

## UNIT - II Design of Tension Members & Compression

Tension Members [Ref Ref Design of Steel structures -

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Tension members are also as "tie members"

The form of tension members is governed to a large extent by the type of structure of which it is a part and by the method of joining it to the adjacent members of the structure.

Forms of a tension member :-

1) Rod : O

3) single angle L

2) flat : □

4) double angle section

5) double channel section

6) T-section

7) I-section

8) Multi-angle section

9) I-section with plat

Design strength of tension members

i) Design strength due to yielding of gross section

$$T_{dg} = \gamma_g f_y A_{gross}$$

Where

$f_y$  = yield stress of the material

$A_{gross}$  = gross area ofcls and

$\gamma_g$  = partial safety factor.

## Design Strength due to Rupture of critical section

$$T_{dh} = 0.9 \frac{f_y}{\gamma_m} A_{bh}$$

Where

$\gamma_m$  = partial safety factor for failure of ultimate stress

$f_y$  = ultimate stress of the material and

$A_{bh}$  = net effective area of the mean bar given by

$$A_{bh} = \left[ b - n d_h + \sum_i \frac{P_s^2}{4 g_i} \right] t$$

Where

$b, t$  = width & thickness of plate

$d_h$  = diameter of the bolt hole

[amm is addition]

$g$  = gauge length b/w the bolt holes

$P_s$  = staggered pitch length b/w line of bolt holes

$n$  = no. of bolt holes in the critical section

$i$  = subscript for summation of all the include legs

③ single angles

$$T_{dh} = \frac{0.9 f_y}{\gamma_m} + \frac{B A_{go} f_y}{\gamma_m}$$

Where

$$B = 14 - 0.076 (\text{wtt}) \left( \frac{f_y}{f_u} \right) \left( \frac{b_s}{l_e} \right) \leq \left( \frac{f_y}{f_u} \gamma_m \right) \left( \frac{f_y}{f_u} \gamma_m \right) \geq 0.7$$

Where

$w_l$  = outstand long width

$b_s$  = shear lag width as shown

$L_c$  = length of the end connection that is the distance b/w the centers most bolt in the end joint measured along the load direction or length of the weld along the load

4: Design strength due to the block shear

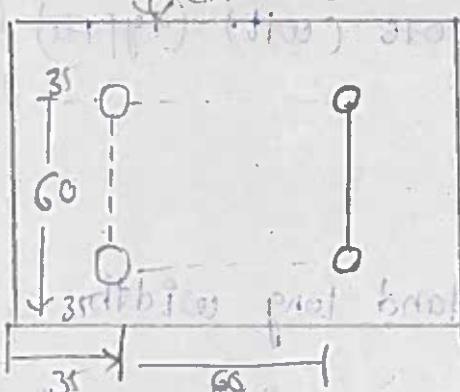
$$T_{db} = \left[ \frac{\text{Arg } f_y}{C\sqrt{3} \gamma_m} + \frac{0.9 P_{th} f_y}{\gamma_m} \right]$$

$$T_{db} = \left[ \frac{0.9 P_{th} f_y}{\sqrt{3} \gamma_m} + \frac{\text{Arg } f_y}{\gamma_m} \right]$$

Where

Arg App.

- ① Problem:- Determine the tensile strength of the plate 130mm x 12mm width the holes for 16mm diameter bolt as shown in fig steel is used is fe 40 grade quality



Sol:- Strength of the plate

A:- Yield of Gross section

B:- Reuse of Critical section

C:- Block shear strength

A:-

$$T_{dg} = \frac{P_g f_y}{8m_s}$$

$$A_g = 132 \times 12 = 1580 \text{ mm}^2$$

$$f_y = 254 \text{ N/mm}^2 \text{ (Assume)}$$

$$8m_s = 1.10$$

$$T_{dg} = \frac{1580}{1.10} (250)$$

$$T_{dg} = 254 \cdot 54 \text{ kN}$$

B:-

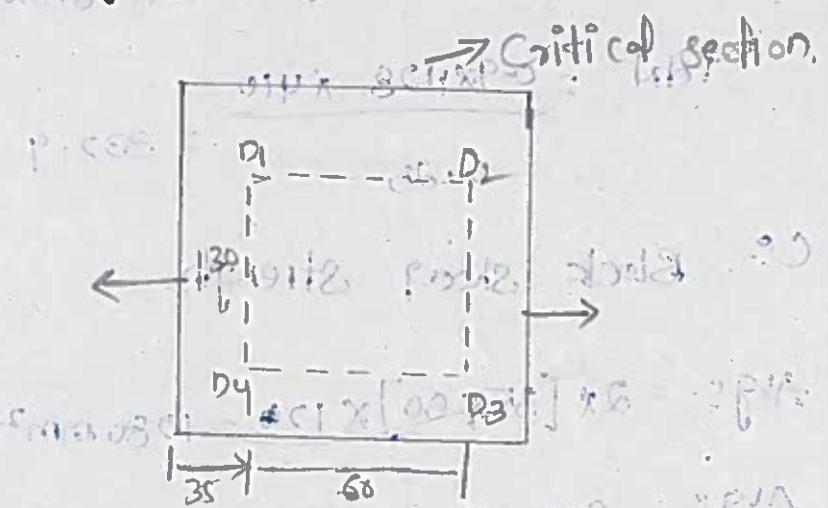
$$T_{dgl} = \frac{0.9 P_{nty}}{8m_s}$$

$$f_y = 410, 8m_s = 1.05$$

Critical section is having 2 holes provided.

A<sub>e</sub> - If the gauge is connected to 60mm leg

B<sub>e</sub> - If the gauge is connected to 6.460 - leg



Case (i) If the gauge is connected to 90mm leg

$$T_{dg} = \frac{A_g f_y}{8m_0}$$

$$T_{dg} = \frac{865 \times 250}{1.10} = 196.59 \text{ kN}$$

$$T_{dN} = \frac{0.9 A_{ns} f_y}{8m_1} + B \frac{A_{go} f_y}{8m_0}$$

$$A_{ns} = [90 - 6/2 - 18] \times 6 = 414 \text{ mm}^2$$

$$A_{go} = (60 - 6/2) \times 6 = 342 \text{ mm}^2$$

$$B = 1.4 - 0.0076 \times \frac{W}{t} \times \frac{f_y}{f_y} \times \frac{bs}{bc}$$

where  $w = 60 \text{ mm}$

$$t = 6 \text{ mm}$$

$$L_c = 4 \times 50 = 200$$

$$bs = 60 + 50 - 6 = 104$$

$$B = 1.4 - 0.0076 \times \frac{60}{6} \times \frac{25}{410} \times \frac{104}{200}$$

$$B = 1.159$$

$$\text{Dia of holes} = 16 \times 2 = 18 \text{ mm}$$

③

$$\therefore A_D = [130 - 2 \times 8] \times 12 = 1128 \text{ mm}^2$$

$$T_{DN} = \frac{0.9 \times 1128 \times 410}{1.25} = 332.9$$

C- Block shear strength

$$A_{qs} = 2 \times [35 + 60] \times 120 = 1280 \text{ mm}^2$$

$$A_{qf} = (35 + 60 - 1.5 \times 18) \times 12 \times 2 = 1632 \text{ mm}^2$$

$$A_{th} = (60 - 18) \times 12 = 504 \text{ mm}^2$$

$$T_{db} = \left[ \frac{A_{qf} f_y}{\sqrt{3} 8m_0} + \frac{0.9 A_{th} f_u}{8m_0} \right]$$

