234

4

234.-

-: HAND WRITTEN NOTES:-

OF

CIVIL ENGINEERING

4

-: SUBJECT:-

DESIGN OF STEEL STRUCTURE

4

2

Design of Steel Structures

Rivetted connection

Welded connection

Eccentric connection < Rivetted / Welded

Tension member

Compression members

Beams

Plate girders

Industrial building } obj

Plastic Analysis - Most Imp

3

IS : 800 - 1984 ✓

IS : 800 - 2007
based on limit state method of analysis
Although W.S.M. given in appendix.

IS 875 - loading

By - Kachan Thakur

Rivetted Connection.

Rivets are —
Generally made of mild steel. and for normal construction purposes Fe 410S grade of steel. 410 N/mm^2 ultimate strength of steel. This grade
- Nominal dia of rivet

Cold Rivetting
Hot Rivetting

is suitable for rivetting for all thicknesses but for welding it is suitable only upto 20mm thickness of member.

For thicknesses more than 20 mm welding, grade of steel is used for member, they are Fe 410 WA
WB
WC

4

The other grades of steel are - Fe 570 HT
HT = high tension
High tension grade of steel has higher strength and they are more resistant to atmospheric corrosion hence this grade is adopted when reduction in dead wt is desired and when str. is prone to atm corrosion

Sp gravity = 7.85

unit wt / density = 7850 Kg/m^3

unit wt γ = 78.5 KN/m^3

$E = 2 — 2.15 \times 10^5$ N/mm^2

$\alpha = 1.2 \times 10^{-6}$ /°C

Advantage of steel structure over Aluminium str.

Strength per unit wt of aluminium is much greater than that of steel.

However $E_{al} = \frac{1}{3} E_{st}$

Hence Al structure is more prone to buckling to over come this crosssectional area of Al str needs to be increased Even then a saving of 50% in wt results by the use of Al str.

However Al is almost 8 times costlier than steel. Thus greater economy is achieved by the use of steel str.

$$\alpha_{al} = 2\, \alpha_{st}$$

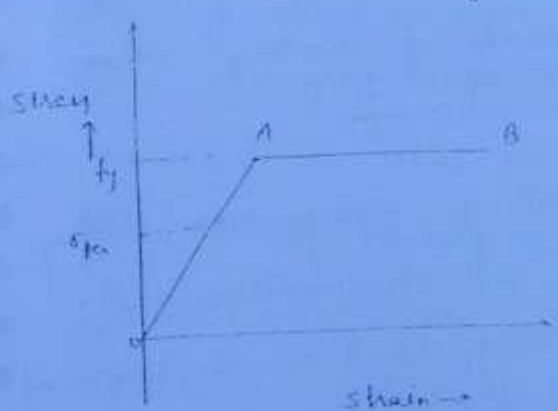
Hence it can not be encased in concrete as steel is normally done.

Note Encasing is normally done for aesthetic & fire resistance

Maintenance cost of Al structure is less as it is not prone to corrosion

Method of Design

Elastic method of design is used



$\sigma_{per} = \frac{f_y}{F.O.S}$

Simplified mild steel stress strain curve

The structure is designed in such a way that stress in the member do not exceed permissible stress.

To design the structure we need to access the loading to which the structure will be subjected to. Hence there is uncertainity in loading similarly there is uncertainity in material property

Also to simplify the analysis certain assumptions are made because of which error will be introduced in the result, to account for all these we use F.O.S.

6

F.O.S. for tension = 1.67 = $\frac{1}{0.6}$ $f_t = 0.6 f_y$

F.O.S for bending = 1.5 = $\frac{1}{.66}$ $f_b = 0.66 f_y$

Note: In case of bending there is a margin to resist additional loading beyond the point of first yield. However in tension there is no margin beyond the point of first yield. Hence FOS in tension is more as compared to bending.

Nominal Dia of rivet

It is the dia of shank of rivet in cold condition

Cold and Hot rivetting.

Cold and hot rivettings are methods of rivetting. cold rivetting is not adopted for dia > 10 mm

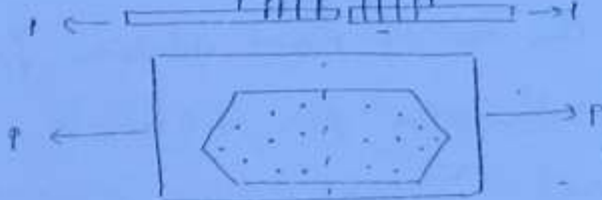
In hot rivetting rivets are heated to 550 — 1000°C. They are inserted in the hole made in the members and hammering is done so as to make head on the hammered side. The diameter of hole in the member should be more than the nominal dia of rivet so that the rivets can be easily inserted in heated condition.

7

In comp. gross area is effective and in tension net area is effective in resisting loads.

Rivets in group subjected to direct load share the load equally (if they are of the same diameter)

Actually stress is assumed to be equally shared



Force taken by one rivet $= \frac{P}{9}$

The outer rivet will be stressed more as compare to the inner rivet hence in one line we do not adopt more than 5 rivets otherwise unbuttoning effect will occur and rivets will start breaking one by one.

Bending stress in rivet is neglected under normal situation but if grip length is more the bending can not be neglected in that case additional precautions are taken as follows. If packing is of larger depth, additional rivets are provided on packing extension. This additional rivets absorbs the effect of (reduces the effect of) bending stress.

Recommendation. If the grip of rivet exceeds 6 times the dia of rivet hole, no of rivets required by normal calculation should be increased by not less than 1% for each additional grip of 1.6 mm ...

provided on packing extension.

Ex. Dia of hole = 20 mm
grip length = 152 mm
load = 100 KN

Strength of 1 rivet = 20 KN
Calculate the no. of rivets required.

6 x dia of hole = 6 X 120 = 120 mm

grip over 120 mm = 152 - 120 = 32 mm

So additional no. of rivets (over and above from normal calc.)
= 1% of the no. of rivet obtained from normal calculation per 1.6 mm

no. of rivet req. by normal cal = $\frac{100}{20} = 5$

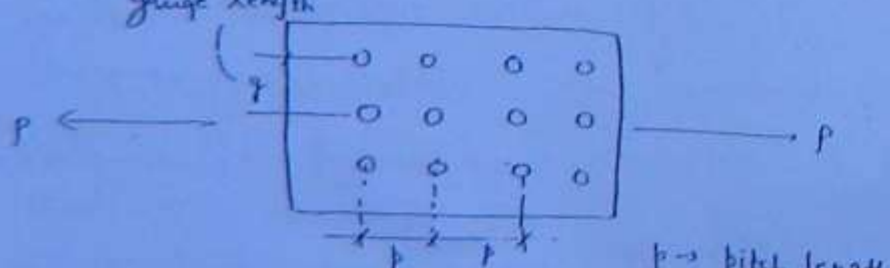
So Add. no. of rivet required : $\frac{32}{1.6}^{20} \cdot \frac{1}{100} \times 5 = 1$

total no. of rivet = 5 + 1 = 6

- Rivet fills the hole completely. i.e. to calculate strength of rivet we use the gross dia of rivet i.e the hole dia.

Pitch & Gauge

gauge length



p → pitch length

Distance b/w centre line of two rivets in the dir of force is called pitch & Dis b/w centre of two rivets at right angle to the dir of force is called gauge

Dia of hole = Nominal dia of rivet + 1.5 mm for dia $\le$ 25mm

Dia of hole = Nominal dia of rivet + 2 mm for dia > 25 mm

Dia of hole is also called as <u>gross dia of rivet</u> (This is under the assumption that rivet fills the hole completely)

9

<u>Unwin's formula</u>

Dia of rivet to suit the thickness of member

$d_{mm} = 6.05\sqrt{t_{mm}}$ (Not recommended by IS code)

$\downarrow$

Nominal dia

It is a guideline

t is the thickness of thinner member being joined

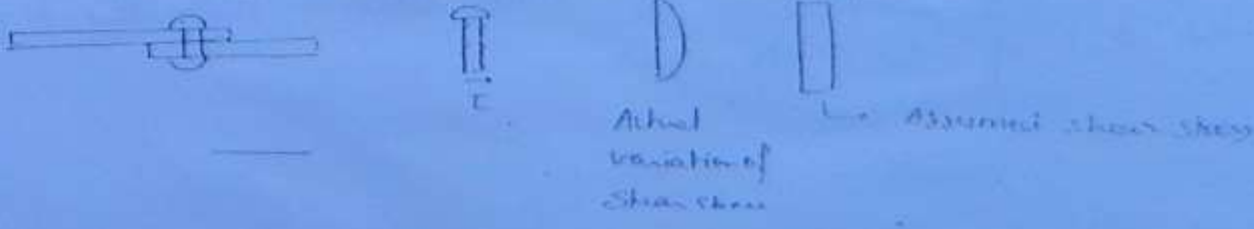
Normally adopted dia of rivets are in (mm) 10, 12, 16, 18, 20, 22, 25, 30 in mm

Assumptions in Rivetted connection

Friction b/w the plates is neglected

In hot rivetting because of gripping friction develops b/w the plates So long as the external force overcomes the frictional force, rivets will not be subjected to any stress Once the friction is overcome, the rivets will deform and hence will start sharing loads but it is difficult to quantify the amount of friction hence it is neglected and all the forces are assumed to be resisted by rivets only

Shear stress is uniform over the Xsecn of rivet



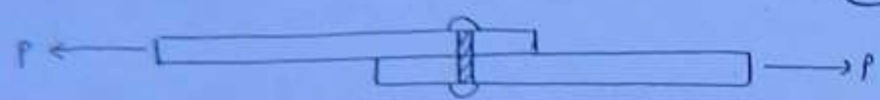
Max S.F. resisted by 1 rivet $= \left\{ \sigma_{perm} \times \frac{\pi d_{hole}^2}{4} \right\}$

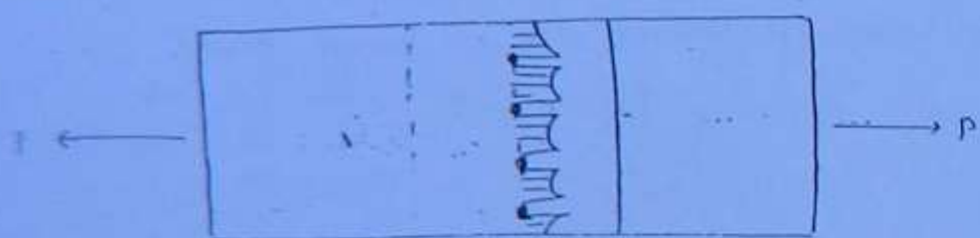
Note: Shear failure is a sudden failure hence FOS for shear is large.

Above assumption is a simplifying assumption.

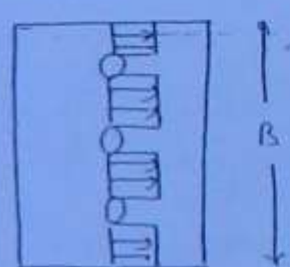
: Distribution of direct stress on the portion of plate b/w the rivet hole is uniform

10





Actual stress variation



Assumed stress variation

B, t thickness of plate

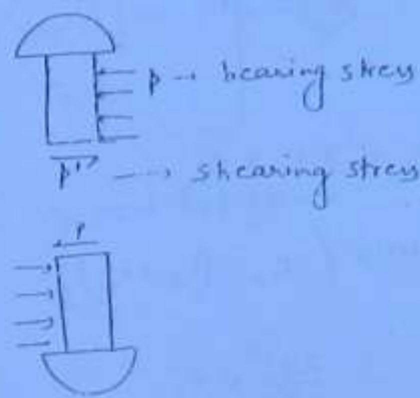
d'. dia of hole

σ_t . per. tensile stress

$$P_{max} = \underbrace{(B - 3d') \times t}_{A_{net}} \times \sigma_t$$

Note: Because of stress concentration stress is normally 2-3 times that of av. stress

Transmission of load through rivets



$P = p \times \pi d' t_1$

$P = p' \dfrac{\pi d'^2}{4}$

(11)

hole dia = d'

Max bearing force resisted by top portion of rivet

$= (d' \times t_1)\,\sigma_{br}$

by bottom $(d' t_2)\,\sigma_{br}$

Max force resisted by rivet in bearing $= d' \times t \times \sigma_{br}$

t → min thickness

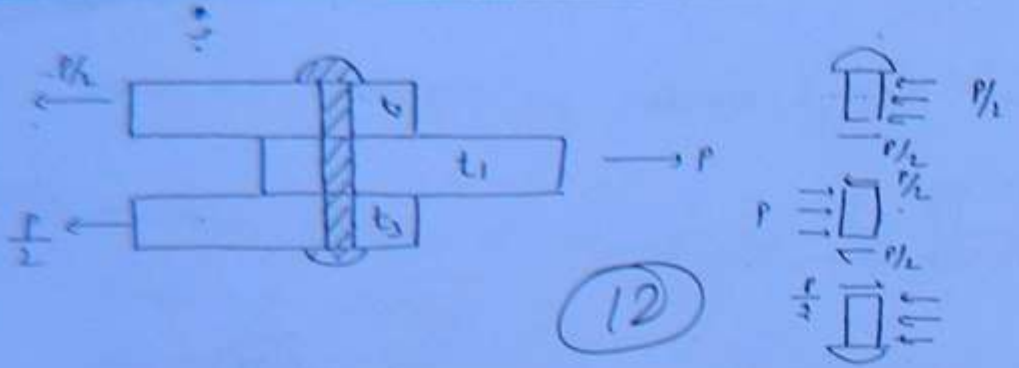
Max force resisted by rivet in shearing $= \dfrac{\pi d'^2}{4}\,\sigma_s$

σ_s → permissible shearing stress

The strength of rivet is min of shearing and bearing strength this is called rivet value of the rivet

→ strength of rivet

If the rivet fails in shear by shearing on one plane the rivet is said to be in single shear



12

d' → Dia of hole

Bearing strength of rivet $= d' \times t \times \sigma_{bt}$

$t \rightarrow \min \{ t_1, (t_2 + t_3) \}$

Shearing strength of rivet $= 2 \frac{\pi d'^2}{4} \times \sigma_s$

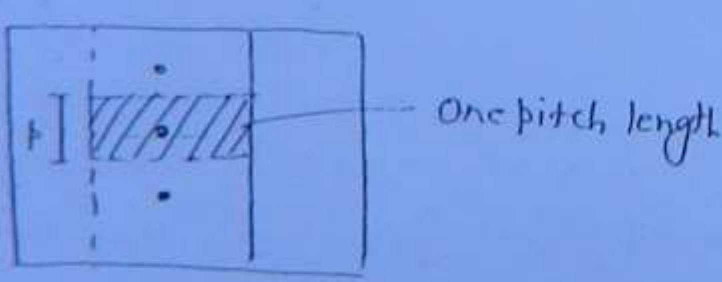
In this case rivet is said to be in double shear.

Note

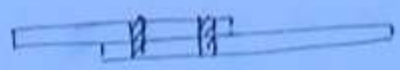
Max. strength of rivet is taken as that corresponding to double shear.

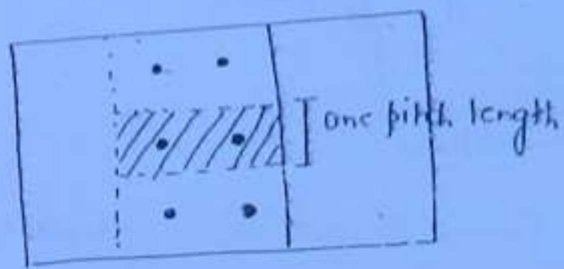
Type of Joints

Lap joint



Single Rivetted Lap joint

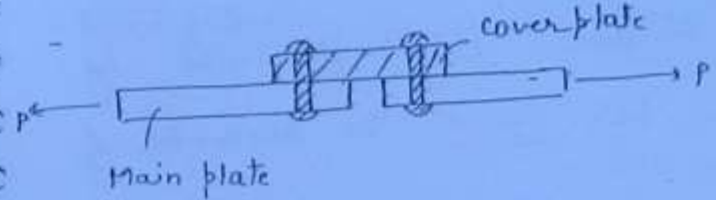




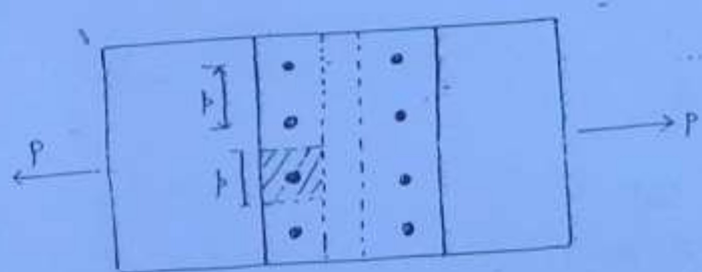
Double rivetted lap joint

13 similary when in one pitch length there are 3 rivets

Triple rivetted lap joint.



Double rivetted double cover butt joint

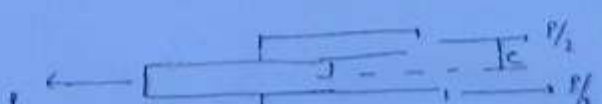


Single rivetted single cover butt joint

Double rivetted single cover butt joint

Note: whenever we concentrate on but joint we concentrate on one side of butt joint of connection.

Eccentricity is eliminated in the case of double cover butt joint.

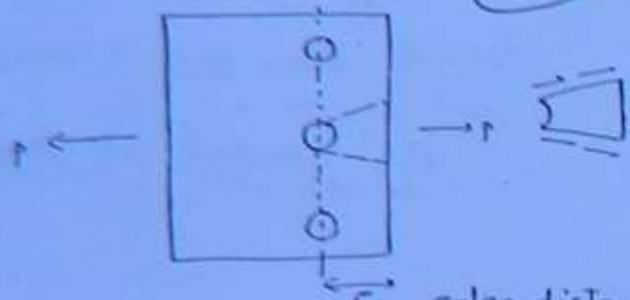


$M = \frac{P}{2}e - \frac{P}{2}e = 0$

Failure of Rivet Joint

1) By tearing of plate b/w rivet hole and edge of plate

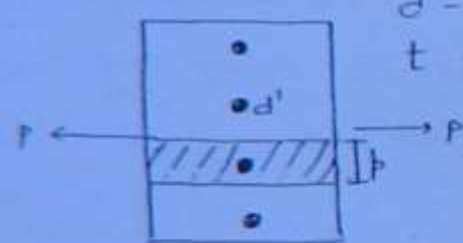
74



It is a type of shearing failure of plate at ends and occurs due to insufficient edge distance. It is prevented by keeping the edge distance to be <u>twice</u> the diameter of rivet hole

2) By tearing of plate b/w rivet.

d' - Dia of hole
t thickness of plate

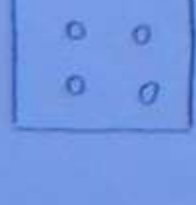


NEA in tension $= (B - 4d')t$

Strength of main plate in tearing $= (B - 4d')t\sigma_{at}$

σ_{at} - permissible stress in axial tension

Strength of main plate / pitch length $= (p - d')t\,\sigma_{at}$



In case of double rivetted butt joint strength of joint per pitch length will be

$(p-d')\,t\,\bar{\sigma}_{at}$

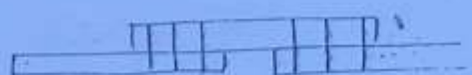
15

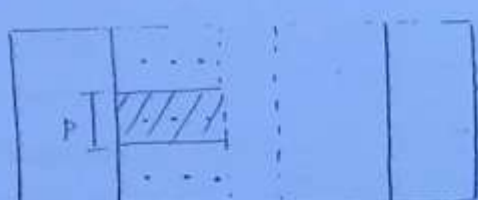
* Shearing failure of joint by shearing of rivets

Strength of joint in shearing is equal to summation of shearing strength of all rivets in the joint

strength of joint in shearing $= \frac{[\pi d']^2}{4} \times \sigma_s \times 3$

$= \frac{3}{4}\sigma_s \pi d'^2$





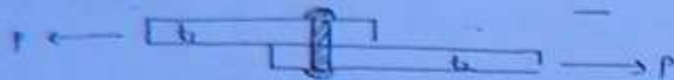
Rivets are in single shear No. of rivet/pitch length = 3

Strength of joint in shearing per pitch length

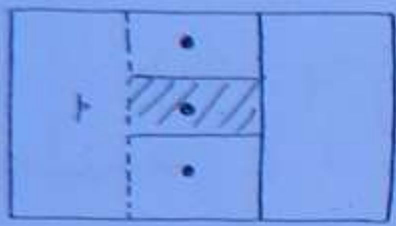
$$= \frac{\pi}{4}(d')^2\,\sigma_s\,\Big] \times 3$$

Strength of triple rivetted double cover butt joint in shearing per pitch length $= \underbrace{2 \times \frac{\pi}{4}(d')^2\,\sigma_s'}_{\text{St. of one rivet in double shear.}} \times 3$

Bearing or crushing of rivet.

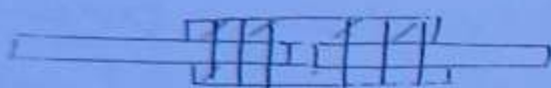


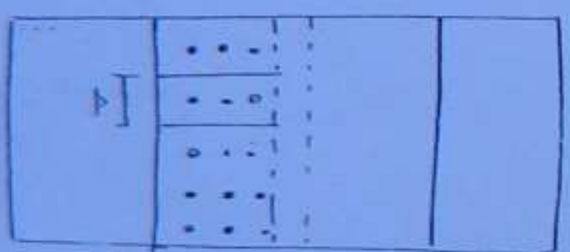
16



Strength of joint per pitch length in bearing = $d' \times t\ \sigma_{br}$

↳ min thickness of plate





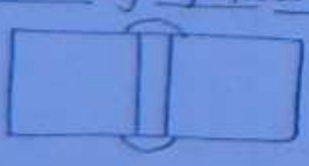
St. of joint per pitch length in bearing

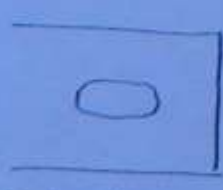
$= d' \times t \times \sigma_{br}) \times 3$

↳ dia of hole

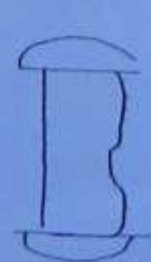
$t \rightarrow$ combined thickness of two cover plate or thickness of main plate whichever is min

* Bearing failure of plate.





bearing failure in ...



Bearing failure of rivet

Bearing failure of plate

Strength of joint is the minimum of shearing, bearing and tearing strength of joint.

Efficiency of Joint

$$\eta = \frac{\text{strength of joint (Min of shearing, bearing, tearing)}}{\text{strength of main plate without deduction for hole}}$$

Note: For efficient utilization of material rivets and plates should fail simultaneously

IS Code Recommendation

1) Permissible stress in rivets

		axial tension	Shearing	bearing
Shop rivet	Power driven	100 MPa	100 MPa	300 MPa
	Hand driven	80 MPa	80 MPa	250 MPa

For field rivetting, the above permissible values are reduced by 10%

Permissible stress values are increased by 25% in case of wind or earthquake loading

(18)

Note-

Wind and or EQ forces act for very short duration for that short duration member force will increase and hence the no. of rivet requirement will increase significantly if permissible stress are kept constant how ever there is a margin between permissible stress and yield stress and if that margin is utilized for that short duration no. of rivet requirement will not increase significantly hence economy will result thus for that short duration FOS is reduced and hence permissible stress is Increased.

NC

Min Pitch-

Min pitch = 2.5 times the nominal dia of rivet

Max pitch -

a) Dis b/w the centres of two consequtive rivets in the dir of stress should not exceed 16t or 200mm whichever is less in tension member.
where t is the thickness of thinner plate

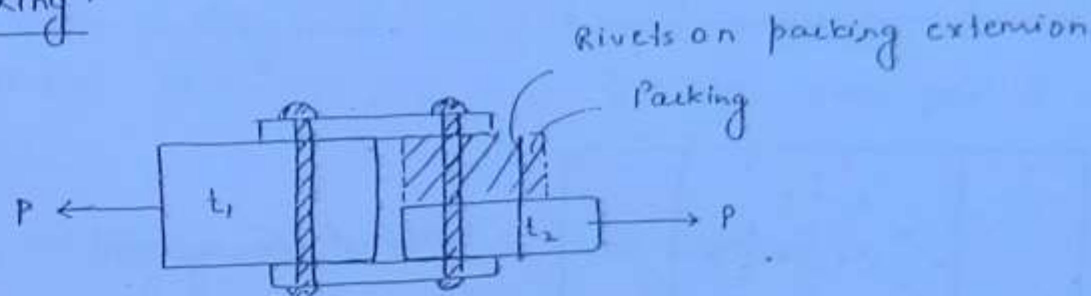
b) In comp. it is

12t or 200 mm
whichever is less

- For tacking rivets its max pitch is 32 t or 300 mm which ever is less.

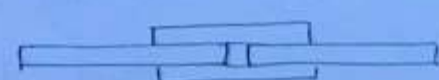
Tacking rivets are used to ensure that the two members being joined along the length behave as a single unit

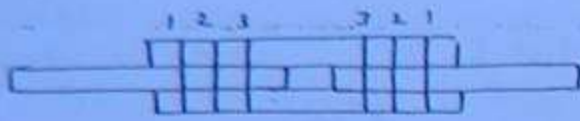
Packing.

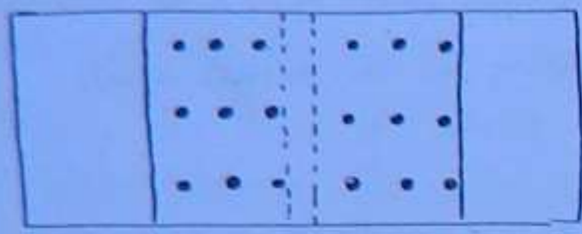


If the difference of thickness b/w two plates being joined ($t_1 - t_2$) is greater than 6 mm. than additional rivets shall be provided on packing extension. No. of additional rivets will be 2.5% of actual no of rivet obtained from normal calculation per 2 mm thickness of packing

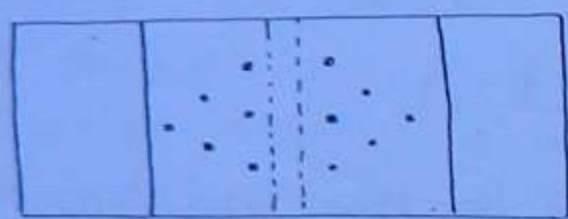
Arrangement of rivets.







Chain Arrangement



Diamond Arrangement

Force in the main plate before (beyond) 1-1 is less than that before 1-1 because rivets at section 1-1 absorb some force and transmits it to the cover plate. The same thing holds for 2-2 and 3-3 therefore main plate required max$_m$ net area at section 1-1 than at other sections. Thus it is desirable to have ~~maxi~~ minimum no. of rivets at section 1-1 providing max NEA. No. of rivets can be increased in inner row that is 2-2 & 3-3. Thus diamond rivetting is better as compare to chain rivetting.

Tearing strength of main plate	Diamond	Chain
1-1	$(B-d')t\sigma_{at} + = P$	$(B-2d)t\sigma_{at} = P$
2-2	$(B-2d')t\sigma_{at} + R_v = P$	$(B-2d)t\sigma_{at} + 2R_v = P$
3-3	$(B-3d')t\sigma_{at} + 3R_v = P$	$(B-2d)t\sigma_{at} + 4R_v = P$

If

$R_v - d' \times t\ \sigma_{at} \leq 0$

secn 2-2 will fail in tearing

2-2 more critical as compare to 1-1

21

At

for chain rivetting most critical section is section 1-1 If plate does not fail at secn 1-1 it will never fail at secn 2-2 & 3-3.

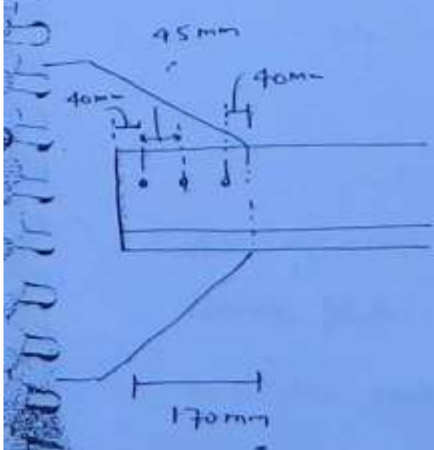
for cover plate secn 3-3 is most critical .

Q. A member of a truss consists of 2 angles 6 mm thick placed back to back. It carries a direct load of 100 kN and is connected to a gusset plate 8 mm thick. Determine the no. of power driven field rivet of 18 mm dia required for the joint:

$d' = 18 + 1.5 = 19.5$

Strength of rivet in shearing $= 2\dfrac{\pi d'^2}{4} \times 90 =$

$= 2 \times \dfrac{\pi\ 19.5^2}{4} \times 90 = 53.757$ kN



in bearing $= 8 \times 19.5 \times 300 \times .9 = \cancel{122.195}$ kN $= 42.12$ ~~38.88~~

rivet value R_v = ~~53.757 kN~~ ~~38.88~~ 42.12 kN

No. of rivet $= \dfrac{100}{\cancel{53.757}\ \cancel{38.88}} = \cancel{2 \text{ rivet}}\ \cancel{2.57}$

$= \dfrac{100}{42.12}$

$= \simeq 3$ rivet

~~≃ 3 rivet~~

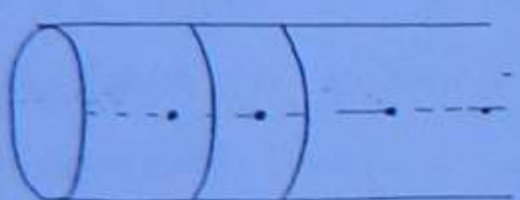
Note :

Shearing strength of rivet $\propto d^2$

Bearing strength of rivet $\propto d$ 22

Hence when shearing governs the design it is better to use small no. of large dia rivets. If however bearing governs the design it is better to use large no. of small dia rivets.

Q: The plate of a boiler 6 mm thick is connected by single rivetted lap joint with 16 mm dia power driven shop driven at 50 mm pitch. Calculate the efficiency of joint.



$d' = 16 + 1.5 = 17.5$

$f_y = 50 \times 6 \times 150 = 45$ KN

Tearing strength of plate $= (50 - 17.5) \times 6 \times 150 = 29.25$ KN

Strength of rivet in shearing $= \dfrac{\pi d'^2}{4} \times 100 =$ ~~24.648 KN~~ 24.053 KN

bearing $= 17.5 \times 6 \times 270 =$ ~~28.95 KN~~ 31.5 KN

$R_v =$ ~~21.648~~ 24.053 KN

$$\eta = \frac{24.053 \times 100}{45} = 53.45\ \%$$

1 \

Q. A tie member has to translate a pull of 400
Design a butt joint to connect two no. of 12mm thick plate and also find it's efficiency use power driven shop rivet.

Sol.

No. of rivets and it's dia

Arrangement of rivets

Width of main and cover plate

Thickness of cover plate

Efficiency.

Dia of rivet $= 6.05\sqrt{12} = 20.95$ mm =

let us adopt 20 mm dia rivet. $\Rightarrow d' = 20+1.5 = 21.5$mm

Rivet value = Min of (shearing strength, Bearing strength)

Shearing strength $= 2 \times \frac{\pi (21.5)^2}{4} \times 100 = 72.61$ KN

Bearing strength =

Assuming that combined thickness of two cover plate is more than main plate

Bearing Strength $= d' \times t \times 300$

$= 77.4$ KN

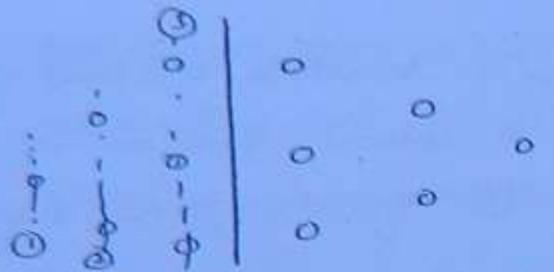
rivet value $= 72.61$ KN

No. of rivets $= \frac{400}{72.61} = 5.5$

Adopt 6 no. of 20 mm ϕ rivet

Arrangement of rivets will decide the dimensions of main & cover plate.

let us adopt diamond rivetting pattern



Width of main plate will be provided such that the main plate does not tear of at any sec^n 1, 2 + 3

Tearing strength of main plate at sec^n 1-1

$$(B-d')\ 12\,mm \times 150 > 400\times10^3$$

$$B-d' > 222.22$$

$$B > 222.22+21.5$$

$$> 243.72\ mm$$

Tearing strength at 2-2

$$(B-2d')\times12\times150+R_v > 400\times10^3$$

$$B > 224.88\ mm$$

at 3-3

$$(B-3d')\times12\times150+3R_v > 400\times10^3$$

$$B > 165.7\ mm$$

Let us adopt width of main plate = 250 mm

Adopting the width of cover plate same as that of main plate

Tearing strength of cover plate at section 3-3

$$(250-3d')\ 2t\ \sigma_{at} > 400$$

$t > \cancel{7.81}\ 7.187$ mm

Recommendation - Min thickness of plate in steel structure should be adopted 6 mm when it is not exposed to atm. and 8 mm if it is exposed to atm.

Let us adopt $t = 8$ mm

25

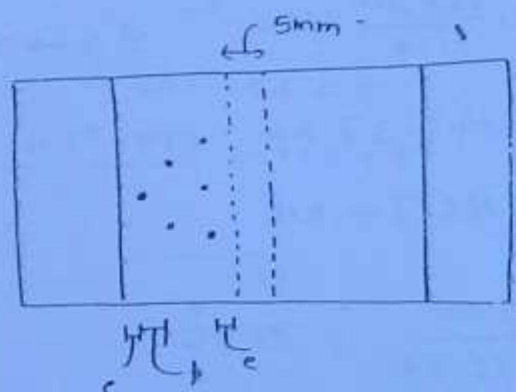
Arrangement — Diamond

No of rivet — 6

Width of main plate - 250 mm

Width of cover " - 250 mm

thickness of cover plate - 8 mm



$e = 2 \times 21.5 = 43$

$p_{min} = 2.5 \times 20 = 50$

$4e + 4p + 5\text{ mm} = 4 \times 43 + 4 \times 50 + 5 = 377$ mm

$$\text{Efficiency} = \frac{\text{Min of shearing, bearing, tearing st. of joint}}{\text{Strength of main plate without deduction for hole}}$$

Min $\begin{cases} \text{Shearing st. of joint} & = 6R_v = 6 \times 72.61 \\ \text{Bearing st. of joint} & = 435.66 \text{ KN} \end{cases}$

Tearing strength of main plate $= (250 - 21.5) \times 12 \times 150 = 411.3$ KN

cover plate $= (250 - 3 \times 21.5) \times 16 \times 150 = 445.2$ KN hence strength of joint = 411.3 KN

$$\eta = \frac{411.3\times10^3\times100}{250\times12\times150} = 91.4\%$$ (26)

Q. Design a rivetted splice for a tie of a steel bridge 20 cm wide 20mm thick carrying an axial load of 50000 kg use 12 mm thick cover plate and 22 mm dia rivet

$\sigma_{at} = 150$ mpa , σ_s = permissible stress in shear = 100 MPa

$\sigma_{br} = 300$ MPa

Sol. $d' = 22+1.5 = 23.5$ m

Shearing

Rivet value $= \min\left(\frac{2\times\pi d'^2}{4}\times100,\ d'\times20\times300\right)$

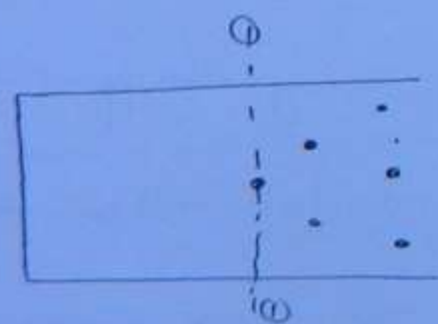
$= 2\times43.37$ KN , 141 KN

Rivet va = 86.74 KN

No. of rivet $= \frac{500}{86.74} = 5.76$

Adopt 6 rivet of 22 mm dia

Arrangement



tearing strength of section 1-1 $= (B-d')\times20\times200\times150$

$= 529.5$ KN > 500 safe

2-2 $= (B-2d')\times20\times150 + 86.74$

$$3\text{-}3 = (B - 3d') \times 20 \times 150 + 3 \times 86.74$$

$$= 648.72 \text{ KN}$$

27

for cover plate (Adopting width of cover plate = width of main plate)

$$= (200 - 3d') \times 24 \times 150$$

$$= \underline{466.2 \text{ KN}} < 500 \text{ KN}$$

So cover plate is not safe in tearing.

After adopting diamond arrangement connection will

sec 3-3

55

$\cancel{2d} = 2.5d = 55$

$2 \times 23.5 = 47$

$55 \times 2 + 47 \times 2 = 204$

So take

45

55

55

45

$e = 1.6$ to 1.7 time d'

So 3 rivets can be accomodated

Hence we can ~~not~~ not apply 3 rivet at sec^n 3-3 b/coz cover plate is not safe

Let us adopt chain rivetting

Check tearing strength of main plate at sec^n 1-1

$$(B - 2d') \times 20 \times 150 = 454 \text{ KN} < 500$$

Not safe

Hence let us adopt

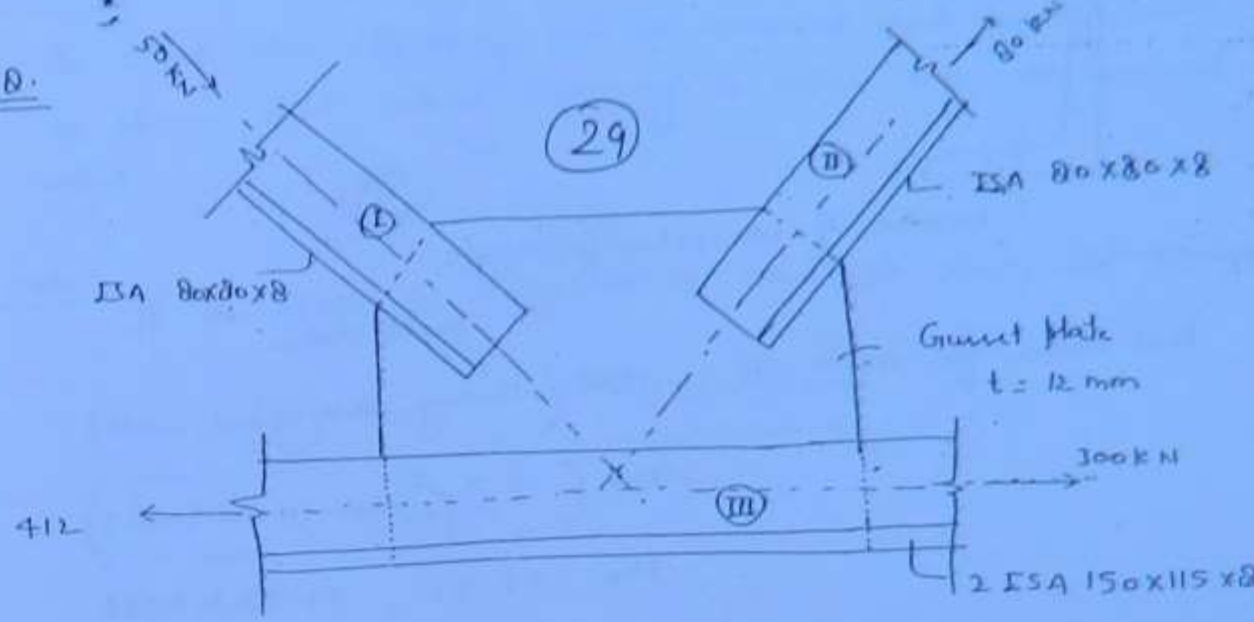
(i) or (ii)

(i)

or

(ii)

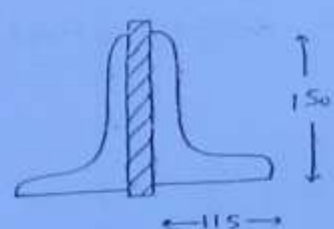
Q.



Determine the no. and pattern of 20 mm dia rivet used for connection

permissible stress in shearing in rivet = 102.5 N/mm²

σ_{br} = 236 N/mm²



for connection of angle I + II rivet is in single shear but for connect of angle III with gusset rivet is in double shear

Hence rivet value for I, II = min (Shearing strength, Bearing st.)

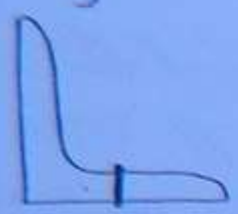
$$= \min\left(\frac{\pi\, 21.5^2}{4} \times 102.5,\ 21.5 \times 8 \times 236\right)$$

$$= \min(37.213\text{ kN},\ 40.59\text{ kN})$$

Rivet value = 37.2 kN

No. of rivet for I $= \frac{50}{37.2} =$ 2 rivet adopt

for II $= \frac{80}{37.2} =$ 3 rivet adopt



Location of connection for angle

Rivet value for angle III $= \text{Min}(\underset{\text{Double shear}}{\text{Shearing}}, \text{Bearing})$

$= \text{Min}\left(2\frac{\pi d'^2}{4} \times 102,\ d' \times t \times 236\right)$

$= \text{Min}\ 74.42,\ 21.5 \times 12 \times 236$

$,\ 60.89$

$= 60.89\ kN$

Net force need's to be transferred to the gusset

$= 412 - 300 = 112\ kN$

No. of rivet $= \frac{112}{60.89}$ = Adopt 2 rivet

For thin shells

For thin shells the connection will be done per pitch length, force in one pitch length will be stress × pitch length × thickness

Stress will be for cylindrical shell

a) hoop stress $= \frac{pd}{2t\eta}$

b) Long. stress $= \frac{pd}{4t\eta}$

$F = \frac{pd}{2t\eta}(s \times t)$

The force for which connection will be designed

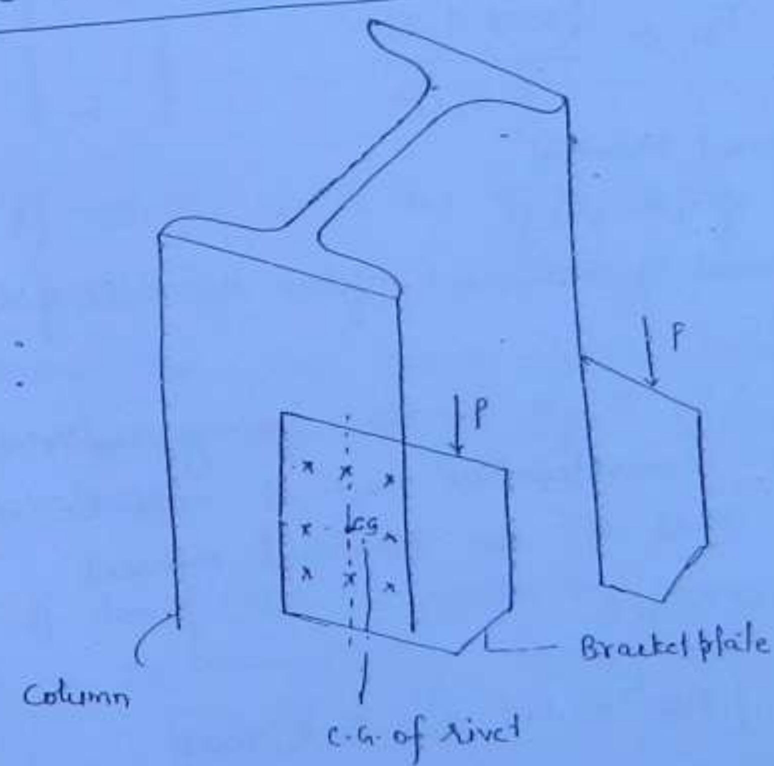
$F = \text{Stress} \times \underset{\text{pitch length}}{s} \times \underbrace{t}_{\text{thickness}}$

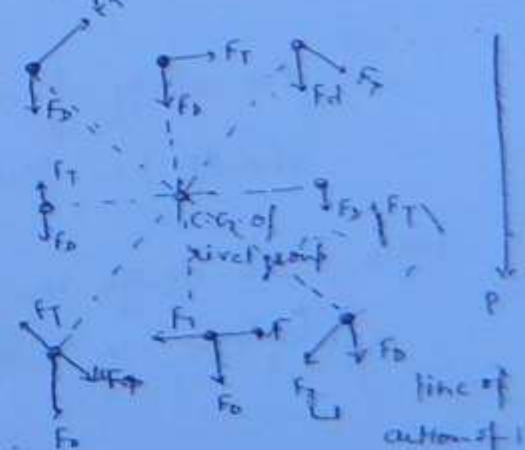
To start with efficiency will be assumed and finally it will be shown that actual efficiency is more than the assumed value.

Note: If the rivet is subjected to combined shear and tension

$$\frac{(\text{Shear stress})_{\text{calc.}}}{\left(\begin{array}{l}\text{Permissible shear stress}\\ \text{in rivet}\end{array}\right)} + \frac{\text{Tensile stress calc.}}{\text{Per.}} \leq 1.4$$

Eccentric Connection -





F_{di} = force on the ith rivet due to direct loading

$$= \frac{P \cdot A_i}{\sum_{i=1}^{n} A_i}$$

under the assumption that direct shear stress is equally shared.

Direction of this force will be parallel to the line of action of force

We know that torsional shear stress $= \frac{T r_i}{J}$

Torsional shear force $= \frac{T r_i}{J} A_i$

$$F_{ti} = \frac{(Pe) A_i r_i}{J}$$

For discrete area $J = \Sigma A_i r_i^2$

$$F_{ti} = \frac{(Pe) A_i r_i}{\Sigma A_i r_i^2}$$

(Pe) → Torsional Moment

A_i → Area of i^{th} rivet

r_i → Distance of ith rivet from the c.G. of rivet group

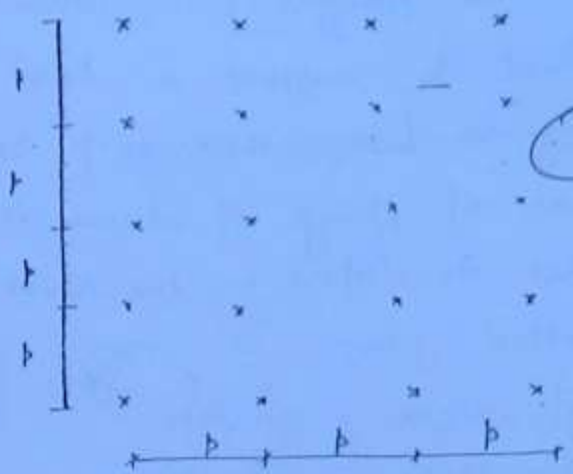
The direction of this S.F. is ⊥ to line joining the c.G. and the rivet under consideration and it will be in the same sense as that of the torsional moment

Resultant force on ith rivet

$$F_{ri} = \sqrt{F_{di}^2 + F_{ti}^2 + 2 F_{di} F_{ti} \cos\theta}$$

For safety of connection resultant force in all rivets should be less than their rivet value.

Note: To design the connection no. of rivets are chosen as follows



33

n - no. of rivets in one column

m - no of such column

$$n = \sqrt{\frac{6(Pe)}{m p R_v}}$$ — Torsional moment

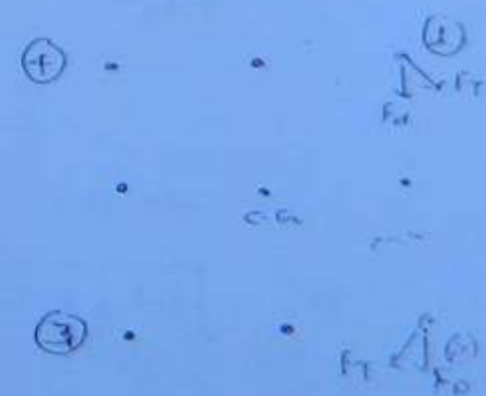
Note

If all the rivets are of same dia then most critical rivit is the one which is farthest from the C.G. If there are more than one rivets equally distant from C.G. then the rivet in which angle b/w F_D & F_t is least is the most critical rivet.

If dia of all the rivets are same

$$F_{di} = \frac{P}{n}$$

$$F_{T_i} = \frac{(Pe) r_i}{\Sigma r_i^2}$$



1, 2, 3 & 4 have same value of F_D and F_T but angle b/w F_D & F_T is smaller in ① & ② hence they are the most critical rivet.

1 & 2 are most critical rivet

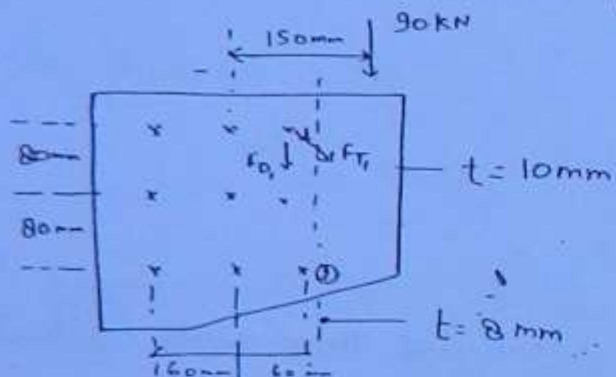
Q. A bracket connected to the flange of a column through a group of rivet to support a load of 90 kN as shown in figure below thickness of bracket plate is 10 mm and that of flange of column is 8 mm determine the max force developed in the rivet and design a suitable rivetted joint.

