Civinnovate

Discover, Learn, and Innovate in Civil Engineering

chapter : 4 Connections in Steel Sections

→ Joining the various members of a structure so that they act as an integral unit.

→ Different techniques used to join steel members are
  → Welding
  → Bolting
  → Riveting

Welding

1) Electric Arc Welding → Structural welding
2) Oxy - acetylene gas welding

~~Types of welds~~ :-

Advantages
→ No holes on the main member are needed.
→ Welding connection are better for fatigue, impact & vibration
→ It saves the weight of structure.
→ Less noise pollution compared to rivetting.

Disadvantages
→ Electricity is required.
→ Failure will be britte
→ Internal stresses are developed due to heating.

Types of weld connection:-

(1) Butt weld (Groove weld).

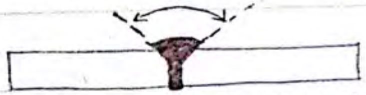
Symbol

(a) Square

↳ plates upto 8 mm thick only.

(b) Single V- butt weld.

45° to 60°



→| |← 1 to 3mm Gap → facilitate contraction of the metal after cooling of weld metal

V

(c) Double V-butt weld

X

When two metal parts are to be connected directly without any lap, the type of weld used is known as butt weld.

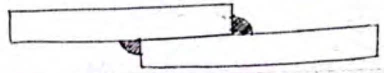
Strength of butt weld ≥ Strength of parent metal.

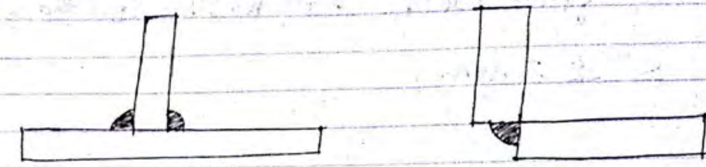
⇒ Better in highly stressed structures.

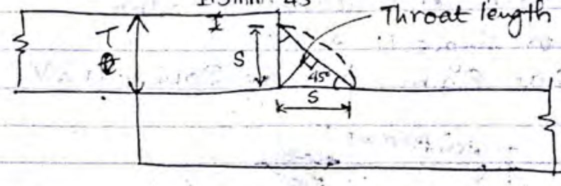
(2) Fillet weld :- If the metal parts are to be connected with a lap or in the form of T or L,

△

Fillet weld is more common than Butt weld. where stiffness rather than strength controls design and fatigue or brittle fracture is not a problem. (For same amt. of weld material, not so strong and cause greater concentration of stress. Triangular c/s.







Throat length $t = \frac{S}{\sqrt{2}} = 0.707\,S$.

$S \leq T - 1.5\,mm$

Idealization of fillet weld :- Right-angled △ with equal sides 'S'

S is size of the fillet weld.

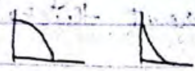
Welds - Symbols.

Square butt weld ⇒

Single V-butt — V

Double V - butt — X

Fillet weld — △

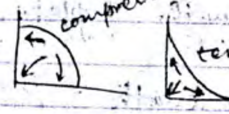


Convex

Concave

↓

smaller throat more favoured cos smoother path for flow of stress. But single pass fillet have great tendency to crack upon cooling.



so cracks on cooling

→ two or more passes/ first slight convex.

(Q) Two plate of 18mm & 15mm thickness are joined by groove or butt weld. The joint is subjected to working load of 300 kN. Calculate the length of the weld considering single groove weld & double groove weld. Use Fe 410 grade.

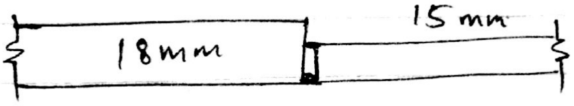
Table 1 (Page 14)
$f_y = 250$ MPa

Soln:-

$P_w = 300$ KN

$P_d = 1.5 \times P_w = 450$ KN (factored load)

Now,



Assume shop welding $\gamma_{mw} = 1.25$
(Table 5 Page 30)

① Single Butt

Throat Thickness (Eff). $= \frac{5}{8} \times$ Thickness of thinner member

$\therefore t_e = 5/8 \times 15 = 9.375$ mm

For the length,

$$T_{dw} = \frac{f_y}{\gamma_{mw}} L_w \cdot t_e$$

or, $$450 \times 10^3 = \frac{250}{1.25} \times L_w \times 9.375$$

or, $L_w = 240$ mm

② Double Butt weld

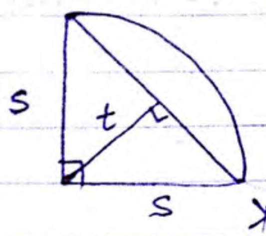
t_e = Thickness of thinner member

= 15 mm

$$T_{dw} = \frac{f_y}{\gamma_{mw}} L_w \cdot t_e$$

$$L_w = \frac{450 \times 10^3 \times 1.25}{250 \times 15} = 150 \text{ mm}$$

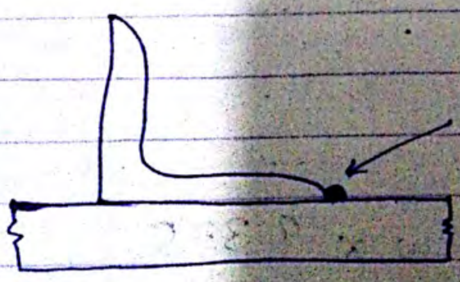
FILLET WELD :-



* Reinforcement.

1) Max. value of s = Thickness − 1.5 mm ..for square edge
[10.5.8.1]

2) For rounded toe of a rolled section, size of weld should not exceed 3/4 th of thickness of section at toe (10.5.8.2)



3) Min. value of 's' (From Table 21)

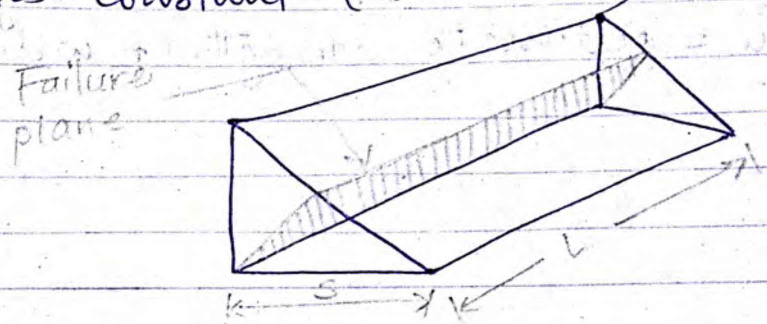
4) Eff. throat thickness $\geq$ 3 mm

$<$ 0.7 x thickness of thinner plate

For stress calculation (10.5.3.2)

5) Effective throat thickness $(t_e) = k \cdot s$.

where, k is constant (Table 22)

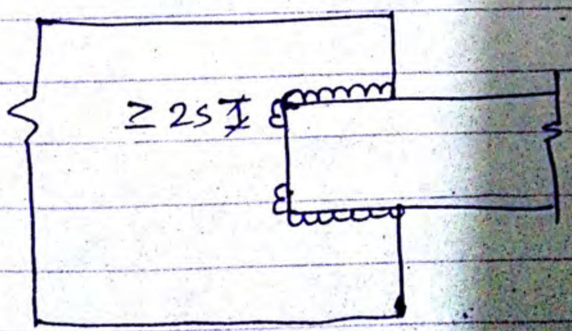


6) Eff. area $\Rightarrow A_w = L_e \times t_e$.

7) Eff. length $l_e = L - 2s$ $(L \geq 4s)$

L is the weld length provided in the field.

(The deduction (2s) is made to allow for craters to be formed at the ends of welded length).



Length of End returns = 2s

↓

To relieve weld length from high stress concentrations at their ends.

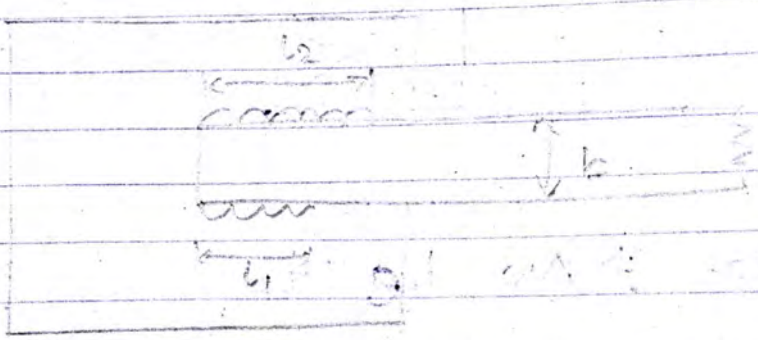
[10.5.1.1]

(8) In lap connections,
Min. length of weld $\geq$ 4 Times thickness of thinner part or 40 mm
[10.5.1.2]

9) Design strengths as per code [10.5.7.1.1]

$$f_{wd} = \frac{f_{wn}}{\gamma_{mw}} = \frac{f_u}{\sqrt{3}\,\gamma_{mw}}$$

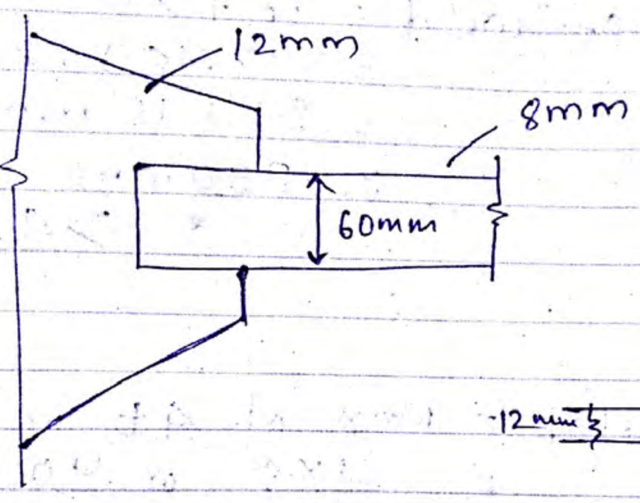
f_{wn} = nominal shear strength of fillet weld
f_u = ultimate strength of weld or parent metal



∴ $l_1 > b$ & $l_2 > b$
(lap connections)

* Service load/Characteristic load $\times \gamma_{mw}$ = Factored Load.

(8) A tie member 60mm x 8mm is connected to a gusset plate of 12mm thick to transmit load of 200kN. Design the fillet weld & necessary overlap. Use Fe 410 grade.



$P_w = 200 \text{ KN}$

$P_{dwd} = 1.5 \times 200 = 300 \text{ KN}$

Size :-

$S_{max} = t - 1.5 = 8 - 1.5 = 6.5 \text{ mm}$

$S_{min} = 5 \text{ mm}$ [Table 21]

Take $S = 6 \text{ mm}$.

Eff. throat thickness :-

$t_e = k \cdot S$

for fusion angle $90°$, $k = 0.7$ (Table 22)

$\therefore t_e = 0.7 \times 6 = 4.2 \text{ mm}$

Fe410 : $f_u = 410 \text{ MPa}$

$f_y = 250 \text{ MPa}$

$$P_{wd} = \frac{f_{wn}}{\gamma_{mw}} \cdot A_w = \frac{f_u}{\sqrt{3}} \cdot \frac{A_w}{\gamma_{mw}}$$

$$300 \times 10^3 = \frac{410 \times l_e \times t_e}{\sqrt{3} \times 1.25}$$

$$l_e = \frac{300 \times 10^3 \times \sqrt{3} \times 1.25}{410 \times 4.2} = 377.189 \text{ mm}$$

Provide length of weld $= l_e + 2s$

$= 377.189 + 2 \times 6$

$= 389.19$ mm

$\simeq 390$ mm

$> 4s = 24$ mm

(OK).

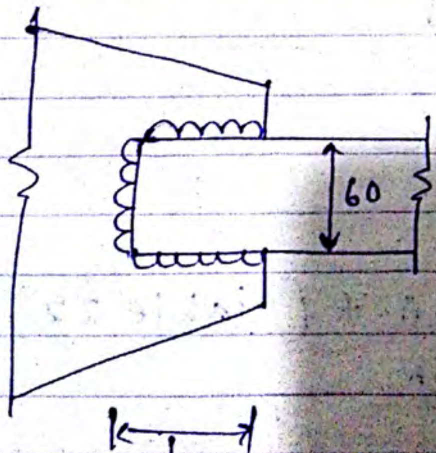
Min Size of Overlap (10.5.1.2) $=$ Max. of $4t$ or 40 mm

$= 4 \times 8$ or 40

$= 32$ or 40 mm

$\simeq 40$ mm

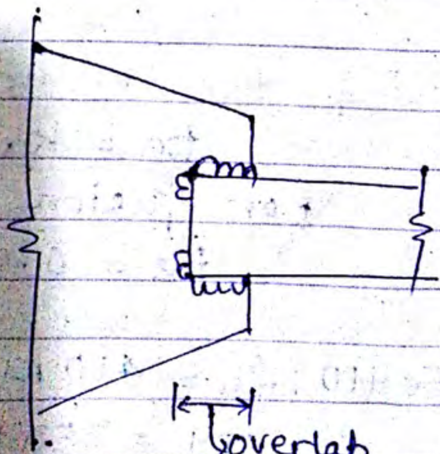
End returns $= 2s = 2 \times 6 = 12$ mm



overlap $= \frac{390-60}{2}$

$= 165\text{mm} > 40\text{mm}$

(OK)



overlap

$= \frac{390}{2} = 195\text{mm}$

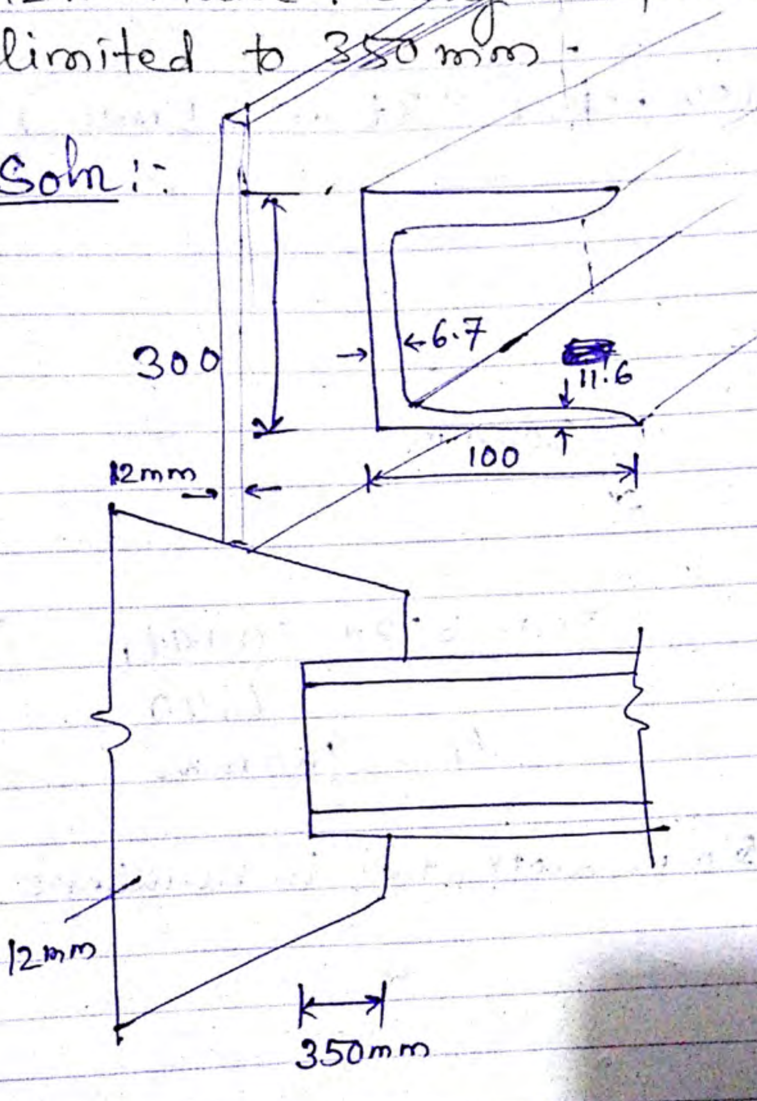
> 40

(OK)

Plug & slot weld

,) An ISLC 300 @ 324.7 N/m (Fe 410) is to carry factored tensile force of 900 kN. The channel section is welded at site to a gusset plate 12 mm thick. Design a fillet weld. If the overlap is limited to 350 mm.

Soln :-



Fe 410

$f_u = 410$ MPa

$f_y = 250$ MPa

Site weld $\gamma_{mw} = 1.5$

Size of weld :-

max size (s) = T − 1.5
(square edge) = 6.7 − 1.5
= 5.2 mm

Min. size = 5 mm

Adopt size (s) = 5 mm

Throat thickness :-

For fusion angle 90°

$t_e = k.s = 0.7 \times 5$
$= 3.5$ mm

$$P_{wd} = \frac{f_{wn}}{\gamma_{mw}} \cdot l_e \cdot t_e$$

or, $900 \times 10^3 = \frac{410}{\sqrt{3} \times 1.5} \times l_e \times 3.5$

$\therefore l_e = 1629.46$ mm

$\therefore$ Length of weld provided $= 1629.46 + 2 \times 5 = 1639.45$ mm
≈ 1640 mm > 4s (OK)

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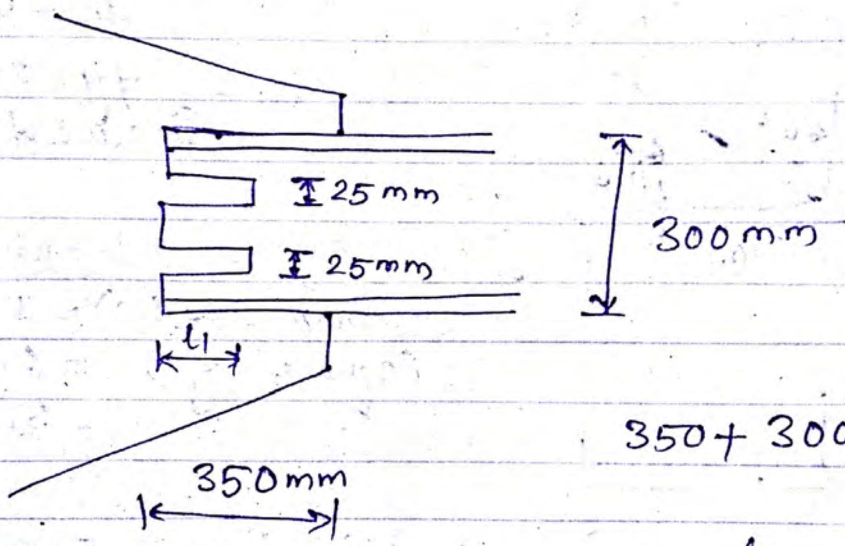
$$\text{Overlap} = \frac{1640 - 300}{2} = 670\,mm > 350\,mm$$
(restriction)

So provide slot,

Provide 2 slot.

Width of slot = Max. of ($\geq 3t$ or 25mm)



$350 + 300 + 350 + 4l_1 = 1640$

$l_1 = 160\,mm$

Provide 160mm x 25mm slots, two in numbers

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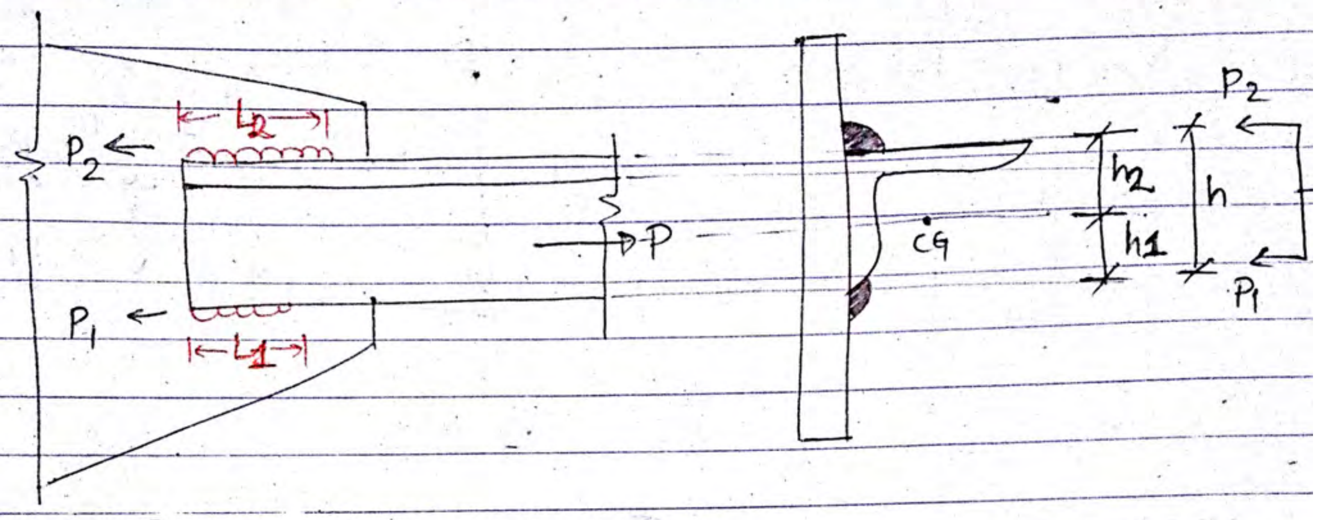
2075/02/04

Fillet Weld for Truss Member



$P = P_1 + P_2$

Taking moment about the welding L_1,

$\Sigma M_{L_1} = 0$

$P_2 \times h = P \times h_1$

$$\boxed{P_2 = \frac{h_1}{h} \cdot P}$$

$\Sigma M_{L_2} = 0$

$P_1 \times h = P \times h_2$

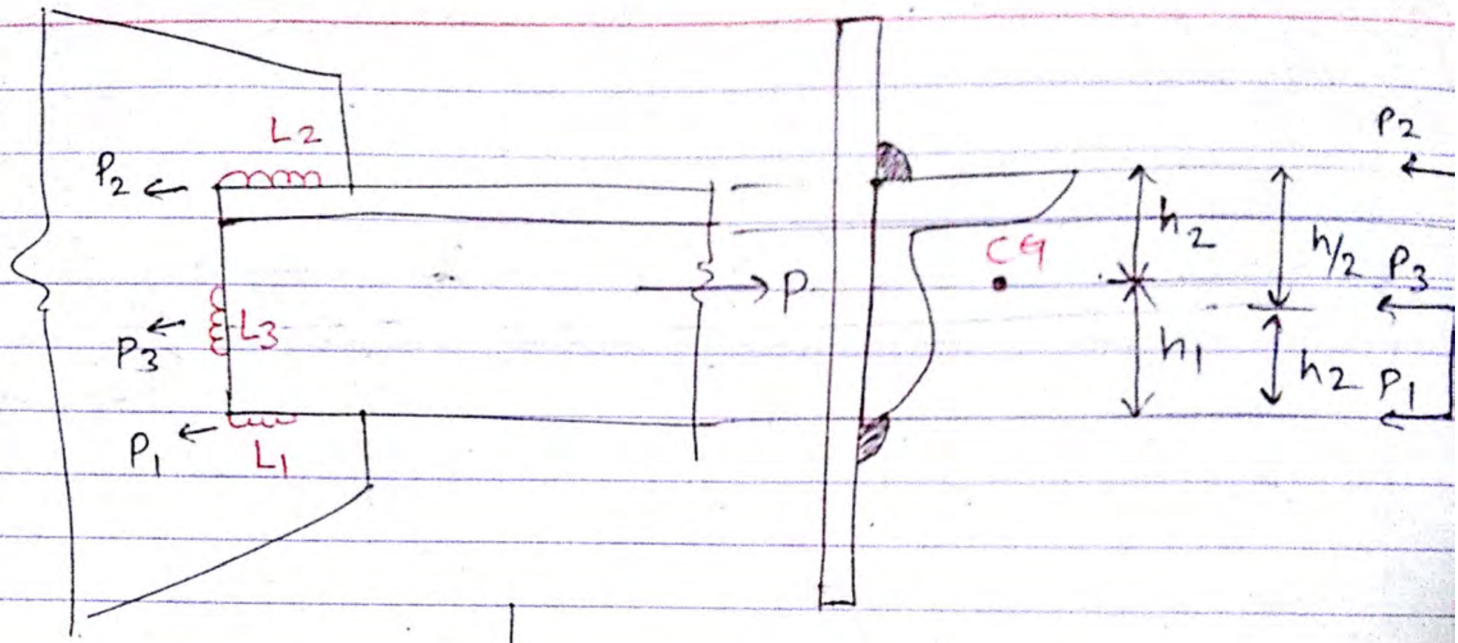
$$\boxed{P_1 = \frac{h_2}{h} \cdot P}$$

L_1 = eff. length of fillet weld on the toe of the angle.

L_2 = eff. length of fillet weld on the back of angle.

P_1 = Design S.F. in weld of eff. length l_1

P_2 = " " " " " " " l_2.



$$\Sigma M_{L_1} = 0$$
$$P_2 \times h - P \times h_1 + P_3 \times h/2 = 0$$
$$2P_2 + P_3 = \frac{2Ph_1}{h} \quad \text{— (I)}$$

$$\Sigma M_{L_2} = 0$$
$$P_1 \times h - P \times h_2 + P_3 \times h/2 = 0$$
$$2P_1 + P_3 = \frac{2Ph_2}{h} \quad \text{— (II)}$$

$$P_1 + P_2 + P_3 = P \quad \text{— (III)}$$

If $L_3 = h$

$$P_3 = \frac{f_u}{\sqrt{3}\,\gamma_{mw}} \cdot l_3 \times t_e$$

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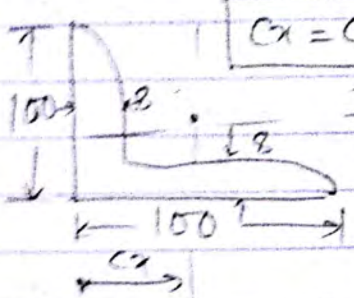
(8) In a truss, ∠ 100 100×8 mm is subjected to factored tension of 200 kN. It is to be connected to a gusset using fillet weld at toe & back. Find length of weld required so that the centre of gravity of weld lies in the plane of CG of angle, $f_u = 410$ MPa i.e. ultimate strength of plate.



Soln: -

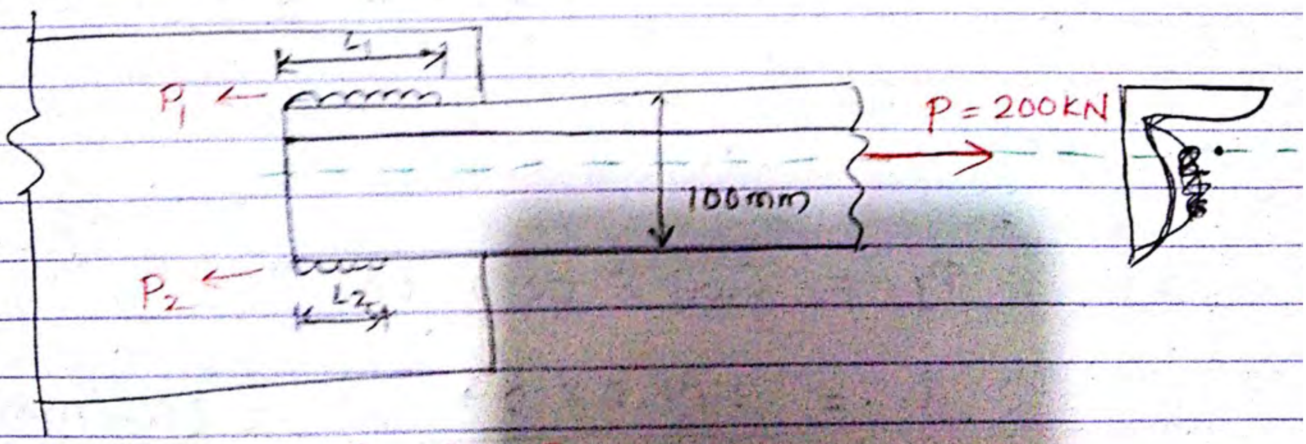
Factored = 200 kN

f_u = 410 MPa

Design strength of weld $f_{wd} = \frac{f_{wn}}{\gamma_{mw}} = \frac{f_u}{\sqrt{3} \cdot \gamma_{mw}}$

$= \frac{410}{\sqrt{3} \times 1.25}$ (shop weld)

$= 189.4$ MPa



$P = P_1 + P_2$

Size of weld $(s) \le \frac{3}{4} \times 8 = 6\,mm$

S_{max} (at toe) $= \frac{3}{4} \times 8 = 6\,mm$

S_{max} (at top) $= t - 1.5 = 8 - 1.5 = 6.5\,mm$

$S_{min} = 3\,mm$

$\therefore$ Provide $\boxed{s = 6\,mm}$

Now,

~~Taking mo~~ Throat thickness $(t_e) = k.s$

$= 0.7 \times S$

$\boxed{t_e = 4.2\,mm}$

Taking moment about welding ~~leg~~ L_1,

$P_2 \times 100 = P \times 27.6$

$P_2 = \frac{200 \times 27.6}{100} = 55.2\,kN$

$P_1 = P - P_2 = 200 - 55.2$

$= 144.8\,kN$

Now,

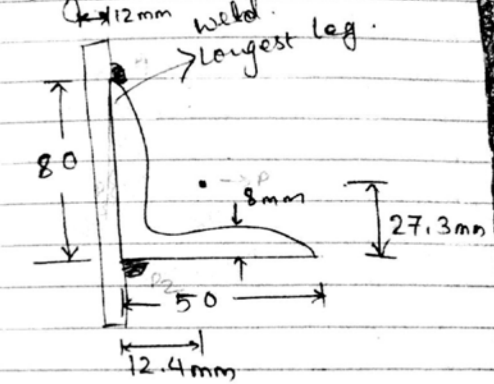
$P_1 = f_{wd} \times t_e \times l_1$

$l_1 = \frac{144.8 \times 10^3}{189.4 \times 4.2} = \underline{182\,mm}$

And, $l_2 = \frac{P_2}{f_{wd} \times t_e} = \frac{144.8 \times 10^3}{189.4 \times 4.2} = \underline{\underline{69.4\,mm}}$

So, it lies in the central axis, there would be no rotation.

(Q) In a truss member, consisting of ISA 80x50x8mm is connected to transmit the working load of 550 kN. This member is connected to gusset plate of 12 mm thickness at the back & toe. Use Fe 410 grade. Design weld to transmit the given load.



Fe 410, $f_u = 410\,MPa$

$f_y = 250\,MPa$

Working load $= 550\,kN$

Factored load $= 1.5 \times 550$

$= 825\,kN$

Size of weld

S_{max} (at ~~toe~~) $= \frac{3}{4} t \rightarrow$ thickness of angle section.

$= 3/4 \times 8 = 6\,mm$.

$S_{min} = 5\,mm$.

Take $\boxed{S = 5\,mm}$

S_{max} at top $= t - 1.5$

$= 8 - 1.5$

$= 6.5\,mm$

$t_e = k.s$.

for fusion angle $90°$, $k = 0.7$

$t_e = 0.7 \times 5 = \underline{3.5\,mm}$.

Take $L_3 = 80\,mm$

Then,

$P_3 = \frac{f_u}{\sqrt{3}\,\gamma_{mw}} . l_3 . t_e$

or, $P_3 = \dfrac{410}{\sqrt{2} \times 1.25} \times 80 \times 3.5$ (Shop welding)

$= 53023.85\ N$

$\therefore \boxed{P_3 = 53.024\ kN}$.