$$= \frac{1280 \times 250}{\sqrt{3} \times 1.10} + \frac{0.9 \times 504 \times 410}{1.25}$$

$$T_{db} = 447.95 \times 10^3 \text{ kN}$$

$$T_{db} = \left[ \frac{0.9 A_{qf} f_y}{\sqrt{3} 8m_1} + \frac{A_{qf} f_y}{8m_0} \right]$$

$$= \frac{0.9 \times 1632 \times 410}{\sqrt{3} (1.25)} + \frac{720 \times 250}{1.10}$$

$$T_{db} = 441.78 \text{ kN}$$

The lower strength of the plate  $441.78 \text{ kN}$

Problem  
2

A single unequal angle  $\text{S} 23 \times 90 \times 60 \times 6 \text{ mm}$  is connected to a  $10 \text{ mm}$  garter plate if the ends is connected to a  $16 \text{ mm}$  bolts to trans for tension as shown in fig , determine design strength of angles

$$T_{DN} = \frac{0.9 \times 414 \times 410}{1.15} + \frac{1.159 \times 342 \times 250}{1.10}$$

$$= 215 \text{ kN}$$

(ii) Block Shear

$$A_{Vg} = 280 \times 6 = 1800 \text{ mm}^2$$

$$A_{Th} = 40 \times 6 = 240 \text{ mm}^2$$

$$A_{Nh} = [40 - 0.5 \times 15] \times 6 = 186 \text{ mm}^2$$

$$A_{VN} = [280 - 4.9 \times 18] \times 6 = 894 \text{ mm}^2$$

$$T_{DN} = \left[ \frac{A_{Vg} f_y}{\sqrt{3} \gamma_m} + \frac{0.9 A_{Nh} f_y}{\gamma_m} \right]$$

$$= \frac{1880 \times 250}{\sqrt{3} \times 1.10} + \frac{0.9 \times 186 \times 410}{1.10}$$

$$= 235.9 \text{ kN}$$

$$T_{Db} = \left[ \frac{0.9 A_{VN} f_y}{\sqrt{3} \gamma_m} + \frac{A_{Vg} f_y}{\gamma_m} \right]$$

$$= \frac{0.9 \times 894 \times 410}{\sqrt{3} \times 1.10} + \frac{240 \times 250}{1.10}$$

$$= 206.91 \text{ kN}$$

the test of the strength of design

case (ii): If the gusset is connected to 60mm

$$T_{dg} = \frac{A_{Vg} f_y}{\gamma_m}$$

$$= \frac{565 \times 250}{1.10} = 196.59 \text{ kN}$$

$$A_{nc} = (60 - 6/2 - 18) 6 = 234 \text{ mm}^2$$

$$A_{go} = (90 - 6/2) 6 = 522 \text{ mm}^2$$

$$B = 1.4 - 0.076 \times 101 + \frac{f_y}{f_y} \times \frac{b_s}{b_c}$$

where  $w = 90 \text{ mm}$

$$t = 6 \text{ mm}$$

$$L_c = 200 \text{ mm}$$

$$b_s = 114$$

$$B = 1.4 - 0.076 \times \frac{90}{410} \times \frac{250}{410} \times \frac{114}{200}$$

$$B = 1.004$$

$$B = 1.004 \leq f_y$$

$$T_{dn} = \frac{0.9 \times 234 \times 410}{1.25} + \frac{10.04 \times 522 \times 25}{1.10}$$

$$= 320.06 \text{ kNm}$$

Block shear

$$A_{vg} = 230 \times 8 = 1840 \text{ mm}^2$$

$$A_{tg} = 30 \times 6 = 180 \text{ mm}^2$$

$$A_{vn} = (230 - 4.5 \times 18) 6 = 894 \text{ mm}^2$$

$$A_{vn} = (30 - 0.5 \times 1.8) 6 = 126 \text{ mm}^2$$

$$T_{db} = \left[ \frac{A_{vg} f_g}{\sqrt{3} P_{mo}} + \frac{0.9 b t n f_y}{8 \text{ mm}} \right]$$

$$T_{db} = \left[ \frac{A_{tg} f_y}{\sqrt{3} 8 \text{ mo}} + \frac{0.9 A_{th} f_y}{8 \text{ mo}} \right]$$

$$= \frac{1350 \times 250}{\sqrt{3} \times 1.10} + \frac{0.9 \times 12.6 \times 410}{1.10}$$

$$T_{db} = 202.83 \text{ kN}$$

$$T_{db} = \left[ \frac{0.9 A_{th} f_y}{\sqrt{3} 8 \text{ mo}} + \frac{A_{tg} f_y}{8 \text{ mo}} \right]$$

$$= \frac{0.9 \times 894 \times 410}{\sqrt{3} \times 1.10} + \frac{180 \times 250}{1.10}$$

$$T_{db} = 193.2 \text{ kN}$$

∴ The least value of the design strength is 193.2 kN

**Step 1:**

find the required gross area to a factored load considering the strength yielding

$$A_g = \frac{1.6 T_y}{f_y}$$

where  $T_y =$  factored tensile force

**Step 2:**

Select suitable section depending upon the type of structure and the location of the member such that gross area is 25 N/mm²

percent more than the  $\sigma_g$  calculated

Step 3:- determine the no. of bolts of the welding required and arranged

Step 4:- fluid the strength considering

- A. Strength in yielding Gross area
- B. Strength in rupture or critical structures
- C. Strength in block shear

The above least strength value have the design is safe for thin condition

$\sigma_g$  provided  $>$   $\sigma_g$  required

Step 4:- The strength is obtained should be more than factor tension

Step 5:- To recommended for IS : 800 - 2007

check strength ratio of tension member

as per the table - No 3 Pg : 20

Prob

① Design a single angle section for tension member of a roof truss to carry of factored tensile force of 225 kN member is subjected to the possible reversal of stress due to the action of the wind the effective length of the member is 3m. use 200mm bolts of great

bolt dia 22 mm pitch 100 mm off center line

$$A_g = \frac{1.1 f_y}{f_g} = \frac{1.1 \times 225 \times 10^3}{250}$$

Take ISH 100 x 75 x 8 mm

$$A_g = 1336 \text{ mm}^2$$

$$d = 20 \text{ mm} \quad d_o = 22 \text{ mm}$$

use gusset plate of thickness 10 mm greatest out bolt in single shear

$$V_{dsb} = V_{nsb} / \gamma_{mb}$$

$$\begin{aligned} V_{nsb} &= \frac{f_y}{\sqrt{3}} [D_n A_{nb} + D_s A_{sb}] / \gamma_{mb} \\ &= \frac{410}{\sqrt{3}} \left[ 0.7 \frac{\pi}{4} \times 20^2 \times 0.78 \right] / 1.25 \end{aligned}$$

$$V_{dsb} = 46404$$

Add plating edge distance  $e = 40 \text{ mm}$   
 $P = 60 \text{ mm}$

$$k_b \text{ is smaller or } \frac{e}{3d_o} \quad \frac{P}{3d_o} = 0.25$$

$$\frac{f_{ub}}{f_y} = 1.0$$

$$V_{dpb} = V_{nsb} / \gamma_{mb}$$

$$R_b = \frac{e}{3d_o} = \frac{40}{3(22)} = 0.606$$

$$V_{dpb} = \frac{1}{\gamma_{mb}} \times k_b \times d \times f_y$$

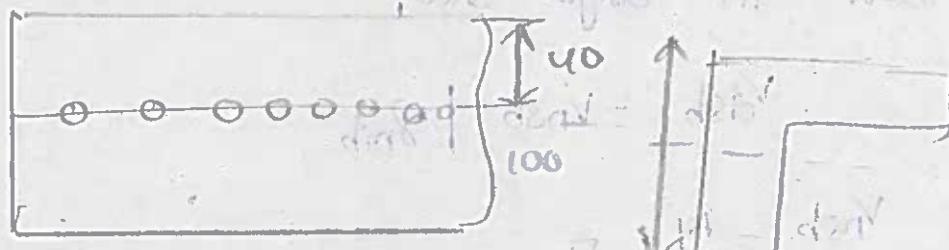
$$= \frac{1}{1.25} \times 0.606 \times 20 \times 8 \times 410 \times 2.5$$

