 - 60} = -1.6$$

$$\frac{y_2 - y_1}{x_2 - x_1} = \frac{y_2 - 168}{60.21 - 60} = -1.6$$

$y_2 = 167.66 \ Mpa = f_{cd}$

Scanned by CamScanner

(3) Design strength $P_d = A_e \times f_{cd}$

$= 2 \times 3053 \times 167.66$

$= 1023\,kN > 600\,kN$

∴ hence safe.

Spacing of battens

The spacing of battens should be such that

$$\frac{C}{r_{min} \text{ of each Component}} = 50 \text{ (or) } 0.7 \times \text{slenderness ratio of Compound Section}$$

(whichever is less)

Note:-

As per Code IS 800-2007, eff. length of battened Column is to be increased by 10%.

∴ slenderness ratio of Compound Column,

$$\frac{le}{r_{min}} \times 1.1 \Rightarrow 60.21 \times 1.1 = 66.23\,mm$$

Sub slenderness ratio in above eqn

$0.7 \times 66.23 = 46.36\,mm$

$$\frac{C}{26.2} = 46.36$$

$C = 1214.6\,mm$

$\simeq 1000\,mm$

∴ provide battens 1000 mm c/c distance.

(4) End batten

overall depth of batten = eff length

$= 180 + (2 \times C_{yy})$

$= 180 + (2 \times 24.6)$

$= 229.3\ mm \cong 230\ mm$

$> 2 \times b = 2 \times 90 = 180\ mm$

hence ok

Providing overlap of batten on flange of channel 60mm.

$\therefore$ Thickness of batten $t_b = (180 + 2 \times 60) \times \frac{1}{50}$

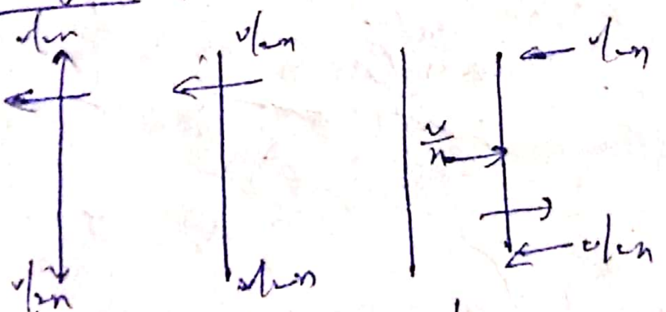
$= 6\ mm$

provide 8mm Thickness.

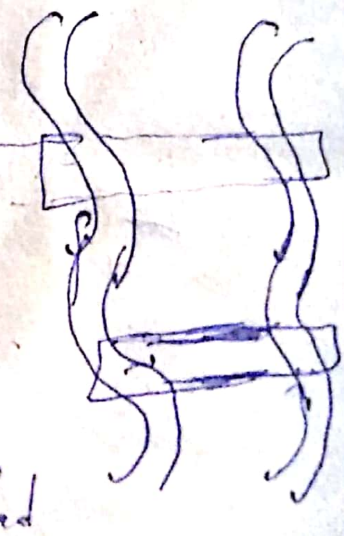
length of batten $L_b = 180 + 2 \times 60 = 300\ mm$

$\therefore$ provide batten plates of $300\ mm \times 230\ mm \times 8\ mm$.

Check for stresses in end batten:-



(loads) Free body diagram



Transverse shear $V = 2.5\%$ of column load

$= \frac{2.5}{100} \times 600 = 15\ kN$

43 → [illegible]

4 → [illegible]

(5) Longitudinal shear $V_1 = \frac{VC}{nS}$

n = no. of parallel plates
S = length of batten = 300 mm
C = c/c distance b/w batten

$V_1 = \frac{15 \times 1000}{2 \times 300} = 25 \text{ kN}$

∴ Moment $M = M_1 \mp M_2$

$= \frac{V}{2n} \times C = \frac{15 \times 10^3 \times 1000}{2 \times 2} = 3.75 \times 10^6 \text{ N-mm}$

$= 3.75 \text{ kN-m}$

longitudinal shear stress

$\tau_{VL} = \frac{\text{load}}{\text{shear area}} = \frac{V_1}{t_b \times D_b} = \frac{25 \times 10^3}{230 \times 9} = 12.\ldots$

$< \frac{f_y}{\gamma_{mo}\sqrt{3}} = \frac{250}{1.1 \times \sqrt{3}} = 131.2 \text{ Mpa}$ hence ok

Bending stress in battens

$\frac{M}{I} = \frac{f}{y} \Rightarrow f = \frac{M}{I} \times y = \frac{M}{\left(\frac{I}{y}\right)} = \frac{\cancel{t_b} D_b^2}{6} = 70.53 \times 10^3 \text{ mm}^3$

$f = \frac{M}{Z} \Rightarrow f = \frac{3.75 \times 10^6}{70.53 \times 10^3} = 53.16 \text{ Mpa}$

This bending stress should not exceed permissable bending stress.

$\left(\frac{f_y}{\gamma_{mo}}\right) \times 1.2 = \frac{250}{1.1} \times 1.2 = 272.72 \text{ mpa}$

hence safe.

Date: 27/08/17

UNIT-III

Design of Beams

1) Design of laterally supported beams
2) Design of laterally unsupported beams (eg: purlin) — 27/8/17

1) Design of laterally supported beams

Procedure :-

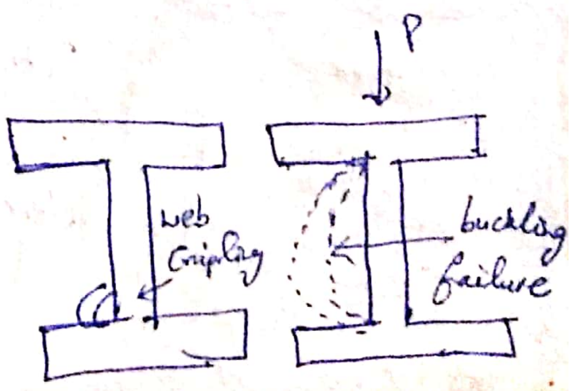
1) → Select section from the steel tables and this section is assumed as plastic section.

Note 1 :- on the basis of IS 800-2007 code clause 3.7 classifies various c/s s are as follows [page 17]

a) class 1 (plastic)
b) class 2 (compact)
c) class 3 (Semi-compact)
d) class 4 (slender)

Note 2 :- To ensure the given section is whether plastic, compact, Semi-compact and slender is found by Table no: 2 of I.S 800-2007 (page 18)

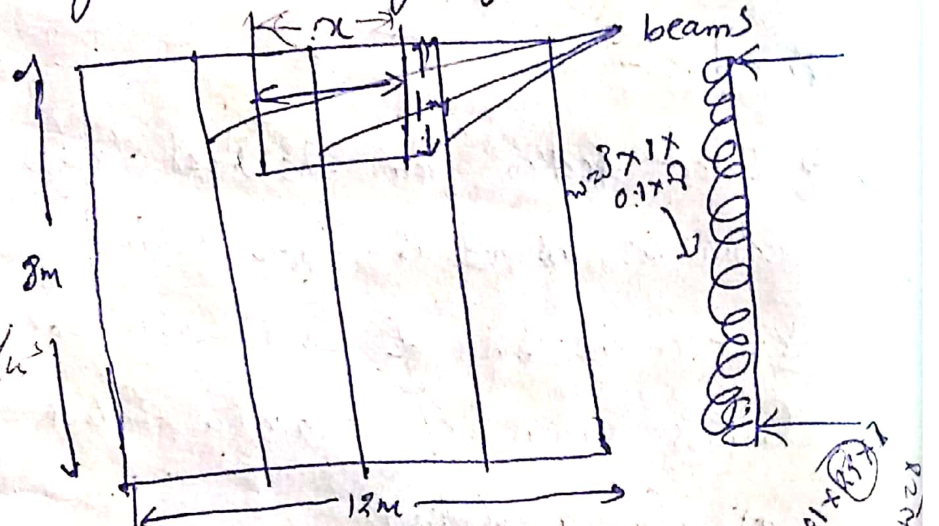
2) check for the class whether it is respective class
3) check for Bending strength
4) check for shear strength
5) check for deflection
6) check for web buckling
7) check for web crippling



if any check fail, redesign   Table 6 page 31

* A roof of a hall measuring $8m \times 12m$, consists of 100mm Thick R.C.C slab, supported on steel I beams of 3m apart as shown in fig below. The finishing load can be taken as 1.5 kN/m² and live load as 1.5 kN/m². Design the beam and also check for web buckling & web crippling if stiff bearing is over a length of 75mm

Sol.



m   m
$x \times 1 \times 1 \times 0.0 \times 25$ kN/m³

: Finding the loads:

i) weight of the R.C.C slab = 7.5 kN/m

ii) finishing load = 1.5 kN/m² = $1.5 \times 3 \times 1$ = 4.5 kN/m

iii) self wt of beam = 0.8 kN/m

live load = 1.5 kN/m² × 3 × 1 = 4.5 kN/m

Total load = 17.3 kN/m

factored load = 1.5 × total load
= 25.95 kN/m

Max B.M $= \frac{wl^2}{8}$

assume width of support wall $= 300mm = \frac{300}{2} = 150mm = 0.15m$

$\therefore$ effective length of beam = clear span of beam $+ 2(0.15)$

$= 8 + 0.3 = 8.3m$

$M = \frac{25.95 \times (8.3)^2}{8} = 223.46$ kN-m

$= 223.46 \times 10^6$ N-mm

Max shear force $= \frac{wl}{2} = \frac{25.95 \times 8.3}{2} = 107.69$ kN.

Section modulus required $Z = \frac{M}{\left(\frac{f_y}{\gamma_{m0}}\right)}$

$= \frac{223.46 \times 10^6}{\left(\frac{250}{1.1}\right)}$ of $\frac{N\text{-}mm}{N/mm^2}$

1 cm = 10 mm

$= 983.2 \times 10^3$

$= 9.83 \times 10^5$ mm^3

$= 983.2$ cm^3

choose ISMB 400 @ 61.6 kg/m

Properties

Z_{ex} = elastic section modulus $= 1022.9$ cm^3 $= 1022.9 \times 10^3 > Z_{req}$

h = depth = 400mm

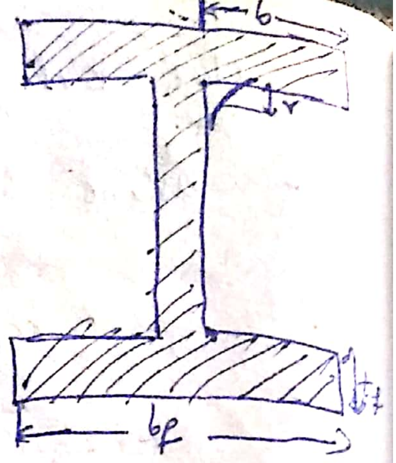
t_f = Thickness of flange = 16mm

[page 1 steel book]

t_w = Thickness of web = 8.9mm

A = Area = 78.46cm^2 = 7846 mm^2

b_f = breadth of flange = 140mm



depth of web = d = $h - 2t_f - 2r_1$

$= 400 - 2\times16 - 2\times14$

$= 340$ mm

where r_1 = root radius = 14 mm

classification of section

$\varepsilon = \sqrt{\frac{250}{f_y}}$ [from page 18]

$= \sqrt{\frac{250}{250}} = 1$

$\frac{b}{t_f} = \frac{\text{out standing width of flange (compression)}}{\text{Thickness of flange}} = \frac{\left(\frac{140}{2}\right)}{16} = 4.375 < 9.4\varepsilon$

$\frac{d}{t_w} = \frac{340}{8.9} = 38.2 < 84\varepsilon$

(Table 2)

∴ The section is classified as plastic section

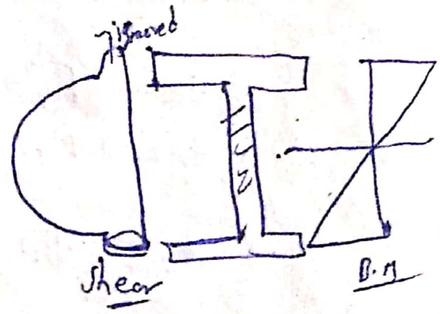
Elastic section modulus $Z_{e\,x\text{-}x} = 1022.9\ cm^3$ [from page 2 steel table]

$Z_{e\,y\text{-}y} = 88.9\ cm^3$.

Z_p = plastic section modulus $= 1176.18\ cm^3$ [from page 138 code book]

$= 1176.18\times10^3\ mm^3$

Check for shear



Design shear force = 107.69 kN

Design shear strength of the section

$$V_d = h \cdot t_w \times \frac{f_y}{\sqrt{3} \times \gamma_{mo}} = 400 \times 8.9 \times \frac{250}{\sqrt{3} \times 1.1} = 467.129\ kN$$

$V_d > 107.69\ kN$ (max S.F)

hence, o.k (safe)

<u>Check for Bending or Moment Capacity</u>

The Moment capacity $M_d = \beta_b \cdot Z_p \times \left(\frac{f_y}{\gamma_{mo}}\right)$

where $\beta_b = 1.0$ for plastic & compact sections

$= \frac{Z_e}{Z_p}$ for semi compact.

$$M_d = 1 \times 1176.18 \times 10^3 \times \left(\frac{250}{1.1}\right)$$

$$= 267.31\ kN\text{-}m$$

This bending strength should not exceed $1.2 \times Z_e \times \left(\frac{f_y}{\gamma_{mo}}\right)$ (for simply supported)

and should not exceed $1.5\ Z_e \left(\frac{f_y}{\gamma_{mo}}\right)$ (for cantilever)

$\therefore$ For S.S.B $= 1.2 \times 1022.9 \times 10^3 \times \left(\frac{250}{1.1}\right) = 278.97\ kN\text{-}m$

$\therefore$ The design bending capacity $M_d = 267.31\ kN\text{-}m$

$M_d >$ Max B.M $\left(\frac{wl^2}{8}\right)$ hence <u>safe</u>.

Therefore, The given section is adequate.