$\Sigma M_{L2} = 0$

or, $P_3 \times 40 + P_1 \times 80 = P \times 27.3$

$P_1 = \dfrac{825 \times 27.3 - 53.024 \times 40}{80}$

$\boxed{P_1 = 255.019\ kN}$

$P_2 = P - P_1 - P_3 = 825 - 255.019 - 53.024$

$\boxed{P_2 = 516.957\ kN}$

$P_1 = \dfrac{f_u}{\sqrt{3}.\gamma_{mw}} \times l_1 \times t_e$

$l_1 = \dfrac{P_1 \times \sqrt{3}.\gamma_{mw}}{f_u \times t_e} = 384.76\ mm$

$l_2 = 779.96\ mm$

Provide $L_1 = 384.76 + 2s = 395\ mm$

$L_2 = 779.96 + 2s = 790\ mm$.

$L_3 = 80\ mm > 4s$

$= 4 \times 5$

$= 20\ mm$ (OK).

Eccentric Welding Connection

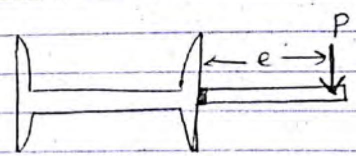
Welding connection subjected to combination of shear, bending & twisting are considered as eccentric welding connection. This type of connection are generally used in beam to column end connection.

Bracket Connection

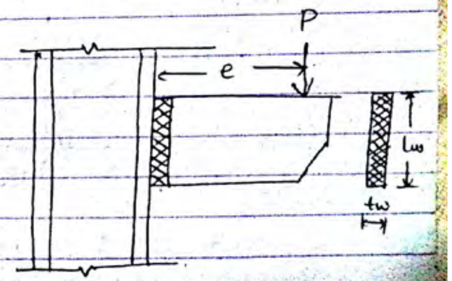
Bracket means a steel plate or set of steel plates connected to columns to support beams. The bracket may be connected to a column using butt or fillet weld. Welds are subjected to shear force, and a bending moment or a twisting moment.

Bracket Connection Using Butt Weld

Butt weld is subjected to vertical shear force V = P and a bending moment M = P.e.

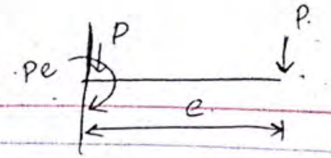


Plan



⇒ Consider a bracket connection to flange of a column by a full penetration butt weld.

⇒ Eccentric load 'P' causes shear force 'P' & moment at the weld section.



$$\frac{M}{I} = \frac{f_b}{y}$$

$$f_b = \frac{M}{I/y} = \frac{M}{Z}$$

Now,
Shear stress in the weld due to factored load.

$$q = \frac{V}{l_w \cdot t_w}$$

Similarly, Bending stress in the weld due to factored load:

$$f_b = \frac{M}{Z} = \frac{P.e}{\frac{t_w \cdot l_w^2}{6}} = \frac{6\,Pe}{t_w \cdot l_w^2}$$

As per code, (Cl. 10.5.10.1 Page 89)
Equivalent stress (f_e)

$$f_e = \sqrt{f_b^2 + 3q^2} \le \cancel{\frac{f_u}{\sqrt{3}\,\gamma_{mw}}}\ f_d$$

where,

$$f_d = \frac{f_y}{\gamma_{mo}} \quad \text{for E250 (Fe 410)}$$

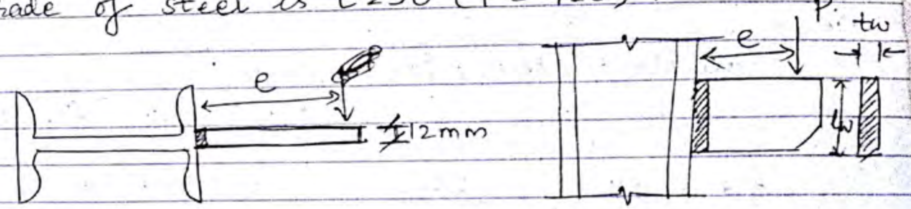
$$= \frac{f_u}{\sqrt{3}\,\gamma_{mw}} \quad \text{for other grade.}$$

$\gamma_{mo} = 1.1$ (from Table 5 Page 30)

Approx. length of weld $= \sqrt{\dfrac{6P.e}{t_w\left(f_y/\gamma_{mo}\right)}}$
↓
Considering only bending stress
(Actual eff. length of weld will be more since weld is subjected to shear stress too).

2075/02/06

(Q) Design a connection for the bracket using Butt weld carrying factored vertical load of 200 kN acting at 300mm from the face of column. Take thickness of bracket as 12 mm, grade of steel is E250 (Fe 410).



Given:-
$P = 200\,kN$.
$e = 300\,mm$.
$t_{bracket} = 12\,mm$ (Generally thickness of bracket >10mm)
E250 ; $f_u = 410\,MPa$
$f_y = 250\,MPa$.

Take thickness of weld $t_w = 12\,mm$.

Approximate length of weld.

$$l_w = \sqrt{\frac{6\,Pe}{t_w\left(f_y/\gamma_{mo}\right)}}$$

$$= \sqrt{\frac{6 \times 200 \times 10^3 \times 300}{12 \times 250/1.1}}$$

$$= 363.318\,mm$$

Take $l_w = 400\,mm$

shear stress $q = \frac{V}{l_w \times t_w} = \frac{200\times10^3}{400 \times 12} = 41.67$ MPa

Bending stress $f_b = \frac{6Pe}{t_w . l_w^2} = \frac{6 \times 200\times10^3 \times 300}{12 \times (400)^2}$

$= 187.5$ N/mm².

Now,

Equivalent stress $f_e = \sqrt{f_b^2 + 3q^2}$
$= 200.91$ MPa $< f_d$ (ok)

$f_d = \frac{f_y}{\gamma_{mo}} = \frac{250}{1.1} = 227.27$ MPa.

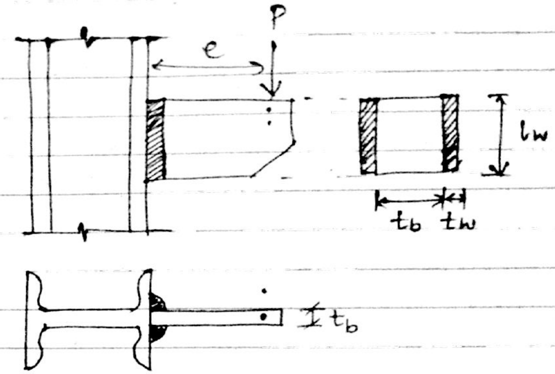
Provide 400mm deep & 12mm thick Bracket (full penetration Butt weld).

Bracket Connection Using Fillet Weld.

These are two ways of connecting a bracket to a column using a fillet weld. Bracket connected perpendicular to the flange of column or in the plane of flange of a column i.e. parallel to flange of column.

(i) Bracket perpendicular to flange of Column.
Bracket is connected to flange of a column with fillet welds on either side of bracket.

The fillet welds are subjected to factored shear force $V = P$ and factored B.M. ($M = P.e$), due to which shear stresses in horizontal & vertical directions are developed in the fillet welds at their throats.



~~Bending Stress q_b =~~

Horizontal Shear stress due to bending moment $q_b = \frac{M}{Z}$

$$q_b = \frac{P.e}{2 \times \frac{t_w . l_w^2}{6}} \Rightarrow q_b = \frac{M}{I} \times y$$

$$= \frac{P.e}{2 \times \frac{t_w . l_w^3}{12}} \cdot \frac{l_w}{2}$$

$$\boxed{q_b = \frac{3P.e}{t_w . l_w^2}} \qquad = \frac{3Pe}{t_w . l_w^2}$$

Vertical shear stress at weld due to shear force

$$q_s = \frac{V}{2 . t_w . l_w} = \frac{P}{2 t_w . l_w}$$

$t_w = k.s$.

Now,

$$q_e = \sqrt{q_b^2 + 3q_s^2} \leq f_{wd}$$

where,

$$f_{wd} = \frac{f_u}{\sqrt{3}\ \gamma_{mw}}$$

Now, Approx. length may be obtained by considering the shear stress in weld due to bending only.

$$q_b = f_{wd} = \frac{3Pe}{t_w . l_w^2}$$

$$l_w = \sqrt{\frac{3Pe}{t_w \cdot f_{wd}}}$$

(Q) Design a connection for bracket using fillet weld when bracket plate is connected to column flange perpendicularly. Take P = 200 kN (factored), e = 300 mm, thickness of bracket = 12 mm. and Fe 410 grade of steel.

Soln. :-

$S_{max} = t - 1.5 = 12 - 1.5 = 10.5$ mm.

$S_{min} = 5$ mm (for $10mm \leq t \leq 20mm$).

Take $\boxed{S = 6\ mm}$.

For fusion angle $90°$, $k = 0.7$ (Table 22) Page 78

$t_w = 0.7 \times 6 = 4.2$ mm

Now,

Effective length of fillet weld may be approximately obtained by considering the shear stress in weld ~~durn~~ due to bending only.

$$l_w = \sqrt{\frac{3Pe}{t_w . f_{wd}}}$$

where,

$$f_{wd} = \frac{f_{wn}}{\gamma_{mw}} = \frac{f_u}{\sqrt{3} . \gamma_{mw}} = \frac{410}{\sqrt{3} \times 1.25}$$

$$= 189.37 \text{ MPa}$$

$$\therefore l_w = \sqrt{\frac{3 \times 200 \times 10^3 \times 300}{4.2 \times 189.37}} = 475.72 \text{ mm}$$

~~Take lw =~~ A 500 mm thick bracket may be tried.

Effective length of fillet weld, $l_w = 500 - 2 \times 6 = 488$

$$q_b = \frac{3Pe}{t_w . l_w^2} = \frac{3 \times 200 \times 10^3 \times 300}{4.2 \times 488^2} = 179.96 \text{ MPa}$$

$$q_s = \frac{V}{2 . t_w . l_w} = \frac{200 \times 10^3}{2 \times 4.2 \times 488} = 48.8 \text{ MPa}$$

Cl. 10.5.10.1.1 Page 80

$$\boxed{f_e = \sqrt{q_b^2 + q_s^2}}$$

$= 186.46 < 189.37$ MPa (ok)

$$f_{wd} = \frac{f_u}{\sqrt{3} . \gamma_{mw}} = \frac{410}{\sqrt{3} \times 1.25} = 189.37$$

Hence, 500mm depth bracket is connected by 6mm size fillet weld on either side.

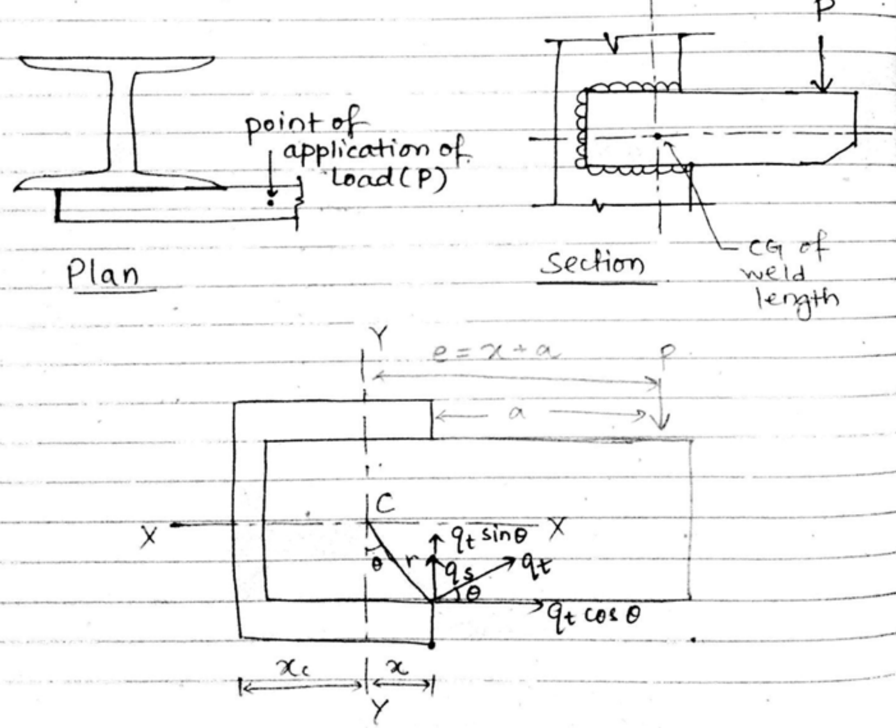
(ii) Bracket in the plane of flange.

→ The fillet weld is subjected to a shear force $V = P$ & a twisting moment $M = P.e$

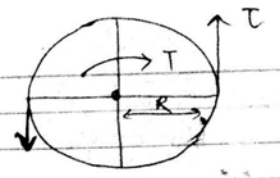
→ Due to shear force P, shear stress (q_s) is produced at the throat of fillet weld.

→ Another shear stress (q_t) is developed due to twisting moment Pe.

→ Dir^n of q_t at any point on length of weld is ⊥ar to the line joining that point with centroid C of the weld.



$T = P.e.$



$$\tau = \frac{T}{I_p} \times R$$

Shear stress due to direct shear.

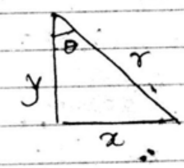
$$q_s = \frac{V}{L_w \cdot t_w} = \frac{P}{L_w \cdot t_w} \quad (\uparrow)$$

Shear stress due to torsion ($M_t = P.e$).

$$q_t = \frac{M_t}{I_p} \times r = \frac{P.e.r}{I_p}$$

where, I_p = Polar moment of inertia of the eff. weld section.

$$q_{tx} = q_t \cos\theta$$
$$q_{ty} = q_t \cdot \sin\theta$$



$$\cos\theta = y/r$$
$$\sin\theta = x/r.$$

$$q_{tx} = q_t \cdot \frac{y}{r}$$
$$= \frac{P.er}{I_p} \cdot \frac{y}{r} = \frac{P.e.y}{I_p} \quad (\rightarrow)$$

$$q_{ty} = q_t \cdot \sin\theta$$
$$= \frac{P.er}{I_p} \cdot \frac{x}{r} = \frac{P.e.x}{I_p} \quad (\uparrow)$$

Net result,

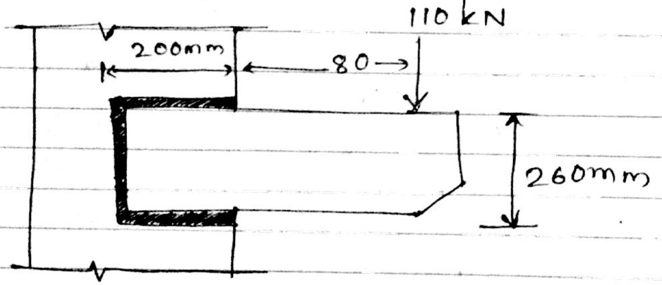
$q_x = q_{tx} = \frac{P e y}{I_p}$

$q_y = q_{ty} + q_s$

$= \frac{P \cdot e \cdot x}{I_p} + \frac{P}{L_w \cdot t_w}$

$q_e = \sqrt{q_x^2 + q_y^2} \le \frac{f_u}{\sqrt{3} \cdot \gamma_{mw}}$.

(Q) A bracket plate is weld to the flange of column section. Factored load = 110 kN. Assume Fe 410 Grade & shop weld.



Soln: -

$P = 110$ kN

$\gamma_{mw} = 1.25$ (shop weld)

$L_w = 200 \times 2 + 260 = 660$ mm

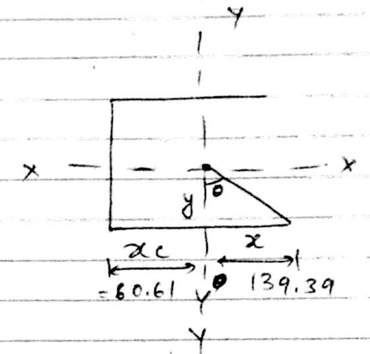
Let t_w be thickness of weld.

Now, Centroid of weld :-

$x_c = \frac{\Sigma l_i x_i}{\Sigma l_i}$

$= \frac{200 \times 100 + 260 \times 0 + 200 \times 100}{660}$

$= 60.61$ mm.



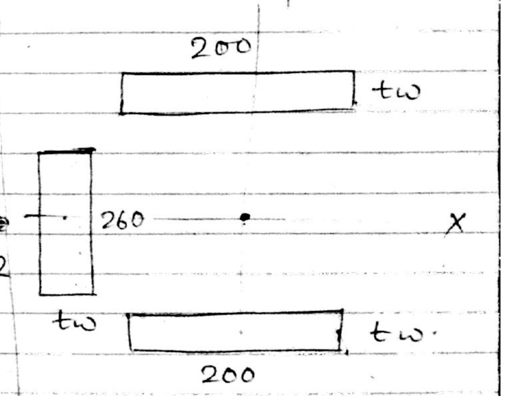
$e = (200 - 60.61) + 80$

or, $\boxed{e = 219.39 \text{ mm}}$

$\begin{cases} x = 139.39 \text{ mm} \\ y = 260/2 = 130 \text{ mm} \end{cases}$

Now,

$I_p = I_x + I_y$.



$I_p = [I_{cg} + Ah^2]_x + [I_{cg} + Ah^2]_y$

$I_x = \left[\frac{200 \times t_w^3}{12} + 200 t_w \times 130^2\right] \times 2 + \left[\frac{260^3 \times t_w}{12}\right]$

$= \cancel{6.76 \times 10^6 \, t_w \text{ mm}^4}$ $2.2217 \times 10^6 \, t_w$

$I_y = \left(\frac{260 \times t_w^3}{12} + 60.61^2 \times 260 t_w\right) + 2 \times \left[\frac{200^3 \times t_w}{12} + 200 t_w \times (100 - 60.61)^2\right]$

$= 2.8091 \times 10^6 \, t_w$

$I_p = 11.1338 \times 10^6 \, t_w$

$$q_x = \frac{P.e.y}{I_p}$$

$$= \frac{110\times10^3 \times 219.39 \times 130}{11.1338 \times 10^6\, t_w}$$

$$= \frac{281.78}{t_w}$$

$$q_y = \frac{P e x}{I_p} + q_s$$

$$= \frac{110\times10^3 \times 219.39 \times 139.39}{11.1338\times10^6\, t_w} + \frac{110\times10^3}{t_w \cdot 660}$$

$$= \frac{468.8}{t_w}$$

$$q_e = \sqrt{q_x^2 + q_y^2}$$

$$= \frac{546.97}{t_w} \le f_d$$

$$\frac{546.97}{t_w} \le \frac{f_u}{\sqrt{3}\,\gamma_{mw}}$$

$$t_w \ge \frac{546.97\sqrt{3} \times 1.25}{410}$$

$$t_w \ge 2.89 \text{ mm}$$

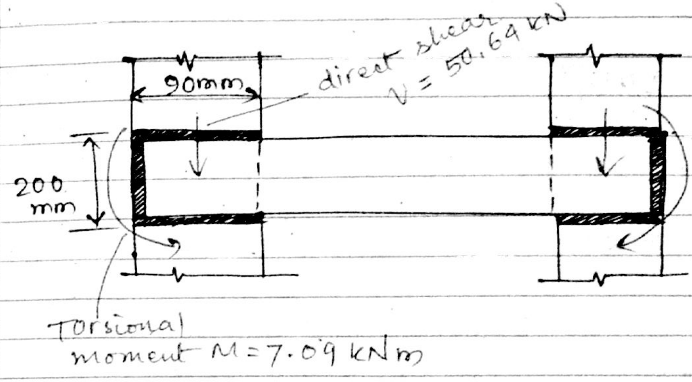
$t_w = ks$

$$\therefore s = \frac{2.89}{0.7} = 4.12 \text{ mm}$$

$s \ge 4.12$ mm

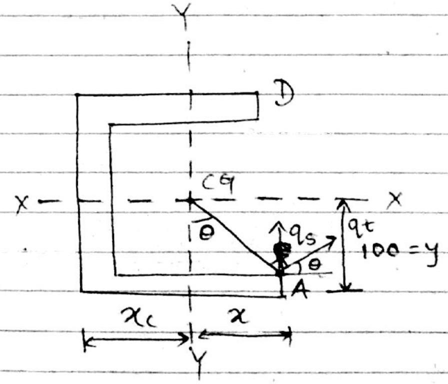
Take $s = 5$ mm

(Q) Design the connection using fillet weld subjected to direct shear 50.64 kN & torsional moment 7.09 kNm.



Soln.

Total length of weld provided = 200+90+90
= 380 mm



$$x_c = \frac{\Sigma l_i x_i}{\Sigma l_i}$$

$$= \frac{90\times45 + 90\times45 + 200\times0}{90+90+200}$$

$$= 21.31 \text{ mm}$$

$x = 90 - 21.31$
$= 68.69$

$y = 100$

$$q_s = \frac{V}{l_w \cdot t_w} = \frac{50.64\times10^3}{380 \cdot t_w} = \frac{133.26}{t_w}\ N/mm^2$$

$$q_x = q_{tx} = 255.01/t_w$$

$$q_y = q_{ty} + q_s = \frac{308.42}{t_w}$$

$$q_e = \sqrt{q_x^2 + q_y^2} = \frac{400.19}{t_w} \le f_d$$

or, $$\frac{400.19}{t_w} \le \frac{410}{\sqrt{3}\times1.25}$$

$$t_w \ge 2.11\ mm$$

Now,

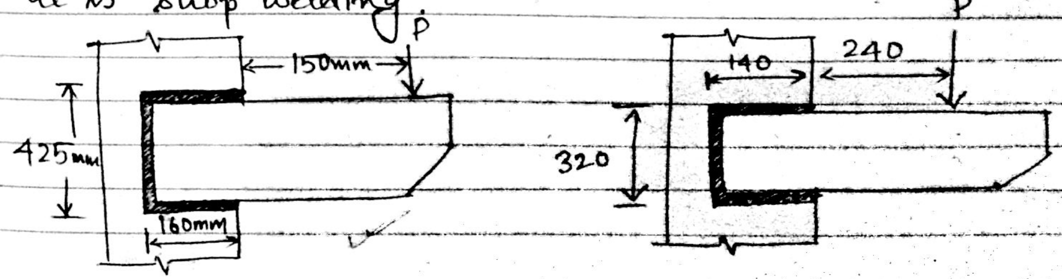
$$t_w = Ks = 0.7s$$

$$s = \frac{t_w}{0.7} = \frac{2.11}{0.7}$$

$$s \ge 3.02\ mm.$$

∴ A 4mm size fillet weld will be provided. (This size of 4mm fillet will hold good for up 10mm thick batten).

(Q) Determine the max. load that can be resisted by the bracket shown by fillet weld if size is 6mm. if it is shop welding.



Design Specification for Bolted Connection

Shear Capacity of Bolt (10.3.3)

$$V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}} = \frac{f_{ub}(n_n A_{nb} + n_s A_{sb})}{\sqrt{3} \cdot \gamma_{mb}}$$

where, V_{nsb} = Nominal shear capacity of bolt.

f_{ub} = Ultimate tensile strength of bolt

n_n = No. of shear plane within threading of bolt

n_s = No. of shear planes within shank of a bolt

IS 1367 (Part 3): 2002

A_{sb} = Sectional area of a shank of bolt

A_{nb} = Net shear area of bolt (Net c/s area at the root of the root of thread)

$\gamma_{mb} = 1.25$ | $A_{nb} = 0.78\, A_{sb}$

Bearing Capacity of Bolt (10.3.4)

$$V_{dpb} = \frac{V_{npb}}{\gamma_{mb}} = \frac{2.5\, k_b \cdot d \cdot t\, f_u}{\gamma_{mb}}$$

k_b is smaller of $\left(\frac{e}{3d_0}, \frac{p}{3d_0} - 0.25, \frac{f_{ub}}{f_u}, 1\right)$

t = least thickness of plates.

Tension Capacity of Bolt ~~(Rupture Strength)~~

$$T_b \le T_{db}$$

$$T_{db} = \cancel{\frac{0.9 A_n f_y}{\gamma_{m1}}} \quad \frac{T_{nb}}{\gamma_{mb}}$$

A_n = Net sectional area of bolt.

$$T_{nb} = 0.9\, f_{ub} \cdot A_n < f_{yb} \cdot A_{sb}\left(\frac{\gamma_{mb}}{\gamma_{m0}}\right)$$

Tension capacity of plate in Rupture

$$T_{dn} = \frac{0.9\, f_u \cdot A_n}{\gamma_{m1}} \quad (\text{Cl } 6.3.1 \text{ Pg } 32)$$

$$A_n = (b - n d_0) \cdot t$$

Dia. of hole d_0 = Nominal dia. of bolt + Clearance.

$\gamma_{m1} = 1.25$.

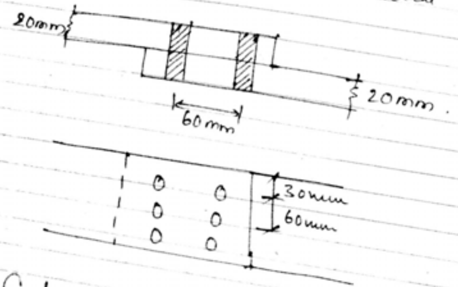
⇒ Strength of the bolted joint is the min. strength based on the shearing of bolt & bearing of bolt.

$$\text{No. of bolt} = \frac{\text{Load to be transferred}}{\text{Bolt Value}}$$

Bolt value = Min. of bearing & shearing

$$\text{Efficiency of joint} = \frac{(\text{Strength of Joint}/\text{pitch length})}{(\text{Strength of Solid plate}/\text{pitch length})} \times 100\%$$

(Q) Find the strength & efficiency of joint. Use M20 bolt of grade 4.6 & material of plate Fe410.



Bolt
$$\begin{cases} d = 20\,mm \\ f_{ub} = 4 \times 100 = 400\,MPa \\ f_{yb} = 400 \times 0.6 = 240\,MPa \end{cases}$$

plate
$$\begin{cases} f_u = 410\,MPa \\ f_y = 250\,MPa \end{cases}$$

Dia. of hole $d_0 = 20 + 2 = 22\,mm$ (Table 19 Page 73).

Design strength in shear,

$$V_{dsb} = \frac{f_{ub}}{\sqrt{3}\,\gamma_{mb}} (n_n A_{nb} + n_s A_{sb})$$

$$\boxed{A_{nb} = 0.78\,A_{sb}}$$ IS 1367 (part 3): 2002

Assume shear plane lies in thread,

$n_s = 0$ $\quad A_{sb} = \frac{\pi}{4}(20)^2 = 314\,mm^2$.

$n_n = 1$.

$$V_{dsb} = \frac{400}{\sqrt{3} \times 1.25} (1 \times 31.4 \times 0.78)$$

$= 45.272\,kN$.

Strength of single bolt in shearing $= 45.272\,kN$

" " 6 " " " $= 45.272 \times 6$

$= 271.632\,kN$

Design strength of bolt in Bearing.

e = Edge distance $= 30\,mm$

p = pitch $= 60\,mm$

$$k_b = \text{Min. of} \left(\frac{e}{3d_0}, \frac{p}{3d_0} - 0.25, \frac{f_{ub}}{f_u}, 1 \right).$$

$$= \left(\frac{30}{3 \times 22}, \frac{60}{3 \times 22} - 0.25, \frac{400}{410}, 1 \right)$$

$$= (0.4545, 0.659, 0.9756, 1)$$

$k_b = 0.4545$

$$V_{dpb} = \frac{2.5\,k_b\,d\,t\,f_u}{\gamma_{mb}} = \frac{2.5 \times 0.4545 \times 20 \times 20 \times 410}{1.25}$$

$149.076\,kN$

Bearing of 6 bolts $= 6 \times 149.076 = 894.456\,kN$

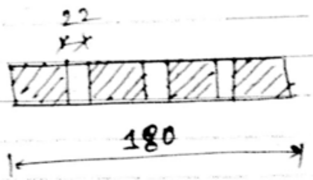
1

Design strength intension of a plate (Rupture)

$$T_{dn} = \frac{0.9\, A_n f_u}{\gamma_{m1}}$$

A_n = Net Area

$= [180 - 3 \times 22] \times 20 = 2280$



$$T_{dn} = \frac{0.9 \times 2280 \times 410}{1.25} = 673.056 \text{ kN}.$$

Strength of Joint = Min. of above values

= 271.632 kN

~~Efficiency = 271.632~~

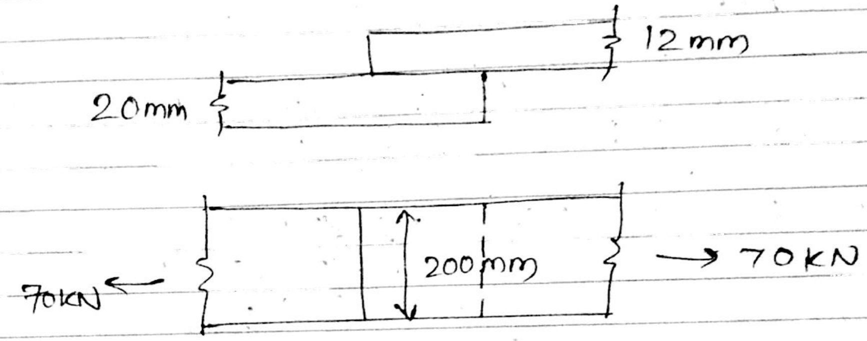
Strength of solid plate $= \frac{A_g f_y}{\gamma_{m0}}$ (Cl 6.2)

$= \frac{180 \times 20 \times 250}{1.1}$

$= 818.182$ kN

$$\text{Efficiency} = \frac{271.632}{818.182} \times 100 = 33.2\%$$

(Q) Design a lap joint between two plate shown in the figure, so as to transmit the factor load of 70kN using M16 bolt of grade 4.6 & the grade of plate material Fe 410.