$$= +9507.2 \text{ N}$$

$\therefore$  Bolt value = 46404 N

$$\text{No. of bolts required} = \frac{\text{factored tensile force}}{\text{Bolt value}}$$

$$= \frac{225 \times 10^3}{46404} = 5 \text{ Nos}$$



$$A_{bc} = 100 \times 40 = 4000 \text{ mm}^2$$

$$T_{dg} = \frac{A_g f_y}{\gamma_m} = \frac{19.98 \times 250}{1.1}$$

$$= 3036.80 > 22500 \text{ N}$$

$$T_{dN} = \frac{0.9 A_n f_y}{\gamma_m} = \frac{0.9 \times A_n \times 410}{1.1}$$

$$= A_n = [b - n d_o] t$$

$$A_n = [100 - 22 - 8/2] 8 = 592 \text{ mm}^2$$

$$A_{g0} = [75 - 8/2] 8 = 56.8 \text{ mm}^2$$

$$B = 1.40 - 0.76 \times \frac{75}{8} \times \frac{250}{410} \times \frac{107}{240}$$

$$= 1.20$$

$$L_c = 60 \times 4 = 240$$

$$T_{db} = 370 \text{ kN}$$

Least value for

$$T_{db} = 370 \text{ kN} > 225 \text{ kN}$$

7 Check for slenderness ratio

$$d = \frac{L}{\gamma_{min}}$$

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

$$\gamma_{min} = 21.8\text{mm}$$

$$d = \frac{300}{21.8} = 137.6 < 180$$

Tension member splice:-

If a single plate of required length is not available the tension members are spliced to transfer the required tension from one piece to another

\* The strength of splice plate and both on weld are connecting them should have strength at least equal to designed load

Packing plates:-

To design shear capacity of bolts carrying shear through a packing plate in excess of 6mm shall be decreased by a factor  $B_{pk}$  given

$$B_{pk} = (1 - 0.0125 + t_{pk})$$

$t_{pk}$  = thickness of packing plate

$\therefore t_{pk}$  = thickness of packing plate

Design a splice to connect a 300 x 20mm plate with a 300 x 10mm plate the design load 500kN use 20mm block bolt faber coated in the shop.

let double corner butt joint width 6mm

Cover plates used

Strength of bolts

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

$$d_0 = 22\text{ mm}$$

$$B_{pk} = 1 - 0.0125x + p_k = 1 - 0.0125 \times 10$$

[∴ min size of plate = 10mm]

$$B_{pk} = 0.875$$

Strength in double shear =  $B_{pk} [\frac{\pi}{4} d^2 + 0.78 \times T_k]$

$$d^2 J \frac{f_y}{\sqrt{3} \delta_m s}$$

$$= 0.875 \left[ \frac{\pi}{4} \times 20^2 + 0.78 \times \frac{\pi}{4} d_0^2 J \times \frac{410}{\sqrt{3} \times 1.25} \right]$$

Strength in boring:

let edge distance ( $e$ ) = 40mm and

pitch ( $P$ ) = 60mm be used

$$\Delta b \text{ is smaller of } e/3d_0 \cdot \frac{P}{3d_0} = 0.25 \frac{f_{eb}}{f_y}$$

∴ Strength in boring against 10mm plate

$$= 2.5 \Delta b = f_y \times \frac{1}{\gamma_m b}$$

$$= 2.5 \times 0.606 \times 20 \times 10 \times 410 \times \frac{1}{1.25}$$

$$= 99384 \text{ N/mm}$$

Bolt value = strength in bearing plate and  
strength in shear

$$\text{Bolt value} = 956.59 \cdot 4 \text{N}$$

$$\begin{aligned}\text{No. of bolts} &= \frac{\text{Load}}{\text{bolt value}} = \frac{500 \times 10^3}{92659} \\ &= 5.39 \approx \text{bolts}\end{aligned}$$

Check for strength of plates:

(i) yielding

$$T_{dg} = \frac{A_g f_y}{\gamma_m} = \frac{300 \times 10 \times 250}{1.1}$$

$$T_{dg} = 681.81 \text{kN}$$

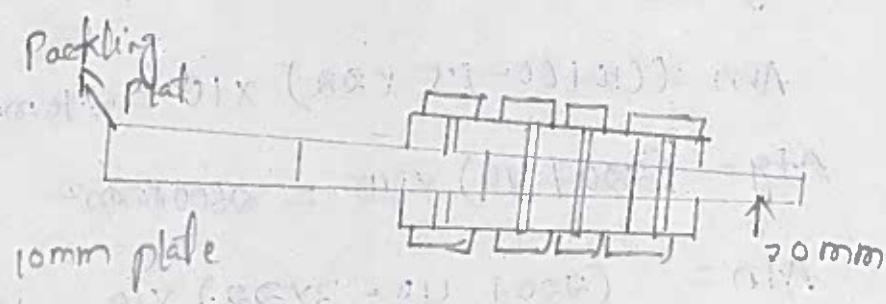
(ii) Check for strength against ruptures

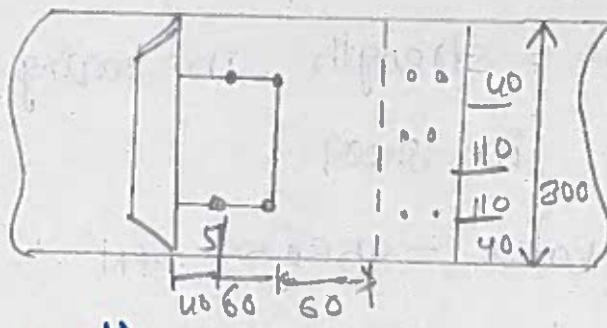
$$T_{dN} = \frac{0.9 A_n f_y}{\gamma_m}$$

$$A_n = \frac{(300 - 3 \times 200) \times 10}{1.25} = 2340 \text{ mm}^2$$

$$T_{dN} = 690.48 \text{kN} > 500 \text{kN}$$

(iii) Block shear





(i) Along the 1-1 - 2-3 - 4-5:

$$A_{rg} = (40+60) \times 2 \times 10 = 2000 \text{ mm}^2$$

$$A_{fg} = 220 \times 10 = 2200 \text{ mm}^2$$

$$A_{rn} = 2(40+60 - 1.5 \times 22) \times 10 = 1340 \text{ mm}^2$$

$$A_{in} = (220 - 2 \times 22) \times 10 = 1760 \text{ mm}^2$$

$$T_{db} = \left( \frac{0.9 A_{rn} f_y}{\sqrt{3} \delta_{mi}} + \frac{A_{fg} f_y}{\delta_{mo}} \right)$$

$$= \frac{0.9 \times 1340 \times 410}{\sqrt{3} \times \delta_{mi}} + \frac{2200 \times 1250}{1.1} = 550 \text{ kN}$$