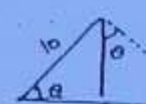
34

Allowable stress in single shear = 100 MPa

in double shear = 200 MPa

σ_{br} = 180 MPa





Assume size of rivets to be Same Hence rivet 1 & 2 will become most critical rivet

Sol. $F_{D_i} = \frac{90}{9} = 10\text{ kN}$

Hence let us calculate force in rivet 1 which is max

$$F_{D_1} = \frac{90}{9} = 10\text{ kN}$$

$$F_{T_1} = \frac{Pe\,r_1}{\Sigma r_i^2} = \frac{90 \times 150 \times 100}{(4 \times 100^2 + 2 \times 80^2 + 2 \times 60^2)}$$

$$= 22.5\text{ kN}$$

$\cos\theta = \frac{3}{5}$

$$F_{r_1} = \sqrt{F_{t_1}^2 + F_{d_1}^2 + 2F_{t_1}F_{d_1}\cos\theta} = 29.6\text{ kN}$$

$$\frac{\pi}{4}(d+1.5)^2 \times 100 \geq 29.6 \times 10^3$$

$$d \geq \cancel{19.41\text{ mm}}\ 17.9\text{ mm}$$

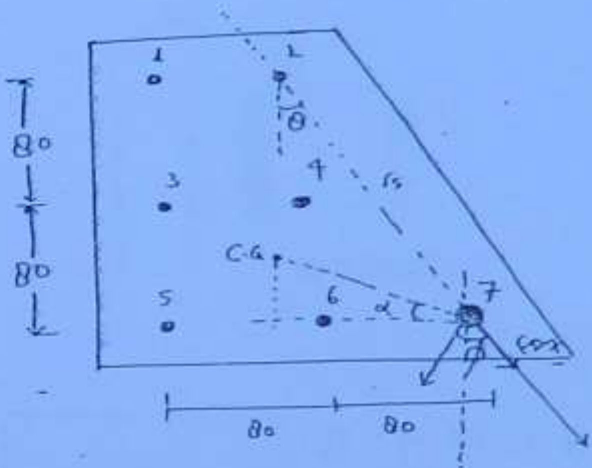
Bearing strength of rivet $= d \times 1.5 \times 8 \times 180 > 29.5 \times 10^3$

$d \geq 19.05$ mm

hence adopt rivet dia $= 20$ mm

35

Q.



Rivet 1 — 6 are of dia d'

& rivet 7 is of dia 1.2d'

Find force in 7^{th} rivet

$\cos\theta = \frac{2}{\sqrt{5}}$

$\sin\theta = \frac{1}{\sqrt{5}}$

$P = 10t = 100$ KN

$\tan\alpha = 2$

$\alpha = 33.69°$

$\beta = 26.565$

$\phi = (\alpha + \beta) = 60.255°$

$$\bar{x} = \frac{A_1 x_1 + A_2 x_2 + A_3 x_3 + \cdots A_n x_n}{A_1 + A_2 + \cdots A_n}$$

$$= \frac{0 + 0 + 0 + 3 \times A \times 80 + 1.44 A \times 160}{7.44 A}$$

$= 63.226$ mm

$$\bar{Y} = \frac{0 + 0 + 0 + 2A \times 80 + 2A \times 160}{7.44 A}$$

$= 64.516$ mm

$$F_{D_7} = \frac{100}{7.44 A} \times 1.44 A = 19.355 \text{ KN}$$

$(Pe) = 100 \cos\theta \, (80 + 80 - 63.226) + \frac{100}{\sqrt{5}} (64.516)$

$q = 115.41$ mm

$Pe = \cancel{1154\ldots}$

$c = 57.7$ mm

$Pe = 5770$ KN mm $= \underline{5.77 \text{ KNm}}$

$r_1 = 114.51\ mm$

$r_2 = 96.94\ mm$

$r_3 = 65.08\ mm$

$r_4 = 22.83\ mm$ (36)

$r_5 = 90.33\ mm$

$r_6 = 56.67\ mm$

$r_7 = 110.31\ m$

$$\frac{(Pe)\ A_7 r_7}{\Sigma A_i r_i^2}$$

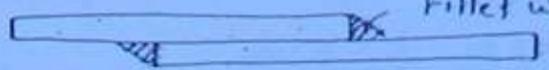
$$= \frac{(Pe)\ \cancel{A(r_1 + \cdots r_6)} + 1.44 A \times r_7}{A(r_1^2 + \cdots r_6^2) + 1.44 A \cdot r_7^2}$$

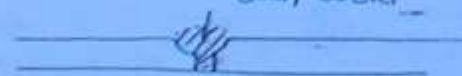
$F_{T_7} = 16.28\ KN$

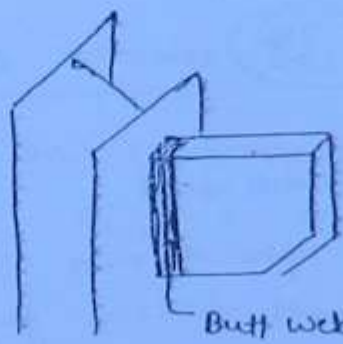
$$F_{R_7} = \sqrt{F_{d_7}^2 + F_{T_7}^2 + 2 F_d F_T \cos(60.25)}$$

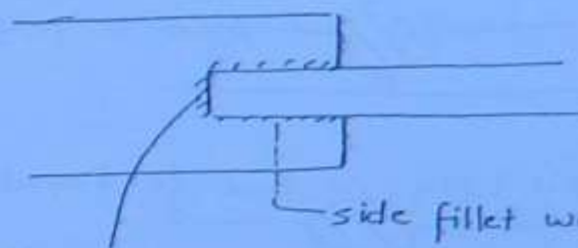
$= 30.86\ KN$

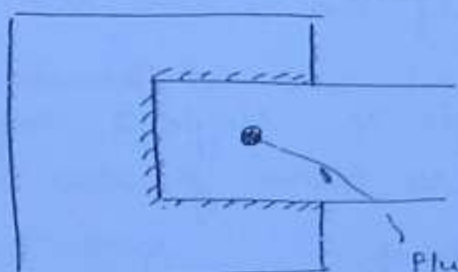
Welded Connection

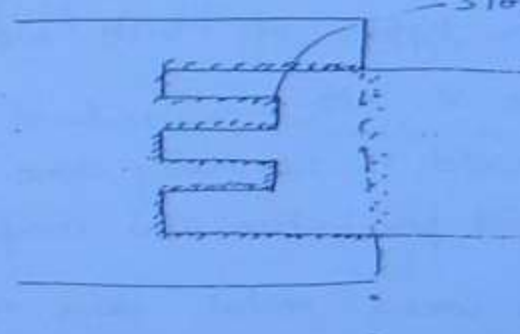












Types of weld.

i) Butt weld or grove weld
ii) Fillet weld
iii) Plug weld
iv) slot weld

Welding is done by electric arc welding

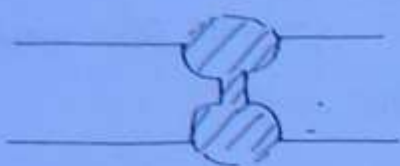
Butt weld -

The various types of butt welds are square butt weld

Adopted upto $t \le 8mm$

Other types of weld are (38)

single U butt weld
Normally upto 40 mm



Double U - butt weld
$> 40mm$

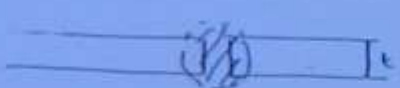
Other types of butt weld are single V. single J single double V. etc.

Butt weld is normally done in the workshop.

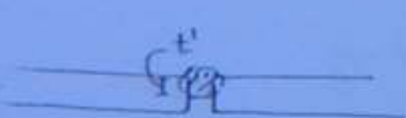
Partial penetration and complete penetration.

When weld metal does not penetrate to the complete depth of the plate.

Weld metal penetrates completely to the full ~~length~~ depth is known as complete penetration.



complete penetration $P_{max} = B \times t \times \sigma_{at}$



Partial or incomplete penetration

$P_{max} = B \times t' \times \sigma_{at}$

If the thickness of penetration is not given t' can be taken $\frac{5}{8}$ times of t_{min} but It is not IS code recommendation.

Fillet weld.

Can be done in the field as lesser precision is required in it. It is cheaper as compared to butt weld.

* Fillet weld is almost always assumed to fail in shear.

(39)

Permissible stresses.

Tension and compression on section through the throat of butt weld

$$\sigma_{perm} = 0.6 f_y$$

Shear of section through the throat of butt or fillet weld.

$$= 0.4 f_y$$ (Normally it is 108 to 110 N/mm^2)

These values are for welding done in workshop they are reduced to 80% in case of field welding.

In case of wind or EQ the permissible values are increased by 25%

Design of butt weld.

- Butt weld or groove weld is more suitable for alternating stresses provided that full penetration of weld is achieved.
- Reinforcement is good for static load condition. It is not suitable for alternating stress condition due to huge stress conc. at this location leading to onset of crack.

However even in this case reinforcement of 0.75 to 3mm is kept.

- The strength of weld in tension and compression is given by

$l_{eff} \times t_{eff} \times 0.6 f_y$

↳ Actual length of weld - Length of full size of weld

t_{eff} - Thickness of thinner part being joined (because of complete penetration)

For incomplete penetration t_{eff} is min thickness of weld metal common to the parts being joined excluding reinforcement.

- When two plates are of different thickness

40

t_1 - thicker

If $(t_1 - t_2) > \frac{t_2}{4}$ or 3mm whichever is greater then we provide tapering in the thickness. The tapering should not be greater than 1 in 5.

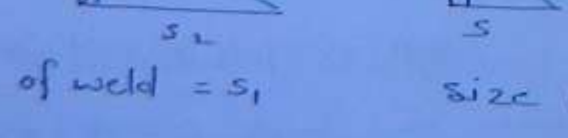
If the thickness of weld common b/w the plates is not given in incomplete penetration weld t_{eff} can be taken as $\frac{5}{8}$ times t_{min}.

Design of Fillet weld.

Size of the weld

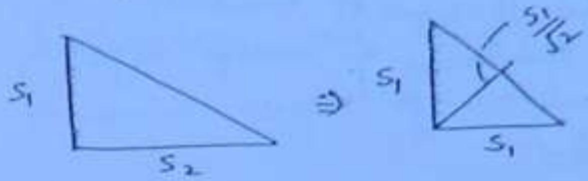
size of the weld is decided on the basis of largest right angle that can be inscribed in the weld for normal fillet weld size is taken as the minimum weld leg size

Note



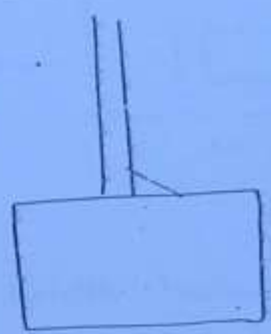
size of weld $= s_1$ size of weld $= s$

For right angle fillet weld minimum throat size will be $\frac{S_1 S_2}{\sqrt{S_1^2 + S_2^2}}$. However as per IS code it is taken as $\left(\frac{S}{\sqrt{2}}\right)$



41

Minimum size of weld.



Thickness of thicker part	min size
0 — 10	3 mm
10 — 20	5 mm
20 — 32	6 mm
32 - 50	8 mm first run, 10 mm final

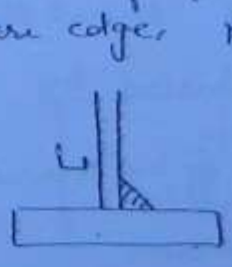
For thickness > 50mm Special technique like pre heating is required. Preheating it is done so that plate does not extract heat from the weld.

Min size however should not be more than thickness of thinner plate being joined

If the weld size is small it will cool faster due to heat extracted by the thicker plate. hence Rapid cooling leads to brittleness in the weld hence there should be min size of the weld controlled by the thickness of thicker plate.

Max size of weld. -

For square edge, Max size = thickness of thinner plate being joined - 1.5 mm



Square edge

For round edge

$$\text{Max size of weld} = \frac{3}{4} \text{ nominal thickness of round edge}$$

42

Note

From economy point of view it is better to use smaller size of large length of weld.
and longer

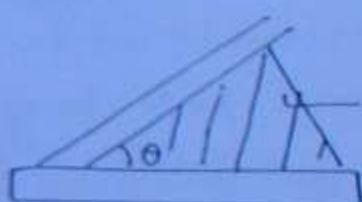
$$\text{quantity of steel} = \frac{1}{2} \times s \times s \times L$$

$$\propto Ls^2$$

$$\text{\& Force} \propto l \times s$$

Throat of fillet weld.

Effective throat thickness is given by = k x size of weld



$= ks$

$$\text{Eff. throat thickness} = k(\text{size of weld})$$
$$= ks$$

θ	60 - 90	91 - 100	101 - 106	107 - 113	114 - 120
k	~~0.8~~ 0.7	0.65	0.6	0.55	0.5

Area resisting shear in fillet weld is $l \times t$
where l is the length of weld t is the eff. throat thickness hence force resisted by fillet weld

$$l \times t \times 0.4 f_y$$

↳ Permissible stress in shear

↳ throat thickness

Effective length of fillet weld.

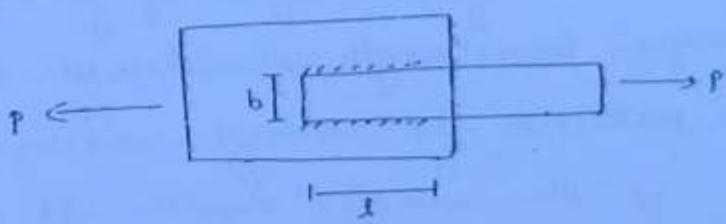
$$L_{eff} = L_{overall} - 2s$$

$s \rightarrow$ size of weld

- In drawings the length shown is the effective length.
- Minimum length of weld $= 4s$ (for weld to be 100% effective in load transfer)

(43)

Side Fillet weld.



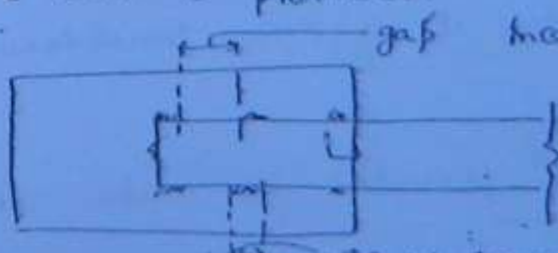
$l \not< b$

As width increases non-uniformity in stress in the plate increases to make it more uniform plug or slot weld can be used

Stress variation on the plate

Intermittent Fillet weld.

When the length of smallest size weld required to transmit in load is less than the continuous length of joint. Intermittent fillet weld is provided.

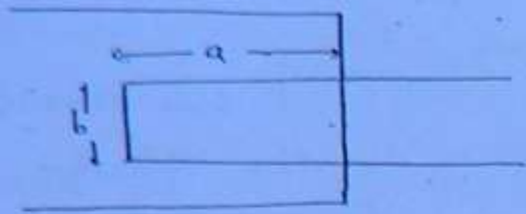


gap

In case of tension → max gap 16t or 200 mm whichever is smaller

t → thickness of thinner plate

4t or [illegible] whichever is greater.



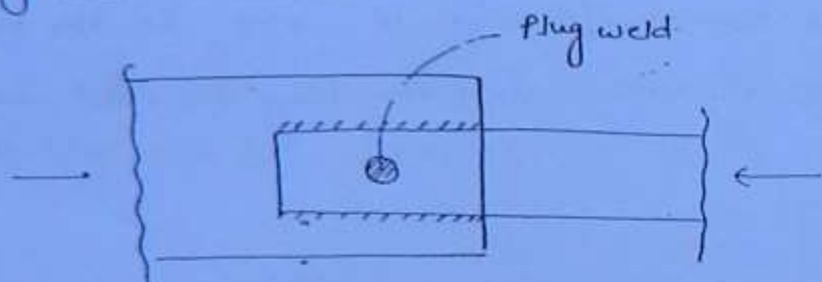
$P = l \times t \times .4 f_y$ ⇒ find l if $l < (2a+b)$

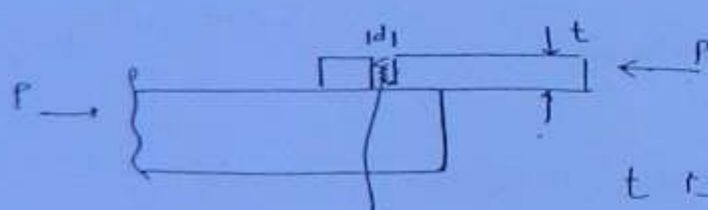
↳ choose min

we provide intermittant fillet weld.

The strength of end fillet weld is generally larger than that of side fillet weld however for strength calculation we take the strengths of the two welds to be same.

Plug and slot weld.





$d \nless 3t$ or 25 mm which ever greater

t is the thickness of plate in which slot is made.

Effective connection b/w top & bottom plate

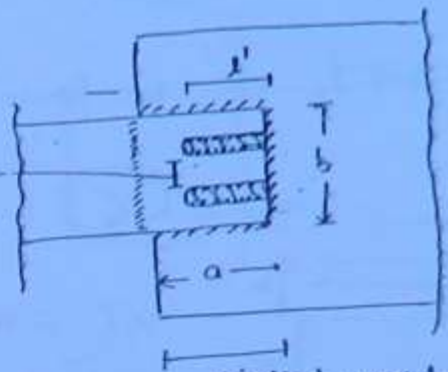
Plug weld is capable of resisting shear however as per codal recommendation it is not considered to be contributing to the strength.

- Plug weld is also provided to make stress variation in top plate more uniform

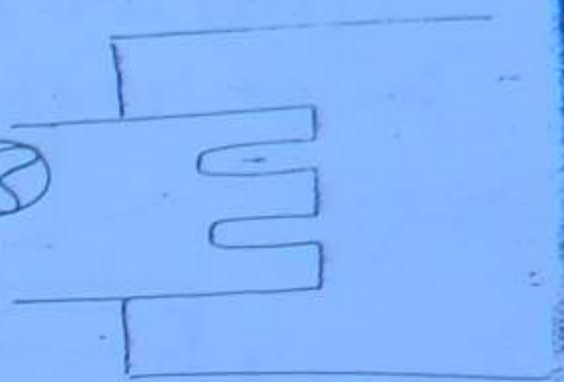
45

slot weld -

gap b/w slots should be 2t



Limited overlap



If the overlap is limited and even by providing the largest size of weld all along the available length, if P force can not be resisted than we provide additional length of connection by making slot. - The additional length of connection in the above figure will be 4 l'

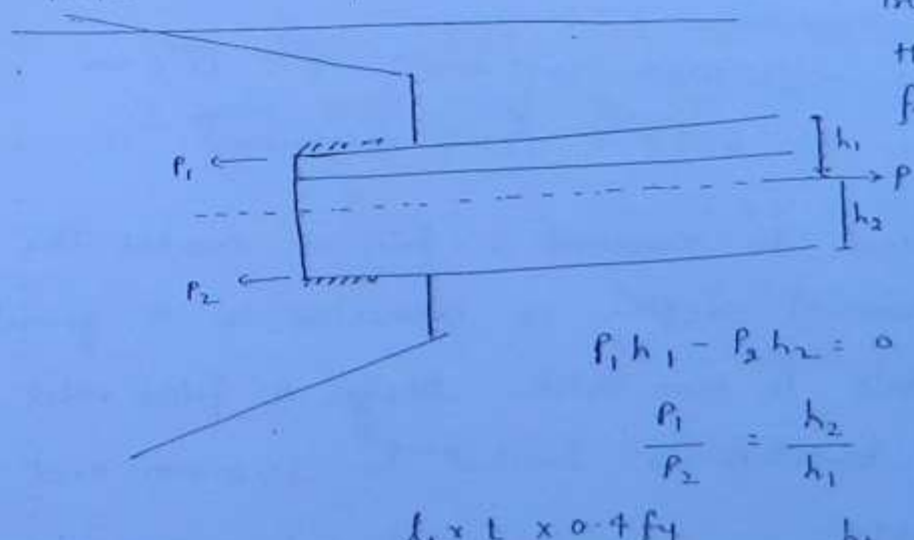
width of slot $\not<$ 3t or 25 mm whichever is greater

r $\not<$ 1.5t or 12 mm " "

Note Min overlap in lap joint $\not<$ 4t or 40 mm whichever is more

Fillet weld for truss member

Truss joint should be such that it should be moment free



$$P_1 h_1 - P_2 h_2 = 0$$

$$\frac{P_1}{P_2} = \frac{h_2}{h_1}$$

$$\frac{l_1 \times t \times 0.4 f_y}{l_2 \times t \times 0.4 f_y} = \frac{h_2}{h_1}$$

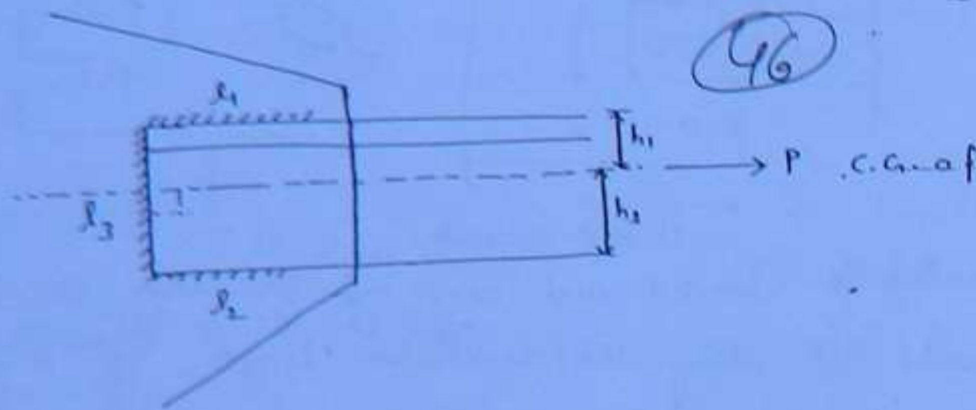
$$\frac{l_1}{l_2} = \frac{h_2}{h_2} \quad \text{—— (1)}$$

$$(l_1 + l_2) \times t \times 0.4\, f_y = P \quad \text{——— (ii)}$$

from ~~left~~ (i) & (ii) l_1 & l_2 can be found out

Case-II

(46)



$$P = (l_1 + l_2 + l_3) \times t \times 0.4\, f_y \quad \text{——— (i)}$$

t, Throat thickness

$$P_1 h_1 - P_2 h_2 - P_3 \left\{ \frac{(h_1 + h_2)}{2} - h_1 \right\} = 0$$

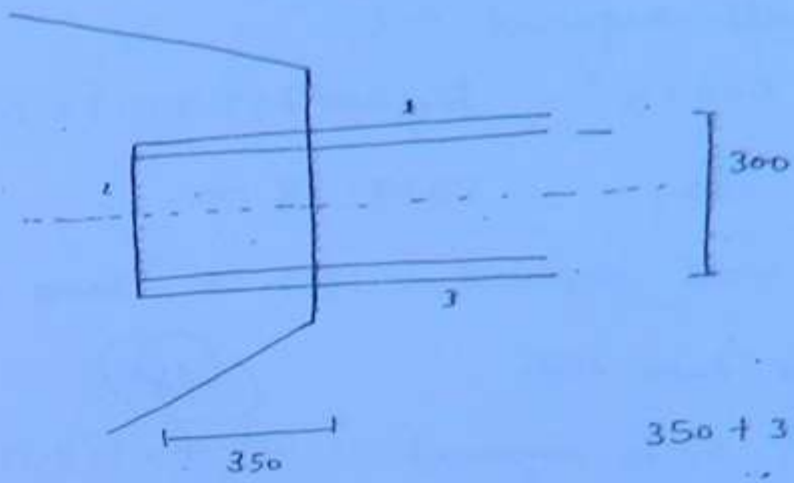
$$P_1 = l_1 \times t \times 0.4 f_y$$

$$P_2 = l_2 \times t \times 0.4 f_y$$

$$P_3 = l_3 \times t \times 0.4 f_y$$

Indian standard light channel.

Q An ISLC 300 with area 4211 mm^2, T = 11.6 mm (flange thickness), t = 6.7 mm (web thickness) is used to transmit a pull of 600 kN. The channel section is connected to a gusset plate 10 mm thick. Design a fillet weld if overlap is limited to 350 mm and welding can be provided only on 3 sides. Use slot weld if required.



47

$350 + 350 + 300 = 1000\,m$

Max size of fillet weld on side 1 = Thickness of thinner part $-1.5\,mm$ (sq. edge)

$= 10 - 1.5 = 8.5\,mm$

Min size $= 5\,mm$

Max size of fillet on side 2 $= 6.7 - 1.5 = 5.2\,mm$

min $= 3\,mm$

If uniform size of weld is to be provided we will adopt 5 mm size

Max force that can be resisted by providing weld on 3 side and adopting the max. possible size.

$P_{max\ possible} = 350 \times 2 \times t_1 \times 0.4 f_y + 300 \times t_2 \times 0.4 f_y$

$t_1 = 0.7 s_1 = 0.7 \times 8.5$

$t_2 = 0.7 \times 5.2$

$P_{max} = 525.7\,kN < 600\,kN$

Hence we need to provide slot welding

Hence we will provide slots by adopting a uniform size of weld = 5 mm length of weld required = l

$$600 \times 10^3 = l_{req} \times 0.7 \times 5 \times 0.4 \times 250$$

$$l_{req} = 1714.28 \text{ mm}$$

Length of slot weld required $= 1714.28 - 1000 = 714.28$ mm

proving two slots

length of slots required $= \frac{714.28}{4} = 178.75$

Adopt 180 mm

Width of slot = 3t or 25 mm whichever is more

$= 3 \times 6.7$ or 25

$= 20.1$ or 25

take $b = 25$ mm

Q. In an industrial shed and edge support consisting of two angles 110 x 110 mm is to be connected to 16 mm gusset plate for a tensile load of 650 KN Design the moment free welded connection.

[illegible] back to back [illegible]

[illegible] Area of [illegible] $= [illegible]$

$= \left(110 - \frac{t}{2}\right) t$

$(220 - t) t$

Force resisted by [illegible] NEA

$(220 - t) t \times$ [illegible] $\geq$ [illegible]

$220t - t^2 - 2166$ [illegible] $= 0$

$t > 10.33$ mm

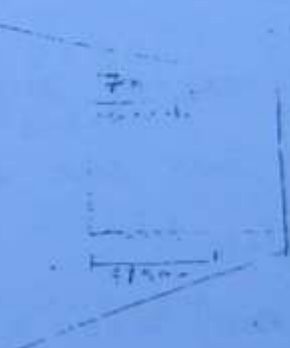
take $t = 12$ mm

$S_{max} = \frac{3}{4}t$ or $t - 1.5$

$= \frac{3}{4} \times 14 = 10.5$

$S_{max} = 10$ mm

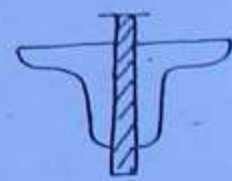
49



$l = 166.71$ — 170 mm

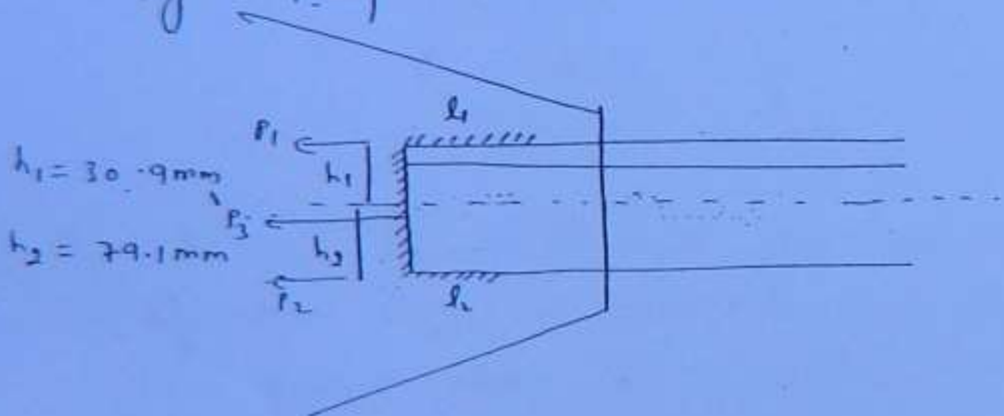
$l = 413.63$ — 415 mm

The best arrangement for transfer of ----
loading to the gusset plate will be when two angles are connected on opp. side of gusset plate.



Hence each angle will carry 325 KN load

Hence we have to decide length of weld, size of weld and arrangement of weld.



As the thickness of angle is not given we will proceed on the basis of min size of weld decided on the basis of thickness of gusset plate

Min size = 5 mm adopted

$$\text{length of weld req.} = \frac{325000}{5 \times 0.7 \times 100} = 928.57\,mm$$

Note: i) Max thickness available for angle

ii) Max. size available for angle

a) equal angle

b) Unequal angle

iii) Max depth of I sec available

(IV) Gauge location for I-secn

(V) Min thickness of plate

VI) Max thickness of plate

$$\text{strength of weld} = 0.7 \times 5 \times 100 = \overline{\underset{}{350\ N/mm}}$$

NC

$l_1 + l_2 = 928.57 - 110 = 818.57\ mm$ —— ①

$P_1 = l_1 \times .75 \times .4 f_y$

$P_2 = l_2 \times 0.75 \times 0.4 f_y$, $P_3 = l_3 \times 0.75 \times 0.4 f_y$

$= 110 \times .7 \times 5 \times 100 = 38.5\ KN$

For moment free connection

$$P_1 \overset{30.9}{h_1'} - P_2 \overset{79.1}{h_2'} - P_3 \overset{24.1}{\left(\frac{h_1 + h_2}{2} - h_1\right)} + 0 = 0$$

$P_1 + P_2 = 325 - 38.5 = 286.5$

$\Sigma M = 0$ about centroidal axis

$$350\, l_1 \times 30.9 - 350\, l_2 \times 79.1 - 38.5 \times 10^3 \times 24.1 = 0$$

~~$l_1 - l_2 =$~~

$30.9\, l_1 - 79.1\, l_2 = 2651$

$l_1 + l_2 = 818.57$

$l_1 = 612.72 = 615\ mm$

$l_2 = 205.84 \simeq 210\ mm$

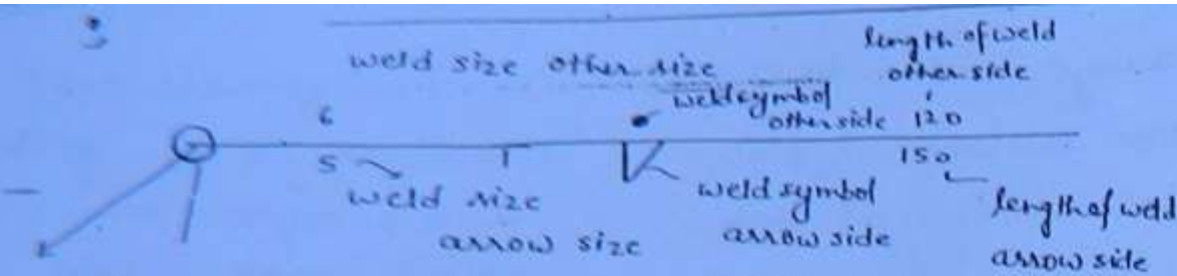
Weld Notations

◺ Fillet weld

|| square butt weld

V single V

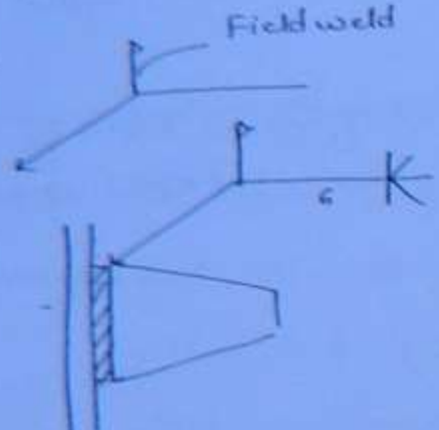
X Double V



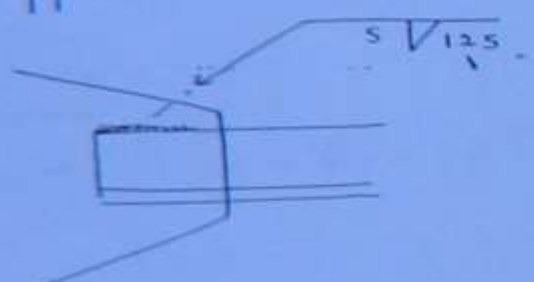
All round welding

can be fillet or butt weld as the case may be.

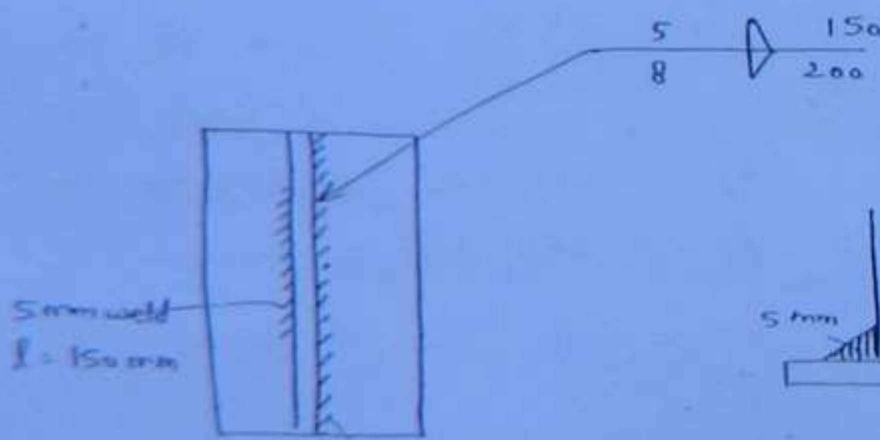
52

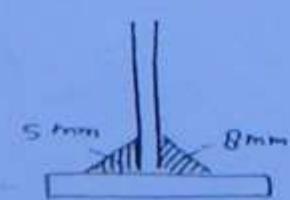


Field butt weld of throat thickness = 6 mm and weld is double bevel.

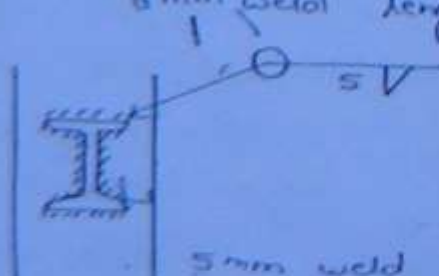


5 mm shop fillet weld of length 125 mm

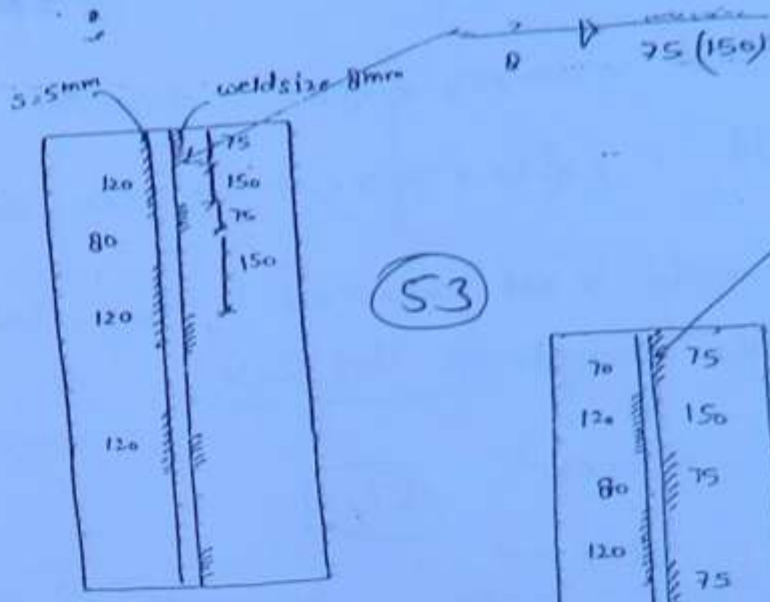




8 mm weld length 200 mm

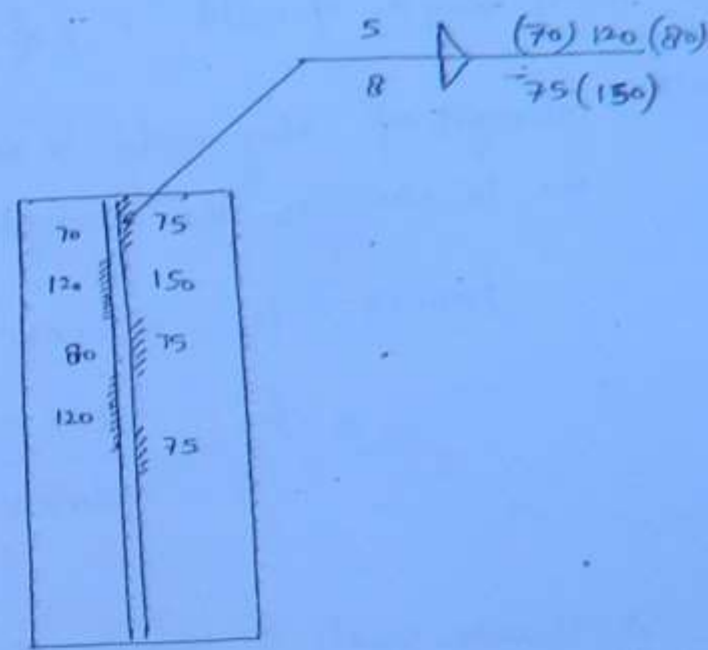


5mm weld shop weld.

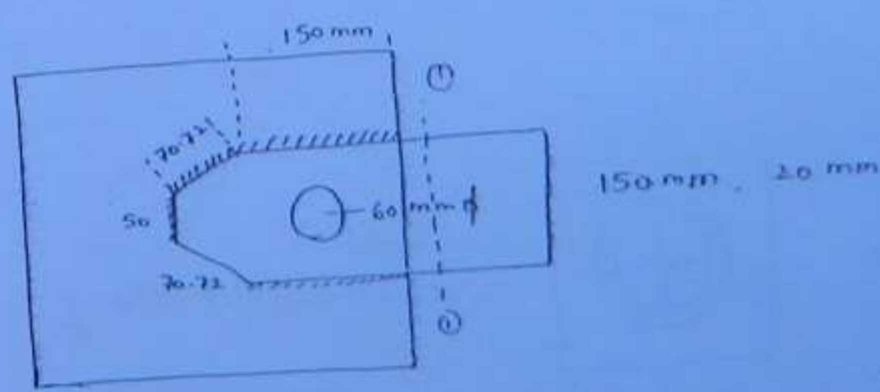


Shop welding

53



Q. A plate of 150 mm width and 20 mm thickness is welded to another plate by fillet weld as shown in figure the size of weld is 12 mm through out compute the av. shear stress produced in the weld for the full strength of plate if the allowable stress is 150 N/mm² in axial tension.



Note: As in this case the location of slot is not given the strength of plate can not be decided hence we will work on full strength of plate at sec 1-1

Full strength of plate $= 150 \times 20 \times 150 = 450\,kN$

total length of weld $= 2 \times (150 + 70.72) + 50 + \pi \times 60 = 680\,mm$

Note: strength of plug weld is not considered in design but we consider the strength of slot weld.

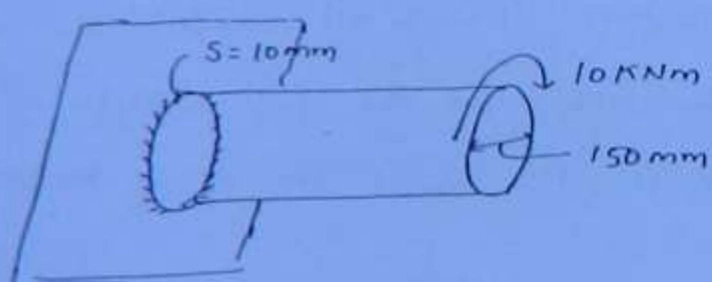
$$l \times 0.75 \times f_s = 450 \times 10^3$$

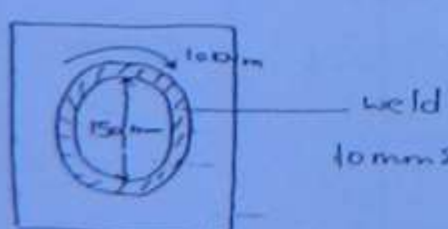
54

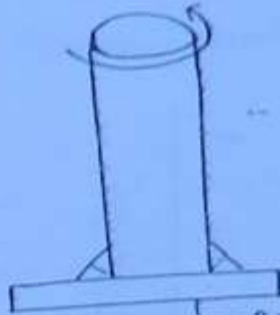
$$f_s = \frac{450 \times 10^3}{680 \times 0.7 \times 12} = 78.78\ N/mm^2$$

Ex 09

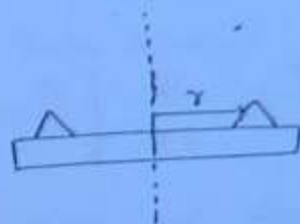
A circular shaft of diameter 150 mm is welded to a a rigid plate by an external all round fillet weld of size 10 mm if a torque of 10 KNm is applied to the shaft find the maximum stress in the weld.







55



Shear stress $= \frac{Tr}{J} \rightarrow$ Polar moment of Inertia of resisting section

Exact Calc. of J

$$\int_0^{.75} 2\pi r\, dx\, r^2$$

$r_0 = 75$ mm

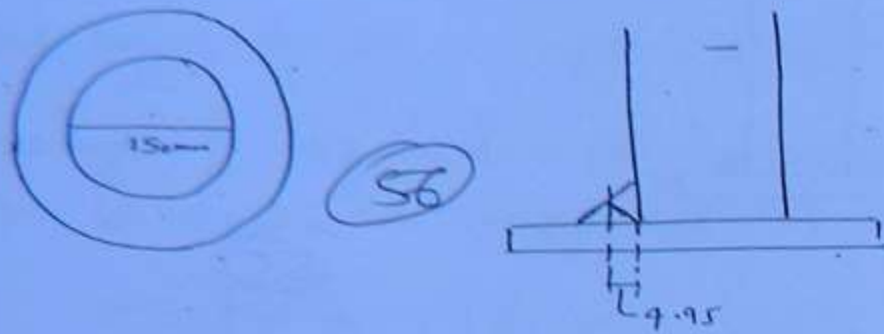
$r = r_0 + \frac{x}{\sqrt{2}}$

$$J = \int_0^{.7\times10} 2\pi \left(r_0 + \frac{x}{\sqrt{2}}\right)^3 dx = 20.474 \times 10^6\ mm^4$$

$r_{max} = r_0 + \frac{0.7\times10}{\sqrt{2}} = 79.949$ mm

Max. shear stress $= \frac{10\times10^6 \times 79.95}{20.474\times10^6} = 39.05\ N/mm^2$

NB: Approximate calc.



$$J = \frac{\pi}{32}\left[(150 + 2\times 4.9)^4 - 150^4\right]$$

$$= 14.3177\times 10^6\ mm^4$$ (x) quite less as compared to actual value.

$$\tau = \frac{Tr}{J} = \frac{10\times 10^6 \times 79.95}{14.31\times 10^6} = 55.83\ N/mm^2$$

$$J = \frac{\pi}{32}\left[(150+14)^4 - 150^4\right] = 21.318\times 10^6\ mm^4$$

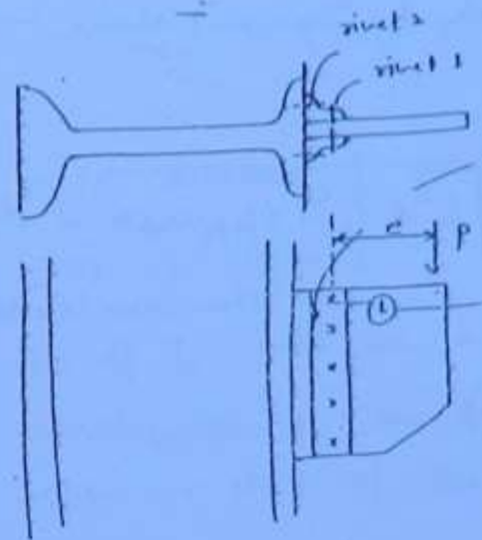
Approximate calc. is quite different from the actual value a logical calculation will be when the projected width is taken as the actual width of resisting section and it is assumed that the stress is uniform over the cross section of throat

τ_0 · Shear stress on throat assuming constant

$$\tau_0 \times 2\pi r \times 0.75 \times r = 10$$

$$\tau_0 = \frac{10\times 10^6}{2\pi\left(80 + \frac{7.5}{\sqrt{2}}\right)^2 0.75} = 35.57\ N/mm^2$$

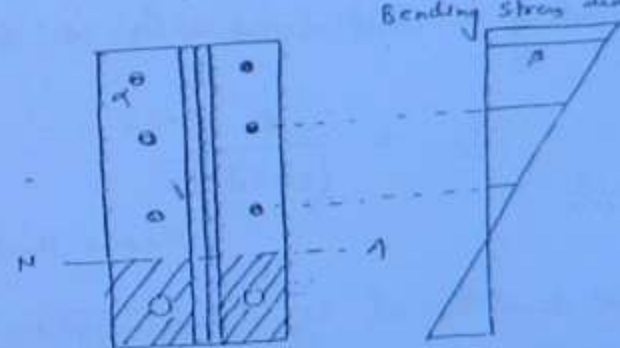
Eccentric connection using weld.



Rivet ② is subjected to shear and axial tension

subjected to direct shear + Torsional moment

Tension in a rivet will be corresponding to β value in Bending stress diagram



Bending stress variation

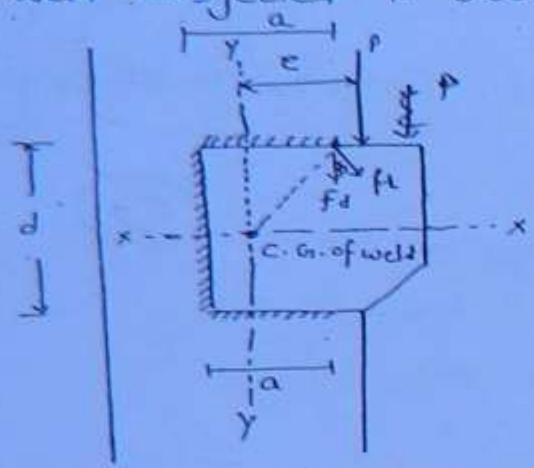
In rivet 2 check for safety is done as follows

$$\frac{\text{Calculated axial force}}{\text{Permissible axial force}} + \frac{\text{Calc. shear force}}{\text{Perm. shear force}} \leq 1.4$$

Eccentric connection using welding.

Case. 1

Weld subjected to direct shear + Torsional shear —



$$f_r = \sqrt{f_d^2 + f_t^2 + 2 f_d f_t \cos\theta} \le p_{er}.$$

Shear stress in welds

(58)

Size of weld = s

As welding is continuous area we will work in terms of stress

$$f_d = \frac{P}{(2a+d)t}$$

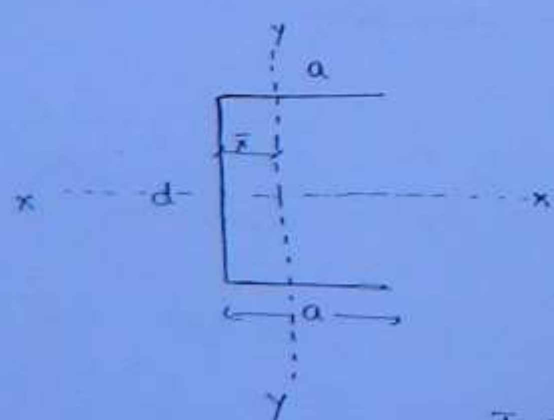
Throat thickness

$$f_d = \frac{P}{(2a+d)(0.7s)}$$

. direct shear on throat of weld.

Direction of f_d is taken along the line of action of force P

$$f_t = \frac{T r}{J}$$



Weld will be considered as a line area as shown above. Thickness of the line will be assumed as throat thickness

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

$$I_{xx} = \frac{td^3}{12} + (a\times t)\left(\frac{d}{2}\right)^2 \times 2$$

$$I_{yy} = (t\times d)\,\bar{x}^2 + \left[\frac{ta^3}{12} + at\left(\frac{a}{2}-\bar{x}\right)^2\right]2$$

r is distance of point under consideration from c.G. of weld

e is $\perp$ distance of line of action of P from the c.G. of weld.

Dir. of f_t will be $\perp$ to the line joining point under consideration to the C.G. of weld and it will be in the same sense as that of the torsional moment.

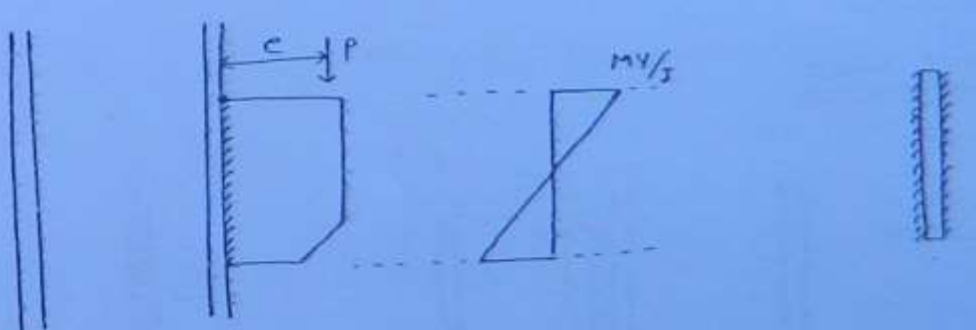
Resultant shear stress

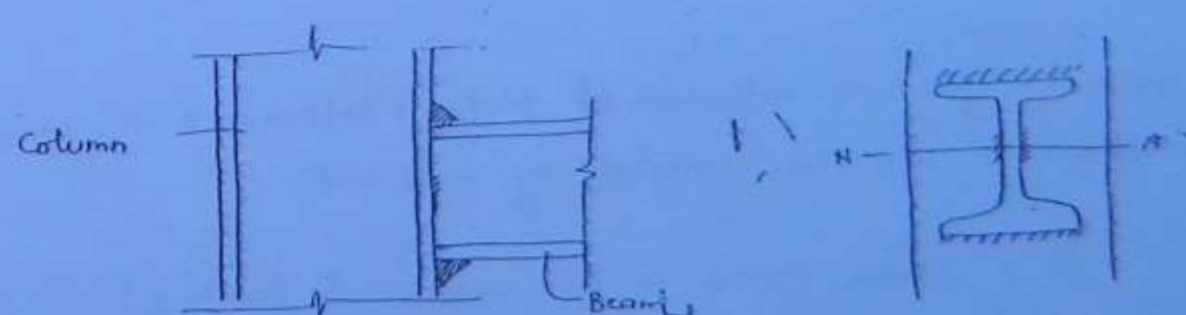
$$f_r = \sqrt{f_d^2 + f_t^2 + 2 f_d f_t \cos\theta}$$

for safety

Case-II

Fillet weld subjected to direct load and bending.