$d = 16$ mm

$f_{ub} = 400$ MPa

$f_{yb} = 240$ MPa

$f_u = 410$ MPa

$f_y = 250$ MPa

Clearance $= 2$ mm (Pg 73 Table 19)

dia. of hole $d_0 = 16 + 2 = 18$ mm

Design Shear Capacity of Bolt (10.3.3)

$$V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}} = \frac{f_{ub}}{\sqrt{3}\,\gamma_{mb}}\left(n_n A_{nb} + n_s A_{sb}\right)$$

Assume shear plane lies on thread,

$n_n = 1$

$n_s = 0$

$A_{sb} = \frac{\pi}{4} \times 16^2 = 200.96 \text{ mm}^2$

$A_{nb} = 0.78\, A_{sb} = 156.75 \text{ mm}^2$

$$V_{dsb} = \frac{400}{\sqrt{3} \times 1.25}(1 \times 156.75)$$

$$= 28.96 \text{ kN}$$

Design Bearing Capacity of Bolt

$$V_{dpb} = \frac{V_{npb}}{\gamma_{mb}} = \frac{2.5\, k_b . d . t . f_u}{\gamma_{mb}}$$

$$k_b = \frac{e}{3d_o},\ \frac{p}{3d_o} - 0.25,\ \frac{f_{ub}}{f_u},\ 1.0$$

Page 73 Cl 10.2.2. Pitch (p) $\nless 2.5d$

$\nless 2.5 \times 16$

$\therefore \nless 40\,mm$

$P_{min} = 40\,mm$

$P_{max} = 16t$ or $200\,mm$

$= 16 \times 12$

$= 192\,mm$ or $200\,mm$

$P_{max} = 192\,mm$

$e_{max} = 12t\varepsilon$

$= 12 \times 12 \times \left(\frac{250}{f_y}\right)^{1/2}$

$= 12 \times 12 \times \left(\frac{250}{250}\right)^{1/2}$

$= 144\,mm$

$e_{min} = 1.7 d_o = 1.7 \times 18$

$= 30.6\,mm$

Take $e = 40\,mm$

$p = 50\,mm$

$$k_b = \frac{40}{3 \times 18},\ \frac{50}{3 \times 18} - 0.25,\ \frac{400}{410},\ 1.0$$

$= 0.75,\ 0.6759,\ 0.9756,\ 1.0$

$k_b = 0.6759$

$$V_{dpb} = \frac{2.5 \times 0.6759 \times 16 \times 12 \times 410}{1.25}$$

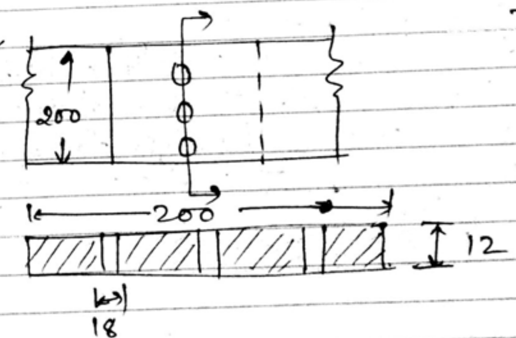
$= 106.414\,kN$

Bolt strength = Min. of shearing & Bearing

$= 28.96\,kN$

Bolt value $= \frac{70}{28.96} = 2.417 \simeq 3$ bolts.

case (I)



Total design load carried by bolt

$= 3 \times 28.96$

$= 86.88\,kN$

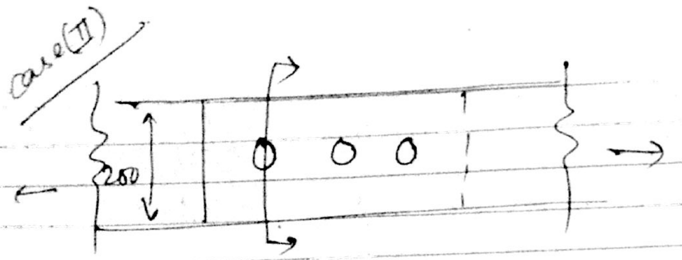
$A_n = [200 - 18 \times 3] \times 12 = 1752$

$T_{dn} = \frac{0.9\, A_n f_u}{\gamma_{m1}}$ (Page 32 Cl 6.3.1)

$= \frac{0.9 \times 1752 \times 410}{1.25} = 517.2\,kN > 86.88\,kN$

(ok)

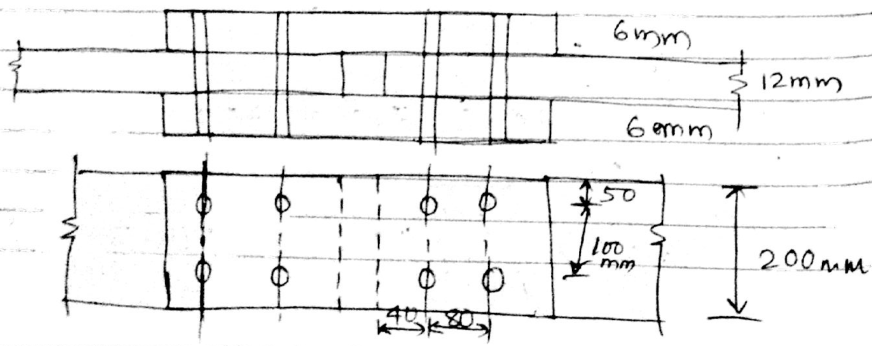
case (II)



$A_n = [200 - 18] \times 12 = 2184 \text{ mm}^2$

$T_{dn} = \frac{0.9 A_n f_u}{\gamma_{m1}} = 644.717 \text{ kN} > 86.88 \text{ kN}$ (safe) OK

(Q) Two 12 mm thick steel plate are spliced by two 6 mm thick plate with four M16 bolt of property class 4.6. Determine the ultimate load carrying capacity of the connection & check for the cover plate. Use Fe 410 grade of steel. & also check for block shear failure.



Soln :-

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

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

$f_{yb} = 240 \text{ MPa}$

$f_u = 410 \text{ MPa}$

$f_y = 250 \text{ MPa}$

Design strength of single bolt

(1) In Shear :- (Pg 75)

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

$= \frac{f_u}{\sqrt{3}\,\gamma_{mb}} (n_n A_{nb} + n_s A_{sb})$

Consider, $n_n = 1$

$n_s = 1$

$A_{sb} = \frac{\pi}{4} \times d^2 = \frac{\pi}{4} (16)^2 = 201.062 \text{ MPa}.$

$A_{nb} = 0.78 A_{sb} = 156.828$

$V_{dsb} = \frac{400}{\sqrt{3} \times 1.25} (1 \times 156.828 + 1 \times 201.062)$

$= 66.121 \text{ kN}$

Strength of 4 bolts in shear $= 4 \times 66.121$

$= 264.484 \text{ kN}$

(2) In Bearing:

$e_{min} = 1.7 d_o = 1.7 \times 18 = 30.6$ mm

$e_{max} = 12t\left(\frac{250}{f_y}\right)^{1/2} = 12 \times 12 \times \left(\frac{250}{250}\right)^{1/2}$

$= 144$ mm

$P_{min} = 2.5 \times d = 2.5 \times 16 = 40$ mm

$P_{max} = 16t$ or 200 mm

$= 16 \times 12$ or 200

$= 192$ or 200

$P_{max} = 192$ mm.

Take $e = 40$ mm

$P = 80$ mm.

$K_b = \frac{e}{3d_o}, \frac{P}{3d_o} - 0.25, \frac{f_{ub}}{f_u}, 1.$

$= 0.7407, \quad 1.2315, \quad 0.9756, \quad 1$

$K_b = 0.7407$.

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

$= \frac{2.5 \times 0.7407 \times 16 \times 12 \times 410}{1.25}$

$= 116.616$ kN.

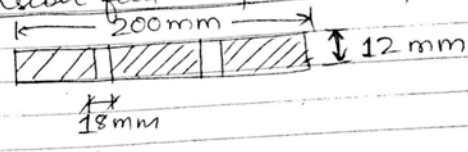
Strength of 4 bolts $= 466.463$ kN.

Now, Bolt value = Min of V_{dsb} & V_{dpb}

$= 264.484$ kN

(3) Check for Plate / Cover Plate :-

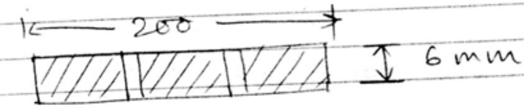
Tension failure of main plate :-



$T_{dn} = \frac{0.9 A_n f_u}{\gamma_{m1}} = \frac{0.9 \times 1968 \times 410}{1.25} = 580.954$ kN

$A_n = (200 - 2 \times 18) \times 12 = 1968$ mm^2

Tension failure of Cover plate :-



$A_n = (200 - 2 \times 18) \times 6 = 984$

$T_{dn} = \frac{0.9 \times 984 \times 410}{1.25} = 290.477$ kN

Strength of two cover plates $= 580.954$ kN > 264.484 kN

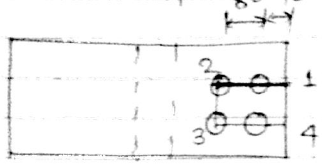
(ok).

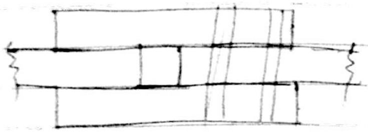
4) Check for Block shear failure :- Pg 33 [6.4]

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

OR

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





Shear plane : 1-2, 3-4

Tension plane : 2-3

$$A_{vg} = (40+80) \times 6 \times 2 \times \underset{\text{2 cover plates}}{\underset{\text{2 shear planes}}{2}}$$

$$= 2880 \text{ mm}^2$$

$$A_{vn} = (40+80-1.5\times 18) \times 6 \times 2 \times 2$$

$$= 2232$$

$$A_{tg} = 80 \times 6 \times 2 = 960$$

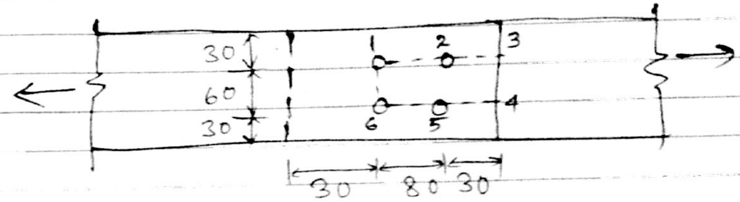
$$A_{tn} = (80-18) \times 6 \times 2 = 744$$

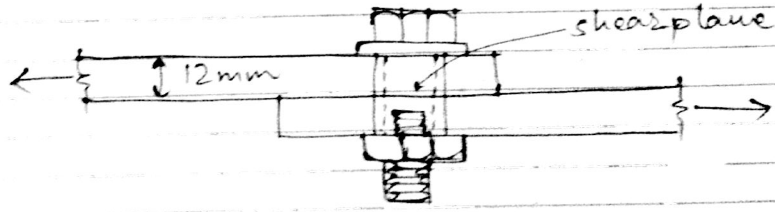
T_{db} = 597.53 kN
= 598.53 kN } Smaller is taken

T_{db} = 597.53 kN

∴ Ultimate Load carrying Capacity of Joint
= 264.421 kN
(Shear failure of Bolt controls)

(5) Determine the ultimate load carrying capacity in tension of lap joint shown in figure. If the bolt threads are outside the shear plane. Use M16 bolts of total grade C & property class 4.6. The yield & ultimate strength of the flats are 250 MPa & 410 MPa respectively.





Property class of bolt - 4.6
f_{ub} = 400 MPa
f_{yb} = 0.6 × 400 = 240 MPa

d_0 = 16 + 2 = 18 mm

Design strength of plate due to rupture at net section

$$T_{dn} = \frac{0.9 A_n f_u}{\gamma_{m1}} = 0.9 \left[\frac{(120 - 2\times 18)\times 12 \times 410}{1.25}\right]$$

$$= \underline{297.56 \text{ kN}} \quad — (1)$$

Design shear strength of bolt (Pg 75)

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

$$= \frac{f_{ub}\,(A_{nb}\,n_n + n_s A_{sb})}{\sqrt{3}\cdot \gamma_{mb}}$$

$$= \frac{400\left(A_{nb} \times 0 + 1 \times \pi/4 \times 16^2\right)}{\sqrt{3} \times 1.25}$$

$$= 37.147 \text{ kN}$$

Load carrying capacity in shear $= 4 \times 37.147$
$= \underline{148.586 \text{ kN}}$.

Design bearing strength of bolt (Pg 75)

$$V_{dpb} = \frac{V_{npb}}{\gamma_{mb}} = \frac{2.5\, k_b\, d\, t\, f_u}{1.25}$$

$$k_b = \frac{30}{3 \times 18},\ \frac{80}{3 \times 18} - 0.25,\ \frac{400}{410},\ 1.0$$

$$= 0.55,\ 1.23,\ 0.9756,\ 1.0$$

$$k_b = 0.55$$

$$V_{dpb} = \frac{2.5 \times 0.55 \times 16 \times 12 \times 410}{1.25}$$

$$= 86.592 \text{ kN}$$

Design strength of connection in bearing
$= 4 \times 86.592 = \underline{346.368 \text{ kN}}$

$\therefore$ Design strength in connection $= 148.586$ kN — (2)

Design strength due to Block shear.

$$T_{db} = \frac{A_{vg} f_y}{\gamma_{m0}\sqrt{3}} + \frac{0.9 A_{tn} f_u}{\gamma_{m1}} = 495.19 \text{ kN}$$

$$= \frac{0.9 A_{vn} f_u}{\gamma_{m1}\sqrt{3}} + \frac{A_{tg} f_y}{\gamma_{m0}} = 503.14 \text{ kN}$$

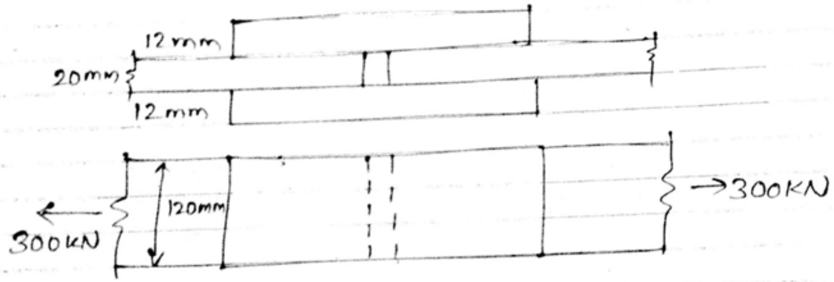
$A_{vg} = (80+30) \times 12 \times 2 = 2640$
$A_{vn} = (80+30-1.5 \times 18) \times 2 \times 12 = 1992$
$A_{tg} = 60 \times 12 = 720$
$A_{tn} = (60-18) \times 12 = 504$

$\therefore$ $T_{db} = 495.19$ kN

$\Rightarrow$ Ultimate load carrying capacity of Joint
$= \underline{\underline{148.586 \text{ kN}}}$

Shear failure of bolt controls.

(Q) Design a double cover plate butt joint to transmit a working load of 300 kN to connect two flats 120 mm wide & 20 mm thick using M16 high strength bolts of property class 10.9 if
(a) Slip is not permitted at working load.
(b) Slip is not " " ultimate load.
The cover plates are 12 mm thick. Assume that one shear plane intersects the threads of the bolt.



Soln:-

$f_{ub} = 1000$ MPa
$f_{yb} = 0.9 \times 1000 = 900$ MPa

$d_0 = 16 + 2 = 18$ mm.

(a) Slip is not permitted at the working load
This means that a slip between the connecting parts is tolerated at the ultimate load. Therefore the joint is designed for no slip connection at the working loads (300 kN) but should be checked for shear & bearing at the ultimate load.

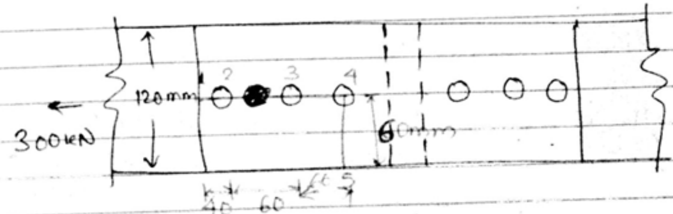
$V_{nsf} = \mu_f \cdot n_e \cdot K_h \cdot F_0 = \cancel{120757.8}$ 120890 N

where, $\mu_f = 0.55$
$n_e = 2$
$K_h = 1.0$
$F_0 = A_{nb} \cdot f_0 = 0.78 \times \frac{\pi}{4} \times 16^2 \times 0.70 f_{ub}$
$= 109779.8137$

$V_{dsf} = \frac{V_{nsf}}{\gamma_{mf}} = \frac{120890}{1.1} = 109900$ N (for 1 Bolt)

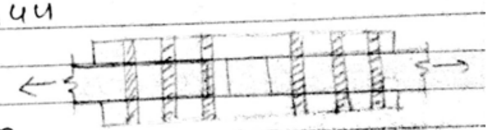
No. of bolt needed $= \frac{300 \times 10^3}{109900} = 2.73 \approx 3$ nos

The joint should be checked for bearing, shear & block shear at ultimate load.



$e_{min} = 1.7 d_0 = 30.6$
$e_{max} = 12t \left(\frac{250}{f_y}\right)^{1/2} = 144$

$P_{min} = 2.5d = 40$



$P_{max} = 16t = 192$ mm
Take $e = 40$ mm & $P = 60$ mm

⇒ Shear capacity of bolt $V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}}$

$$= \frac{f_u(n_n A_{nb} + n_s A_{sb})}{\sqrt{3}\,\gamma_{mb}}$$

$$= \frac{1000(1\times201 + 1\times157)}{\sqrt{3}\times1.25}$$

$$= 165.35\ kN$$

⇒ Bearing capacity of Bolt

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

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

$$= \frac{40}{3\times18},\ \frac{60}{3\times18} - 0.25,\ \frac{1000}{410},\ 1$$

$$= 0.7407,\ 0.8611,\ 2.44,\ 1$$

$\therefore\ K_b = 0.7407$

$$V_{dpb} = \frac{2.5\times0.7407\times16\times20\times410}{1.25}$$

$$= 194.36\ kN$$

Design strength of a bolt = 165.35 kN.

Design strength of 3 bolts in ultimate load = 496.05 kN $> 300\times1.5$ $> 450\ kN$ (ok)

⇒ Check for block shear.

$A_{vg} = (40+60+60)\times20 = 3200\ mm^2$ (along 1-2-3-4)

$A_{vn} = (160 - 2.5\times18)\times20 = 2300\ mm^2$ (along 1-2-3-4)

$A_{tg} = 60\times20 = 1200\ mm^2$ (along 4-5)

$A_{tn} = (60 - \frac{18}{2})\times20 = 1020\ mm^2$ (along 4-5)

$$\frac{A_{vg}f_y}{\gamma_{m0}\sqrt{3}} + \frac{0.9A_{tn}f_u}{\gamma_{m1}} = \frac{3200\times250}{1.1\times\sqrt{3}} + \frac{0.9\times1020\times410}{1.25}$$

$$= 720995.1\ N$$

$$\frac{0.9A_{vn}f_u}{\gamma_{m1}\sqrt{3}} + \frac{A_{tg}f_y}{\gamma_{m0}} = \frac{0.9\times2300\times410}{1.25\times\sqrt{3}} + \frac{1200\times250}{1.1}$$

$$= 664725\ N$$

$T_{db} = 664725\ N = 664.725\ kN > 450\ kN$ (ok).

Hence, 3 bolts may be provided.

(b) Slip is not permitted at ultimate load

$V_{nsf} = 120890\ N$

Design frictional force produced by a bolt

$$V_{dsf} = \frac{V_{nsf}}{\gamma_{mf}} = \frac{120890}{1.25} = 96712\ N$$

$$\text{No. of bolts needed} = \frac{1.5\times300\times10^3}{96712} = 4.65$$

Five bolts may be provided.

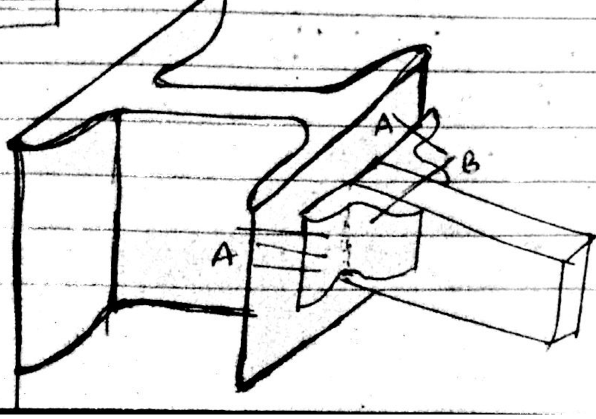
Design of Eccentric Bolted Connection

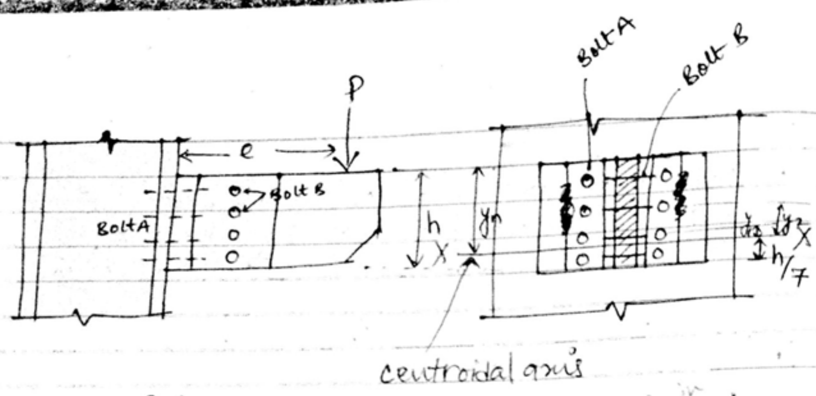
→ Bolted connections subjected to combined actions such as shear, ~~tor~~ twisting or bending are considered in this type of eccentric connection.

→ Types of eccentric bolted connections
(i) Bracket Connections
(ii) Beam-to-column end connections
(iii) Splicing ~~of~~ beams.

(i) Bracket Connections:- Brackets are used to support beams at some eccentricity from the ~~com~~ column face. Brackets are connected to the flange of a column in either of following ways:-
(a) Perpendicular to flange of column
(b) ~~Parall~~ Bracket connected in the plane of flange of a column.

(a) When Bracket is connected at right angle to the flange of column.





⇒ Bolt A (Direct shear & Bending Moment) ⇒ Tension in bolts
Bolt B (Direct shear & twisting)

⇒ Bracket is connected to the column with the help of angle section and it is assumed that depth of neutral axis ~~ex~~ $(NA) = \frac{h}{7}$ from the bottom of the bracket.

⇒ X - X is centroidal axis → Neutral axis.

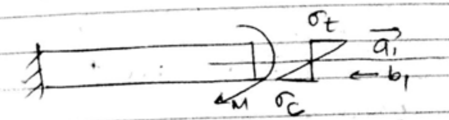
(1) Factored moment $M = P.e$ is resisted by the tensions developed in the bolts above the N.A.
(2) For equilibrium, the moments of tension in the bolts above the NA should be equal to P.e.

$$P.e = 2T_1 y_1 + 2T_2 y_2 + \dots + 2T_n y_n \quad \text{—— (1)}$$

$T_1, T_2 \dots T_n$ = Tensions in each of the bolts
$y_1, y_2 \dots y_n$ = distances from the NA to the respective bolts.

Assumptions : The tension in a bolt is proportional to the distance $(y_1, y_2 \dots y_n)$ from the neutral axis.

$$\frac{T_1}{y_1} = \frac{T_2}{y_2} = \dots = \frac{T_n}{y_n}$$



$M = Ta_1 + Cb_1$

Then,
$$T_1 = \frac{y_1}{y_n} T_n$$
$$y_2 = \frac{y_2}{y_n} T_n$$
$$T_n = \frac{y_n}{y_n} T_n \quad \text{—— (2)}$$

Substituting in eqn (1) yields.
$$\frac{2y_1^2}{y_n} T_n + \frac{2y_2^2}{y_n} T_n + \dots + \frac{2y_n^2}{y_n} T_n = P.e.$$

or, $$\frac{T_n}{y_n}\left[2y_1^2 + 2y_2^2 + \dots + 2y_n^2\right] = P.e.$$

or, $$\frac{2T_n}{y_n} \Sigma y_i^2 = P.e.$$

or, $$\boxed{T_n = \frac{P.e.y_n}{2\Sigma y_i^2}}$$ ⇒ Max. tension developed in bolt of topmost row.

Shear force in each bolt due to direct shear
$$V_{sb} = \frac{P}{N} \quad (N = \text{No. of bolts})$$

Bolt subjected to tension & shear should satisfy the interaction eq^n

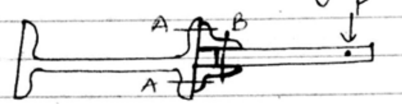
$$\left(\frac{V_{sb}}{V_{db}}\right)^2 + \left(\frac{T_b}{T_{db}}\right)^2 \le 1$$

Approximate no. of bolt in a single vertical line

$$n = \sqrt{\frac{6M}{m\,p\,T_{db}}}$$

where
m = no. of vertical line
p = pitch of bolt
$M = P.e$

(Q) Design a connection for a bracket using M20 bolts of product grade & property class 4.6 to carry factored vertical load of 200 kN acting at 300 mm from the face of the column. 12 mm thick bracket is connected by 2 ∠100 100 X 10. The grade of steel is E250.



Given :-
Dia. of bolt (d) = 20 mm
f_{ub} = 400 MPa
f_{yb} = 240 MPa
P = 200 kN
e = 300 mm
$M = P \cdot e = 60$ kNm

DESIGN OF BOLT 'A' :-
Bolts 'A' are subjected to SINGLE SHEAR & BENDING MOMENT.

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

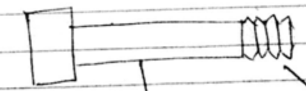
Assume, bolt threads are outside shear plane.

$$V_{nsb} = \frac{400}{\sqrt{3}}\left(0 \times A_{nb} + 1 \times 314\right) = 72.52 \text{ kN}$$

$$V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}} = \frac{72.52}{1.25} = 58 \text{kN}$$

Design tension capacity of a bolt

T_{db} = Least of T_{dbg} or T_{dbn}.



T_{dbn} = Design strength of a bolt due to rupture at the net section (i.e. at root of thread)

$$T_{dbn} = \frac{0.9\,A_{nb}\,f_{ub}}{\gamma_{mb}} = \frac{0.9 \times 245 \times 400}{1.25 \cancel{\times 10}} = 70.56 \text{kN}$$

T_{dbg} = Design strength of a bolt due to yielding of a gross section (i.e. at the shank).

$$T_{dbg} = \frac{A_{sb} f_{yb}}{\gamma_{mo}} = \frac{314 \times 240}{1.1} = 68.5 \text{ kN}$$

$\therefore T_{db} = 68.5$ kN.

Let n = approx. no. of bolts required in a vertical line.

$$n = \sqrt{\frac{6M}{mp\,T_{db}}}$$

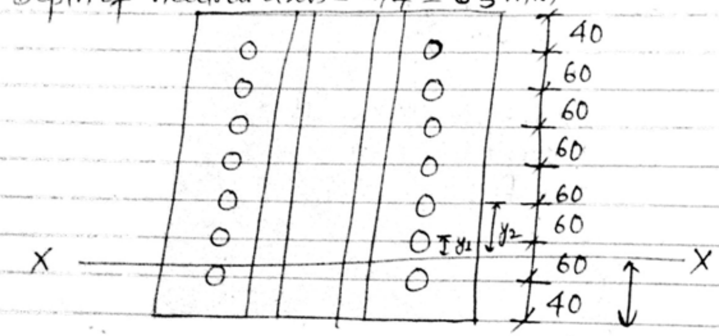
Take
e = 40mm
p = 60mm

$$= \sqrt{\frac{6 \times 60 \times 10^6}{2 \times 60 \times 68.5 \times 10^3}}$$

$= 6.62 \simeq 7$ nos. in a line.

Total no. of bolts $N = 2 \times 7 = 14$

Depth of bracket, $h = 2 \times 40 + 6 \times 60 = 440$mm
Depth of neutral axis $= h/7 = 63$ mm



$y_1 = 100 - 63 = 37$
$y_2 = 37 + 60 = 97$
$y_3 = 97 + 60 = 157$
$y_4 = 157 + 60 = 217$
$y_5 = 217 + 60 = 277$
$y_6 = y_n = 337$ mm.

Max. tension developed in the bolts of topmost row

$$T_n = \frac{P e\, y_n}{2 \Sigma y_i^2} = \frac{60 \times 10^6 \times 337}{545628}$$

$\therefore T_b = T_n = 37.06$ kN

$$V_{sb} = \frac{P}{N} = \frac{200 \times 10^3}{14} = 14.286 \text{ kN}.$$

↑ factored SF acting on a bolt.

Now,

$$\left(\frac{V_{sb}}{V_{db}}\right)^2 + \left(\frac{T_b}{T_{db}}\right)^2 \le 1.0$$

$$\left(\frac{14.286}{58}\right)^2 + \left(\frac{37.06}{68.5}\right)^2 = 0.353 < 1.0$$

(ok).

Oversafe

(0.8 to 0.9 is suitable).

So, the no. of bolts may be reduced to 12 (6 in each row/line) and the calculation is repeated.

$h = 2\times40+5\times60 = 380\,mm$

$h/7 = 54.286\,mm \simeq 54\,mm$

$y_1 = 100-54 = 46\,mm$

$y_2 = 46+60 = 106\,mm$

$y_3 = 166\,mm$

$y_4 = 226\,mm$

$y_5 = y_n = 286\,mm$

$$T_n = \frac{P.e.y_n}{2\sum y_i^2} = 49.37\,kN$$

$$V_{sb} = \frac{P}{N} = \frac{200}{12} = \underline{\underline{16.67\,kN}}.$$

$$\left(\frac{V_{sb}}{V_{db}}\right)^2 + \left(\frac{T_b}{T_{db}}\right)^2 \le 1.0$$

or, $\left(\frac{16.67}{58}\right)^2 + \left(\frac{49.37}{68.5}\right)^2 = 0.6 < 1.0$ (OK)

⇒ Check for Bearing

$$K_b = \frac{e}{3d_0}, \left(\frac{p}{3d_0} - 0.25\right), \frac{f_{ub}}{f_u}, 1.0$$

$$= \frac{40}{3\times22}, \frac{60}{3\times22} - 0.25, \frac{400}{410}, 1.0$$

$$= 0.606,\ 0.659,\ 0.97,\ 1.0$$

$\therefore k_b = 0.606$

$$V_{dpb} = \frac{2.5\times k_b.d.t.f_u}{\gamma_{mb}}$$

Bearing on angle

$$= \frac{2.5\times0.606\times20\times10\times410}{1.25} = 99.384\,kN$$

$> V_{sb}$ (16.67 kN)

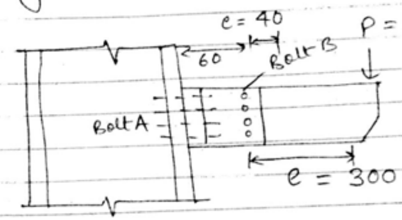
(OK)

DESIGN OF BOLTS 'B':

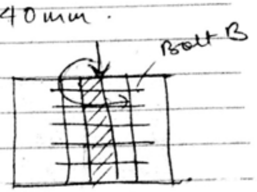
These bolts are subjected to DIRECT SHEAR & TWISTING (TORSION)

Twisting moment $= 200\times240 = 48\,kNm$

$\left(\begin{matrix}100-40\\=60\end{matrix}\right)$



$e = 300-60 = 240\,mm$.



As these bolts are in double shear,

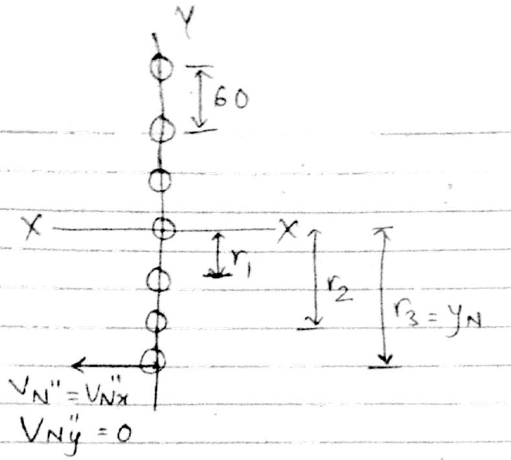
$V_{dsb} = 2\times58 = 116\,kN$

Since there is only one vertical line $m = 1$.

$$n = \sqrt{\frac{6M}{m\,p\,V_{db}}}$$

$$= \sqrt{\frac{6\times48\times10^6}{1\times60\times116\times10^3}} = 6.43 \simeq 7.$$

So, 7 Bolts may be provided.