$$T_{db} = \frac{A_{rg} f_y}{\sqrt{3} \delta_{mo}} + \frac{0.9 A_{in} f_y}{\delta_{mi}}$$

$$= \frac{2200 \times 1250}{\sqrt{3} \times 1.1} = \frac{0.9 \times 1760 \times 410}{1.25}$$

$$= 781 > 500 \text{ kN}$$

(ii) Along the 1-1 - 2-3 - 6-1:

$$A_{rg} = (40+60) \times 10 = 1000 \text{ mm}^2$$

$$A_{rn} = (10+60 - 1.5 \times 22) \times 10 = 670 \text{ mm}^2$$

$$A_{fg} = (220 + 40) \times 10 = 2600 \text{ mm}^2$$

$$A_{in} = (220 + 40 - 3 \times 22) \times 10 = 1940 \text{ mm}^2$$

$$T_{db} = \frac{A_{rg} f_y}{\sqrt{3} \sqrt{\delta_{mo}}} + \frac{0.9 A_{in} f_y}{\delta_{mi}}$$

$$\frac{0.9 \times 670 \times 40}{\sqrt{3} \times 1.25} + \frac{2600 + 250}{1.01}$$

$$= 405.09 > 500 \text{ kN}$$

Provide an extra bolt in packing plate or packing material

are in compression vertical comp in buildings

are called as columns posts and stanchions

comp numbers in trusses are called as short

\* The job or crans which carries compression is called 'Boom'

\* compression Number in bridges called pier

\* Imperfection of fabrication resulting into accidental eccentricity largely depends upon cls of the comp members

\* IS 800 - 2007 divided various cls into 4 buckling classes a, b, c & d shown in table -

design compressive strength  $P_d$  of a members is given by

$$P_c \leq P_d$$

where

$$P_d = A_e f_{cd}$$

$A_e$  = eff sectional area

$f_{cd}$  = designed compressive stress

\* The design compressive stress fed of axially loaded comp members shall be calculated by following egn

$$f_{cd} = \frac{f_y l}{\gamma_m}$$

$$\phi + (\phi - d^2)^{0.5} = \lambda f_y l / \gamma_m \leq f_y l / \gamma_m$$

$$\text{where } \phi = 0.5 (C_L + \alpha (d^{-0.2}) + d^2)$$

$d$  = non - down load or slenderness ratio.

$$= \sqrt{f_y l / \gamma_m} = \sqrt{f_y \left(\frac{k_1}{r}\right)^2 / \pi^2 E}$$

$$f_{cc} = \text{Euler buckling stress } \frac{\pi^2 E}{(k_1/r)^2}$$

Where

$k_1/r$  = eff slenderness ratio  $k_1$  to appropriate radius of gyration

$\alpha$  = imperfection factor (table-7)

$d$  = stress reduction factor (table-8)

$$= \frac{1}{\phi + (\phi - d^2)^{0.5}}$$

$\gamma_m$  = partial safety factor

## Table for design stress

for the table are given in IS-800 - 2007 table - 9 (Pg - 48, 49) depends upon backling class

- Determine the design axial load capacity of column ISMB 400 at 616 N/mm if the length of column is 8m both ends

Sol:

$$f_y = 410 \text{ N/mm}^2$$

Both ends are fixed

$$L = 0.65L = 0.65 \times 8 = 5.2 \text{ mm}$$

for ISMB 400 @ 616 N/mm

$$h = 400 \quad b_f = 410 \quad t_f = 16$$

$$A_e = A = 7846 \text{ mm}^2$$

$$\frac{h}{b_f} = \frac{400}{140} = 2.8 > 1.2 \rightarrow \text{class a } t_f < 40 \text{ mm}$$

- class b

$$\sigma_{min} = \gamma_{yy} = 28.2 \text{ mm}$$

$$180 \quad 49.5$$

$$184.3 \quad ?$$

$$190 \quad 44.7$$

$$f_{cd} = 47.43 \text{ N/mm}^2 \quad - \text{class a}$$

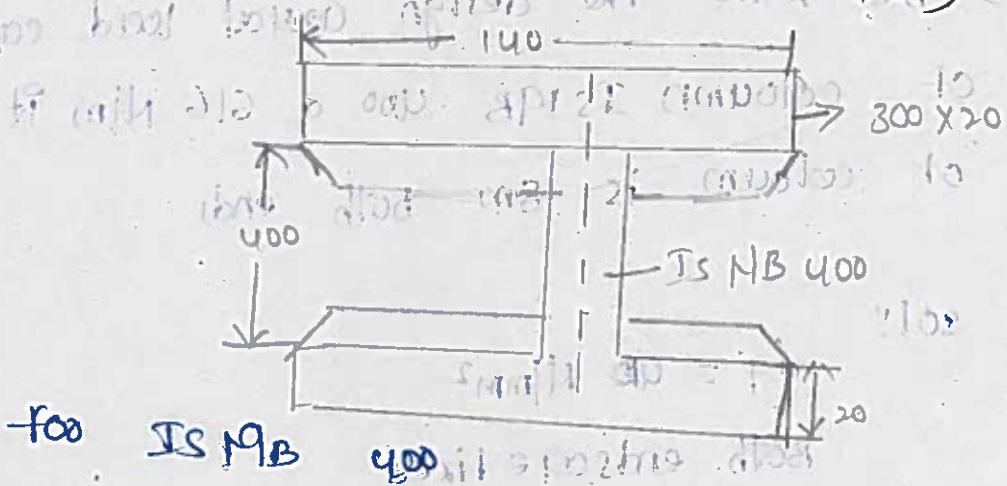
$$f_{cd} = 44.6 \text{ N/mm}^2 \quad - \text{class b}$$

$$P_d = f_{le} f_{cd} = 784.6 \times 44.6$$

$$P_d = 349.93 \text{ kN}$$

$$P = \frac{349.93}{1.5} = 233.28 \text{ kN} \text{ (unit-2; page no-19 / 39)}$$

- ③ Determine the load carrying capacity of column shown in fig if its actual length is 4.5m its end fixed and other end hinged. The grade of steel Fe 415 ( $f_e = 250$ )



$$l = 0.81 \times 4.5 = 3.600 \text{ mm}$$

Buckling class C

$$I_{zz} = I_{xx} + gah^2$$

$$\begin{aligned} I_{zz} &= 20458.4 \times 10^4 \times 2 \times 300 \times 20 \times (200+10)^2 \\ &= 773 \times 8 \times 4000 \text{ mm}^4 \end{aligned}$$

$$\begin{aligned} I_{yy} &= 612.1 \times 10^4 + \frac{1}{12} \times 200 \times 300^3 \times 2 \\ &= 9822 \times 1000 \text{ mm}^4 \end{aligned}$$

$$I_{zz} > I_{yy}$$

$$A = 7848 + 2 \times 300 \times 20$$

$$= 19846 \text{ mm}^2$$

$$r_{yy} = \sqrt{\frac{I_{yy}}{A}} = \sqrt{\frac{9822000}{19846}}$$

$$= 69.63 \text{ mm}$$

$$\text{Slender ness ratio } \lambda = \frac{\kappa L}{\gamma_{\min}} = \frac{3600}{69.63} \\ = 51.70$$