<u>Check for deflection:-</u>

The limiting deflection for a building for brittle cladding

$= \frac{le}{300}$ [From page 31 Table 6 code book]

$$= \frac{8.3 \times 10^3}{300} = 27.66\ mm$$

The actual deflection on the beam $Y_{max} = \delta_{max} = \frac{5}{384}\frac{w l^4}{EI}$

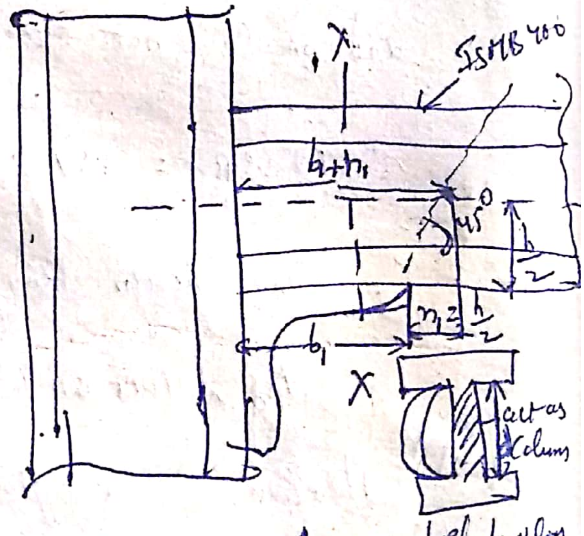
$$= \frac{5}{384} \times \frac{17.3 \times (8.3 \times 10^3)^4}{2 \times 10^5 \times 20458.4 \times 10^4}$$

$$= 26.12\,mm < \text{limiting deflection } (27.66\,mm)\ (\text{mdr})$$

$\therefore$ hence safe.

Check for web buckling (no need)

Certain portion of beam at supports acts as a column transfer the load from beam to the support assume load dispersion 45°, hence as per IS 800-2007 effective web buckling strength is to be found based on the c/s of web.



Area $= (b_1 + n_1)\,t_w$

where b_1 = width of stiff bearing on the flange

$n_1 = \frac{h}{2}$

where h = depth of the section

$\therefore$ web buckling strength

$F_{cdw} = (b_1 + n_1)\,t_w \times f_c$
(Area × stress)

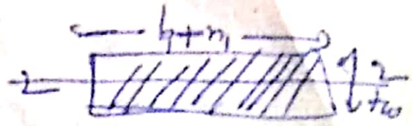
where f_c is the allowable compressive stress corresponding to the assumed web column

Effective length = 0.7 times the total thickness

$l_e = 0.7 \times d = 0.7 \times 340 = \underline{238\,mm}$

r_y = least radius of gyration about minor axis of assumed column

$$= \sqrt{\frac{I}{A}} = \sqrt{\frac{\frac{bd^3}{12}}{A}}$$



$$= \sqrt{\frac{\frac{(b_1+n_1)(t_w)^3}{12}}{t_w(b_1+n_1)}} = \frac{t_w}{2\sqrt{3}}$$

$$= 2.56 \text{ mm}$$

Slenderness ratio $\lambda = \frac{l_e}{r_{yy}} = \frac{238}{2.56} = 92.96$

From Table No: 9c (page 42)

$\frac{l_{ef}}{r_{yy}}$	$f_y = 250$
90	121 ~~0.573~~
92.96	$f_{cd} = y =$
100	107

slope $= \frac{107-121}{100-90} = -1.4$

$\frac{y-121}{92.96-90} = -1.4$

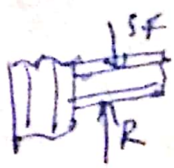
$f_{cd} = y = 116.85 \text{ N/mm}^2$

$\therefore$ design buckling strength $F_{cdw} = (b_1+n_1)\, t_w \times f_{cd}$

assume bearing width $b_1 = 150$mm $\left(\frac{\text{wall width}}{2}\right)$

$$F_{cdw} = \left(150 + \frac{400}{2}\right) 8.9 \times 116.85$$

$$= 363.98 \text{ kN} > \text{Max S.F}$$



hence safe

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Design of laterally unsupported beams :-

(*) An ISHB 500 Section is used as a beam over a span of 6m, with simply supported ends. (8-11m) Determine the Max Factored U.D.L that the beam can carry. if the ends are restrained against torsion but compression flange is laterally unsupported.

Sol:- Given Section ISHB 500

Properties:

overall depth $h = 500\,mm$

[page ①]

width of flange $b_f = 180\,mm$

Thickness of flange $t_f = 17.2\,mm$

Thickness of web $t_w = 10.2\,mm$

least radius of gyration $r_{yy} = 3.52\,cm = 35.2\,mm$

Effective length for Torsional buckling $l_e = 6m$

To find F_{bd} : (reduced bending strength)

[From page 45 code book K = 1]

Find F_{crb}

$$\text{slenderness ratio} = \frac{K.L}{r_{min}} = \frac{1 \times 6000}{r_{yy}} = \frac{6000}{35.2} = 170.45$$

depth to Thickness of flange ratio

[From page 57 code book]

$$\frac{h}{t_f} = \frac{500}{17.2} = 29.06\,mm$$

$\frac{k \cdot L}{r}$	$h/t_f = 29.06$		
$\lambda = \frac{kL}{r}$	25	29.06	30
170	136.7	X = 124.19	121.3
170.45		M = ?	
180	127.1	Y = 115.001	112.2

Finding x :-

$$x = 136.7 + \frac{121.3 - 136.7}{30 - 25}(29.06 - 25)$$

$$= 124.19$$

Finding y :-

$$y = 127.1 + \frac{112.2 - 127.1}{30 - 25}(29.06 - 25)$$

$$= 115.001$$

To find M

	$h/t_f = 29.06$
170	124.19
170.45	M = ?
180	115.001

$$M = 124.19 + \frac{115.001 - 124.19}{180 - 170}(170.45 - 170)$$

$$M = 123.77$$

$$\therefore \boxed{f_{cb} = 123.77\,N/mm^2}$$

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From Table 13(a) Page 55 Code book.

f_{crb}	$f_y = 250$
150	106.8
123.78	$y = ?$
100	77.3

$$y = 106.8 + \frac{77.3 - 106.9}{100 - 150}(123.78 - 150)$$

$y = 91.33$

$$\boxed{f_{bd} = 91.33 \ N/mm^2}$$

Finding β_b :-

It is a factor, it has 1.0 for plastic & Compact sections and $\beta_b = \frac{Ze}{Zp}$ for semi compact sections.

classification of section :-

It's depends upon Table no: 2 of IS 800 (Page 18)

class	$b = \frac{bf}{2} = \frac{180}{2} = 90$, $\frac{b}{t_f} = \frac{90}{17.2} = 5.23$	$d = h - 2 \cdot t_f =$, $\frac{d}{t_w} = 45.64$
plastic $\beta_b = 1$	$< 9.4\varepsilon$	$< 84\varepsilon$
Compact. $\beta_b = 1$	$< 10.5\varepsilon$	$< 105\varepsilon$
semi-Compact $\beta_b = \frac{Ze}{Zp}$	$< 15.7\varepsilon$	$< 105\varepsilon$

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

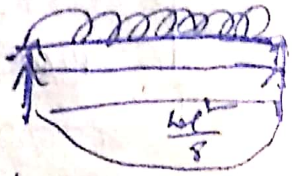
$\therefore$ From Table, section is plastic $\beta_b = 1$

Moment Capacity $M_d = \beta_b \cdot Z_p \cdot f_{bd}$ (Page 138 Z_p)

$$\Rightarrow \quad = (1.0) \times (2074.7 \times 10^3) \times 91.33$$

$$= 189.48 \times 10^6 \text{ N-mm}$$

$$= 189.48 \text{ kN-m}$$



For the equilibrium condition

Max B.M $\frac{wl^2}{8}$ should be equal to design Moment.

$$\therefore \frac{wl^2}{8} = M_d = 189.48 \times 10^6$$

$$\Rightarrow \quad w = \frac{189.48 \times 8}{l^2} = \frac{189.48 \times 8}{6^2}$$

$$w = 42.10 \text{ kN/m}$$

∴ The max U.D.L carried by the member is 42.10 kN/m

This includes the self wt of the beam also.

∴ to get the net U.D.L, deduct the self wt.

Net U.D.L $= 42.10 - 0.852$

$= 41.24$ kN/m.

86.9 kg/m
$86.9 \times 9.81 =$ 852.48 N/m
$= 0.852$ kN/m

$$\therefore \text{Working load} = \frac{41.24}{F.S.} = \frac{41.24}{1.5}$$

$$= \underline{27.5 \text{ kN/m}}$$

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Design of Purlin

Design Procedure:-

Step:1 Resolve the factored forces $\parallel^{lar}$ to and $\perp^{lar}$ to sheet.

Step:2 : Determine moments and shear forces about 2-2 & Y-Y axis

Step:3 To account for biaxial bending, The required value of section modulus about 2-2 axis may be taken as

$$Z_{P_z} = \frac{M_z}{f_y}\gamma_{mo} + 2.5\frac{d}{b}\frac{M_y}{f_y}(\gamma_{mo})$$

where γ_{mo} = P.S.F = 1.1

d is the depth of trial section (choosen sec)

b is the breadth or width of trial section

The second term in the expression above makes an approximate relation b/w Z_{P_z} and Z_{P_y}

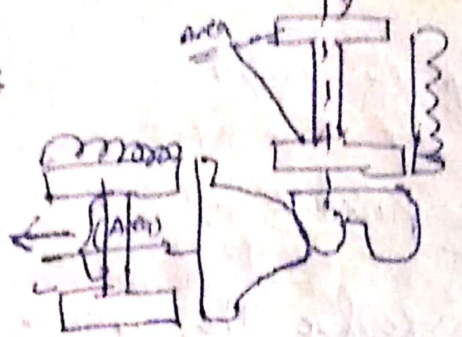
Since d, & b values will be known, only after section is selected, first trial section is selected and Z_P required is found. if it is sufficient, proceed for that otherwise try another section

Step: 4 Check the Section for shear Capacity, for which the following expressions can be used.

i.e $$V_{dz} = \frac{f_y}{\sqrt{3}} \times \frac{1}{\gamma_{mo}} \times A_{vz}$$

$$V_{dy} = \frac{f_y}{\sqrt{3}} \times \frac{1}{\gamma_{mo}} \times A_{vy}$$

where $A_{vy} = 2 b_f t_f$



$A_{vz} = h \cdot t_w$

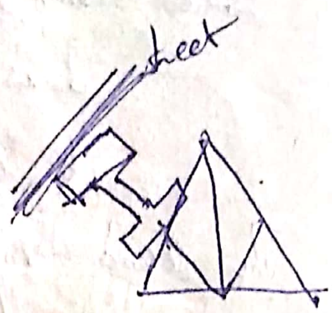
<u>Step:5</u> Conclude the design capacity of the section in both axis

$$M_{dz} = Z_{pz}\left(\frac{f_y}{\gamma_{m0}}\right) \le 1.2\, Z_{ez}\left(\frac{f_y}{\gamma_{m0}}\right)$$

$$M_{dy} = Z_{py}\left(\frac{f_y}{\gamma_{m0}}\right) \le 1.5\, Z_{ey}\left(\frac{f_y}{\gamma_{m0}}\right)$$

<u>Step:6</u> The design should satisfy the interaction formula.

$$\frac{M_z}{M_{dz}} + \frac{M_y}{M_{dy}} \le 1.0$$



<u>Step:7</u> check for deflection

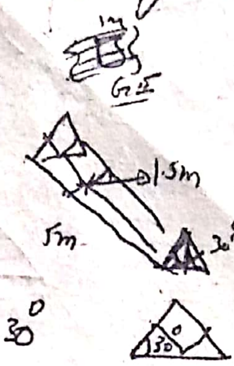
<u>Step:8</u>:- check for wind suction

when dead load & live loads are predominant, Top flange of purlin is under compression, but this flange is laterally restrained by sheeting, hence for those loads, analysis as laterally restrained compression flange was adequate

2) When <u>wind suction</u> acts, bottom flange will be in compression, which is not restrained laterally obviously, in this case, wind suction along with dead load only should be considered as critical load

Considering Torsional buckling M_{dz} and M_{dy} should be found & the interaction formula should be checked like previously.

⊛ Design an 'I' section purlin for an Industrial building to Support a Galvanised curvegated Iron sheets.



Given: Spacing of the trusses - 5m.