∴ Direct shear stress

vertical $f_v = \frac{P}{2dt}$

60 $= \frac{P}{2d(0.7S)}$

At any point along the length of weld the bending stress will be resisted by the fillet weld through shearing action on it's throat hence horizontal shear stress in fillet weld (f_H)

$$f_H = \left(\frac{My}{I_{xx}}\right)$$

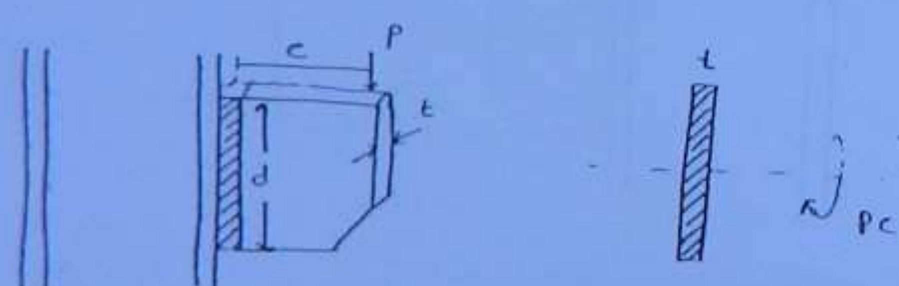
$$I_{xx} = 2\left(\frac{t d^3}{12}\right) = \frac{2(0.7S)d^3}{12}$$

$$f_r = \sqrt{f_H^2 + f_v^2}$$

for safety of connection $f_r <$ permissible shear stress in weld.

~~Case~~ Case. III

Butt Weld



Butt weld is just an extension of plate whatever is the stress in the plate, it will be resisted by butt weld

f_s = shear stress in butt weld $= \frac{P}{td}$ (vertical)

f_b = bending stress in butt weld $= \dfrac{Pe\, y_{max}}{I_{xx}} = \dfrac{Pe\,(d/2)}{\frac{td^3}{12}}$

Checking safety using interaction formula

$$\left(\frac{f_s}{\text{per. shear stress in butt weld}}\right)^2 + \left(\frac{f_b}{\text{perm. bending stress in butt weld}}\right)^2 \leq 1.0$$

Equivalency Method -

$$\sqrt{f_b^2 + 3 f_s^2} \leq 0.9 f_y$$

based on max distortion energy theorem

Permissible bending stress for flanged section $= 165\ N/mm^2$
$= (0.67 f_y)$

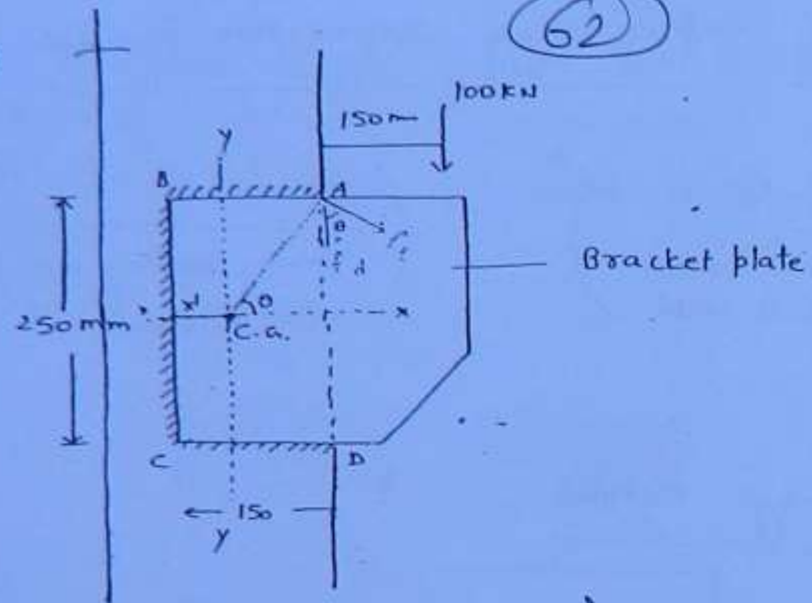
for solid sections () permissible

bending stress is $185\ N/mm^2$

Q. A welded bracket ~~connects the a~~ plate is connected to a column flange as shown in the fig. below determine the size of weld if the allowable stress in the weld is 110 N/mm^2

(62)

ISHB 300



Sol

$$x' = \frac{A_1 x_1 + A_2 x_2}{A_1 + A_2 + A_3}$$

$$= \frac{2(150 \times t \times 75) + 0}{550 \times t} = 40.91 \text{ mm}$$

Maximum stress will be either at A or at D

At A $\quad f_d = \frac{100 \times 10^3}{550 \times t} = \frac{181.818}{t} \; N/mm^2$

$$f_t = \frac{Tr}{J} = \frac{T r_A}{I_{xx} + I_{yy}}$$

$$I_{xx} = 2(150 \times t \times 125^2) + \frac{t\, 250^3}{12} = 5.9896 \times 10^6 \, t \text{ mm}^4$$

$$I_{yy} = 250\, t\, x'^2 + \left[\frac{150\, t^3}{12} + (t \times 150)(75 - 40.91)^2\right] \times 2$$

$$= 1.3295 \times 10^6 \, t \text{ mm}^4$$

$$T = 100 \times 10^3 (300 - 40.91) = 25.9 \times 10^6 \text{ N mm}$$

$$r_n = \sqrt{(150 - 40.91)^2 + 125^2} = 165.908 \text{ mm}$$

$$f_t = \frac{T r_n}{J} = \frac{587.8096}{t} \text{ N/mm}^2$$

63

$$\cos\theta = 0.657$$

$$f_r^2 = f_d^2 + f_t^2 + 2 f_d f_t \cos\theta$$

$$f_r = \frac{720.009}{t} \frac{\text{N}}{\text{mm}^2}$$

for safety of connection $f_r <$ per. shear stress

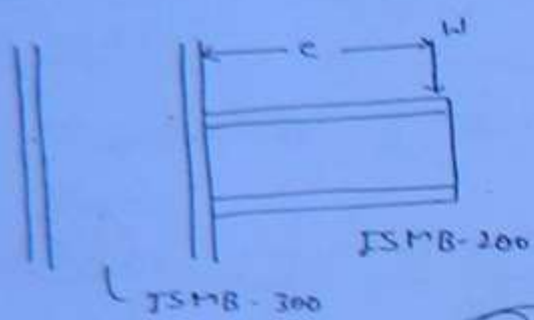
$$\frac{720.009}{t} < 110$$

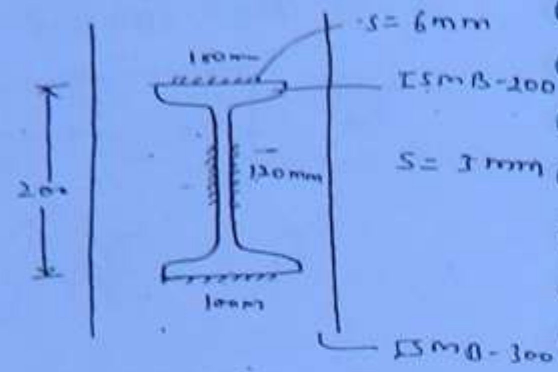
$$t > 6.545 \text{ mm}$$

$$s > \frac{6.545}{.7}$$

$$> 9.35 \text{ mm}$$

8.





64

Weld is subjected to direct shear W
and BM $= We$

These external things are resisted by fillet weld through shearing action

Direct shear stress $= \dfrac{W}{(100 \times .7 \times 6 + 120 \times .7 \times 3) \times 2}$ N/mm²

$= \dfrac{W}{1344}$ N/mm²

Hor. shear stress $= \dfrac{My}{I} = \dfrac{W(200) \times 100}{I}$

$I = 2 \times (100 \times .7 \times 6)\, 100^2 + \dfrac{.7 \times 3 \times 120^3 \times 2}{12}$

$= 9.0048 \times 10^6$ mm⁴

$f_h = \dfrac{W}{450.24}$ N/mm²

$f_r = \sqrt{f_h^2 + f_s^2} = \sqrt{\left(\dfrac{W}{1344}\right)^2 + \left(\dfrac{W}{450.24}\right)^2}$

$= 2.342 \times 10^{-3} W \le 100$

$W \le 42.69$ kN

Note: Bracing system resist lateral loads.

Tension Member

Comparison between tension and compression member.

Tension member	Compression member
1) In tension net area is effective	1) Gross area is effective.
2) There is no stability problem.	2) There is stability problem b/cos member may buckle before achieving full strength
3) Permissible stress $\sigma_{at} = 0.6\, f_y$	3) $\sigma_{ac} < 0.6\, f_y$
4) Design is straight forward.	4) Design is based on trial and error.

65

Note: Theoretically there is no limitation on slenderness ratio of tension member since stability is of little concern. However the member may be subjected to compressive load during transportation and erection hence in order to provide adequate rigidity to prevent undesirable lateral buckling and excessive vibration IS code limits the slenderness ratio values as follows

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

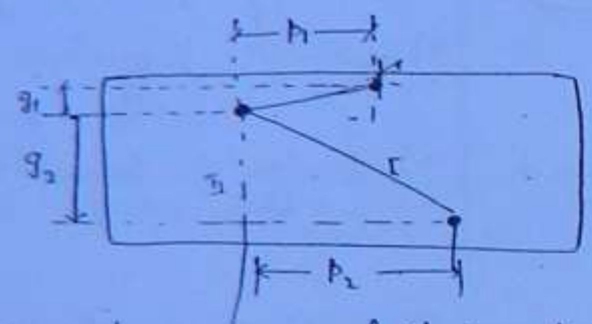
S.No.	Description	~~Max S.R.~~	Max S.R.
1)	A tension member in which reversal occurs due to loads other than wind or EQ		180
2)	A member normally acting as a tie (tension) in a roof truss or bracing system but subjected to compression due to wind or EQ		350

Net Sectional Area

(66)

For plates -

A plate may have various modes of failure in tension hence all the modes are required to be examined and the mode which gives the min net area will be the most critical mode.



The member may fail in tearing, either along 1-1 or 1-2 or may be 2-3 Net area for all of these failure modes will be calculated and the minimum of it will be the most critical

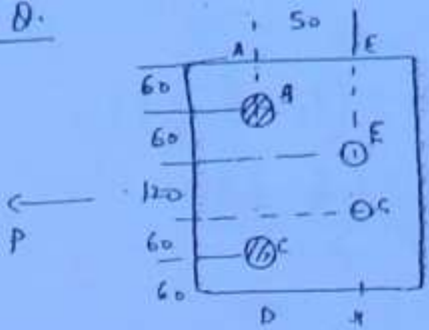
$$A_{net \text{ for I}} = Bt - nd't + \left(\frac{p_1^2}{4g_1} + \frac{p_2^2}{4g_2}\right)t = B_{eff} \times t$$

n ↓ No. of rivets along failure line

for each inclined line $\left(\frac{p^2}{4g_1} \times t\right)$ is added

$$A_{net \text{ for II}} = Bt - 2d't + \frac{p^2}{4g_1}t$$

Q.



The thickness of plate is 10 mm & hole dia 22 mm find the min net Area provided by the Plate.

(67)

$$NEA_{A_1 ABCD} = 360 \times 10 - 2 \times 22 \times 10 = 3160 \text{ mm}^2$$

$$NEA_{ABFCD} = 360 \times 10 - 3 \times 22 \times 10 + \left(\frac{50^2}{4 \times 60} + \frac{50^2}{4 \times 180}\right) \times 10$$

$$= 3078 \text{ mm}^2$$

$$NEA_{ABFGCD} = 360 \times 10 - 4 \times 22 \times 10 + \left(\frac{50^2}{4 \times 60} + 0 + \frac{50^2}{4 \times 60}\right) \times 10$$

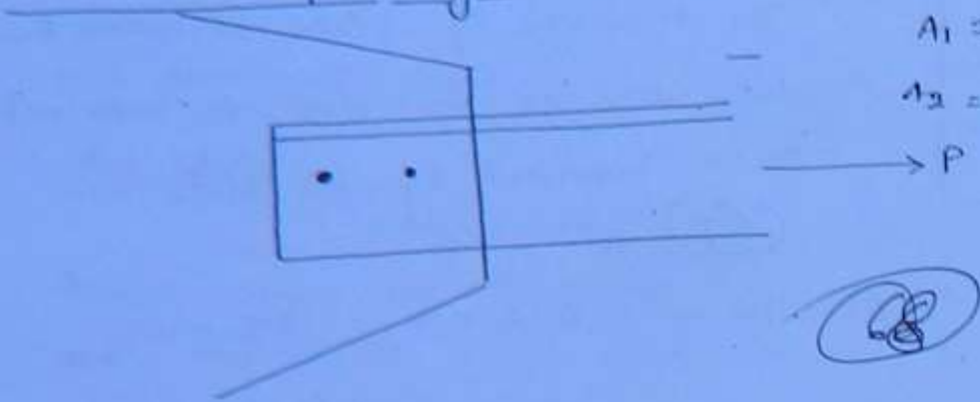
$$= 2928.33 \text{ mm}^2$$

$$NEA_{ABFGH} = 360 \times 10 - 3 \times 22 \times 10 + \frac{50^2}{4 \times 60} \times 10 = 3044.17$$

☆ In this case for calculation of force we have to add the rivet value of c

Hence the most critical section is ABFGCD

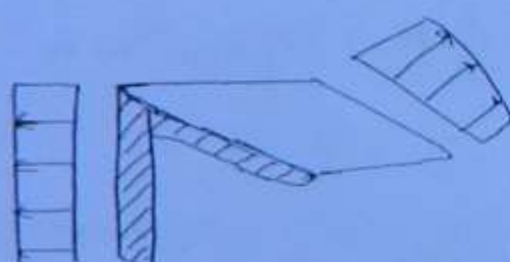
Net Area for angles



$$A_1 = \left(l_1 - d' - \frac{t}{2}\right) t$$

$$A_2 = \left(l_2 - \frac{t}{2}\right) t$$

Force is transferred in the member through the gusset plate location of transfer is at the location of rivets/welds this force is then distributed in the whole member through shearing. The connected legs leads over the outstanding leg in the distribution through shear stress thus there is a shear lag existing between connected and outstanding leg. The stresses in the angle at the location of joint can be shown as follows

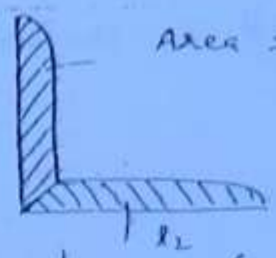


As the stress in outstanding leg is less the force carried by it, will be less to account for this net area is taken as $(A_1 + K A_2)$

Where K is a reduction factor for outstanding leg

$A_1 \rightarrow$ Area of connected leg

A_2 — outstanding leg



Area = $\left(l_1 - \frac{t}{2}\right) t$

Total area = $\left(l_1 + l_2 - t\right) t$

Area $\left(l_2 - \frac{t}{2}\right) t$

69

single angle connected through one leg only

$$A_{net} = A_1 + k A_2$$

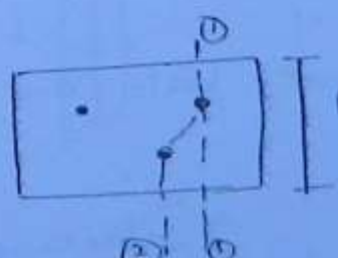
$$k = \frac{3A_1}{3A_1 + A_2}$$

$$A_1 = \left(l_1 - d' - t/2\right) t$$

$$A_2 = \left(l_2 - t/2\right) t$$

$$A_{net} = A_1 + k A_2$$

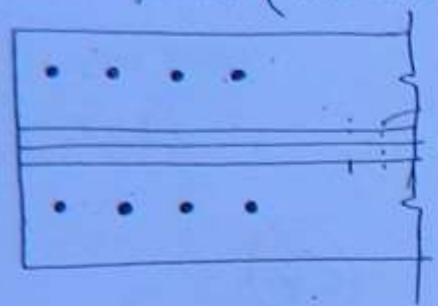
$$= \left(l_2 - \frac{t}{2}\right) t$$



$\left(l_1 - \frac{t}{2}\right) t$

$$A_{net} = \min \text{ of } \left\{ \left(l_1 - \frac{t}{2} - d'\right) t, \left(l_1 - \frac{t}{2} - 2d' + \frac{p^2}{4g}\right) t \right\}$$

Case-I (a) When two angles are connected on the same side of gusset plate (and are tacked)



70

$$A_{net} = A_1 + kA_2$$

$$k = \frac{5A_1}{5A_1 + A_2}$$

$A_1 \rightarrow$ Area of connected legs

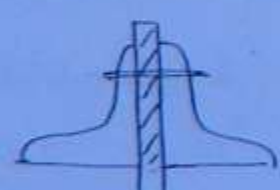
$A_2 \rightarrow$ " " outstanding legs

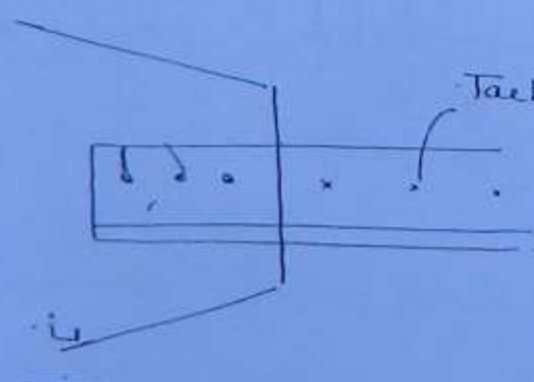
(b) When two angles are connected on the same side of gusset plate and not tack rivetted.

In this case two angles will behave individually hence net area would be twice the area corresponding to single ~~oute~~ angle connected to a gusset plate.

Case-II (a)

Two Angles connected on the opposite side of gusset plate and tacked along the length.





$$A_{net} = \left(A_{gross} - \text{Area of hole}\right)$$

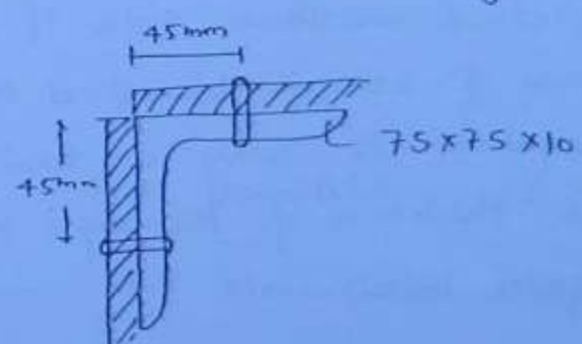
$$= 2(A_1 + A_2) - \text{Area of hole}$$

$$= 2(l_1 + l_2 - t)t - 2d't$$

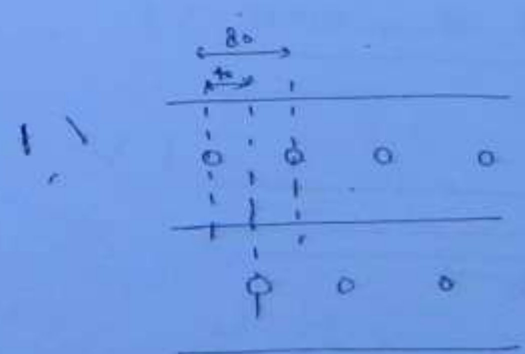
(b) When there is no tack rivetting along the length The two angles will behave individually and net area would be twice that corresponding to single angle connected to gusset plate.

Note: Shear lag effect is non existing in this case (case a)

Q. 04 ES An ISA 75×75×10 is connected to gusset plate by 16 mm diameter rivet through both legs the pitch on each leg is 80 mm and the rivets on one leg are staggered by 40 mm w.r.t. those other. Find allowable tensile load on the angle.

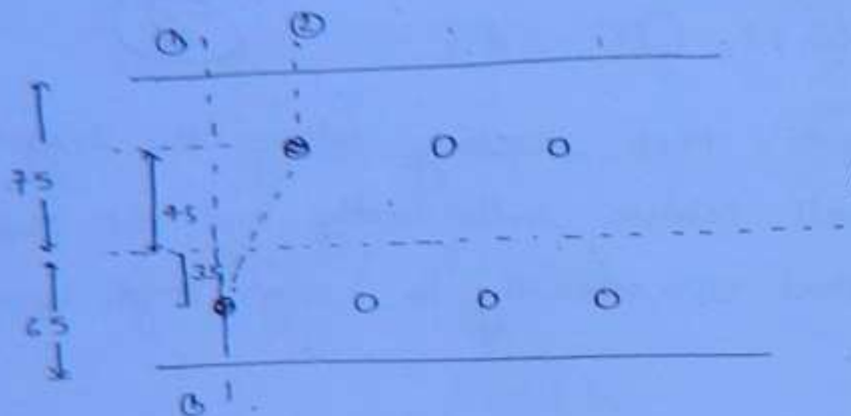


Sol



As both the legs are connected legs the angle can be considered as a plate

$d' = 16 + 1.5 = 17.5$



$A_{net\ 1-1} = (140 - 17.5) \times 10 = 1225\ mm^2$

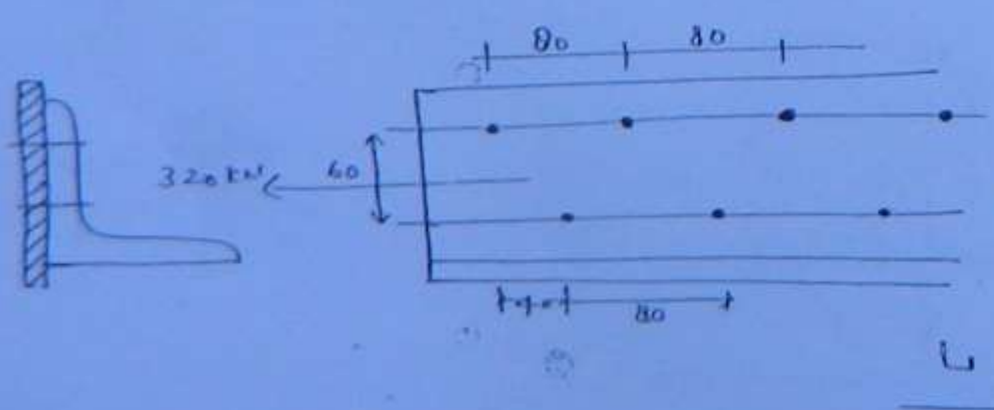
$A_{net\ 2-2} = \left(140 - 2 \times 17.5 + \frac{40^2}{4 \times 80}\right) \times 10 = 1100\ mm^2$

$A_{net\ critical} = 1100\ mm^2$

$P_{max} = 1100 \times 0.6\ f_y$

$= 165\ kN$

Q. An ISA 150 × 115 rivetted on one side of a gusset plate by two rows of 22 mm dia rivet through 150 mm leg. It is required to carry a tensile force of 320 kN. Find the thickness of the angle required the diagram is as shown below.



Sol.

[illegible] $150 - $ [illegible] $= 15$[illegible]

$150 - $ [illegible] $\frac{40^2}{4 \times 60} \cdot \frac{1}{2} = 109.67$ [illegible]

$$A_{net}\ req. = \frac{320}{0.6 f_y} = \frac{320 \times 10^3\ N}{150\ N/mm^2} = 2133.33\ mm^2$$

$$A_{net}\ available = A_{1net} + k A_2$$

$$A_2 = \left(115 - \frac{t}{2}\right) \times t$$

$$A_{1net} = \left[150 - \frac{t}{2} - 2 \times 23.5 + \frac{40^2}{4 \times 60}\right] \times t$$

$$= \left(\cancel{118.67} - \frac{t}{2}\right) t \quad \left(109.67 - \frac{t}{2}\right) t$$

The thickness will be assumed in such a way net area available should be more than Net area required.

Assume $t = 8mm$

$$A_{1net} = 845.336\ mm^2$$

$$A_2 = 888$$

$$k = .74$$

A_{net} available $= 1734.08 < A_{net}$ req. not safe

Assume $t = 10mm \Rightarrow$ unsafe

$t = 12mm \Rightarrow$ safe

$$k = \frac{3A_1}{3A_1 + A_2} = \frac{3\left(109.07 - \frac{t}{2}\right)}{3\left(109.07 - \frac{t}{2}\right) + \left(115 - \frac{t}{2}\right)}$$

$$=$$

74

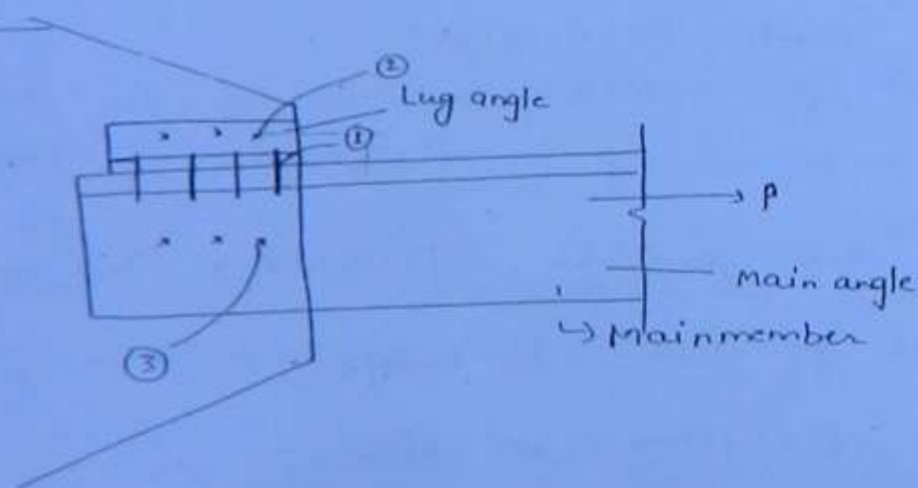
$$NEA = A_1 + kA_2$$

$$2133.33 = \left(109.07 - \frac{t}{2}\right)t + \frac{327.21 - 1.5t}{442.21 - 2t}\left(115 - \frac{t}{2}\right)t$$

$$t = 11.59 \text{ mm}$$

take $t = 12$ mm

Lug Angles



Lug angle is a short length of angle section used at a joint to connect the outstanding leg of main member thereby reducing the length of joint.

The rivet connecting the outstanding leg of main memb with the lug angle should start in advance of all other rivets this is done to ensure that force in the outstanding leg is effectively transferred to the lug angl

When angle members are main members → (B)

Lug angle and their connection with the gusset plate are designed for forces greater than equal to 1.2 times the force in outstanding leg of the main member.

Force in the outstanding leg $= \frac{F A_2}{(A_1 + A_2)}$

$A_2 \rightarrow$ Area of outstanding leg

$A_1 \rightarrow$ Area of connected leg

Hence designed force for lug angle and rivet 2 is

$$F_{D_2} = 1.2 \left(\frac{F A_2}{A_1 + A_2} \right)$$

connection of outstanding leg of main member with the lug angle should be designed for force $\geq 1.4 \cdot$ force in outstanding leg

Rivet 1 will be designed for

$$F_{D_0} = 1.4 \left(\frac{F A_2}{A_1 + A_2} \right)$$

When channel sections are main members.

- Lug angles and it's connection with the gusset plate should be designed for the force 1.1 times force in out standing legs.

(76)

- Connection of lug angles with the out standing legs of channel should be designed for a force greater than equal to 1.2 times the force in the outstanding legs.

- Minimum no. of rivets in lug angles should be 2.

Rivet 3 should be designed for force in connected leg.

ES-95 Q.2 Page 45

An

As force is not given the maximum force that the angle can resist will be found out under the assumption that, at any section main angle is reduced by 1 rivet hole.

$$F_{max} = \left(A_{gross} - A_{hole}\right) \times \sigma_{at}$$

$$A_{gross} = \left(75 + 100 - 10\right) \times 10 = 1650 \text{ mm}^2$$

$$A_{hole} = d' \times t = 21.5 \times 10 = 215 \text{ mm}^2$$

$$F_{max} = \left(1650 - 215\right) \times 150 = 215.25 \text{ kN}$$

$$A_1 = \left(100 - \frac{t}{2}\right) \times t = 950$$

$$A_2 = (75 - 5) \times 10 = 700$$

$$F_{outstanding} = \frac{F A_2}{A_1 + A_2} = \frac{215.25 \times 700}{1650} = 91.318\,kN$$

$A_1 = (100 - 5) \times 10 = 950\ mm^2$

$A_2 = 700\ mm^2$

77

$F_{connected} = 123.932\ kN$

Rivet value = Min (shearing, bearing), bearing

10mm, 8mm

$$R_v = Min\left(\frac{\pi d'^2}{4} \times 100,\ d' t \times 300\right)$$

$$= \left(\frac{\pi\, 21.5^2}{4} \times 100,\ 21.5 \times 10 \times 300\right)$$

$= (36.305\ kN,\ 64.5\ kN)$, $51.6\ kN$

$R_v = 36.305\ kN$

No. of rivet 3 $= \frac{123.932}{36.305} = 3.41$

Adopt 4 no.

No of rivet 2

Design of lug Angle

$F_{design\ lug\ angle} = 91.318 \times 1.2 = 109.582\ kN$

A_{net} req. $= \frac{109.582 \times 10^3}{150} = 730.55\ mm^2$

Let us adopt ISA 60×60×8

$A_{net} = 896 - 21.5 \times 8 = 724\ mm^2 < A_{net}$ req.

Not safe

Hence adopt a bigger section

Let us adopt ISA 60×60×10

A_{net} provided $= 1100 - 21.5 \times 10 = 885\ mm^2 > A_{net}$ req. o.k.

No. of rivet 1 $= \dfrac{1.4 \times F_{out}}{R_v}$ 78

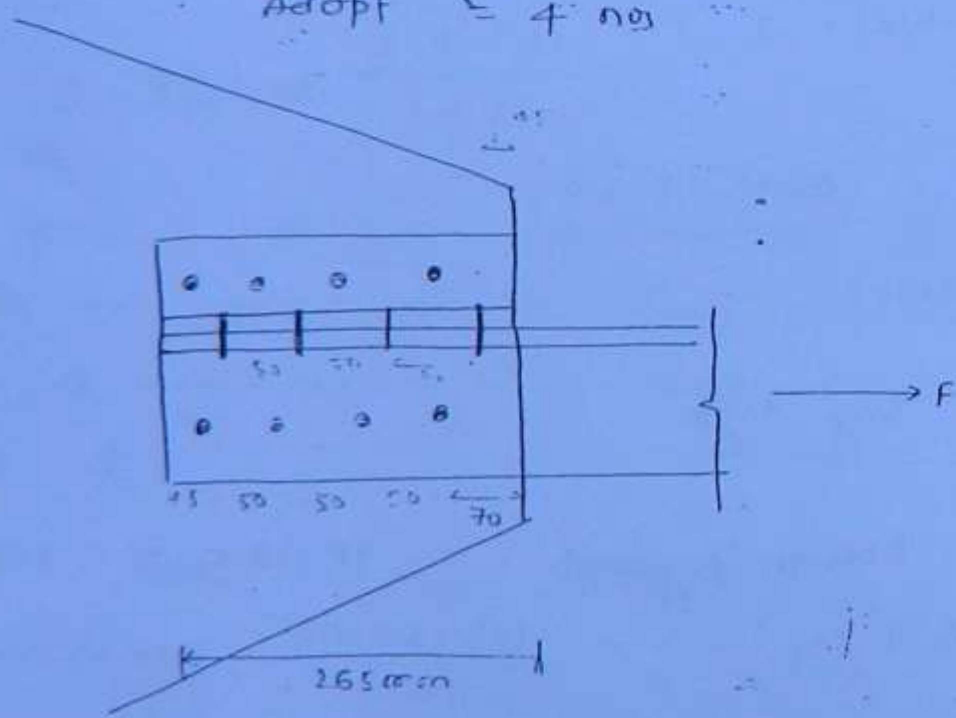
$= \dfrac{1.4 \times 91.318}{36.305} = 3.52$

adopt 4 no

No of rivet 2 $= \dfrac{1.2 \times F_{out}}{R_v}$

$= \dfrac{1.2 \times 91.318}{36.305} = 3.018$

Adopt = 4 nos



Sol. 1

[illegible] : 50 mm

500 kN

ϕ = 22 mm

The thickness of main plate depends on the tearing strength of the main plate that is

tearing strength $\geq$ 500 kN

- Tearing strength can be known only if arrangement of rivet is known.
- Arrangement of rivet is known only when no. of rivets are known.
- No. of rivets are known only when rivet value is known
- Rivet value is known only when we know whether shearing or bearing govern.
- The rivets are in double shear hence shearing strength is known.
- Bearing strength will depend on the thickness of main plate

By equating shearing and bearing strength of rivet we get the thickness of main plate under the assumption that combined thickness of cover plate is more than that of main plate.

If thickness is not capable of resisting 500 kN load, thickness has to be increase. If thickness is increased bearing strength will become more the shearing str. of rivet they

Hence the rivet value will be governed by shearing strength of rivet. hence no. of rivet will be = $\frac{500}{[illegible]}$

No. of rivet will be arranged and tearing strength a will be calculated at various sections in terms of thickness t

taking per. shear = 100 N/mm² 8

Sol. Rivet is in double shear

Shearing st. of rivet = $2 \frac{\pi}{4} \times (23.5)^2 \times 100$

$= 86.75$ kN

Assuming the thi combined thickness of cover plate to be more than main plate

Bearing strength $= d' \times t\, \sigma_{br}$

$= 23.5 \times 300 \times t = 7.05\, t$ kN (9 mm)

$86.747 = 7.05\, t$

$t = 12.305$ mm

Max. force that gross area corresponding to this thickness can resist $= .6 \times 12.305 \times 250 \times 260 = 480$ kN < 500 Not safe

Hence thickness of main plate has to be ↑

thus shearing strength govern the rivet value.

$\Rightarrow R_v = 86.748$ kN

$n = \frac{500}{86.748} = 5.7$

adopt n = 6 rivet

The 6 no. of rivet can be arranged in diamond pattern

Tearing strength of sec^n 1-1

$$= (250 - 23.5) \times t \times 0.6 \times 260 \geq 500 \times 10^3$$

$$t \geq \cancel{14.716\ mm}\ 14.151\ mm$$

81

at 2-2

$$(250 - 2 \times 23.5) \times t \times .6 \times 260 + 86.748 \times 10^3 \geq 500 \times 10^3$$

$$t \geq 13.572\ mm$$

$$\geq 13.05\ mm$$

Hence we will adopt thickness 15 mm

let us adopt 16 mm thickness of main plate

check for strength of cover plate

For cover plate sec^n 3-3 is critical

$$(250 - 3d') \times t \times .6 \times 260 \geq 500$$

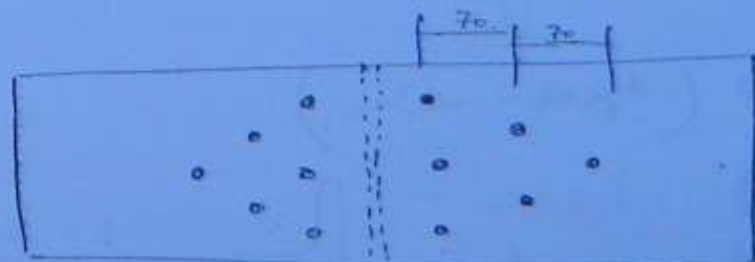
$$t \geq 17.85\ mm$$

Adopting thickness of cover plate as 10 mm each

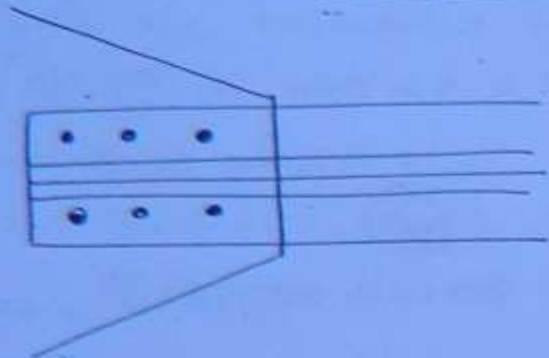
For gusset plates as the thickness of main gusset plate as well as combined thickness of two cover plate are more than 12.3 mm hence shearing will govern the rivet value.

$$\text{hence} = \frac{500}{86.74} = 5.7$$

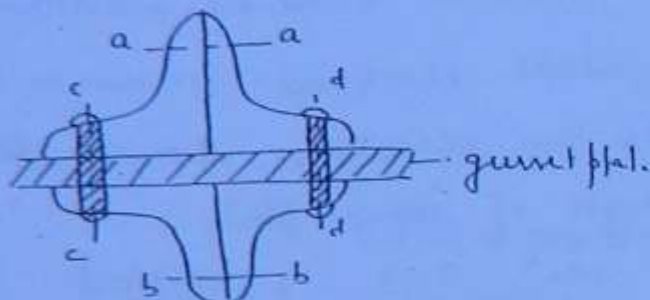
adopt 6 rivet



Compression Member



82



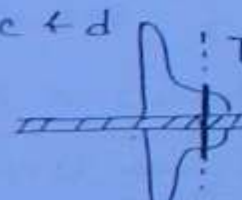
1) Tacking along a, b, c, d

$$A_{net} = \left(A_{gross} - 4\,A_{hole}\right)$$

2) Tacking along a & b

$A_{net} = \Big[\,2 \quad \times$  $\Big]$ $\qquad k = \dfrac{5A_1}{5A_1 + A_2}$

3) Tacking along c & d

$A_{net} = \Big[\,2 \times$  $\Big]$ $\qquad k = 5A_1,$

$$= 2 \times 2\left(A_{gross} - A_{hole_1}\right)$$

4) No tacking
↳

$A_{net} = 4 \times$  $\qquad k = \dfrac{3A_1}{3A_1 + A_2}$

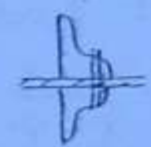
Ex. Angle $60 \times 60 \times 10$

$A_{gross} = (60 + 60 - 10) \times 10 = 1100\ mm^2$

Case 1 $\quad 4 \times 1100 - 4 \times 21.5 \times 10 = 3540\ mm^2$

Case 2 $\quad k = \dfrac{5A_1}{5A_1 + A_2} = \dfrac{5 \times 670}{5 \times 670 + 1100} = .7528$

$A_{net} = 2\left[670 + .7528 \times 1100\right] = 2996.16\ mm^2$

Case 3 $\quad 2 \times \Big[$  $\Big] = 2 \times \left[2 \times 1100 - 2 \times 21.5 \times 10\right]$

$= 3540\ mm^2$

Case 4 $\quad 4 \times$  $= 4\left[A_1 + kA_2\right]$

$A_1 = 335$

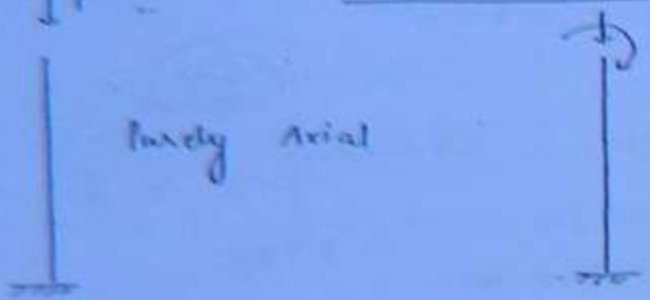
$A_2 = \left(60 - \dfrac{t}{2}\right) t = 550$

$k = \dfrac{3A_1}{3A_1 + A_2} = 0.6463$

$A_{net} = 2761.86\ mm^2$

Note: Max. deduction due to all hole at any section along the length of member should never be more than max deduction of hole at any secn at the joint. This is done to ensure that if the joint is not failing member will not fail.

Compression Member
(Purely Axial)



84

For long columns under purely axial loading the failure is always by buckling and buckling load is given by Euler's load

$$P = \frac{\pi^2 EI}{l^2}$$

The buckling stress is given by Euler's stress

$$\text{Euler stress} = \frac{\pi^2 E}{\lambda^2}$$

$$\lambda = \frac{l_{eff}}{r_{min}}$$

$r_{min} \rightarrow$ Min radius of gyration of the section and l_{eff} is eff. length of compression member which depend on end support condition. However the IS code takes into account the failure by buckling and crushing simultaneously through the use of Merchant Rankine formula

$$\sigma_{ac} = \text{Per. stress for axial comp} = \frac{0.6\, f_{cc} \cdot f_y}{\left[(f_{cc})^{1.4} + (f_y)^{1.4}\right]^{\frac{1}{1.4}}}$$

f_{cc} → Euler stress

f_y - Yield stress.

For a given grade of steel $\sigma_{ac} = f(\lambda)$

As $\lambda \uparrow$ σ_{ac} decreases  $\frac{l}{2}$. leff

leff as b/w point of inflexion

Effective length (for columns)

S. No.	Description	(leff)	leff theoretical
(1)	 l	$0.65l$	
(2)	 l	$0.8l$	$.707l$
(3)		l	l
(4)	 Fixed at one end and at other end restrained against rotation but fixed in position	$1.2l$	
(5)	 Fixed at one end but at other end partially restrained against rotation but not fixed in position	$1.5l$	

Note

length adopted by IS code is more than the theoretical value because it is impossible to simulate perfect fixity of support. So we $\uparrow$ leff $\Rightarrow \lambda \uparrow \Rightarrow \sigma_{ac} \downarrow$

(6)		$2l$	
(7)		$2l$	

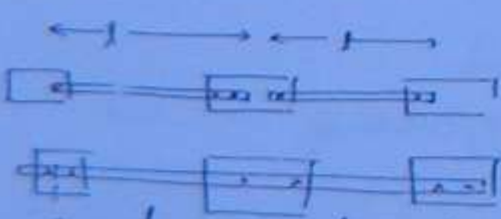
The above values are $\uparrow$ by 10% in case of battened column

Effective length of angle strut

used in trusses and bracing system

		Description	l_{eff}	Allowable compressive stress (σ_{ac})
1	Continuous	Single or double angle	$.7l$ — l	σ_{ac}
2.	Discontinuous	Single angle connected with one rivet only	l	$0.8\,\sigma_{ac}$
3.	"	single angle with more than one rivet or weld	$.85l$	σ_{ac}
4.	"	Double angle back to back. On opposite side of gusset plate	$.7$ to $.85l$	σ_{ac}
5.	"	Double angle on same side of gusset plate	l	$.8\,\sigma_{ac}$

* Actual value of effective length depend on the rigidity of joint



Discontinuous

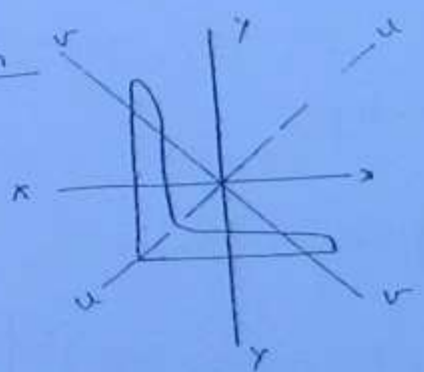
continuous

{ As fixity of joint ↑ l_{eff} ↓ & viceversa }

Maximum slenderness ratio

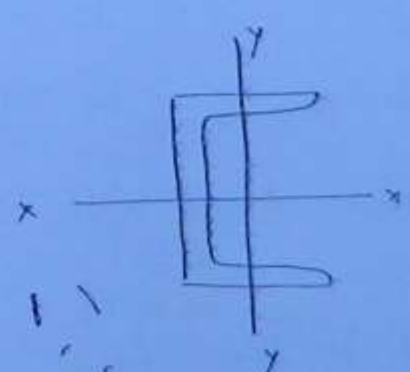
S.No.	Description	λ_{max}
(1)	Member carrying compressive load due to dead & live load only	180
(2)	Member subjected to compression due to wind or EQ	250
(3)	For compression flange of beam	300
(4)	Member normally acting as a tie in a roof truss or bracing system but subjected to reversal of stress due to wind or EQ.	350

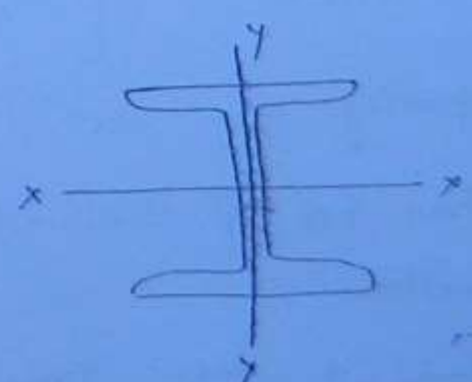
Rolled Section



$I_{min} = I_{vv}$

$r_{min} = r_{vv}$

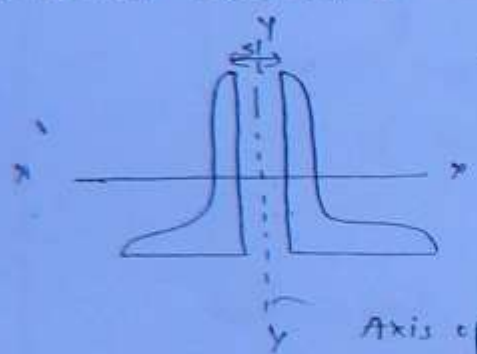




I_{min} ↳ I_{xx} or I_{yy}

for rolled sections M.O.I. can not be changed. Hence to find out the load carrying capacity I_{min} needs to be located. If the section is having at least one axis of symmetry, $r_{min} = \sqrt{\frac{I_{min}}{A}}$ will be about the axis of symmetry or about an axis ⊥ to the axis of symmetry. If however the section is having no axis of symmetry then major and minor principal axis needs to be located.

In case of built up sections we can change the MOI of combined section a



Axis of symmetry

88

$$I_{min} < \begin{matrix} I_{xx} \text{ combination} \\ I_{yy} \text{ combination} \end{matrix}$$

$$I_{xx_{comb.}} = 2 \times I_{xx} \text{ of one section}$$

$$I_{YY_{comb}} = 2\left[I_{YY_{one}} + A\left(C_{YY} + \frac{S}{2}\right)^2\right]$$

If spacing b/w secⁿ is zero $I_{YY\,comb.}$ in this case will be less than I_{xx} of combination hence I_{min} will be I_{yy} of combination.

If I_{min} is less r_{min} will be less fo

for this λ will be more.

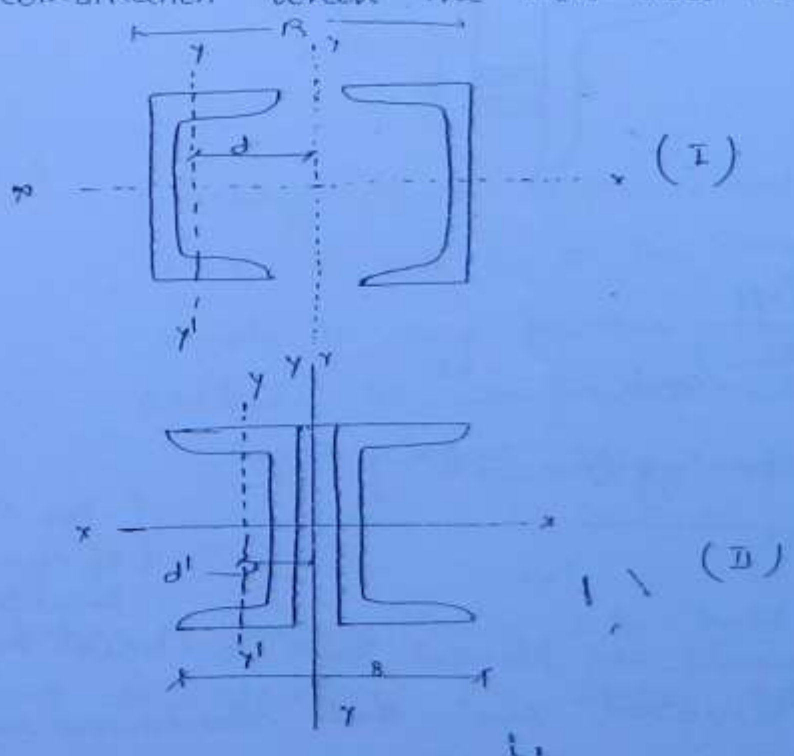
and hence σ_{ac} will be less.

Thus load carrying capacity will be less.

To increase the load carrying capacity for the given area. I_{yy} of combination needs to be increased - This can be achieved by increasing the spacing. for a given spacing S_0, I_{xx} of combination will become I_{yy} of comb. For $S > S_0$, $I_{min} = I_{xx}$ of comb. which is fixed. Hence load carrying capacity becomes fixed for all $S > S_0$ and this spacing corresponds to maximum load carrying capacity for a given sectional area. Thus for best utilization of material I_{xx} of comb = I_{yy} of comb.

(89)

Radius of gyration of a sections will not change when put in combination when the axis does not shift

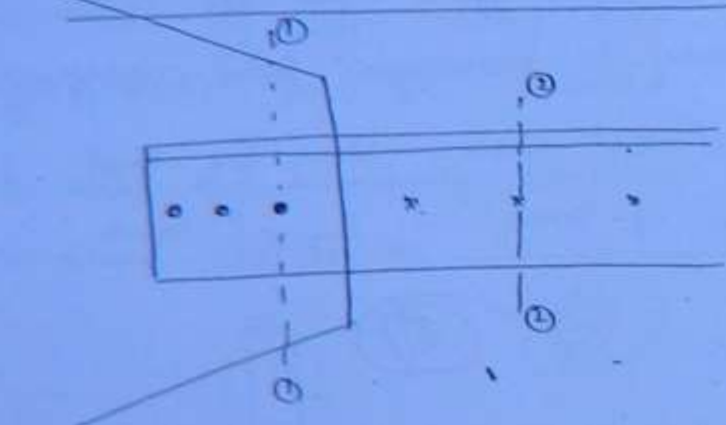


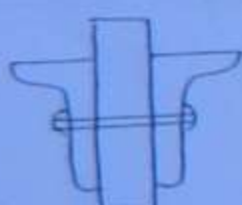
For a given overall size of column strength of I will be more than the strength of II because shifting of axis will be more in (I) than in (II)

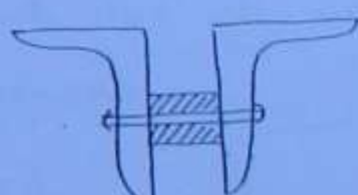
$$d > d'$$

For maintenance point of view II will be better.