Buckling Class C (Pg-42)

$$\frac{\kappa L}{\gamma} = 51.70 \quad f_y = 250 \text{ N/mm}^2$$

$$50 - 183$$

$$51.7 - 2$$

$$60 - 168$$

$$f_{cd} = 180.4 \text{ N/mm}^2$$

$$f_{pd} = 19846 \times 18.4$$

$$= 3580218.4$$

$$= 3580.21 \text{ kN}$$

$$P = \frac{3580.21}{1.5} = 2386.8 \text{ kN}$$

Design of comp member

→ Design stress in comp is to be assumed for rolled steel beam sections

$\lambda = \text{Slender ness ratio: various } 70-90 \text{ Hence design stress may be assumed as } 135 \text{ N/mm}^2$

for angle strands these slender ness ratio value may be assumed as  $90 \text{ N/mm}^2$

for comp members carrying large load

The slenderness ratio comparatively is small for each members design stress may be assumed as  $800 \text{ N/mm}^2$

- eff sectional area  $A = \frac{P_d}{f_{cd}}$

Select a section to give eff area required and calculate  $\tau_{min}$

\* Now the end condition and decreasing the type connection determines the effective length

\* find the slenderness ratio & hence  $f_{cd}$  and  $P_d$

\* Perve the section is calculated  $f_d$  differs considerable design load

i) Design a single angle strut connection to quest plate to carry 18 kN factored load the length of strait below center to center connection is 3m

Angle strait  $f_{cd} = 90 \text{ N/mm}^2$

Calculate eff area

$$A = \frac{P}{f_{cd}} = \frac{180 \times 10^3}{90} = 2000 \text{ mm}^2$$

choose the selection Isha 150x 150x 10 mm

$$\delta_W = 29.4 \text{ mm}$$

Calculate width length

(12) Assuming both ends are pinned

$$d = \frac{4L}{\delta_{min}} = \frac{3000}{19.4} = 154.63$$

$$d = 154.63 \quad f_y = 250 \text{ N/mm}^2$$

choose beekling class C

150

59.7

160

53.3

154.6

,

$$f_{cd} = 24.91 \text{ N/mm}^2$$

$$P_d = A \times f_{cd} = 2502 \times 24.91$$

$$P_d = 120.1 > 180$$

Hence section is safe

Check is ok

2)

A column  $4m$  long has to support a factored load of  $600kN$  the column is effective that at both ends and restrained in directions at one end of the end design the column beam section & plate.

$$P_u = 600kN$$

$$d = 4m$$

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

$$\text{Area required} = \frac{600 \times 10^3}{200}$$

using IEB 450 @  $907 \text{ N/mm}$

Area provided by plates =  $3000 - 1178.9$

$\approx 18211 \text{ mm}^2$  and follow 2A  
 Select the 200mm plate breadth required  
 obtained from  $ab \times 20 \approx 18211$

$$ab = \frac{18211}{20} = 910.55$$

$$b = 455.27 \approx 500 \text{ mm}$$

Provide  $= 20 \times 50 \text{ mm plate}$

total area provided

$$A_e = 11789 + 2 \times 500 \times 200$$

$$= 31489 \text{ mm}^2$$

$$I_{zz} = I_{xx} + \alpha h^2$$

$$I_{zz} = 40349 \cdot 9 \times 10^9 + 2 \times 500 \times 200 (205+10)^2$$

$$= 1507 \cdot 94 \times 10^6 \text{ mm}^4$$

$$I_{yy} = I_{yy} + \alpha \times \frac{ab^3}{12}$$

$$= 3045 \times 10^9 + 2 \times \frac{20 \times 500^3}{12} = 4471.166 \times 10^6$$

$$\sigma_{\min} = \frac{r_{yy}}{A} = \sqrt{\frac{I_{yy}}{A}}$$

$$= \sqrt{\frac{4471.166 \times 10^6}{31489}} = 114 \times 10^3$$

$$\sigma_{\min} = 118.54 \text{ mm}$$

$$\text{eff length} = \frac{d}{2} = 0.8L = 0.8 \times 4$$

Buillap the members to choose the section buckling class 2.

$$\frac{d}{\sigma_{\min}} = \frac{3200}{118.54} = 26.98$$

20 - 224

30 - 211

26.98 - ?

$$f_{cd} = 220 \text{ N/mm}^2$$

$$P_d = f_{cd} \times A = 220 \times 314.89$$

$$= 6995.8 > 6000 \text{ kN}$$

It is safe

Design of longer

Rolled steel plate and angles are used for locating on can be used

i) single locating system

ii) double locating system

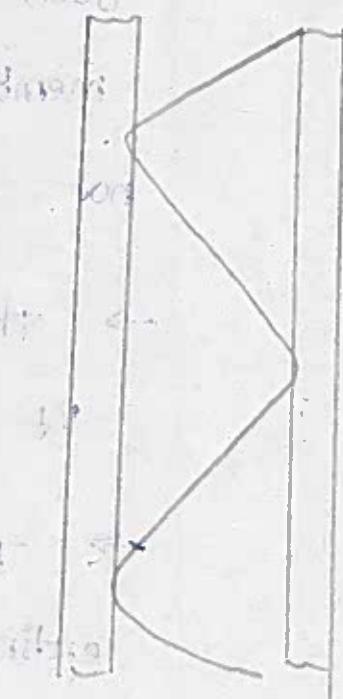
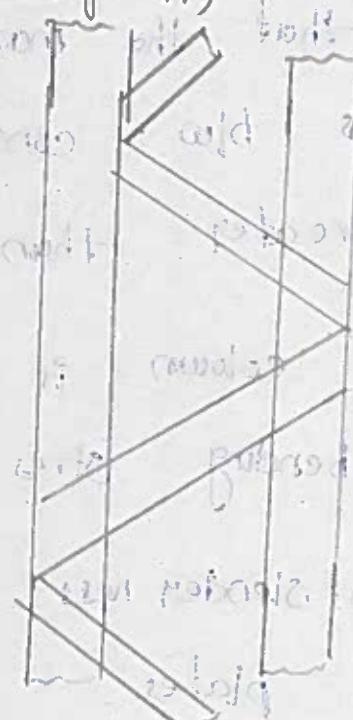
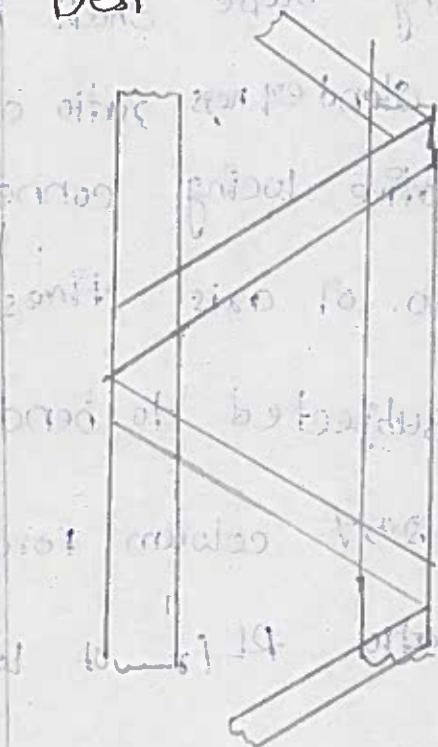
\* other object or providing later system

is to keep the main members of column

away from principle axis

Desi

(Pg - 49)



## Design procedure of loceng:

- \* The loceng system shall be uniform throughout
- \* In single loceng should be saddle of the other
- \* IP bottom rivited connection
- \* In Min width of loceng bars shall be 3 times in normal dia of loceng bars shall not <  $\frac{1}{16}$  th of eff length for single loceng &  $\frac{1}{18}$  th of eff length for double or system
- Loceng bars weather in double or single system shall be inclind at angle  $\leq 40^\circ$  to axis of built up members.
- distance b/w 2 main members should be kept so as the  $r_{eff} \geq r_{322}$ .
- $r_{322}$  = radius of gyration of stronger axis
- Max spacing of loceng bars shall be such that the max slenderness ratio of main members b/w consecutive loceng connection is not greater than 50 or axis times more
- The column is subjected to bending also + bending stress  $\pm 2.5\%$  column force
- The slenderness ratio  $PL/r$  of loceng with entire plates.