Spacing of the purlins - 1.5m

Inclination of main rafter to horizontal = 30^0

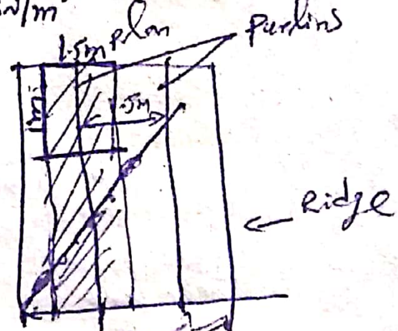
Weight of Galvanised sheets taking into accounts, laps & connecting bolts = $130 N/m^2$

Imposed snow load = $1.5 kN/m^2$

Wind load = $1 kN/m^2$ (suction upward blow)

Sol:

Dead load = $130 N/m^2$ = $0.13 kN/m^2$ ~~x1m~~ =

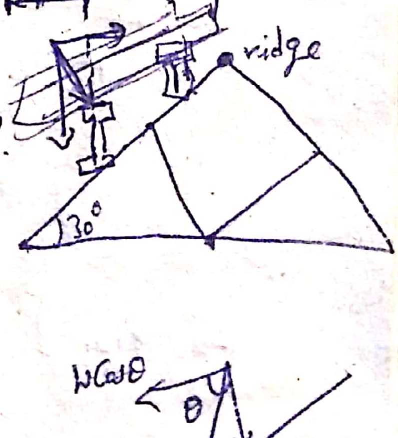


Snow load = $1.5 kN/m^2$

∴ Dead load for 1m length of purlin = $0.13 kN/m^2 \times 1.5m$ = $0.195 kN/m$

Snow load/meter length = $1.5 kN/m^2 \times 1m \times 1.5m$

= $2.25 kN/m$



∴ Total load = $2.445 kN/m$

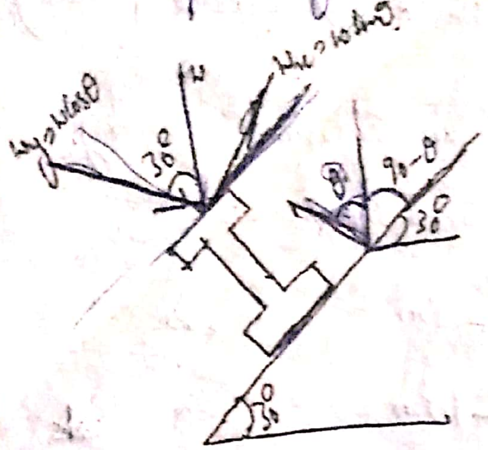
Factored load = 2.445×1.5

= $3.6675 kN/m$

Component of load Normal to sheet $W_y = W \sin\theta$

$= 3.667 \sin 30^\circ$

$= 1.83$ kN/m



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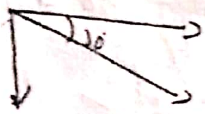
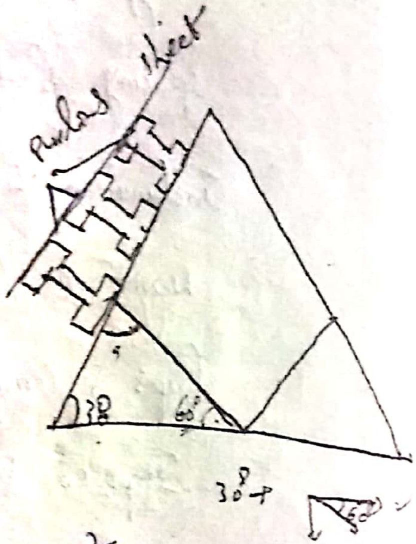
$W_z = 3.667 \cos(30^\circ)$

$= 3.17$ kN/m





Moments

Moment about z-direction $M_z = \frac{W_z l^2}{8}$

$= \frac{3.17 \times 5^2}{8} = 9.94$ kN-m

Moment about y-direction $M_y = \frac{W_y l^2}{8} = 5.73$ kN-m

Shear force

$F_z = \frac{W_z l}{2} = 7.92$ kN

$F_y = \frac{W_y l}{2} = 4.57$ kN

Z_p required

$$Z_{pz}\ req = \frac{M_z}{f_y} \times \gamma_{mo} + 2.5 \frac{d}{b} \frac{M_y}{f_y} (\gamma_{mo})$$

Trail any section find Z_{pz}, From Table that section have same Z_{pz} (page 135)

∴ Z_{pz} calcu < Z_{pz} (Table)

Z_{pz} (req) = assume (or) select ISMB 500 @ 86.9 kg/m

$d = h - 2t_f = 500 - 2(17.2) = 465.6\,mm$

$b = \frac{b_f}{2} = 90\ 180$

$$Z_{pz}(req) = \frac{9.94\times10^6}{250}\times(1.1) + 2.5\times\frac{465.6}{180}\times\frac{5.73\times10^6}{250}(1.1)$$

$= \cancel{364\times10^3\,mm}\ 206.77\times10^3$

$= \cancel{369.7}\ 206.77\,cm^3$

322.47×10^3

From page 138 $Z_{pz} = 2074\,cm^3$

$\therefore Z_{pz}$ req < 2074

$\therefore$ hence ISMB 500 adequate.

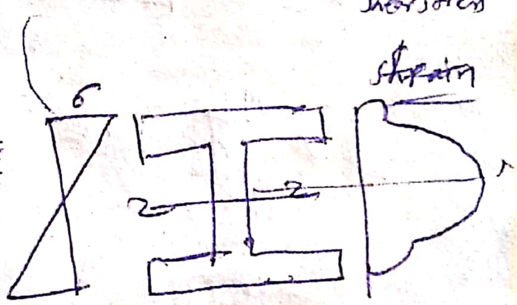
check for shear strength

shear stress

~~strain~~

1. shear capasity in z direction

$\tau = \frac{F}{A}$



$$V_{dz} = \tau\times A$$

$$= \frac{h\times t_w}{\sqrt3}\times\frac{f_y}{\gamma_{m0}}$$

$$= \frac{500\times10.2}{\sqrt3}\times\frac{250}{1.1} = 669.201\,kN > 7.9\,kN\,(F_z)$$

$\therefore$ hence o.k.

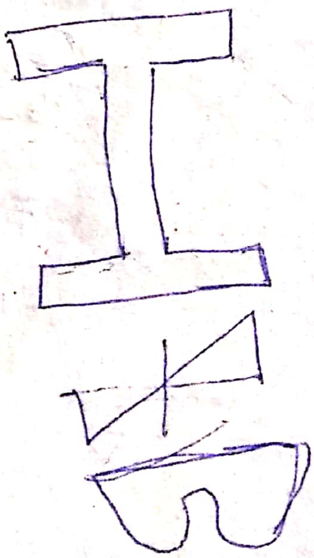
2. shear capasity in y direction

$$V_{dy} = (2\times t_f\times b_f)\frac{f_y}{\sqrt3\,\gamma_{m0}}$$

$$= 2\times17.2\times180\times\frac{250}{\sqrt3\times1.1}$$

$$= 812.48\,kN > (419\ F_y)$$

hence o.k



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Moment Capacity in z direction

$$M_{dz} = \beta_b \cdot Z_{pz}\left(\frac{f_y}{\gamma_{m0}}\right) \leq 1.2\left(\frac{f_y}{\gamma_{m0}}\right) Z_{ey}$$

$$= 1 \times 2074.67 \times 10^3 \times \left(\frac{250}{1.1}\right) \leq 1.2\left(\frac{250}{1.1}\right)(152.2)10^3$$

$$= 471.51 \text{ kN-m} \leq 41.50 \text{ kN-m}$$

$$= 471.51 > M_z \ (9.9 \text{ kN-m})$$

hence o.k.

Moment Capacity in y direction

$$M_{dy} = \beta_b \cdot Z_{py}\left(\frac{f_y}{\gamma_{m0}}\right) \leq 1.5\left(\frac{f_y}{\gamma_{m0}}\right) Z_{ey}$$

redu

$$= 1.5 \times \frac{250}{1.1}(152.2 \times 10^3)$$

$$= 51.77 \text{ kN-m} > M_y \ (5.7 \text{ kN-m})$$

hence o.k

Check for Interaction formulae

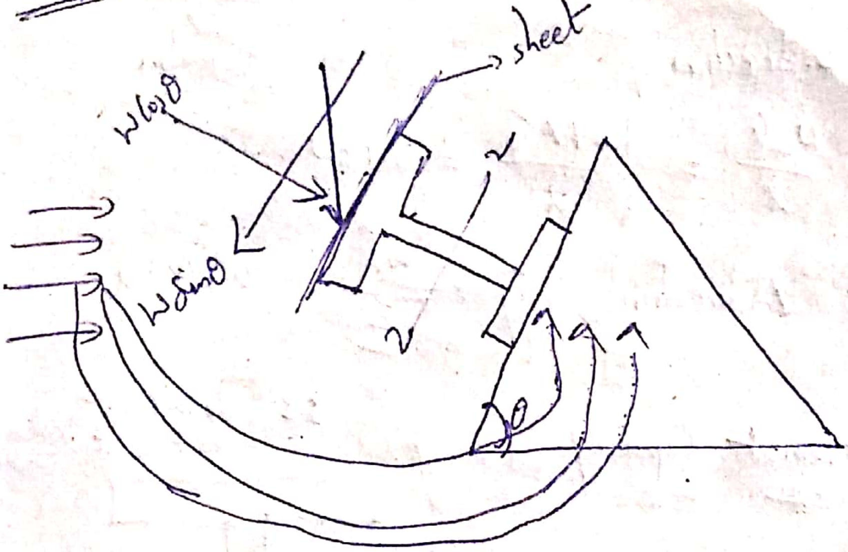
$$\frac{M_z}{M_{dz}} + \frac{M_y}{M_{dy}} \leq 1$$

$$\Rightarrow \frac{9.9}{471.51} + \frac{5.7}{51.88} \leq 1$$

$$\Rightarrow 0.13 \leq 1.$$

page 70

Step: 7 Check for Wind Condition



dead load = 0.195 kN/m

Factored load = 1.5 × 0.195 Cos 30° = 0.2533 kN/m
(Normal to sheet)

Wind load Normal to the sheet = 1 kN/m² × 1.5 m × 1
= 1.5 kN/m (Suction upward)

∴ Net Load (Normal to sheet) = 1.5 × 0.195 Cos (30°) − 1.5

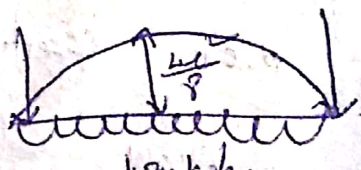
= −1.24 kN/m

w = 1.24 kN/m (upward)

Load parallel to the sheet = 0.195 × 1.5 × sin (30°)

= 0.146 kN/m

Finding the Moment :-



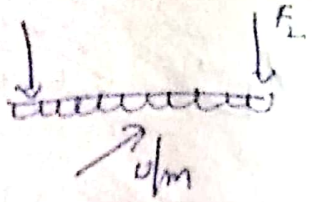
Moment about z direction :-

$$M_z = \frac{W_z l^2}{8} = \frac{1.24 \times 5^2}{8} = 3.875 \text{ kN-m}$$

M about Y-direction :-

$$M_y = \frac{W_y l^2}{8} = \frac{0.146 \times 5^2}{8} = 0.456 \text{ kN-m}$$

Max S.F or reactions



S.F about z-direction

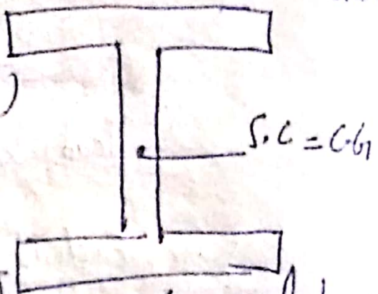
$$F_z = \frac{w_z l}{2} = \frac{1.24 \times 5}{2} = 3.1\ kN$$

S.F about y-direction $F_y = \frac{w_y l}{2} = 0.365\ kN$

S.C = shear center
= Point about section rotates

Check for Moment Capacity

$M_{dz} = \beta_b \cdot Z_{p_z} \cdot f_{bd}$ (un supported)

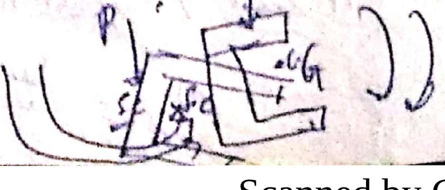


From Table 14

Symmetrical k = 1
Un symmetrical k = 1.2

$\lambda = \frac{k L}{r_{min}} = \frac{1 \times 5000}{35.2} = 142.045$	$\frac{h}{t_f} = \frac{500}{17.2} = 29.069$		
	25	29.069	30
140	177.5	x	160.2
142.04		M	
150	161.5	y	144.8

Note:- effective length of unsymmetrical sections like channel purlin, angle purlin & T-section purlin is taken as 1.2 x actual length because for these sections shear center shear center should not coinside with the C.G Therefore increase slenderness ratio, it should be taken as 1.2 x L



By Interpolation formulae

$$X = 177.5 + \frac{160.2 - 177.5}{30-25}(29.06-25) = 163.45$$

$$Y = 161.5 + \frac{144.9 - 161.5}{30-25}(29.06-25) = 147.93$$

To find $f_{crb} = M$

$$f_{crb} = 163.45 + \frac{147.93-163.45}{150-140}(142.04-140)$$

λ	$\frac{h}{t_f} = 29.06$
140	163.45
142.04	f_{crb}
150	147.93

$= 160.28$ Mpa

Critical stress in bending $f_{crb} = 160.28$ Mpa

To find f_{bd}:-

$$f_{bd} = 134.1 + \frac{106.8-134.1}{150-200}(160.28-200)$$

f_{crb}	$f_y = 250$
200	134.1
160.28	f_{bd}
150	106.8

$= 112.41$ Mpa

$M_{dz} = \beta_b \cdot Z_p \times f_{bd}$

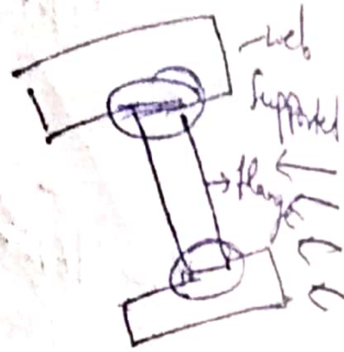
$= (1.0) \times 2074.67 \times 10^3 \times 112.41$ (from page 138 Z_p)

$= 233.21$ knm $> M_z$ (3.87 knm)

hence o.k. $\therefore$ Section is adequate

Moment Capasity about Y-direction (M_{dy}) :-

$$M_{dy} = \beta_b . Z_{py} \times \left(\frac{f_y}{\gamma_{mo}}\right)$$



$\therefore$ W.K.T $\quad S = \dfrac{Z_{py}}{Z_{ey}} = \dfrac{Z_{pz}}{Z_{ez}}$

$$\Rightarrow Z_{py} = S \times Z_{ey}$$

$$= 1.471 \times 152.2 \times 10^3 \quad \text{(Page 11}$$

$$= 174.58 \times 10^3$$

$$\therefore M_{dy} = (1.0) \times 174.58 \times 10^3 \times \left(\frac{250}{1.1}\right)$$