IS code Recommendation.







$$\lambda_{\text{whole sec}^n} = \frac{l_{eff}}{(r_{min})_{comb}}$$

$$\lambda_{one} = \frac{\text{spacing b/w tacking rivets}}{r_{vv}}$$

{ For the point of view of local buckling

When two components are placed back to back they should be tack rivetted such that slenderness ratio of individual secⁿ b/w tacking rivet should not be more than 40 nor more than 0.6 × slenderness ratio of whole secⁿ

(91)

$\lambda_{one} \not> 40$

$\not> 0.6\,\lambda_{whole\ sec^n}$

This recommendation safeguards against local buckling of individual sections between tacking rivets.

The spacing b/w tacking rivet in line however should not be more than 600 mm.

The dia of tacking rivet should not be ~~more~~ less than the min dia given below.

Thickness of member	Dia of tacking rivet upto
upto 10 mm	18 mm
10 – 16 mm	20 mm
> 16 mm	22 mm

Note: Tacking rivet recommendation given before 32 t or 300 mm is the max. spacing b/w taking rivets in any position. However the limiting criteria in compression will become

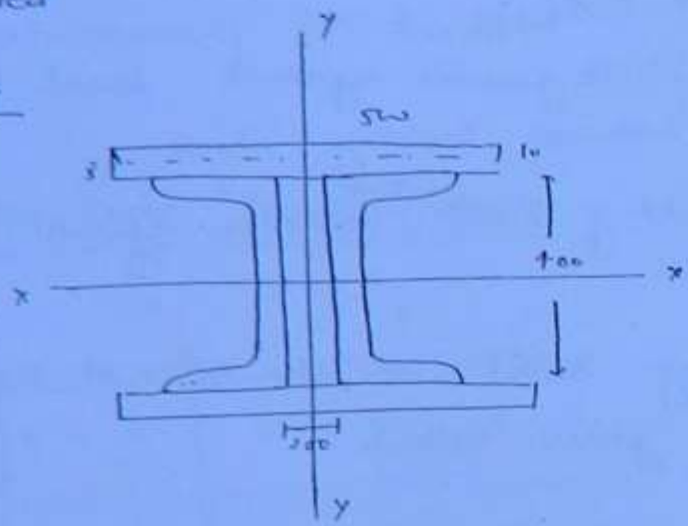
$\lambda_{one} \not> 40$

$\not> 0.6\,\lambda_{whole\ sec^n}$

- Purpose of tacking rivet is to hold the two member together and to equalize the stress ~~b/w~~ in the component memb

If the web of channel is > 150 mm in depth, two rows of tacking rivet can be provided. If angles are 125 mm or more two rows of tacking rivet can be provided

8.10 P.49



92

$$I_{xx} = 2 \times 150828 \times 10^3 + \left[\frac{500 \times 10^3}{12} + 500 \times 10 \times 205^2 \right] \times 2$$

$$= 721.9893 \times 10^6 \text{ mm}^4$$

$$I_{yy} = 2\left[5048 \times 10^3 + (100 + 24.2)^2 \times 6.293 \times 10^3 \right]$$

$$+ 2 \times \frac{10 \times 500^3}{12}$$

$$= 412.5764 \times 10^6 \text{ mm}^4$$

I_{yy} is min

$$r_{min} = \sqrt{\frac{I_{yy}}{A_{combined}}} = \sqrt{\frac{412.5764 \times 10^6}{2 \times (6.293 \times 10^3 + 5000)}}$$

$$= 135.155 \text{ mm}$$

$$\sigma_{ac} = 500$$

$$\lambda = \frac{l_{eff}}{r_{min}} = \frac{5000}{135.155} = 36.995$$

$$\sigma_{ac} = 145 - \frac{(145-132)}{20} \times 6.995$$

93

$$= 140.453 \text{ N/mm}^2$$

$$\text{safe load} = 140.453 \times 2(6293 + 5000)$$

$$= 3172.277 \text{ kN}$$

if effective length is 6m

$$\lambda = \frac{6000}{135.155} = 44.393$$

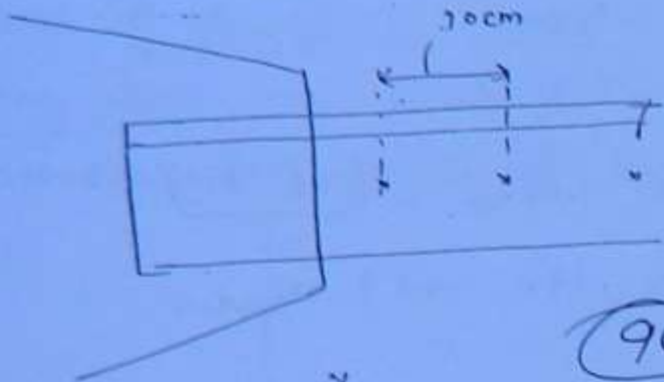
$$\sigma_{ac} = 145 - \frac{(145-132)}{20} \times 14.393$$

$$= 135.645 \text{ N/mm}^2$$

$$\text{Safe load} = 135.645 \times 2(6293 + 5000)$$

$$= 3063.667 \text{ kN}$$

Q.9

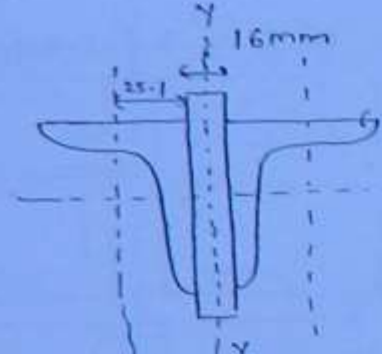


2 ISA 90×90×8

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

$I = Ar^2$

(94)



ISA 90×90×8

$A = 1379 \ mm^2$

$C_{xx} = C_{yy} = 25.1 \ mm$

$r_{xx} = 27.5 \ mm$

$l_{eff} = 3 \times .85 = 2.55 \ m$

$I_{xx} = 2 A r_{xx}^2 +$

$r_{xx} = 27.5 \ mm$

$$I_{yy} = \left[Ar^2 + A \times 33.1^2 \right] \times 2$$

$$= A \left[27.5^2 + 33.1^2 \right] \times 2 = 5.107429 \times 10^6$$

$$r_{yy} = \sqrt{\frac{I_{yy}}{A}} = 43.03 \ mm \quad \boxed{\text{not need}}$$

r_{xx} is min

$$\lambda = \frac{l_{eff}}{r} = \frac{2550}{27.5} = \cancel{59.257} = 92.727$$

$$\lambda_{one} = \frac{300}{17.5} = 17.143$$

$$f = \cancel{1320 - \frac{1320 - 1120}{20} \times 9.257} = 1227.43 \ kg/cm^2$$

$= 120.288 \ N/mm^2$

$P =$

$f = \cancel{4}\ 900 - \frac{900 - 720}{20} \times 2.727$

$= 875.45\ kg/cm^2 = 85.882\ N/mm^2$

$P = 85.882 \times 2 \times 1379 = \underline{236.863\ kN}$

$g = 9.81\ N/s^2$

95 if $g = 10 \Rightarrow P = \underline{241.325\ kN}$

Note

As the angle is equal angle $I_{xx} = I_{yy}$

$r_{xx} = r_{yy}$

When put in combination x axis does not shift but y axis shifts such that

I_{yy} of comb $>$ I_{xx} of comb.

hence

I_{min} of comb. $=$ I_{xx} of comb.

Check for local buckling.

$\lambda_{one} = \frac{30}{r_{vv}} = \frac{30}{1.75} = 17.143$

$\lambda_{whole} > \lambda_{one}$

$$\left.\begin{array}{l} 17.14 \not> 92.7\ \ 270.6 \\ \quad\quad\ \not> 40 \end{array}\right\} \Rightarrow \text{safe in local buckling.}$$

local buckling will only be checked load can not be calculated on the basis of this

8. A stanchion of eff. length 6 m consists of twin box sec^n using an ISMB 250 with two plates 260×10 mm size welded each to the tips of two flanges of ISMB 250 on both sides with 4 mm fillet weld continuous through out the height properties of ISMB 250 are

$D = 250$ mm

$B = 125$ mm

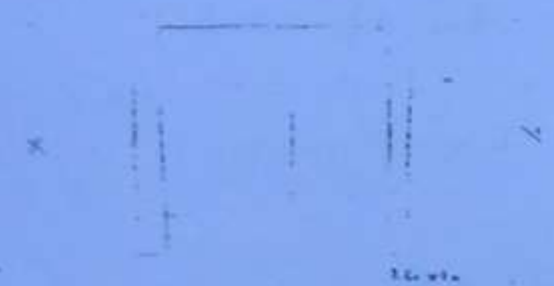
$A = \cancel{40}\ 47.55\ cm^2$

$I_{xx} = 5131.6\ cm^4$

$I_{yy} = 334.5\ cm^4$

(96)

Calculate the load carrying capacity of the section.



$$I_{xx_{tot}} = 5131.6 + 2 \times \frac{1 \times 26^3}{12} = 8060.933\ cm^4$$

$$I_{YY} = 334.5 \times 10^4 + 2\left[\frac{260 \times 10^3}{12} + 67.5^2 \times 260 \times 10\right]$$

$$= 27.08 \times 10^6\ mm^4$$

$$I_{min} = 27.08 \times 10^6$$

$$r_{min} = \sqrt{\frac{I_{min}}{A}} = \sqrt{\frac{27.08 \times 10^6}{(4755 + 260 \times 10 \times 2)}}$$

$$= 52.1567\ mm$$

$$\lambda = \frac{l_{eff}}{r_{min}} = \frac{6000}{52.1567} = 115.038$$

110 —— 75.3 N/mm²

120 —— 67.1 N/mm²

$\sigma_{ac} = 71.168\ N/mm^2$

$P = \sigma_{ac} \times A = 71.168 \times (4755 + 260 \times 20)$

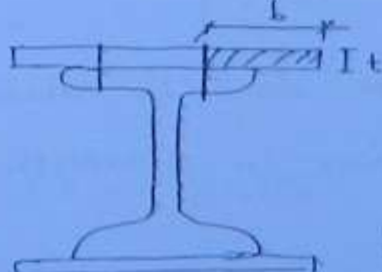
$= 708.47\ kN$

97

* Local buckling Phenomenon in cover plates and web plates.

Local buckling phenomenon in webs and cover plate of compression member occurs in the form of waves or wrinkles. The critical stress at which local buckling in the form of waves or wrinkle starts is given by

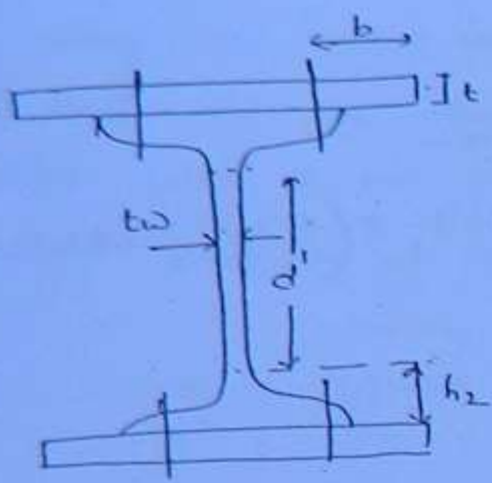
$$f_{cr} = \frac{K\pi^2 E}{12(1-\mu^2)\left(\frac{b}{t}\right)^2}$$

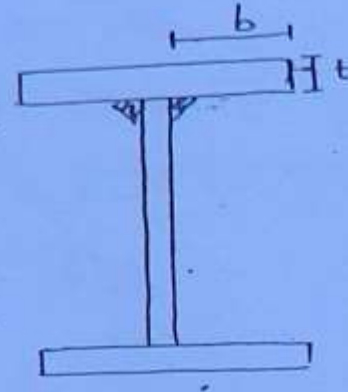


$$f_{cr} \propto \frac{1}{(b/t)^2}$$

Larger the value of b/t smaller is the stress at which local buckling starts in the form of waves or wrinkles.

if $\left(\frac{b}{t}\right)$ is so adjusted that critical stress for local buckling becomes more than the stress corresponding to overall failure of column, then local buckling is will not occur before overall failure of column on this basis : IS code has given the following recommendation





98

$\left(\frac{b}{t}\right) \ngtr 16$

Any portion of b greater than 16 t should not be taken into account in strength calculation

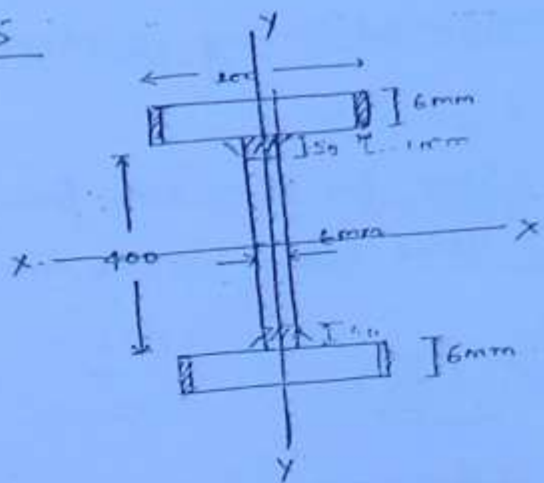
<u>For web</u> $\left(\frac{d'}{t_w}\right) \ngtr 50$

Any portion of d' > 50 tw should not be taken into account in strength calculation.

<u>Note →</u>

The area reduction should be done in such a way that it leads to maximum reduction in strength.

Q.5



99

100 from

$$I_{yy} = 2 \times \frac{198^3 \times 6}{12} + \frac{6^3 \times 300}{12} = 7.7677 \times 10^6 mm^4$$

$$I_{xx} = \frac{6 \times 300^3}{12} + 2\left[\frac{198 \times 6^3}{12} + 198 \times 6 \times 203^2\right]$$

$$= 111.42 \times 10^6 mm^4$$

$$r_{min} = \sqrt{\frac{I_{min}}{A}} = \sqrt{\frac{7.7677 \times 10^6}{2 \times 198 \times 6 + 300 \times 6}} = 43.29 \text{ mm}$$

$\lambda = 81.15$

$\sigma_{ac} = 997.91 \text{ kg/cm}^2$

$P_{safe} = 997.91 \times (2 \times 198 \times 6 + 300 \times 6) \times 10^{-2}$

$= 41672.17 \text{ kg}$

$= 416.727 \text{ kN}$

Q. 6 Page 47

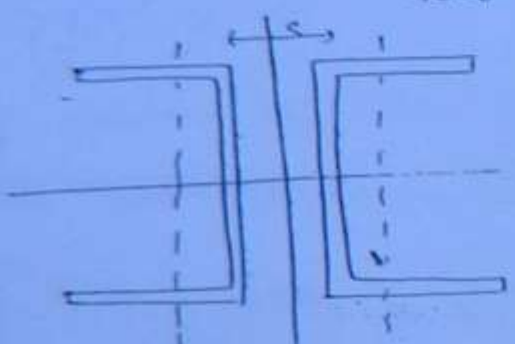
A compression is to be constructed using 2 ISMC 300 placed back to back

Cyy for ISMC 300 from steel table $= 23.6$ mm

$$\sigma_{ac} = \frac{fc / f.o.s}{1 + a\lambda^2}$$

a ↓ Rankine constant 100

$$= \frac{3200/3}{1 + \frac{1}{7500}\lambda^2} = \frac{8000000}{7500 + \lambda^2} = 759.79 \ \frac{kg}{cm^2}$$



For max. effect.

$I_{xx} = I_{yy}$

$$I_{xx} = 2 \times 6362.2 \times 10^4 =$$

$$I_{yy} = \left[310.8 \times 10^4 + 4564 \times \left(23.6 + \frac{s}{2}\right)^2\right] \times 2$$

$S = 183.095$ mm

$r_{min} = (r_{xx})_{comb} = (r_{xx})_{one} = 118.1$ mm

$l_{eff} = .65 \times l = 6.5m = 6500$

$\lambda = 55.038$

↳ σ

$P_{safe} = \sigma_{ac} \times Area$

$= 759.79 \times 4564 \times 2$

$= 69353.63$ kg

(18)

$$\boxed{\frac{1\ kgf}{cm^2} = \ldots \frac{N}{mm^2}}$$

use this conversion

$f \rightarrow$ force

Design of compression member required

Design of compression member required finding out area of cross secⁿ which depend on σ_{ac} $\left(\text{area req} = \frac{P}{\sigma_{ac}}\right)$ but σ_{ac} also depends on the section hence we will not have a direct solⁿ as in the case of tension member where permissible stress was constant. Hence we will go for a trial and error solution

Step 1 Assume the value of σ_{ac}

a) for rolled secⁿ choose σ_{ac} between 60 — 80 MPa

b) For built up section choose σ_{ac} 110 MPa

Step 2 $\frac{P}{(\sigma_{ac})_{chosen}}$ = Area required

Choose see trial secⁿ and find the slenderness ratio and hence σ_{ac} for the chosen area

Calculate safe load carrying capacity of the choosen section

$$P' = \text{area choosen} \times \sigma_{ac\,\text{chosen}}$$

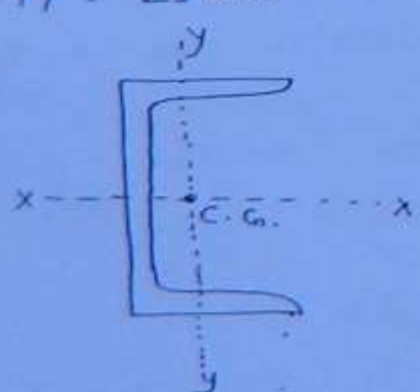
$$P' = \text{area}_{\text{provided}} \times \sigma_{ac}$$

P' - safe load carrying capacity of trial sec^n

- If $P' > P$ sec^n is sufficient otherwise chose another larger section and repeat. 102
- check for max. limit of slenderness ratio
- check for slenderness ratio can be done at the time when slenderness ratio of chosen sec^n was found out

Q Design a built up column 10 m long to carry an axial load of 750 KN. Use 2 channels placed back to back l_{eff} = 10 m. The section available are

	ISMC 250	ISMC 300
A =	3867 mm^2	4564 mm^2
r_{xx} =	99.4 mm	118.1 mm
r_{yy} =	23.8 mm	26.1 mm
C_{yy} =	23 mm	23.6 mm



1 - Chose

$$\sigma_{ac} = 110 \ N/mm^2$$

$$A_{req.} = \frac{P}{\sigma_{ac}} = \frac{750 \times 10^3}{110} = 6818 \ mm^2$$

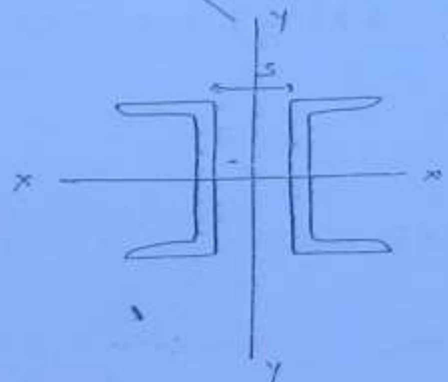
Chose 2 nos of ISMC 250

$$A \text{ provided} = 2 \times 3867 = 7734 \ mm^2$$

For the chosen area find $r_{min} \rightarrow \lambda \rightarrow \sigma_{ac}$

r_{min} < min of r_{xx} & r_{yy}

(103)



r_{xx} of combination = r_{xx} of 1 sec^n b/coz x axis has not shifted and only 2 similar rolled sec^n has been used.

r_{yy} of combination depends on spacing between the sec^n

for most efficient design

$$r_{xx \ comb} = r_{yy} \ comb.$$

thus spacing is so chosen that r_{xx} of comb = r_{yy} of com

and under the situation

$$r_{min} = (r_{xx} \text{ of comb})$$

$$r_{min} = r_{xx} \text{ of one sec}^n = 99.4 \ mm$$

$$l_{eff} = 10 \ m$$

$$\lambda = \frac{l_{eff}}{r_{min}} = 100.6 < 180$$

ok. from the limit

λ	σ_{ac}
60	100.7
90	92.8
100	84
110	75.3

104

$\sigma_{ac} = 85.478\ N/mm^2$

$P' = \sigma_{ac}$ provided $\times$ Area $_{pro}$

$= 84.478 \times (2 \times 3867) = 645.6\ KN < 750$ not safe.

Choose 2 ISMC 300

$A_{provided} = 2 \times 4564 = 9128\ mm^2$

$r_{min} = r_{xx}$ of one $= 118.1\ mm$

$$\lambda = \frac{l_{eff}}{r_{min}} = \frac{10000}{118.1} = 84.67 < 180 \text{ ok}$$

$$\sigma_{ac} = 100.7 - \frac{100.7 - 92.8}{30} \times 24.67$$

$= 94.2\ N/mm^2$

$P' = \sigma_{ac} \times A_{pro}$

$= 94.2 \times 9128 = 859.89\ KN > 750\ KN$ ok

spacing calculation

$$I_{xx})_c = I_{yy})_c$$

$$2 I_{xx\,one} = 2\left[I_{yy} + A\left(c_{yy} + \frac{s}{2}\right)^2\right]$$

$$2 A r_{xx}^2 = 2\left[A \times r_{yy}^2 + A\left(c_{yy} + \frac{s}{2}\right)^2\right]$$

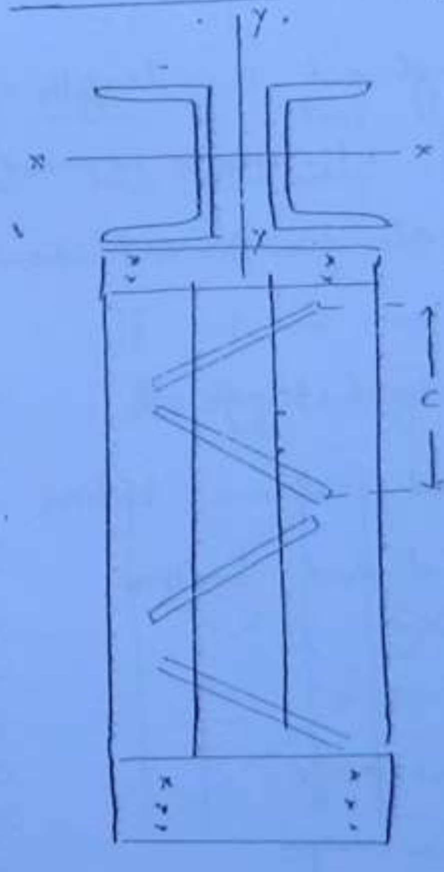
$$r_{xx}^2 = r_{yy}^2 + \left(C_{yy} + \frac{S}{2}\right)^2$$

$S = 183.1$ mm 185

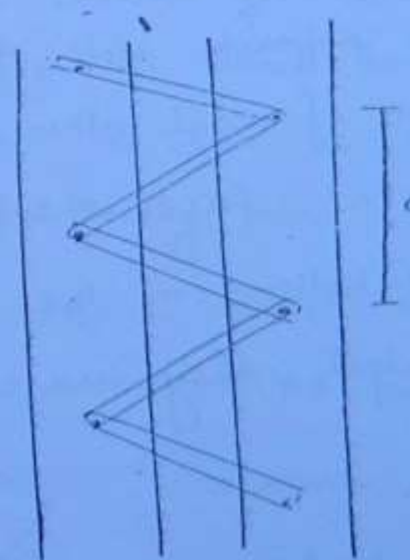
Spacing should be chosen 185 mm

under this spacing r_{min} will remain r_{xx}

Design of lacing and batten



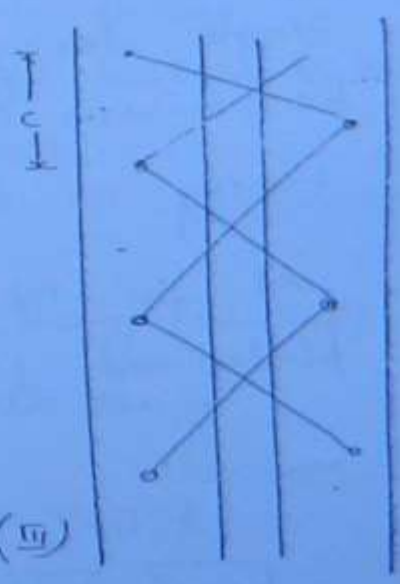
(I)



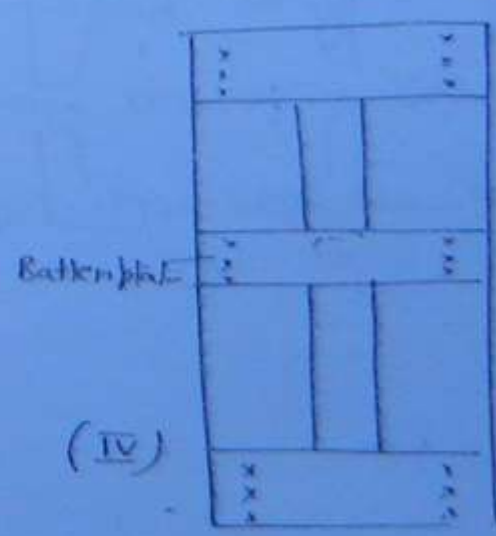
single lacing

(II)

Double lacing



(III)



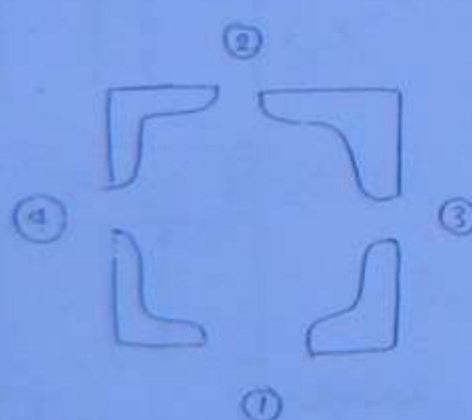
(IV)

1) Out of (I) & (II) I is better because acciden-
tal failure of 1 joint will not ~~decre~~ increase the
distance b/w intermediate connection significantly.
2) If the single lacing is not sufficient to safeguard
against local buckling. double lacing will be provided

106.

Lacing
lacing is in the form of a flat plate or
angle. r_{yy} is kept slightly more than r_{xx}

- Lacing should not be varied through out the length of compression member (angle, ~~of~~ x^{secn}, etc should be same)
- Single lacing on opposite side of main component should be mirror image of each other.
- lacing on adjacent face should be staggered.
- At the top and bottom in the laced column batten plates are provided they are called end battens



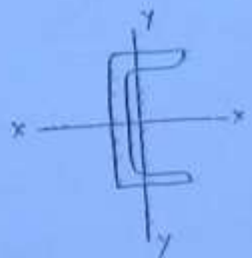
1- 2 mirror image
3- 4 mirror image
1- 3 Staggered.

Design

1) Angle of inclination of lacing with the longitudinal axis of cloumn should be between 40 - 70 degrees. $\in \theta$

2) Slenderness ratio of individual member of column b/w intermediate connections should not be more than 50 neither more than 0.7 time slenderness ratio of whole section.

107



$$\frac{C}{(r_{min})_{one}} \not> 50$$

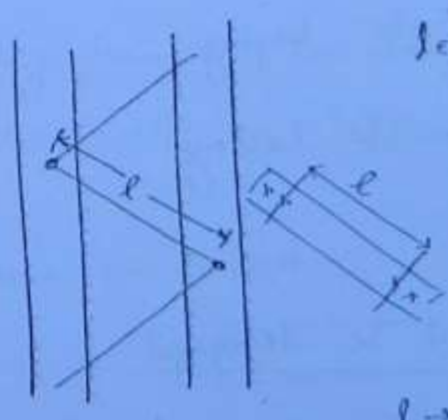
$$\not> 0.7 \lambda_{whole}$$

} criteria for local buckling

Note To start with θ angle will be chosen. As spacing has already been chosen for component members, C becomes fixed this C check for local buckling criteria

- If it is not met θ can be increased.

- Effective length of lacing bar



l_{eff} = l for single lacing (rivetted)

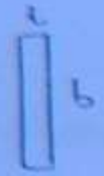
= $0.7 l$ for double lacing (when there is connection at the overlap)

= $0.7 l$ for welded lacing

$l \rightarrow$ Distance b/w inner ends of rivet or weld

- Slenderness ratio of lacing bar should not be more than 145

$$\frac{l_{eff}}{r_{min}} \not> 145$$



$$r_{min} = r_{YY} = \sqrt{\frac{I_{YY}}{A}} = \sqrt{\frac{bt^3}{bt\,12}} = \frac{t}{\sqrt{12}}$$

$$\frac{l_{eff}}{t/\sqrt{12}} \not> 145$$

108

Min width of lacing bar

Nominal dia of rivet (mm)	Min width (mm)
22	65
20	60
18	55
16	50

{ 3 times the nominal dia of rivel ; [illegible]

Min thickness of lacing bar

Min thickness $\not<$ $\frac{l}{40}$ for single lacing

$\frac{l}{60}$ for double lacing

Force for which lacing should be designed.

The lacing should be designed for shear force in colu[illegible] arising due to

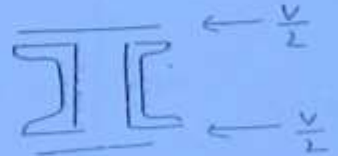
a) B.M.

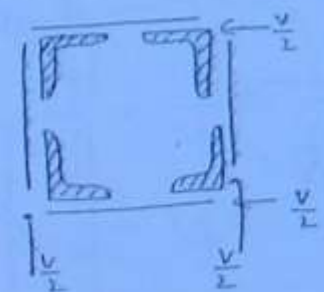
b) Lateral loading

c) Incidental Eccentricity

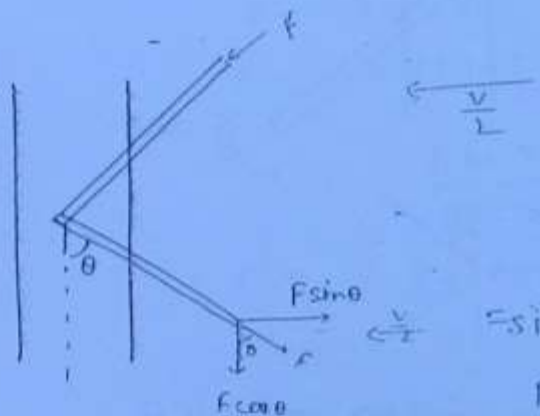
↳ unavoidable

For incidental eccentricity transverse shear = 2.5% of axial load in the member is taken into account.
This shear is divided equally between all transverse lacing system in parallel planes.





109

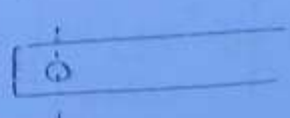


$F\sin\theta = \frac{V}{2}$

$F = \frac{V}{2\sin\theta}$

Lacing bar will be designed for tension and compression the force will be taken equal to $\left(\frac{V}{2\sin\theta}\right)$

Check for tension

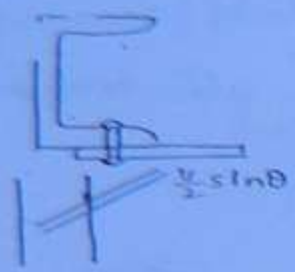


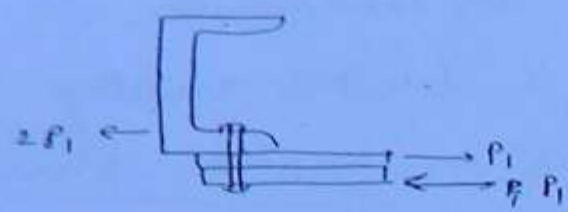
$(b-d')\,t\,\sigma_{at} \geq \frac{V}{2\sin\theta}$

Check for compression

$$\frac{\frac{V}{2\sin\theta}}{b \times t} < \sigma_{ac} \left(\text{corresponding to } l_{eff}/r_{min},\ r_{min} = \frac{t}{\sqrt{12}} \right.$$

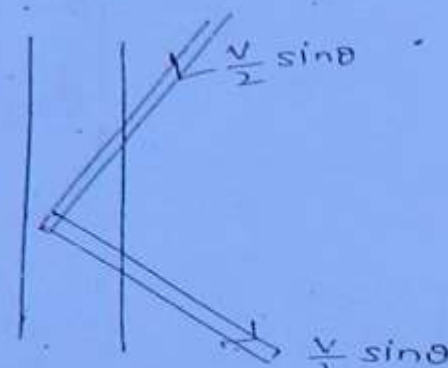
Design for connection of lacing bar

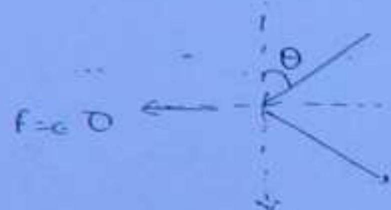




Single shear

$$\frac{V}{2\sin\theta} \le R_v$$

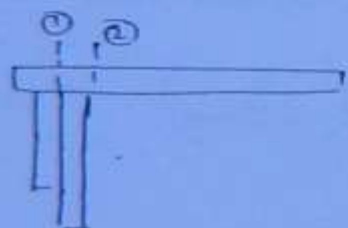




$$\frac{V}{2}\frac{\cancel{\sin\theta}}{\sin\theta}\cos\theta + \frac{V}{2}\frac{\cancel{\sin\theta}}{\sin\theta}\cos\theta$$

$$V\cot\theta \le R_v$$

Note



Shear at secⁿ 1 will correspond to $\frac{V}{2\sin\theta}$

Shear at secⁿ 2 " " $V\cot\theta$

Normally check is done for $V\cot\theta$ However if $\frac{V}{2\sin\theta}$ becomes more than $V\cot\theta$ means

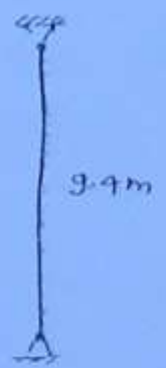
$\frac{V}{2\sin\theta} > V\cot\theta \Rightarrow \theta > 60°$

Secⁿ ① will become critical here

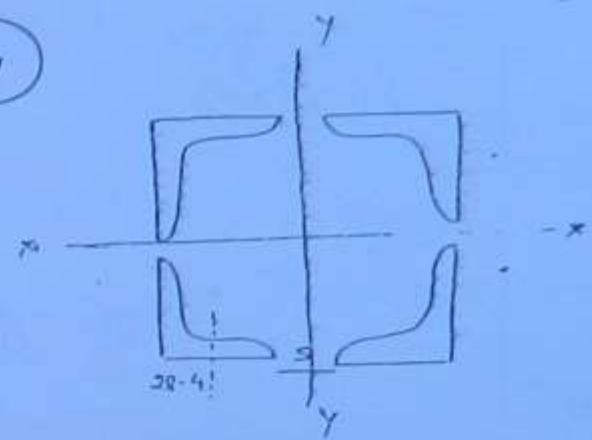
Check should be done for $\frac{V}{2\sin\theta}$ force but normally θ is around 45° so $(V\cot\theta)$ governs.

NC

9.8



(7.1)



$P = 750\,kN$

$A = 4 \times 1903 = 7612\ mm^2$

$\sigma_{ac} = \frac{750 \times 10^3}{7612} = 98.13\ N/mm^2$

$\lambda_{req} = 82.247$

$\lambda_{req} \le 82.247$

$\frac{l_{eff}}{r_{min}} < 82.247$

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

$r_{min} \ge 114.29\ mm$

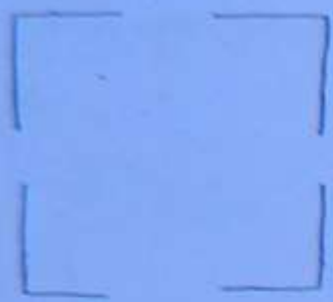
$I = A r^2$

$= (4 \times 19.03)\ 114.29^2 = 99.43 \times 10^6\ mm^4$

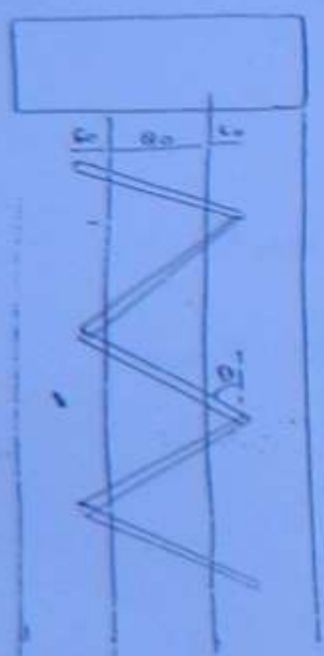
$I_{min} = 4 \times \left[177 \times 10^4 + \left\{(100 - 28.4) + \frac{s}{2}\right\}^2 \times 1903\right] = 99.43 \times 10^6$

$s = 77.09\ mm$

Adopt $s = 80\ mm$



(112)



taking $g = 40$ mm $\left(> \overset{c.c}{C_{yy}} \right)$

$$\tan\theta = \frac{200}{c/2}$$

$$c = \frac{400}{\tan\theta}$$

assuming $\theta = 45$

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

Check for local buckling

$$\left.\frac{c}{r_{min}}\right)_{indi} \not> 50$$

$$\not> 0.7\,\lambda_{whole}$$

$$\left.\frac{400}{r_{vv}}\right) \not> 50$$

$$\frac{400}{19.4} = 20.62 \not> 50 \quad \underline{ok}$$

Note: for slenderness ratio 82.247 spacing is 77.09

If spacing is increased r_{min} will increase hence λ will fall thus for spacing equal to λ would be less than 82.247 if local buckling criteria is satisfied for larger slenderness ratio

(113)

Design of lacing.

Local buckling criteria is satisfied for, $\theta = 45$ in single lacing hence single lacing is sufficient.

$$l = 200\sqrt{2} = 282.843 \text{ mm}$$

Dia of rivet : $6.05\sqrt{t} = 6.05\sqrt{10} = 19.13$

$= 20$ mm

Min width of lacing = 60 mm

min thickness $= \frac{l}{40} = \frac{282.843}{40} = 7.07$

adopt 8mm

check for slenderness ratio of lacing

$$S.R.)_{lacing} \ngtr 145$$

$$\frac{l_{eff}}{t/\sqrt{12}} \ngtr 145$$

$$\frac{282.843\sqrt{12}}{8} \ngtr 145$$

$$122.47 \ngtr 145$$

ok

check for tension

114

$V = 2.5\%$ of P

$= \frac{2.5}{100} \times 750 = 18.75$

Tensile force in lacing $F = \frac{V}{2\sin\theta} = 13.25$ KN

Tearing strength > 13.25

$(60 - 21.5) \times 8 \times 0.6 f_y > 13.25$

46.2 KN > 13.25 KN

OK.

check for compression

$\lambda = 122.5$

λ	σ_{ac}
120	67.1
130	59.7

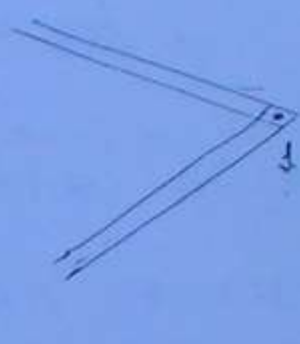
$\sigma_{ac} = 65.27$ N/mm^2

$P' = \sigma_{ac}\ b \cdot t$

$= 65.27 \times 60 \times 8 = 31.33$ KN > 13.25 KN

Safe in compression

Design for connection.



$V\cot\theta = 18.75$ KN

~~Rivet value = Min(shearing, bearing)~~

~~$= \min\left(\frac{\pi d'^2}{4} \times 100,\ 300 \times 21.5 \times 8\right)$~~

Adopting power driven shop rivet $= \min\left\{36.3\ \text{KN},\ 51.6\ \text{KN}\right\}$

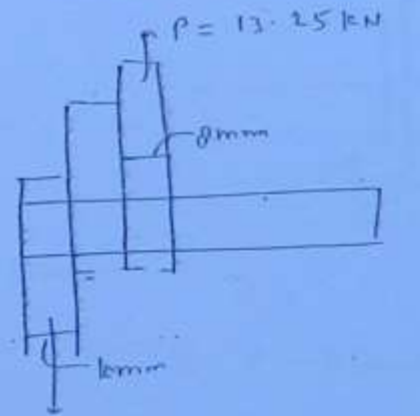
shearing strength of rivet = 36.3 KN

115

No. of rivet $= \frac{18.75}{36.3}$

Adopt 1 rivet

Check for bearing

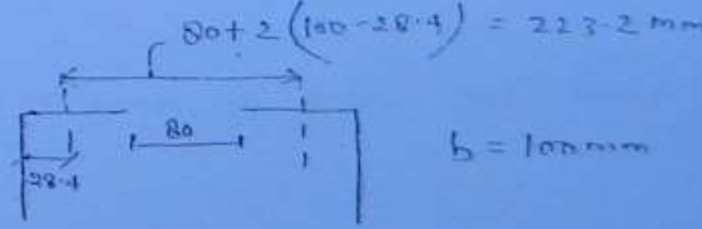


$64.5 > 18.75$ OK.

$51.6 > 13.25$ OK

} safe in bearing.

Design of End batten.



b = 100 mm

d → Eff. depth of end batten

$d \nless a \nless 223.2$

$d \nless 2b \nless 200$

$\Rightarrow$ Adopt d = 225 mm

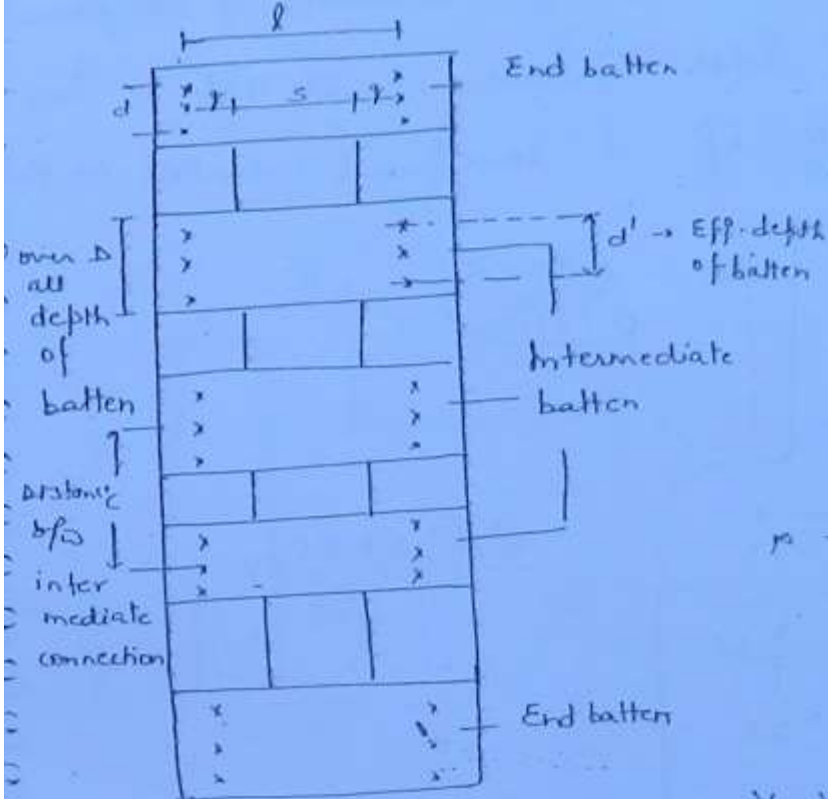
$D = d + 2$ edge distance $= 225 + (2 \times 20) \times 2 = 305$ mm

Thickness $= \frac{l}{50} = \frac{200}{50} = 4$

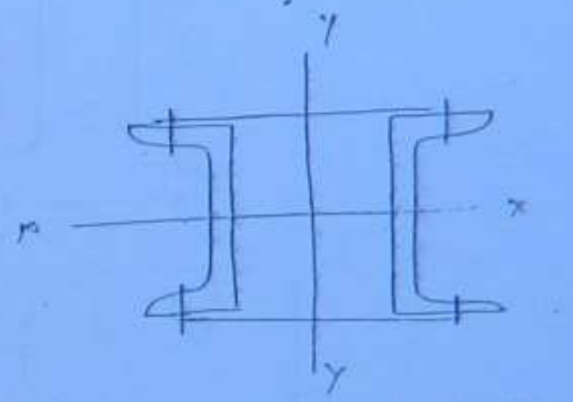
Adopt 8 mm

116

Design of End batten



The effective length of batten column is taken 10% more than that of laced column



$r_{yy} > r_{xx}$

y - y plane is the plane ⊥ to the plane of batten

Batten on opposite faces should be mirror image. No of batten should be such that it divides the column longitudinally in not less than 3 parts

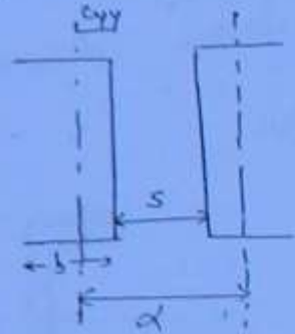
Effective length of battened column is taken 10% more

Design specification

$$\frac{c}{(r_{min})_{one}} \not> 50$$

$\not>$ 0.7 d whole

Effective depth of intermediate batten should not be less than $\frac{3}{4}$ times the distance b/w centroid of component member, the eff. depth however should not be less than twice the width of 1 component member in the plane of batten



118

$$\alpha = s + 2C_{yy}$$

$$\boxed{\begin{aligned} d' &\nless \tfrac{3}{4}\alpha \\ &\nless 2b \end{aligned}}$$

- For End batten

Effective depth of end batten $d \nless \alpha$

$\nless 2b$

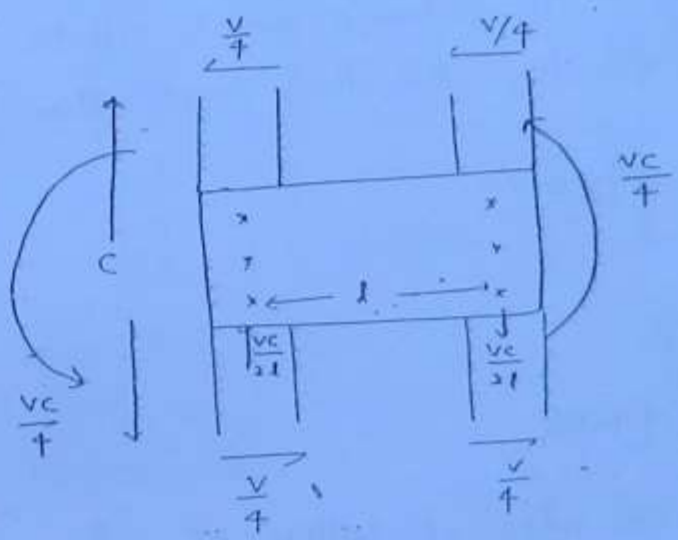
- The thickness of batten should not be less than $\frac{l}{50}$ where l is the distance b/w innermost connecting line of rivet or weld

$$l = (s + 2g)$$

Check for safety

Batten should be designed to carry B.M. and shear arising due to transverse shear. Transverse shear for incidental

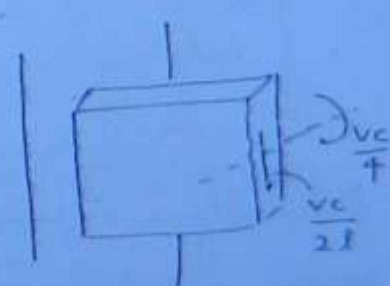
Eccentricity $= V = \frac{2.5}{100} P_{axial}$



Hence batten plate will be designed for

$$\text{Shear} = \frac{Vc}{2l}$$

$$\text{Moment} = \frac{Vc}{4}$$



Check for shear

Permissible average shear stress $= .4\, f_y$

$$\frac{\left(\frac{Vc}{2l}\right)}{Dt} \le 0.4\, f_y$$

Check for bending

Permissible av. bending stress $= 185\ N/mm^2$

i.e. $$\frac{\left(\frac{Vc}{4} \times \frac{D}{2}\right)}{\left(\frac{tD^3}{12}\right)} \le 185\ N/mm^2$$

or

$165\ N/mm^2$.

Design of connection will be done as eccentric connection in which rivet group is subjected to transverse shear of $\left(\frac{Vc}{2l}\right)$ and torsional moment $\left(\frac{Vc}{4}\right)$

From this torsional moment ($\Rightarrow$ torsional shear) and transverse shear, shear force in extreme rivet will be found out and this should be less than rivet value for safety of connection.

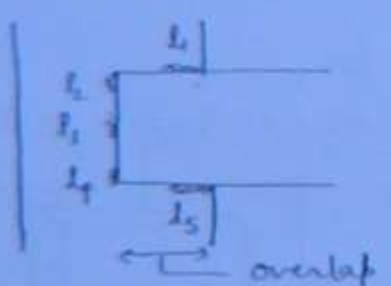
120

Welded Connection

overlap $\nless 4t$

$t \rightarrow$ thickness of plate

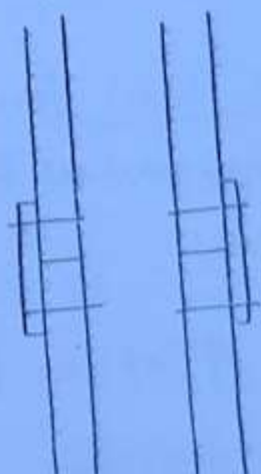
Total length of weld at the edge of batten $\nless \frac{D}{2}$



$$(l_1 + l_3 + l_4) \nless \frac{D}{2}$$

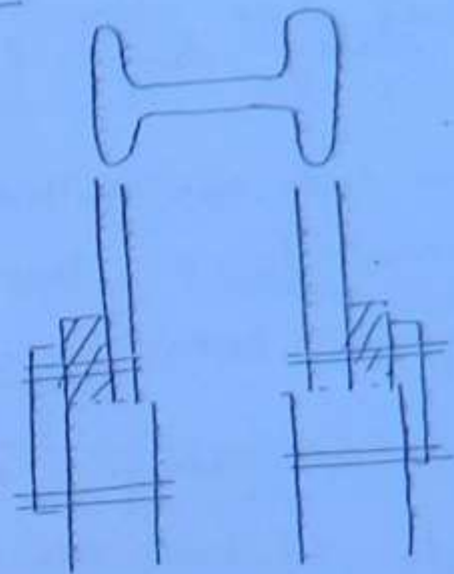
Practically we provide continuous connection hence these recommendation will not be required to be checked.