$r_1 = 60$
$r_2 = 120$
$r_3 = 180$

$$V_N'' = \frac{P \cdot e \cdot r_n}{\Sigma r_i^2}$$

$$\Sigma r_i^2 = 2(r_1^2 + r_2^2 + r_3^2) = 100800 \text{ mm}^2$$

$$V_{Nx} = V_{Nx}'' = \frac{P \cdot e \cdot y_n}{\Sigma r_i^2}$$
$$= \frac{48 \times 10^6 \times 180}{100800}$$
$$= 85.71 \text{ kN}$$

$$V_{Ny} = V' + V_{Ny}''$$
$$= \frac{P}{N} + \frac{P \cdot e \cdot x_N}{\Sigma r_i^2} \qquad [V_{Ny}'' = 0]$$
$$= \frac{P}{N} + 0$$
$$= \frac{200}{7}$$
$$= 28.57 \text{ kN}$$

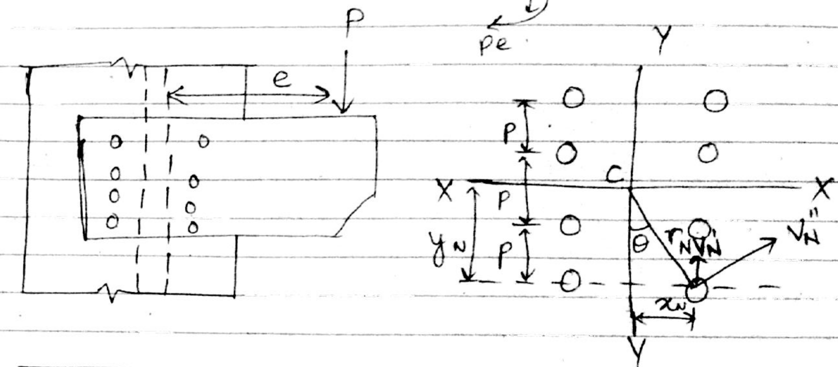
$$V_N = \sqrt{85.71^2 + 28.57^2} = 90.35 \text{ kN} < V_{db} \ (116 \text{ kN})$$

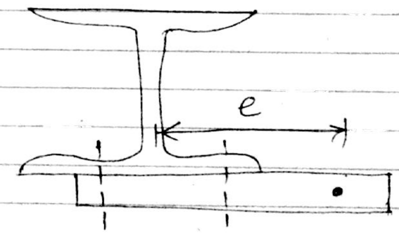
(OK)

Check for Bearing

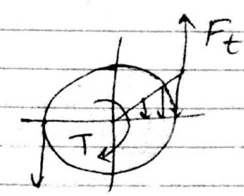
$$V_{dpb} = \frac{2.5 \, k_b \, d \, t \, f_u}{\gamma_{mb}}$$
$$= \frac{2.5 \times 0.606 \times 20 \times 12 \times 410}{1.25}$$
$$= 119.26 \text{ kN} > 90.35 \text{ kN} \ \text{(OK)}$$

(b) Bracket connected in the plane of flange of column.





⇒ Bolts are subjected to direct shear & torsion.



$$\tau = \frac{T}{I_p} \times R$$

Direct shear is equally resisted by all bolts so.

$$\boxed{V' = \frac{P}{N}} \quad (\uparrow)$$

Twisting moment $M = P.e.$ is resisted by the shear forces V_1'', V_2'', ..., V_N'' developed in the bolt. If the applied moment is clockwise, then the ~~clockwise~~ direction of these shear forces are such that they produce an anticlockwise moment. So for equilibrium,

$$V_1''r_1 + V_2''r_2 + \dots + V_N''r_N = M = P.e. \quad (1)$$

It is assumed that the shear forces V_1'', V_2'' ... V_N'' are proportional to their distances $r_1, r_2 \dots r_N$ respectively from the CG of bolt.

$$\frac{V_1''}{r_1} = \frac{V_2''}{r_2} = \dots = \frac{V_N''}{r_N}$$

or,
$$V_1'' = \frac{r_1}{r_N} V_N''$$
$$V_2'' = \frac{r_2}{r_N} V_N''$$
$$V_N'' = \frac{r_N}{r_N} V_N''$$

V_N'' = Max S.F. in the extreme bolt due to twisting moment Pe.

Substituting in (1) we get

$$\frac{r_1^2}{r_N} V_N'' + \frac{r_2^2}{r_N} V_N'' + \dots + \frac{r_N^2}{r_N} V_N'' = P.e.$$

or,
$$\frac{V_N''}{r_N} (\Sigma r_i^2) = Pe.$$

or,
$$V_N'' = \frac{P.e\,r_N}{\Sigma r_i^2}$$

$$V_{Nx}'' = V_N'' \cos\theta = \frac{P.e.r_N}{\Sigma r_i^2} \cdot \frac{y_N}{r_N} = \frac{P.e.y_N}{\Sigma r_i^2}$$

$$V_{Ny}'' = V_N'' \sin\theta = \frac{P.e.r_N}{\Sigma r_i^2} \cdot \frac{x_N}{r_N} = \frac{P.e.x_N}{\Sigma r_i^2}.$$

$$V_{Nx} = V_{Nx}'' = \frac{P.e.y_N}{\Sigma r_i^2}$$

$$V_{Ny} = V' + V_{Ny}'' = \frac{P}{N} + \frac{P.e.x_N}{\Sigma r_i^2}$$

V_N = Max. Resultant SF in the extreme bolt

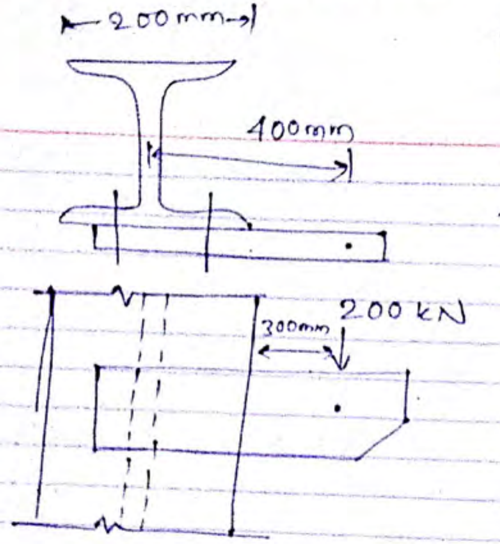
$$V_N = \sqrt{V_{Nx}^2 + V_{Ny}^2} \le V_{db}$$

V_{db} = least of V_{dsb} & V_{dpb}

Approx. no. of bolts $n = \sqrt{\dfrac{6M}{m p\, V_{dsb}}}$

m = No. of vertical lines of bolt.

(Q) Design a connection for bracket using high strength M20 bolts a property class 8.8 if the bracket is connected in the plane of the flange. The width of column is 200mm. The bracket has to carry a factored vertical load of 200kN. The eccentricity of load is 300mm from the edge of the column flange. Also design the connection if
(a) No slip is permitted at the ultimate load.
(b) A slip is permitted at the ultimate load but not a working load.



Dia of bolts $d = 20\,mm$

$d_0 = 20+2 = 22\,mm$

$f_{ub} = 800\,MPa$

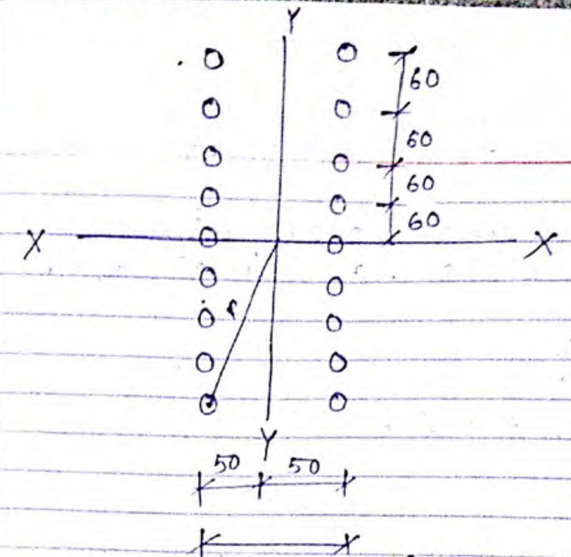
$f_{yb} = 640\,MPa$

$e = 300+100 = 400\,mm$

$M = P.e$

$= 200 \times 0.4$

$= 80\,kNm$



$g = 100\,mm$ for HB200 section.

(a) Slip is not permitted at ultimate load

$V_{nsf} = \mu_f . n_e . K_h . F_o$

$= 0.55 \times 1 \times 1 \times 137200$

$= 75.46\,kN$

$F_o = A_{nb}\, f_o$

$= 0.78 \times \frac{\pi}{4} \times 20^2 \times 0.7 \times 800$

$= 245 \times 0.7 \times 800$

$= 137200$

$V_{dsf} = \frac{V_{nsf}}{\gamma_{mf}} = \frac{75.46}{1.25}$

$= 60.37\,kN$

$n = \sqrt{\frac{6\,M}{m\,p\,V_{dsf}}} = \sqrt{\frac{6 \times 80 \times 10^6}{2 \times 60 \times 60.37 \times 10^3}} = 8.14 \simeq 9$

Hence 18 bolts may be provided.

$\Sigma r_i^2 = 4\left[(50^2+240^2)+(50^2+180^2)+(50^2+120^2)+(50^2+60^2)\right]$

$= 472000\,mm^2$

$V_{Nx} = \frac{P.e.y_N}{\Sigma r_i^2} = \frac{80 \times 10^6 \times 240}{472000} = 40.68\,kN$

$V_{Ny} = \frac{P}{N} + \frac{P.e.x_N}{\Sigma r_i^2} = \frac{200}{18} + \frac{80 \times 10^6 \times 50}{472000} = 19.6\,kN$

$V_N = \sqrt{V_{Nx}^2 + V_{Ny}^2} = 45.15\,kN < 60.37\,kN$

(OK).

(b) Slip is permitted at ultimate load but not at working load.

If slip is permitted at Ultimate load, bolts are designed for bearing & shear at ultimate load.

⇒ Shear capacity of bolt

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

$$= \frac{800}{\sqrt{3}}(314)$$

$$= 145 \text{ kN}$$

$$V_{dsb} = \frac{145}{1.25} = 116 \text{ kN}$$

Take $e = 40$ mm, $p = 60$ mm

⇒ Bearing capacity of bolt.

$$K_b = \frac{e}{3d_0}, \frac{p}{3d_0} - 0.25, \frac{f_{ub}}{f_u}, 1.0$$

$$= \frac{40}{3 \times 22}, \frac{60}{3 \times 22} - 0.25, \frac{800}{410}, 1.0$$

$$= 0.606, 0.659, 1.95, 1.0$$

$$K_b = 0.606$$

$$V_{dpb} = \frac{2.5\, K_b \cdot d \cdot t \cdot f_u}{\gamma_{mb}}$$ (Take thickness of bracket = 12 mm)

$$= \frac{2.5 \times 0.606 \times 20 \times 12 \times 410}{1.25}$$

$$= 119.26 \text{ kN}$$

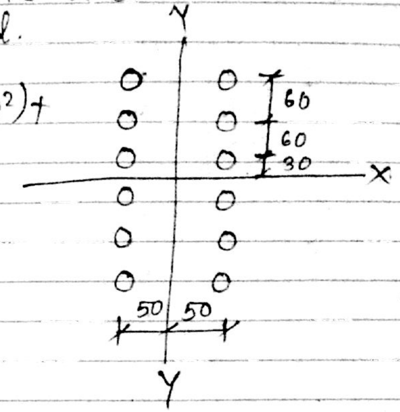
$$\therefore \boxed{V_{db} = 116 \text{ kN}}$$

No. of bolts in a line $n = \sqrt{\frac{6M}{m p V_{db}}}$

$$= \sqrt{\frac{6 \times 80 \times 10^6}{2 \times 60 \times 116 \times 10^3}}$$

$$= 5.9 \simeq 6$$

Twelve bolts may be provided.



$$\Sigma r_i^2 = 4\left[(50^2+30^2)+(50^2+90^2)+(50^2+150^2)\right]$$

$$= 156000$$

$$V_{Nx} = \frac{P \cdot e \cdot y_N}{\Sigma r_i^2}$$

$$= \frac{80 \times 10^6 \times 150}{156000 \times 10^3}$$

$$= 77 \text{ kN}$$

$$V_{NY} = \frac{P}{N} + \frac{P e x_N}{\Sigma r_i^2}$$

$$= \frac{200}{12} + \frac{80 \times 10^3 \times 50}{156000} = 42.3 \text{ kN}$$

$$\therefore V_N = 87.85 \text{ kN} < 116 \text{ kN} \quad (\underline{OK}).$$

The provided should be checked for no-slip at the working load.

$$V_{dsf} = \frac{V_{nsf}}{\gamma_{mf}} = \frac{75.46}{1.1} = 68.6 \text{ kN}.$$

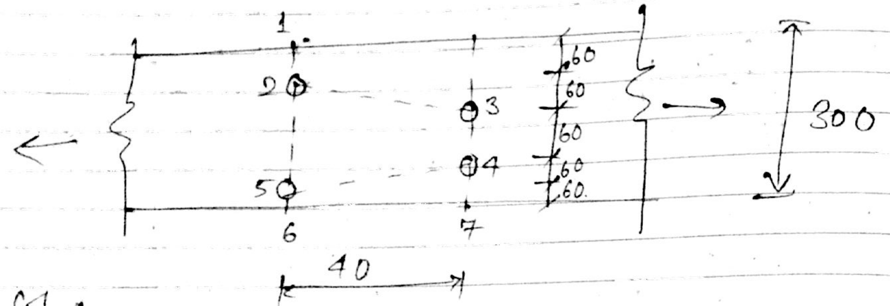
$$\text{Working load} = \frac{200}{1.5} = 133 \text{ kN}$$

$$V_{Nx} = \frac{133 \times 400 \times 150}{156000} = 51 \text{ kN}$$

$$V_{Ny} = \frac{133}{12} + \frac{133 \times 40 \times 50}{156000} = 28 \text{ kN}$$

$$V_N = 58.18 \text{ kN} < 68.6 \text{ kN} \quad (\underline{OK})$$

Q) Determine min. net area of c/s. of 300mm × 12mm shown in fig below. The dia. of the holes are 17.5 mm



Soln:-

Along 1-2-5-6

$$b_e = b - nd$$
$$= 300 - 2 \times 17.5$$
$$= 265\,mm$$
$$A_{net} = b_e \times t$$
$$= 265 \times 12 = 3180\,mm^2$$

Along 1-2-3-4-7

$$b_e = b - nd + \sum_i \frac{p_i^2}{4g_i}$$
$$= 300 - 3 \times 17.5 + \frac{40^2}{4 \times 60}$$
$$= 254\,mm$$
$$A_{net} = 254 \times 12 = 3048\,mm^2$$

Along 1-2-3-4-5-6

$$b_e = b - nd + \frac{\sum p_i^2}{4g_i}$$
$$= 300 - 4 \times 17.5 + \frac{40^2}{4 \times 60} + \frac{40^2}{4 \times 60}$$
$$= 243\,mm$$
$$A_{net} = 243 \times 12$$
$$= 2916\,mm^2$$

$\therefore$ $A_{min} = 2916\,mm^2$ (Critical Section).

Empirical eqn by V.H. Cochrane.

→ Along inclined/diagonal line there is a combination of direct tensile stress & shear stress. and thus a smaller area should be used.

$$\text{Deduction} = \text{Holes c/s area} - \frac{\sum p^2 t}{4g}$$

→ $\frac{p^2 t}{4g}$ for each line extending betn holes at others than right angles to the stress dirn. The chain of lines should be choosen so as to produce max. deductions.

→ $\frac{p^2}{4g}$ is merely an approximation or simplification of complex stress variations that occur in the member with staggered arrangement of bolts.

30th May 2018 Wed

CHAPTER : 5 TENSION MEMBER

→ Members carrying a tensile force is called tension member.

→ Tension members may be wires, cables, bars & rod & flat plates.

→ Tension members may fail by
(a) Yielding of gross - section
(b) Rupture of net section
(c) Block shear failure

(a) Yielding of gross-section :- It means considerable deformation of tension member before fracture.
Design strength of tension member due to yielding

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

(b) Rupture of net-section :- Rupture of tension member occurs when tension member reaches the ultimate stress at net section.

For Plates :-

$$T_{dn} = \frac{0.9 A_n f_u}{\gamma_{m1}}$$

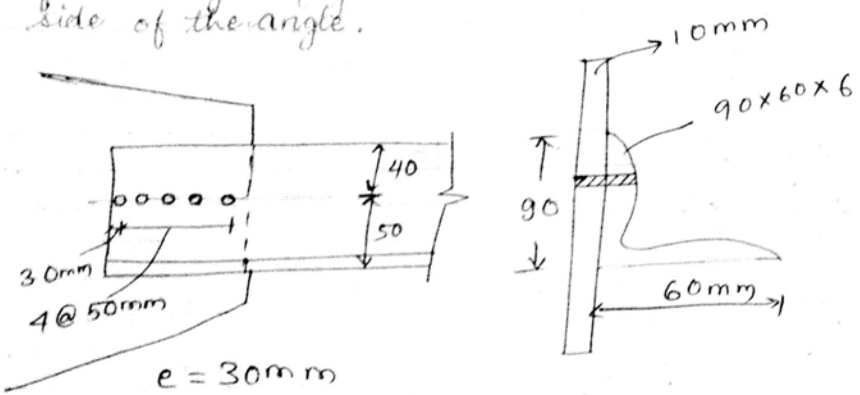
For Threaded rod :-

$$T_{dn} = 0.9 A_n \frac{f_u}{\gamma_{m1}}$$

For single angles :-

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

(Q3) A single unequal angle ISA $90 \times 60 \times 6$ mm is connected to a 10mm thick gusset plate at the end with 5 no. of 16mm bolts to transfer tension. Determine the design tensile strength of the angle if gusset is connected to longer side of the angle.



$e = 30$ mm
$p = 50$ mm

Soln :-

Check → Yielding
→ Rupture
→ Block failure.

(a) Design Strength due to yielding.

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

$$A_g = \left(90 - \frac{6}{2}\right) \times 6 + \left(60 - \frac{6}{2}\right) \times 6$$

$$= 864 \text{ mm}^2$$

from code $A_g = 865 \text{ mm}^2$

Assume Fe 410 ; $f_y = 250$ MPa.

$$T_{dg} = \frac{864 \times 250}{1.1} = 196.36 \text{ kN}$$ #

(b) Design Strength due to Rupture.

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

A_{nc} = Area of connected leg

$$= \left(90 - d_o - \frac{t}{2}\right) \times t$$

$$= (90 - 18 - 3) \times 6 = 414 \text{ mm}^2$$

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

$\therefore$ ~~Tdn~~ =

$$\beta = 1.4 - 0.076 \times \frac{w}{t} \times \frac{f_y}{f_u} \times \frac{b_s}{L_c} \le \frac{f_u \cdot \gamma_{mo}}{f_y \cdot \gamma_{m1}}$$

$$\ge 0.7$$

$w = 60$
$t = 6$
$b_s = w + w_1 - t = 60 + 50 - 6 = 104$ mm
$L_c = 4 \times 50 = 200$ mm

$\therefore \boxed{\beta = 1.16}$ < 1.44, > 0.7 (ok).

$\therefore T_{dn} = 212.38$ kN #

(c) Design strength due to block shear.

$A_{vg} = (4 \times 50 + 30) \times 6 = 1380 \text{ mm}^2$

$A_{vn} = (4 \times 50 + 30 - 4.5 \times 18) \times 6 = 894 \text{ mm}^2$

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

$A_{tn} = (40 - 0.5 \times 18) \times 6 = 186 \text{ mm}^2$

$$T_{db} = \left.\begin{array}{l} 235.98 \text{ kN} \\ \text{OR} \\ 206.91 \text{ kN} \end{array}\right\}.$$

$\therefore \boxed{T_{db} = 206.91 \text{ kN}}$

$\therefore$ Design Strength of Angle Section $= 196.36$ kN

(Q4) Find the ultimate design strength of angle $100 \times 100 \times 10$ in tension which is connected to gusset 12 mm thick using M20 bolt of property class 4.6 in a single line. Assume the bolt threads are outside the shear plane. The yield & ultimate strengths are 250 MPa & 410 MPa respectively.

Soln:-

Steps

1). Design strength of angle due to yielding
$T_{dg} = 431.82$ kN.

2) Design strength of bolt $\left.\begin{array}{l} \to \text{Shear} \\ \hookrightarrow \text{Bearing} \end{array}\right\} = 58.04$ kN

No. of bolts $= 8$.

3) Design strength of angle in rupture

4) Design strength for Block shear failure.

5) Ultimate Design strength $\Rightarrow$ Min of 1, 3, 4.

(Q5) Design tension member carrying a service load of 300 kN using
(a) Single angle section with bolted & welded end connection separately.
(b) Double angle section with bolted & welded end connection separately.

Design of Single angle section with Bolted connection

Gross-Sectional area $A_g = \dfrac{T \cdot \gamma_{m0}}{f_y}$

$= \dfrac{(1.5 \times 300) \times 10^3 \times 1.1}{250}$

$= 1980 \text{ mm}^2$

Take ISA $150 \times 115 \times 10$ mm

Area $= 20.1 \text{ cm}^2 = 2010 \text{ mm}^2 > 1980 \text{ mm}^2$.

Design of end Connection

Take M20 Bolts.

$d = 20$ mm

$d_0 = 22$ mm

Shear capacity of bolt $V_{dsb} = \dfrac{f_u}{\sqrt{3}\,\gamma_{mb}} (n_n A_{nb} + n_s A_{sb})$

Assume threads of bolt lie in shear plane & Class of 4.6

$n_n = 1$

$n_s = 0$

$A_{nb} = \dfrac{\pi d^2}{4} \times 0.78 = 245.04 \text{ mm}^2$

$\therefore V_{dsb} = 45.27$ kN

Bearing strength of Bolts.

$V_{dpb} = \frac{2.5 K_b \cdot d \cdot t \cdot f_u}{\gamma_{mb}} = \frac{2.5 \times 0.606 \times 20 \times 10 \times 400}{1.25}$

$K_b = \frac{e}{3d_0}, \frac{p}{3d_0} - 0.25, \frac{f_{ub}}{f_u}, 1.0$

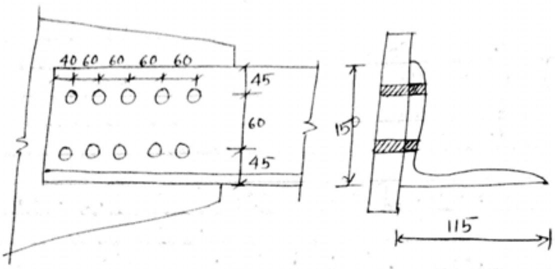
Assume, $e = 40mm$, $p = 60mm$

$K_b = 0.606$

$\therefore V_{dpb} = 96.96$ kN

$\therefore$ Strength of bolt = 45.27 kN.

No. of Bolts required = $\frac{1.5 \times 300}{45.27} = 9.94 \approx 10$ nos.



Check :-

Strength due to yielding $T_{dg} = A_g \frac{f_y}{\gamma_{m0}}$

$= \frac{2010 \times 250}{1.1}$

$= 456.82$ kN $> (1.5 \times 300)$
> 450 kN
(OK)

Strength due to rupture :-

Strength due to Block shear :-

Design of Single Angle Section with Welded Connection

Design Load = 450 kN

Gross-sectional Area $A_g = \frac{T \cdot \gamma_{m0}}{f_y} = 1980 mm^2$

Take $\angle$ 150 115 x 10 mm

Fe 410, $f_y = 250$ MPa

Design strength of weld $P_u = \frac{f_u}{\sqrt{3}\, \gamma_{mw}} \cdot l_w \cdot t_w$

$\gamma_{mw} = 1.25$ (Assume shop weld)

$t_w = k \times s = 0.7 \times s$

Size of weld (s) $\Rightarrow$ $s_{min} = 5mm$

$s_{max} = \frac{3}{4} \times 10 = 7.5mm$

Provide $s = 6mm$

$t_w = 0.7 \times 6 = 4.2mm$

Now, $450 \times 10^3 = \frac{410}{\sqrt{3} \times 1.25} \times l_w \times 4.2$

$\therefore l_w = 565.78mm$

Provide $l_w = 600mm$

Check for yielding :-

$T_{dg} = A_g \frac{f_y}{\gamma_{m0}} = \frac{2010 \times 250}{1.1} = 456.8$ kN
> 450 kN
(OK)

Check for Rupture :-

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

$\beta = 1.4 - 0.076 \frac{w}{t} \times \frac{f_y}{f_u} \times \frac{b_s}{l_c} \leq \frac{f_u \gamma_{m0}}{f_y \gamma_{m1}}$

≥ 0.7

https://civinnovate.com/civil-engineering-notes/

$w = 115 \text{ mm}$
$t = 10 \text{ mm}$
$b_s = w = 115 \text{ mm}$
$l_c = 600 \text{ mm}$
$\beta = 1.297 < 1.44$
> 0.7 (ok)

$$A_{nc} = \left(150 - \frac{10}{2}\right) \times 10 = 1450 \text{ mm}^2$$

$$A_{go} = \left(115 - \frac{10}{2}\right) \times 10 = 1100 \text{ mm}^2$$

$\therefore T_{dn} = 752.29 > 450 \text{ kN}$

Design of Double angle section

Gross-sectional area $A_g = 1980 \text{ mm}^2$
Area of each angle $= 990 \text{ mm}^2$
Take 2 ∠ $70 \times 70 \times 8$ mm
Total area $= 2 \times 1060 = 2120 \text{ mm}^2 > 1980 \text{ mm}^2$

Assuming angles are provided at one side of gusset plate

Use M20 Bolts of property class 4.6.
$d = 20$ mm
$d_0 = 22$ mm
$f_{ub} = 400$ MPa.

→ Shear strength of bolt

$$V_{dsb} = \frac{f_{ub}}{\sqrt{3}\,\gamma_{mb}} (n_n A_{nb} + n_s A_{sb})$$

Assume threads of bolt lies in shear plane
$n_n = 1$ & $n_s = 0$

$$V_{dsb} = \frac{400}{\sqrt{3} \times 1.25}\left(1 \times 0.78 \times \frac{\pi}{4}(20)^2\right)$$

$= 45.27$ kN.

→ Bearing capacity of bolt :-

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

$$K_b = \frac{e}{3d_0},\ \frac{p}{3d_0} - 0.25,\ \frac{f_{ub}}{f_u},\ 1.0.$$

Min $p = 2.5d = 2.5 \times 20 = 50$ mm
Max $p = 16t = 16 \times 8$ or 200 mm $= 128$ mm.
Min $e = 1.7 d_0 = 37.4$
Max $e = 12t\sqrt{\frac{250}{f_y}} = 12 \times 8 = 96$ mm

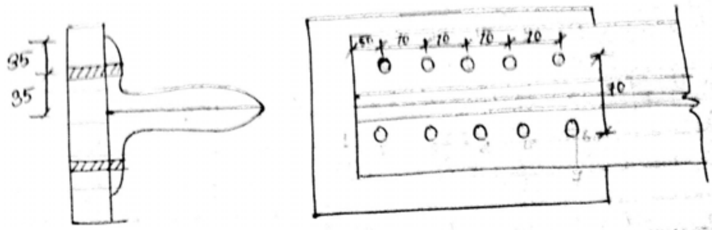
Take $p = 70$ mm, $e = 50$ mm

$K_b = 0.76$

$\therefore V_{dpb} = 2.5 \times 0.76 \times 20 \times 8 \times \frac{410}{1.25} = 99.7$ kN

Strength of bolt $= 45.27$ kN

No. of bolts $= \frac{450}{45.27} = 9.94 \simeq 10$ Nos.



check for yielding.

$T_{dg} = \frac{A_g f_y}{\gamma_{m0}} = \frac{2120 \times 250}{1.1} = 481.8\,kN > 450\,kN$

Check for rupture.

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

$A_{nc} = \left(70 - 22 - \frac{8}{2}\right) \times 8 = 352\,mm^2$

$A_{go} = \left(70 - \frac{8}{2}\right) \times 8 = 528\,mm^2$

\# $\beta = 1.4 - 0.076 \frac{w}{t} \times \frac{f_y}{f_u} \times \frac{b_s}{L_c} \le \frac{f_u \gamma_{m0}}{f_y \gamma_{m1}}$
≥ 0.7

$w = 70$ $\quad f_u = 410$
$t = 8$ $\quad b_s = w + w_1 - t = 70 + 35 - 8 = 97$
$f_y = 250$ $\quad L_c = 70 \times 4 = 280$

$\therefore \beta = 1.26 \le 1.44$
> 0.7

$\therefore T_{dn} = 255.11\,kN \times 2 = 510.22 > 450\,kN$ (ok).

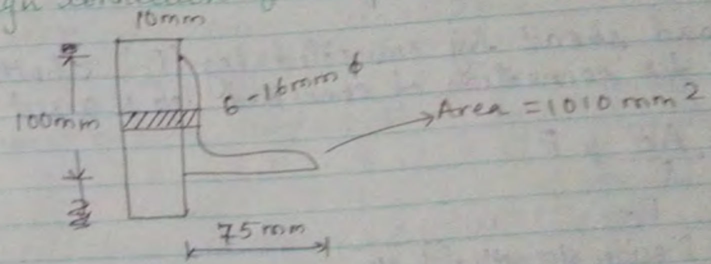
Check for Block Shear:

$A_{vg} = (70 \times 4 + 50) \times 8 = 2640$
$A_{vn} = [(70 \times 4 + 50) - 4.5 \times 22] \times 8 = 1848$
$A_{tg} = 35 \times 8 = 280$
$A_{tn} = \left(35 - \frac{22}{2}\right) \times 8 = 192$

$T_{db} = 403.1\,kN$ OR $378.6\,kN$.

$\therefore T_{db} = 378.6 \times 2 = 757.2\,kN > 450\,kN$

Q.6) A single unequal angle 100 75 X 6mm is connected to a 10mm thick gusset plate at the end with a 6-16mm dia bolts to transfer the tension. Determine design tensile strength of the angle assuming that $f_y = 250\,MPa$, $f_u = 410\,MPa$. If the gusset plate is connected to the 100mm leg & also design connection for its full capacity.



Soln:-

⇒ Design tensile strength: Min. of (a), (b), (c)

(a) Gross-section yielding :- $T_{dg} = A_g \times f_y / \gamma_{m0} = 229.55\,kN$

(b) Net-section rupture :- T_{db}
$\beta \rightarrow w = 75$; $b_s = 75 + 50 - 6$; $L_c = 5 \times$ pitch.

(c) Block shear strength :- T_{db}

⇒ Design for connection:
(a) Shear strength of bolts.
(b) Bearing " " "

Design strength of bolt = ?
No. of bolts required for full capacity (n) = ?

Design of Lug-angle

Angle section which is used in connection to reduce the length of connection so the additional angle is used along the tension member to reduce the length of joint & consequently reduce size of gusset.

Design steps:-

1) Find load shared by connected leg (P_c) & bolts required for connection of connected leg & gusset plate (n_c).

$$P_c = \frac{A_c}{A} \times P$$

$$n_c = \frac{P_c}{\text{Design strength of bolts}}$$

2) Find load shared by outstanding leg (P_o) & bolts required for connection of ~~with~~ lug angle (n_o) & with outstanding leg and lug angle with gusset plate (n_g).

$$P_o = \frac{A_o}{A_g} \times P$$

$$P_o = P - P_c$$

$$n_o = \frac{1.4\, P_o}{\text{Design strength of bolt}}$$

$$n_g = \frac{1.2\, P_o}{\text{Design strength of bolt}}$$

3) Find sectional area of lug-angle and select appropriate section for lug-angle.

$$A_g = \frac{T\, \gamma_{mo}}{f_y} = \frac{1.2\, P_o \times 1.1}{250}$$

(Q) Design a tension member to carry a factored tensile force of 450 kN. The length of connection of angle section to the gusset plate is limited to 400 mm. Take M20 bolts of grade 4.6.