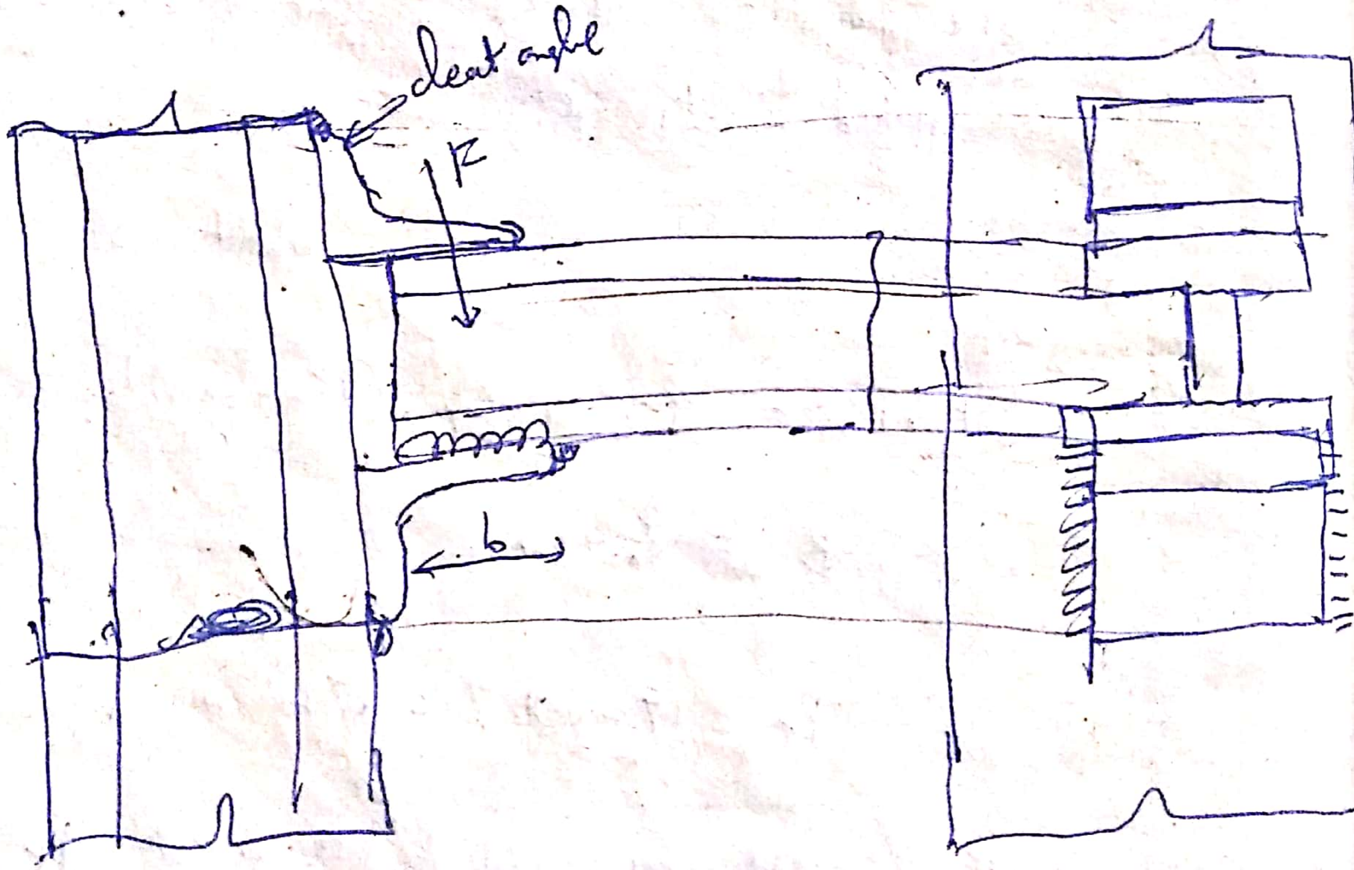
$$= 39.67 \text{ kN-m} > 0.456 (M_y)$$

Hence ok.

Unstiffened Seated Connection :-

1. An ISMB 400 transverse an end reaction of 160 kN to the flange of an ISHB 300 @ 577 N/m. Design an unstiffened welded seat connection. Take permissable bearing stress $= 0.75 \times f_y = 0.75 \times 250 = 187.5 \ N/mm^2$

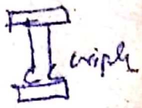
Sol:



bearing length $b = B - \sqrt{3}\, h_2$

$$= \frac{F}{f_b \cdot t_w} - \sqrt{3}\,(t_f + r_1)$$

where B = bearing length to avoid crippling failure



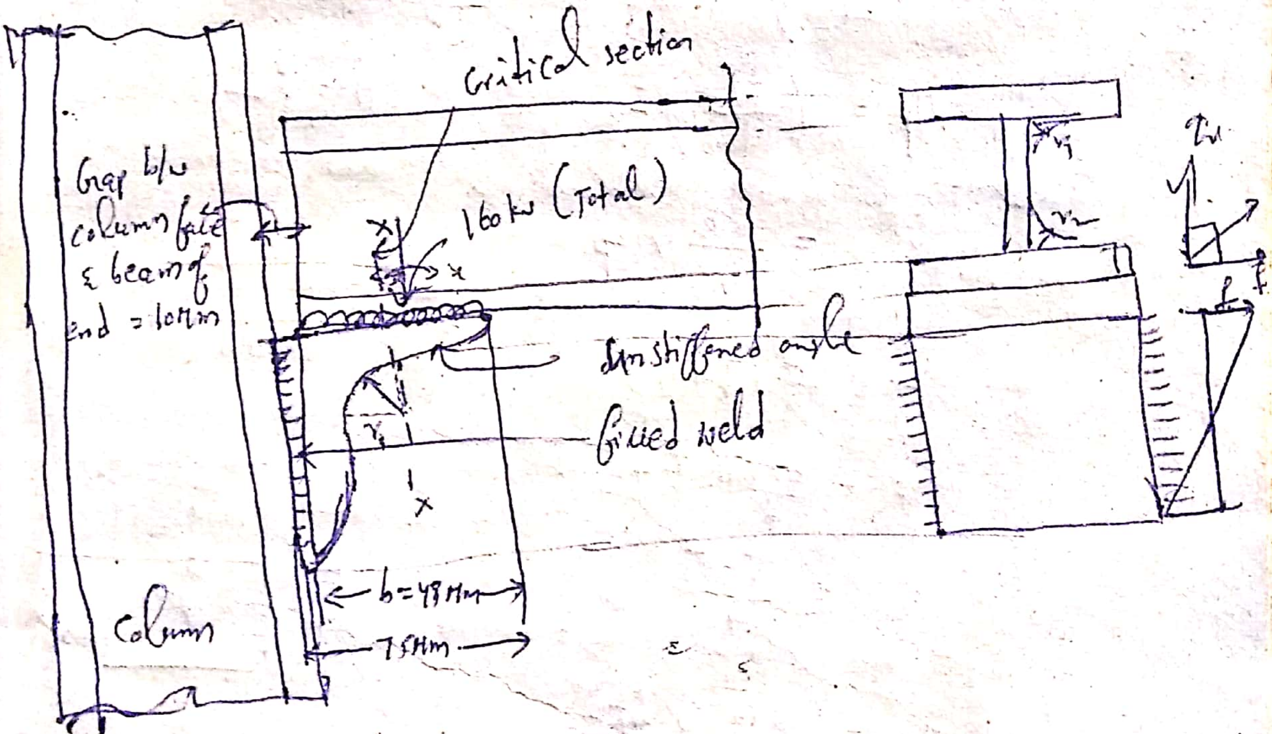
$$= \frac{160 \times 10^3}{0.75 \times 250 \times 8.9} - \sqrt{3}\,(16 + 14)$$

$= 43.97$ mm

but $b \nless \frac{B}{2} \Rightarrow b \nless \frac{F}{2 f_b \cdot t_w} = \frac{160 \times 10^3}{0.75 \times 250 \times 8.9 \times 2}$

$b \nless 47.94$

$b \simeq 48$ mm



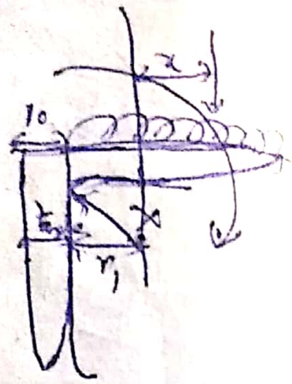
Size of the unstiffened angle (Page 2)

width of the angle = breadth of flange of beam = 140mm

t = Thickness of the angle = 12mm (assume)

∴ provide unstiffened angle ISA 150×75×12

Moment at critical section ~~at~~ X-X



$M = F \times x$

$= \cancel{F\left(10+\frac{48}{2}\right)}$

$= F\left(10 + \frac{48}{2} - (t_{angle} + r_1)\right)$

$= 160 \times 10^3 \left(10 + 24 - (12 + 10)\right)$

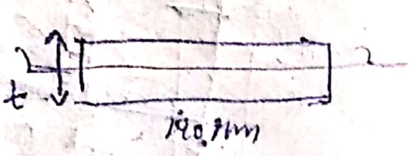
$= 1.92 \times 10^6$ N-mm $= 1.92$ kN-m (due to external force F)

Let 't' be the thickness of the angle

Moment capacity of the thickness of the angle

$M_p = \left(\frac{f_y}{\gamma_{m0}}\right) Z_p$ $\boxed{M = f Z}$

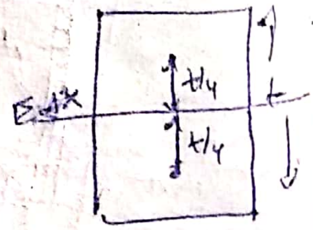
$= \frac{250}{1.1} \times Z_p$



$Z_p = \frac{A}{2}(\bar{y}_1 + \bar{y}_2)$

$= \frac{b \times t}{2}\left(\frac{t}{4} + \frac{t}{4}\right)$

$= \frac{bt^2}{4}$



$\therefore M_p = \frac{250}{1.1} \times \frac{bt^2}{4}$

To be in the state of equilibrium moment capacity equal to Max B.M at X-X

$\therefore M_p = \frac{250}{1.1} \times \frac{bt^2}{4} = 1.92 \times 10^6$

$\Rightarrow \frac{250}{1.1} \times \frac{140 \times t^2}{4} = 1.92 \times 10^6$

$\Rightarrow \quad t = 15\,mm \cong 16\,mm$ (provide)

$\therefore$ provide ISA $150 \times 75 \times 16\,mm$, which is not available in steel table

$\therefore$ provide welded plated angle manually such a way that root radius $= 10\,mm$.

Design of weld

Bending stress in weld due to moment (M). $f = \frac{M}{I} \times y$

$$\Rightarrow f = \frac{M}{I_{zz}} \times y$$

$$= \frac{M}{2\left(\frac{t_e (150)^3}{12}\right)} \times \frac{150}{2}$$

$$= \frac{6M}{2t_e (150)^2}$$

$$= \frac{6 \times 1.92 \times 10^6}{2 \times t_e \times (150)^2}$$

$$= \frac{256}{t_e}\ N/mm^2$$

$$\text{direct stress} = \frac{160 \times 10^3}{2 \times t_e \times 150} = \frac{533.33}{t_e}\ N/mm^2$$

$$\text{Resultant} = \sqrt{f^2 + \tau_v^2 + 2f\tau_v \cos 90^\circ}$$

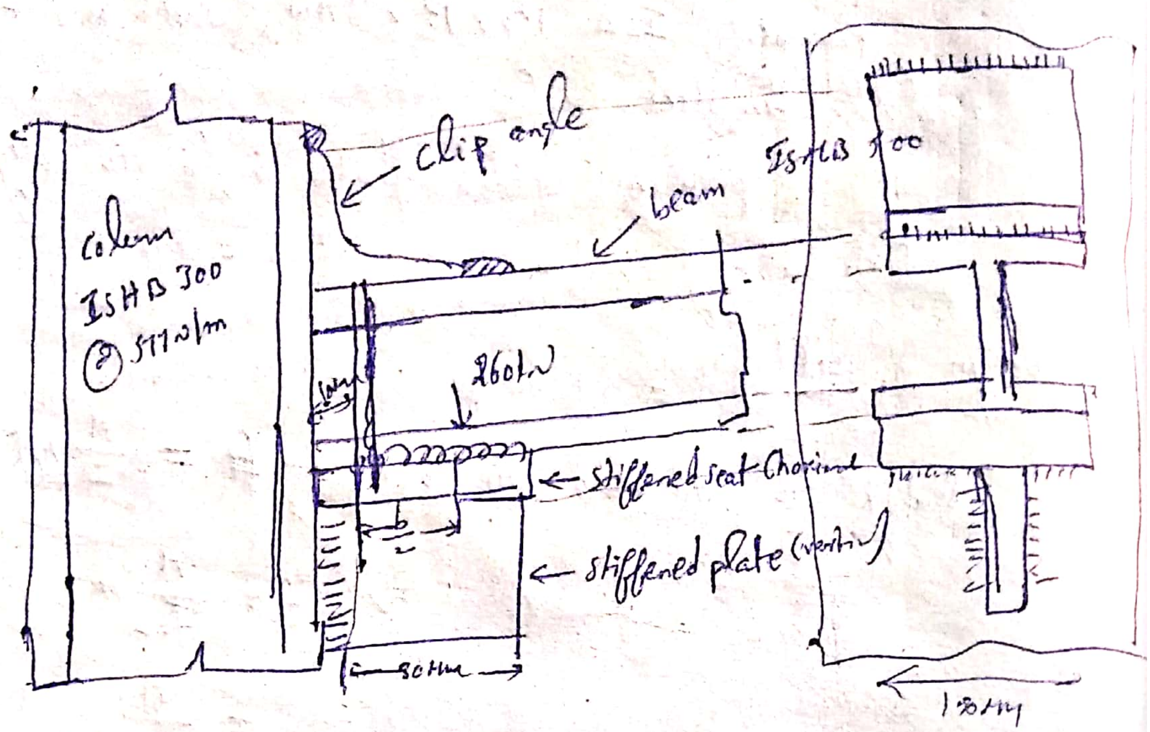
$$= \sqrt{f^2 + \tau_v^2}$$

$$= \sqrt{\left(\frac{256}{t}\right)^2 + \left(\frac{533.3}{t}\right)^2} = 189\ N/mm^2$$

$\Rightarrow \quad t_e = 3.12 \quad \Rightarrow \quad 0.7 \times S = 3.12$

$\Rightarrow \boxed{S = 4.4 \cong 6\,mm}$

Stiffened Seated Connection



(*) Design a stiffened seat connection, to connect ISMB 500, transfering a load of 260 kN to an ISHB 300 @ 577 N/m