Column splikes



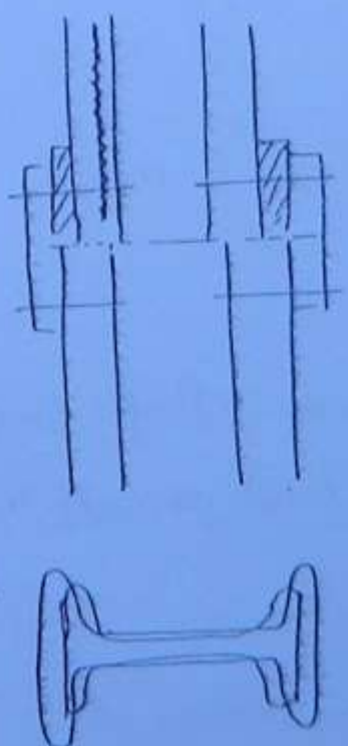
case(a)

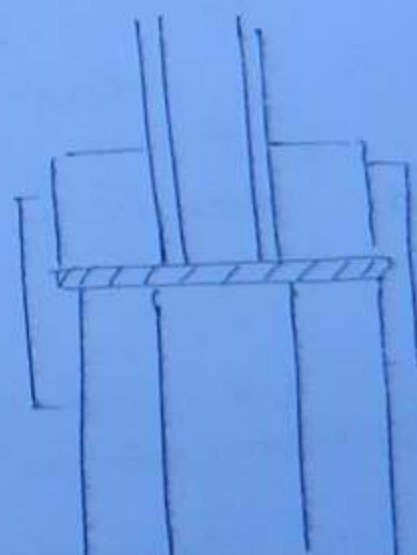
Top storey column & bottom storey column are of same size



case(b)

top & bottom storey column are of different size but top storey column flange resting completely on bottom storey flange





In case of complete bearing of top storey column flange on bottom storey column flange the design is as follows

If the column ends are milled all of the direct force will be assumed to be transferred through direct bearing of top storey to bottom storey. * (IS code)

122

If moments are not acting. the purpose of splice plate will only be to ~~wl~~ hold the two column together

Not IS code learn — However to find out the secn of splice plate it will be assumed that 50% of the direct load is transferred through splice plates

Hence force on one splice plate $= \frac{P}{4}$

$$\text{Area of splice plate} = \frac{P/4}{(\sigma_{ac})}$$

As there is no buckling i.e. $\sigma_{ac} = 0.6 f_y$

$$b \times t = \frac{(P/4)}{0.6 f_y}$$

$b \to$ width of column flange

- If the column ends are not milled design force $= \frac{P}{2}$ for one splice

If the column is subjected to direct load as well as moment

Case 1

When column ends are milled

All direct force will be assumed to transferred through direct bearing. Splice plate however will be designed for forces generated due to moment. Hence

design force $= \left(\frac{M}{l}\right)$

$l \rightarrow$ spacing b/w splice plate.

123

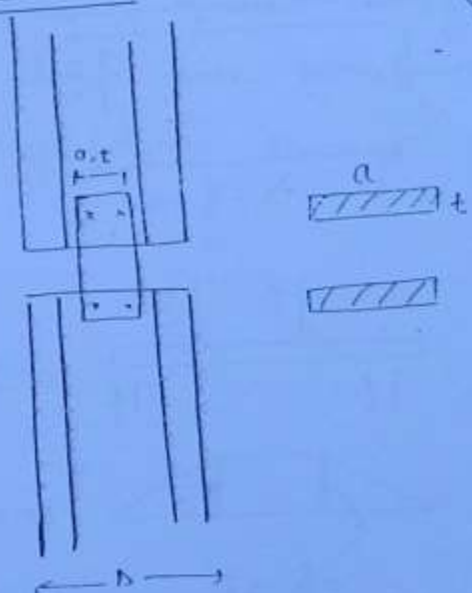
(Case. b)

When column ends are not milled

Design force $= \left(\frac{P}{2} + \frac{M}{l}\right)$ in compression.

$= \left(\frac{P}{2} - \frac{M}{l}\right)$ in tension. (When value becomes (-)ve)

Web splice



$$\frac{V}{2(a)t} \leq 0.4\, f_y$$

Web splice are provided to resist shear in the column

Max space available for providing splice plate is

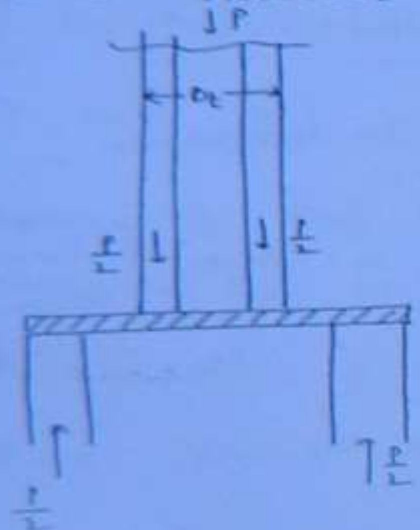
$(D - 2h_2)$

a should be chosen smaller than this

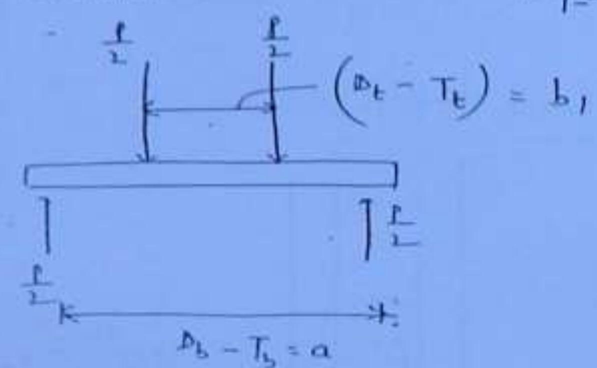
Rivets here will be designed to be in double shear.

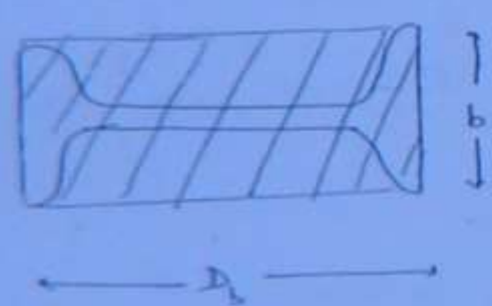
* When top storey column flange is not resting on the bottom storey column flange:

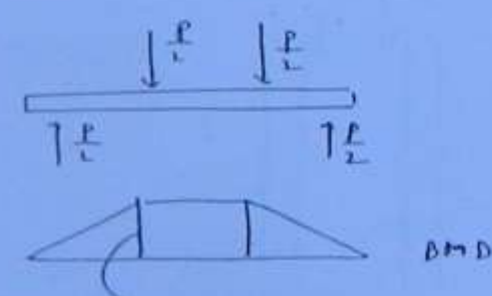
In this case the total tot. load is assumed to have been transferred through the column flanges to the base plate and then through the base plate to the bottom storey column flanges. The connection will be designed by usual methods. However the base plate needs to be designed.



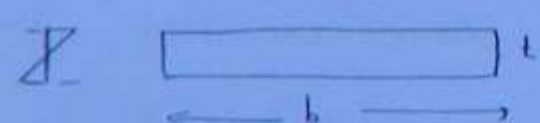
$D_t \rightarrow$ Depth of top storey column
$T_t \rightarrow$ Thickness " " flange







$$M_{max} = \frac{P(a-b)}{4}$$



Per Bending stress $\underbrace{\sigma_{pt}}_{\text{165 for flanged, 185 for solid sec}^n} \times \underbrace{Z}_{M.O.R.} = \dfrac{P(a-b)}{4}$

$$\sigma_{per} \cdot \frac{bt^2}{6} = \frac{P(a-b_1)}{4}$$ from here calc. t

125

Note:
i) If packing is required, thickness of packing will be equal to the gap to be filled

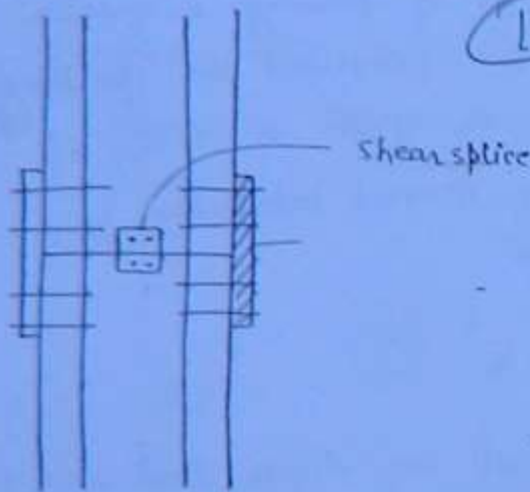
ii) In this case if thickness of packing is greater than 6mm additional rivets will be provided on packing extension and no. of such rivets is equal to 2.5% of the no. of rivets obtained from normal calculation per 2 mm thickness of packing.

- In case of partial bearing all the direct load will be assumed to have been transferred from top storey column to bottom storey column through splice plate.

Question.

Q. 2 ISHB 300 sections are spliced using compression splice the applied load is 300 KN. S.F. is 75 KN and applied moment is 20 KNm assume column ends to be not milled Design column splice

126



As the column ends are not milled all of the direct force will be transfered through the splice plate.

Hence force in splice plate $= \frac{P}{2} \pm \frac{M}{l}$

Adopt $l = 300$ mm

to be on safer side

The forces in splice are

$\frac{P}{2} + \frac{M}{l}$ $\qquad\qquad$ $\frac{P}{2} - \frac{M}{l}$

$\frac{300}{2} + \frac{20}{.3}$ $\qquad\qquad$ $\frac{300}{2} - \frac{20}{.3}$

216.07 KN comp. $\qquad\qquad$ 83.33 KN comp.

Design force for column splice = 216.67 KN

Permissible stress in compression in splice = $0.6 f_y$

= b/c as column

127

splice will not buckle individually

area required = $\dfrac{216.67 \times 10^3}{0.6 \times 250}$ Assuming $f_y = 250$

$= 1444.47 \text{ mm}^2$

width of splice plate is taken as the width of column flange

$$b \times t = 1444.47$$

$$b = 250$$

$$\text{so } t = \frac{1444.47}{250} = 5.7$$

adopt $t = 8$ mm

Length of column splice will depend on no. of rivets

Design of connection.

Rivets will be designed for force of 216.67 KN

Adopting the dia of rivet as 18 mm and choosing power driven field driven

Per stress in rivet in shear = 90 MPa

bearing : 270 MPa

Rivet value min {shear, Bearing}

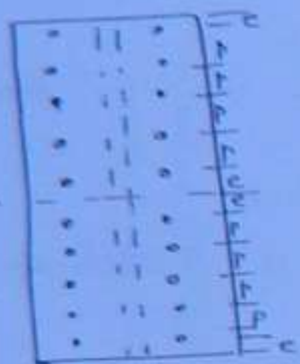
$$= \frac{\pi}{4} \times 19.5^2 \times 90 , \quad 19.5 \times 8 \times 270$$

$$\min \{ 26.87 \text{ KN} , \quad 42.12 \text{ KN} \}$$

$\dot{R}_v = 26.87$ kN

No of rivet $= \dfrac{216.67}{26.87} = 8.06$

Adopt 10 No. (even no. due to symmetry)



128

$$l = 8p + 4e$$
$$= 8 \times 2.5d + 4 \times 2d'$$
$$= \cancel{28d = 28 \times 18}$$
$$= \cancel{504 \text{ mm}}$$
$$= 20 \times d + 8d'$$
$$= 520 \text{ mm}$$

splice plate on the other side will also be of same x-sec^n & connection detailing will also be same.

Design for shear splice

Area required to resist shear

$$= \frac{75}{0.4 f_y}$$
$$= \frac{75 \times 10^3}{100} = 750 \text{ mm}^2$$

$2at \geq 750$

adopt width of splice 125 mm

$$t_{req} = \frac{750}{125}$$
$$\geq 3 \text{ mm}$$

Adopt thickness of plate = 6 mm

Rivets are in double shear hence

129

$$R_v = \min\left\{ 2 \times \frac{\pi d'^2}{4} \times 90,\ 6 \times d \times 270 \times 7.6 \right\}$$

$$= \min\{53.75,\ 40.014 \text{ KN}\}$$

$$R_v = 40.014 \text{ KN}$$

$$\text{No. of rivet} = \frac{75}{40.014} \simeq 1.8$$

Adopt 2 rivet.

Note:
ISSC → Is a I section

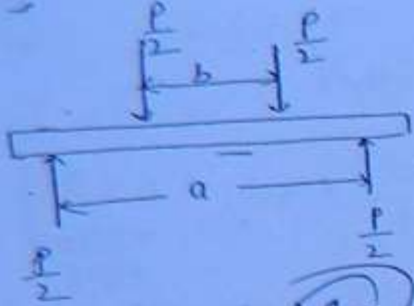
Q. An ISHB 300 in lower story and ISHB 200 in the upper story has been used

	ISHB 300	ISHB 200
D	= 300	200
B	= 250	200
T	= 10.6	9
t_w	= 7.6	6.1

Column load is 650 KN design the base plate placed between top & bottom story column.

Sol. The top story column flange is not resting at all on the bottom story column flange hence we require the base plate.

Size of the base plate will be = 300 × 250 × t



130

$a = 300 - 10.6 = 289.4$ mm

$b = 200 - 9 = 191$ mm

$P = 650$ kN.

$$\frac{P\,(a-b)}{4} \le M.O.R$$

Adopting $\sigma_{per} = 185 \frac{N}{mm^2}$ for solid plate

$$10^3 \times \frac{650}{4}(289.4 - 191) \le \sigma_{per} \cdot Z$$

$$\le \cancel{0.66 \times 250}\ 185 \times \frac{250 \times t^2}{6}$$

$$t \geq 45.54 \text{ mm}$$

Adopting $t = 50$ mm

Column Bases

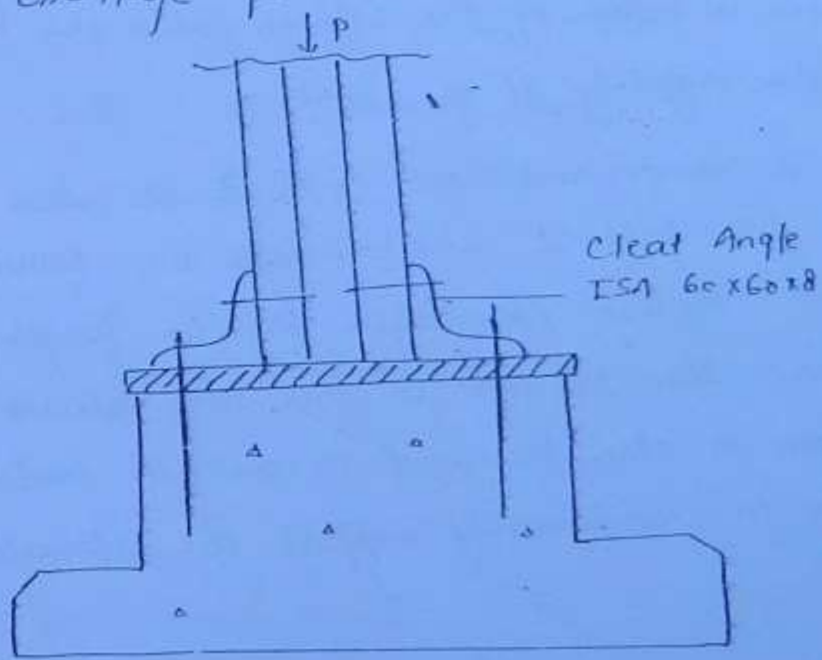
Column Bases

(131)

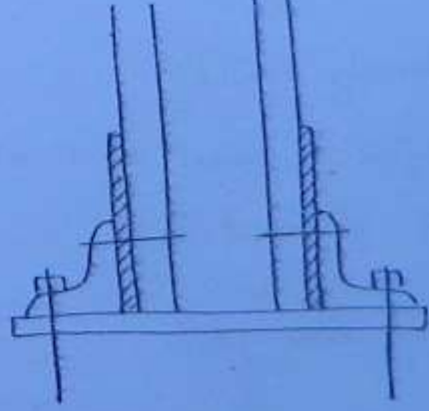
If steel column is directly placed on a concrete pedestal the concrete will get crushed. to safeguard against crushing of concrete, area of load transferred should be increased so that the resulting stress on concrete is less than the permissible stress in direct compression in concrete.

The various types of column bases are.

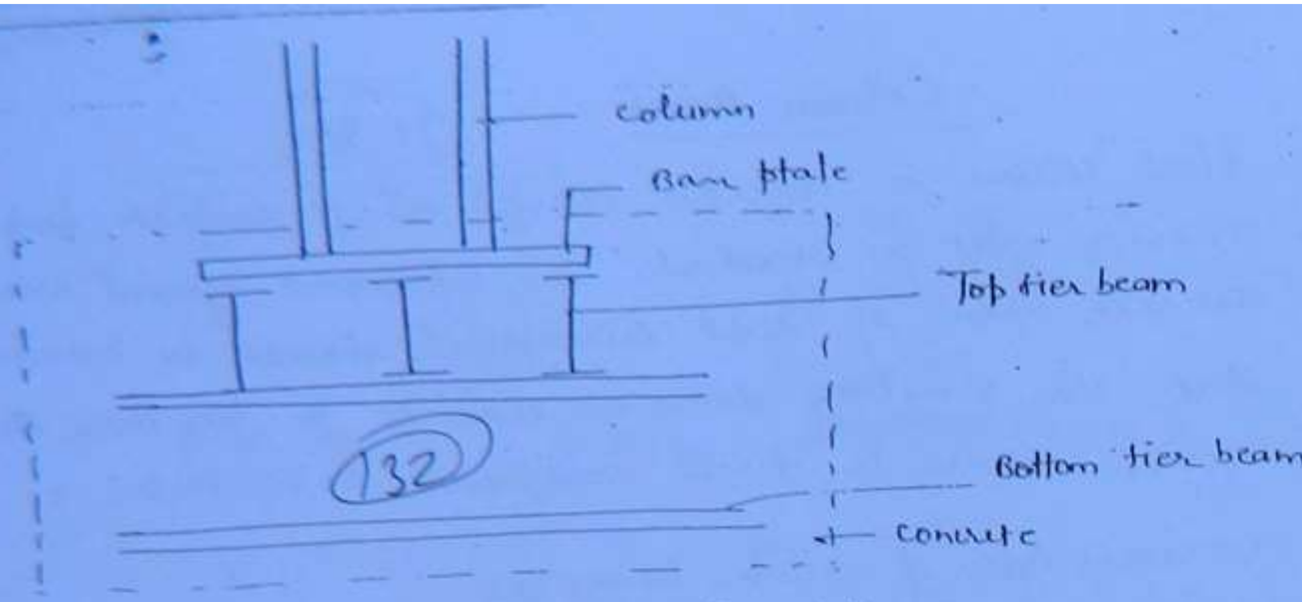
a) Slab Base
b) Gussetted base
c) Grillage foundation



(slab base)



(Gussetted base)



Grillage Foundation

Note: Gusseted base is better if the column loads are heavy (as compare to slab base). It provides greater rigidity at the joint.

- When the load is heavy and soil is weak or when the foundation is to be laid at shallow depth we provide grillage foundation. Grillage foundation provides larger area at the base hence the pressure on the soil reduces. Grillage foundation is also provided in case of temporary construction & or to temporarily support the structure.

When the column load is purely axial

Assuming that column transfers a load of P at the base plate size of base plate will be decided as

$$\frac{P}{\sigma_c} = \text{Area of base plate}$$

σ_c . Per. stress in concrete in compression

Note:- If applied load is given we should take into account the dead wt. also hence the design force for base plate will be $1.1 \times P$ where P = Applied load on column and 1.1 takes into account the dead wt. which is taken as 10% of applied load.

For purely axially loaded column

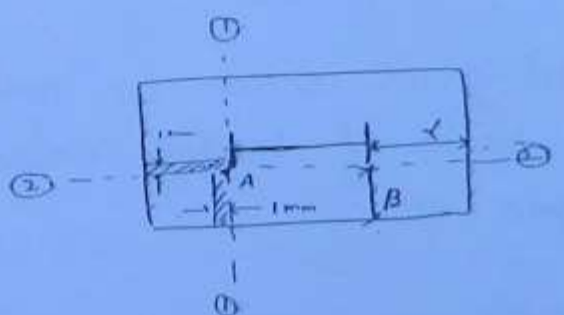
$$t = \sqrt{\frac{3w}{\sigma_{bs}}\left(a^2 - \frac{b^2}{4}\right)}$$

a Larger overhang
b Smaller overhang

$t \rightarrow$ Thickness of base plate

σ_{bs} - Per. bending stress in base plate ($185\ N/mm^2$)

b & a are smaller and larger overhangs



Larger (α & A) = a
Smaller (α, B) = b

$$w = \frac{\text{load on base plate}}{\text{Area of base plate}}$$

$$M_{11} \text{ at } A = \frac{wa^2}{2}$$

$$M_{22} = \frac{wb^2}{2}$$

$$\sigma_{11\,A} = \frac{M_{11} \times \frac{t}{2}}{1 \times \frac{t^3}{12}} \quad \& \quad \sigma_{22\,A} = \frac{M_{22}\ t/2}{1 \times \frac{t^3}{12}}$$

$$\frac{\sigma_A}{E} = \frac{\sigma_{11A}}{E} - \frac{\mu\,\sigma_{22A}}{E}$$

$$\sigma_A = \frac{(M_{11} - \mu M_{22}) \times 6}{t^2} = \frac{\left(\frac{w a^2}{2} - \frac{\mu w b^2}{2}\right) 6}{t^2}$$

$$= \frac{3w(a^2 - \mu b^2)}{t^2}$$

134

$$\sigma_A \leq \sigma_{bs} \Rightarrow \quad t \geq \sqrt{\frac{3w}{\sigma_{bs}}(a^2 - \mu b^2)}$$

Assuming $\mu = 0.25$ for steel

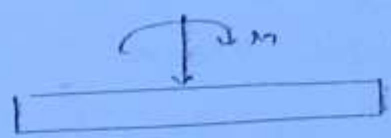
$$t_{min} = \sqrt{\frac{3w}{\sigma_{bs}}\left(a^2 - \frac{b^2}{4}\right)}$$

Note: The plate is critical in bending at section 1-1 hence thickness is decided on the basis of bending however check for shear can be done as

$$\frac{w \times 1 \times a}{1 \times t} \leq 0.4\, f_y$$

If the column is subjected to load as well as moment Net base pr. is to be calculated and bending stress at critical section is evaluated. This bending stress is shown to be less than the permissible bending stress this analysis helps us in finding out thickness of base plate

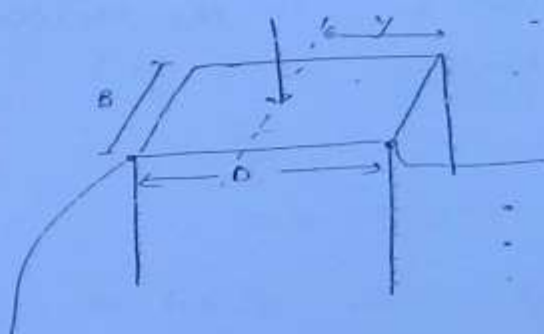
Slab base subjected to Moment and axial loading →



$\frac{P}{A}$ effect of axial loading $= \frac{P}{BD}$

+

$\frac{My}{I}$ effect of bending



Net stress comp $= \frac{P}{BD} + \frac{6Pe}{BD^2}$

$\frac{P}{BD}\left(1 + \frac{6e}{D}\right)$

Net stress $= \frac{P}{BD}\left(1 - \frac{6e}{D}\right)$

If $e < \frac{D}{6}$

$\frac{6e}{D} < 1$

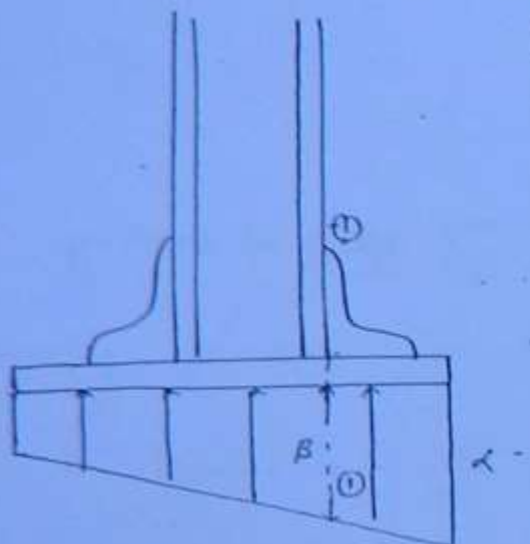
For design $\frac{P}{BD}\left(1 + \frac{6e}{D}\right) \leq \sigma_c$

↳ per. stress in comp.

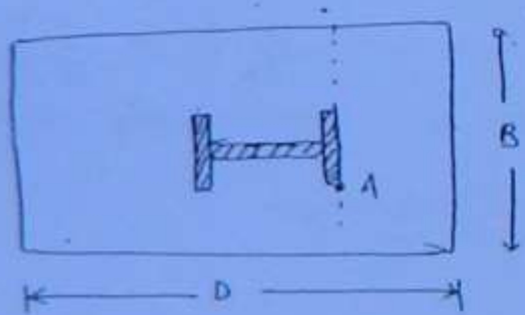
using this formula we can choose 1 & find out other
(choose either of B & D & calc. other)

thus size of base plate will be known -

Thickness of base plate



section 1 - 1 is the critical secⁿ for bending



$$\sigma_A = \sigma_{11} - \mu \sigma_{22}$$

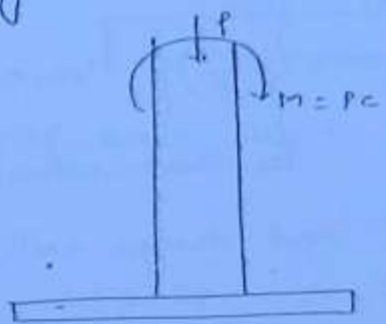
$$= \frac{M_{11} \frac{t}{2}}{t^3/12} - \frac{\mu M_{22}\, t/2}{1 \times t^3/12}$$

$$\sigma_A = \frac{6}{t^2}\left(M_{11} - \mu M_{22}\right) \leq \sigma_{bs}$$

$\therefore$

$$t \geq \sqrt{\frac{6\left(M_{xx} - \mu M_{yy}\right)}{\sigma_{bs}}}$$

If value of μ is not given we can neglect it by doing so we will be on safer side.



04/10/11

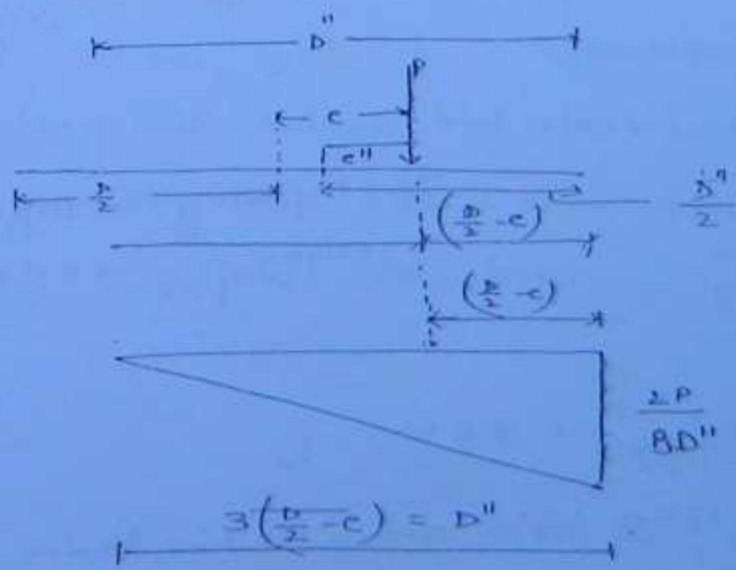
$$\text{Max. stress} = \frac{P}{BD}\left(1 + \frac{6e}{D}\right)$$

$$\text{Min stress} = \frac{P}{BD}\left(1 - \frac{6e}{D}\right)$$

If $e > \frac{D}{6}$

Min stress $= \frac{P}{BD}\left(1 - \frac{6e}{D}\right)$ becomes $(-)$ve $\Rightarrow$ Tensile

Due to tensile stress there will be loss of contact b/w base plate and concrete. In the final length of contact situation minimum stress will be zero. For this



Hence final length of contact $= 3\left(\frac{D}{2} - e\right)$

$$e = \frac{M}{P}$$

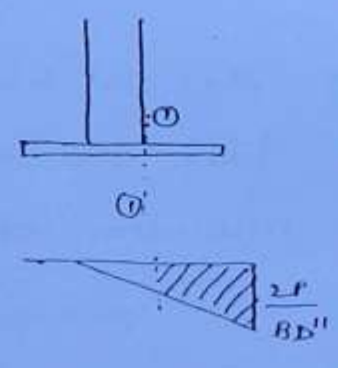
and Max. comp. stress $= \dfrac{2P}{3B\left(\frac{D}{2}-e\right)} = \dfrac{2P}{B \cdot 3\left(\frac{D}{2}-e\right)}$

138

$e > \frac{D}{6}$

In the starting e will be known $\left\{\left(\frac{M}{P}\right)\right\}$ Hence D will be chosen such that $e < \frac{D}{6}$. In that situation stre will be compressive through out and design will be done as discussed in previous case.

However if D is given, then if $e > \frac{D}{6}$, there would be loss of contact. Hence final length of contact and max^m compressive stress will be calculated. From this stress distribution max^m bending stress at critical sec^n 1-1 will be calculated and should be $\leq$ per. Bending stress



P. 41
Q. 11

SC 200

$P = 700$ kN

$M = 55$ kNm

$\sigma_c = 3.75$ MPa

$f_y = 250$ MPa

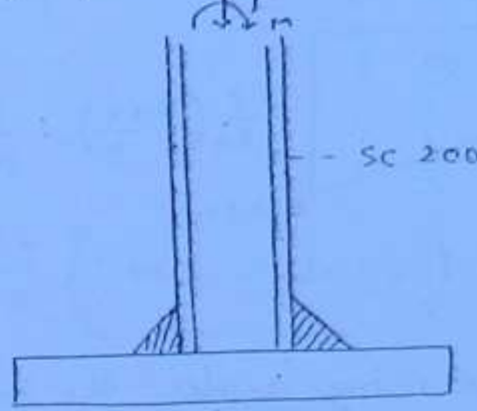
$T = 15$ mm

Stren = 1mm weld / mm = 76 N

$f_{bt} = 0.7 f_y = 175 \frac{N}{}$ perm. stress in bending tension

Sol. Neglecting the self wt. of column

Shearing force that can be resisted by weld $= 5.2 \times 76$



132

Design of base plate

$$e = \frac{M}{P} = \frac{55}{700} = 78.57 \text{ mm}$$

Choose $D > 6e$

> 471.43 mm

Choose $D = 500$ mm

Max. stress

$$\frac{P}{BD}\left(1 + \frac{6e}{D}\right) \leq \sigma_c$$

$$\frac{700}{B \times 500}\left(1 + \frac{6 \times 78.57}{500}\right) \leq 3.75$$

$B \geq 725.33$ mm

Adopt $B = 730$ mm

Note: by increasing the value of D overhang portion will (↑) but stress will ↓. However by choosing smaller D overhang will reduce but stress will ↑. Final selection is generally done based on economy i.e. vol. of steel $B \times D \times t$ should be less.

D = 500 m   P = 700
B = 730 mm   e = 78.57

stress DFS

$\frac{P}{BD}\left(1-\frac{6e}{D}\right)$

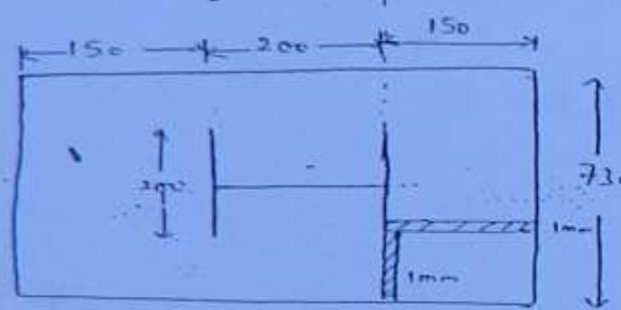
$$\frac{P}{BD}\left(1+\frac{6e}{D}\right) = \frac{700}{500\times730}\left(1+\frac{78.57\times6}{500}\right)$$

$$= 3.726\ \frac{N}{mm^2}$$

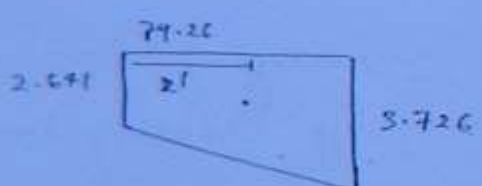
140

Note

If connection has been made using angles the length of plate should be choosen such that the connecting angle could be accomodated in the choosen length



$$\sigma_n = \frac{M_{11}\ t/2}{\frac{t^2}{12}} - \frac{ll\ M_{22}\ t/2}{L\ \frac{t^3}{12}} \leq \sigma_{bs}$$



$\bar{x} = 79.26$ mm

Force on 1 mm width $= \frac{1}{2}(2.641 + 3.726) \times 150 \times 1$

$F = 477.45$ N

$M_{11} = 477.45 \times 79.26 = 37848.275$ N mm

$$M_{22} = \frac{2.641 \times 265^2}{2} = 92728.6 \text{ Nmm}$$

$$\frac{6}{t^2}\left(M_{11} - \mu M_{22} \right) \leq 0.7 \times 250$$

$$t \geq 34.135 \text{ mm}$$

Similarly

$$\frac{6}{t^2}\left(M_{22} - \mu M_{11} \right) \leq 0.7 \times 250$$

$$t \geq 53.42 \text{ mm}$$

Let us adopt max. thickness

$$t = 55 \text{mm}$$

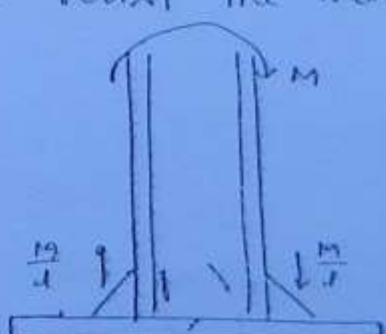
$B = 730$ mm

$D = 500$ mm

$t = 55$ mm

Design of weld.

As the column ends are milled all the direct load will be assumed to have been transferred through direct bearing on the base plate hence weld should be designed only to resist the moment.



$$\frac{M}{d} = \frac{55 \times 10^6}{200} = 275 \text{ kN}$$

$$l \times 5 \times 76 \geq 275 \times 10^3$$

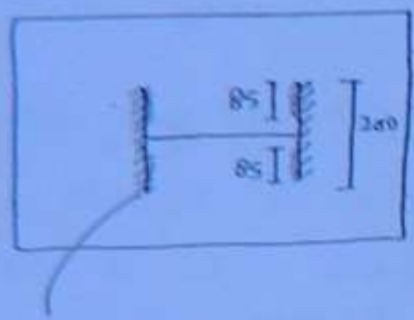
min. size - 10 mm

max. size - 15 - 1.5 = 13.5 mm max.

$l \; 5\times76 \geq 275\times10^3$

$l \geq 361.84\,mm$

Adopt $l = 370\,mm$



10 mm fillet weld

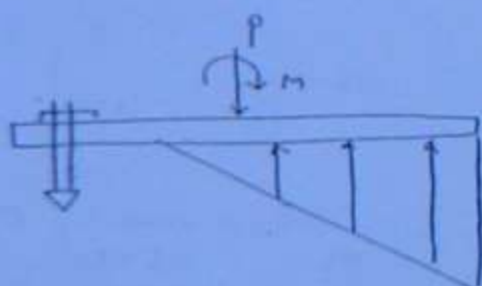
∴ there !

Note -

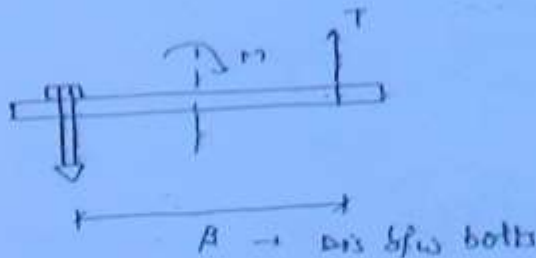
If there is tension at the base the bolt will come under tension and we need to design the bolt as well

Bolt can be designed if the tension in the bolt can be estimated.

Actual determination of T will be based on following fig.



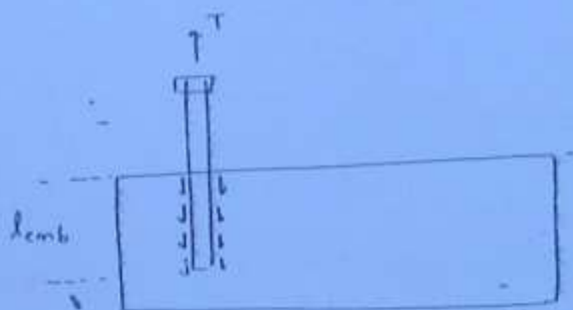
However a conservative estimate of tension in the bolt is as follows



$TB = M$

$$\boxed{T = \frac{M}{B}}$$

Once tension is known the dia of bolt and length of embedment can be calculated.



$$\pi\phi \times l_{emb} \times \sigma_a \geq T \quad \text{—— (1)}$$

↓

Per. adhesive stress b/w bolt & conc. depends on grade of steel

$$\frac{\pi\phi^2}{4} \cdot \sigma_t \geq T \quad \text{—— (ii)}$$

$\phi \rightarrow$ A Dia at the root of thread

From (i) & ii Dia of bolt & length of embedment can be calculated

- If a solid round steel column is supported over a square base plate then for purely axial load thickness is given by

$$\sqrt{\frac{90W}{16\sigma_{bs}} \frac{B}{(B-d_o)}}$$

:

$B \nless 1.5(d_o + 75)$ mm

$W \rightarrow$ KN

$t \rightarrow$ mm

$\sigma_{b} \rightarrow$ MPa

144

Design of Beam

For a beam to be safe

a) It should be safe in bending
b) Safe in shear
c) Deflection

} Primary criteria

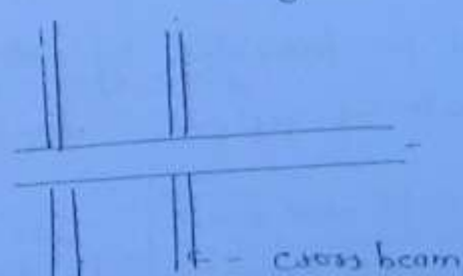
d) safe in local buckling of flange plate
e) Safe in web crippling
f) Safe in web buckling

} Secondary criteria

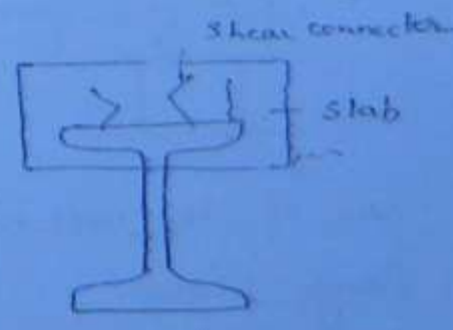
\# Laterally restrained and unrestrained Beams

Beam can buckle laterally if the compression flange is weak hence to safeguard against lateral buckling

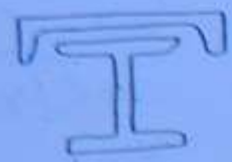
a) Beam can be laterally restrained by using cross beams



b) By inserting the compression flange inside the slab



c) By making compression flange heavy.



For laterally restrained beam

$\sigma_{bc} = \sigma_{bt}$ ⟶ per. stress in bending tension $= 0.67 f_y$

$= 165\ N/mm^2$

↓

per. stress in bending comp.

For laterally unrestrained beam $\sigma_{bc} < \sigma_{bt}$

Design of beam (laterally restrained beams)

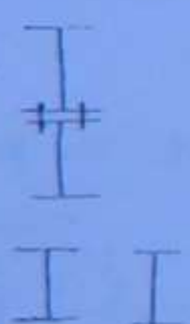
1.) Beam is designed for bending and checked for other criteria.

$$\frac{M_{max}}{\sigma_{bc} \text{ or } \sigma_{bt}} = Z_{req.}$$

Choose section that provides $Z > Z_{req.}$

When $Z_{req.}$ can not be provided by using single rolled secn we will adopt built up section.

Note

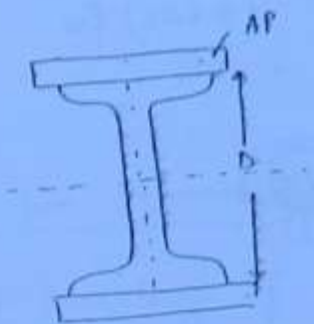


Most suitable arrangement for deflection criteria.

One of the most common built up section is I secn with flange plates

In I secⁿ and flange plate combination the largest I secⁿ available is used and deficiency in Z is met by using plates

$$I_{req} = I_{rolled} + I_{plates}$$



$$\frac{I_{req.}}{D/2} = \frac{I_{rolled}}{D/2} + \frac{I_{plate}}{D/2}$$

$$Z_{req.} = Z_{rolled} + \frac{A_p \times (D/2)^2 \times 2}{D/2}$$

$$\frac{Z_{req.} - Z_{rolled}}{D} = A_p$$

The above approach gives the approximate value of A_p the area of plate chosen is more than A_p.

Note The increase in A_p should cater for the area of hole on the tension flange because in tension net area is effective.

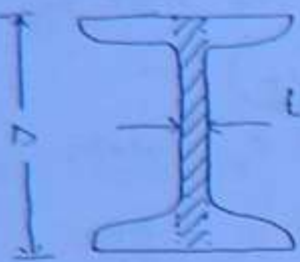
The built up beam should be checked for safety in bending as follows

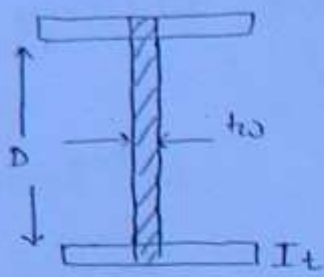
$$\frac{M\, Y_{comb}}{I_{gross}} \le \sigma_{bc}$$

$$\frac{M\, Y_{ten}}{I_{gross}} \times \left(\frac{\text{Gross area of tension flange}}{\text{Net area of tension flange}} \right) \le \sigma_{bt}$$

I_{gross} corresponds to NA of section neglecting holes

Check for shear



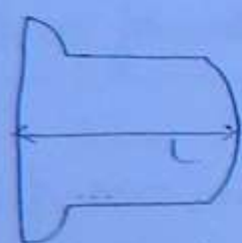


$$\frac{V_{max}}{(D+2t)\,t_w} \le 0.4 f_y$$

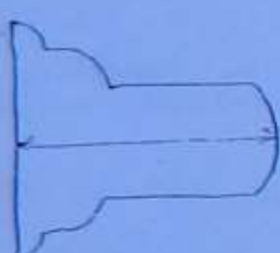
$V_{max} \to$ Max. s.f. in Beam

$$\frac{V_{max}}{D \times t_w} = \text{Av. Shear stress} \le \underbrace{0.4 f_y}_{\text{per. av. shear stress}}$$

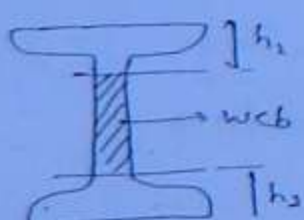
Note



Max. shear stress $\le 0.45 f_y$



Max. shear stress $\le 0.45 f_y$.



$$\frac{V_{max}}{(D-2h_2)\,t_w} = \text{Max. shear stress} \le 0.4$$

Check for Deflection.

Max. permissible deflection $= \dfrac{\text{span}}{350}$

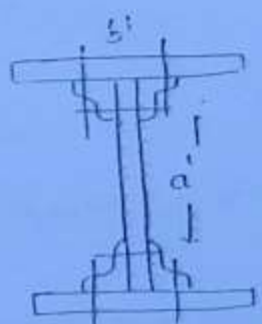
$$\Delta_{max} = \frac{5}{384}\frac{wl^4}{EI} \le \frac{l}{325}.$$

Check for secondary criteria.

149

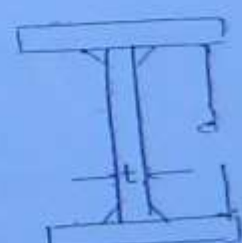
Local buckling of flange plate -

Note: Secondary criteria need not to be checked for rolled section because sections are designed in such a way that overall failure of the section takes place before the local failure has had the chance to take place

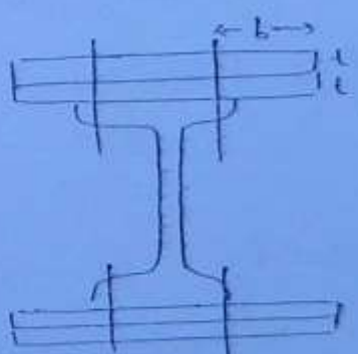


$$\frac{a'}{t} \not> 50$$

$$\frac{b'}{t_f} \not> 50$$



$$\frac{d}{t} \not> 50$$



$$\frac{b}{t} \not> 16$$

plates are simply supported placed one over the other

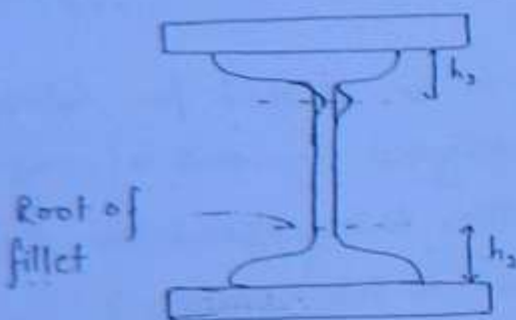
if however two plates are welded so that they behave as 1 unit

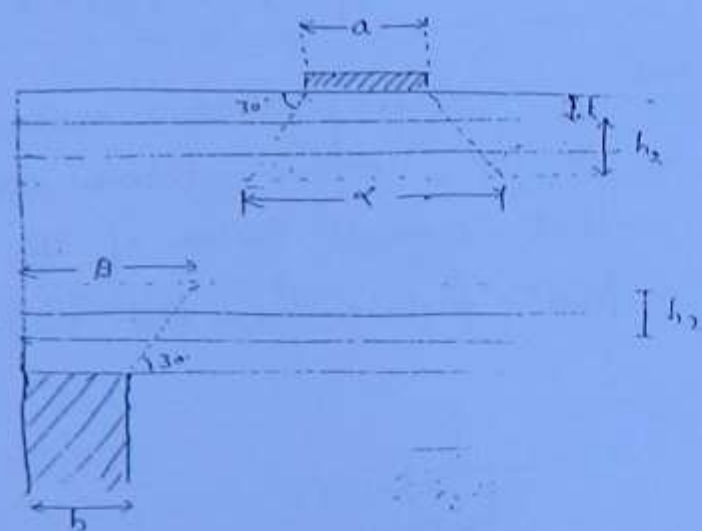
$$\frac{b}{2t} \not> 16$$

This criteria should be checked at the time of selection of plate only

Check for web cripling.

Web cripling takes place at the location of heavy point load The heavy point load can come in the span and at the support the point load are dispersed as shown below





for no web crippling

$$\frac{P}{\alpha\, t_w} \le \sigma_{br} \quad (\text{per. bearing stress})$$

Web cripling takes place at location where section provides least area of resistance such location is first encountered at the root of fillet. Hence web crippling takes place at the root of fillet.

For safety against web crippling, bearing stress at the root of fillet should be less than permissible bearing stress ($.75 f_y$)

$$\frac{P}{\left\{a + 2\sqrt{3}\left(h_2 + t\right)\right\} t_w} \leq \sigma_{br}$$

(151)

At support

$$\frac{R}{\left\{a + \sqrt{3}\left(h_2 + t\right)\right\} t_w} \leq \sigma_{br}$$

Note: If the beam is carrying UDL web crippling needs to be checked at the support.
The support width is decided based on the bearing failure of supporting structure as well as web crippling of steel section.