Design a facing column with two channels back to back of length 10 mm to carry a axial factored load of 1400kN. The column may be assumed restrained position but not in direction at both ends Chinged ends.

Given data -

$$P_d = 1400 \text{ kN}$$

$$l = 10 \text{ cm}$$

$$dL = l = 12 = 10 \text{ cm}$$

$$f_{ck} = 135 \text{ N/mm}^2 \text{ (assumed)}$$

Area required

$$\frac{P_d}{f_{cd}} = \frac{1400 \times 10^3}{135}$$

$$= 10870 \text{ mm}^2$$

Try 2ISPC 350 @ 41.3 N/m

$$\text{Area provided} = 2 \times 5366 = 10732 \text{ mm}^2$$

$$\text{Distance between } \delta_{22} = 136.6 \text{ mm}$$

Distance will be maintained  $\delta_{22} > \delta_{22}$

$$\frac{\delta L}{\delta} = \frac{1000}{136.6} = 73.2$$

Hence faced column =  $1.05 \times 43.2$

$$\frac{\delta L}{\delta_{22}} = 78.66$$

$$f_y = 250 \text{ (Book Long)}$$

$$\delta L_{12}$$

Fed

$$40$$

$$152$$

$$80$$

$$136$$

$$76.86$$

$$9$$

$$f_{cd} = 146 \text{ N/mm}^2$$

1 load carrying capacity  $\geq 146 \times 10732$

Spacing b/w channels

$$I_{xx} = 2 \times 1008 \times 10^4 = 200,16 \times 10^4 \text{ mm}^4$$

$$I_{yy} = 2[I_{yy} \alpha + h^2]$$

$$20016 \times 10^4 = 2[430 \cdot 6 \times 10^4 + 5366 (d_1 + 242)]$$

$$\frac{C_d}{2} + 24.4)^2 = \frac{10008 \times 10^4 - 430 \cdot 8 \times 10^4}{5366}$$

$$\left(\frac{C_d}{2} + 24.4\right)^2 = 17848 \cdot 3$$

$$= \sqrt{17848 \cdot 3}$$

$$\frac{d}{2} = 133.59 - 24.4 = 109.19$$

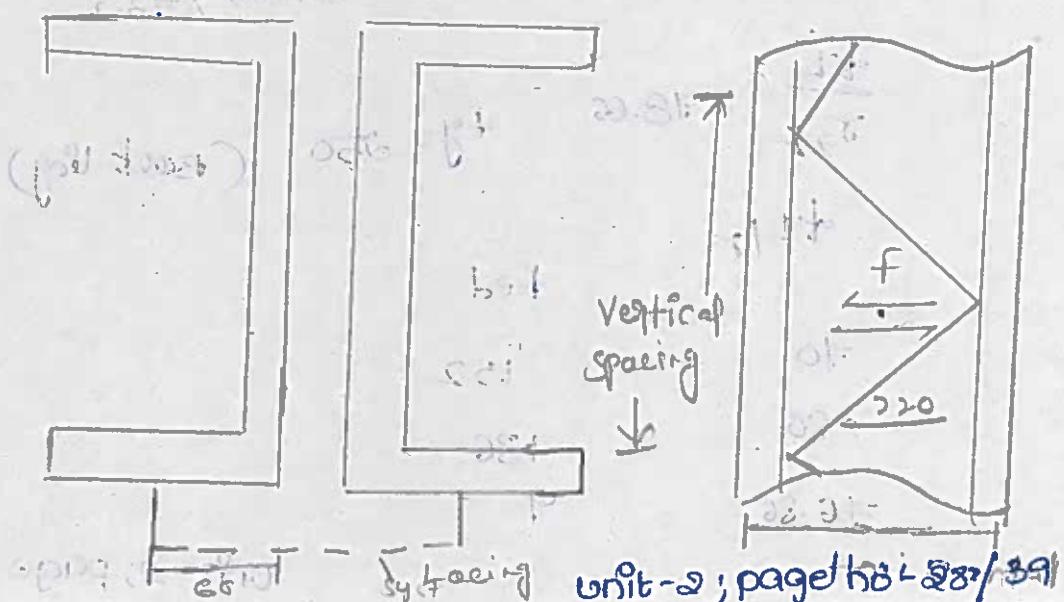
$$d = 109.19 \times 2 \approx 220 \text{ mm}$$

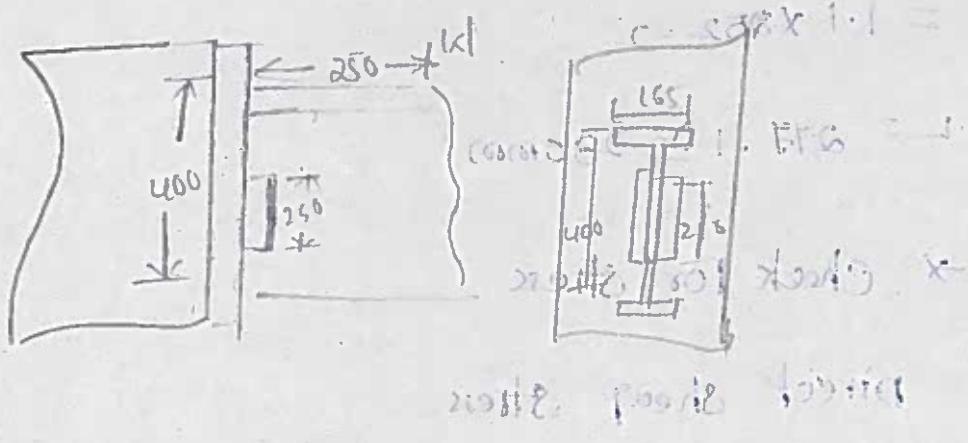
Let the long be provided at  $45^\circ$

Horizontal spacing of facing =  $220 + 60 + 60$

$$= 340 \text{ mm } (g = 60 \text{ mm})$$

Vertical spacing =  $340 \tan 45^\circ \times 2 = 680 \text{ mm}$





Assuming Normal fillet welds:

$$\text{Throat thickness of flange welds} = 0.7 \times 10 = 7 \text{ mm}$$

$$\text{Throat thickness of web welds} = 0.7 \times 0.5 = 3.5 \text{ mm}$$

$\therefore$  Total throat area of weld

$$\text{Resistance of weld found} = \frac{f_u}{\sqrt{3}} \times 1$$

$$f_{wd} = \frac{410}{\sqrt{3}} \times \frac{1}{2.5} = 189.57 \text{ N/mm}^2$$

Depth of the weld general to prevent bending along

$$h' = \sqrt{\frac{G \times N}{\sigma_{at-fwd}}}$$

$$h' = \sqrt{\frac{G \times W \times 250}{2 \times 5.6 \times 189.57}}$$

$$h' = \frac{h}{1.1} = \frac{400}{1.1} = 363.63 \text{ mm}$$

$$363.63 = \sqrt{\frac{G \times W \times 250}{2 \times 5.6 \times 189.57}}$$

$$432.37 = \sqrt{W}$$

$$W = 186.94 \text{ kN}$$

$$= 1.1 \times 252.2$$

$$L = 277.1 \approx 280 \text{ mm}$$

→ Check for stress

Direct shear stress

$$\tau = \frac{P}{2ht} = \frac{150 \times 10^3}{2 \times 280 \times 3.6} = 6.94 \text{ N/mm}^2$$