Sol:

For ISMB 500

$b_f = 180$ mm

$t_f = 17.2$

$t_w = 10.2$, $r_1 = 17$ mm

bearing length $b = \frac{F}{f_y \cdot t_w} * \sqrt{3}\,(t_f + r_1)$

$= \frac{260 \times 10^3}{250 \times 0.75 \times 10.2} - \sqrt{3}\,(17.2 + 17)$

$= 76.71$ mm

but $b \nless \frac{B}{2}$ $\Rightarrow$ $b \nless \frac{F}{2 f_y \cdot t_w} = \frac{260 \times 10^3}{0.75 \times 250 \times 10.2}$

$b \nless 67.97$ mm

$\therefore$ provide bearing length $b = 76.71 \cong 80$ mm

$\therefore$ provide 80mm seat plate

Thickness of the seating plate = Thickness of flange of beam

$= 17.2$ mm $\cong 18$ mm

$\boxed{\text{Thickness seating plate} \nless \text{Thickness of flange of beam}}$

Width of seating plate = b_f of beam

$= 180$ mm

Stiffened plate:

Width of the stiffening plate $= b_f = 180$ mm

Thickness of the stiffening plate $\nless$ Thickness of web of beam

$= t_w = 10.2$ mm

$\therefore$ provide 12 mm

Length of the stiffening plate = bearing length + 10 mm

$= 80 + 10 = 90$ mm

Conclusion:-

finally provide $90 \times 150 \times 18$ mm seat plate and

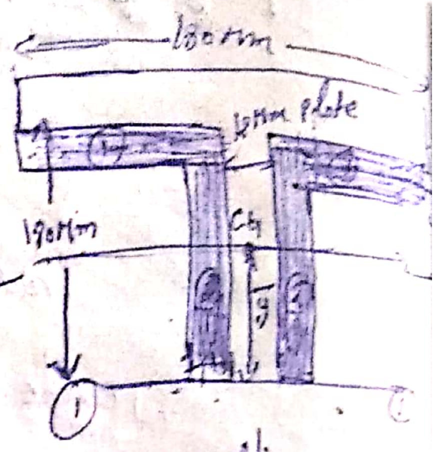
$90 \times 150 \times 12$ mm stiffened plate

Moment $M = F \times r$

$$= F \times \left(\frac{\frac{F}{f_b t_w}}{2} \right)$$

$$= \frac{260 \times 10^3}{2} \left(\frac{260 \times 10^3}{0.7 \times 250 \times 10.2} \right)$$

$$= 18.93 \times 10^6 \text{ N-mm}$$



$(180 - t_e)(t_e)$

M.O.I of weld :-

Let $\bar{y}$ be the distance of C.G of the weld group from ①-① reference

$$\bar{y} = \frac{a_1 y_1 + a_2 y_2}{a_1 + a_2}$$

$$= \frac{(190-12) \times t_e \times \left(180 - \frac{t_e}{2}\right) + 2(180 - t_e)\, t_e \times \left(\frac{180 - t_e}{2}\right)}{(180-12)\, t_e + 2(180 - t_e)\, t_e}$$

$$= \frac{[(180)^2 + t_e^2 - 2(t_e)(180)] \times t_e}{168 t_e + 360 t_e - 2t_e^2}$$

$$= \frac{(180)^2 t_e + t_e^3 - 2t_e^2\, 180}{168 t_e + 360 t_e - 2t_e^2}$$

$$= \frac{(180)^2 t_e}{528 t_e}$$

$$= 61.36 \text{ mm}$$

from the top $= 180 - 61.36 = 118.63$ mm

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I_{zz} of weld $= I_{zz_1} + I_{zz_2}$

$$= \frac{(180-12)\,t_e^3}{12} + (180-12)\,t_e\,(61.36)^2 + \frac{t_e\,(180-t_e)^3}{12} + t_e\,(180-t_e) \times \left(\frac{180}{2} - 61.36\right)^2$$

$2(180-t_e) \times t_e \times (61.36)^2$

$$= 180 \times 10^3\, t_e\ \text{mm}^4$$

From BM eqn

$$f = \frac{M}{I} \times y$$

$$= \frac{18.78 \times 10^6}{180.7 \times 10^3 \times t_e} \times (118.63)$$

$$= \frac{2876.47}{t_e}\ \text{N/mm}^2$$

<u>Direct stress</u>

$$q_v = \frac{260 \times 10^3}{(180-12)\,t_e + (180-t_e)(t_e)\,2}$$

$$= \frac{492.42}{t_e}$$

Permissible stress $= \dfrac{f_u}{\sqrt{3}\,\gamma_{mw}} = \dfrac{410}{\sqrt{3} \times 1.25} = 189$

$$q_R = \sqrt{\left(\frac{492.42}{t_e}\right)^2 + \left(\frac{2876.47}{t_e}\right)^2} = (187)$$

$$\Rightarrow \quad t_e = 15.44\ \text{mm}$$

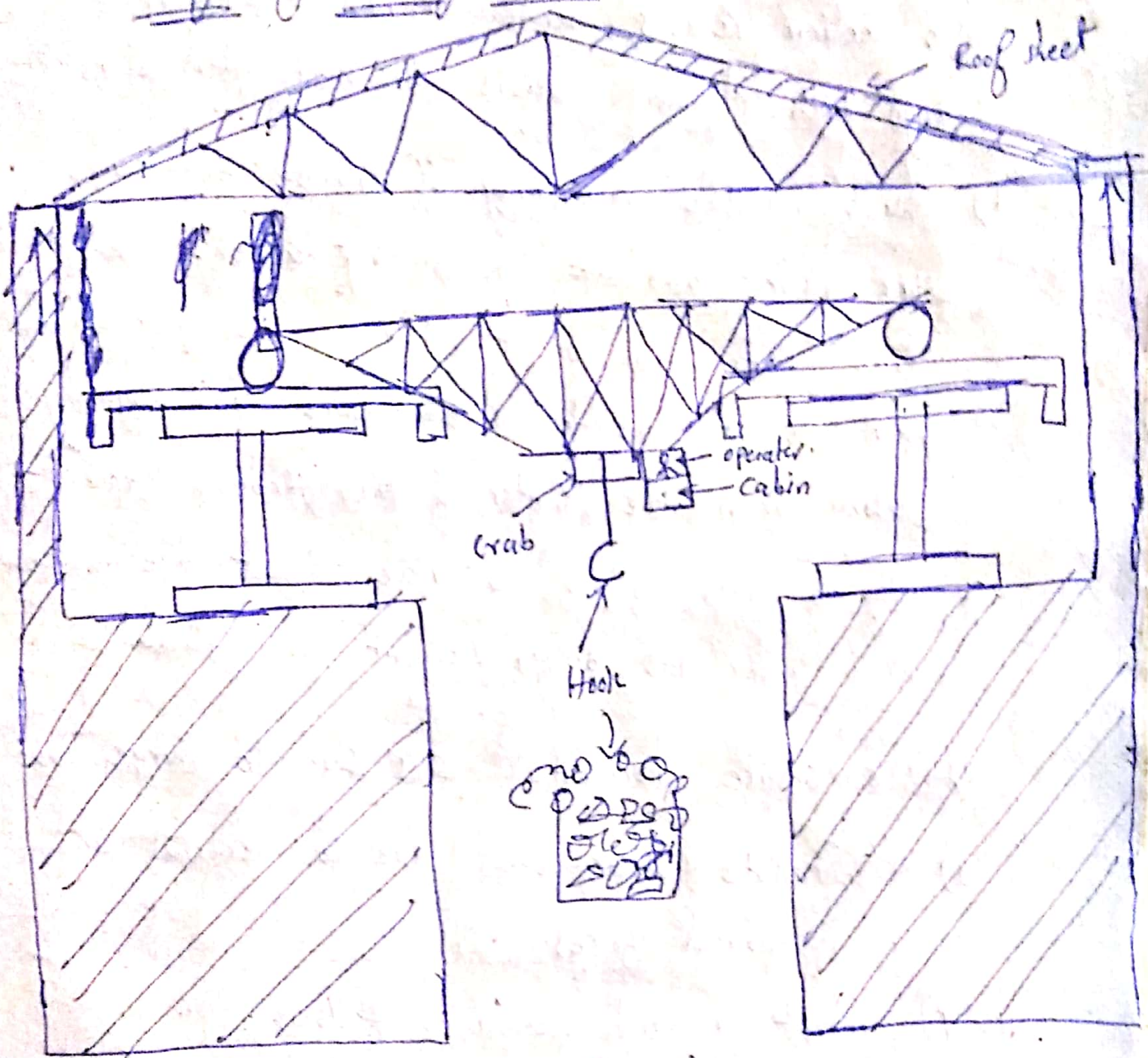
$\therefore \quad 0.7 \times s = 15.44 \quad \Rightarrow \boxed{s = 22.05\ \text{mm}}$

$$\frac{22}{3} = \underline{6\ \text{times}}$$

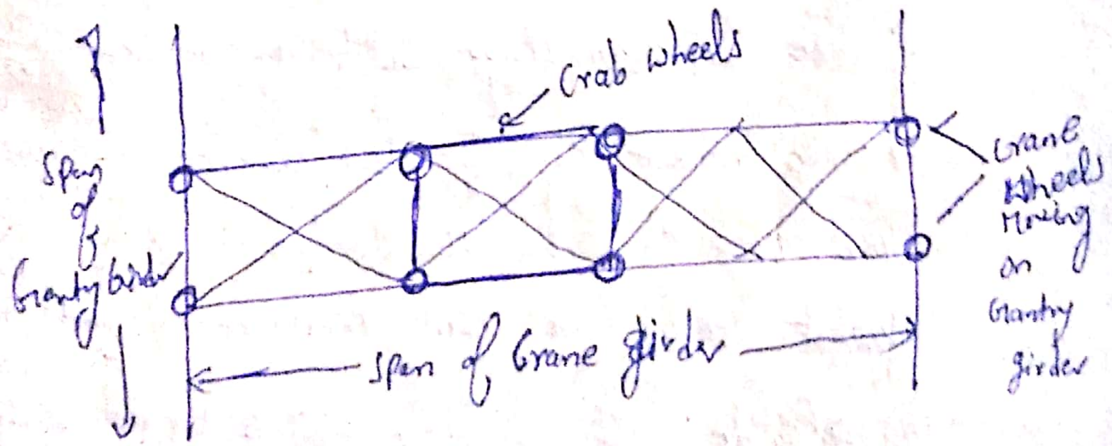
UNIT-V

Gantry Girder and plate Girder

(36) Design of Gantry Girder :-



c/s - Inner view or Elevation



plan

Design procedure

The Gantry girder should be designed such a way that Entire section resists vertical loads. and compression flange with channel resists the horizontal forces (Surge).

1) With suitable positioning of crane, determine Max Moment and S.F on Gantry Girder. Add impact load contribution to it. Though the position for Max Moment due to wheel load is slightly away from the center of the girder (under the wheel it is just added to Max Moment due to U.D.L on girder. and design moment is found.

2) Calculate horizontal B.M due to surge load

3) Calculate shear forces due to vertical and horizontal forces

4) Select a trial section as follows

5) The economic depth is about $\frac{1}{12} \times$ span and The compression flange width may be kept $\frac{1}{25} \times$ span

6) The moment capasity for vertical loads should be 40% more than the moment due to vertical load, so, that section can resist combined moment safely.

7) Calculate I_{zz}, I_{yy} & Z_p of the trial section selected

8) check for Moment capasity of the section

9) check for buckling resistance as per clause 8.8.2

10) Check for biaxial bending

11) check for shear capasity.

12) check for web buckling & web bearing

13) check for deflection

14) Design welds

⊛ Design a simply supported Gantry Girder to carry an electric over head travelling Crane, given

Span of Gantry Girder = 6.5 m

Span of Crane Girder = 16 m

Crane capasity = 250 kN

Self wt of Crane Girder excluding trolley = 200 kN

Self wt of trolly = 50 kN

Max hook approch = 1 m

Distance b/w wheels = 3.5 m

Self wt of rails = 0.3 kN/m

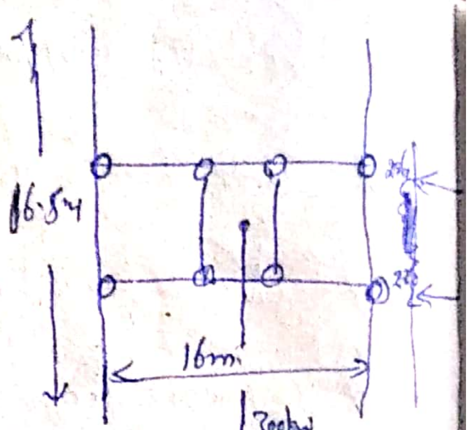
Sol:

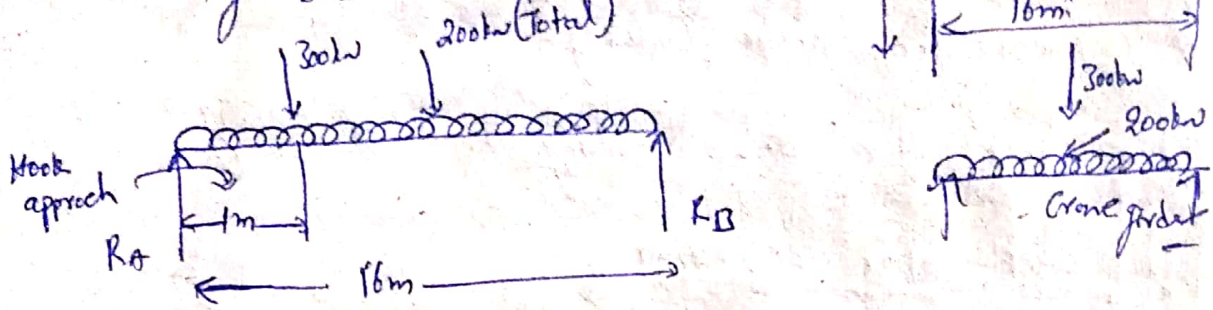
Finding the Moments

Self wt of Crane girder = 200 kN (total)

wt of trolly + lifted load = 50 + 250 = 300 kN

For Max reaction on Gantry girder, the moving load should be as close to Gantry girder as possible as shown in fig below