Web Buckling: Web buckling occurs due to heavy conc load, diagonal compression due to shear. To check for safety in web buckling

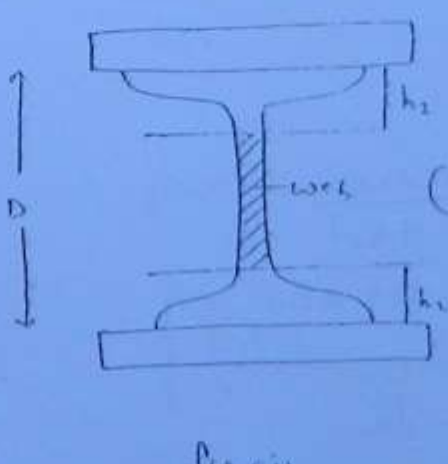


Fig (i)

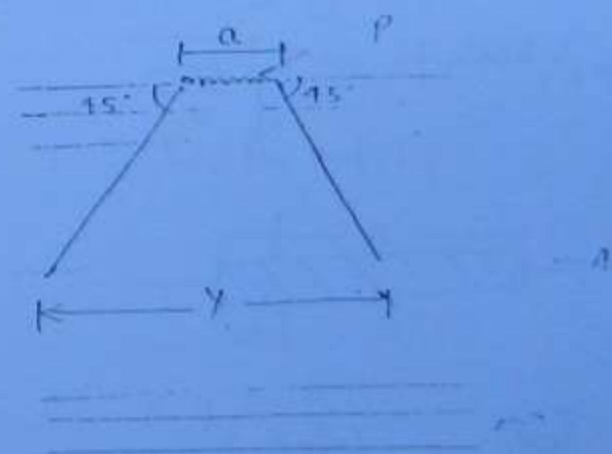


Fig (ii)

web buckling ↳ Area at N.A. = $Y t_w = A$

To check for web buckling web is treated as a column of slenderness ratio = $\frac{d_w \sqrt{3}}{t_w}$ where

$d_w = D - 2h_2$

t_w - thickness of web

The area resisting compression is calculated as follows (fig ii)

For safety against web crippling

carrying capacity = $\sigma_{ac} \times A \;> P$

$\sigma_{ac} \rightarrow$ Per. stress in axial compression obtained from slenderness ratio = $\frac{d_w \sqrt{3}}{t_w}$

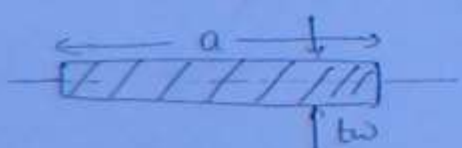
$A \rightarrow$ Applied load

Note

Note The web can be treated as a column which is fixed at the two flange ends hence

$$l_{eff} = \frac{d_w}{2}$$

$$r_{min} = \sqrt{\frac{I_{min}}{A}}$$



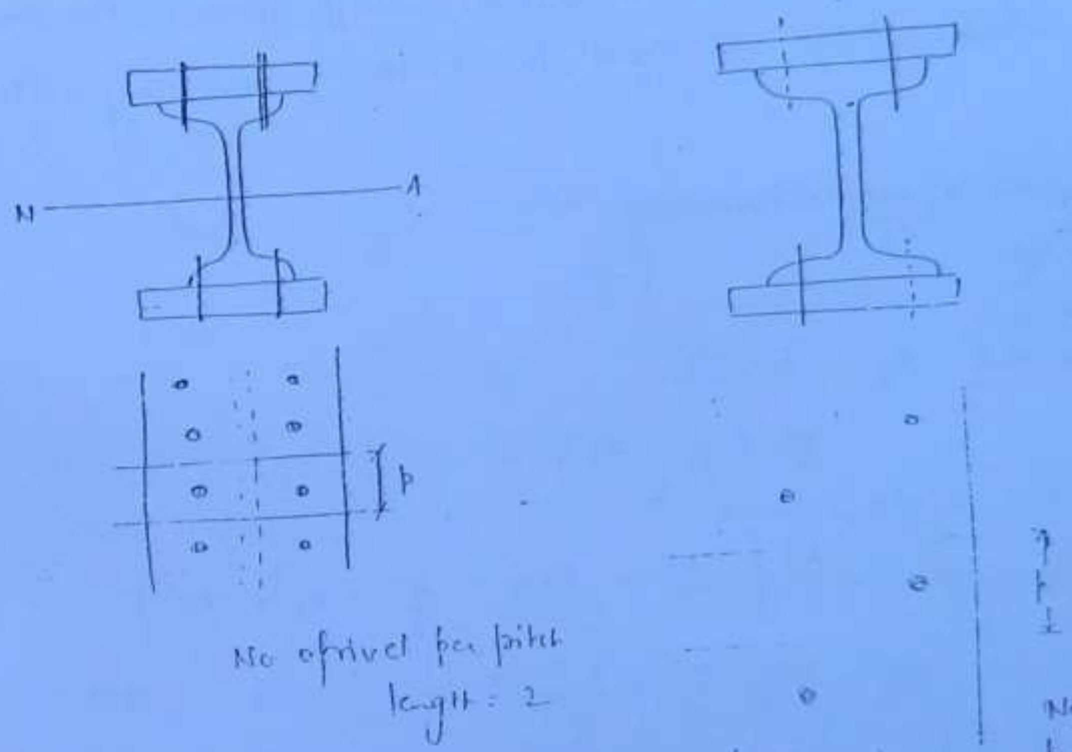
$$r_{min} = \sqrt{\frac{a t_w^3}{12 \times a t_w}} = \frac{t_w}{\sqrt{12}}$$

$$\frac{l_{eff}}{r_{min}} = \frac{(d_w/2)}{t_w/\sqrt{12}} = \frac{d_w\sqrt{3}}{t_w}$$

Generally if beam is safe in web crippling it will be safe in web buckling

153

Design of rivets



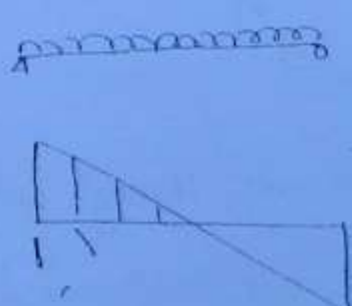
No of rivet per pitch length : 2

No of rivet per pitch length = 1

$\frac{VA\bar{y}}{I_{xx}} \times p$ = Shear force per pitch length $\leq n R_v$

$n \to$ no of rivet / pitch length

2 for safety



Rivet diameter will be assumed and pitch will be calculated for max^m shear.

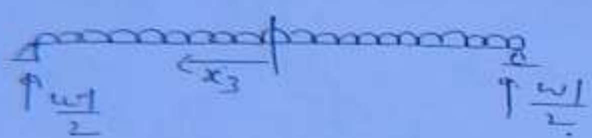
Same pitch can be adopted through out the span

In reality, as shear force decreases towards the mid span, the pitch should increase towards the mid span.

Curtailment of Plates

154

Theoretical cutoff point is the location at which B.M. is equal to MOR of continuing section.



B.M. at x_3 location

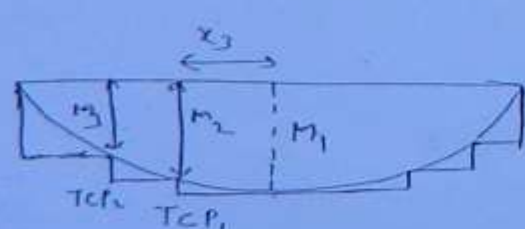
$$= \frac{wl}{2}\left(\frac{l}{2} - x_3\right) - \frac{w\left(\frac{l}{2} - x_3\right)^2}{2}$$

$$= \frac{w}{2}\left[\frac{l^2}{2} - \cancel{l x_3} - \frac{l^2}{4} - x_3^2 + \cancel{l x_3}\right]$$

$$= \frac{w}{2}\left(\frac{l^2}{4} - x_3^2\right)$$

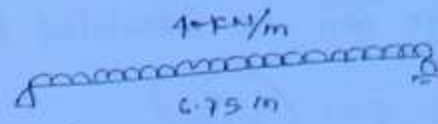
$$\frac{w}{2}\left(\frac{l^2}{4} - x_3^2\right) = \sigma_b Z_2$$

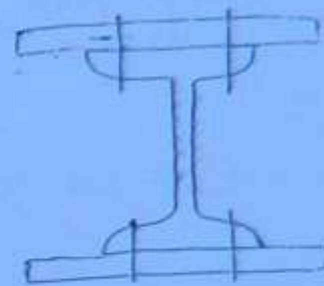
find x_3



At least one plate must continue through out the span.

Q.3 Page. 52





	ISMB 300	ISMB 400
I_{xx}	$= 89.9\times10^6\ mm^4$	205×10^6
Z	$= 59.9\times10^4$	102×10^4
B	$= 140\ mm$	140
t	$= 7.7\ mm$	8.9

$$M_{max} = \frac{wl^2}{8} = \frac{40\times6.75^2}{8}\ KNm = 227.813\ KNm$$

$$Z_{req.} = \frac{M_{max}}{\sigma_{bc}\ \text{or}\ \sigma_{bt}} = \frac{227.813\times10^6}{165} = 138\times10^4\ mm^3$$

No rolled secⁿ can provide $Z = Z_{req.}$

Hence plates will be used to increase the z value

Hence let us use ISMB 400 and cover plate as the secⁿ

$$\text{Approx area of plate req} = \frac{Z_{req} - Z_{rolled}}{D}$$

$$= \left(138\times10^4 - 102\times10^4\right)/D$$

$$= \frac{36\times10^4}{400} = 900\ mm^2$$

let us choose $A_p = 1200\ mm^2$

Adopting $t = 8\ mm$

$$\text{width req.} = \frac{1200}{8} = 150\ mm$$

Check for bending-

Area of tension flange = Area of flange plate + Area of flange of rolled secⁿ

As the thickness of flange of rolled secn is not give the gross and Net area of flange can be calculated from the area of flange plate only

$$\frac{\text{Gross area}}{\text{Net area}} = \frac{150 \times 8}{150 \times 8 - 21.5 \times 8 \times 2}$$

$$= 1.4$$

$$I_{gross} = 205 \times 10^6 + 2\left[\frac{150 \times 8^3}{12} + 150 \times 8 \times 204^2\right]$$

$$= 304.89 \times 10^6 \text{ mm}^4$$

$$y_{max} = 208 \text{ mm}$$

$$\frac{M_{ax} \cdot Y_T}{I_{gross}} \times \frac{GA}{NA} = \frac{227.813 \times 10^6 \times 208}{304.89 \times 10^6} \times 1.4$$

$$= 217.58 > 165$$

let us adopt Area of plate 1800 mm² and staggering of rivets hence

$$B \times t = 1800 \text{ mm}^2$$

$$\text{adopt } t = 10 \text{ mm}$$

$$B = 180 \text{ mm}$$

$$\frac{GA}{NA} = \frac{1800}{1800 - 21.5 \times 10} = 1.136$$

$$I_{gross} = 205 \times 10^6 + 2\left[\frac{180 \times 10^3}{12} + 1800 \times 205^2\right]$$

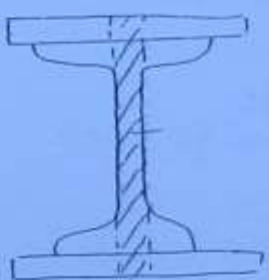
$$356.32 \times 10^6 \text{ mm}^4$$

check for bending $\frac{my}{I} \times \frac{SA}{NA}$

$$= \frac{227.813 \times 10^6 \times 210}{356.32 \times 10^6} \times 1.136$$

$$= 152.52\ N/mm^2 < 165\ N/mm^2$$

check for shear



Area $= 420 \times 8.9 = 3738\ mm^2$

Av. shear stress $= \frac{V_{max}}{3738}\ N/mm^2$

$$= \frac{135 \times 10^6}{3728} = 36.12\ \frac{N}{mm^2} < \frac{100N}{mm^2}$$

o.k.

Check for Deflection

$$a_{max} = \frac{5}{384} \frac{wl^4}{EI_{zz}} = \frac{5}{384} \times \frac{40 \times 6750^4}{2 \times 10^5 \times 356.32 \times 10^6}$$

$$= 15.17\ mm$$

$$\frac{Span}{325} = \frac{6750}{325} = 20.77$$

$$15.77 < 20.77$$

OK.

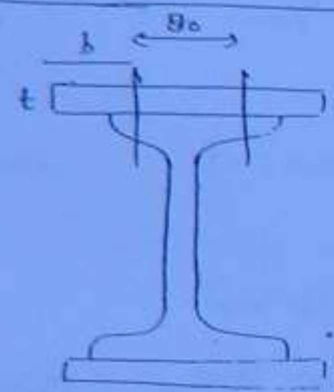
Note → [illegible] from the [illegible] point of view.

Check for secondary criteria.

Assuming $g = 80mm$

$h_2 = 32.8mm$

- Check for local buckling of flange plate.



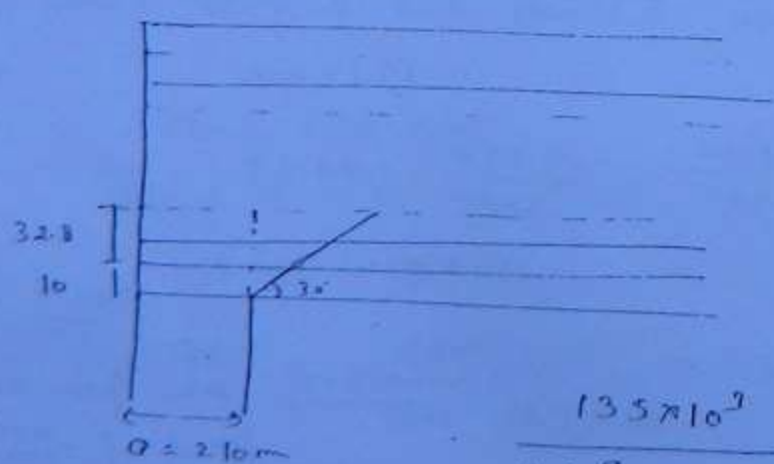
158

$$b = \frac{180-80}{2} = 50mm$$

$$\frac{b}{t} = \frac{50}{10} = 5 < 16$$

Check for web cripling.

The beam is carrying UDL hence point loading will only be at support thus web crippling will be checked at support.



$$\frac{135 \times 10^3}{8.9 \left\{ 210 + \sqrt{3}\,(32.8 + 10) \right.} \leq .75 f_y$$

$$53.39 \leq 187.5$$

Safe in web crippling

Check for web buckling

$$\lambda = \frac{d_w \sqrt{3}}{t_w} = \frac{(D - 2h_2)\sqrt{3}}{t_w}$$

$$= \frac{(400 - 2 \times 32.8)\sqrt{3}}{8.9} = 65.071$$

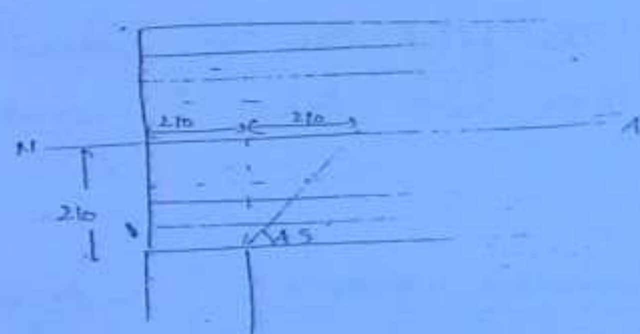
$\sigma_{ac} = 110.207\ N/mm^2$

λ	σ_{ac}
60	113
70	107.5

Area of resisting section

$= (210 + 210) \times 8.9$

$= 3738\ mm^2$



Carrying capacity

$= 3738 \times 110.207 = 411.95\ kN$

$> 135\ kN$ Safe

Rivet Design

$$\frac{VA\bar{y}}{I} \cdot p \le R_v$$

because no of rivet per pitch length = 1

$180 \times 10 = 1800\ mm^2$

$\bar{y} = 201\ mm$

$I_{xx} = 356.32 \times 10^6$

$V = 135 \times 10^3\ N$

Rivet dia = 20mm

$R_v = 36.3\ kN$

$p \leq 259.68\,mm$

Note:

16t or 12t, 200 mm is a recommendation for direct loading. In this case limit would be 32t or 300 mm. hence max. pitch will be

160 min { 320, 300 }

Max pitch : 300 mm

Adopt $p = 250$ mm

The pitch can be increased towards the mid span. However we take it constant through out.

Permissible Stresses

$$\sigma_{bc} = \sigma_{bt} = 0.67\, f_y$$

$$\sigma_{shear} = 0.4\, f_y$$

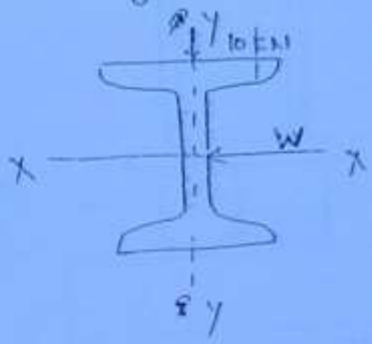
$$\sigma_{br} = \cancel{0.75 f_y}\;\; 0.75\, f_y$$

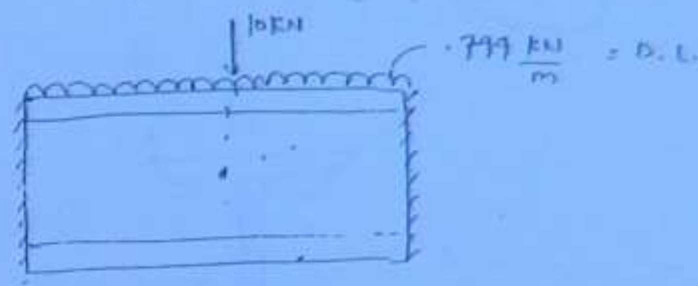
All these permissible values are increased by $33\frac{1}{3}\,\%$ if effect of wind or EQ is taken into account

Sol. 6 ISWB 450 @ 79.4 kgf/m

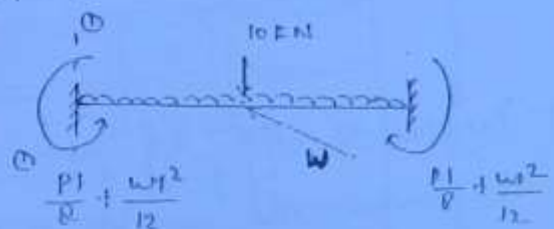
1000 kgf = 10 kN

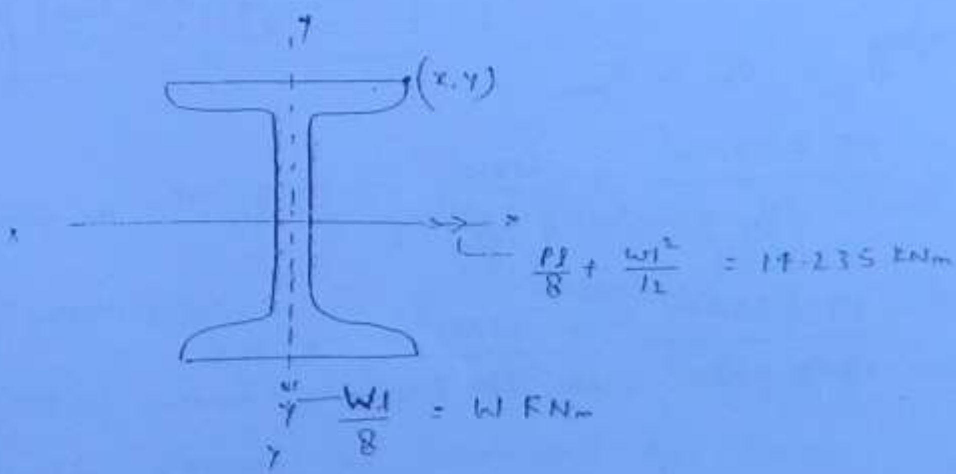
g = 10





for the restrained beam like this max. B.M. will occur at the support





Tension (+)ve
$$\sigma = \frac{14.235 \times 10^6 \, y}{I_{xx}} + \frac{1W \times 10^6 \, x}{I_{yy}}$$

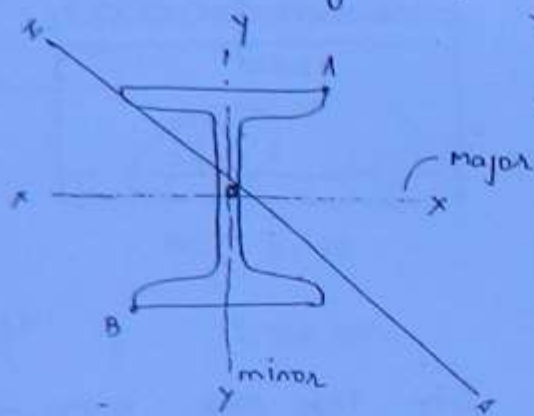
at N.A. Bending stress is zero

$$\left(\frac{M \cdot 235\times10^6}{I_{xx}}\right) y + \left(\frac{w\times10^6}{I_{yy}}\right) x = 0$$

162

$$ax + by = 0$$

$$y = -\frac{a}{b} x$$



location of N.A shows B. stress will be max_m either at A or B

Note Instead of finding out location of N.A by above approach we could have located it under the concept that N.A lies b/w minor axis and the direction of resultant moment

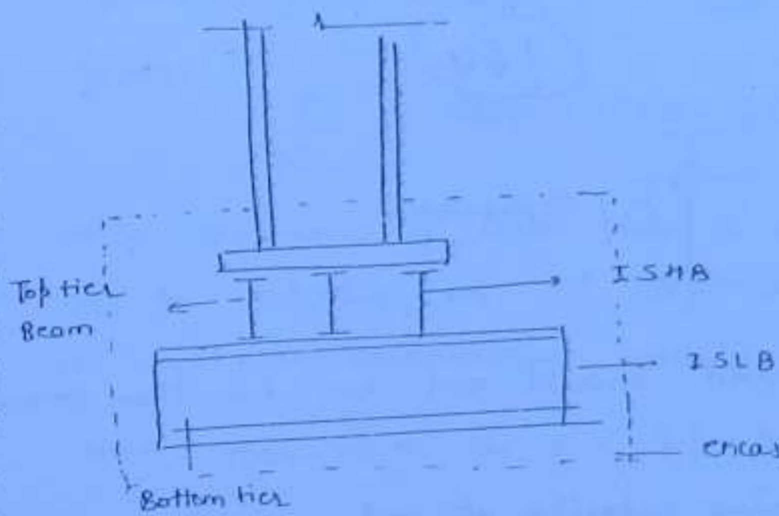
for safety $\quad \sigma_A \le \sigma_{perm}$

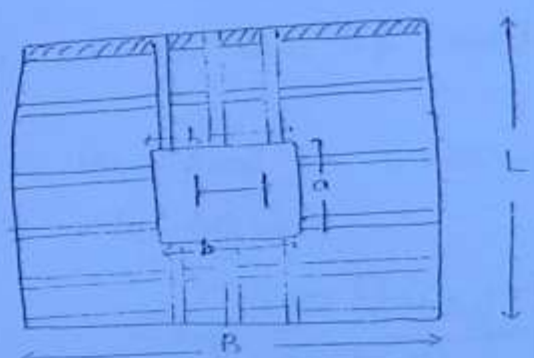
$$\frac{14 \cdot 235\times10^6}{\left(\frac{I_{xx}}{y}\right)} + \frac{w\times10^6}{\left(\frac{I_{yy}}{x}\right)} \le 165 \text{ mPa}$$

$$\frac{14 \cdot 235\times10^6}{1558 \cdot 1\times10^3} + \frac{w\times10^6}{170 \cdot 7\times10^3} \le 165$$

$$w \le 26 \cdot 6 \text{ KN}$$

Grillage foundation





Permissible str — If the grillage foundation is encased in conc. the permissible stress are increased by $33\frac{1}{3}$ %.

If effect of wind and EQ is also taken into account permissible stresses are increased by 50%.

Note

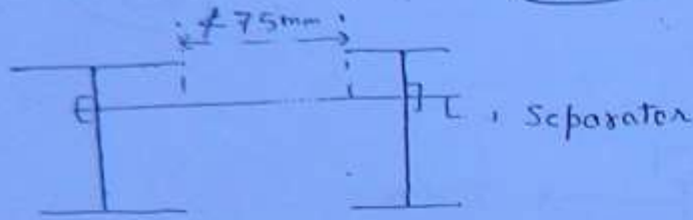
Per. Bending stress normally is $0.67 f_y$

All the stresses are ↑ by this percentage

Per. Bending stress in grillage foundation $= 1.33 \times 0.67 f_y$

Per. Bending stress in grillage foundation when effect of wind or EQ is also taken in account $= 1.5 \times .67 f_y$

Seperator must keep the beam properly spaced such that the distance between edges of adjacent beam is not less than 75 mm

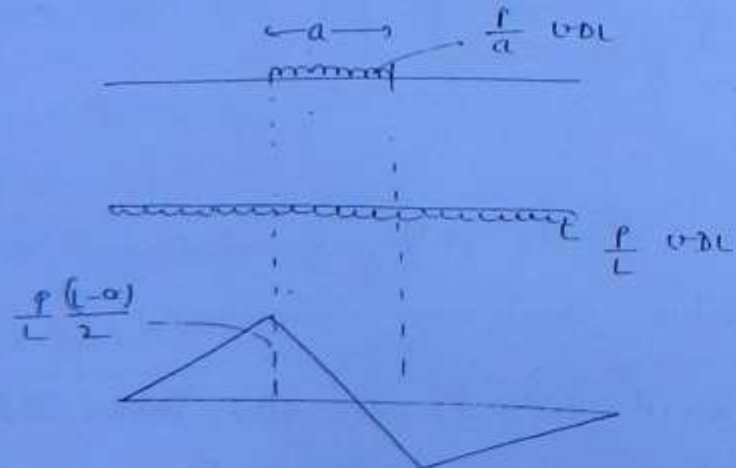


- Clear cover all around should not be less than 100mm

$$\therefore \frac{P}{L \cdot B} \leq \text{Bearing capacity of soil}$$

Choose L & B on this criteria

Top tier Beams



$$M_{max} = \frac{1}{2} \frac{P}{L} \frac{(L-a)}{2} \times \frac{L}{2}$$

$$M_{max} = \frac{P(L-a)}{8}$$

The above max SF & BM are for all top tier beam taken together. Hence to design top tier beam no. of beams needs to be selected.

To start with area of base plate can be calculated as follows

$$\frac{P}{\sigma_{conc}} = A_{req.} = a \times b$$

once the size of base plate is known no of top tier beam can be selected, no of top tier beam should be such that it could be accomodated with in width b

Hence Max. B.M. in one top tier beam $= \dfrac{P(l-a)}{8n}$

where $n \rightarrow$ no of top tier beam.

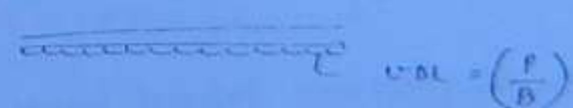
Max. S.F. in one top tier beam $= \dfrac{P(l-a)}{2L \times n}$

P is the load in which effect of dead wt had already been taken into account.

$$Z_{req.} = \frac{\left\{\dfrac{P(l-a)}{8 \times n}\right\}}{165 \times 1.33}$$

Choose secⁿ and check for shear.

Bottom Tier beam.



Mmax. for all bottom tier beam taken together $= \dfrac{P(B-b)}{8}$

V_{max} for all bottom tier beam taken together

$$= \frac{P(B-b)}{2B}$$

166

Note :

If there is a chance of web crippling it will be there in top tier beam only hence top tier beam can be checked for web crippling also.

8

Load on a column is 2500 KN the safe bearing capacity of soil is 250 $\frac{KN}{m^2}$ & permissible bearing stress in conc. is 4000 $\frac{KN}{m^2}$ Design grillage foundation.

Sol. Load on base plate $= 1.1 \times 2500 = 2750$ KN

Choosing square base plate

$$a \times a = \frac{1.1P}{\sigma_c} = \frac{2750}{4000}$$

$$a = .829$$

$$= 850 \text{ mm}$$

$$\frac{1.1P}{250} = L^2$$

$$L = \frac{1.1 \times 25\cancel{0}0^{10}}{25\cancel{0}} = 3.316 \text{ m}$$

adopt $= 3.4$ m

$$M_{max} = \frac{1.1P(L-a)}{8} = \cancel{847.68 \text{ KNm}} \quad 876.56 \text{ KNm}$$

$$V_{max} = \frac{1.1P(L-a)}{2L} = 1031.25 \text{ KN}$$

Z_{req} for all beams taken together

$$= \frac{876.56 \times 10^6}{165 \times 1.33} = 4000 \text{ cm}^3$$

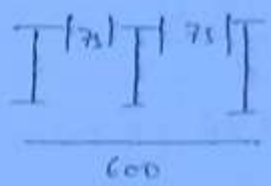
let us choose 3 no. of top tier beams

Z req. for 1 $= \dfrac{4000}{3} = 1333.33$

(167)

3 let us choose ISMB 450

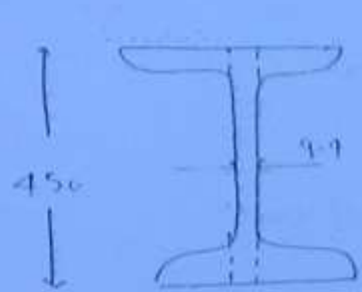
$Z_{avai} = 1350.7\ cm^3$



$b = 150$

So 3 no. of ISMB 450 can be accomodated with in the base plate

$$V_{max\ one} = \frac{1.1\,P(L-a)}{2L \times 3} = 343.75\ KN$$

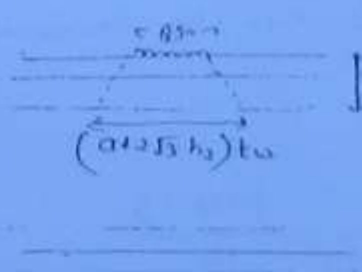


$$\text{Av shear stress} = \frac{343.75 \times 10^3}{450 \times 9.4}$$

$$= 81.264 < 133$$

100

safe



$h_2 = 35.4\ mm$

$$\frac{1.1 \times 2500}{3} = 916.7\ KN$$

$$\frac{916.7 \times 10^3}{(a + 2\sqrt{3}\,h_2)\,t_w} \le 1.33 \times 0.75\,f_y$$

$$100.26 \le 247.375\ N/mm^2$$

OK.

Max spacing b/w bottom tier beam is generally 200 to 250 mm.

168

No. of bottom tier beam

M_{max} bottom tier for all beams taken together

$$= \frac{P(B-b)}{8}$$

$$= 876.56 \text{ KNm}$$

$$Z_{req.} = \frac{876.56 \times \cancel{1.33}}{1.33 \times 165} = 4000 \text{ cm}^3$$

Space available = 3.9 m

No. of bottom tier beams are generally 10 – 20

Let us choose 15 no.

$$Z_{req.} \text{ one} = \frac{4000}{15} = 266.7 \text{ cm}^3$$

choose ISMB 225

$$Z_{provided} = 305.9 \text{ cm}^3$$

$$14 \times 75 + 15 \times 110 < 3400$$

$$2700 < 3400$$

hence can be accomodated

Check for shear.

Shear in one bottom shear beam $= \frac{1031.25}{15}$

$= 68.75$ KN

$$\text{Av. shear stress} = \frac{68.75}{6.5 \times (225)} = 47 \text{ N/mm}^2$$

$< 1.33 \times 100$

ok.

Note:

If the size of column is given to us thickness can be calculated

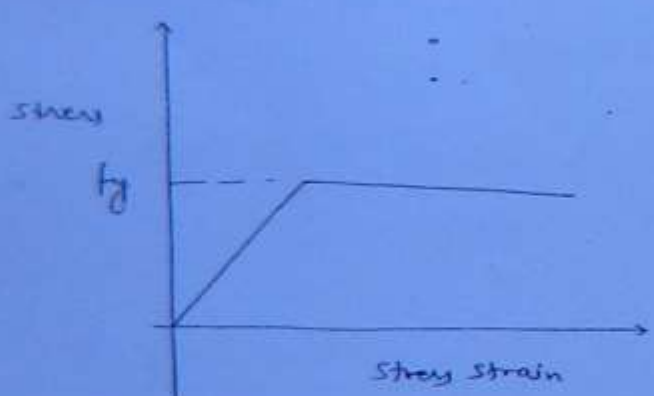
PLASTIC ANALYSIS

In conventional design method structure is designed for strength of steel till it yields at a point in the secn.

However if yielding occurs at one point in section it does not mean collapse of the member. Due to plastic deformation and strain hardening of material, the particles which were less stressed will be brought into action so that structure is actually able to resist greater load.

In the modern design strength of steel beyond the point of first yield is utilized and this method of design is called plastic method of design.

Simply Stress strain curve.



- Strain hardening range has been omitted In this case which infact will add to the margin of safety.
- Plastic design can be applied only to redundant structure.
- In the case of simple members load causing first yield is most critical because at that load only large unacceptable deformation occurs.

str. deformation even at the time of collapse is not much
hence plastic analysis can be applied only to redundant str.

Assumptions in Plastic Analysis

1. Material should possess ductility so that it can be deformed in the plastic stage.
2. Strain distribution is linear.
3. Relation b/w tensile stress and tensile strain & comp. stress & comp. strain should be same.
4. Joints should be sufficiently strong to transfer the moment. All joints should be rigid joint.

Plastic Bending of Beams.

T = C gives N.A.

$$C = \frac{1}{2} f_c (D-x)\, b$$

$$T = \frac{1}{2} f_t\, x\, b$$

$$\frac{f_c}{(D-x)} = \frac{f_t}{x}$$

$$f_c (D-x) \cancel{b} = f_t\, x\, \cancel{b}$$

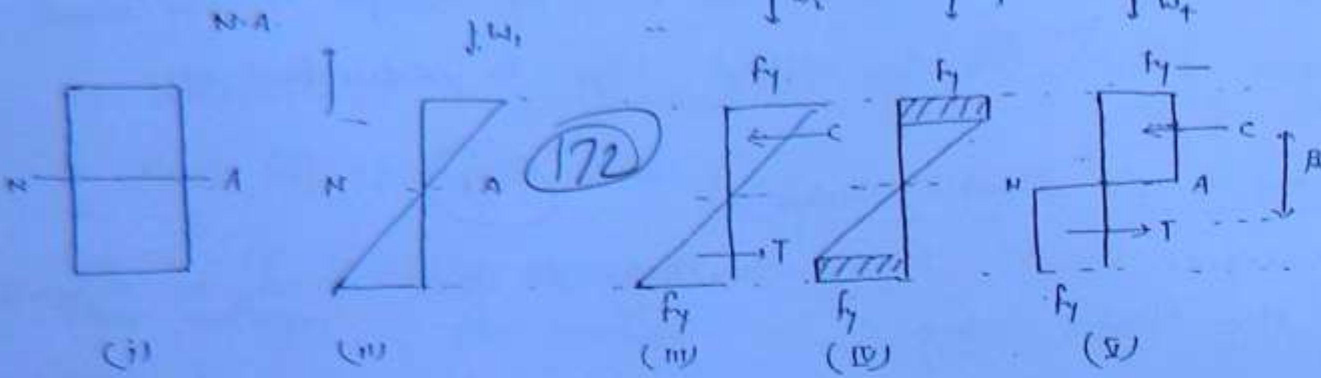
$$(D-x)^2 = x^2$$

$$b(D-x) \cdot \frac{(D-x)}{2} = \frac{b}{2} x \cdot x$$

Moment of comp area about N.A = Moment of tension area about N.A i.e.

This is the location of C.G.

Within elastic limit centroidal axis is the N.A.



N.A. will remain at centroidal axis if section is symmetrical about the axis of bending. (in fig IV & V)

In fig (v)

$$C = T$$

$$f_y A_c = f_y A_T$$

$$A_c = A_T = \frac{A}{2}$$

N.A. is equal area axis

$$C\beta = T \cdot \beta = M_p$$

M_p is full plastic moment capacity of secn

In fig (iii)

N.A. is centroidal axis

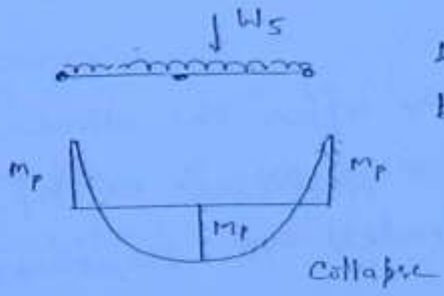
$$T\alpha = C\alpha = M_y$$

$M_y \rightarrow$ M.O.R. at the time of first yield = Yield moment

- M_p is plastic moment capacity which depends on section only.
- A fully plastic secn can not resist any further moment and if loading is applied beyond W4. Unrestrained rotation will take place at the fully yielded section thus we assume as if a plastic hinge has formed. Plastic hinge i.e. can be understood as rusted hinge in which rotation does not take place upto certain load and beyond that load a large deformation in the form of rotation will take place.

Formation of plastic hinge at one location does not mean the collapse of the structure. For collapse sufficient no. of plastic hinge must develop to make the structure unstable.

173

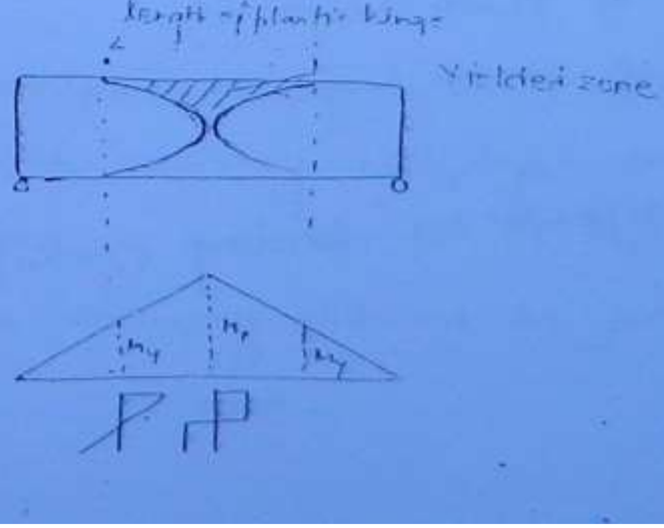


Due to plastic hinge formation fixed end start behaving as hinge.

For complete collapse of structure the margin available is $(W_s - W_2)$ beyond the point of first yield. $(W_s - W_2)$

For complete yielding of the secⁿ margin available beyond the point of first yield is $(W_4 - W_2)$

Definition of Plastic hinge – A plastic hinge can be defined as a yielded zone due to flexure in a structure in which infinite rotation can take place at a constant resisting moment (M_p) of the secⁿ



Some important point about plastic hinge.

1. A secⁿ is said to develop a plastic hinge when due to flexure, stress at every point of the secⁿ is equal to yield stress.

174

2. Plastic hinge develops first at secⁿ subjected to greatest curvature

3. Due to formation of plastic hinge one after the other, re distribution of moments take place. Sufficient no. of plastic hinges have to be developed to render the structure to unstable or collapse state.

4. No. of plastic hinge required for <u>complete collapse of</u> the structure is $(r+1)$.
Where r is the degree of redundancy.

For vertical loading
$D_s = 2$

No of plastic hinge required for complete collapse of str $= r+1 = 3$

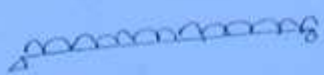
$D_s = 2$

partial collapse of str.
No. of plastic hinge $= 2$

for complete collapse no. of plastic hinge $= 2+1 = 3$

5. Length of plastic hinge depends on loading & shape of section.

l_p : length of P.H.

  l_{p_2}

  l_{p_3}

6) For purpose of analysis plastic hinge will be assumed at a point about which plastic rotation takes place

7) M.O.R. of normal hinge is zero & M.O.R. of P.H. is M_p

8) Plastic hinge is expected to form at

a) Fixed ends
b) At the location of point load.
c) At the point of sudden change in geometry.
d) At points of zero shear in a span subjected to U.D.L. or U.V.L.

9) When two sections join at a point plastic hinge forms in a section of smaller M_p

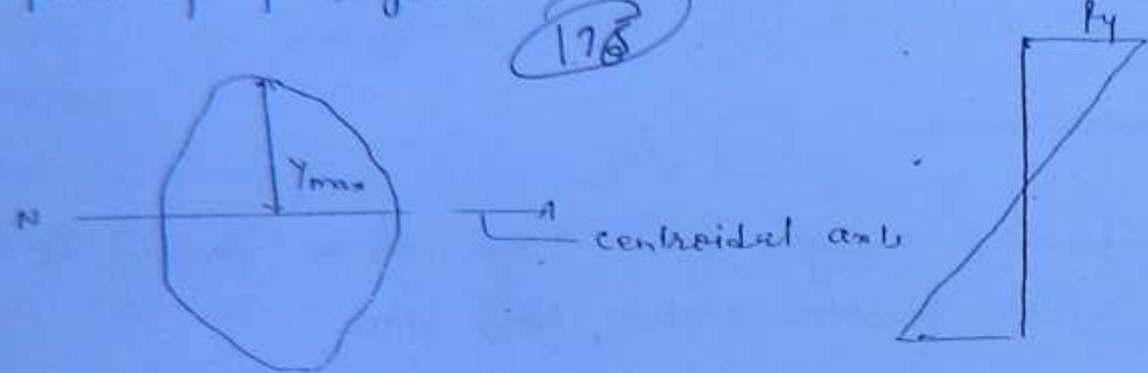
If section is loaded upto a moment of M_y and then unloaded complete recovery will take place. However if moment applied on the secn is $> M_y$ & then unloading takes place recovery will not be complete

Shape factor

$$\text{Shape factor} = \frac{M_p}{M_y}$$

It shows the reserve of strength of a sec^n beyond the point of first yield.

178

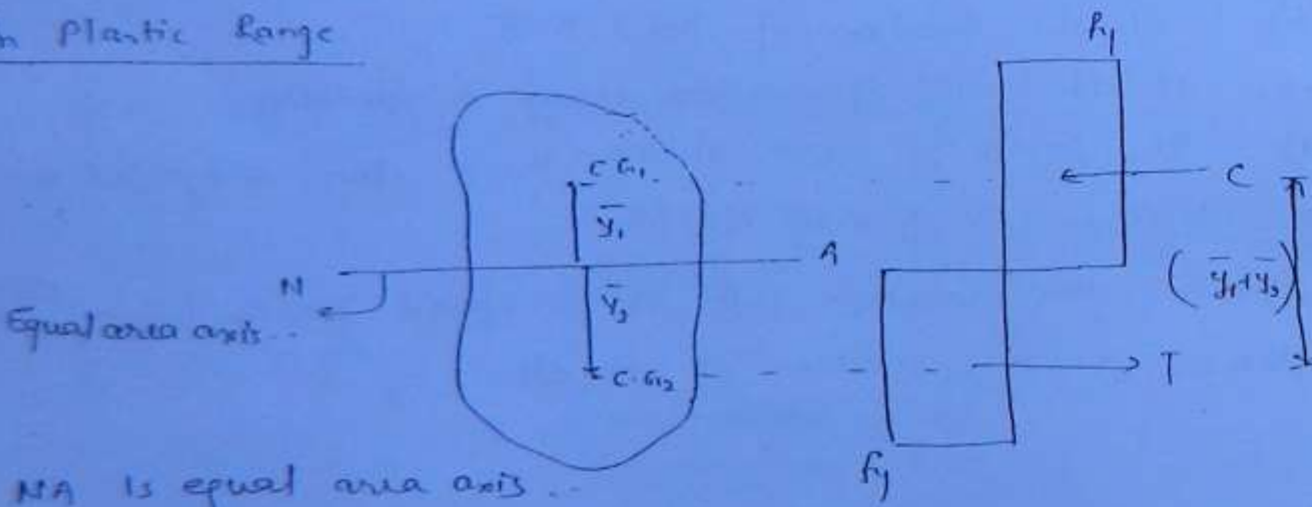


In elastic range

$M_y = f_y \cdot Z$, $Z \rightarrow$ Section modulus

$$Z = \frac{I_{NA}}{y_{max}} = \frac{\text{M.O.I about C.G. axis}}{y_{max}}$$

In Plastic Range



NA is equal area axis

$$M_p = C(\bar{y}_1 + \bar{y}_2) = T(\bar{y}_1 + \bar{y}_2)$$

$$= f_y \cdot \frac{A}{2}(\bar{y}_1 + \bar{y}_2)$$

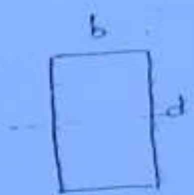
$M_p = f_y S$ where $S \rightarrow$ Plastic Modulus

$$= \frac{A}{2}(\bar{y}_1 + \bar{y}_2)$$

Shape factor $= \dfrac{M_p}{M_y} = \dfrac{f_y \cdot S}{f_y \cdot Z} = \dfrac{S}{Z} = \dfrac{\text{plastic modulus}}{\text{section modulus}}$

$$= \frac{\frac{A}{2}\left(\bar{y}_1 + \bar{y}_2\right)}{Z}$$ 177

Shape factor of Rectangular secⁿ –



equal area axis & centroidal axis is same

$$SF = \frac{\frac{A}{2}\left(\bar{y}_1 + \bar{y}_2\right)}{Z} = \frac{\left(\frac{bd}{2}\right)\left(\frac{d}{2}\right)}{\frac{bd^2}{6}}$$

$$= \frac{6}{4} = \frac{3}{2} = 1.5$$

circular secⁿ

$$S.F = \frac{\frac{A}{2}\left(\bar{y}_1 + \bar{y}_2\right)}{Z}$$

$$= \frac{\frac{\pi d^2}{8}\left(\frac{4\cdot(D/2)}{3\pi} \times 2\right)}{\frac{\pi d^4}{64}\,\frac{2}{d}}$$

Plastic modulus for circular secⁿ $= \dfrac{d^3}{6}$

$$= \frac{\frac{1}{8} \times \frac{4}{3\pi}}{\frac{1}{32}}$$

$$= \frac{1}{6\pi}\,32 = 1.698$$

$$S.F. = \frac{\frac{A}{2}\left(\bar{y}_1+\bar{y}_2\right)}{Z}$$

$$= \frac{\frac{A}{2}\left(\frac{a}{\sqrt2}\times\frac{1}{3}\times 2\right)}{\frac{a^4}{12}\frac{\sqrt2}{a}}$$

178

$$S = \frac{a^3}{3\sqrt2}$$

$$= \frac{\frac{a^3}{3\sqrt2}}{a^3\frac{\sqrt2}{12}}$$

$$= \frac{12}{\sqrt2}\times\frac{1}{3\sqrt2} = 2$$

$$\frac{t}{d} << 1$$

$$S.F. = \frac{\frac{A}{2}\left(\bar{y}_1+\bar{y}_2\right)}{Z} = \frac{S}{Z}$$

S = Plastic modulus $= \frac{d^3}{6} - \frac{(d-2t)^3}{6}$

$$= \frac{d^3}{6}\left[1-\left(1-\frac{2t}{d}\right)^3\right]$$

$$= \frac{d^3}{6}\left[1-\left\{1-\frac{8t^3}{d^3}+3\,\frac{4t^2}{d^2}+\frac{3\times2t}{6d}\right\}\right]$$

$$= \frac{d^3}{6}\left[\frac{6t}{d}\right] = d^2t$$

$$Z = \frac{I}{y_{max}} = \frac{\frac{\pi d^4}{64}-\frac{\pi(d-2t)^4}{64}}{\frac{d}{2}}$$

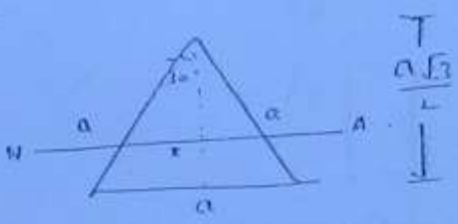
$$\frac{\pi d^3}{32}\left[1-\left(1-\frac{2t}{d}\right)^4\right]$$

$$=\frac{\pi d^3}{32}\left[1-\left(1-\frac{8t}{d}+\cdots\right)\right]$$

179

$$=\frac{\pi d^3}{32}\times\frac{8t}{d}=\frac{\pi d^2 t}{4}$$

$$S.F.=\frac{d^2 t}{\pi d^2 t}\times 4 = 1.273$$



$$S.F=\frac{5}{2}$$

$$Z=\frac{I}{y}=\frac{a\cdot\left(\frac{a\sqrt{3}}{2}\right)^3}{36\left(\frac{a\sqrt{3}}{2}\right)\frac{2}{3}}$$

$$=\frac{a\left(\frac{a\sqrt{3}}{2}\right)^2}{\frac{24}{1}}=\frac{a^3\cdot 3}{4\times 24}=\left(\frac{a^3}{32}\right)$$

S

N.A is equal area axis

$$\frac{Area}{2}=\frac{1}{2}\left\{a\cdot\frac{a\sqrt{3}}{2}\times\frac{1}{2}\right\}=\frac{a^2\sqrt{3}}{8}$$

Area of equilateral triangle $=\frac{a^2\sqrt{3}}{4}$

$$\frac{x^2\sqrt{3}}{4}=\frac{a^2\sqrt{3}}{4}\times\frac{1}{2}$$

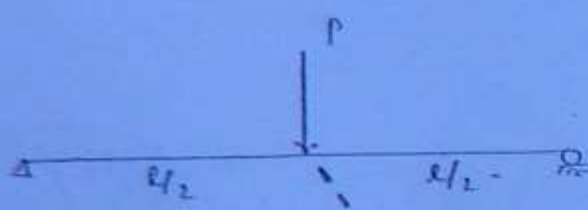
$$x=\frac{a}{\sqrt{2}}$$

$$\bar{y}=\frac{1}{3}\left(\frac{2a+x}{a+x}\right)\times\left(\frac{\sqrt{3}a}{2}-\frac{x\sqrt{3}}{2}\right)$$

$$=\frac{1}{3}\left(\frac{2\sqrt{2}+1}{\sqrt{2}+1}\right)\frac{\sqrt{3}}{2}\left(\sqrt{2}-1\right)=.19x=.189x$$

$$=.268a$$

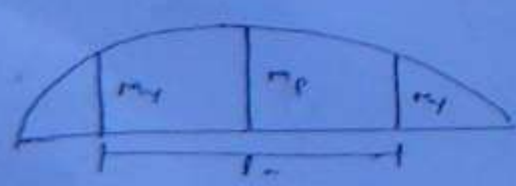
Length of Plastic hinge.



$$\frac{M_1}{M_y} = \frac{\frac{l}{2}}{\frac{(l-l_p)}{2}} = 1.5$$

$$l_p = \frac{l}{3}$$

3)



$$\frac{M_p}{M_y} = \frac{\frac{wl^2}{8}}{\frac{wl^2}{8} - \frac{w l_p^2}{2 \times 4}} = \cancel{\frac{l^2}{l^2 - 4 l_p^2}}$$

only when s.f. is zero at mid point

$\cancel{l^2 = 1.5 l^2 - 6 l_p^2}$

$\cancel{6 l_p^2 = 0.5 l^2}$

$\cancel{l_p^2 = \frac{l^2}{12}}$

$\cancel{l_p = \frac{l}{2\sqrt{3}}}$

(1.5)

$$\frac{l^2}{l^2 - l_p^2} = 1.5$$

$$l^2 = 1.5 l^2 - 1.5 l_p^2$$

$$.5 l^2 = 1.5 l_p^2$$

$$l_p^2 = \frac{l^2}{3}$$

$$l_p = \frac{l}{\sqrt{3}}$$

Load factor and factor of safety

$$\text{load factor} = \frac{\text{collapse load}}{\text{working load}}$$

Note:

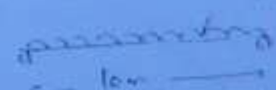
Plastic Design

(5 kN/m × load factor)

$$\frac{(5 \times L.F)\, l^2}{8} \le M_p$$

Elastic Design

5 kN/m



$$\frac{\frac{wl^2}{8} \times y}{I} \le \frac{f_y}{F.O.S}$$

Ne.