$$\tau = 6.94 \text{ N/mm}^2$$

Bending stress

$$\sigma = \frac{M}{I} = \frac{6m}{2 + h^2} = \frac{6 \times 150 \times 10^3}{2 + 150^2} = 153.4 \text{ N/mm}^2$$

$$\sigma = 153.4 \text{ N/mm}^2$$

equivalent stress

$$\sigma_e = \sqrt{\sigma^2 + 3\tau^2} = \sqrt{153.4^2 + 3 \times 6.94^2}$$

$$\sigma_e = 174.6 < 189.39$$

Hence check is ok

- (5) An I-section bracket is connected to a column by drill as shown fig determine the load which can be safely carried. This size of the package weld is 5mm while the size of the package weld is 10mm. Assume  $f_e$  fillet.  $k_{f,ld}$

(16) Least value of  $\gamma_{yy}$  = 28.3

$\frac{dL}{s}$  of channel b/w loosing

$$\Rightarrow \frac{\text{Vertical spacing}}{\gamma_{yy}} = \frac{680}{28.3}$$

Transverse shear to be exerted loosing

System = 215% of design load

$$= \frac{215}{100} \times 1400 \times 10^3 = 3500 \text{ N}$$

Shear to be resisted by each loosing =  $\frac{3500}{2}$

$$\text{length spacing} = \frac{\text{horizontal spacing}}{\cos 45}$$

$$= 480 \cdot 9 \text{ mm}$$

Min thickness of loosing if 20mm bolts

$$= 3 \times 20 = 60 \text{ mm}$$

$$\text{sectional area} = 60 \times 4 = 240 \text{ mm}^2$$

$$\gamma_{min} = \sqrt{\frac{bd^3/12}{bd}} = \sqrt{\frac{60 \times 1.43}{12 / 160 \times 4}} = 4.04 \text{ mm}$$

$$\frac{dL}{\gamma_{min}} = \frac{480.9}{4.04} = 119.8 < 145$$

It's check is ok

a) Strength of slope bolts

$$\text{In single shear} = 0.78 \times W_{1/4} (20)^2 \times \frac{400}{\sqrt{3} \times 1.25}$$

$$= 45272 \text{ N}$$

Edge thickness  $e = \frac{60}{2} = 30\text{mm}$

$t_b$  is small or  $\frac{e}{3d_0} = \frac{30}{300} = 0.1 = 25 \cdot \frac{f_e}{f_y} \cdot 1.10$

$$\frac{30}{3 \times 22} = 0.45 \quad \frac{60}{3 \times 22} = 0.25 = 0.65$$

$$\frac{440}{410} = 1.05$$

b) Strength of bearing  $= \frac{25 \cdot k \cdot b \cdot t}{\delta_m} \cdot f_y$

$$= 2.5 \times 0.45 \times 20 \times 10 \times 410$$

$$= 78300 \text{ N}$$

Min bolt value = 4527 N

$$\text{No. of bolt required} = \frac{17500}{45272} = 0.386$$

Provide one bolt

$$\frac{4kL}{\delta} = 119.82 \quad f_y = 250$$

Buckling Class

110

$\sim 94.6$

120

$\sim 83.7$

119.82

$\sim 95.0$

$$f_{ed} = 83.89 \text{ N/mm}^2$$

load carrying capacity of comp

## Column Base:-

Column base transmit the column load to the corner or side of masonry foundation blocks. The column spread the load on wider area so that the intensity of bearing pressure on the foundation block is within the bearing. There are two types of column bases commonly used in practice.

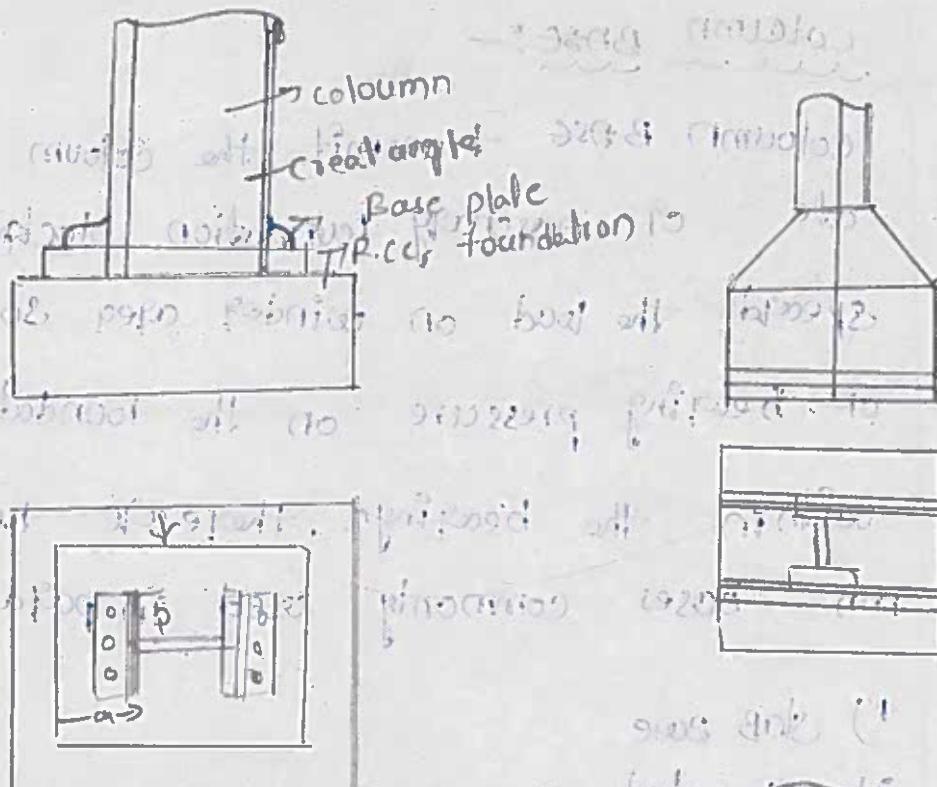
- 1) Slab Base
- 2) Gusseted Base

### Slab Base:-

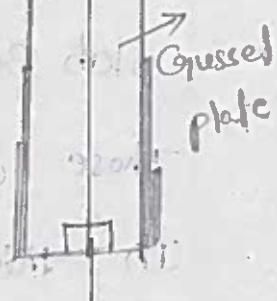
These are used in column carrying small loads. In this type the column is directly connected to base plate through cleat angles as shown. The load is transferred to the base plate through bearings.

### Gusseted Base

for column carrying heavy loads gusseted bases are used. In gusseted base the column is connected to gusseted base shown a typical gusseted base connection.



Slab base



### Design of Slab Base:-

The design of slab consist in finding the size and thickness of slab. In the procedure given below. It is assumed the the pressure is distributed uniformly under that slab base.