Taking M about 'B'

$$\Rightarrow \quad R_A \times 16 - 300 \times (15) - (200 \times 8) = 0$$

$$\Rightarrow \quad R_A \times 16 = 6100$$

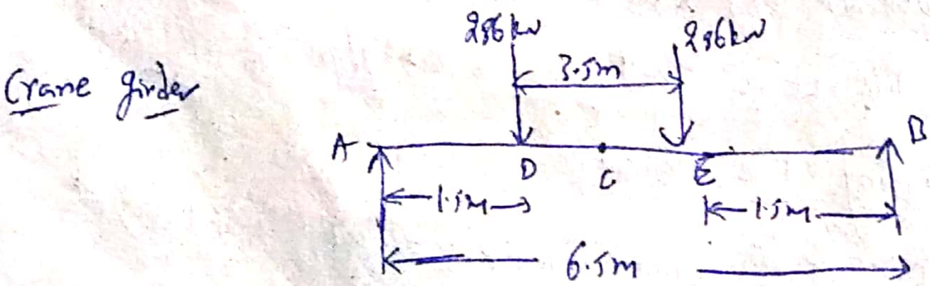
$$\boxed{R_A = 381.25\,kN}$$

This load is transfered to Gantry girder through 2 wheels. The wheel base being 3.5m (distance b/w wheels)

$\therefore$ load on Gantry Girder from each wheel $= \frac{R_A}{2}$

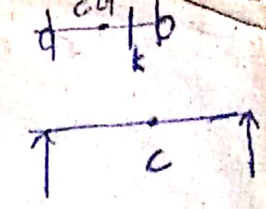
$$= \frac{381.25}{2} = 190.63\,kN$$

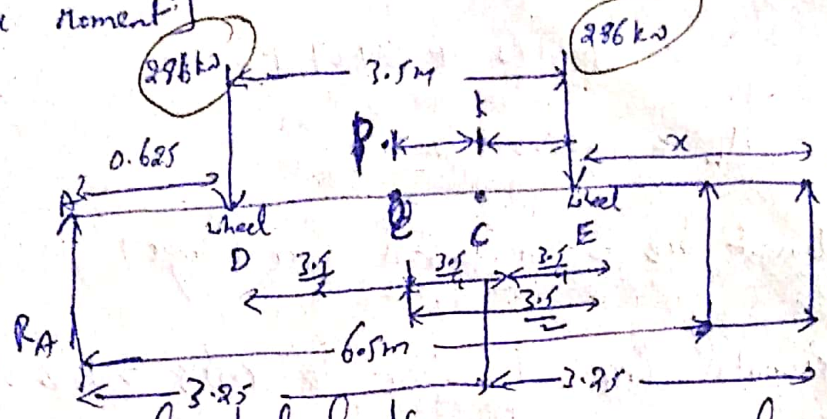
Factored wheel load $= 190.63 \times 1.5 = 286\,kN$



Statement: Max moment due to moving loads occur under a wheel when the C.G of wheel load and the wheel are equidistant from the center of girder as shown in fig below

[k & C are should coinside, Then we get Max Moment]





P is C.G of wheel loads, k is C.G of P and wheel

C is C.G of girder

Consider

$$\therefore R_B = \frac{0.625 \times 296}{6.5} + \frac{296 \times (3.5 + 0.625)}{6.5}$$

$$= 209 \text{ kN}$$

Max B.M will occur under a wheel considered (E)

$\therefore$ Max B.M at E $= R_B \times x$

$$= 209 \times 2.375$$

$$= 496.37 \text{ kN-m}$$

Moment due to impact $= 0.25 \times 496.375$

$$= 124.09 \text{ kN-m}$$

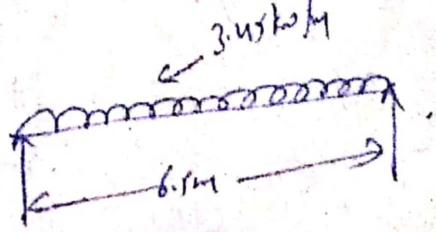
assume self wt of girder $= 2$ kN/m

$\therefore$ Dead load due to self wt + rails $= 2 + 0.3$

$$= 2.3 \text{ kN/m}$$

$\therefore$ Factored dead load $= 2.3 \times 1.5 = 3.45$ kN/m

Moment due to dead load



$$M \text{ due to U.D.L} = \frac{3.45 \times (6.5)^2}{8}$$

$$= 18.22 \text{ kN-m}$$

Factored Moment due to all vertical loads

$= 496.373 + 124.09 + 18.22$

$M_L = 638.699$ kN-m

Max Moment due to horizontal force (Surge)

horizontal force Transferse to rails = 10% of [Trolley weight + load lifted]

$= \frac{10}{100} \times [250 + 50]$

$= \frac{10}{100} \times 300$

$= 30$ kN

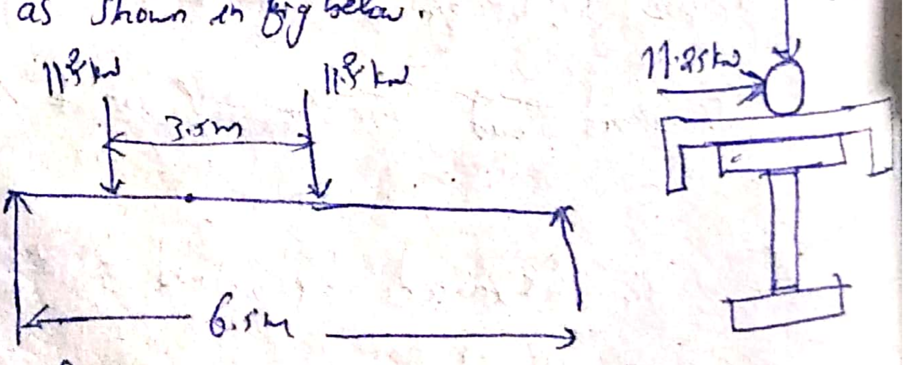
Assuming double flamed wheels, This is distributed over 4 wheels

$\therefore$ horizontal force on each wheel $= \frac{30}{4} = 7.5$ kN

$\therefore$ Factored horizontal force on each wheel $= 1.5 \times 7.5$

$= 11.25$ kN.

For Max Moment in Gantry Girder, The position of load is same as shown in fig below.



horizontal two wheel loads on Gantry Girder

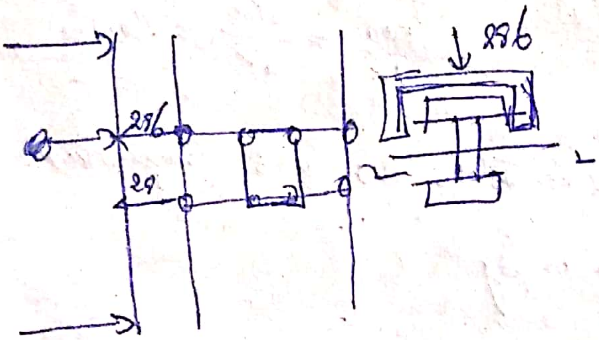
B.M in Y-direction due to horizontal wheel loads by proportioning, we get

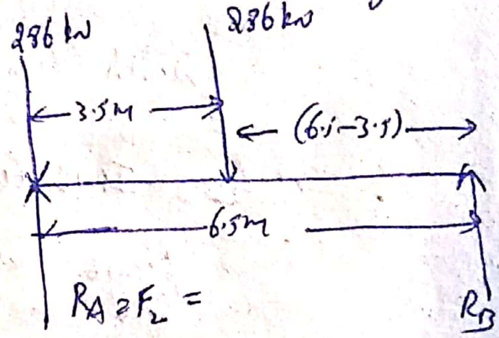
$M_y = 496.37$ when load is 286

$$M_y = \frac{496.37}{286} \times 11.25 = \underline{19.52\ kN\text{-}m}$$

Shear forces

For max shear force on the girder, the trailing wheel should be just on the girder as shown in fig below.





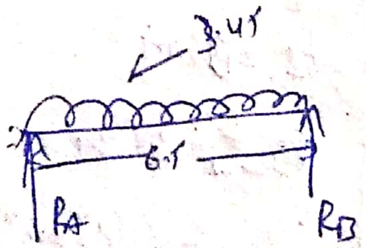
$$R_A = F_2 = 286 + \frac{286 \times (6.5 - 3.5)}{6.5}$$

$$= \underline{418\ kN}$$

vertical shear due to impact $= 0.25 \times 418$

$= 104.5\ kN$

Vertical shear due to self wt $= \dfrac{3.45 \times 6.5}{2}$



$= 11.21\ kN.$

∴ Total vertical shear $= 418 + 104.5 + 11.21$

$= \underline{533.71\ kN}$

Similarly by proportioning horizontal shear force due to horizontal surge

$$418 \rightarrow 286$$
$$= \quad ? \rightarrow 11.25$$
$$= \frac{418}{286} \times 11.25$$
$$= \underline{16.44\ kN}$$

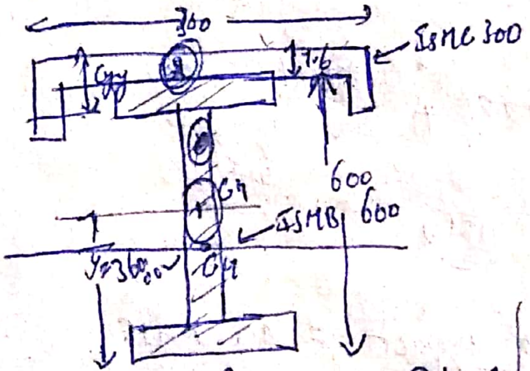
$6.5m = 6500\ mm$

Step: 2 Priliminary section

Economical depth of Gantry Girder $= \frac{L}{12} = \frac{6500}{12} = 541.7mm$ $\approxeq 600mm$

Width (flange) " " $= \frac{Span}{25} = \frac{6500}{25} = 260mm$ $\approxeq 300$

Choose ISWB 600 with ISMC 300 and compression flange as shown in fig.



Properties of ISWB600 @ 1.312kN/m

$A = 17038\ mm^2$

$b_f = 250mm$

$t_f = 21.3mm$

$t_w = 11.2\ mm$

$I_{zz} = 106199.5 \times 10^4\ mm^4$

$I_{yy} = 4702.5 \times 10^4\ mm^4$

Properties of ISMC 300

$A = 4564mm^2$

$b_f = 90\ mm$

$t_f = 13.6mm$

$t_w = 7.6\ mm$

$I_{zz} = 6362.6 \times 10^4\ mm^4$

$I_{yy} = 310.8 \times 10^4\ mm^4$

$C_{yy} = 23.6mm$

Let $\bar{y}$ be the distance of C.G for the total Gantry Girder section from the bottom most tension fiber.

$$\bar{y} = \frac{a_1 y_1 + a_2 y_2}{a_1 + a_2}$$

$$= \frac{(17038 \times 300) + 4564 \times (600 + 7.6 - 23.6)}{17038 + 4564}$$

$$= 360.002 \text{ mm}$$

$$I_z = I_1 + I_2$$

$$I_1 = I_{zz} \text{ of ISWB 600} + a_1 h_1^2$$

$$= 106198.5 \times 10^4 + (17038) \times (360.002 - 300)^2$$

$$= 1.123 \times 10^9$$

$$I_2 = I_{yy} \text{ of ISMC 300} + a_2 h_2^2$$

[reverse position ZZ = YY]

$$= 310.8 \times 10^4 + (4564) \times [(600 + 7.6 - C_{yy}) - 360.002]^2$$

$$= 310.8 \times 10^4 + 4564 \times (223.998)^2$$

$$= 232.1 \times 10^6 \text{ mm}^4$$

$$\therefore I_z = 1.123 \times 10^9 + 232.1 \times 10^6$$

$$= 1.3551 \times 10^9 \text{ mm}^4$$

$$= 1355.1 \times 10^6 \text{ mm}^4$$

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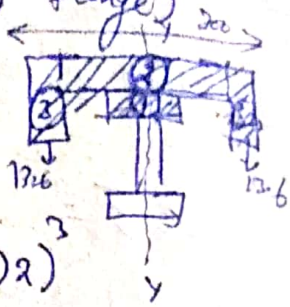
$I_{yy} = I_1 + I_2$ (for compression flange)



~~I_y~~ - I_{yy} for hatching portion only

$$I_y = \frac{21.3\times(250)^3}{12} + \frac{7.6\times(300-(13.6)2)^3}{12}$$

$$+ \left[\frac{\cancel{2}\times 90\times(13.6)^3}{12} + (13.6\times 90)\times\left(150-\frac{13.6}{2}\right)^2\right]\times 2$$

$$I_y = 90.82\times 10^6\ \mathrm{mm}^4$$

Z_{ez} = elastic section modulus about z-z axis

$$= \frac{I_{zz}}{y_{max}}$$

$$= \frac{1355.1\times 10^6}{360.002}$$

$$= 3.76\times 10^6\ \mathrm{mm}^3$$

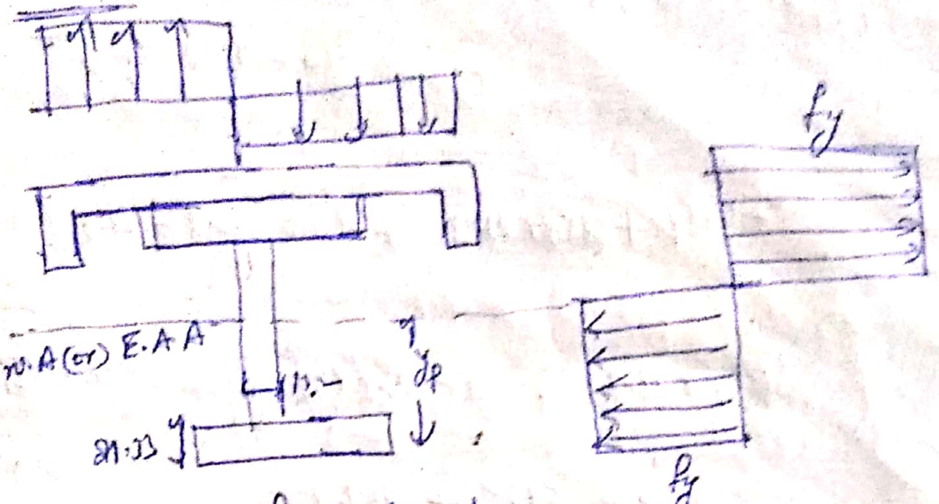
$$Z_{ey} = \frac{I_{yy}}{y_{max}}$$

$$= \frac{90.82\times 10^6}{150}$$

$$= 605.4\times 10^3\ \mathrm{mm}^3$$

~~Zp = plastic section modulus~~

plastic Section Modulus



Fully plastified section

Let y_p be the distance of plastic N.A [E.A.A from bottom edge] as shown in fig above

$$(y_p - 21.3)\,11.2 + 250 \times (21.3) = \frac{A}{2}$$

where A = Total area of Gantry Girder.