Soln:-

Design load = 450 kN

M20, $d = 20\,mm$, $d_o = 22\,mm$

Length of connection $\leq 400\,mm$.

Sectional area of angle section: $A_g = \frac{T \times \gamma_{mo}}{f_y}$

$= 1980\,mm^2$

Take a trial section ISA 150×75×10 mm

$A_g = 2160\,mm^2 > 1980\,mm^2$

Design of connection

$$V_{dsb} = \frac{400}{\sqrt{3} \times 1.25}\left(1 \times 0.78 \times \frac{\pi}{4} \times 20^2\right) = 45.3\,kN$$

Take $e = 40\,mm$

$p = 50\,mm$

$$V_{dpb} = 2.5\, k_b \times d \times t \times \frac{f_u}{\gamma_{mb}} = 2.5 \times 0.507 \times 20 \times 10 \times 410/1.25 = 83.6\,kN$$

$$k_b = \frac{40}{3 \times 22}, \frac{50}{3 \times 22} - 0.25, \frac{f_{ub}}{f_u}, 1$$

$= \underline{0.606}, 0.507, 0.976, 1$

$\therefore$ Design strength of bolt = $\underline{45.3\,kN}$

No. of bolt required $= \frac{450}{45.3} = 9.94 \approx \underline{10}$

Length of connection $= 9 \times 50 + 2 \times 40 = 530\,mm > 400\,mm$.

So, provide lug angle.

$P_c = \frac{A_c}{A} \times P$

$P_c \rightarrow$ connecting angle section load
$P_o \rightarrow$ lug angle load

$A_c = \left(150 - \frac{10}{2}\right) \times 10 = 1450\,mm^2$

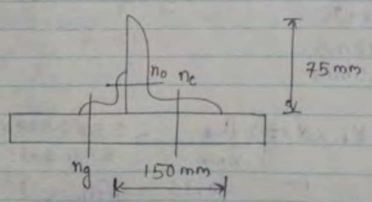
$P_c = \frac{1450}{2160} \times 450 = 302.08\,kN$

$n_c = \frac{P_c}{\text{Design strength}} = \frac{302.08}{45.3} = 6.67 \simeq 7$

$P_o = P - P_c = 450 - 302.08 = 147.92\,kN$

$n_o = \frac{1.4\,P_o}{45.3} = \frac{1.4 \times 147.92}{45.3} = 4.57 \simeq 5$

$n_g = \frac{1.2\,P_o}{45.3} = 3.92 \simeq 4$



Area of lug-angle $= 1.2\,P_o \cdot \frac{\gamma_{mo}}{f_y} = \frac{1.2 \times 147.92 \times 1.1}{250}$

$= 781\,mm^2$

Provide $\angle 70 \times 70 \times 6\,(800\,mm^2)$ as lug angle.

[Cl.10.7 Pg 82]

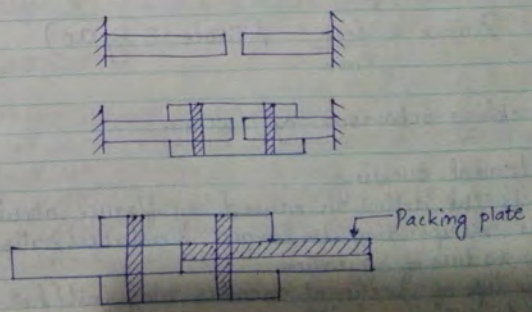
Tension Splice

When the available length is less than the required length of tension member, then splices are provided. Splice connection for tension member is usually provided in the bolted connection. As per IS specification, the splice connection should be designed for a force of atleast 0.3 times the member design capacity in tension or design action (load) whichever is more. If the sections to be spliced are not of same thickness steel packing should be provided. The design shear capacity of bolt carrying calculated shear through a packing plate in excess of 6mm should be decreased by the factor β_{pkg}

$\beta_{pkg} = (1 - 0.0125\,t_{pk})$ where t_{pk} = thickness of thicker packing in mm.

[Cl.10.3.3.3 Pg 75]

In design of tension splice the effect of eccentricity can be neglected.



CHAPTER: 6 COMPRESSION MEMBER

Classification :-

1) Long column – Buckling
2) Intermediate – Axial shortening at initial stage & then deformation in dir^n normal to axis of loading
3) Short – Crushing

Effective length :-

→ The distance between two points of contraflexure in longitudinal dir^n of column.

→ Can be calculated from the actual length of column by applying a factor that depends upon the end conditions of column provided. (Table : 11 Pg 45)

Slenderness Ratio :-

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

$$\lambda_{max} = \frac{l_{eff}}{r_{min}} \quad \text{(Table : 3 Pg 20)}$$

Buckling behaviour of Column

1. Flexural Buckling
- It is the deflection caused by flexure about the axis corresponding to largest slenderness ratio i.e. min. radius of gyration.
- Any type of sectional member may fail (hot-rolled section, built-up section)
- Valid for elastic behaviour of structure.

2. Torsional Buckling
- This type of buckling occurs in compression member that are doubly symmetric & have very slender c/s element.
- It is caused by turning of longitudinal axis.
- Torsional buckling occurs mostly in built-up section and almost never in rolled section.

3. Flexural Torsional buckling
- This type of buckling only occurs in compression members that have unsymmetrical c/s with one axis of symmetry.
- It is the result of simultaneous bending & twisting of member.
- Mostly occurs in channel sections, T-section & Double angle section.

Design of Axially Loaded Compression member

Assumptions :

1) Column is absolutely straight.
2) Modulus of Elasticity is assumed to be constant f[or] built-up column.
3) Secondary stresses (Temperature stress & settlemen[t]) are neglected which may be equal to 25 to 4[0%] of primary stresses.

Cl 7.1 (Page 34) * Assume comp stress about 80-120 MPa.

Design Steps

1) Assume value of slenderness ratio based on column height
   3 to 5 m → λ (40-60)
   > 5m → λ > 60
2) Calculate the design compressive stress (f_{cd}) using above slenderness ratio & buckling class of required section.
3) Calculate the gross-sectional area of the member using $A_g = \dfrac{\text{factored load}}{\text{Design compressive stress}}$
   $= \dfrac{P_u}{f_{cd}}$
4) Take a trial section having area greater than A_g.
5) Calculate eff. length of column as per given boundary conditions.
6) Calculate actual slenderness ratio.
   $\lambda_{actual} = \dfrac{l_{eff}}{r_{min}}$
   Compare with permissible slenderness ratio. (Table:3 Pg 20)
7) Calculate actual design compressive stress by using actual slenderness ratio & buckling class of section.
8) Calculate design capacity of section by using
   $P_d = A_e \times f_{cd} > \text{factored Load}$.

(for angle refer pg 48)

$E = 2 \times 10^5$ MPa

(Q1) Determine the load carrying capacity of a single discontinuous angle 80 X 80 X 6 mm which is used as a compression member in a roof truss if it is connected to a gusset plate by 2 bolts. The centre-to-centre distance betn the end connection is 2m. Take E250 grade.

Soln:-

For ISA 80 x 80 x 6 mm
Area $A_e = 929\ mm^2$
$r_{min} = r_{vv} = 15.6$ mm.

Cl 7.5.1.2. Pg 48

$$\lambda_e = \sqrt{k_1 + k_2 \lambda_{vv}^2 + k_3 \lambda_\phi^2}$$

$$\lambda_{vv} = \frac{\left(\frac{l}{r_{vv}}\right)}{\varepsilon\sqrt{\frac{\pi^2 E}{250}}} = \frac{\left(\frac{2000}{15.6}\right)}{1\sqrt{\frac{\pi^2 \times 2\times10^5}{250}}} = 1.44$$

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

$$\lambda_\phi = \frac{(b_1+b_2)/2t}{\varepsilon\sqrt{\frac{\pi^2 E}{250}}} = \frac{(80+80)/(2\times6)}{1\sqrt{\frac{\pi^2 \times 2\times10^5}{250}}} = 0.1$$

End connection assumed as fixed (in Roof truss).
$k_1 = 0.20$
$k_2 = 0.35$
$k_3 = 20$

Equivalent Slenderness Ratio
$\lambda_e = 1.173$

$\phi = 0.5[1+\alpha(\lambda - 0.2)+\lambda^2]$

α = Imperfection factor (Table 7, Pg 35)

$\alpha = 0.49$ (Buckling class 'c') $\because$ angle section.

$\phi = 0.5[1+0.49(1.173-0.2)+1.173^2]$
$= 1.43$

Now, Design compressive stress.

$f_{cd} = \dfrac{f_y/\gamma_{mo}}{\phi + [\phi^2 - \lambda^2]^{0.5}}$

$= \dfrac{250/1.1}{1.43+[1.43^2 - 1.173^2]^{0.5}}$

$= 101.104$ MPa.

Design capacity $= A_{e} \cdot f_{cd}$
$= 929 \times 101.104$
$= 93.93$ KN

(Q2) Design a column to support a factored load of 1050 kN. The column section taken as I-section & the effective length in the direction of X is 7m & in dir^n of Y is 5m. Use Fe 410.

Soln :-

Design of I-section.

Assume $\lambda = 100$.

Assuming buckling class in (from Table 10)

Y-dir^n → class b

X-dir^n → class a

Using Table 9(b) $f_{cdy} = 118$ MPa (for $\lambda = 100$ & $f_y = 250$ MPa)

Using Table 9a

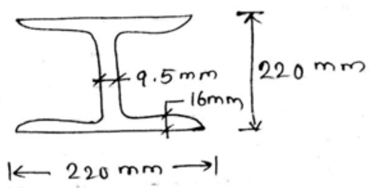
$f_{cdx} = 132$ MPa

Area $= \dfrac{1050 \times 10^3}{118} = 8898.30\ mm^2$

Take SC 220, Area $= 8980 mm^2$

$r_x = 9.35cm = 93.5$ mm

$r_y = 49.0$ mm



Buckling class classification (Table 10)

$\dfrac{h}{b_f} = \dfrac{220}{220} = 1 < 1.2$

$t_f = 16$ mm < 100 mm

X - X axis → Class b

Y - Y axis → Class c.

Actual Slenderness ratio

$\lambda x = \dfrac{KLx}{r_x} = \dfrac{7 \times 10^3}{93.5} = 74.86 < 180$ (ok)

Table 3 Page 20.

$\lambda y = \dfrac{KLy}{r_y} = \dfrac{5000}{49} = 102.04 < 180$ (ok)

Actual compressive stress (f_{cd})

In X-dir^n (f_{cd})

$\lambda x = 74.86$, $f_y = 250$

$f_{cdx} = 158$ MPa

In Y-dir^n.

$\lambda_y = 102$, $f_y = 250$

$f_{cdy} = 104.52$ MPa.

Capacity :-

X-dir^n = $f_{cd} \times$ Area
$= 158 \times 8980$
$= 1418.84$ kN > 1050 kN (safe)

Y-dir^n = 104.52×8980
$= 938.59$ kN < 1050 (unsafe)

Since, column is unsafe in Y-dir^n take another I-section SC 250 & repeat the process.

(Q3) Design of compressive members to carry a compressive load (factored) of 150 KN. The centre-to-centre distance between the end connection is 3m. & use M16 bolts of 4.6. Design connection also. Take Fe 410.

Soln :

Assume $\lambda = 100 = \frac{KL}{r}$

Assume angle section.
Using Buckling class C (Table 9c)

$\lambda = \frac{KL}{r} = 100$, $f_y = 250$

$f_{cd} = 107$ MPa

Area of Section = $A_g = \frac{150 \times 10^3}{107} = 1401 \text{ mm}^2$

Provide single angle section

Take ∠90 90×10 mm $A_g = 1700 \text{ mm}^2$

$r_{min} = r_{vv} = 1.74$ cm $= 17.4$ mm

Assume $l_{eff} = l = 3$m

$\lambda_{actual} = \frac{3 \times 10^3}{17.4} = 172.4 < 180$ (ok)

Actual capacity = $f_{cd} \times$ Area.

For, f_{cd}

$$f_{cd} = \frac{1}{\phi + [\phi^2 - \lambda^2]^{0.5}} \times \frac{f_y}{\gamma_{mo}}$$

$$\lambda = \lambda_e = \sqrt{k_1 + k_2 \lambda_{vv}^2 + k_3 \lambda_\phi^2}$$

$\left.\begin{array}{l} k_1 = 0.7 \\ k_2 = 0.6 \\ k_3 = 5 \end{array}\right\} \geq 2$ bolts & Hinged.

$$\lambda_{vv} = \frac{\left(\frac{l}{r_{vv}}\right)}{\varepsilon\sqrt{\frac{\pi^2 E}{250}}} = \frac{\left(3000/17.4\right)}{1\sqrt{\frac{\pi^2 \times 2 \times 10^5}{250}}}$$

$\therefore \lambda_{vv} = 1.94$

$$\lambda_\phi = \frac{(b_1+b_2)/2t}{\varepsilon\sqrt{\frac{\pi^2 E}{250}}} = \frac{(90+90)/(2 \times 10)}{1\sqrt{\frac{\pi^2 \times 2 \times 10^5}{250}}}$$

$\therefore \lambda_\phi = 0.101$

$\lambda = \lambda_e = 1.735$

$\phi = 0.5[1 + \alpha(\lambda - 0.2) + \lambda^2]$
$= 0.5[1 + 0.49(1.735 - 0.2) + 1.735^2]$
$= 2.381$

$\therefore f_{cd} = 56.65$ MPa. (UNSAFE)

$P_d = f_{cd} \times$ Area $= 56.65 \times 1700 = 96.311 < 150$ kN

Design steps of lacing :-

1) Assume design compressive stress (f_{cd}) = 100-175 MPa
2) Calculate approx. area by using

$$A_{approx} = \frac{P_u}{f_{cd}}$$

3) Take a trial section having Area $\geq A_{approx}$.

For two channel sections $A_{req} = \frac{A_{approx}}{2}$

For four angle sections $A_{req} = \frac{A_{approx}}{4}$

4) The sections are so spaced that the radius of gyration of built-up column about the centroidal axis perpendicular to plane of lacing (r_y) is not less than radius of gyration about the centroidal axis in the plane of lacing. i.e. $r_y \geq r_x (= r_z)$.

5) Calculate eff. slenderness ratio

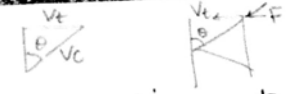
$$\lambda_e = 1.05 \; \lambda_{actual}$$

6) Calculate actual design comp. stress using λ_e & buckling class. (from Table 9(c))
7) Calculate design load carrying capacity

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

8) Assume angle of inclination of lacing bar betn 40-70° to the longitudinal axis of built-up column.
9) Calculate spacing of lacing bar.
10) Max. spacing of lacing bar should be

$\sin\theta = \frac{V_t}{V_c}$ $\quad V_c = V_t/\sin\theta$



such that max. slenderness ratio not greater than 50 or 0.7 times max. slenderness ratio of column as a whole whichever is less.

11) Calculate length of lacing bar & spacing of lacing bar diagrammatically.
12) Lacing bar should be designed to resist a transverse shear (V_t) = 2.5% of design axial load
13) The compressive force in lacing bar

$$= \left(\frac{V_t}{N}\right) \operatorname{cosec}\theta \text{ for single lacing}$$

$$= \left(\frac{V_t}{2N}\right) \operatorname{cosec}\theta \text{ for double lacing. } (N=2)$$

14) Width of lacing bar should not be less than 3 times the nominal dia. of end connector.
15) Thickness of lacing bar

$$t \nless \frac{l_e}{40} \text{ for single lacing}$$

$$t \nless \frac{l_e}{60} \text{ for double lacing}$$

16) The min. radius of gyration of lacing bar

$$r_{min} = \sqrt{\frac{I}{A}} = \frac{t}{\sqrt{12}}$$

17) The slenderness ratio of lacing bar should not be greater than 145

$$\frac{KL}{r_{min}} < 145 \qquad K = 0.7$$

→ In case of single lacing, the eff. length is distance betn inner end rivet / bolt.

→ In case of double lacing, the eff. length is 0.7 times distance betn inner end rivet / bolt

→ In case of welding, eff. length is 0.7 times inner end of weld connecting lacing bar.

18) Calculate design comp. stress in lacing bar by using Table 9C.

19) $P_{lacing} = A_{lacing} \times f_{cd,lacing} > \text{Load on lacing}$

(Q) Design a built-up column 10 m long. It carry an axial load of 1200 kN and composed of 2 channels placed back-to-back. Design the lacing (single) system. Take $f_u = 410$ MPa. Design end connection of lacing bar using site weld.

Soln:-

Design load $= 1.5 \times 1200 = 1800$ kN.

$L_{eff} = KL = 10$ m.

Assume, $f_{cd} = 150$ MPa.

$$A_{approx} = \frac{P_u}{f_{cd}} = \frac{1800}{150} = 12000 \text{ mm}^2$$

$$A_{req} = \frac{12000}{2} = 6000 \text{ mm}^2$$

Take ISMC 400 @ 50.1 kg/m.

$A = 6380 \text{ mm}^2$

$I_x = 15200 \times 10^4 \text{ mm}^4$

$I_y = 508 \times 10^4 \text{ mm}^4$

$C_y = 24.2$ mm

$r_x = 154$ mm

$r_y = 28.2$ mm

Actual slenderness ratio,

$$\lambda_{actual} = \frac{L_{eff}}{r_{min}} = \frac{L_{eff}}{r_x} = \frac{10 \times 10^3}{154} = 64.94$$

$\lambda_e = 1.05\, \lambda_{actual} = 68.18$

Design compressive stress (f_{cd})

For $\lambda_e = 68.18$, buckling class c, $f_y = 250$

$\therefore f_{cd} = 154.91$ MPa

Design capacity $P_d = 2 \times 6380 \times 154.91 \times 10^{-3}$

$= 1976.65 \text{ kN} > 1800 \text{ kN (OK)}$

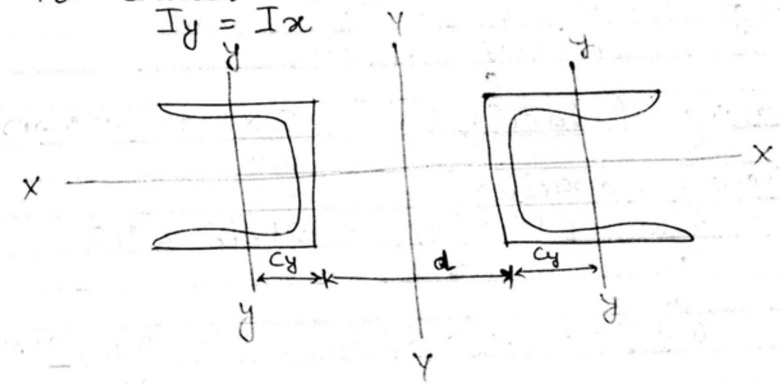
Design of Spacing (d)

$r_y \geq r_x$

or, $\sqrt{\frac{I_y}{A}} \geq \sqrt{\frac{I_x}{A}}$ or, $I_y \geq I_x$

For critical value,

$I_y = I_x$



$$2\left[I_y + A\left(C_y + \frac{d}{2}\right)^2\right] = 2I_x$$

or, $508 \times 10^4 + 6380\left(24.2 + \frac{d}{2}\right)^2 = 15200 \times 10^4$

$\therefore d = 255.1$ mm

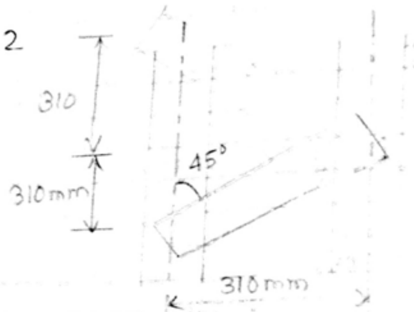
Provide $d = 260$ mm

Design of lacing

Angle of inclination $\theta = 45^\circ$
Centroidal distance between two channels.

$= d + c_y + c_y$
$= 260 + 24.2 + 24.2$
$= 308.4 \text{ mm}$

Provide 310 mm.



Length of lacing bar

$\sin 45^\circ = \dfrac{310}{\text{length of lacing bar}}$

$\therefore$ Length of lacing bar $= 438.4$ mm
Provide 440 mm

Spacing of lacing bar $= 2 \times 310 = 620$ mm.

Check for spacing

$\dfrac{620}{r_y} = \dfrac{620}{28.2} = 21.98 < 50$

$0.7\lambda_e = 0.7 \times 68.18 = 47.726 > 21.98$
(ok)

Width of lacing

Take $b = 50$ mm (50 to 70 mm)

Thickness of lacing bar

$t \nless \dfrac{l_e}{40} \nless \dfrac{440}{40} \nless 11$

Provide $t = 12$ mm

Forces in lacing bar

Transverser shear $V_t = 2.5\%$ of P_u
$= \dfrac{2.5}{100} \times 1800 = 45 \text{ kN}$

Compressive force in lacing bar

$V_{c,} = \dfrac{V_t}{N} \operatorname{cosec}\theta = \dfrac{45}{2} \operatorname{cosec} 45^\circ = 31.82 \text{ kN}$

Slenderness ratio of lacing bar

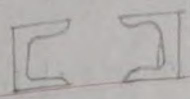
$\lambda = \dfrac{l_e}{r_{min}} = \dfrac{440 \times 0.7}{3.464} = 88.9 < 145$
ok.

$r_{min} = \dfrac{t}{\sqrt{12}} = \dfrac{12}{\sqrt{12}} = 3.464$

For fcd,
for $\lambda = 88.9$, $f_y = 250$ MPa, Table 9C
$f_{cd, lacing} = 121$ MPa.

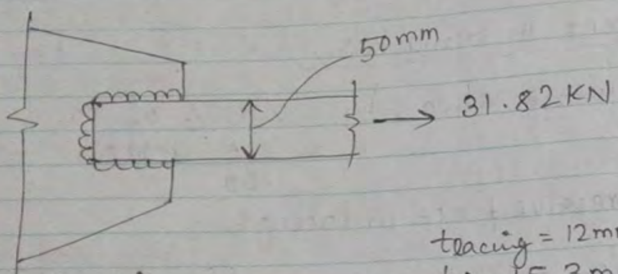
Capacity of lacing bar $= 121 \times (12 \times 50)$
$= 72.6$ kN
> 31.82 kN
(ok).

Design for Toe to Toe



Design of end connection

Strength of weld



$t_{lacing} = 12mm$
$t_f = 15.3mm$

Size of weld.

$S_{min} = 5mm$

$S_{max} = 12 - 1.5 = 10.5mm$

Provide $S = 8mm$

$t_e = K.S = 0.7 \times 8 = 5.6mm$

$$P_d = t_e \cdot l_w \cdot \frac{f_u}{\sqrt{3}\,\gamma_{mw}}$$

$$31.82 = 5.6 \times l_w \times \frac{410}{\sqrt{3} \times 1.25}$$

$$l_w = 36.006 \, mm + 2 \times 8$$

~~≥ 40mm~~

∴ length of both = 53 mm

Design of tie plate → Refer design of batten.

Design Steps of Battens (Page 50 Cl. 7.7)

Steps 1 to 4 Same as lacing

5) $\lambda_e = 1.1 \lambda_{actual}$

6) Calculate f_{cd} ; $P_d = A_e \times f_{cd}$

7) Battenings will be designed to a total shear force 2.5% of total axial load.

8) No. of battens should be provided such that column is divided into at least 3 bays within its actual length from centre-to-centre of end connections (i.e. atleast 3 battens provided)

9) Effective depth of end batten shall not be less than the centroidal distance between built-up component members. ($s' + 2c_y$) width
Effective depth of intermediate batten is taken as 3/4 th of depth of end batten.

10) Thickness of batten shall not be less than 1/50th distance betw innermost connecting lines of bolts or welds.

12) Spacing of batten shall be such that slenderness ratio of any component over that distance shall neither be greater than 50 nor greater than 0.7 times slenderness ratio of the member/column.

11) Overlap of batten ≮ 4 × thickness of batten

13) The longitudinal shear & B.M.

$$\left.\begin{aligned} V_b &= \frac{V_t \cdot C}{N S} \\ M &= \frac{V_t \cdot C}{2 N} \end{aligned}\right\}$$

failure by yielding

14) Check for shear stress & bending stress of batten plates

Shear stress $= \tau_s = \frac{V_b}{A} = \frac{V_b}{t \times d} < \frac{f_y}{\sqrt{3}\,\gamma_{m0}}$

Bending stress $= \tau_b = \frac{6M}{td^2} < \frac{f_y}{\gamma_{m0}}$

(8) Design a built-up column of effective length of 6 m to carry an axial load of 1000 kN using 2 channels & battens. Design connection using weld. Grade of steel E250.

Soln :-

$L_{eff} = KL = 6\,m$.

Axial load $= 1000\,kN$

Factored load $P_u = 1.5 \times 1000 = 1500\,kN$

Assume permissible stress $f_{cd} = 150\,MPa$.

$A_{approx} = \frac{1500 \times 10^3}{150} = 10{,}000\,mm^2$

Area of each channel $A = 5000\,mm^2$

Take ISMC ~~350~~ 400

Area $= 6380\,mm^2$

$I_x = 15200 \times 10^4\,mm^4$

$I_y = 508 \times 10^4\,mm^4$

$r_x = 154\,mm$

$r_y = 28.2\,mm$

$C_y = 24.2\,mm$

$b_f = 100\,mm$

$\Rightarrow$ for design of spacing $r_y \geq r_x$

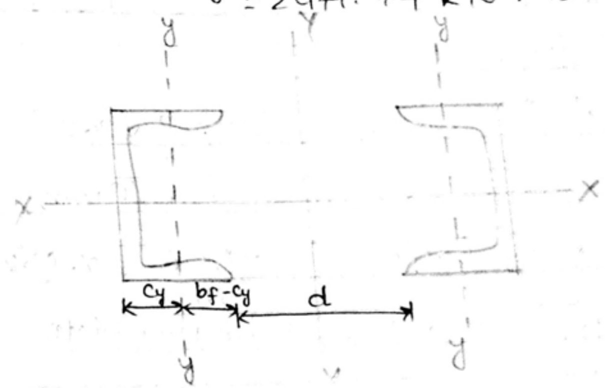
$r_{min} = r_x = 154\,mm$

Slenderness ratio $(\lambda_{actual}) = \frac{KL}{r_{min}} = \frac{6 \times 10^3}{154} = 38.96$

Effective $\lambda_{eff} = 1.1 \times 38.96 = 42.86$

Using Table 9C, $f_{cd} = 193.71\,MPa$.

Design capacity $= 193.71 \times 2 \times 6380 = 2471.74\,kN > 1500\,kN$ (ok).



Design of spacing

$r_y \geq r_x$

$I_{yy} \geq I_{xx}$

Critical condition, $I_{YY} = I_{xx}$

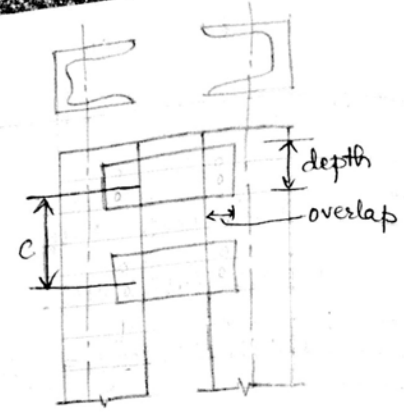
or, $2\left[I_y + A\left(b_f - C_y + \frac{d}{2}\right)^2\right] = 2 I_x$

or, $\left[508 \times 10^4 + 6380\left(100 - 24.2 + \frac{d}{2}\right)^2\right] = 15200 \times 10^4$

$\therefore d = 151.9\,mm$

Provide 200 mm spacing.

Design of Batten
End Batten :-



Min depth (width) of end batten

= Centroidal distance betn 2 channels

$= 200 + 2(b_f - c_y)$

$= 200 + 2(100 - 24.2)$

$= 351.6$ mm

Provide depth $= 360$ mm

~~∴~~ Depth of intermediate batten $= \frac{3}{4} \times 360$

$= 270$ mm

Thickness of batten $\nless \frac{351.6}{50} = 7.032$ mm

Provide 8 mm thick batten plate.

~~Provide~~ Overlap $\nless 4 \times$ Thickness of batten

$\nless 4 \times 8$

$\nless 32$ mm

Provide overlap 40 mm

Length of batten $= 200 + 40 + 40$

$= 280$ mm

Provide $280 \times 360 \times 8$ mm end batten and $280 \times 270 \times 8$ mm intermediate batten

N is. no. of plane

Spacing of Batten (c)

$\frac{c}{r_y} < 50$ or $0.7\lambda_e$

$\frac{c}{28.2} < 50$ or 0.7×42.86

< 50 or 30.002.

$c < 28.2 \times 30.002 = 846.06$ mm

Provide $c = 800$ mm.

Design strength of Batten

Batten is designed for bending moment & shear force developed due to transverse shear force.

$V_t = 2.5\%$ of $1500 = 37.5$ kN.

Shear force in batten $V_b = \frac{V_t \cdot C}{NS}$

$S = 200 + \frac{40}{2} + \frac{40}{2} = 240$ mm

$\therefore V_b = \frac{37.5 \times 800}{2 \times 240} = 62.5$ kN

Moment $M_b = \frac{V_t \cdot C}{2N} = \frac{37.5 \times (800 \times 10^{-3})}{2 \times 2}$

$= 7.5$ kNm

$\boxed{\text{S – distance betn CG of weld}}$

$A = d \times t$

Check:-

End batten:-

Shear stress $= \frac{V_b}{A} = \frac{62.5 \times 10^3}{8 \times 360} = 21.7$ MPa

$< \frac{250}{\sqrt{3} \times 1.10}$

< 131.2 MPa (ok)

Bending stress $= \frac{6M}{td^2}$

$= \frac{6 \times 7.5 \times 10^6}{8 \times 360^2}$

$= 43.4$ MPa $< \frac{250}{1.1}$

< 227.27 MPa

(ok)

Intermediate batten:-

Shear stress $= \frac{62.5 \times 10^3}{8 \times 270} = 28.94$ MPa (ok)

Bending stress $= \frac{6 \times 7.5 \times 10^6}{8 \times 270^2} = 77.16$ MPa (ok).

Design of connection

Eccentric welded connection in the plane of column.

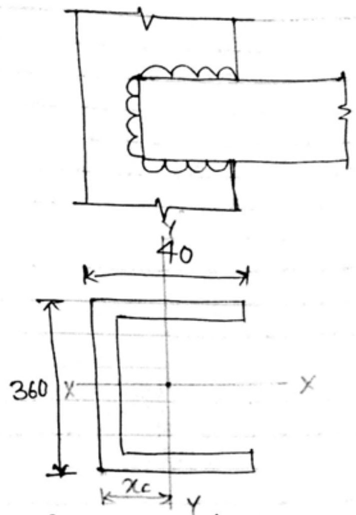


fig:- End batten connection

$x_c = \frac{\Sigma L_i x_i}{\Sigma L_i}$

$= \frac{40 \times 20 + 360 \times 0 + 40 \times 20}{40 + 360 + 40}$

$= 3.64$ mm.