1.

$$\text{load factor} = \frac{\text{collapse load}}{\text{working load}} = \frac{W_c}{W_w}$$

$$= \frac{M_p}{\sigma_{per} \cdot Z} = \frac{f_y \cdot S}{\left(\frac{f_y}{F.O.S}\right) Z} = F.O.S \times \left(\frac{S}{Z}\right)$$

$$\text{load factor} = (\text{factor of safety}) \times \text{shape factor}$$

182

Moment curvature Relationship (Rectangular sec^n)

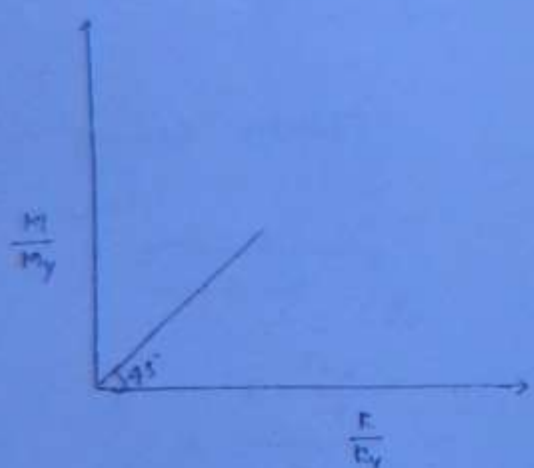
So long as moment is less than M_y

Elastic condition prevails

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

$$M = \frac{EI}{R} = EI\,K \text{ , curvature}$$

$$\frac{M}{M_y} = \frac{K}{K_y}$$

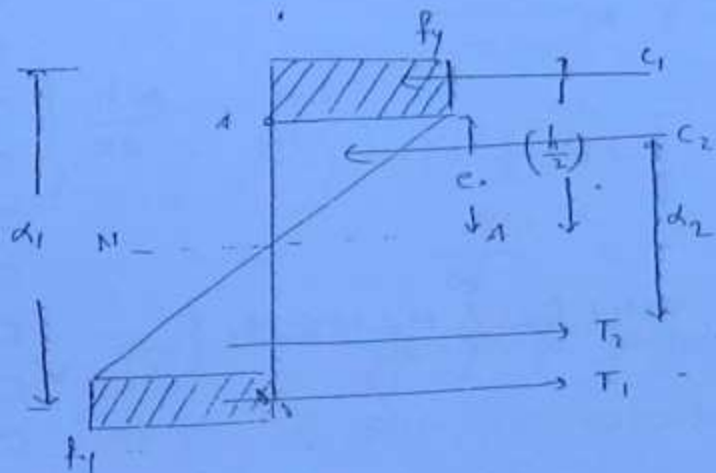


As the sec^n start yielding moment curvature relationship becomes non linear

Even in the yielded zone the strain variation is assumed to be linear.

strain $\varepsilon = \frac{y}{r} = yK$

$$\boxed{K = \frac{\varepsilon}{y}}$$



stress variation

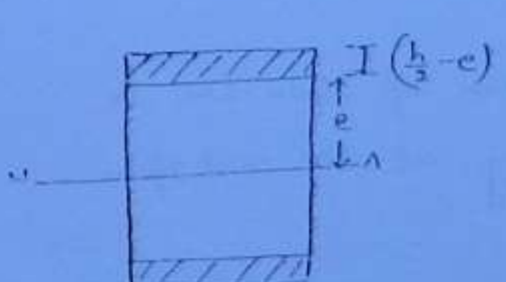
Strain at $A = \frac{fy}{E} = Ke$

curvature $(K) = \left\{\frac{fy}{Ee}\right\}$

$$Ky = \frac{fy}{E \cdot \frac{h}{2}}$$

$$\boxed{\frac{K}{K_y} = \frac{h}{2e}}$$

$$M = c_1 d_1 + c_2 d_2 = T_1 d_1 + T_2 d_2$$



$$c_1 = fy \cdot \left(\frac{h}{2} - e\right) b$$

$$d_1 = h - \left(\frac{h}{2} - e\right) = \left(\frac{h}{2} + e\right)$$

$$c_2 = \frac{1}{2} fy \cdot e \cdot b$$

$$d_2 = \frac{4e}{3}$$

$$M = C_1 x_1 + C_2 x_2$$

$$= f_y\, b\left(\frac{h^2}{4} - e^2\right) + \frac{2 f_y e^2 b}{3}$$

$$= f_y \cdot \frac{bh^2}{6}\left[\frac{6}{4} - \frac{6e^2}{h^2} + \frac{2}{3}\frac{e^2}{h^2}\times 6\right]$$

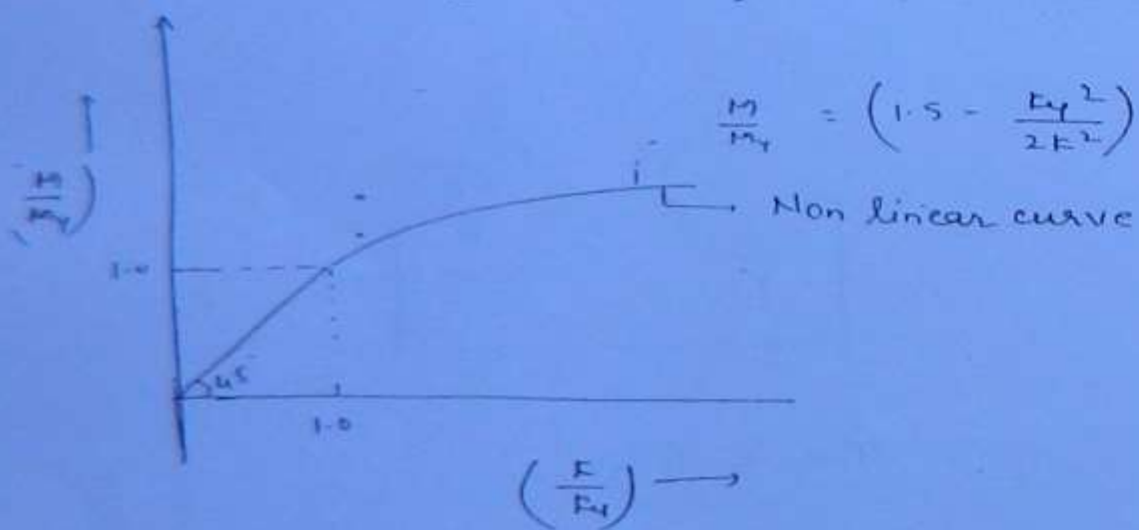
184 $$= f_y \cdot \frac{bh^2}{6}\left[1.5 - 6\;2\frac{e^2}{h^2}\right]$$

$$= M_y\left[1.5 - \frac{2e^2}{h^2}\right] = M$$

$$\left\{\frac{k}{k_y} = \frac{h}{2e}\right\}$$

$$\frac{M}{M_y} = \left[1.5 - 2\left(\frac{k_y}{k}\right)^2 \cdot \frac{1}{4}\right]$$

$$\left(\frac{M}{M_y}\right) = \left[1.5 - \frac{k_y^2}{2k^2}\right] \quad \text{valid for } \left\{M_y \le M \le M_p\right\}$$

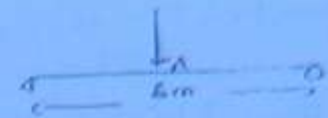


Question.

Design a simply supported beam of span 6m to support a point load of 100 kN acting at its mid span.

$f_y = 250$ MPa & load factor $= 1.75$

collapse load = 100×1.75 = 175 KN



Beam will collapse when plastic hinge forms at midspan below the point load.

$$B.M. \text{ at } A = M_p = \frac{175 \times l}{4} = 262.5 \text{ KNm}$$

185

$$M_p = f_y \cdot S$$

$$S = \frac{262.5 \times 10^6}{250} = 10.5 \times 10^5 \text{ mm}^3$$

$$\frac{S}{Z} = \text{shape factor}$$

$$Z = \frac{S}{SF} =$$

By choosing shape factor i.e by choosing the sec^n shape Z can be calculated

Note

Shape factor for I sec^n = 1.15

$$Z_{req.} = \frac{10.5 \times 10^5}{1.15} = 913.04 \text{ cm}^3$$

Choose

ISMB - 400

$$Z = 1022.9 \text{ cm}^3$$

Important theorems in plastic analysis

In the plastic analysis following conditions must be satisfied.

1. Equillibrium condition

$$\Sigma F = 0$$ 186

$$\Sigma M = 0$$

In all types of analysis equillibrium eq. is always satisfied

2. Mechanism condition.

At collapse sufficient no of plastic hinge must be develop so as to transform a part or whole of the structure into a mechanism leading to collapse.

For complete collapse of the structure no of plastic hinge required to be formed $= (r+1)$

$r \rightarrow$ Degree of redundancy of structure.

3. Yield condition.

At collapse B.M. at any section must not exceed the fully plastic moment capacity of the section

If all the above 3 condn are satisfied, a unique lowest value of collapse load will be achieved.

Based on the above 3 condn we get the following theorems based on which plastic analysis is performed / done.

a) Upper Bound theorem. or kinematic theorem

b) Lower bound theorem or static theorem.

Upper Bound theorem. This theorem satisfies equillibrium and mechanism condition.

Load determined by assuming a mechanism will always be greater than equal to the collapse load ($P \geq P_u$).

$P_u \rightarrow$ collapse load.

This theorem can also be stated as

Of the various possible mechanism, the correct mechanism is the one for which the collapse load is min.

Lower Bound Theorem.

Load determined on the basis of any collapsed B.M.D, in which B.M. at any secn is less than plastic moment, will be less than or equal to actual collapse load i.e $P \leq P_u$

This theorem satisfies equillibrium and yield condn

Methods of Analysis.

a) Static Method.
b) Kinematic Method.

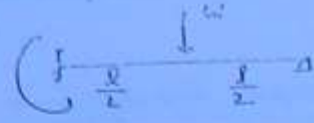
Static Method.

Steps

1) Select the redundant force
Moment will be taken as redundant

$D_s = 1$

2) Draw free BMD and redundant BMD.



Free BMD

188

Redundant BMD

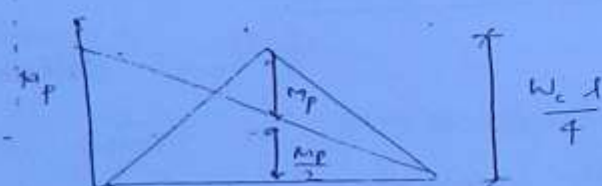
3) A combing BMD is drawn in such a way that a mechanism is formed.

Degree of redundancy = 1

hence no of plastic hinge req. for complete collapse = 2

They will form at support & below point load.

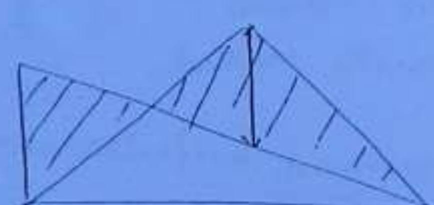
4) collapse load is found out by working out the equilibrium equation



5) It is checked that B.M at every sech is less than M_p

$$M_p + \frac{M_p}{2} = \frac{W_c l}{4}$$

$$W_c = \left(\frac{6M_p}{l}\right)$$



Kinematic Method.

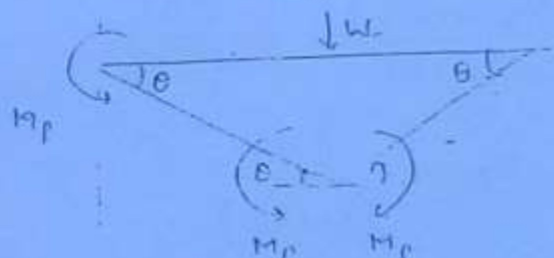
- Locate the possible places of plastic hinges and ascertain the various possible mechanics

collapse load is found out by applying the principal of virtual work.

BMD of collapse mechanism is drawn and it is checked that BM at any secⁿ is not more than M_p

(189)

No. of plastic hinge required = 2



$$-M_p\theta - M_p\theta - M_p\theta + W_c\frac{l}{2}\theta = 0$$

$$3M_p\theta = \frac{W_c\,\theta\,l}{2}$$

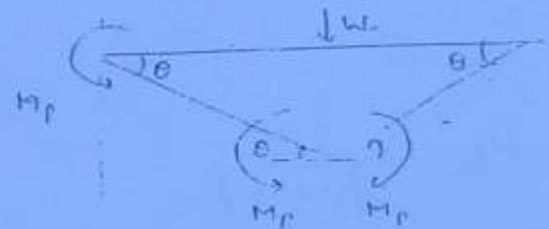
$$W_c = \frac{6M_p}{l}$$

Kinematic Method

- Locate the possible places of plastic hinges and ascertain the various possible mechanics
- collapse load is found out by applying the principal of virtual work
- BMD of collapse mechanism is drawn and it is checked that BM at any secn is not more than M_p.

189

No. of plastic hinge required = 2



$$-M_p\theta - M_p\theta - M_p\theta + W_c\frac{l}{2}\theta = 0$$

$$3M_p\theta = \frac{W_c\,\theta\, l}{2}$$

$$W_c = \frac{6M_p}{l}$$

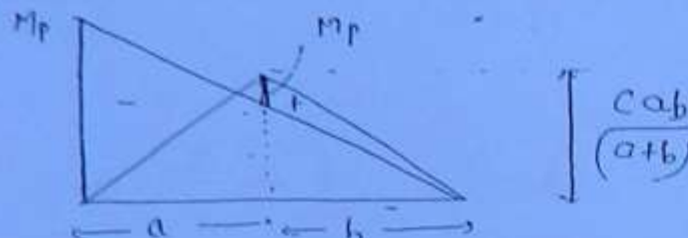
8 i) Find M_p if c is the collapse load

ii) Find the position of load for collapse load to be min.

$R = 1$ (for vertical loading)

No of hinge required for complete collapse $= R+1 = 2$

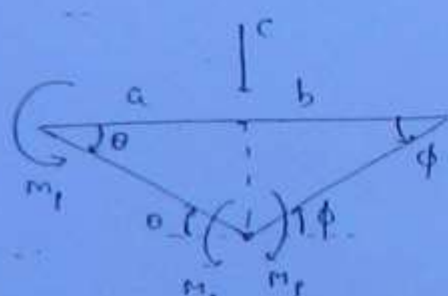
190



$$M_p + \frac{M_p \cdot b}{(a+b)} = \frac{cab}{(a+b)}$$

$$M_p = \frac{cab}{(a+2b)}$$

Kinematic method



$$-M_p\theta - M_p\theta - M_p\phi + c \cdot b\phi = 0$$

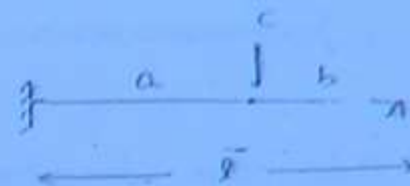
$$a\theta = b\phi$$

$$-2M_p\theta - M_p\frac{a\theta}{b} + ca\theta = 0$$

$$M_p = \frac{cab}{a+2b}$$

(ii) $$\frac{Cab}{a+2b} = M_p$$

$$C = \frac{M_p(1+2b)}{ab}$$



$$= \frac{M_p(l+b)}{(l-b)b}$$

for C to be min

$$\frac{dC}{db} = 0$$

$$\frac{M_p(l-b)b - M_p(l+b)(l-2b)}{\{(l-b)b\}^2} = 0$$

$$(l-b)b - (l+b)(l-2b) = 0$$

$$lb - b^2 - (l^2 - 2bl + bl - 2b^2) = 0$$

$$lb - b^2 - l^2 + bl + 2b^2 = 0$$

$$b^2 + 2bl - l^2 = 0$$

$$b = \frac{-2l \pm \sqrt{4l^2 + 4l^2}}{2}$$

$$= -l + \sqrt{2}\,l$$

$$= l(\sqrt{2} - 1)$$

$$b = 0.414\,l$$

That in case of propped cantilever with point load collapse load will be min when load is at $0.414\,l$ from the hinged end c — collapse load

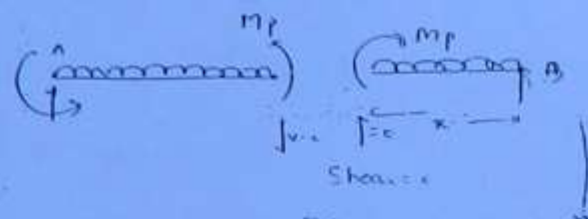
Find M_p

In this case location of plastic hinge needs to be decided first.

192

Method 1

The plastic hinge will form at support and second plastic hinge will form somewhere in span, the location of plastic hinge will be at the point of zero shear because the beam is loaded with u.d.l.

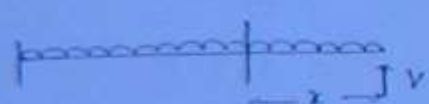


$\Sigma M_B = 0$

$$M_p - \frac{Cx^2}{2} = 0$$

$$M_p = \frac{Cx^2}{2}$$

SF = 0 at x distance from hinged end



$$V - Cx = 0$$

$$Vx - \frac{Cx^2}{2} = M_p$$

$$Cx^2 - \frac{Cx^2}{2} = M_p$$

$$\frac{Cx^2}{2} = M_p$$

At support A

$$Vl - \frac{Cl^2}{2} = -M_p$$

$$Clx - \frac{Cl^2}{2} = \frac{-Cx^2}{2}$$

$\Sigma M_A = 0$

$$2M_p - \frac{C(l-x)^2}{2} = 0$$

$$Cx^2 - \frac{C(l-x)^2}{2} = 0$$

$$2x^2 - l^2 - x^2 + 2lx = 0$$

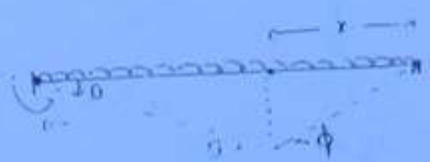
$$x^2 + 2lx - l^2 = 0$$

$$x = 0.414\,l$$

$$x^2 + 2lx - l^2 = 0$$

$$x = 0.414\,l$$

$\Rightarrow$ By Kinematic method



$$-M_p\theta - M_p\theta - M_p\phi + \text{UDL (area under the deflected curve)} = 0$$

$$-2M_p\theta - M_p\phi + c\,\frac{1}{2}\,l.(l-x)\,\theta = 0 \quad \text{--- (I)}$$

$$x\phi = (l-x)\,\theta$$

$$\phi = \frac{(l-x)}{x}\,\theta \quad \text{--- (II)}$$

$$-2M_p\theta - M_p\frac{(l-x)}{x}.\theta + c\,\frac{l\,(l-x)}{2}\,\theta = 0$$

$$-M_p\left(2 + \frac{l-x}{x}\right) + \frac{cl\,(l-x)}{2} = 0$$

$$-M_p\,\frac{(l+x)}{x} + \frac{cl\,(l-x)}{2} = 0$$

$$M_p = \frac{cl(l-x)\,x}{2(l+x)}$$

For M_p to be max. $\frac{dM_p}{dx} = 0$

$$\frac{2(l+x)\,cl\{l-2x\} - cl(l-x)x(2)}{\{2(l+x)\}^2} = 0$$

$$(l+x)(l-2x) - (l-x)\,x = 0$$

$$l^2 - 2xl + \cancel{lx} - 2x^2 - \cancel{lx} + x^2 = 0$$

$$-x^2 - 2xl + l^2 = 0 \Rightarrow x = 0.414\,l$$

Note $M = f(x)$
we calc the max value of M
& maximum value of M is M_p
So we use cond^n $\frac{dM_p}{dx} = 0$

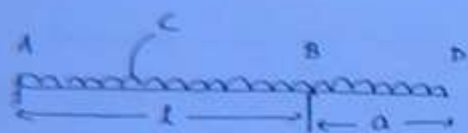
$$M_P = \frac{cx^2}{2}$$

$$= \frac{c(.414l)^2}{2} = (.085)cl^2 = \frac{(3-2\sqrt{2})}{2}cl^2$$

$$= 0.6863\left(\frac{cl^2}{8}\right)$$

194

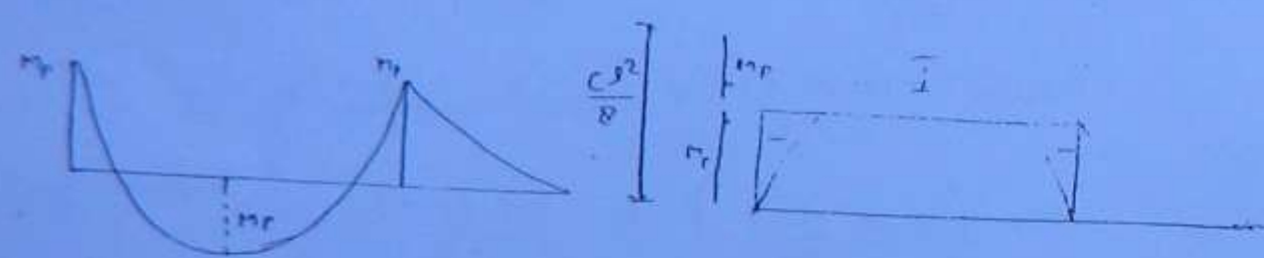
Q.



Find value of a for simultaneous collapse of AB & BD

<u>Collapse BMD</u>

For simultaneous collapse of AB & BD plastic hinge should form at B, at A & in b/w A & B



BD will collapse

<u>Note</u>

Plastic hinge forms at .414 l when moment at the hinged end = 0. But if there is moment at the hinged end plastic hinge will not form at 0.414 l

$$\frac{ca^2}{2} = M_p$$

$$\frac{cl^2}{8} = 2M_p$$

$$\frac{cl^2}{8} = 2\frac{ca^2}{2}$$

195

$$a^2 = \frac{l^2}{8}$$

$$a = \frac{l}{2\sqrt{2}} = 0.353l$$

Note:

If $a < .353 l$

AB will collapse

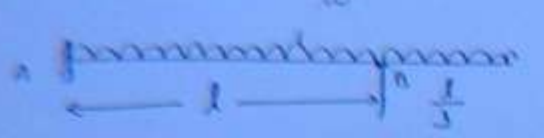
If $a = 0.353 l$

AB & BD will collapse

if $a > .353 l$

BD will collapse

8.



Plastic moment = Mp

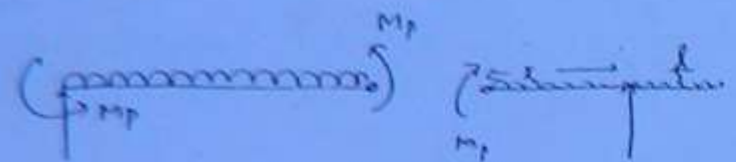
find collapse load = w

196

$R = 1$

No of P.H. for complete collapse = 1+1 = 2

One will form at support and other at x dis from B



$$2M_p - \frac{w(l-x)^2}{2} = 0 \quad \text{——— (I)}$$

$\Sigma M_B = 0$

$$M_p - \frac{wx^2}{2} + \frac{w(l/3)^2}{2} = 0 \quad \text{——— (II)}$$

from (I) & (II)

$$\frac{w(l-x)^2}{2 \times 2} = \frac{wx^2}{2} - \frac{wl^2}{18}$$

$$(l-x)^2 = 2x^2 - \frac{2l^2}{9}$$

$$9(l-x)^2 = 18x^2 - 2l^2$$

$$9(l^2 + x^2 - 2lx) = 18x^2 - 2l^2$$

$$11l^2 - 9x^2 + 18lx = 0$$

$$9x^2 + 18lx - 11l^2 = 0$$

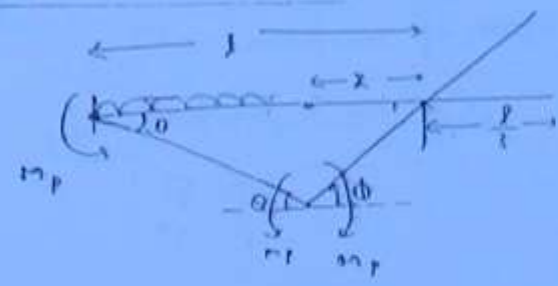
$$x = 0.4907\,l$$

By putting

$$M_p = \frac{w(l-x)^2}{4}$$

collapse load $\quad w = \dfrac{4M_p}{(l-x)^2} = 15.421 \dfrac{M_p}{l^2}$

Kinematic Method



$$(l-x)\,\theta = x\,\phi$$

$$\phi = \frac{(l-x)\,\theta}{x}$$

197

$$-M_P\theta - M_P\theta - M_P\phi + \frac{1}{2}w\,l\,x\,\phi - \frac{1}{2}w\,\frac{l}{3}\times\frac{l}{3}\,\phi = 0$$

$$-2M_P\theta - M_P\frac{(l-x)\theta}{x} + \frac{wl}{2}(l-x)\theta - \frac{wl^2}{18}\frac{(l-x)\theta}{x} = 0$$

$$-M_P\theta\left[\frac{l+x}{x}\right] + \frac{wl\theta}{2}(l-x) - \frac{wl^2}{18x}(l-x)\theta = 0$$

$$M_P = \frac{\frac{w(l-x)l}{2}\left[1-\frac{l}{9x}\right]}{\frac{(l+x)}{x}}$$

$$= \frac{wl\,(l-x)(9x-l)}{18\,(l+x)}$$

$$= k\,\frac{(l-x)(9x-l)}{(l+x)}$$

$$= \frac{k}{l+x}\left(9xl - l^2 - 9x^2 + xl\right)$$

$$\frac{dM_P}{dx} = 0 \qquad = \frac{k}{l+x}\left(-9x^2 + 10lx - l^2\right)$$

$$(l+x)(-18x + 10l) - (-9x^2 + 10lx - l^2) = 0$$

$$-18lx + 10l^2 - 18x^2 + \cancel{10}lx + 9x^2 - \cancel{10}lx + l^2 = 0$$

$$-9x^2 - 18lx + 11l^2 = 0$$

Q.

Beam of uniform x^{sec} —

M_p is constant

198

$R = 2$

No. of P.H. = 3

find collapse load ω

3 hinges will form. 2 at the support &

1 will form b/w A B b/cos S.F. will be zero only in AB

$\Sigma M_A = 0$ of left FBD

$$2M_p - \frac{\omega x^2}{2} = 0$$

$$M_p = \frac{\omega x^2}{4}$$

$\Sigma M_D = 0$

$$2M_p = \omega\left(\frac{l}{2} - x\right)\left[\frac{l}{2} + \frac{l}{4} - \frac{x}{2}\right]$$

$$= \omega\left(\frac{l}{2} - x\right)\left[\frac{3l}{4} - \frac{x}{2}\right]$$

$$2M_p = \omega\frac{(l-2x)}{2}\frac{(3l-2x)}{4}$$

$$4\frac{\omega x^2}{4} = \frac{\omega}{4}\left[3l^2 - 2lx - 6lx + 4x^2\right]$$

$$4x^2 = 4x^2 - 8lx + 3l^2$$

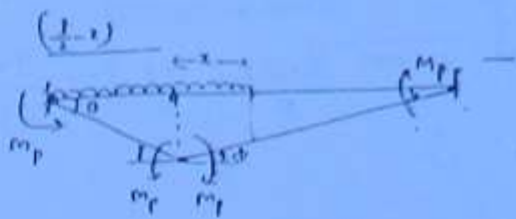
$$8lx = 3l^2$$

$$x = \left(\frac{3l}{8}\right)$$

collapse load $\omega = \dfrac{4M_p}{x^2} = \dfrac{4M_p \times 64}{9l^2} = \dfrac{256}{9}\dfrac{M_p}{l^2}$

$= 28.44 M_p$

By kinematic method.



$$\left(\frac{l}{2}-x\right)\theta = \left(\frac{l}{2}+x\right)\phi$$

199

$$-M_p\theta - M_p\theta - M_p\phi - M_p\phi + \frac{1}{2}\omega\left(\frac{l}{2}-x\right)\left(\frac{l}{2}+x\right)\phi$$

$$+ \left(\frac{l}{2}+x\right)\phi + \frac{l}{2}\phi\Big]\frac{1}{2}\times x\,\omega$$

$$-2M_p\frac{\left(\frac{l}{2}+x\right)}{\left(\frac{l}{2}-x\right)}\phi - 2M_p\phi + \frac{1}{2}\omega\phi\left(\frac{l^2}{4}-x^2\right) + \frac{\omega x\phi}{2}(l+x) = 0$$

$$-2M_p\frac{(l+2x)}{(l-2x)} - 2M_p + \frac{\omega}{2}\left[\frac{l^2}{4} - \cancel{x^2} + lx + \cancel{x^2}\right] = 0$$

$$+2M_p\left[\frac{l+\cancel{2x}+l-\cancel{2x}}{(l-2x)}\right] = +\frac{\omega}{2}\left(\frac{l^2}{4}+lx\right)$$

$$4M_p\frac{l}{(l-2x)} = \frac{\omega}{8}\left(l^2+4lx\right)$$

$$\omega = (32M_p)\frac{l}{(l-2x)(l^2+4lx)}$$

$$\frac{d\omega}{dx} = 0 \qquad = \frac{kl}{\left(l^3+4l^2x-2xl^2-8lx^2\right)}$$

$$= -l\left(4l^2-2l^2-16lx\right) = 0$$

$$2l^2-16lx = 0$$

$$x = \frac{1}{8}l$$

M_1 (l+2x)(l-2x)

$$\left(\frac{l}{2}-x\right) = \frac{l}{2}-\frac{l}{8} = \frac{4-1}{8} = \left(\frac{3l}{8}\right)$$

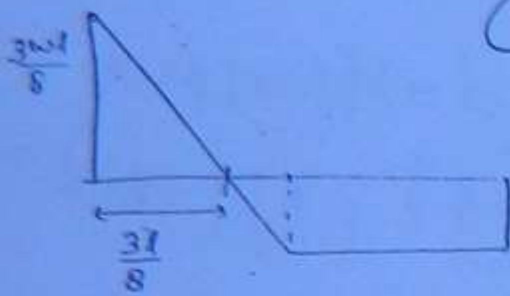
Ans $x = \frac{3}{8}l$

Static method

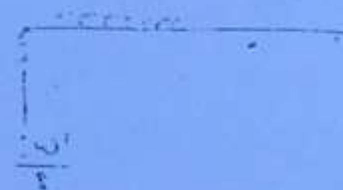
Free BMD

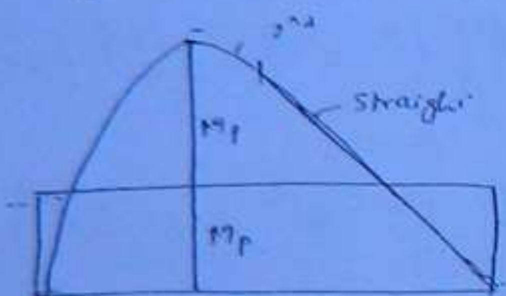
$$M_{max} = \frac{1}{2} \times \frac{3wl}{8} \times \frac{3l}{8} = \frac{9wl^2}{128}$$

200



SFD



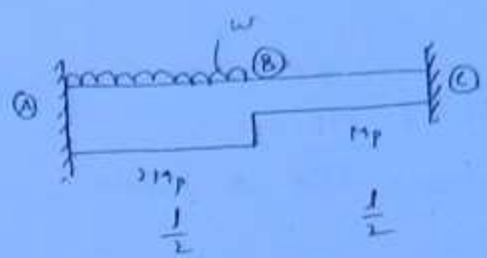


$$2M_p = \frac{9wl^2}{128}$$

$$wl = \frac{256 M_p}{9l^2}$$

Note: In case of fixed beam with constant x^{sec^n} location of plastic hinge in the span will be at the location of max. moment in free BMD

6.



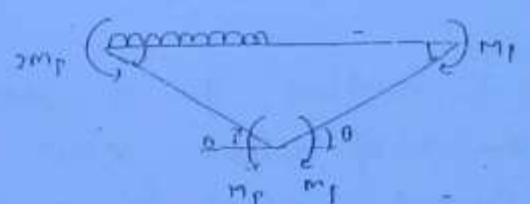
Find collapse load w

Degree of redundancy = 2

29

No. of P.H. req. = 3

Case I Mode of failure

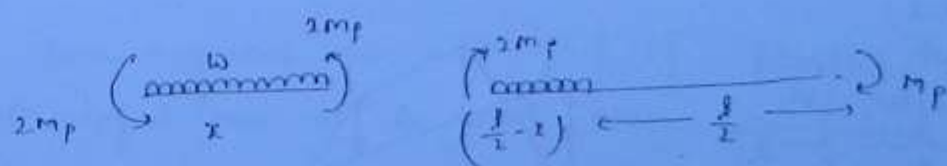


When two member join at a point Plastic hinge form in the member of smaller M_p

$$-5M_p\theta + \frac{1}{2}\,\frac{l}{2}\theta \times \frac{l}{2}\,w = 0$$

$$w = \frac{40M_p}{l^2}$$

Case II



$$4M_p = \frac{wx^2}{2}$$

$$M_p = \frac{wx^2}{8}$$

$$3M_p = w\left(\frac{l}{2}-x\right)\left[\frac{l}{2}+\frac{l}{4}-\frac{x}{2}\right]$$

$$\frac{3wx^2}{8} = w\left(\frac{l}{2}-x\right)\left(\frac{3l}{4}-\frac{x}{2}\right)$$

$$\frac{3x^2}{8} = \frac{(l-2x)(3l-2x)}{8}$$

$$3x^2 = 3l^2 - 8xl + 4x^2$$

$$x^2 - 8xl + 3l^2 = 0$$

$$x = .3944\,l$$

$$W = \frac{8 M_p}{x^2}$$

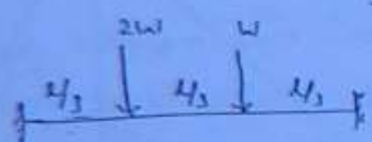
$$= \frac{8 \quad M_p}{(\cdot 3944\, l)^2} = 51.43 \frac{M_p}{l^2}$$

202

Out of various possible mechanism correct mechanism is the one for which loading is min hence mechanism 1 is the correct mechanism & correct collapse load will be

$$\left(\frac{40 M_p}{l^2}\right)$$

Q.

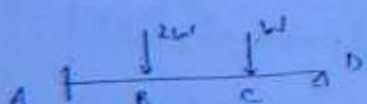


Beam is of uniform x sec^n

find W collapse load

5 l

Note



Two mechanism needs to be checked

Mechanism 1 - P.H. at A & B

" 2 - " A & C

For location of plastic hinge in the span, S.F. Needs to be checked for 3rd hinge

293

Continuous Beam.

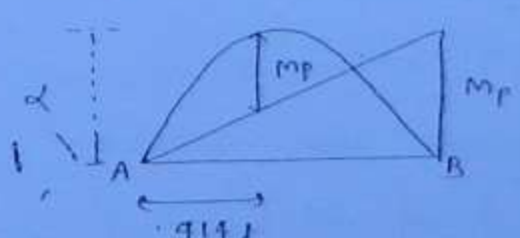
Beam is of uniform x secn find the collapse load.

In case of continuous beam failure of individual span is considered and the correct mechanism will be the mechanism corresponding to which loading will be minimum.

In this case failure of AB & BC is considered as two mechanism.

Mechanism 1. Failure of AB

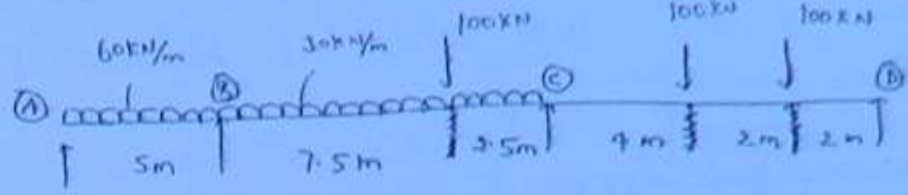
Beam segment AB will fail if plastic hinges are developed one at B and other somewhere b/w A & B



$$\alpha = \frac{wl}{2} \cdot .414l - \frac{w(.414l)^2}{2}$$

$$\alpha = .1213\, wl^2$$

If Plastic moment at B and in the span are same, plast hinge will form at .414 l from the hinged end A

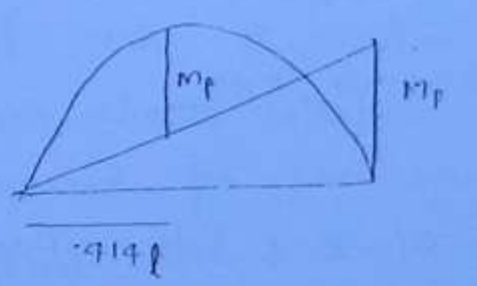


(205)

continuous ~~the~~ beam shown above has uniform x secⁿ find the value of M_p. The given loads are collapse load.

For the continuous beam each of the span will be considered separately for it's failure and M_p required for each of the span will be calculated. if a uniform secⁿ is to be provided, we will adopt secⁿ corresponding to largest M_p

for failure of AB two hinges are required 1 will form at B and other at $\cdot 414\, l$ from hinged end (end A)



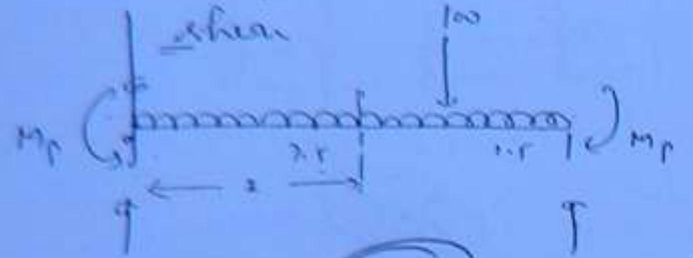
$$M_p = \cdot 6864 \frac{wl^2}{8}$$

$$= 128\cdot 7 \text{ kNm.}$$

If instead of collapse load working load is given and M_p required is to be found out we will have to load the structure with collapse load

Collapse load = load factor x working load.

Span BC :- BC will fail with 3 P.H. Two will form at B & C and one at location of zero shear



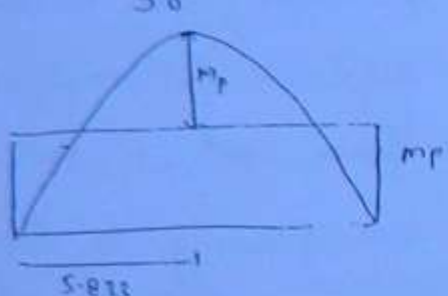
150+25 = 175

206

150+75 = 225

Location of zero shear

$$x = \frac{175}{30} = 5.833 \text{ m}$$

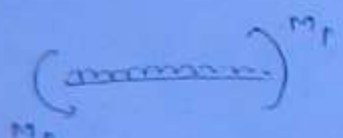


Free BMD ordinate $= 175 \times 5.833 - \frac{30 \times 5.833^2}{2}$

$= 510.47$ KNm

$2M_p = 510.47$

$M_p = 255.237$ KNm.

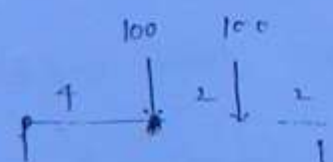


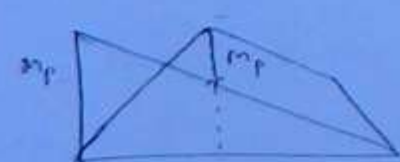
$$2M_p = \frac{30 \times 5.833^2}{2}$$

$M_p = 255.17$ KNm

For failure of CD hinge will form at C & below either of point loading

CD Mechanism 1



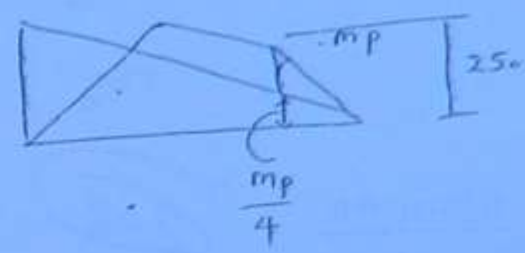


Free B.M.D ordinate :-

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

$\frac{3}{2} M_p = \cancel{300}\ 100$

$M_p = 200$



Mech. 2

$M_p + \frac{M_p}{4} = 250$

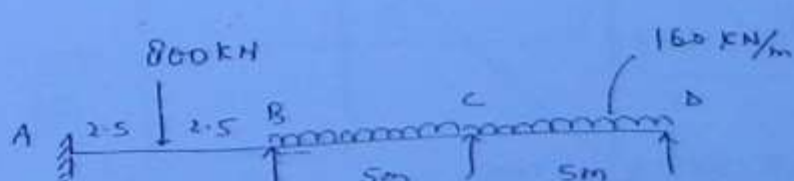
$\frac{5 M_p}{4} = 250$

$M_p = 200$

hence largest Mp will be selected thus M_p required is 255.178 KNm

Q. A continuous beam as shown below is subjected to a collapse load system each span has a uniform secn. If under the action of collapse load system all the spans should collapse, determine the plastic moment required for each span. Assume middle span to be lightest

$M_p\ BC < M_p\ AB$



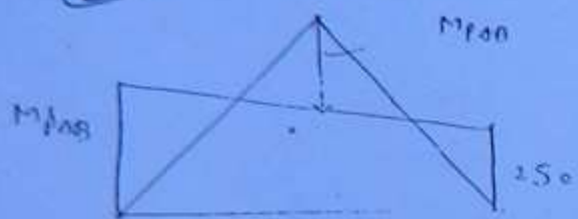
When two sections meet at a point plastic hinge forms corresponding to smaller M_p

Span BC.

$$2M_{p_{BC}} = \frac{wl^2}{8}$$

$$M_{p_{BC}} = \frac{wl^2}{16} = \frac{160 \times 25}{16} = 250 \text{ kNm}$$

Span AB 208

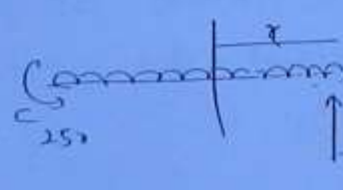


$$M_{pAB} + \frac{M_{pAB} + 250}{2} = 1000$$

$$M_{pAB} = 583.33 \text{ kNm}$$

Span CD

Zero shear location



$$-\frac{250}{5} + 400$$

$$= 350$$

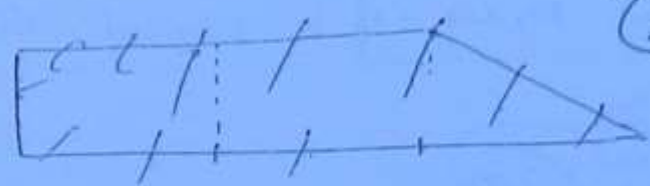
$$x = \frac{350}{160} = 2.1875 \text{ m}$$

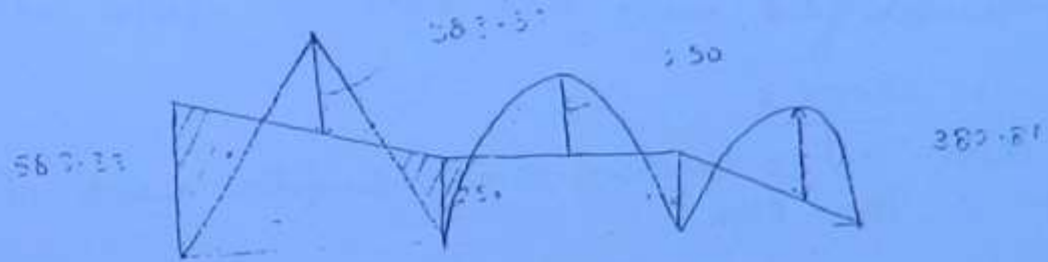
$$M_{pCD} = \frac{160 \times 2.1875^2}{2}$$

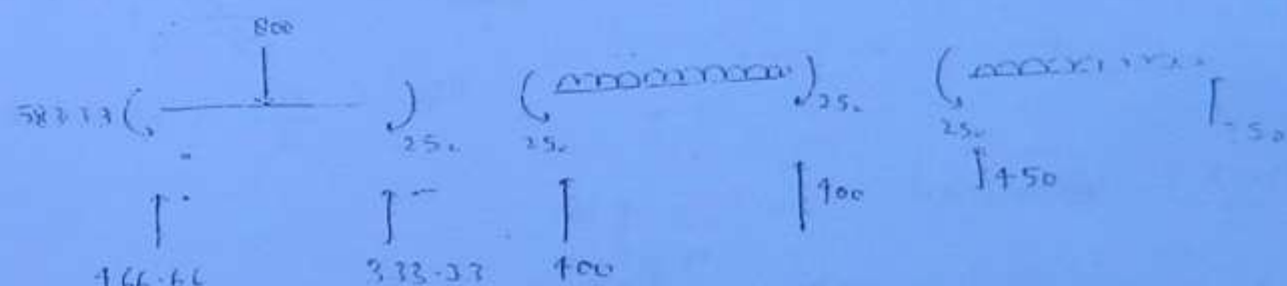
$$= 382.8125 \text{ kNm}$$

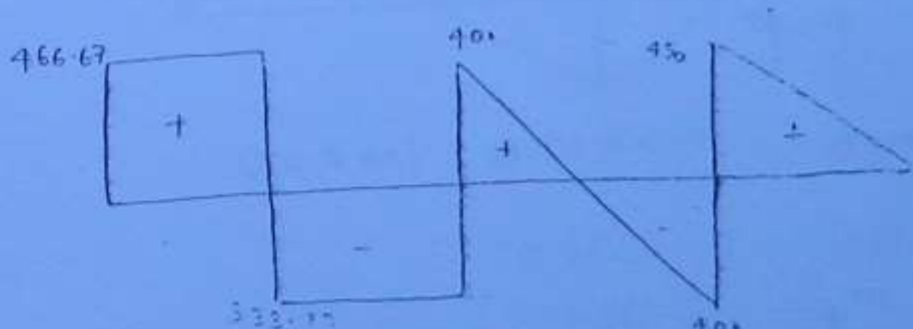
Collapsed BMD

209



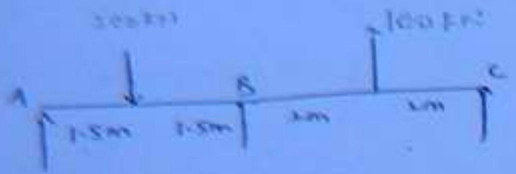






Collapsed SFD

Q



Secⁿ is uniform find M_p req. to be provided. Given loadings are collapsed loading.

210

Draw mechanism are

Mech 1

$$M_p + \frac{M_p}{2} = \frac{200 \times 3}{4}$$

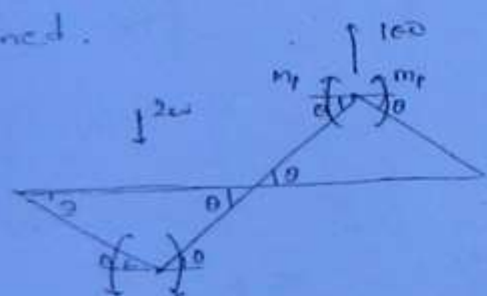
$$M_p = 100 \text{ KNm}$$

Mech 2

$$\frac{3M_p}{2} = \frac{100 \times 4}{4}$$

$$M_p = \frac{200}{3} = 66.67 \text{ KNm}$$

Mech 3 combined.



$$2M_p\theta + 2M_p\theta = 200 \times 1.5\theta + 100 \times 2\theta$$

$$4M_p\theta = 500\theta$$

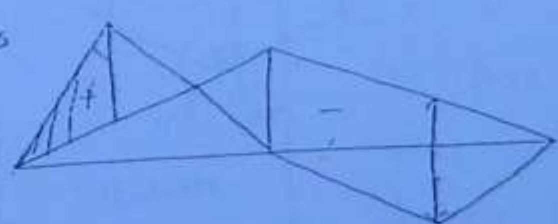
$$M_p = 125 \text{ KN.m}$$

Section will be provided corresponding to $M_p = 125$ KNm collapsed BMD

211

The collapse of beam will occur only corresponding to Mech - 3

To draw collapsed BMD moment at B needs to be calculated



Plastic Analysis for Portal Frames.

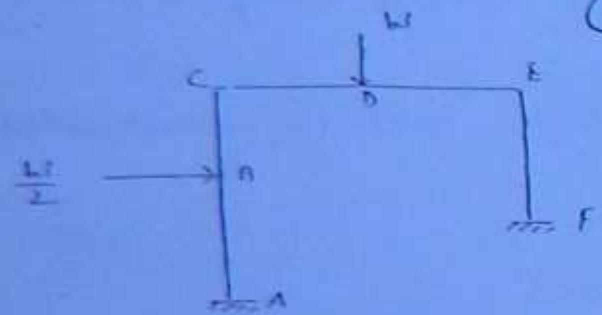
In the portal frames various mechanism considered are

I) Beam Mechanism
II) Sway Mechanism
III) Gable Mechanism

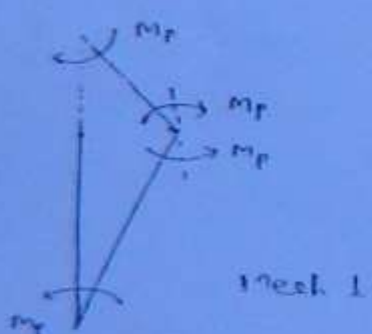
} Independent Mechanism

IV) Combined Mechanism
V) Joint Mechanism.