$$\Rightarrow (y_p - 21.3)\,11.2 + 5325 = \frac{17038 + 4564}{2}$$

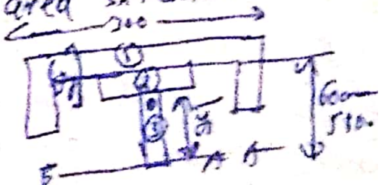
$$(y_p - 21.3)\,11.2 = 5476$$

$$y_p = 510.22 \text{ mm}$$

plastic Section Modulus

$$Z_p = \frac{A}{2}(\bar{y}_1 + \bar{y}_2)$$

where $\bar{y}_1$ = distance of C.G of an area situated above E.A.A



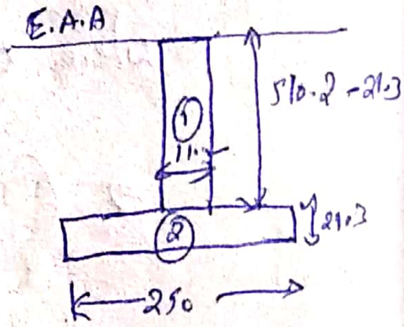
$$\bar{y}_1 = \frac{4564 \times (600 - 510.2 + 7.6 - 23.6) + 250 \times 21.3 \times (600 - 510.2 - \frac{21.3}{2}) + 11.2 \times \frac{(600 - 510.2 - 21.3)^2}{2}}{}$$

$$\bar{y}_1 = \frac{4564 \times (600 - 510.2 + 7.6 - 23.6) + 250 \times 21.3 \times \left(600 - 510.2 - \frac{21.3}{2}\right) + \frac{11.2 \times (600 - 510.2 - 21.3)^2}{2}}{4564 + 250 \times 21.3 + (600 - 510.2 - 21.3)(11.2)}$$

$$= \frac{4564 \times 73.8 + 250 \times 21.3\,(79.15) + \frac{11.2}{2} \times 4692.25}{4564 + 5325 + 767.2}$$

$$= \underline{73.68\,MM}$$

Let $\bar{y}_2$ be the distance of C.G of an area situated below the equal Area axis



$$\bar{y}_2 = \frac{a_1 y_1 + a_2 y_2}{a_1 + a_2}$$

$$= \frac{(510.2 - 21.3)\, 11.2 \times \left(\frac{510.2 - 21.3}{2}\right) + (21.3 \times 250)\left(510.2 - 21.3 + \frac{21.3}{2}\right)}{(510.2 - 21.3)\, 11.2 + (250) \times (21.3)}$$

$$= \frac{(488.9) \times 244.45 + 5325 \times 499.85}{(488.9 \times 11.2) + 5325}$$

$$= \underline{257.5\ mm}$$

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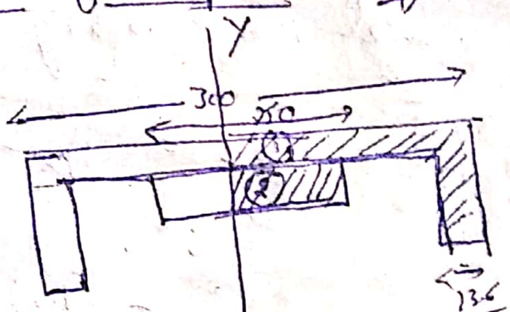
plastic section modulus $Z_{P_z} = \frac{A}{2}(\bar{y}_1 + \bar{y}_2)$

$$= \frac{17039 + 4564}{2}(73.62 + 257.5)$$

$$= 3.57 \times 10^6 \ mm^3$$

plastic section about YY axis for compression flange



$A = 4564 + 250 \times 21.3$

$= 9889 \ mm^2$

$\bar{y}_1$ = C.G of section right of YY-axis

$$\bar{y}_1 = \frac{\left(\frac{4564}{2}\right) \times \left(\frac{150}{2} + 150 - \frac{13.6}{2}\right) + \left(\frac{250 \times 21.3}{2}\right) \times \frac{250}{4}}{\left(\frac{4564}{2}\right) + \left(\frac{250 \times 21.3}{2}\right)}$$

$= 134.5 \ mm$

$\bar{y}_2$ = distance of C.G of an area situated left of YY-axis

$\bar{y}_1 = \bar{y}_2 = 134.53 \ mm$

$\therefore \ Z_{py} = \frac{A}{2}(\bar{y}_1 + \bar{y}_2)$

$$= \frac{9889}{2} \times (134.53 + 134.53)$$

$$= \frac{2.66 \times 10^6}{2}$$

$$= 1.33 \times 10^6 \ mm^3$$

Check for Moment Capacity

$\frac{b}{t_f}$ of flange of ISMB 600 $= \frac{\frac{210}{2}}{21.3} = 5.86 < 8.4\varepsilon$

$\frac{d}{t_w} = \frac{600 - 2 \times 21.3}{11.2} = 49.76 < 84\varepsilon$

(Page 18)

The given Section is plastic

Similarly for Compression flange

$$\frac{b}{t} = \frac{90-7.6}{13.6} = 6.05 < 8.42$$

$\therefore$ it is a plastic section

<u>Local moment capacity for bending</u> (z-direction) in vertical plane

$$M_{dz} = \beta_b \cdot Z_{pz} \times \frac{f_y}{\gamma_{mo}}$$ page no - 53

$$= (1.0) \times 3.57 \times 10^6 \times \left(\frac{250}{1.1}\right)$$

$$= 811.36 \times 10^6 \text{ N-mm}$$

but $M_{dz} \leq 1.2\, Z_{ez}\left(\frac{f_y}{\gamma_{mo}}\right)$

$$\leq 1.2 \times 3.76 \times 10^6 \times \frac{250}{1.1}$$

$$\leq 1025.4 \times 10^6 \text{ N-mm}$$

ie $\therefore$ $M_{dz} = 811.36 \times 10^6 > M_z$ (639.68 kN-m)

$M_{dz} = 811.36$ kN-m > (Max BM in z-direction)

hence ok.

<u>Moment Capacity in Y-direction</u>

$$M_{dy} = \beta_b \cdot Z_{py} \times \left(\frac{f_y}{\gamma_{mo}}\right) \leq 1.2 \times Z_{ey} \times \left(\frac{f_y}{\gamma_{mo}}\right)$$

$$= (1.0) \times 1.73 \times 10^6 \times \left(\frac{250}{1.1}\right) \leq 1.2 \times 605.4 \times 10^3 \times \left(\frac{250}{1.1}\right)$$

$$= 393.27 \times 10^6 \text{ N-mm} \geq 165.109 \times 10^6$$

$\therefore$ hence take $M_{dy} = 165.109 > M_y = 17.52$

hence ok.

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Check for Interaction formula (Combined local capacity)

$$\frac{M_z}{M_{dz}} + \frac{M_y}{M_{dy}} \leq 1$$

(Page 70)

$$\Rightarrow \frac{638.68}{811.367} + \frac{19.52}{165.109} \leq 1$$

$$\Rightarrow 0.905 \leq 1$$

hence ok

Check for buckling resistance:-

This check need not be checked

for this check $M_d = \beta_b \cdot Z_{p_z} \times f_{bd}$ (treated as laterally un supported)

f_{bd} = reduced bending strength which can be obtained from table no. 13(a).

Proceed from Table no: (14).

$F_z = 533.7$ kN

$F_y = 16.55$ kN

∴ Consider max of above two = F_z = 533.7 kN

$$\text{shear Capacity} = \frac{A_v(f_y)}{\sqrt{3}\,\gamma_{m0}}$$

$$= \frac{h t_w}{\sqrt{3}}\left(\frac{f_y}{\gamma_{m0}}\right)$$

$$= \frac{600 \times 11.2}{\sqrt{3}} \times \left(\frac{250}{1.1}\right)$$

$$= 881.77 \text{ kN} > 533.7 \text{ kN}$$

hence ok

Check for web buckling

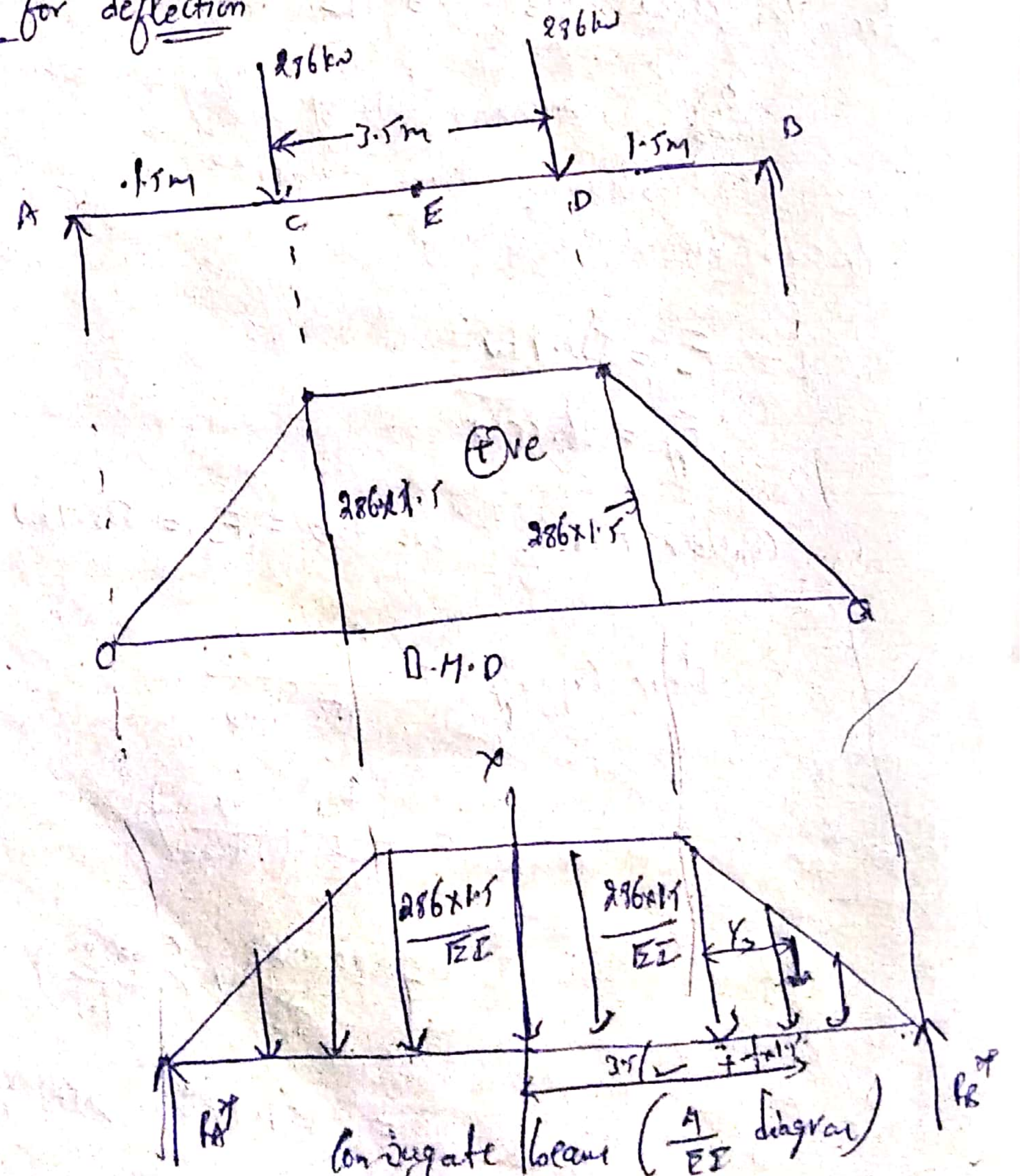
$$\frac{d}{t} = \frac{600 - 2(21.3 + 17)}{11.2} = 46.73 < 67$$

as per 8.4.2.0 (page 59)

$\frac{d}{t_w} < 67\varepsilon$ There is no need to check for web ~~crippling~~ buckling

check for web Crippling, it is no need to check because ISWB 600, it is capable to resist Crippling.

Check for deflection



Conjugate beam ($\frac{M}{EI}$ diagram)

working load, deflection is to be limited = $\frac{Span}{750}$ for Gantry Girder

(Table 6, Page 31)

For Max deflection, wheel load as shown in fig above.