$I_x =$

$I_y =$

$I_p = I_x + I_y$

$q_x = \frac{P.e.y}{I_p}$

$q_y = \frac{P.e.x}{I_p} + \frac{P}{l_w . t_w}$

$q = \sqrt{q_x^2 + q_y^2}$

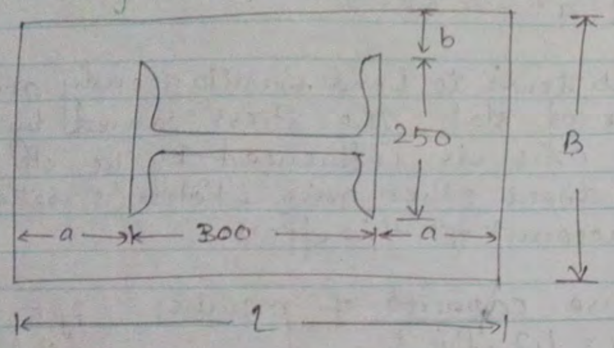
$< \frac{f_y}{\sqrt{3}\, \gamma_{mw}}$

$t_w = 0.7 S$.

here,

$$\boxed{\begin{array}{l} P = V_b \\ P.e = M_b \end{array}}$$

(Q) Design the base plate for a column section ISHB 300 which carries a factored axial compression of 1500 kN. Grade of steel is E250 & grade of concrete is M20.



Soln :-

Bearing pressure of concrete $= 0.6 f_{ck}$ (cl. 7.4.1)

$= 0.6 \times 20$

$= 12$ MPa

Factored load = 1500 kN

Area required $= \dfrac{P_u}{\text{Bearing pressure}}$

$= \dfrac{1500 \times 10^3}{12}$

$L \times B = 125000 \text{ mm}^2$

$\dfrac{L}{B} = \dfrac{300}{250} = 1.2$

$L = 1.2B$

$1.2B \times B = 125000$

$B = 322.75 \text{ mm} > 250 \text{ mm}$

$L = 387.3 \text{mm} > 300 \text{mm}$

Provide $L = 400$ mm

$B = 350$ mm

Projection along $L = a = \dfrac{400-300}{2} = 50$ mm

" " $B = b = \dfrac{350-250}{2} = 50$ mm

$w = \dfrac{P}{L \times B} = \dfrac{1500 \times 10^3}{400 \times 350} = 10.71 \text{ N/mm}^2 < 12 \text{ MPa}$ (ok)

$t = \sqrt{2.5\, w (a^2 - 0.3 b^2) \dfrac{\gamma_{mo}}{f_y}}$

$= \sqrt{2.5 \times 10.71 \times (50^2 - 0.3 \times 50^2) \times \dfrac{1.1}{250}}$

$= 14.36 \text{mm} > 10.6 \text{ mm}$ (ok)

Provide $400 \times 350 \times 16$ mm slab base.

Provide 4 no. of M20 bolt to connect base plate with the concrete.

(Q) Design a slab base for a beam column SC 250 to transfer a factored axial compression of 750 kN & bending moment of 75 kNm. Take M30 concrete & E250 grade of steel.

$M = 75$ kNm

$P_u = 750$ kN

M30, E250.

$e = \dfrac{M}{P} = \dfrac{75}{750} = 0.1 \text{ m} = 100 \text{ mm}$

Entire base plate is under compression.

Length of slab base $\geq 6 \times e$

$\geq 6 \times 100$

≥ 600 mm

Provide $L = 600$mm

Bearing pressure $= 0.6 \times 30$ $(0.6 f_{ck})$

$= 18$ MPa

$$q_{max} = \frac{P}{L \times B}\left(1 + \frac{6e}{L}\right)$$

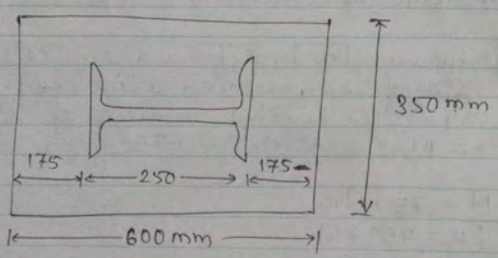
$$18 = \frac{750 \times 10^3}{600 \times B}\left(1 + \frac{6 \times 100}{600}\right)$$

$\therefore B = 138.89$ mm < 250 mm

Min. width $= 250 + 2 \times$ projections (50 – 100mm)

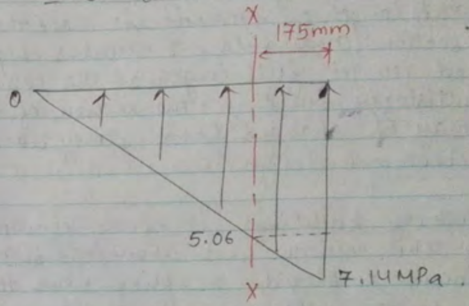
$= 250 + 2 \times 50$

$= 350$mm



$$q = \frac{750 \times 10^3}{600 \times 350}\left(1 \pm \frac{6 \times 100}{600}\right)$$

$= 0$ to 7.14 MPa < 18 MPa. (OK).



In case of slab base, max. bending moment occurs at the face of column.

M_{max} (at X – X) per mm length

$$= 5.06 \times 175 \times \frac{175}{2} + \frac{1}{2} \times 175 \times (7.14 - 5.06) \times \frac{2}{3} \times 175$$

$= 98714.58$ Nmm/mm

$= 98.71$ kN/mm

Now,

$$M = 1.2\, Z_e\, f_y / \gamma_{mo}$$

$$98.71 = 1.2 \times \frac{1 \times t^2}{6} \times \frac{250}{1.1}$$

or, $t = 46.6$ mm $> t_f$ (17mm) (OK).

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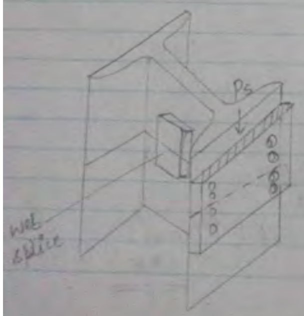
Column Splices

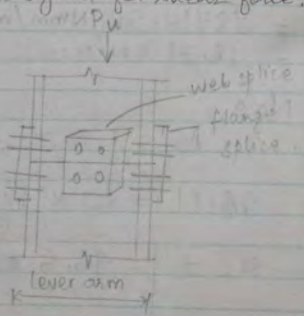
Connecting two pieces of sections to get the required length of column is called column splicing. Column sections are spliced in following cases:

1) When the length of column is more than the column section available, a number of pieces are joined to get full length of the column.
2) In multistorey building, the section of the column may be changed from storey to storey for ~~eco~~ economical design.

In multistorey buildings, it is very convenient to splice the column just above the floor & it is economical to provide a splice near the inflection point in the column.

In case of column splices with bolting, splice plates are to be used on the flanges as well as on the web. Splice plates are designed to resist an axial compression and bending moment (on flange). Splice plates on web are designed for shear force.





→ Column is preferably spliced at point of inflection which is usually 150 to 300mm above floor line.

Specification for Column Splicing

(1) The splice plate & the connections should be designed to carry 50% of axial load & tension (if any due to bending moment).

(2) Column splice plate may be assumed to act as a short column of zero slenderness ratio i.e. these plates will be subjected to yield stress.
Assume $\boxed{f_{cd} = f_y/\gamma_{m0}}$ & calculate required area of splicing plate.

(3) The width of splicing plate is taken equal to that of width of flange of column & calculate required thickness. The thickness of splicing plate shouldnot be less than 6mm for mild exposure condition.

(4) For selected dia. of bolt, the bolt value is calculated and required number of bolt can be calculated by = $\frac{\text{design load}}{\text{bolt value}}$

(5) If moment & shear force are also acting in addition to axial load, splices plates are provided to the flange as well as web.
Splice plates attached to the flange are designed to resist additional axial load equal to M/a where 'a' is distance between centre-to-centre of flange splice plate (lever arm)

Web splice plates are designed to resist max. shear force.

Total load $= P_{u1} + P_{u2}$

P_{u1} = Design splice load

$= \frac{P_u}{2}$ for non-machined ends

$= \frac{P_u}{4}$ for machined ends

P_{u2} = Load due to B.M. $= \frac{M_u}{\text{Lever arm}}$.

(6) Centroidal axis of splices should coincide with centroidal axis of members jointed.

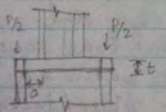
(7) When bearing plates are to be provided to ~~de~~ join two columns of unequal size, following steps are followed:

(a) Bearing plate may be assumed as short beam to transmit axial load to the lower column.

(b) Axial load of the column is assumed to be taken by flanges only.

Max. moment in bearing plate $= \frac{P}{2} \times a$

Thickness of bearing plate $= t = \sqrt{\frac{6M}{b\, f_{bs}}}$



$f_{bs} = f_y / \gamma_{m0}$.

(8) A column ISHB 300 @ 576.8 N/m is to support a factored load of 900 kN. The column section is to be spliced at a height of 2.5 m. Design the splice plate at connection using M16 bolts of property class 4.6. Use Fe410.

Soln:

Factored load = 900 kN

Load to ~~de~~ be designed for column splice

= 50% of 900 kN

= 450 kN

As we have to provide two column splices so load to be designed for each splice plate

= 225 kN.

Permissible stress $f_{cd} = \frac{f_y}{\gamma_{m0}} = \frac{250}{1.1} = 227.27$ MPa

Area of each splice plate $= \frac{225 \times 10^3}{227.27}$

$= 990\ mm^2$.

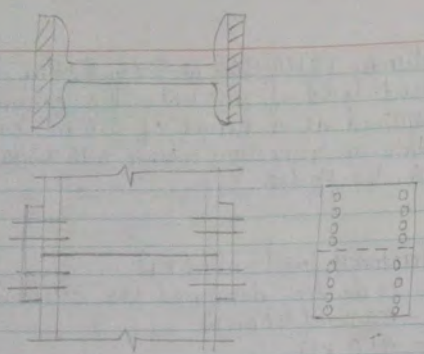
For ISHB 300 @ 576.8 N/m, $b_f = 250$ mm

Width of splice plate = Width of flange (b_f)

= 250 mm

Thickness of plate $= \frac{990}{250} = 3.96\ mm < 6\ mm$

Provide 250 mm × 6 mm splice plate



Design of connection.

M16 Bolts of property class 4.6.

Shear Strength $= \frac{400}{\sqrt{3} \times 1.25} \times 1 \times 0.78 \times \frac{\pi}{4} \times 16^2$

$= 28.97$ kN

Bearing strength $= 2.5\ K_b \times d \times t \times \frac{f_u}{\gamma_{mb}}$

$=$

$\therefore$ Bolt value $= 28.97$ kN

No. of bolts $= \frac{225}{28.97} = 7.76 \simeq 8$ Nos.

$P_{min} = 2.5d = 2.5 \times 16 = 40$ mm

$P_{max} = 32t$ or 300 mm

Provide Pitch $= 60$ mm &

edge distance $= 50$ mm

$\therefore$ Length of splice plate $= 6 \times P + 4 \times e$

$= 6 \times 60 + 4 \times 50$

$= 560$ mm

Provide two 560mm × 250 × 6mm splice plates.

(8) An ISHB250 column carrying a factored axial load of 900kN, bending moment 30 kNm & shear force of 100 kN. The column end is machined. Design the splice connection using M16 bolt of 4.6 grade.

Soln.

$P_u = 900$ kN

$M = 30$ kNm

S.F. $= 100$ kN.

Assume thickness of splice $= 6$ mm

lever arm $= 250 + \frac{6}{2} + \frac{6}{2}$

$= 256$ mm

Splicing on flange.

$P_{u1} = \frac{P_u}{4} = \frac{900}{4} = 225$ kN

$P_{u2} = \frac{M_u}{\text{lever arm}} = \frac{30 \times 1000}{(250+6)} = 117.19$ kN

Total design load for splice $= 225 + 117.19$

$= 342.19$ kN

Area = P/f_{cd} =

Area = $\dfrac{342.19 \times 10^3}{250/1.1}$

= ~~1368.76 mm^2~~ . ($1505.636 mm^2$)

Width of splice = Width of flange

= 250 mm

Thickness of splice = $\dfrac{1505.636}{250}$

= 6.022 mm .

Adopt t = 6 mm

Design of connection

i) Strength of bolt in single shear

V_{dsb} = 28.975 kN

ii) Strength of bolt in bearing

e = 40

p = 60

V_{dpb} = 84.56 kN

Bolt value = 28.975 kN .

No. of bolts = $\dfrac{342.19}{\text{Bolt value}}$

= 11.81

≈ 12 bolts (multiple of 2)

L = 4 × e + 10 × p

= 4 × 40 + 10 × 60 = 760 mm

Splicing of web .

Width of web between the flanges = 250 − 2×t_f

= 250 − 2 × 9

= 230.6 mm

Adopt width of web = 200 mm

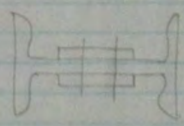
Now,

$V_d = \dfrac{f_y}{\sqrt{3}\,\gamma_{m0}} \times$ Area of splice

$100 \times 10^3 = \dfrac{250}{\sqrt{3} \times 1.1} \times (200 \times t) \times 2$

∴ t = 1.9 mm

Adopt t = 6 mm



Connection design .

Strength of bolt in Double shear .

V_{dsb} =

Strength of bolt in bearing

e = 40 mm

p = 60 mm

V_{dpb} =

∴ Bolt value =

No. of bolts = 2

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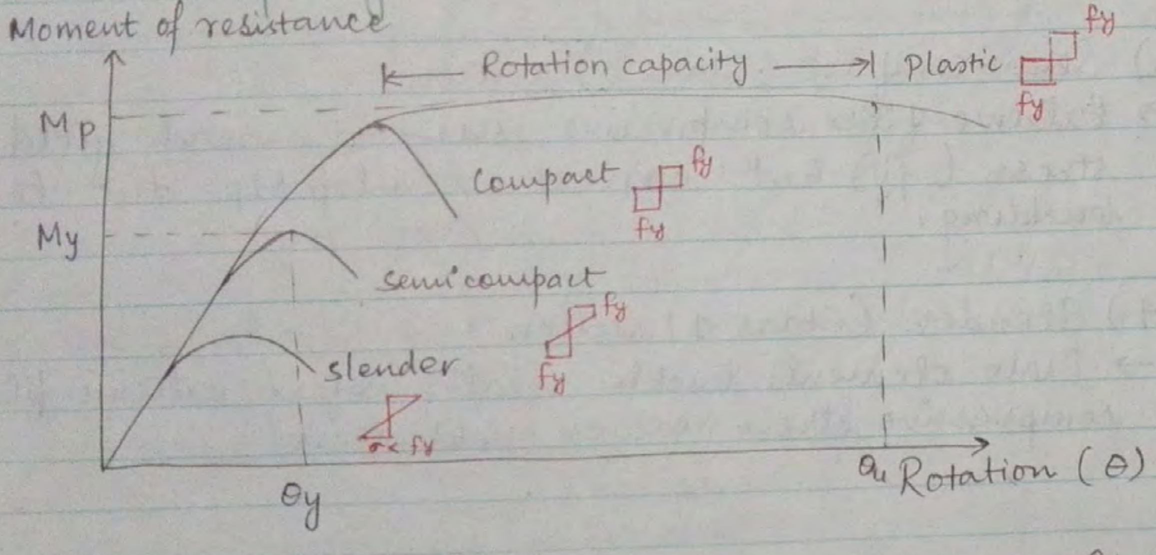
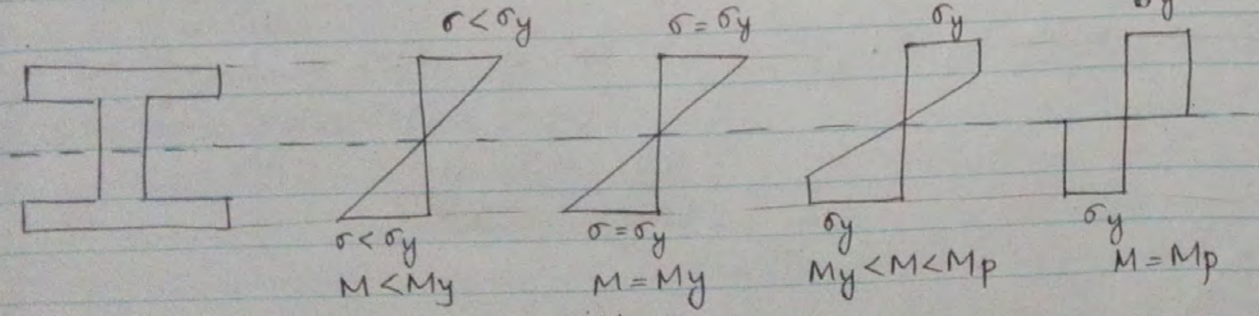
Chapter : SEVEN FLEXURE MEMBER

Any structural member subjected to transverse loading is called flexure member. The most economical & efficient section for the flexure member is I-section.

Beam → Simple beam I-section
→ Built-up beam
Plate Girder.

Classification of section based on moment rotation (M-θ) capacity.





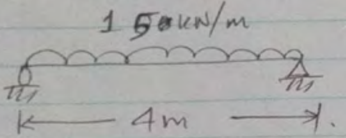
M is moment of resistance
My is Yield moment ; Mp is full plastic moment.

(8) A simply supported steel joist of 4m effective span is laterally supported throughout. It carries a total UDL of 40 kN/m inclusive of self weight. Design appropriate section. Fe 410.

Soln :-

Total UDL = 10 kN/m

Factored load = $1.5 \times 10 = 15$ kN/m



Max B.M $= \dfrac{w l_e^2}{8}$

$= \dfrac{15 \times 4^2}{8}$

$\therefore M = 30$ kNm

Max. S.F $= \dfrac{w l_e}{2} = \dfrac{15 \times 4}{2} = 30$ kN.

Required plastic modulus,

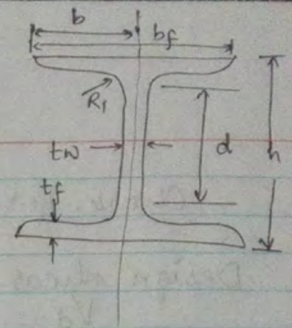
$Z_{p,req} = \dfrac{M \cdot \gamma_{mo}}{f_y}$

$= \dfrac{30 \times 10^6 \times 1.1}{250} = 132000\ mm^3$
$= 132\ cm^3$

Take a trial section ISLB200 @ 19.8 kg/m

$Z_{pz} = 184.34\ cm^3$ (plastic)

$Z_e = 170\ cm^3$ (elastic).



Depth of section (h) = 200 mm

Width of flange bf = 100 mm

Thickness of web tw = 5.4 mm

Thickness of flange tf = 7.3 mm

Radius of root R_1 = 9.5 mm

$b = \dfrac{b_f}{2} = \dfrac{100}{2} = 50$ mm

$d = h - 2(t_f + R_1)$
$= 200 - 2(7.3 + 9.5)$
$= 166.4$ mm

$I_z = 1700\ cm^4 = 1700 \times 10^4\ mm^4$.

Classification of section Pg 18, Table 2

$\dfrac{b}{t_f} = \dfrac{50}{7.3} = 6.85 < 9.4\varepsilon$ plastic

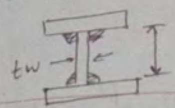
$\dfrac{d}{t_w} = \dfrac{166.4}{5.4} = 30.81 < 84\varepsilon$ plastic

Section is plastic.

No web buckling under shear before yielding since $\dfrac{d}{t_w} = 30.81 < 67\varepsilon$.

here check for shear buckling of web not required.

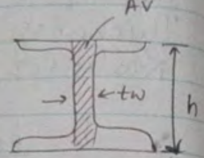
For welded :-



$A_v = d \times t_w$

Check for Shear Capacity

Design shear strength of section



$$V_d = \frac{V_n}{\gamma_{m0}} = \frac{A_v \cdot f_y}{\sqrt{3}\ \gamma_{m0}}$$

$$= \frac{(h \times t_w) \times f_y}{\sqrt{3} \times \gamma_{m0}}$$

$$= \frac{(200 \times 5.4) \times 250}{\sqrt{3} \times 1.1}$$

$$= 141.713\ kN > 30\ kN \quad \text{(OK)}.$$

Now,

$$0.6\, V_d = 0.6 \times 141.713$$
$$= 85.03\ kN > 30\ kN.$$

$V < 0.6\, V_d \Rightarrow$ Low Shear Case.

Check for Design Bending Strength (M_d)

$$M_d = \beta_b \cdot Z_p \cdot f_y / \gamma_{m0}$$

$$= \left(\frac{1 \times 184.34 \times 250 \times 10^3}{1.1} \right) \times 10^{-6}$$

$$= 41.895\ kNm > 30\ kNm$$

$M_d > M$ Safe in Bending.

OK.

For simply supported beam,

$$M_d \le 1.2 Z_{ez} \cdot f_y / \gamma_{m0}$$

$$= 1.2 \times \cancel{x}\ 170 \times 10^3 \times 250 / 1.1$$

$41.895 < 46.36\ kNm$ (OK)

Check for Deflection. (under working load)

Permissible deflection $\delta = \frac{l}{300} = \frac{4 \times 10^3}{300}$

$$= 13.33\ mm.$$

Max. deflection $\delta_{calc} = \frac{5}{384} \frac{w l^4}{EI}$

$$= \frac{5 \times 10 \times (4 \times 10^3)^4}{384 \times 2 \times 10^5 \times 1700 \times 10^4}$$

$$= 9.8\ mm < 13.33\ mm \quad \text{(OK)}$$

Check for web bearing (web crippling)

Assume stiff bearing length $= \cancel{75}\ 100\ mm$ (at support)

$(b = 100\ mm)$

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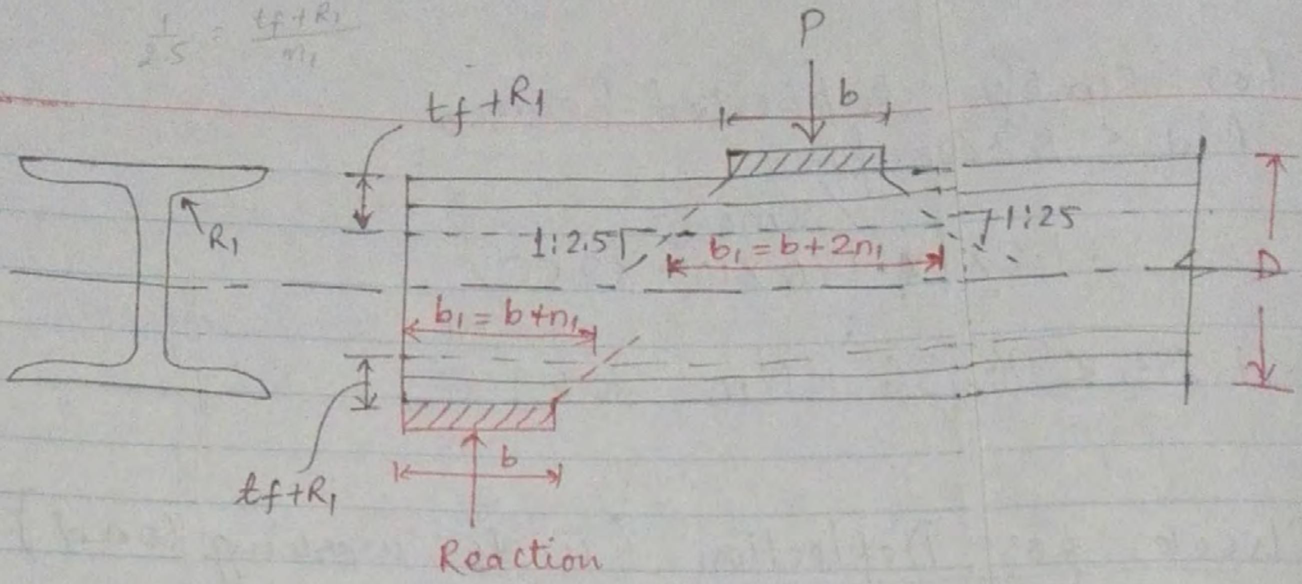
1V : 2.5 H

$\frac{1}{2.5} = \frac{t_f + R_1}{n_1}$



$$n_1 = 2.5(t_f + R_1)$$
$$= 2.5(7.3 + 9.5)$$
$$= 42 \text{mm}$$

$$b_1 = 100 + 42$$
$$= 142 \text{mm}$$

Bearing strength $F_w = A_e \times \frac{f_y}{\gamma_{m0}}$

$$= (b_1 \times t_w) \times \frac{250}{1.1}$$

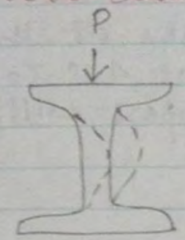
$$= 142 \times 5.4 \times \frac{250}{1.1}$$

$$= 174.27 \text{kN} > 30 \text{kN} \text{ (OK)}$$

Rxn at support.

30 June 2018, Sat

Web Buckling : Rolled I - section & deep plate Girder



The web in a rolled steel section behaves like a column when placed under concentrated loads. The webs are quite thin and therefore subjected to buckling.

Web buckling (vertical buckling) occurs when the intensity of vertical compressive stress near the center of section becomes greater than the critical buckling stress for the web acting as column.

Generally vertical buckling of webs is not a problem with rolled beam sections. This possibility exists mainly in thin webs of deep plate girder where thin ratio may exceed the permissible limit.

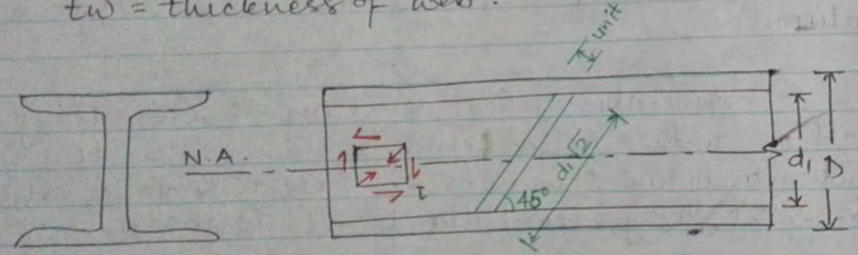
The max. diagonal compression occurs at the neutral axis and will be inclined at 45° to it. The web buckling strength at support will be

$$F_{wb} = B \cdot t_w \cdot f_{cd}$$

f_{cd} = compressive stress for web strut according to buckling curve or Class 'C'.

B = length of stiff portion of the bearing plus the additional length given by the dispersion at 45° to the level of neutral axis.

t_w = thickness of web.



Consider an element at neutral axis. The element is subjected to vertical shear τ on its vertical faces and complementary shear stresses of same magnitude are developed on the horizontal faces. The stresses produce diagonal compression & may cause buckling of the web.

Consider a column of unit thickness & of length equal to $d_1\sqrt{2}$ (as inclined at 45° to Neutral axis).

Effective length $= \frac{d_1\sqrt{2}}{2}$; if the ends are assumed to be fixed (Both flanges fixed)

$= d_1\sqrt{2}$; for pinned end.

Ends are assumed to be fixed for simplicity,

$$l_{eff} = \frac{d_1\sqrt{2}}{2} = \frac{d_1}{\sqrt{2}}$$

Minimum radius of gyration,

$$A = 1 \times t_w$$

$$I_{min} = \frac{1 \times t_w^3}{12}$$

$$r_{min} = \sqrt{\frac{I}{A}} = \sqrt{\frac{t_w^3}{12 \times t_w}} = \frac{t_w}{\sqrt{12}}$$

Slenderness ratio, $\lambda = \frac{l_{eff}}{r_{min}} = \frac{d_1 \times \sqrt{12}}{\sqrt{2} \times t_w} = \frac{2.45\, d_1}{t_w}$

From Table 9(C), find $f_{cd} > \tau_{max}$

here

$$d_1 = h - 2(t_f + R_1)$$

$\boxed{OR}$

$$r_{min} = \frac{I}{A} = \frac{100 \times t_w^3}{12 \times (100 \times t_w)} = \frac{t_w}{\sqrt{12}}$$

$$\lambda = \frac{l_{eff}}{r_{min}} = \frac{0.7\, d_1}{t_w/\sqrt{12}}$$

(5) Design laterally supported beam which is subjected to factored B.M of 150 kNm, factored shear force of 210 kN. Use Fe410. Effective span is 6 m.

Soln :-

$M_u = 150 \text{ kNm}$

$V_u = 210 \text{ kN}$.

Plastic section modulus required,

$$Z_{pz,req} = \frac{M \gamma_{mo}}{f_y} = \frac{150 \times 10^6 \times 1.1}{250} = 660 \times 10^3 \text{ mm}^3$$

Take ISLB 350 @ 49.5 kg/m

$h = 350$ mm

$b_f = 165$ mm

$t_w = 7.4$ mm

$t_f = 11.4$ mm

$R_1 = 16$ mm

$I_z = 13200 \times 10^4 \text{ mm}^4$

$Z_{ez} = 752 \text{ cm}^3 = 752 \times 10^3 \text{ mm}^3$

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

Depth of web, $d = h - 2(t_f + R_1)$

$= 350 - 2(11.4 + 16)$

$= 295.2$ mm

low shear $V \leq 0.6 V_d$

High shear $V > 0.6 V_d$

Section Classification

$$b = \frac{b_f}{2} = \frac{165}{2} = 82.5$$

$$\frac{b}{t_f} = \frac{82.5}{11.4} = 7.24 < 9.4\varepsilon \quad \text{Plastic}$$

$$\frac{d}{t_w} = \frac{295.2}{7.4} = 39.89 < 84\varepsilon \quad \text{Plastic}$$

Check for shear capacity :

$$V_d = \frac{V_n}{\gamma_{mo}} = \frac{A_v \cdot f_y}{\sqrt{3}\ \gamma_{mo}} = \frac{h \times t_w \times f_y}{\sqrt{3} \cdot \gamma_{mo}}$$

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

$$= 339.85 \text{ kN} > 210 \text{ kN} \quad \underline{(\text{OK})}$$

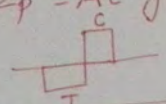
Check for high or low shear.

$0.6 V_d = 203.91 \text{ kN} < 210 \text{ kN}$

$\therefore \quad V > 0.6 V_d$, High Shear

Since $V > 0.6 V_d$, the design bending strength is less than the moment capacity of section due to the interaction between the moment & the shear. The moment capacity of the section will be reduced & is denoted by M_{dv}. The web area is ineffective in resisting the moment & only the flanges will resist the moment.

$Z_p = A_c\, y_c + A_t \cdot y_t$



for Plastic Section, Pg 70.