212

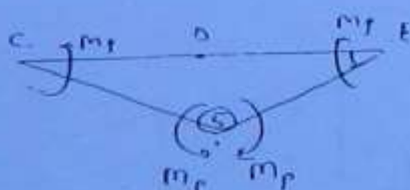


Beam mechanism in AC



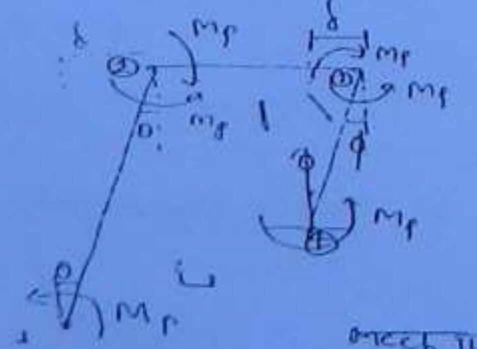
Mech I

Beam mechanism in CE



Mech II

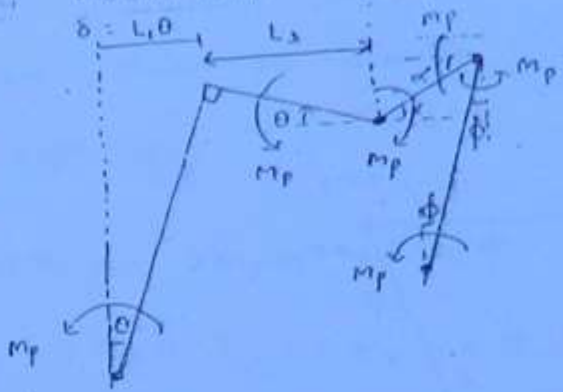
Sway Mechanism.

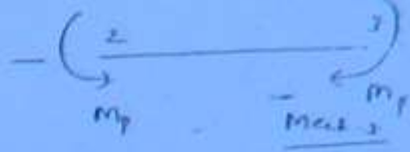


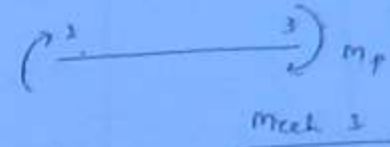
Mech III

Combined Mechanism.

(2+3)







$L_1\theta = L_2\phi$

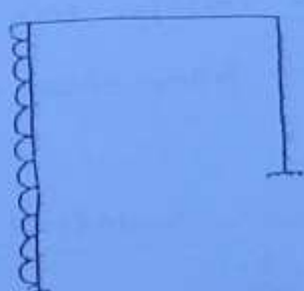
$L_3\theta = L_4\alpha$

Eliminate ② due to opposite moments at 2

213

Mech 1 + Mech 3

eliminate ②

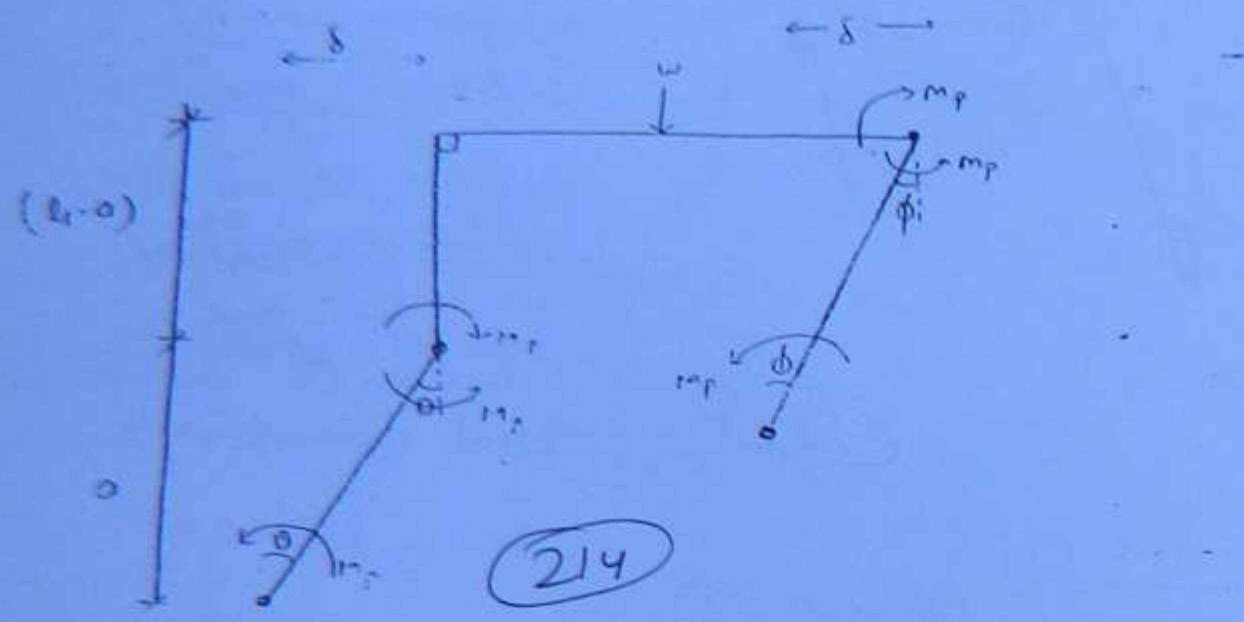


Q.

When beam & sway mechanism are considered combindely as in this case if we consider mech 2 & 3, No of hinges will be five but max. no. of hinge that can be formed is 4 only hence one of the hinge needs to be eliminated. the common hinge b/w beam & sway mechanism are ② & ③

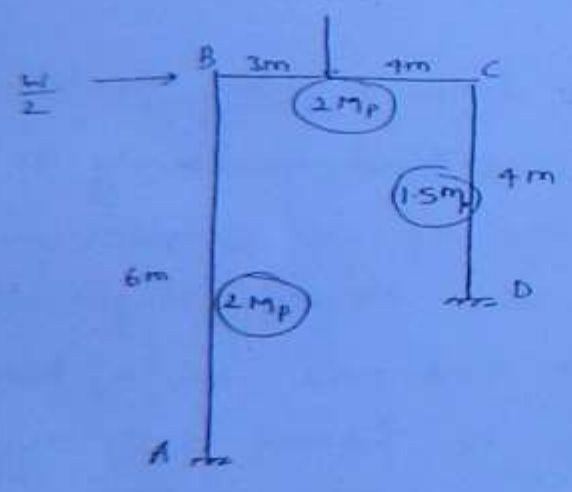
At ② the moment from beam and sway mechanism are opposite in nature but they are of same nature at 3 hence in combined mechanism hinge at 2 will be eliminated.

Combined Mechanism:
(1+3)



$a\theta = L_2 \phi$

Q.

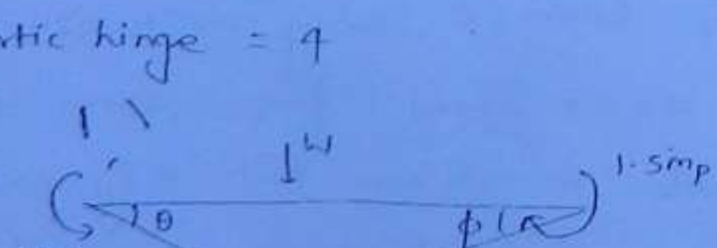


Find collapse load W and draw collapse BMD for the portal frame shown above

R = 3

Max. no. of plastic hinge = 4

Beam mechanism BC



At C, P.H. form due to 1.5 Mp. Also P.H. forms at joint due to smaller Mp.

$$3\theta = 4\phi$$

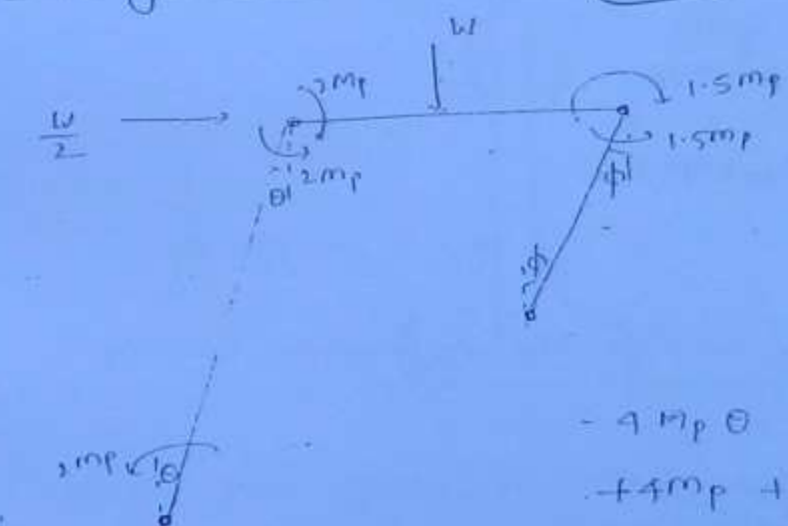
$$-2M_p\theta - 2M_p\theta - 2M_p\phi - 1.5M_p\phi + W \cdot 3\theta = 0$$

$$+4M_p\theta + 3.5M_p \cdot \frac{3}{4}\theta = 3W\theta$$

$$\theta = \frac{53}{24} M_p = 2.208 M_p$$

Sway mechanism.

215



$$6\theta = 4\phi$$

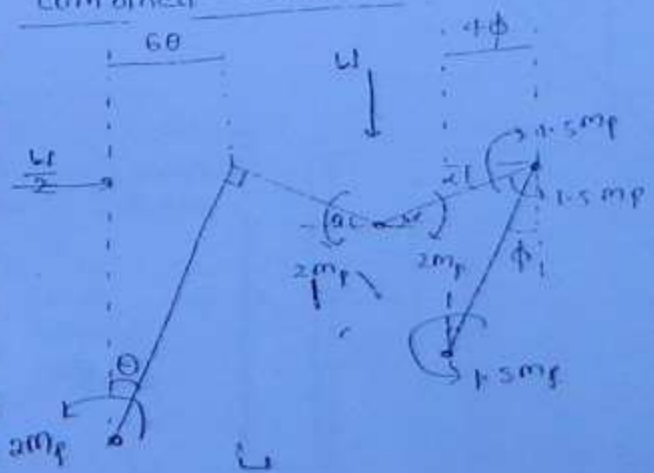
$$-4M_p\theta - 3M_p\phi = -\frac{W}{2}6\theta$$

$$+4M_p + 4.5\theta M_p = 3W$$

$$W = \frac{8.5}{3}M_p$$

$$= 2.833 M_p$$

Combined Mechanism



$$6\theta = 4\phi \quad \text{—(i)}$$

$$3\theta = 4\alpha \quad \text{—(ii)}$$

$$-2M_p\theta - 2M_p\theta - 2M_p\alpha - 1.5M_p\alpha - 3M_p\phi = -W3\theta - \frac{W}{2}6\theta$$

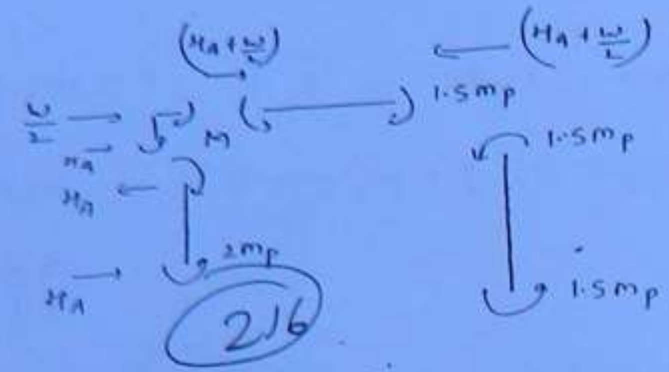
$$-4M_p\theta - 3.5 \times \frac{3}{4}M_p\theta - 3\times1.5M_p\theta = 6W\theta$$

$$W = 1.854 M_p$$

Out of various possible mechanism correct mech. is one for which collapse loading is min

hence collapse load is 1.854 Mp.

Collapsed B.M.D.



$$H_A + H_B + \frac{W}{2} = 0 \quad \text{—(1)}$$

$\Sigma M_D = 0$ for CD portion

$$1.5m_p + 1.5m_p + H_B \times 4 = 0$$

$$H_B = -\frac{3}{4} M_p = -.75\ M_p$$

$$H_A = -H_B - \frac{W}{2} = -.177\ M_p$$

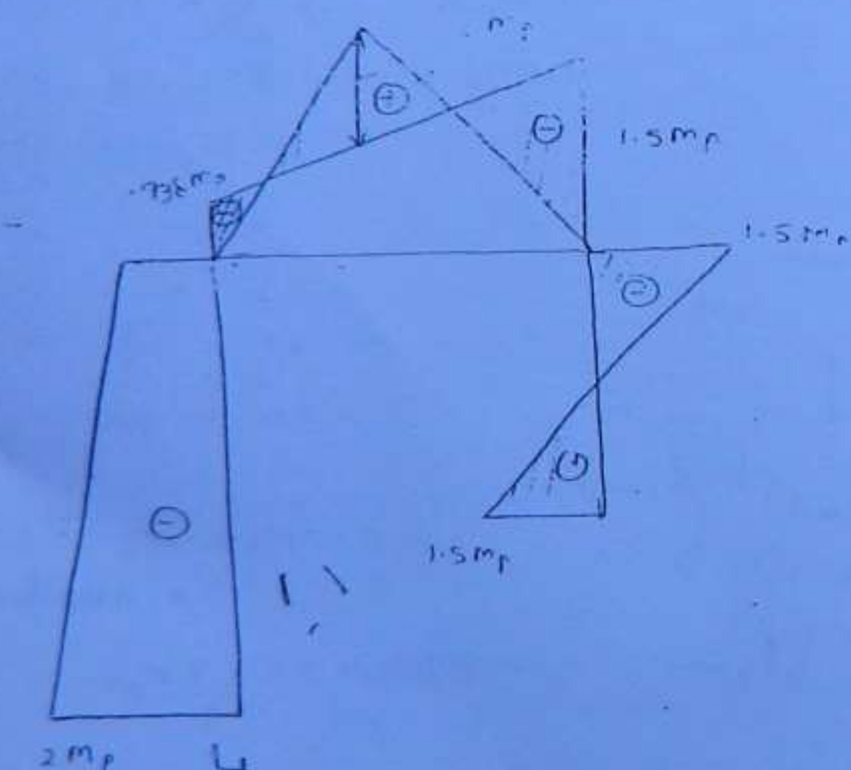
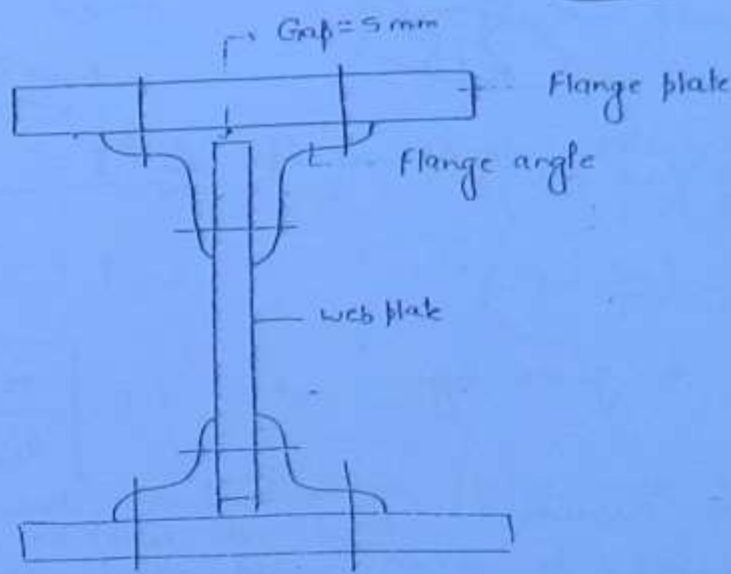


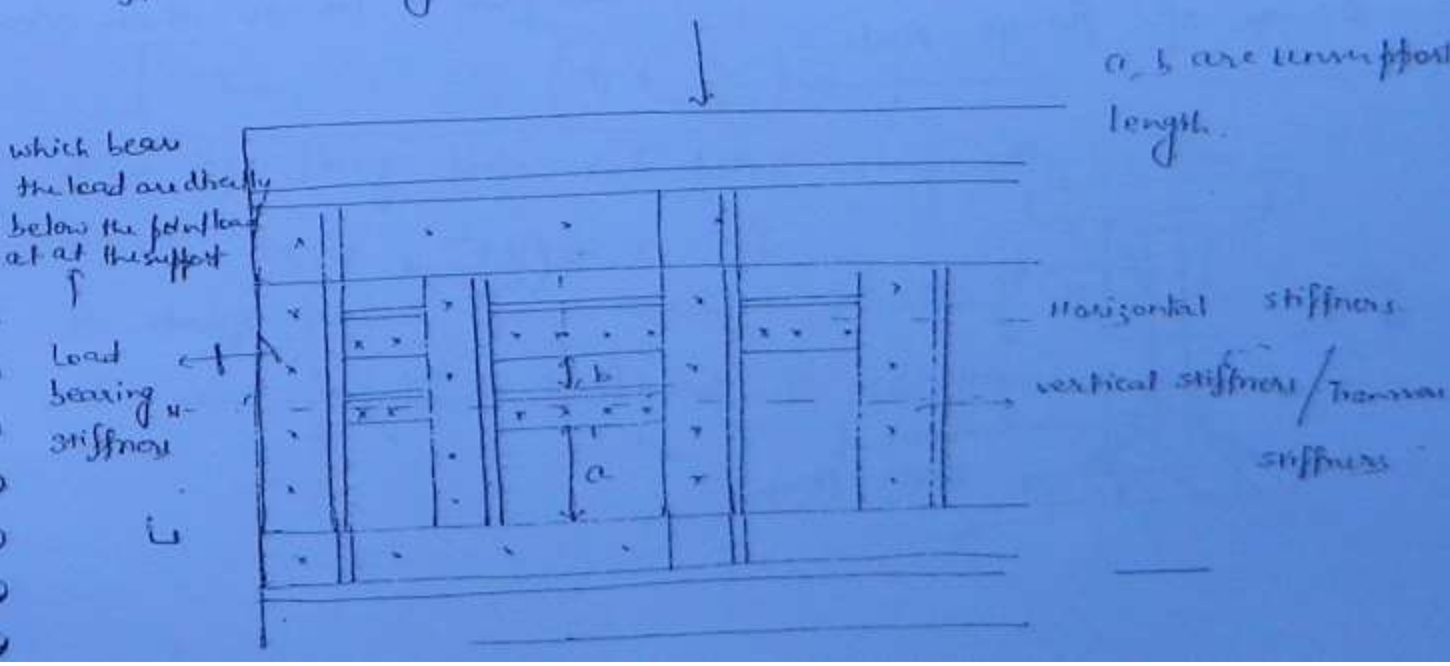
PLATE GIRDERS.

Plate girders (beams made using plates) adopted when the span and loading becomes large. For span upto 10 m I sec^n + plates may be used but for larger span 15 to 30 m plate girders are used

21.7



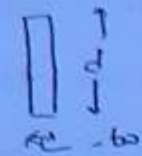
As the plate girder are deeper chances of web buckling is more hence web has to be supported by using stiffners. stiffners actually increase the moment of inertia



larger unsupported length $\not> 270\, t_w$

Smaller " " " $\not> 180\, t_w$

Design of Web Plate



Web primarily resist shear hence web is designed for shear

For unstiffened web permissible web av shear stress $= 0.4 f_y$

stiffened " 218 " " $= f\left(\frac{d}{t_w}, c\right)$

per. av shear stress decreases if $\frac{d}{t_w} \uparrow$

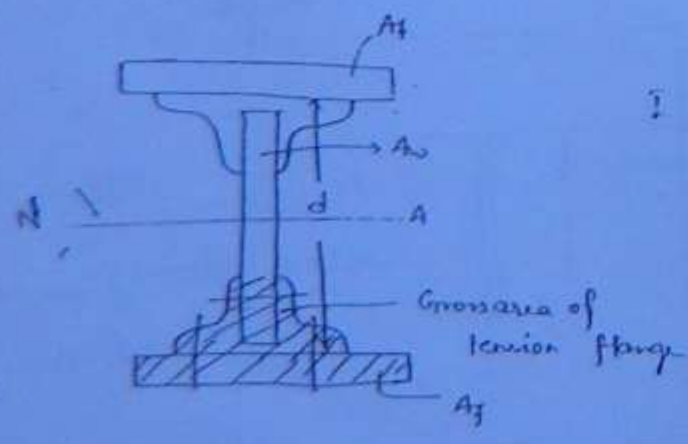
" " " " $c \uparrow$

Economical depth is given by $d = 1.1 \sqrt{\frac{M}{\sigma_{bt} \times t_w}}$

Thickness will be assumed and d will be calculated from economical depth formula and it will be ensured that with this dimension $\frac{V_{max}}{d \times t_w} \leq$ per. av. shear stress.

Spacing b/w vertical stiffners will be choosen such that the calculated average shear stress is less than equal to per. av shear stress

Design of flange Plate



$$I = 2 A_f \left(\frac{d}{2}\right)^2 + \frac{t_w d^3}{12}$$

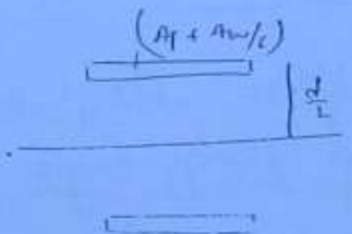
$$= \left(A_f + \frac{A_w}{6}\right)\frac{d^2}{2}$$

flange plate is designed to resist bending moment

$$MOR = \sigma_{bt} \times Z$$

$$M_{max} = \sigma_{bt} \times \frac{I}{d/2}$$

$$= \underset{165\,MPa}{\sigma_{bt}} \underbrace{\left(A_f + \frac{A_w}{6}\right)}_{\text{Effective flange area on compression side}} d$$



$$I = 2\left(A_f + \frac{A_w}{6}\right)\left(\frac{d}{2}\right)^2$$

$$= \left(A_f + \frac{A_w}{6}\right)\frac{d^2}{2}$$

On tension side reduction in flange area will occur due to formation of holes hence the area of flange plate has to increase. The effective flange area for tension side is taken as $\left(A_f + \frac{A_w}{8}\right)$

$$\boxed{M_{max} = \sigma_{bt}\left(A_f + \frac{A_w}{8}\right) d}$$

Area of flange plate calculated using this formula will be more.

For design purpose flange area is calculated than on tension side and same is adopted for compression side.

Check for bending

$$\frac{M_{max} \cdot Y_{max}}{I_{gross}} \leq \sigma_{bc}$$

$$\frac{M_{max} \cdot Y_{max}}{I_{gross}} \times \frac{(\text{G.A. of tension flange})}{(\text{Net area of tension flange})} \leq \sigma_{bt}$$

220

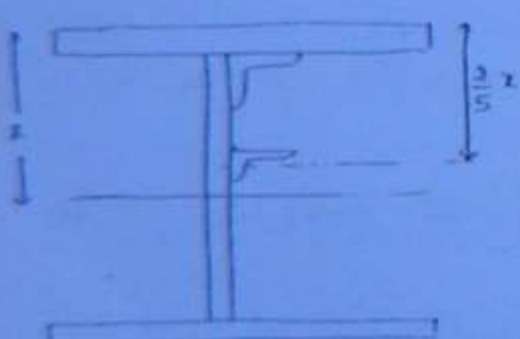
Tension flange will comprise of flange plate, flange angles and area of web plate included b/w flange angle

Design of stiffner

i) $\frac{d}{t_w} < 85$ No stiffner required (No hor. or vert. stiffner)

ii) $85 < \frac{d_2}{t_w} \leq 200$ only vertical stiffner is required

iii) $200 < \frac{d_2}{t_w} \leq 250$ Apart from vertical stiffner one horizontal stiffner is provided at a distance from compression flange equal to $\frac{2}{5}$th of the distance of compression flange from N.A.



iv) if $\frac{d}{t_w} \in [250 - 400]$ One more hor. stiffner is provided at N.A.

$250 < \frac{d}{t_w} \leq 400$

t_w

$t_w \geq \frac{d}{400}$ for stiffened web min thickness of web $= \left(\frac{d}{400}\right)$

for unstiffened web $t_{w_{min}} = \left(\frac{d}{85}\right)$

221

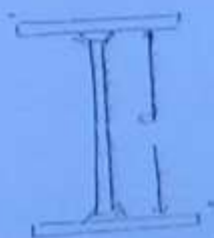
Design of vertical stiffners

spacing is selected such that permissible shear stress is grater than $\frac{V}{d t_w}$.

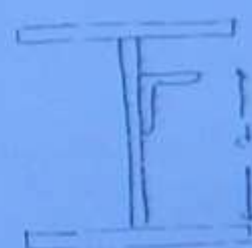
The max. & min spacing of vertical stiffner are $1.5\,d$, or $0.33\,d$, respectively.

Note

fig(i)

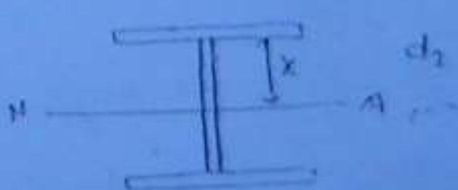


fig(ii)



To decide about the type of stiffner to be provided we work on actual length of web however to design the vertical stiffner i.e. to decide on the cross section of stiffner required we work on d_1 (actual unsupported length)

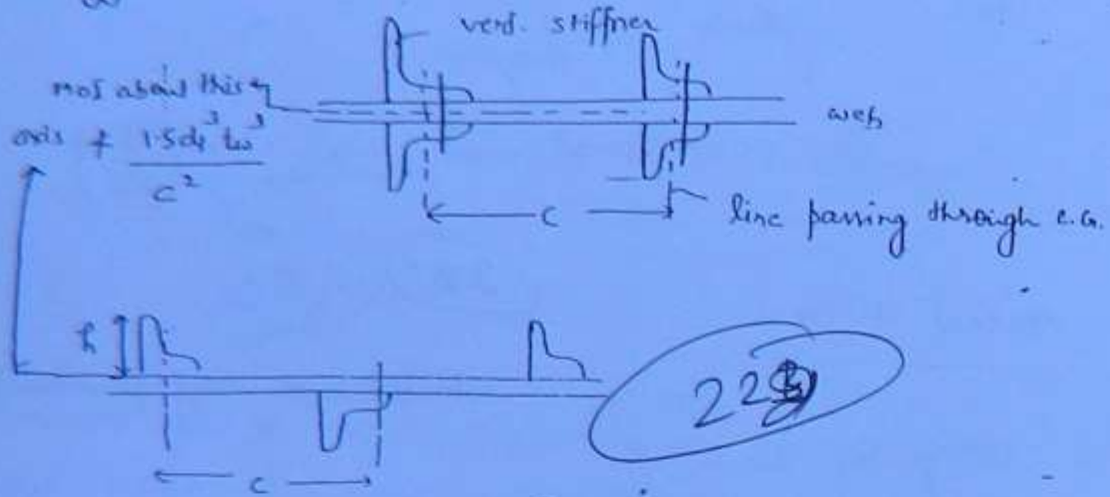
d_2 = twice the clear distance from N.A of the beam to the compression flange



$d_2 = 2x$

Larger & smaller clear dimension should be maintained below $270\,t_w$ & $180\,t_w$

vert. stiffner should be provided as an angle two sides of web plate. If it is provided on one side it should be staggered.



229

the moment of Inertia of angle should not be less than

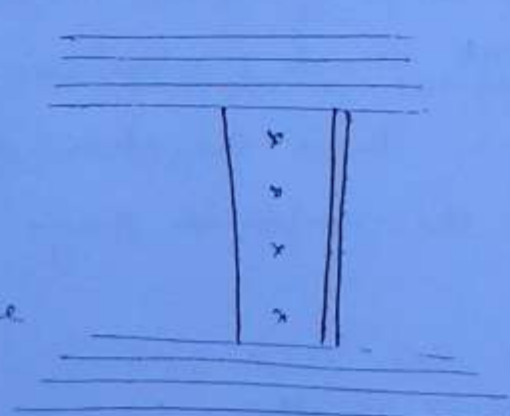
$$I \not< \frac{1.5 d_1^3 t_w^3}{c^2}$$

Connection Design

Connection should be designed for s.f

$$SF = \frac{125 t_w^2}{h} \ N/mm$$

where h is the outstand of stiffner from the web



$$\frac{F_v}{\frac{125 t_w^2}{h}} = \text{Pitch}$$

$f \not> 16\, t$ for angle where
t is the thickness of member

In case of flat plate overhang $\ngtr 12t$

$t \rightarrow$ thickness of plate.

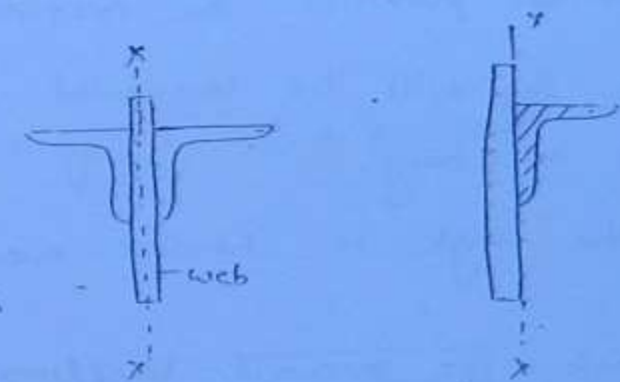
vertical stiffner is primary supposed to resist shear buckling.

Horizontal stiffner

(223)

i) $200 < \frac{d_2}{t_w} \leq 250$ $\qquad I \ngtr 4c\, t_w^3$

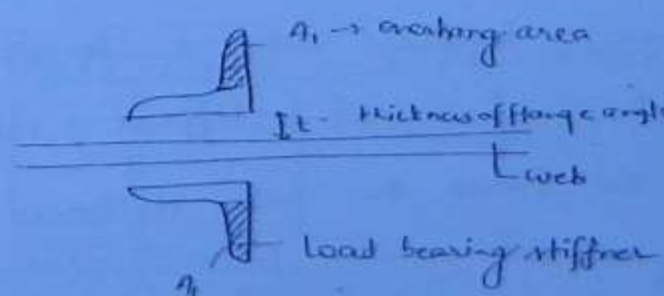
ii) $250 < \frac{d_2}{t_w} \leq 400$ $\qquad I_{xx} \ngtr d_2 t_w^3$

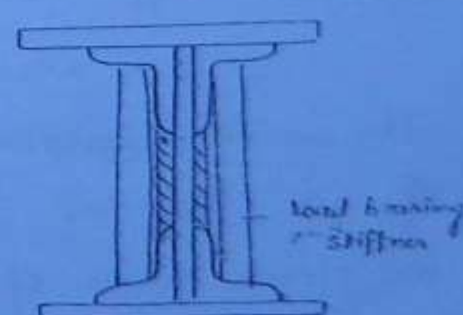


connection of hor. stiffner will be done in same way as vert stiffner.

Horizontal stiffner primarily safeguards against buckling due to bending compression.

Imp. Design of load bearing stiffner.





1 outstanding leg of the stiffner is assumed to provide the necessary bearing area hence bearing area required is found out

224

$$\frac{P}{\sigma_{br}} = \text{bearing area req.}$$

$\sigma_{br} \rightarrow$ perm. bearing stress $= 0.75 f_y$

→ Unequal angle is choosen in such a way that the over hang portion provides the necessary bearing area.
smaller leg will be connected to the web larger leg will be over hang.

3 Once the angle is choosen overall buckling is to be checked.

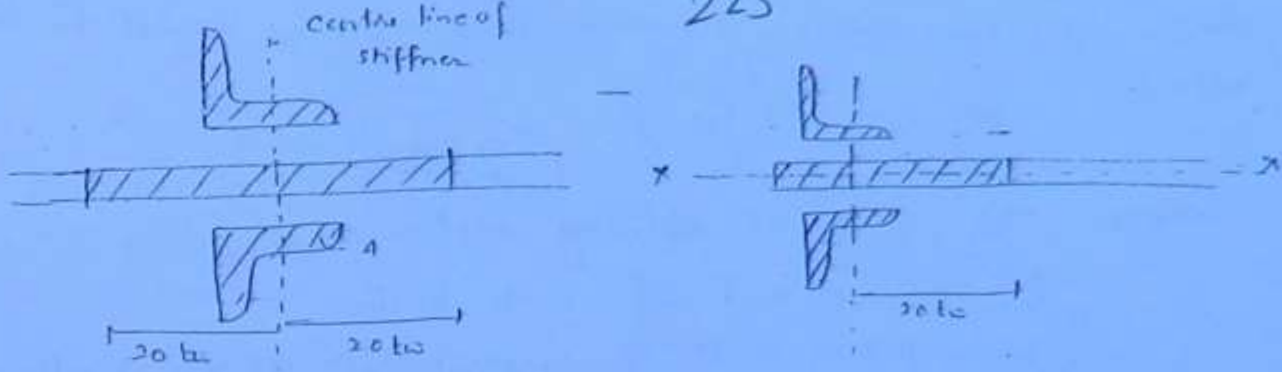
4 To check for overall buckling the stiffner is assumed to be a column. The permissible load capacity of that column should be greater than applied load.
To find out σ_{ac} λ is required and

$$\lambda = \frac{l_{eff}}{r_{min}}$$

Where l_{eff} is taken as $0.7\,l$

$l \rightarrow$ overall depth / length of stiffner

The section resisting compression is assumed to be consisting of a pair of stiffners together with a length of web on either side of centre line of stiffner, where available, equal to 20 times the web thickness.

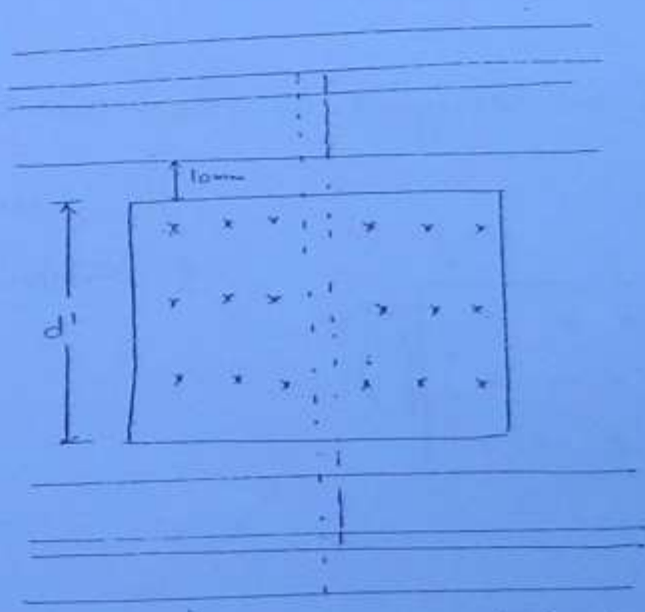


A : Shaded area

= 40 tw × tw + 2A

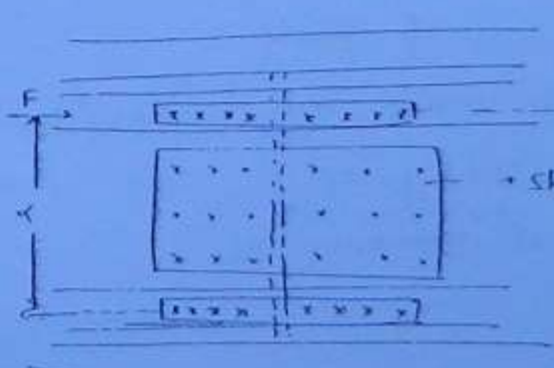
Load carrying capacity : $\sigma_{ac} \times A > P$

Web splices



for exam

In fig (i)



better way.

In fig (i) web splice plate will be assumed to carry 100% shear at the splice location and moment equal to M_w

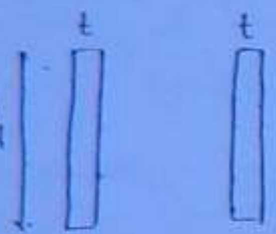
where $$M_w = \frac{M I_w}{I}$$

226

where M → B.M. at splicing section

I_w → M.O.I. of web plate

I → gross M.O.I of original secn about bending axis.



$$\frac{V}{2d't} \le 0.4 f_y$$

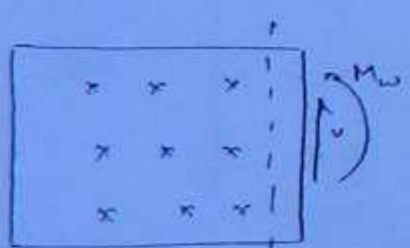
$$MOR = \sigma_{bt} \times Z$$

$$= \sigma_{bt} \frac{2t \times d'^2}{6} \geq M_w$$

connection will be designed as eccentric connection

for direct S.F. = V

& Torsional moment = M_w



In fig (ii) we provide two sets of plate

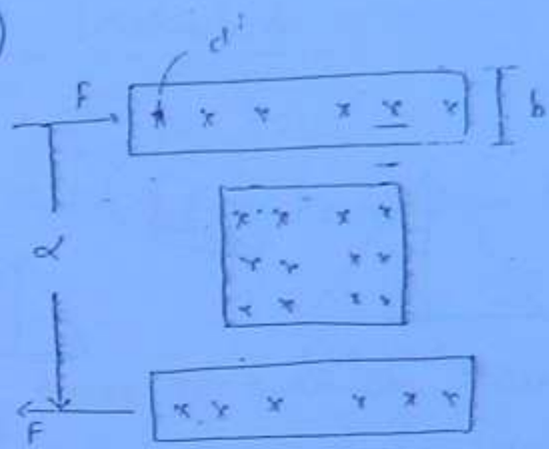
1- Shear splice → Assumed to resist only shear (100% shear at splicing secn)

11- moment splice ↳ Assumed to resist moment M_w

Moment splice

$$F\alpha = M_w$$

$$F = \frac{M_w}{\alpha}$$



$$2(b-d') \times t_f \times \sigma_{at} \geq F$$

$$n = \frac{F}{R_v}$$

This design is for tension and same area is provided for compression

Shear splice

$$\frac{V}{2 d_w \times t} \leq 0.4 f_y$$

$$n = \frac{V}{R_v}$$

Industrial Building

228

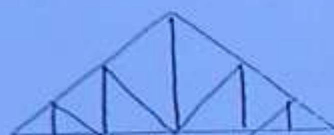
King Post

span < 6 m

Queen post truss

9 m

Fink truss



Howe truss

12 – 18 m

Howe truss (flat)

upto 24 m

Compound fink

12 – 18 m

glass

glass for sun light

Saw tooth

Span

H → Height of truss

$$\text{Slope of truss} = \frac{H}{L/2} = 2\left(\frac{H}{L}\right)$$

$$\text{pitch of truss} = \frac{H}{L}$$

$$\boxed{\text{Slope} = 2 \times \text{Pitch}}$$

Deeper trusses have smaller deflection as compare to shallow truss.

Member forces are larger in shallow truss as compare to deep truss.

For smaller span height of truss is normally taken as $\frac{1}{6}$th of the span.

$$\frac{H}{L} = \frac{1}{6}$$

$$\text{Slope} = \frac{1}{3}$$

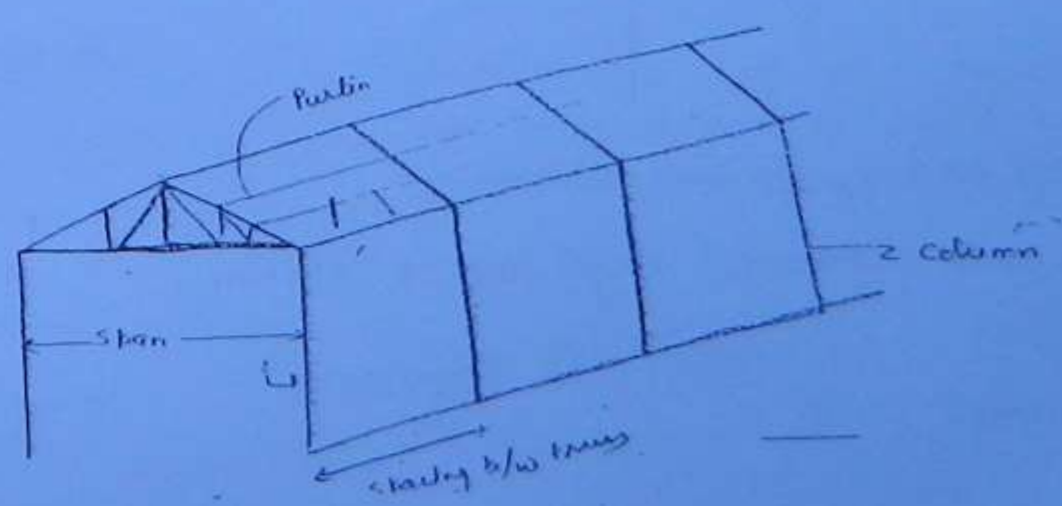
For larger span depth may be taken as $\frac{1}{10}$th of the span

* If GI (Galvanised iron) sheet is used as roof covering pitch of trusses taken as 1 in 6

For AC (Asbestos cement) sheet roof covering pitch may be taken as 1 in 12

Lower pitch is advantageous for wind pressure

Spacing of truss



Generally spacing is taken $\frac{1}{4}$ of span, span upto 15m

& $\frac{1}{5}$th of span, for span > 15m

For economy in roof

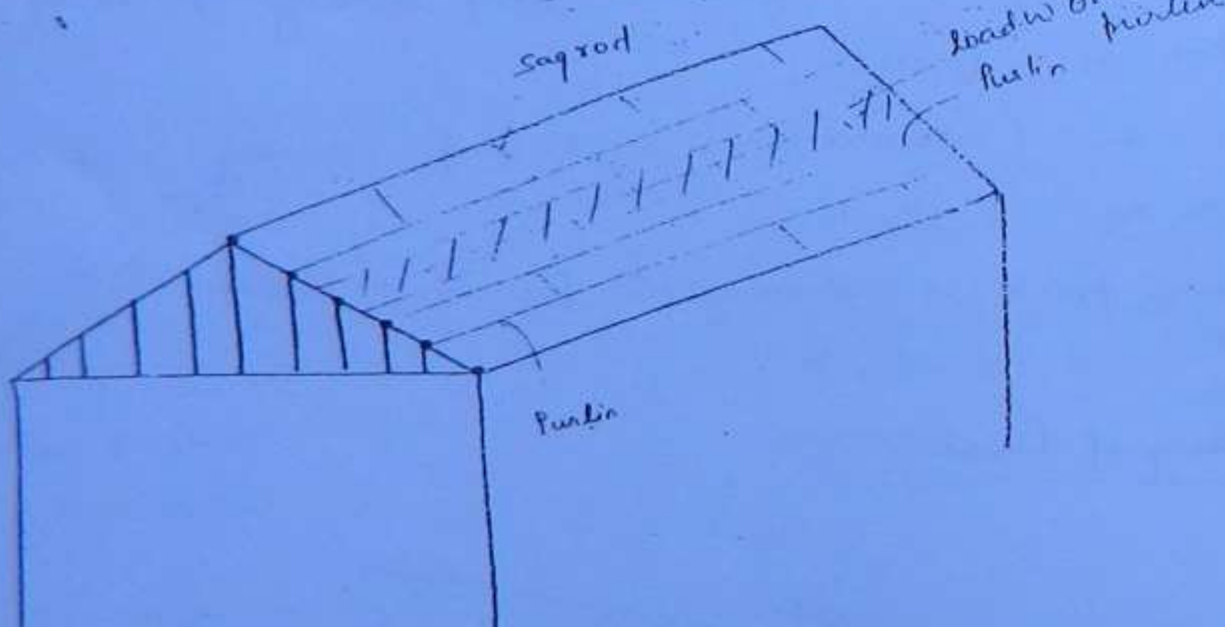
230

$$\frac{\text{cost of truss}}{\text{Area}} = \frac{\text{cost of purlin}}{\text{Area}} \times 2 + \text{cost of roof covering}$$

Area $\Rightarrow$ Area of floor or plan area of building.

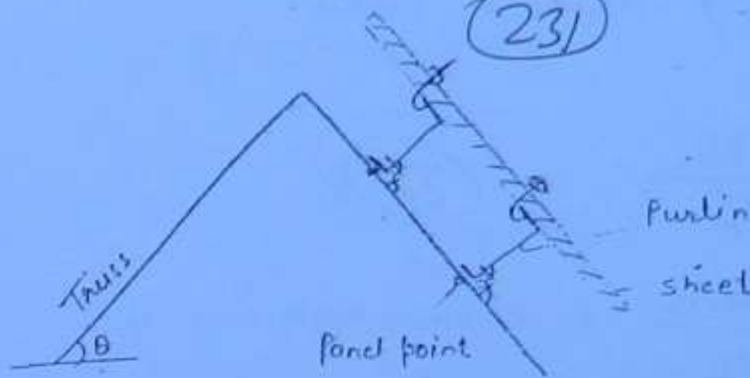
Purlins

Purpose of purlin is to support the roof sheet.

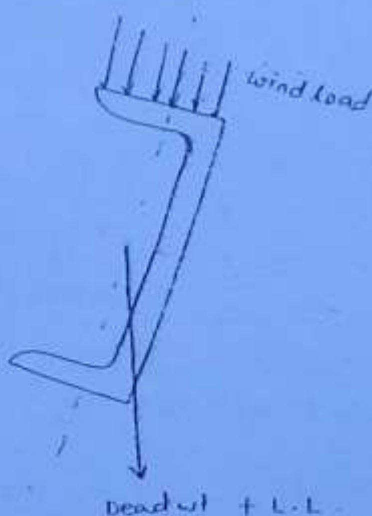


sag rod - Tension member.

Purlin are channel, angle or I section.



Max spacing b/w purlin is 1.4 m



Purlins are subjected to biaxial loading

Hence design of it should be done as biaxial bending.

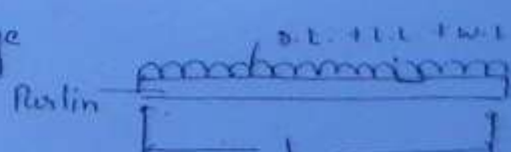
However if slope of roof truss is < 30°

We may avoid design purlin by biaxial bending.

Hence desi

In this case (θ < 30°) angle purlins can be used and it will be designed as uniaxial bending condition.

In this case D.L. , L.L. & W.L. are assumed to be acting ⊥ to the roof coverage



$M_{max} = \frac{wL}{10}$

l → Span of purlin

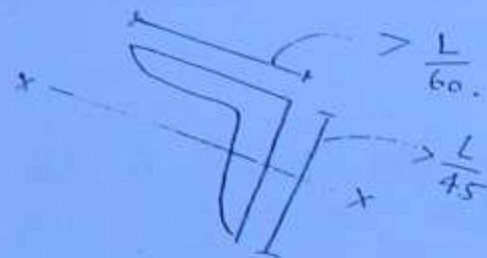
= spacing b/w truss

W → Total load resisted by purlin

$$Z_{req} = \frac{Wl/10}{\sigma_{bc}}$$

238

$$Z_{xx} \geq \frac{Wl/10}{\sigma_{bc}}$$ ← Choose angle such that

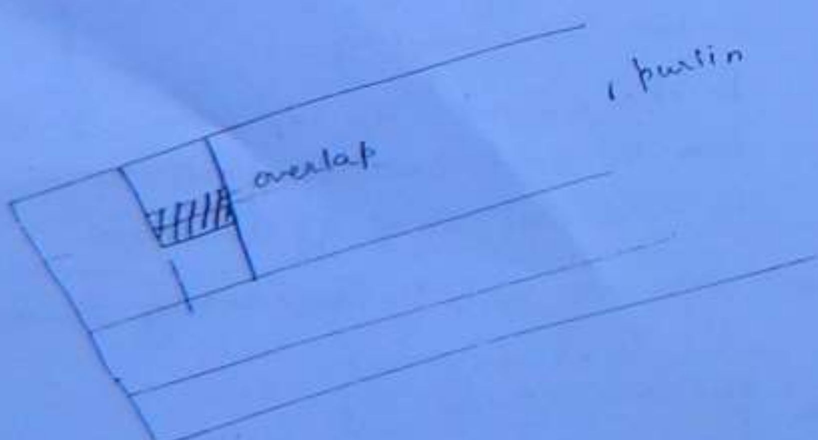


Angle choosen should be such that

width of angle ⊥ to roof covering $> \frac{L}{45}$
" " ∥ to " " $> \frac{L}{60}$

Recommendation only for angle purlin

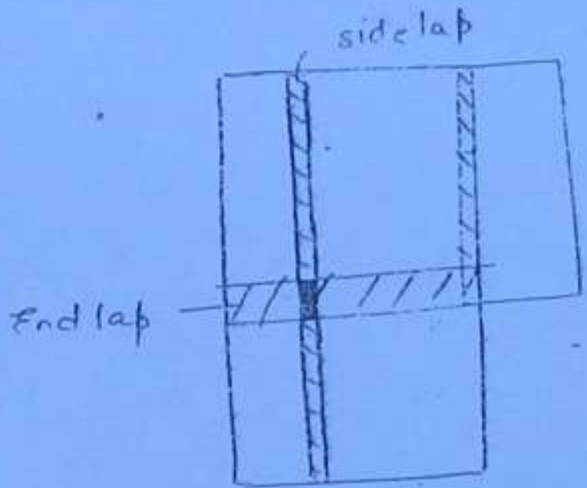
- Sag rod resist tangential component of D.L due to sheet (roof covering) & purlin itself.

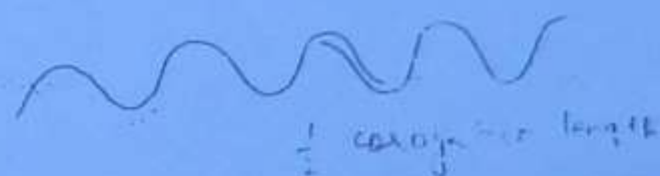


Try to make overlap connection at the location of purlin overlap length

for slope upto 1:3

→ For flatter slope overlap length can be ↑ed





Sid lap for A.C sheet $\frac{1}{2}$ corongation length lap
for G.I sheet it is full corrugation length lap

<u>Note</u>: when 4 sheets overlap at location two sheets are cut at corners this process is called mitring

Northern Lightning

234

Most of the time in a yr. sun moves from east to west through south hence direct sunlight will come in the building from south which will cause shadow formation. To avoid it windows shold be provided on north side so that diffused light come inside the buiding which does not cause any shadow formation.

DSS 190