Max deflection will occur at E

$\therefore$ $Y_{max} = Y_{at E}$ = BM at E for Conjugate beam

Let R_A^* & R_B^* be the support reactions for the Conjugate beam

$$R_A^* = R_B^* = \frac{1}{2} \times \text{Total } \frac{M}{EI} \text{ diagram}$$

$$= \frac{1}{2} \times \left[\frac{a+b}{2} \times h \right]$$

$$= \frac{1}{2} \times \left[\frac{6.5+3.5}{2} \times \frac{286 \times 1.5}{EI} \right]$$

$$= \frac{1}{2}\left(\frac{10}{2} \times \frac{286 \times 1.5}{EI} \right)$$

$$= \frac{4290}{4EI}$$

$$= \frac{1072.5}{EI}$$

M at E = - Moment of $\square$ loading - Moment of Δ Area + Moment of R_B^*

=

$$= R_B^* \times \frac{6.5}{2} - \left[\left(\frac{1}{2} \times 1.5 \times \frac{286 \times 1.5}{EI} \right) \times \left(\frac{3.5}{2} + \frac{1}{3} \times 1.5 \right) \right]$$

$$- \left(\frac{286 \times 1.5}{EI} \times \frac{3.5}{2} \times \frac{3.5}{4} \right)$$

$$= \frac{1072.5}{EI} \times \frac{6.5}{2} - \cancel{28}\ \frac{321.75}{EI} \times (2.25) - \frac{656.90}{EI}$$

$$= \frac{2104.78 \text{ kN-m}}{EI}$$

$$= \frac{2104.78 \times 10^6}{2 \times 10^5 \times 1355.1 \times 10^6}$$

$$Y_{max} = 7.76 \times 10^{-6} = 0.0000076$$

limiting deflection (permissable) $\delta_{limit} = \frac{L}{750} = \frac{6500}{750} = 8.6\,mm > Y_{max}$

hence <u>ok</u>.

plate Girders

When Span and load increase, The available rolled Section may not be sufficient, even after stiffening with Cover plates.
In that Situation, The beams are Manufactured by using different plates by welding or rivetting or bolting is known as plate girder, as shown in fig below.

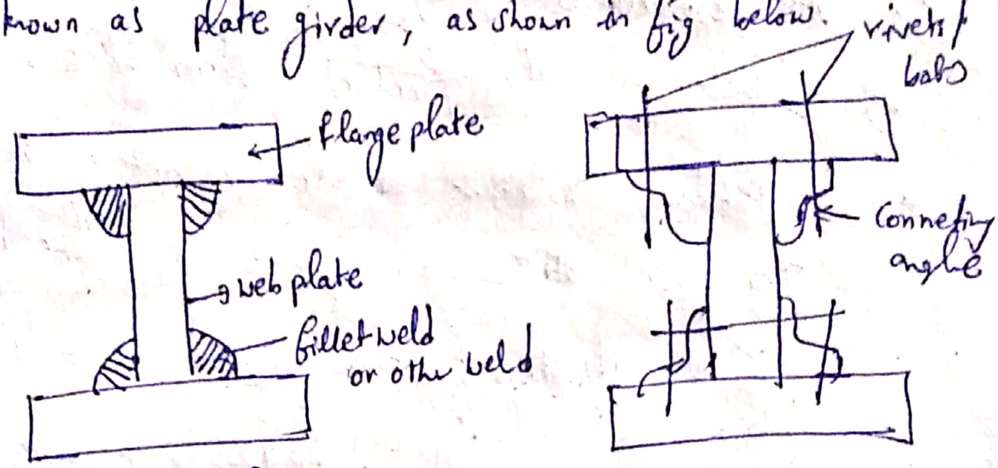
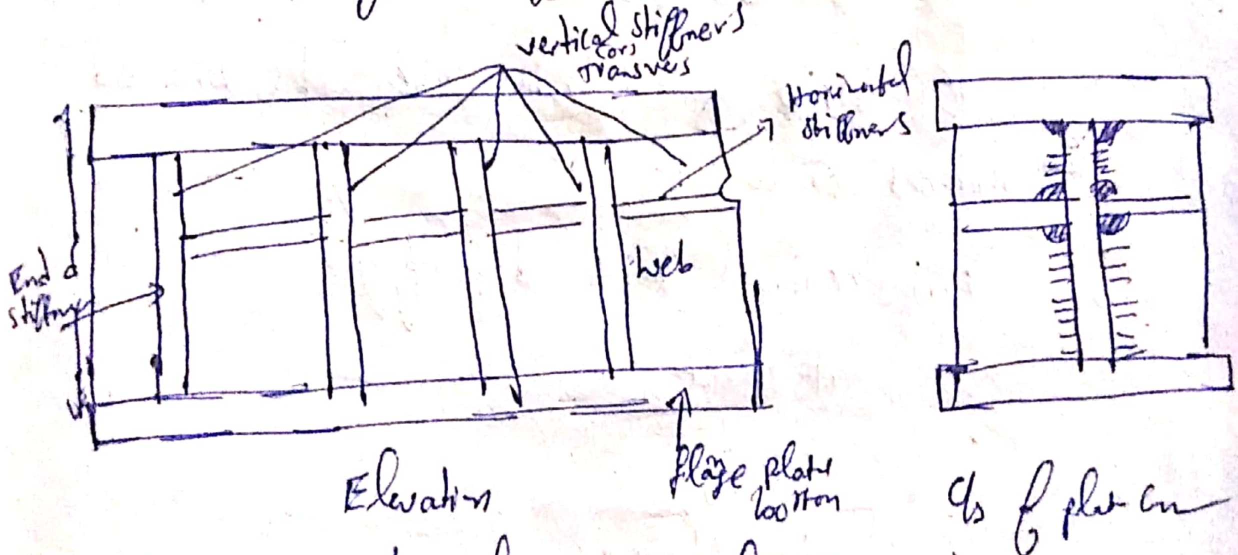


fig plate girder.



Following are the elements typical plate girder.

1) web
2) Flanges
3) stiffners as shown in fig

Notes:- Flanges are assumed to be resist bending therefore the flanges are designed to resist moment.

The web of a beam is assumed to be resist shear. Therefore the web is designed to resist shear

Design Procedure

1) Assume self wt $= \frac{w}{200}$ where w = Total factored load

~~2)~~ Determine the factored shear force & moment

2) Decide whether to use or not to use Transverse stiffners & assume the value $k = \frac{d}{t_w}$

if the stiffness of the avoided $k \leq 67$ (normal plate as girder)

if $k \geq 200$, intermediate Transverse stiffners are to be provided.

if $k = 100$ that indicates Thin webs with end ~~bear~~ stiffners can be used

3) Find economical depth $d = \left(\frac{Mk}{f_y}\right)^{1/3}$

M = Moment

$k = \frac{d}{t_w}$

f_y = Yield stress

4) Determine the area of flange to resist Moment. Proportion it so that $\frac{b}{T_f}$ satisfies requirements of plastic, compact, semi-compact.

5) Check for Moment Capacity of girder
6) Design of weld
7) Design of end bearing stiffners
8) Design the Connection of stiffner
9) Design intermediate stiffner if req.

(7) Design a welded plate girder of span 24m to carry Superimposed load of 35 kN/m. Avoid use of bearing and intermediate stiffners. use Fe415 steel.

Sol) **Design:-**

1) Calculation of Moment and shear forces:

Span = 24m

Superimposed load = 35 kN/m

$\therefore$ factored load = $35 \times 1.5 = 52.5$ kN/m

Self wt $= \frac{W}{200} = \frac{52.5}{200} = 6.3$ kN/m

$\therefore$ Total factored load $= 52.5 + 6.3 = 58.8$ kN/m

Moment (Max) $= \frac{Wl^2}{8} = \frac{58.8 \times 24^2}{8}$

$= 4233.6$ kN-m

Shear force $= \frac{wl}{2} = \frac{58.8 \times 24}{2} = 705.6$ kN.

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ii) Economical depth (depth of web plate) :-

If stiffeners are to be avoided

$\therefore k \leq 67$

$\frac{d}{t_w} \leq 67$

$\therefore$ economical depth of web $d = \left(\frac{Mk}{f_y}\right)^{1/3}$

$= \left(\frac{4233.6 \times 10^6 \times 67}{250}\right)^{1/3}$

$= 1043$ mm

$\therefore$ use 1000 mm plates

$\therefore$ t_w :- $\frac{d}{t_w} \leq 67 = 1$

$\Rightarrow \frac{d}{67} \leq t_w$

$\Rightarrow \frac{1000}{67} \leq t_w = 1$

$t_w = 14.91 \cong 16$ mm

$\therefore$ web plate selected is 1000 mm $\times$ 16 mm

iii) selection of flanges

neglecting moment capacity of web, area of flange required

$$\frac{A_f \cdot f_y \cdot d}{\gamma_{m0}} \geq M$$

$A_f \times \frac{250}{1.1} \times 1000 \geq 4233.6 \times 10^6$

$A_f \geq 18628$ mm²

To keep the flange in plastic category $\frac{b}{t_f} \leq 8.4 \varepsilon$

$$\therefore \frac{b_f}{2t_f} \leq 8.4$$

assume $b_f = 16.8\, t_f$

$$A_f = 18628 = b_f \times t_f = 16.8\, t_f \times t_f$$

$$\Rightarrow \quad 16.8\, t_f^2 = 18628$$

$$\Rightarrow \quad t_f = \sqrt{\frac{18628}{16.8}} = 33.29 \cong 40\ \text{mm}$$

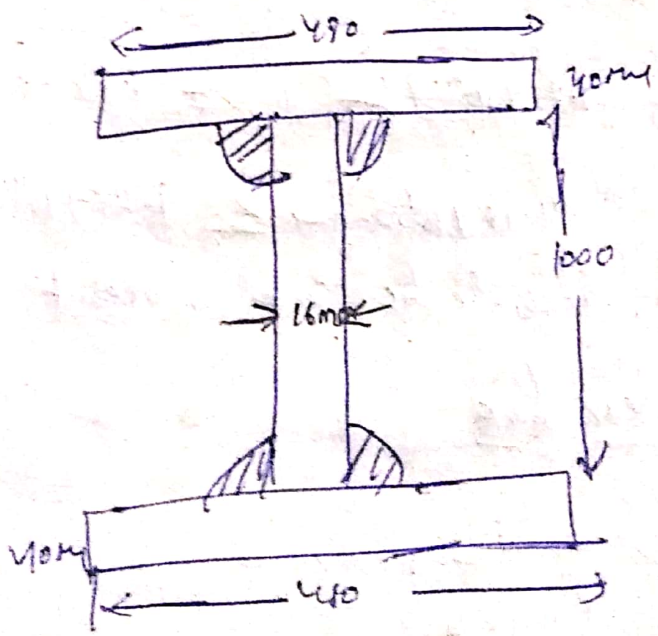
So provide 40 mm thickness of flange

b_f (breadth of flange)

$$b_f = \frac{A_f}{t_f} = \frac{18628}{40} = 465.7$$

So, provide 480 mm breadth of flange.

here we 480 mm wide & 40 mm Thick flange plates & plastic section is shown in fig below



(iv) Check for Moment capacity of the girder.

Since it is assumed that only flanges resist the moment and flange is a plastic section

$$M_d = Z_e\left(\frac{f_y}{\gamma_{mo}}\right)$$

Z_e = elastic section modulus @ major axis

$$Z_e = Z_{e_z} = \frac{I_{zz}}{y_{max}}$$

$$I_{zz} = \left[\frac{480\times40^3}{12} + 480\times40\times\left(\frac{1000}{2}+\frac{40}{2}\right)^2\right]\times 2$$

$$= 1.03\times10^{10}\ mm^4$$

$$\therefore Z_{e_z} = \frac{I_{zz}}{y_{max}} = \frac{1.03\times10^{10}}{500+40} = 1903\times10^6\ mm^3$$

$$\simeq 20745\times10^6\ mm^3$$

$$\therefore M_d = 19.23\times10^6\times\left(\frac{250}{1.1}\right) \leq 1.2\times Z_e\left(\frac{f_y}{\gamma_{mo}}\right)$$

$$4.75\times10^9\ N\text{-}mm \geq 4.25\times10^9$$

hence ok, adequate the section to resist Moment

(v) Design shear force

$$V_d = \left(\frac{f_y}{\sqrt{3}\,\gamma_{mo}}\right)\times A_v$$

$$= \frac{250}{\sqrt{3}\times1.1}\times1000\times16$$

$$= 2099.45\ kN > 7.5\ kN\ (\text{Max SF})$$

$\therefore$ hence, assumed section is adequate to resist shear force.

Check for end bearing:-

bearing strength of web

$$f_w = (b_1 + n_2)\, t_w \times \left(\frac{f_y}{\gamma_{m0}}\right)$$

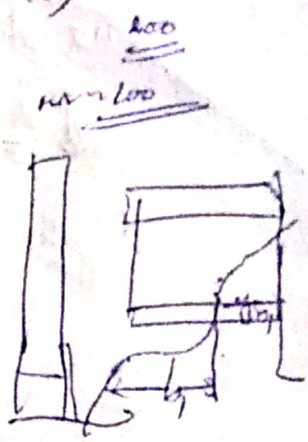
assume $b_1 = 1000$ mm (width of support (min))

Defined length $n_2 = 2.5 \times 40$

$= 100$

$$f_w = (1000 + 100) \times 16 \times \left(\frac{250}{1.1}\right)$$

$= 727$ kN > 7.5 kN.



Design of weld:-

Max shear force $= 705.6$ kN.

Shear stress/mm length at the level of the flange & web

$$\tau/mm = \frac{F\,(a\bar{y})}{I\,b}$$

$$= \frac{705.6 \times 10^3 \times (480 \times 40) \times \left(\frac{40}{2} + 500\right)}{1.03 \times 10^{10}}$$

$= 6833$ N/mm.

This equals to force of weld product of the junction is

$$S \times t_e \times \frac{f_u}{\sqrt{3}\,\gamma_{m0}} \quad (1)$$

$$= 0.7 \times S \times \frac{410}{\sqrt{3} \times 1.0} \longrightarrow (2)$$

$$0.7 \times S \times \frac{410}{\sqrt{3} \times 1.1} = 6833$$

$S = 5.15 = 6$ mm

size of weld $S = 6$ mm

Conclusion :-

web = 1000 x 16 mm

flange = 490 x 40 mm

no stiffners are required

weld = provide 6mm size continuously along the length of girder at the contact of flange to web.

Note :- To find the selection of plates (of flange & web plates) including bearing stiffners the procedure is same as above upto last & k should be taken as 100 mm ie $\frac{d}{t_w}$ for finding economical depth.

2) Similarly to provide intermediate stiffners also along with the end stiffners $k = \frac{d}{t_w} = 200$ mm, the procedure is same as previous.