$M_{dV} = M_d - \beta\,(M_d - M_{fd}) \le 1.2 Z_e f_y / \gamma_{mo}$

$\le \dfrac{1.2 \times 752 \times 10^3 \times 250}{1.1}$

≤ 205.09 kNm

$\beta = \left(\dfrac{2V}{V_d} - 1\right)^2$

$= \left(\dfrac{2 \times 210}{339.85} - 1\right)^2$

$= 0.056$

$M_d = \beta_b \cdot Z_p \cdot f_y / \gamma_{mo}$ (Md is plastic moment of whole section disregarding high shear but considering web buckling effect)

$= 1 \times 851.11 \times 10^3 \times \dfrac{250}{1.1}$

$= 193.43$ kNm

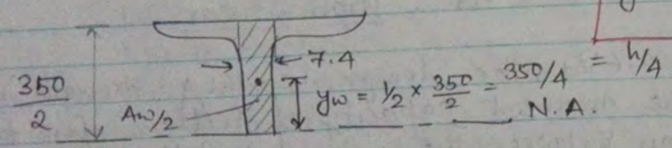
$M_{fd} = Z_{fp} \cdot f_y / \gamma_{mo}$ (here $Z_{fp} \Rightarrow$ plastic section modulus of flanges)

$Z_{fp} = Z_{pz} - A_w \cdot y_w$

$= Z_{pz} - \left(\dfrac{A_w}{2} \times y_w\right) - \left(\dfrac{A_w}{2} \times y_w\right)$

$= Z_{pz} - A_w \cdot y_w$

$\boxed{y_w = h/4}$



$Z_{fp} = 851.11 \times 10^3 - (350 \times 7.4) \times \dfrac{350}{4}$

$= 624485$ mm^3

$\therefore M_{fd} = 141.93$ kNm

$\therefore M_{dV} = M_d - \beta_b\,(M_d - M_{fd})$

$= 193.43 - 0.056\,(193.43 - 141.93)$

$= 190.546$ kNm > 150 kN

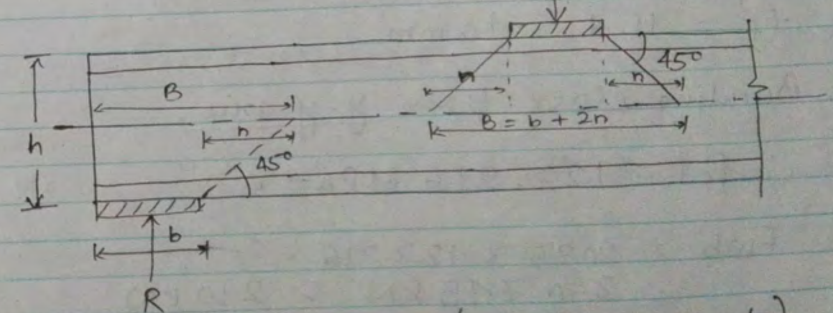
$M_{dV} > M$ (OK)

Also, $M_{dV} = 190.546 < 205.09$ kNm (OK)

Check for web buckling (at support or under concentrated load).

$\dfrac{d}{t_w} = 39.89 < 67\,\varepsilon$.

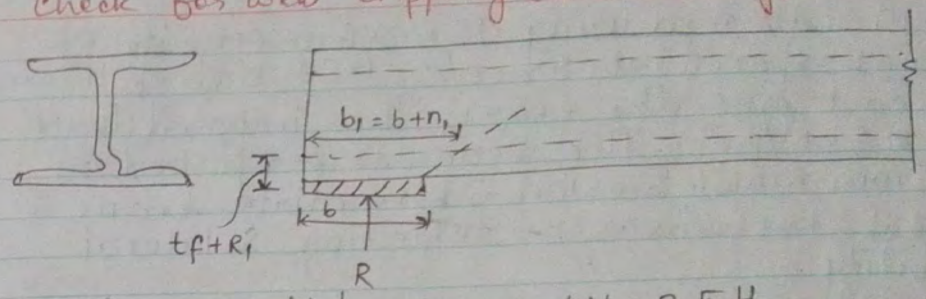
Check for web buckling not required.



$B = b + n = b + \dfrac{h}{2}$ (at the support)

$B = b + 2n = b + h$ (under concentrated load)

Check for web crippling (web bearing)



Assuming dispersion at 1V : 2.5H

$n_1 = 2.5(t_f + R_1) = 2.5(11.4+16) = 68.5\,mm$

$b_1 = 100 + 68.5 = 168.5\,mm$.

Web bearing strength at support,

$F_w = A_e \cdot \frac{f_y}{\gamma_{m0}}$

$= b_1\, t_w \cdot \frac{250}{1.1}$

$= \frac{168.5 \times 7.4 \times 250}{1.1} = 283.39\,kN > 210\,kN$ (OK)

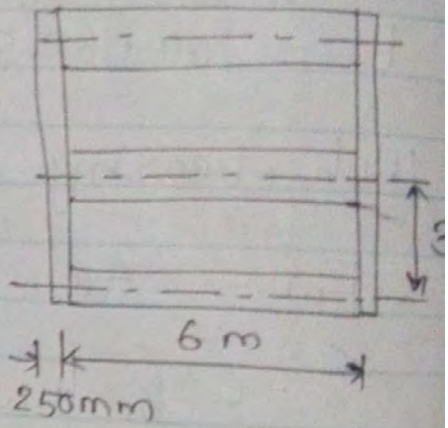
Hence, the section ISLB 350 is safe. #

2071 Bhadra

(Q) A hall measuring 15 m × 6 m consists of beams spaced at 3 m c/c. RCC slab of 110mm is cast over the beam. The imposed load is 4 kN/m². The beam is supported on 250mm wall. Design intermediate beam & check for shear, deflection & lateral stability.

Soln :-

Since RCC is casted on beam, it is laterally supported.



Here,

Clear span of beam = 6 m

Load per metre length of beam is :—

$$\text{Wt. of RCC slab} = \gamma \times l \times h = 25 \times 3 \times 0.11 = 8.25 \text{ kN/m}$$

Imposed load = 4 × 3 = 12 kN/m

Self weight = 0.8 kN/m (Assumed).

∴ Total working load = 8.25 + 12 + 0.8 = 21.05 kN/m

∴ Total factored load = 1.5 × 21.05 = 31.58 kN/m.

$$\text{Eff. length } (l_{eff}) = 6 + \frac{0.25}{2} + \frac{0.25}{2} = 6.25 \text{ m}$$

$\therefore$ Design moment $M = \frac{wl^2}{8}$

Design Shear force $V = \frac{wl}{2}$

Also See Eg : 8.7 Pg 444 Duggal.

(clause 8.2.2)

Laterally Unsupported Beam (Pg 54-59)

(Q) Design a laterally unsupported beam for the following data:

Eff. span = 4m
Max. B.M. = 550 KNm
Max. S.F. = 200 kN
Steel Grade = Fe 410
Conditions of restraints at supports :-
Torsional restraint - Fully restrained
Warping restraint - Warping not restrained in both flanges.

Soln:-

From Table 15 Pg 58.

$L_{LT} = KL = 1.0L = 1.0 \times 4 = 4m = 4000mm$
(Normal loading).

Plastic section modulus (Z_{pz}) required,

$Z_{pz,req} = 1.30 \times \frac{M \gamma_{mo}}{f_y}$

$= 1.30 \times \frac{550 \times 10^6 \times 1.1}{250}$

$= 3146000 \; mm^3$

$= 3140 \cancel{\times 10} \; cm^3$

Zpz may be increased by 25-50% for lateral torsional buckling

Try ISMB600 @ 1202.71 N/mm

$h = 600$ $\quad r_y = 41.2 \; mm$
$b_f = 210$ $\quad r_z = 242 \; mm$
$t_f = 20.3 \; mm$
$t_w = 12 \; mm$
$R_1 = 20 \; mm$

Depth of web $d = h - 2(t_f + R_1)$
$= 519.4 \; mm$

$I_z = 91813 \times 10^4 \; mm^4$
$I_y = 2651 \times 10^4 \; mm^4$
$Z_{pz} = 3510.63 \times 10^3 \; mm^3$
$Z_{ez} = 3060.4 \times 10^3 \; mm^3$.

Section classification.

$\frac{b}{t_f} = \frac{210/2}{20.3} = 5.172 < 9.4\varepsilon$ Plastic flange

$\frac{d}{t_w} = \frac{519.4}{12} = 43.28 < 84\varepsilon$ Plastic web.

$\therefore$ Section is Plastic.

Check for Design Bending strength

$M_d = \beta_b \cdot Z_{pz} \cdot f_{bd}$

$f_{bd} = \chi_{LT} \cdot \frac{f_y}{\gamma_{mo}}$

$\chi_{LT} = \frac{1}{\{\phi_{LT} + [\phi_{LT}^2 - \lambda_{LT}^2]^{0.5}\}} \leq 1.0$

$\phi_{LT} = 0.5[1 + \alpha_{LT}(\lambda_{LT} - 0.2) + \lambda_{LT}^2]$

here, $\alpha_{LT} = 0.21$ for Rolled section.

Simplified formula $M_{cr} = 1190796962$ Nmm
$\lambda_{LT} = 0.859 < 0.8781$
$\phi_{LT} = 0.9381$ $\quad \chi_{LT} = 0.7604$ $\quad f_{bd} = 172.814$

$$\lambda_{LT} = \sqrt{\beta_b \cdot Z_p f_y / M_{cr}} \le \sqrt{1.2 Z_e f_y / M_{cr}}$$

$$= \sqrt{\frac{f_y}{f_{cr,b}}}$$

$$f_{cr,b} = \frac{1.1 \pi^2 E}{(L_{LT}/r_y)^2} \left[1 + \frac{1}{20}\left(\frac{L_{LT}/r_y}{h_f/t_f}\right)^2\right]^{0.5}$$

$E = 2 \times 10^5$ N/mm^2
$L_{LT} = 4000$ mm
$r_y = 41.2$ mm
h_f = centre-to-centre distance betn flanges
$= 600 - 20.3$
$= 579.7$ mm
$t_f = 20.3$ mm

$\therefore f_{cr,b} = 289.36$ N/mm^2

$$\lambda_{LT} = \sqrt{\frac{250}{289.36}} = 0.9295 > 0.4,$$

the effect of lateral torsional buckling has to be considered.

$M_{cr} = \beta_b \cdot Z_p \cdot f_{cr,b}$
$= 1 \times 3510.63 \times 10^3 \times 289.36$
$= 1.015836 \times 10^9$ Nmm

$$\sqrt{1.2 Z_e f_y / M_{cr}} = \sqrt{0.90381} = 0.9507$$

$\lambda_{LT} = 0.9295 < 0.9507$ (ok).

Table 13a $f_{bd} = 161.195$

$$\phi_{LT} = 0.5\left[1 + \alpha_{LT}(\lambda_{LT} - 0.2) + \lambda_{LT}^2\right]$$
$= 1.0086$.

$\chi_{LT} = 0.7142 \le 1.0$

$\therefore f_{bd} = \chi_{LT}\, f_y / \gamma_{m0}$
$= 162.32$ N/mm^2.

$M_d = \beta_b \cdot Z_p \cdot f_{bd}$
$= 1 \times 3510.63 \times 10^3 \times 162.32$
$= 569.85$ kNm > 550 kN.
(ok).

Check for shear capacity

$V = 200$ kN.

$$V_d = \frac{f_y \cdot A_v}{\sqrt{3} \cdot \gamma_{m0}}$$

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

$= 944.75$ kN > 200 kN (ok).

~~Check for deflection~~

Check for web crippling at support.

$$F_w = (b + n_1) \cdot \times t_w \times \frac{f_y}{\gamma_{m0}}$$

$$= \{100 + 2.5(t_f + R_1)\} \times 12 \times \frac{250}{1.1}$$

$= 547.5$ kN > 200 kN (ok).

https://civinnovate.com/civil-engineering-notes/

Check for web buckling :

$\frac{d}{t_w} = 43.28 < 67\varepsilon$.

Check for web buckling under shear is not required.

$F_w = (b+n_1) \times t_w \times f_{cd} > 200\,kN$

here, $b = 300\,mm$ (Assume)

$n_1 = \frac{h}{2} = \frac{600}{2} = 300\,mm$

$t_w = 12\,mm$

for f_{cd} :-

$$\boxed{\lambda = \frac{KL}{r} = 3.57 \times \frac{h}{t_w}}$$

$$= 3.57 \times \frac{600}{12}$$

$$= 178.5$$

$\frac{h}{b_f} = \frac{600}{210} = 2.857 > 1.2$

$t_f = 20.3\,mm < 40\,mm$

Buckling class (b) in y-axis.

$\therefore f_{cd} = 47.25\,MPa$.

$\therefore F_w = 340.2\,kN > 200\,kN$

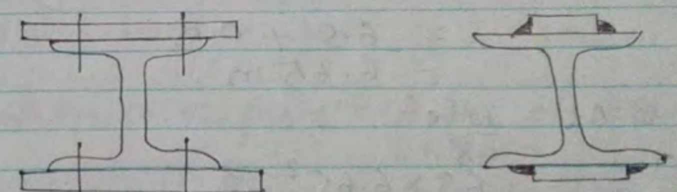
(ok).

BUILT-UP BEAM

Built up beams or compound beams are provided:

1) When large B.M. is generated due to long span and heavy loads.
2) When available rolled beam sections donot have sufficient strength to resist the external bending moment, then the I-section (rolled) may be reinforced along the entire length or part of it.
3) When the depth of the beam is restricted due to head room requirements.

In the above situations, the top & bottom flanges of rolled I-sections are reinforced along the length of beam by providing plates (i.e. cover plates).



Riveted or Bolted

Welded

(Q) Design a steel beam section for supporting roof of big hall for the following data & apply usual checks. Assume steel of Fe 410.

Clear span 6.5 m

End bearings = 150 mm

c/c spacing of beams = 3 m

Imposed load on beam = $10\ kN/m^2$
Dead load inclusive of self wt = $4\ kN/m^2$
Restriction on beam depth = 375 mm
The compression flange of beam is laterally supported throughout.

Soln :

Factored DL = $1.5 \times 4 = 6\ kN/m^2$
Factored L.L = $1.5 \times 10 = 15\ kN/m^2$
c/c spacing = 3 m.

Hence,

Total UDL = $(6\times3 + 15\times3)$
$\therefore \omega = 63\ kN/m$

Eff. span (l_e) = Clear span + $2 \times \frac{\text{Bearing length}}{2}$
$= 6.5 + 0.15$
$= 6.65\ m$

Max B.M = $\frac{wl_e^2}{8}$
$= \frac{63 \times 6.65^2}{8}$
$= 348.25\ kNm$

Max. S.F. = $\frac{wl_e}{2} = \frac{63 \times 6.65}{2} = 209.475\ kN$

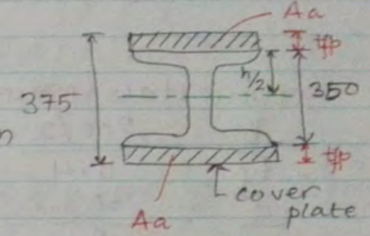
For a beam laterally supported throughout,

$Z_{pz,req} = \frac{M \cdot \gamma_{mo}}{f_y}$
$= 1532300\ mm^3$

Since the depth of beam is restricted to 375 mm.
Try ISWB 350 @ 558.19 N/m

$h = 350\ mm$	$I_z = 15500 \times 10^4\ mm^4$
$b_f = \cancel{165}\ 200\ mm$	$I_y = 1180 \times 10^4\ mm^4$
$t_w = \cancel{7.4}\ 8\ mm$	$r_z = 146\ mm$
$t_f = 11.4\ mm$	$r_y = 40.3\ mm$
$R_1 = 12\ mm$	$Z_{ez} = 887 \times 10^3\ mm^3$
$d = h - 2(t_f + R)$	$Z_{pz} = 995.49 \times 10^3\ mm^3$.
$= 303.2\ mm$	

Section modulus required by plates



$Z_a = Z_{pz,req} - Z_{pz}$ of I-section
$= 1532300 - 995.49 \times 10^3$
$= 536810\ mm^3$

$Z_a = A_a \cdot h = A_a \cdot h/2 + A_a \cdot h/2$

$A_a = \frac{Z_a}{h} = \frac{536810}{350} = 1533.743\ mm^2$

(Assumption: distance betn c.g. of flange cover plates equal to depth of I-section)

Width of cover plate = Width of flange
$= 200\ mm$

Thickness of flange cover plate $t_{fp} = \frac{A_a}{B}$
$= \frac{1533.743}{200}$
$= 7.67\ mm$
$\not< 6\ mm$

Provide 8mm thick cover plate on each side of flange.

Overall beam depth $= 350 + 2 \times 8$
$= 366\,mm < 375\,mm$ (ok)

Z_p, provided $= 995.49 \times 10^3 + [(200 * 8) * (350 + 8)]$
$= 1568290\,mm^3$
$> 1532300\,mm^3$
(ok).

Section classification

$$\frac{b}{t_f} = \frac{200/2}{11.4} = 8.77 < 9.4\varepsilon$$

$$\frac{d}{t_w} = \frac{303.2}{8} = 37.9 < 84\varepsilon$$

∴ Section is plastic.

Since $\frac{d}{t_w} < 67\varepsilon$, check for web buckling is not required.

Check for shear capacity

$$V_d = \frac{A_v \cdot f_y}{\sqrt{3}\,\gamma_{mo}}$$

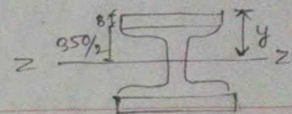
$$= \frac{h \times t_w \times f_y}{\sqrt{3} \times \gamma_{mo}} = \frac{350 \times 8 \times 250}{\sqrt{3} \times 1.1}$$

$= 367.405\,kN > 209.475\,kN$ (ok).

Now, $0.6 V_d = 220.44\,kN > 209.475\,kN$

Since, $V < 0.6 V_d$, Low shear case.

$$M_d = \beta_b \cdot Z_{pz} \cdot \frac{f_y}{\gamma_{mo}} \le 1.2\, Z_e \cdot \frac{f_y}{\gamma_{mo}}.$$



or, $M_d = 1 \times 1568290 \times \frac{250}{1.1}$

∴ $M_d = 356.43\,kNm > 348.25\,kNm$ (ok)

Now, $Z_{ez} = \frac{I_z}{y}$.

$$I_z = 15500 \times 10^4 + 2\left[\frac{200 \times 8^3}{12} + (200 \times 8)\left(\frac{350+8}{2}\right)^2\right]$$

$= 257.5483 \times 10^6\,mm^4$

$$\therefore Z_{ez} = \frac{I_z}{y} = \frac{257.5483 \times 10^6}{\left(\frac{350}{2} + 8\right)} = 1408553.37\,mm^3$$

$$\therefore 1.2\, Z_e \frac{f_y}{\gamma_{mo}} = 384.15\,kNm.$$

$M_d = 356.43\,kNm < 384.15\,kNm$ (ok).

Check for deflection

Service load $= \frac{63}{1.5} = 42\,kN/m$.

Permissible deflection $\delta = \frac{L}{300} = \frac{6.5 \times 10^3}{300} = 21.67\,mm$

[Table 6 Pg 31 : For simply supported beam, Brittle cladding]

$$\text{Max. deflection} = \frac{5}{384} \frac{wl^4}{EI}$$

$$= \frac{5}{384} \times \frac{42 \times (6500)^4}{2 \times 10^5 \times 257.5483 \times 10^6}$$

$= 18.95\,mm < 21.67\,mm$

(ok).

Check for web bearing at support.

Take $b = 100mm$

$$n_1 = 2.5\,(t_f + R_1)$$
$$= 2.5\,(11.4 + 12)$$
$$= 58.5\ mm$$

$$F_w = (b + n_1) \times t_w \times \frac{f_y}{\gamma_{m0}}$$
$$= (100 + 58.5) \times 8 \times \frac{250}{1.1}$$
$$= 288.18\ kN > 209.47\ kN$$
(ok)

Design of connection

Bolts/ welds joining the plates & flanges are to be designed for the horizontal shear at that level.

Shear stress at level of plate & flanges

$$= \frac{V}{b_f \cdot I_z} (A\bar{y})$$

$$= \frac{209.475 \times 10^3}{200 \times 257.5483 \times 10^6} \times (250 \times 8 \times 175)$$

$$= 1.4233 \text{ N/mm}^2$$

$$\text{Strength in single shear} = \frac{f_{ub}}{\sqrt{3}} \times 0.78 \times \frac{\pi}{4} d^2 \times \frac{1}{1.25}$$

$$= 28974.36 \text{ N}$$

Shear strength of two bolts = Shear force per pitch length.

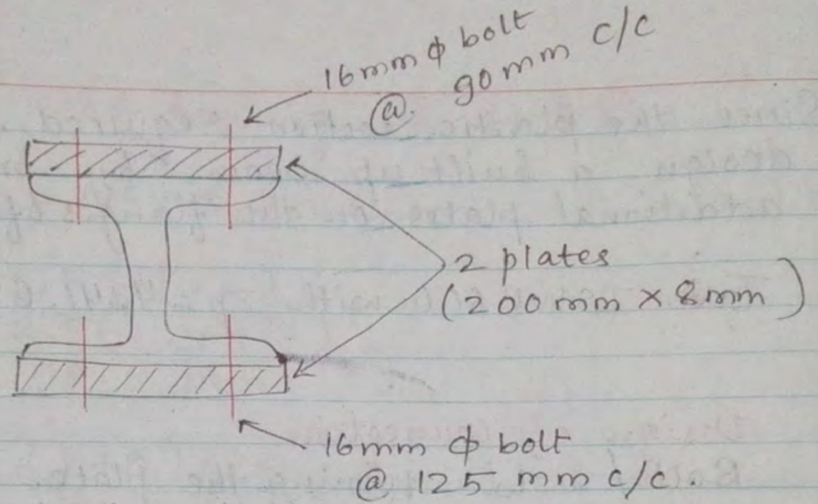
or, $2 \times 28974.36 = 1.4233 \times (200 \times P)$

$\therefore P = 203.57$ mm

Max Pitch in tension member = 16t or 200mm = 128 mm

Max. Pitch in comp. member = 12t or 200 mm = 96mm

Hence, provide 125mm pitch at tension side & 90mm pitch at compression side.



Note :-

Check for deflection :.

Consider pt. load on midspan of simply supported beam

$$M = \frac{PL}{4} \Rightarrow P = \frac{4M}{L} =$$

$$\text{Working load} = \frac{P}{1.5} =$$

$$\Delta max = \frac{PL^3}{48EI}.$$

Static Wind method

Wind load F on a building is given by

$F = C_f \times A_e \times P_d$ - for the building as a whole.

$= (C_{pe} - C_{pi}) \times A \times P_d$ - for individual structural elements as roofs, walls, glazing & their fixings.

where,

C_f = Force coeff. for building

C_{pe}, C_{pi} = Force coeff. for exterior & interior of bldg.

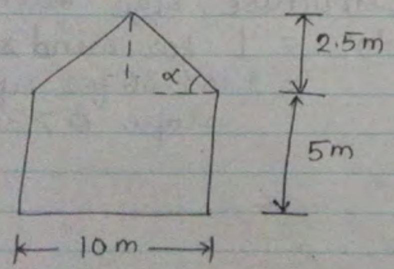
A_e = eff. area of structure

A = eff. area of member.

A_e is taken less than total area due to opening & shielding.

(Q1) Find wind pressure for design of a sloping roof of span 10m & pitch 1/4. The height of eaves is 5m above the ground. The building is situated in Madras & permeability is normal.

Soln:-



$\frac{h}{w} = \frac{5}{10} = 0.5$

$\frac{h}{w} \leq \frac{1}{2}$

$\text{Pitch} = \frac{1}{4} = \frac{\text{Rise}}{\text{Span}} = \frac{\text{Rise}}{10} = \frac{1}{4}$

$\therefore \text{Rise} = 10/4 = 2.5\text{ m}$.

Roof angle $\alpha = \tan^{-1}\left(\frac{2.5}{5}\right) = 26.56°$.

Assume Probability factor $K_1 = 1.0$ (General building)

Terrain / Height Structure size factor $K_2 = 0.8$

Topography factor $(K_3) = 1.0$

Basic wind speed at Madras $V_b = 50\text{ m/s}$.

Design wind speed $V_z = K_1 K_2 K_3 V_b$

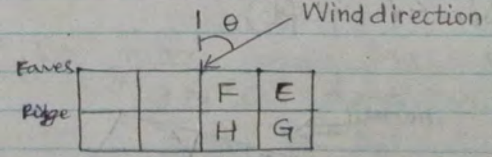
$= 1 \times 0.8 \times 1 \times 50$

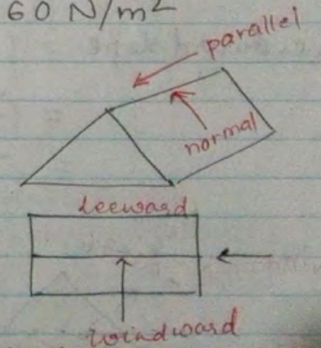
$= 40\text{ m/s}$.

Design wind pressure $P_d = 0.6 V_z^2$

$= 960\text{ N/m}^2$

(1) Wind normal to Ridge





External wind pressure coefficient C_{pe}:-

$\frac{h}{w} \leq \frac{1}{2}$

Roof angle $\alpha = 26.56°$.

Wind angle $\theta = 0°$

EF = windward slope.

GH = leeward slope.

− minus = upward

+ plus = downward

On windward slope EF $= -0.1376$ (By interpolation)
On leeward slope GH $= -0.4$

Internal wind pressure coefficient Cpi :-
Wall opening = 0 to 5% of wall area.
$\therefore$ $C_{pi} = \pm 0.20$ (Normal permeability).

$$\boxed{F = (C_{pe} - C_{pi}) \times A \times P_d}$$

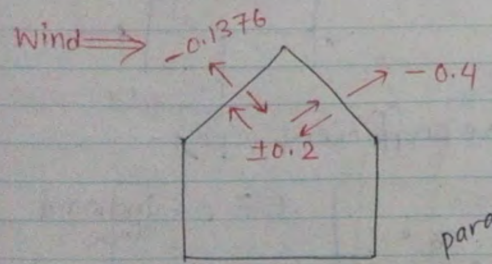
Combined external + internal wind pressures =

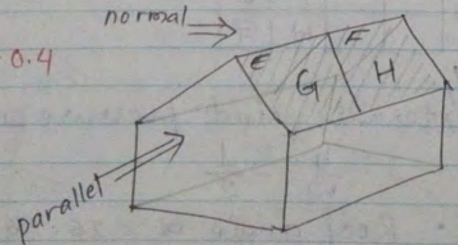
Windward slope $= (-0.1376 \pm 0.20) \times 960$

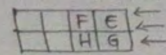
$= \cancel{19.867.904}$ $59.9\ N/m^2$ (downward)
$= -324.1\ N/m^2$ (uplift)

Leeward slope $= (-0.4 \pm 0.20) \times 960$

$= -192\ N/m^2$ (uplift)
$= -576\ N/m^2$ (uplift)







(2) Wind parallel to ridge ($\theta = 90°$)

External pressure coeff Cpe: on both slopes for 1/4th length of building $= -0.7$ (EG)

On both slopes for mid 1/2 ~~lot~~ length of bldg $= -0.6$ (FH)

Internal air pressure coeff. for normal permeability
$C_{pi} = \pm 0.2$
Combined external + Internal pressures :-

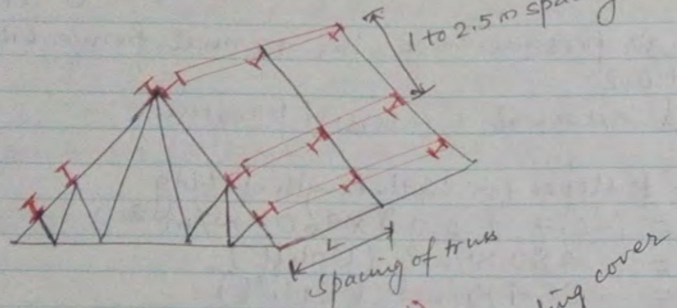
On both slopes for 1/4th length of bldg
$= (-0.7 \pm 0.2) \times 960\ N/m^2$
$= -480\ N/m^2$ (Uplift)
$= -864\ N/m^2$ (Uplift)

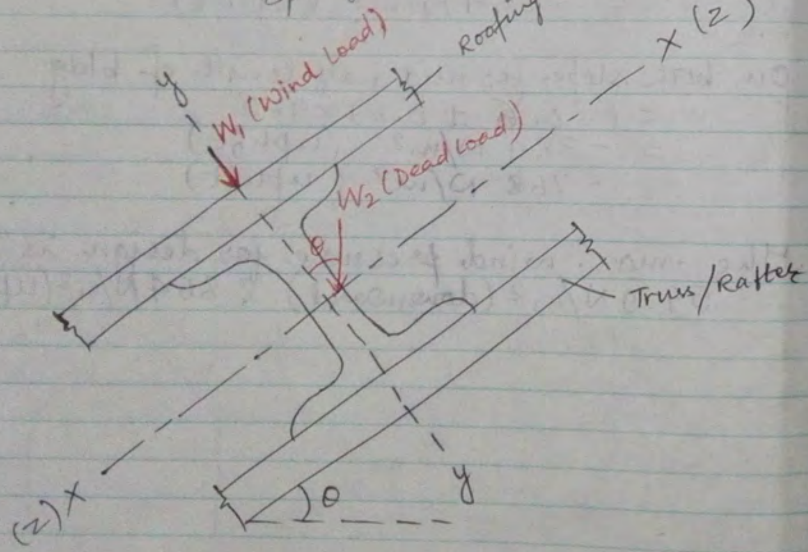
On both slopes for mid half length of bldg
$= (-0.6 \pm 0.2) \times 960$
$= -384\ N/m^2$ (uplift)
$= -768\ N/m^2$ (uplift).

The max. wind pressure for design is
$59.9\ N/m^2$ (downward) & $864\ N/m^2$ (uplift)

Design of Purlin:-

Purlins are members spanning on the roof frames to support the roof coverings. The spacing of truss is the span of the purlin. The spacing of purlin varies from 1 to 2.5m. Purlin consists of angles, channels or I-sections.

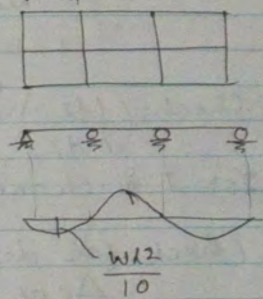




→ Purlin is continuously supported. $BM = \frac{WL^2}{10}$

Design steps:-

1) Resolve factored force parallel & perpendicular to the sheeting (Roofing).
2) Determine the moment & shear force about X & Y axis.



$$W_x = W_2 \sin\theta$$
$$W_y = W_1 + W_2 \cos\theta$$

(Purlins are designed as beams subjected to bending moment about z & y-axis).

$$M_x = M_z = \frac{W_y \cdot l^2}{10} = (W_1 + W_2 \cos\theta)\frac{l^2}{10}$$

$$M_y = \frac{W_x \cdot l^2}{10} = W_2 \sin\theta \cdot \frac{l^2}{10}$$

3) Determine the section modulus required for biaxial bending

$$Z_{pz} = M_z \cdot \frac{\gamma_{mo}}{f_y} + 2.5\left(\frac{d}{b}\right) M_y \cdot \frac{\gamma_{mo}}{f_y}$$

Take $\frac{d}{b} = 1$ to 2.

4) Determine the bending capacity of the section by using. (Page 53 Cl 8.2.1.2)

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

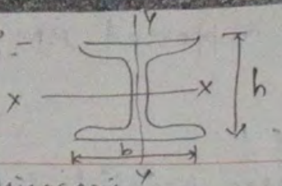
$$M_{dy} = \beta_b \cdot Z_{py} \frac{f_y}{\gamma_{mo}} \le 1.2\, Z_{ey} \frac{f_y}{\gamma_{mo}}$$

5) Check for shear (Page 59 Cl 8.4)

$$V_{dz} = \frac{f_y}{\sqrt{3}\,\gamma_{mo}} \cdot A_{vz} \quad ; \quad V_{dy} = \frac{f_y}{\sqrt{3}\,\gamma_{mo}} \cdot A_{vy}$$

shear area ⇓ $h t_w$

Shear area :-



X–X – Minor axis
Y–Y – Major axis
Shear area ⇒ Major axis bending ⇒ $h\,t_w$
Minor " " ⇒ $2b\,t_f$

(6) Check $\left(\frac{M_z}{M_{dz}}\right) + \left(\frac{M_y}{M_{dy}}\right) \le 1.0$

~~For I & channel sections $\alpha_1 = \alpha_2 =$~~.