- Prob:- Design a slab for a column ISHB 300 @ 5 ft  
 ① 11mm carrying an axial factored load of 1000  
 & N.M. Concrete is used for the foundation  
 Provide welded connection between column and  
 base plate

Bearings Strength of concrete =  $0.45 f_{ck}$

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

factored load  $P_u = 1000 \text{ kN}$

$\therefore$  area of base plate required =  $\frac{1000 \times 10^3}{9} = 1111 \text{ mm}^2$

$$= 360 \times 310 \text{ mm}^2$$

Provide 360x310 size plate

$$\text{area provided} = 360 \times 310 = 111600 \text{ mm}^2$$

$$\text{Pressure} = \frac{1000 \times 10^3}{111600} = 8.96 \text{ N/mm}^2$$

Projected area

$$a = \frac{360 - 300}{2} = 30 \text{ mm}$$

$$b = \frac{310 - 250}{2} = 30 \text{ mm}$$

$$t_s = \left[ \frac{2.5 \times 8.96 (30^2 - 0.3 \times 30^2) \times 1.1}{250} \right]^{0.5}$$
$$= 7.88 \text{ mm}$$

thickness of flanged of ISHB 300 @ 577 N/mm

Steel stiffener thickness of plate 0.6 m

to provide 12 mm thick plate

connecting 360 x 310 x 12 mm plate to concrete

foundation

use 4 bolts of 20 mm diameter 300 mm long

welds:- properly machined column is to be connected to plate using fillet weld

$$= 2(250 + 250 - 7.6 + 300 - 2 \times 0.6) = 1542.4 \text{ mm}$$

$$\text{Strength of weld} = \frac{0.10}{\sqrt{3}} \times \frac{1}{1.85} = 189.87 \text{ N/mm}^2$$

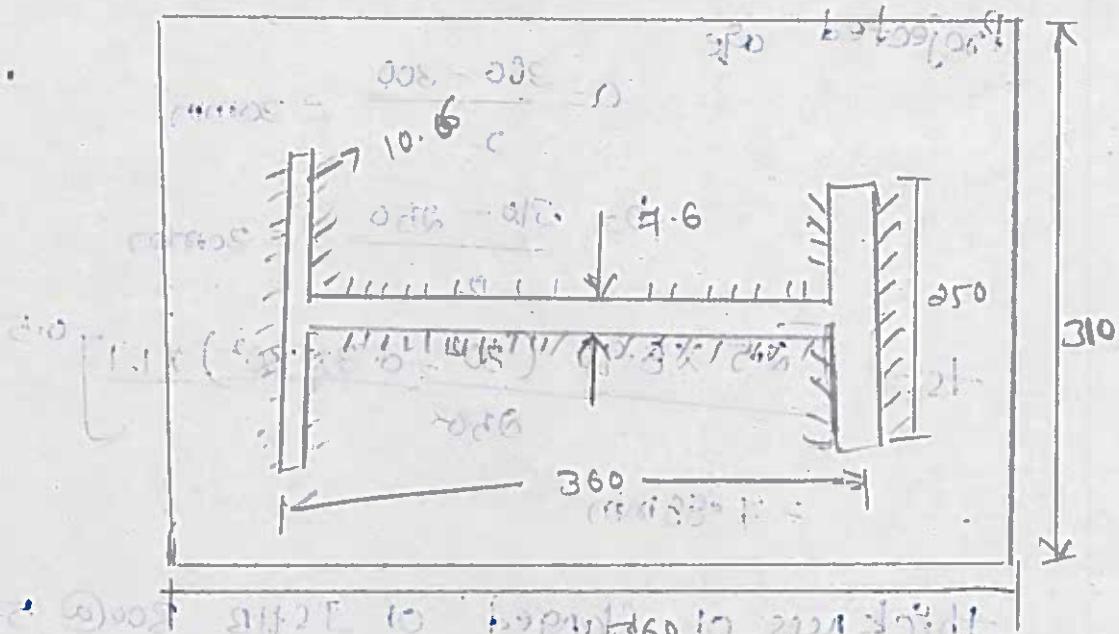
$$\text{Area of weld} = 0.75 \text{ Le}$$

$\therefore$  The design condition is  $0.75 \text{ Le} \times 189.34$

$$= 1000 \times 10^3$$

$$SLe = 1548.8$$

using 6mm weld  $Le = 125 \text{ mm}$



after deducting for end return of the weld  
size or twice size of the weld at each  
end available effective length

$$= 1542.4 - 2 \times 6 \times \text{No. of return}$$

$$= 1542.4 - 2 \times 6 \times 12$$

$$= 1398.4 > 125 \text{ mm}$$

Hence 6mm weld is adequate

DESIGN OF GUSSETED BASE

IS 800 - 2007 specifies that the gusset plate angles cleats stiffener and fastenings etc. in combination with the bearings area - shall be sufficient to take the loads bending moment and reactions to the base plate with the bearings without exceeding specified strength.

for following design procedure be followed

- 1) area of base plate = factored load
- 2) Assume various number of gusset base
  - (a) Thickness of gusset plate is assumed as 16mm
  - (b) Sizing of the gusset angles is assumed such that it's vertical leg can accomm - odote two bolts
  - (c) thickness of angles is kept approximately equal to the thickness of length = area of plate  
width
- (d) when the end of the column is machine.

Design a gusseted base for a column ISHB 316@ 410 N/mm two plates 450 mm x 50 mm carrying a factored load of 350 kN.

$$f_{ck} = 20 \text{ N/mm}^2$$

$$A_z = \frac{P_y}{0.45 f_{ck}} = \frac{3600 \times 10^3}{0.45 \times 20} = 40000 \text{ mm}^2$$

Selecting IS A 150 L5 - 15 mm angle and 16 mm thick nos. gusset plate

$$\text{Minimum width required} = 250 + 2 \times 80 + 2 \times 16 \\ + 2 \times 15 \\ = 152 \text{ mm}$$

use 300 mm wide plate

$$\therefore \text{length of plate} = \frac{40000}{300} = 133.3 \text{ mm}$$

Provide 300 x 600 mm plate

Pressure under base plate

$$= \frac{3600 \times 10^3}{300 \times 600} = 8.57 \text{ N/mm}^2$$

$$d = 300 - \frac{(250 + 80 \times 2 + 16 \times 2 + 2 \times 15)}{2}$$

$$= 124 \text{ mm}$$

B.M at section X-X per mm width

$$= 8.57 \times \frac{124^2}{2} = 65886 \text{ N-mm}$$

At section Y-Y bending moment

$$M_{yy} = 8.57 \times \frac{250^2}{2} - \frac{300 \times 8.57 \times}{2}$$

$$\left[ \frac{350}{2} + 80 + \frac{160 + 15}{2} \right] = 106482 \text{ N-mm}$$

design moment = 106482 N-mm

$$\text{Bending strength} = \frac{f_y l}{8 m_0} = \frac{250}{1.1} = 227.2 \text{ N/mm}^2$$

Equating Bending moment of resistance

$$1.2 \times \frac{1}{6} \times 1 \times t^2 \times 227.27 = 106482$$

$$\therefore t = 48.4 \text{ mm}$$

$\therefore$  Use 56 mm plate of size 300 x 600 mm

Assuming ends column are fixed

$$\text{Design load} = 0.5 \times 3600 = 1800 \text{ kN}$$

$$\text{Load on each splice} = \frac{1800}{2} = 900 \text{ kN}$$

Bearing 24 mm

Strength of bolt in single shear

$$= 0.78 \times \frac{\pi}{4} \times 24^2 \times \frac{400}{\sqrt{3}} \times \frac{1}{1.25}$$

$$= 65192 \text{ N}$$

Strength in bearing is higher

$$\therefore \text{Bolt value} = 65192 \text{ kN}$$

$$\therefore \text{No. of bolts required} = \frac{900 \times 10^3}{65192} = 13.8.$$