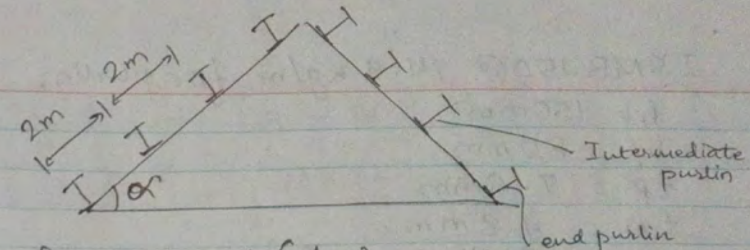
(7) Check for deflection

$\Delta_{cal} < \Delta_{per}$.

__Note:__ (purlin is not checked in shear & bearing since loads & shear are small).

(8) Design an I-section purlin for a roof truss from following data:
Span of roof = 12 m
Spacing of purlin along slope of roof = 2 m
Slope of roof truss = 1V : 2H
Spacing of truss = 5 m
Wind load on roof surface in normal direction = 1000 N/m²
Vertical load from roof sheet = 200 N/m²

Soln :-



Slope of roof $\alpha = \tan^{-1}(1/2) = 26.56°$
D.L. on Roof = 200 N/m²
Wt. of Roof sheeting = 200 × 2 = 400 N/m.
Assume self wt. of purlin = 150 N/m² (100 to 200)
∴ Self wt = 150 × 2 = 300 N/m
Total vertical load (W_2) = 400 + 300
= 700 N/m.
Total wind load (W_1) = 1000 × 2 = 2000 N/m

$W_x = W_2 \sin\alpha = 700 \sin 26.56° = 313$ N/m
$W_y = W_1 + W_2 \cos\alpha = 2626$ N/m

$L = 5$ m.
$M_x = M_z = \frac{W_y \cdot l^2}{10} = 6565$ N m
$M_y = \frac{W_x l^2}{10} = 782.5$ N m
Factored moment : $M_x = M_z = 9847.5$ N m
$M_y = 1.5 \times 782.5 = 1173.75$ Nm

Section modulus required :
$$Z_{p,req} = M_z \frac{\gamma_{mo}}{f_y} + 2.5\left(\frac{d}{b}\right)\frac{M_y}{f_y}\gamma_{mo}$$
$= 69.15$ cm³ $\left(\text{Take } \frac{d}{b} = 2\right)$

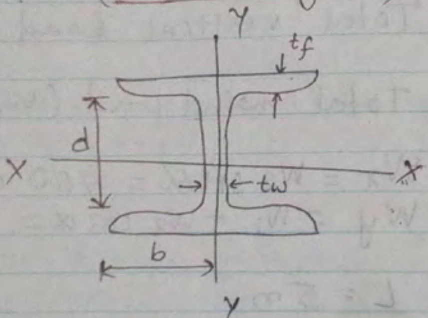
ISMB 150@ 14.9 kg/m for purlin,

$h = 150mm$
$b_f = 80mm$
$t_f = 7.6mm$
$t_w = 4.8mm$
$Z_{pz} = 110.48 cm^3$
$Z_{ez} = 96.9 cm^3$

Shape factor $(Z_{pz}/Z_{ez}) = 1.1401$

Now,

Section Classification :- (Table 2 Page 18)



$\frac{b}{t_f} = \frac{(80/2)}{7.6}$

$= 5.263 < 9.4\varepsilon$

$\frac{d}{t_w} = \frac{(150 - 2\times 7.6)}{4.8}$

$= 28.08 < 84\varepsilon$

The Section is PLASTIC,

$\beta_b = 1.0$

Design capacity,

$M_{dz} = \beta_b . Z_{pz} \frac{f_y}{\gamma_{m0}} \leq 1.2 Z_{ez} \frac{f_y}{\gamma_{m0}}$

$= 1 \times 110.48 \times \frac{250}{1.1} \leq 1.2 \times 96.9 \times \frac{250}{1.1}$

$= 25.109 kNm \leq 26.43 kNm$

(OK)

$Z_{py} = \frac{t_f b_f^2}{2} + \frac{(h - 2t_f)}{4} . t_w^2$

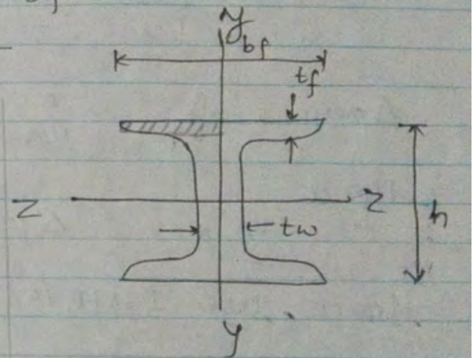
$= \frac{7.6 \times 80^2}{2} + \frac{(150 - 2\times 7.6)}{4} \times 4.8^2$

$= 25096.448 mm^3$

$Z_{ey} = \frac{I_y}{y} = \frac{I_y}{b_f/2} = \frac{2 I_y}{b_f}$

$= \frac{2 \times 46.8 \times 10^4 mm^4}{80}$

$= 11700 mm^3$



$M_{dy} = \beta_b . Z_{py} . \frac{f_y}{\gamma_{m0}}$

$= 1 \times 25096.448 \times \frac{250}{1.1}$

$= 5.704 kNm$

$\leq 1.2 Z_{ey} \frac{f_y}{\gamma_{m0}}$

$\leq \frac{1.2 \times 11700 \times 250}{1.1}$

$\nleq 3.19 kNm$

$Z = \frac{I}{y} = \frac{A . y^2}{y} = A . y$

$\left(t_f \times \frac{b_f}{2} \times \frac{b_f}{4}\right) \times 4 = \frac{t_f b}{2}$

$\left[(h - 2t_f) \times \frac{t_w}{2} \times \frac{t_w}{4}\right] \times 2 =$

$\frac{(h - 2t_f) t_w^2}{4}$

So,

Take $M_{dy} = 3.19 kNm$.

Check

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

$\frac{9.8475}{25.109} + \frac{1.17375}{3.19}$

$0.76 < 1$ safe (OK)

Check for deflection.

$$\Delta_{max} = \frac{5}{384} \cdot \frac{wl^4}{EI} = 0.0149 m = 14.9 mm$$

$w = w_1 + w_2 \cos\theta$

$= 2626 \, N/m$

$l = 5 m$

$E = 2 \times 10^5 \, N/mm^2 = 2 \times 10^{11} \, N/m^2$

$I = I_x = I_z = 718 \, cm^4 = 7.18 \times 10^{-6} m^4$

$$\Delta_{perm} = \frac{l}{150} = \frac{5}{150} = \frac{1}{30} m = 33.33 mm.$$

Hence,

$\Delta_{max} < \Delta_{perm}$ (OK)

Hence, use ISMB 150 @ 14.9 kg/m for purlin

Chapter 11: DESIGN OF COMPRESSION MEMBER

Types of Column

1. Solid Column:-

Any section (rectangular, circular, square) having solid core throughout the length of the column is called solid column.

→ Short $S/d < 11$
→ Intermediate $11 < S/d < K_8$
→ Long $S/d > K_8$

$$K_8 = 0.702\sqrt{\frac{E}{f_{cp}}}$$ (Page 2)

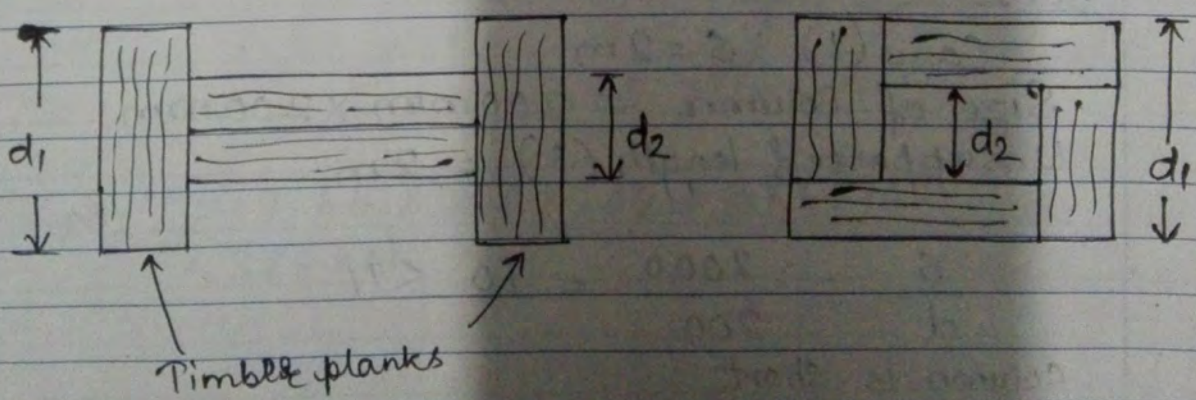
f_{cp} ← permissible compressive stress along grain

S is unsupported length of column; d is least dimension

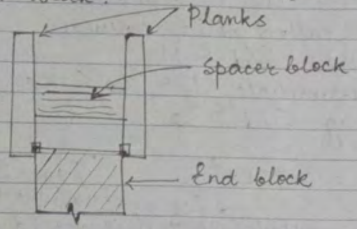
2. Box & Built up columns:-

Built-up columns are made of two or more rectangular sections (or planks) properly bolted together by metal connector.

The width of each planks should not be more than 5 times its thickness. The planks of built up column are fastened together at interval not exceeding 6 times of thickness of the individual plank.



3. Spaced Columns : These are special form of built up column of hollow construction consisting of two or more planks with their axis parallel and separated by spacer block and end-block.



(Q) A column 200 mm × 200 mm is made of sal wood. Determine the safe load for the column if unsupported length is
1) 2 m
2) 3.5 m
3) 5 m

Inside location
Solid Column

Take $f_{cp} = 10.6\ N/mm^2$
$E = 12700\ N/mm^2$

Soln:-
Case (I) : $S = 2m$
Size of Column = 200 mm × 200 mm
Unsupported length (S) = 2 m

$$\frac{S}{d} = \frac{2000}{200} = 10 < 11$$

Column is short
Permissible compressive stress $f_c = f_{cp} = 10.6\ N/mm^2$

(permissible ~~grade~~ stress for grade I & for inside location of compression parallel to grain)
$f_{cp} = 10.6\ N/mm^2$

Load = $10.6 \times 200 \times 200$
= 424000 N
= 424 kN

Case (II)
$$\frac{S}{d} = \frac{3.5 \times 10^3}{200} = 17.5 > 11$$

$$K_8 = 0.702\sqrt{\frac{E}{f_{cp}}}$$
$$= 0.702\sqrt{\frac{12700}{10.6}}$$
$$= 24.3$$

$11 < 17.5 < 24.3$
∴ Intermediate Column.

Permissible stress $f_c = f_{cp}\left[1 - \frac{1}{3}\left(\frac{S}{K_8 d}\right)^4\right]$
$$= 10.6\left[1 - \frac{1}{3}\left(\frac{17.5}{24.3}\right)^4\right]$$
$$= 9.65\ N/mm^2$$

Load = $9.65 \times 200 \times 200 = 386 \times 10^3$ N
= 386 kN

https://civinnovate.com/civil-engineering-notes/

Case (III)

$\frac{S}{d} = \frac{5\times10^3}{200} = 25 \quad 711$

$K_8 = 0.702\sqrt{\frac{E}{f_{cp}}}$

$= 24.3$

$\frac{S}{d} > K_8$

∴ Column is long column.

Permissible stress $f_c = \frac{0.329\,E}{\left(\frac{S}{d}\right)^2}$

$= \frac{0.329 \times 12700}{(25)^2}$

$= 6.69\ N/mm^2$

Load $= 6.69 \times 200 \times 200 = 267600\ N$

$= 267.6\ KN$

Q2) A column of 300 mm diameter is made of deodar wood. Determine safe axial load if its eff. length is 3m. $E = 94.8\times10^2\ N/mm^2$, $f_{cp} = 7.8\ N/mm^2$

Soln:-

Assume solid column & Inside location

From Table 1, for Deodar wood

$E = 94.8\times10^2\ N/mm^2$

Permissible stress for grade I & for inside location of compression parallel to grain,

$f_{cp} = 7.8\ N/mm^2$

For classification of column, check S/d ratio.

For finding 'd'

A square = A circle

$d^2 = \frac{\pi}{4} D^2$

$d = 265.87\ mm$

$\frac{S}{d} = \frac{3000}{265.87} = 11.28 > 11$

$K_8 = 0.702\sqrt{\frac{E}{f_{cp}}}$

$= 0.702\sqrt{\frac{94.8\times10^2}{7.8}} = 24.47$

$11 < 11.28 < 24.47$ (∴ Intermediate Column)

Permissible stress $f_c = f_{cp}\left[1 - \frac{1}{3}\left(\frac{S}{K_8 d}\right)^4\right]$

$= 7.68\ N/mm^2$

Load $= \frac{7.68 \times (265.87)^2}{10^3} = 543\ KN$

Note: For circular column, least lateral dimension is taken as equal to size of the equivalent square of same sectional area.

(Q3) Design solid deodar column for the following data. Load on the column is 500 kN, column to be located at open area. Effective length is 4m. Take select grade timber.

(For Deodar $E = 9480\ N/mm^2$, $f_{cp} = 6.9\ N/mm^2$

Soln :-

For select grade timber,

Permissible stresses $= f_{cp}' = 6.9 \times 1.16$

$= 8.004 \ N/mm^2$

$S = 4 m$

Assume column is short.

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

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

$d > \frac{4000}{11}$

$d > 363.64 \ mm$

Take $d = 400 mm$

$P = 500 \ kN$

$\text{Area} = \frac{500 \times 10^3}{8.004} = 62468.77 \ mm^2$

$\text{Other side} = \frac{62468.74}{400} = 156.17 \ mm$

Take 200 mm × 400 mm size column.

$S/d = 4000/200 = 20 > 11$

$K_8 = 0.702 \sqrt{\frac{E}{f_{cp}}} = 24.16$

$11 < \frac{S}{d} < K_8$ ($\because$ Intermediate column)

$f_c = f_{cp} \left[1 - \frac{1}{3} \left(\frac{S}{K_8 d} \right)^4 \right]$

$= 8.004 \left[1 - \frac{1}{3} \left(\frac{20}{24.16} \right)^4 \right] = 6.75 \ N/mm^2$

Permissible compressive stress $= 6.75 \ N/mm^2$

$\text{Area of column} = \frac{500 \times 10^3}{6.75}$

$= 74074.07 \ mm^2$

Smaller dimension $= 200 mm$

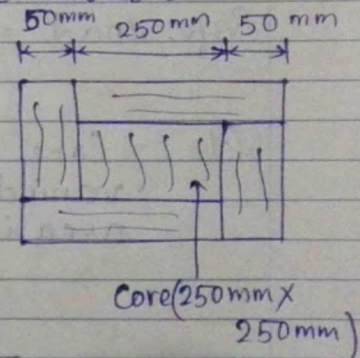
Larger " $= \frac{74074.07}{200} = 370.37 \ mm$

Provide 200 mm × 380 mm size solid column.

$\text{Load} = \frac{200 \times 380 \times 6.75}{1000} = 513 \ kN > 500 kN$

(OK)

(Q4) A built-up column of salwood consist of four planks of 50 mm × 300 mm with the central core of 250 mm × 250 mm. Calculate safe load capacity of column if its effective length is 4 m. Take sal wood ~~of~~ used in wet location.



Soln :-

Min. outer dimension of column

$d_1 = 300 + 50$

$= 350 \ mm$

Min. dimension of core

$d_2 = 250 \ mm$

$\frac{S}{\sqrt{d_1^2 + d_2^2}} = \frac{4 \times 10^3}{\sqrt{350^2 + 250^2}}$

$= 9.3 > 8$

$K_9 = \frac{\pi}{2} \sqrt{\frac{U \times E}{5 \, q \, f_{cp}}}$

$E = 126.7 \times 10^2 \ N/mm^2$

$f_{cp} = 7.7 \ N/mm^2$

for plank thickness 50mm (cl 7.6.2.5)

$U = 0.6$

$q = 1.0$

$$K_g = \frac{\pi}{2}\sqrt{\frac{0.6 \times 126.7 \times 10^2}{5 \times 1 \times 7.7}}$$

$= 22.07$

$$8 < \frac{s}{\sqrt{d_1^2 + d_2^2}} < K_g$$

∴ Column is Intermediate.

Permissible stress

$$f_c = q \cdot f_{cp}\left[1 - \frac{1}{3}\left(\frac{s}{K_g\left(\sqrt{d_1^2 + d_2^2}\right)}\right)^4\right]$$

$$= 1 \times 7.7\left[1 - \frac{1}{3}\left(\frac{9.83}{22.07}\right)^4\right]$$

$= 7.62\ N/mm^2$

$$\text{Safe Load} = \frac{7.62 \times 350 \times 350}{10^3} = 933.45\ KN$$

Note :- Core vayana vane BOX column vanincha & to find safe load, area is subtracted.

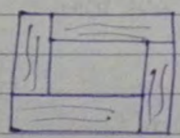


fig: Box Column.

(Q5) Design a built up sal wood column fabricated with 50 mm thick and 250 mm width planks to carry an axial load of 800KN. The effective length of column is 3.5 m. Take $E = 12700\ N/mm^2$, $f_{cp} = 10.6\ N/mm^2$, $U = 0.6$, $q = 1$

Soln :-

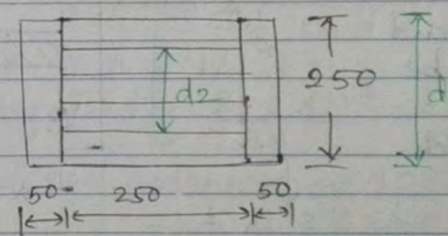
Take 90% of f_{cp}

$f_c = 0.9 \times 10.6 = 9.54\ N/mm^2$

Area of each plank $= 50 \times 250 = 12500\ mm^2$

$$\text{No. of planks} = \frac{800 \times 10^3}{12500 \times 9.54} = 6.7.$$

Provide 7 planks.



$d_1 = 250\ mm$

$d_2 = 3 \times 50 = 150\ mm$

$$\frac{s}{\sqrt{d_1^2 + d_2^2}} = \frac{3.5 \times 1000}{\sqrt{250^2 + 150^2}} = 12.005 > 8$$

$$K_g = \frac{\pi}{2}\sqrt{\frac{UE}{5q\, f_{cp}}} = 18.83$$

$$8 < \frac{s}{\sqrt{d_1^2 + d_2^2}} < K_g \text{ (Intermediate Column.)}$$

Permissible stress

$f_c = 9 \cdot f_{cp}\left[1 - \frac{1}{3}\left(\frac{S}{K_9\sqrt{d_1^2 + d_2^2}}\right)^4\right]$

$= 10.016\ N/mm^2$

$\therefore$ Safe load $= \frac{10.016 \times 350 \times 250}{10^3}$

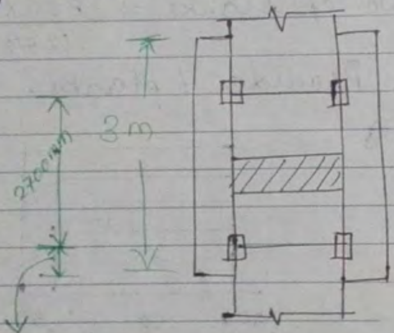
$= 876.4\ kN > 800\ kN$ (ok)

(Q6) A timber column 3m long has to support a load of 200 kN taking column to be of deodar wood, design column as a spaced column considering inside location.

Soln :-

$S = 3000\,mm$

Assume the end connector to be provided at $S/20$ from each end



$\frac{S}{20} = \frac{3000}{20}$

$= 150\,mm$

$\frac{S}{20} \rightarrow L/\sqrt{2.5}$

$\frac{S}{10}$ to $\frac{S}{20} \rightarrow \frac{L}{\sqrt{3}}$

Distance between centre of end connector

$l_1 = 3000 - 150 - 150$

$= 2700\,mm$

Effective length $s = l_1/\sqrt{2.5} = 2700/\sqrt{2.5} = 1708\,mm$

Assume thickness of plank $= 60\,mm = d$

Slenderness ratio $= \frac{s}{d}$

$= \frac{1708}{60} = 28.46 > 11$

$K_{10} = 0.702\sqrt{\frac{2.5\,E}{f_{cp}}}$

From IS 883

$E = 94.8 \times 10^2\ N/mm^2$

$f_{cp} = 7.8\ N/mm^2$

$K_{10} = 0.702\sqrt{\frac{2.5 \times 94.8 \times 10^2}{7.8}} = 38.69$

$11 < \frac{s}{d} < K_{10}$ (Intermediate Column)

$f_c = f_{cp}\left[1 - \frac{1}{3}\left(\frac{S}{K_{10}\,d}\right)^4\right]$

$= 7.04\ N/mm^2$

Area required $= \frac{200 \times 10^3}{7.04} = 28409.1\ mm^2$

Width of each plank $= \frac{28409.1}{2 \times 60}$

$= 236.74\,mm \simeq 240\,mm$

(Let us provide 2 planks of equal size).

Provide 2 planks of size 60mm × 240mm.

Check :

Safe load $= 7.04 \times 60 \times 240 \times 2 = 202.75\ kN$

$> 200\ kN$.

(ok).

Structural Members subjected to Bending & Axial Stress

For Compression

$$\frac{f_{ac}}{f_c} + \frac{f_{ab}}{f_b} \le 1$$

For tension

$$\frac{f_{at}}{f_t} + \frac{f_{ab}}{f_b} \le 1 .$$

$f_t \rightarrow$ perm. stress parallel to grain

$f_{ac} = \frac{P}{A}$, $f_{ab} = \frac{M}{Z} = \frac{M \times y}{I}$

$f_c =$, $f_b =$ from code (Permissible bending stress on extreme fibre)

(Q7) Check the safety of a square column 200mm × 200mm in c/s. ~~eff~~ The effective length of the column is 3m, the axial load and bending moment in the column are 25kN & 2kN m respectively. The material is timber of Salwood and the column is located inside the building.

Soln :- $E = 126.7 \times 10^2$ N/mm^2

$f_{cp} = 10.6$ N/mm^2

$f_b = 16.9$ N/mm^2

Check S/d ; find f_c ; $f_{ac} = \frac{P}{A}$; $f_{ab} = \frac{M}{Z}$

Check $\frac{f_{ac}}{f_c} + \frac{f_{ab}}{f_b} \le 1.0$.

Chapter 12 : DESIGN OF FLEXURE MEMBER

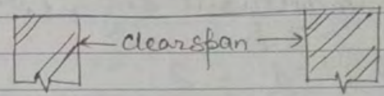
(a) Effective span

l_{eff} = Centre-to-centre distance between two supports

OR

Clear span + $\frac{\text{bearing length}}{2}$ + $\frac{\text{bearing length}}{2}$

whichever is minimum



(b) Bending stress

The max. bending stress due to dead load & superimposed load shall not exceed permissible stress specified by the code.

i.e. $f_{ab} = \frac{M}{Z} \le f_b$

The allowable stress of beam slightly decreases with the increase of depth so we have to modify the section modulus by a factor known as form factor.

If depth of beam ≥ 300 mm we should take a form factor K_3 for rectangular section, (Pg 11 Cl 7.5.4)

$$M = K_3 \cdot f_{ab} \cdot Z$$

Box Beam & I-section, $M = K_4 f_{ab} \cdot Z$

Solid circular, $K_5 = 1.18$; $M = \frac{f_{ab} \cdot Z_{sq}}{K_5}$

Square section, $K_6 = 1.414$; $M = \frac{f_{ab} \cdot Z_{sq}}{K_6}$

(c) Width of Beam
The min. width of beam shall not be less than 50mm or span/50 whichever is greater.

(d) Depth of Beam
Depth of beam shall not be taken 3 times of width of beam without any restrainer.

$$d \ngtr 3b$$

In practical, $\frac{d}{b} = 2$

(e) Shear (Pg 12 cl 7.5.7)

$$H = \frac{3V}{2bd} < f_{shear}$$

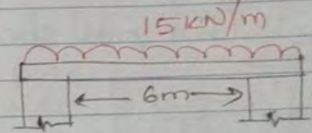
(f) Bearing
Flexural member directly supported on masonry wall or concrete wall, the bearing length provided at each end of the support shall not be less than 75mm.
In design, bearing length generally taken as 200 – 300 mm.

(g) Deflection
The max. deflection can be calculated for any type of loading then this calculated deflection shall not exceed permissible deflection specified by code.

$$\Delta_{calculated} \leq \Delta_{permissible}.$$

(Q) A timber beam having a clear span of 6m carries a UDL of 15 KN/m including the self weight of the beam assuming beam to be made of deodar wood, design the beam.

Soln:



Assume bearing length at each end be 300mm

Effective length $l_e = 6 + \frac{0.3}{2} + \frac{0.3}{2}$

$= 6.3\ m$

Max. B. M $= \frac{W l_e^2}{8} = \frac{15 \times 6.3^2}{8} = 74.42$ KNm

Max. permissible bending stress for Deodar

$f_b = 10.2\ N/mm^2$.

Now,
Section modulus required.

$$Z_{req} = \frac{M}{f_b} = \frac{74.42 \times 10^6}{10.2} = 7.3 \times 10^6\ mm^3$$

Width of Beam $\nless 50mm$, $\nless span/50$ } whichever is greater.

Width of beam = 50mm OR $\frac{6300}{50} = 126$ mm

Adopt b = 126mm
Provide 200mm width.

Section modulus $= Z = \frac{bd^2}{6} = 7.3 \times 10^6\ mm^3$

or, $200 \times d^2 = 6 \times 7.3 \times 10^6$

$\therefore\ d = 467.97\ mm$

Take 200mm x 470mm size beam.

Check $d \ngtr 3b$

$470 \ngtr 600$ (ok)

Check for Shear :- (Horizontal shear)

$$V = \frac{Wl_e}{2}\left[1 - \frac{2d}{l_e}\right]$$

$$= \frac{15 \times 6.3}{2}\left[1 - \frac{2 \times 0.47}{6.3}\right]$$

$$= 40.2\ kN$$

$$H = \frac{3V}{2bd} = \frac{3 \times 40.2 \times 10^3}{2 \times 200 \times 470} = 0.641\ N/mm^2$$

$$H = 0.641\ N/mm^2 < 0.7\ N/mm^2 \text{ (ok)}$$

Check for deflection :- (Cl 7.5.9 Pg 13)

Max deflection $\Delta_{max} = \frac{5}{384}\frac{Wl^4}{EI}$

$$= \frac{5}{384} \times \frac{15 \times (6.3 \times 10^3)^4}{94.8 \times 10^2 \times 1.73 \times 10^9}$$

$$= 18.76\ mm$$

$$I = \frac{bd^3}{12} = \frac{200 \times \cancel{470}^3}{12} = 1.73 \times 10^9$$

$$\Delta_{permissible} = \frac{span}{240} = \frac{6300}{240} = 26.25\ mm$$

$\Delta_{max} < \Delta_{permissible}$ (ok).

Check for Bearing :-

$$\text{Bearing stress} = \frac{\text{Reaction}}{\text{Bearing area}}$$

$$\text{Reaction} = \frac{Wl_e}{2} = \frac{15 \times 6.3}{2} = 47.25\ kN$$

Bearing area $= 200 \times \cancel{470}\ 300 = 60000\ mm^2$

$$\text{Bearing stress} = \frac{47.25 \times 10^3\ N}{60000\ mm^2}$$

$$= 0.7875\ N/mm^2 < 2.7\ N/mm^2$$

Permissible bearing stress = Permissible compressive stress perpendicular to grain

$= 2.7\ N/mm^2$

Safe (ok)

(Q) Design one of the intermediate Deodar joist used in timber roof of a hall from the following data:

Spacing of joist (c/c) = 50 cm

Clear span of joist = 2m

Bearing at each end = 8cm

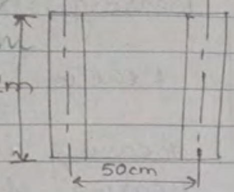
DL = $2\ kN/m^2$

LL = $1.5\ kN/m^2$

Timber of standard grade & inside location.

Q) Design one of the intermediate Deodar joist used in the timber roof of a hall front following data.

Spacing of joist (c/c) = 50 cm
Clear span of joist = 2 m
Bearing at each end = 8 cm



Solution :—

For Deodar, $E = 94.8 \times 10^2 \ N/mm^2$

Dead load = $2 \times 0.5 = 1 \ kN/m$

Live load = $1.5 \times 0.5 = 0.75 \ kN/m$

Assuming self wt. of beam = $0.1 \ kN/m$

Total load = $W = 1.85 \ kN/m$

Effective span l_{eff} = Clear span + Bearing length

$\therefore L_{eff} = 2 + 0.08 = 2.08 \ m$

Max. B.M. = $\frac{W \cdot l_e^2}{8} = \frac{1.85 \times 2.08^2}{8} = 1.0005 \ kNm$

Max. permissible stress for Deodar $f_b = 10.2 \ N/mm^2$

Section modulus required

$Z_{req} = \frac{M}{f_b} = 98086.27 \ mm^3$

Now, $Z = \frac{bd^2}{6}$

or, $bd^2 = 588517.65$

Taking $b/d = 0.5 \Rightarrow b = 0.5d$

$\therefore d = 105.58 \ mm$ & $b = 52.79 \ mm$.

Min width of beam $b > 50mm$ or $l/50 (= 2.08/50)$

$b > 50 \ mm$ or $41.6 \ mm$

$52.79 > 50 \ mm$ (ok)

Provide $b = 60mm$ & $d = 120mm$ Deodar beam.

Check for shear

Shear $V = \frac{W l_e}{2}\left[1 - \frac{2D}{l_e}\right] = \frac{1.85 \times 2.08}{2}\left[1 - \frac{2 \times 0.14}{2.08}\right]$

$\therefore V = 1.665 \ kN$

Shear stress $(\tau) = \frac{3V}{2bD} = \frac{3 \times 1.665 \times 10^3}{2 \times 60 \times 120} = 0.347 \ N/mm^2$

From code, permissible shear stress = $0.7 N/mm^2 > 0.347 N/mm^2$
(OK) safe in shear.

Check for Deflection :-

$\Delta_{calculated} = \frac{5 W l_e^4}{384 EI} = \frac{5 \times 1.85 \times (2080)^4}{384 \times 94.8 \times 10^2 \times 8.64 \times 10^6}$

$I = \frac{60 \times 120^3}{12} = 8.64 \times 10^6 \ mm^4$

$\therefore \Delta = 5.505 \ mm$

$\Delta_{permissible} = span/240 = 2080/240 = 8.67 mm > 5.505 mm$
(ok).

Check for Bearing :-

Bearing stress = $\frac{\text{Reaction}}{\text{Bearing area}}$

Reaction = $\frac{W l_e}{2} = \frac{1.85 \times 2080}{2} = 1924 N$

Bearing area provided = $60 \times 80 = 4800 \ mm^2$

Bearing stress = $\frac{1924}{4800}$

$= 0.4 \ N/mm^2$

Permissible bearing stress = $2.7 N/mm^2 > 0.4 N/mm^2$
(ok).

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