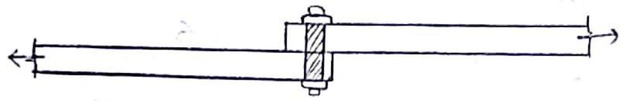
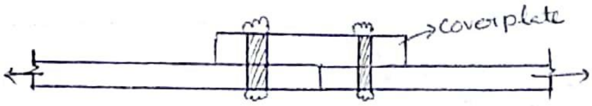
DESIGN OF BOLTED CONNECTION

- Types of joint:

  - Lap joint



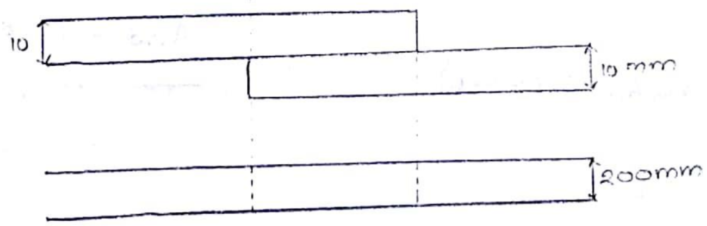
  - Butt joint



- failure of joint:

  - Shear failure of bolt
  - Bearing failure of bolt
  - Bearing failure of plate
  - Tension failure of bolt
  - Tension or tearing failure of bolt plate (yield, rupture failure)
  - Block shear failure

Q Two plates of 10mm thickness & width 200mm are to be joined using high strength friction bolt (HSFG) & 20mm diameter. Design the joint for maximum strength. Given that 4.6 (x.y) grade bolts & Fe 410 steel plate is used.



Step 1: Given data

Thickness of plate = 10 mm

width of plate = 200 mm

Bolt diameter, d = 20 mm

ultimate strength of ~~bolt~~ Steel, $f_u = 410\ N/mm^2$

$x \cdot y = 4.6$

ultimate strength of bolt, f_{ub}:

$$\boxed{x = \frac{1}{10} f_{ub}\ kgf/mm^2} \quad , \quad x = 4 \ , \ y = 0.6$$

$\boxed{kgf/mm^2 \rightarrow N/mm^2 \ * 9.81 \text{ or } 10}$

$\therefore f_{ub} = 4 \times 10\ kgf/mm^2$

$= 40 \times 10\ N/mm^2 = \underline{400\ N/mm^2}$

Yield strength of bolt, f_{yb}:

$$\boxed{f_{yb} = y\, f_{ub}}$$

$f_{yb} = 0.6 * 400$

$= \underline{240\ N/mm^2}$

Step 2: Strength calculation of bolt

a) Design shear strength of bolt

(cl. 10.3.3, Pg 75, IS 800:2007)

V_{dsb} : design shear strength of bolt

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

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

$\boxed{A_{nb} = 0.78\ A_{sb}}$

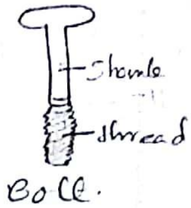
$\boxed{A_{nb} = 0.78 \times \frac{\pi}{4} d^2}$

$f_{ub} = 400\ N/mm^2$

n_n = no. of shear plane with thread

$n_n = 1$

n_s = no. of shear planes without thread



$n_s = 0$

$A_{sb} = \frac{\pi}{4} \times 20^2 = 314.159 \text{ mm}^2$

$A_{nb} = 0.78 A_{sb} = 245.04 \text{ mm}^2$

γ_{mb} : Partial safety factor , $\gamma_{mb} = 1.25$ (code)

$$\therefore V_{dsb} = \frac{400}{\sqrt{3} \times 1.25} (1 \times 245.04 + 0 \times 314.159)$$

$$= 45272.39 \text{ N}$$

$$= 45.27 \text{ kN}$$

b) Bearing strength of Bolt

(Pg. 75, Cl. 10.3.4, IS 800:2007)

V_{dpb} : design bearing capacity of bolt.

$$V_{dpb} = \frac{V_{npb}}{\gamma_{mb}}$$

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

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

e : end distance

p : pitch distance ~~(Fig)~~

d_0 : dia of hole (Table 19, Pg 73, IS 800:2007)

d : dia of bolt

t : thickness of plate

$\boxed{e_{min} = 1.5 \times d_0}$

$\boxed{d_0 = \text{bolt } \phi + \text{clearance}}$ $\Rightarrow d_0 = 20 + 2$

$= 22 \text{mm}$

$e_{min} = 33 \text{ mm} \approx 35 \text{ mm}$

$$k_b = \begin{cases} \dfrac{e}{3d_o} = 0.53 \\ \dfrac{P}{3d_o} - 0.25 = 0.507 \checkmark \\ \dfrac{f_{ub}}{f_u} = \dfrac{400}{410} = 0.97 \\ 1 \end{cases}$$

$$\boxed{P_{min} = 2.5\,d}$$

$P_{min} = 2.5 \times 20 = 50$ mm

$$V_{dpb} = \frac{2.5 \times 0.507 \times 20 \times 10 \times 410}{1.25}$$

$= 83148$ N

$= 83.148$ kN

~~consider~~

Step 3: Number of bolts

Strength of bolt = smaller of V_{dsb} & V_{dpb}

$= 45.27$ kN

$$\boxed{\text{No. of bolts required} = \frac{\text{Strength of plate}}{\text{Strength of bolt}}}$$

(Strength of plate : Pg: 32, cla 6.2, IS 800:2007)

Strength of plate,

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

A_g : gross area of c/s, $A_g = 200 \times 10 = 2000$ mm²

f_y : yield strength, $f_y = 250$ N/mm² (code)

γ_{mo} : Partial safety factor in tension by yielding, $\gamma_{mo} = 1.1$ (code)

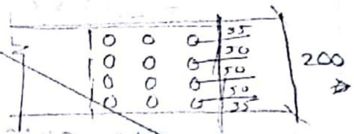
$T_{dg} = 454545.45 \text{ N}$

$= 454.54 \text{ kN}$

$\therefore$ No. of bolts $= \dfrac{454.54}{45.27}$

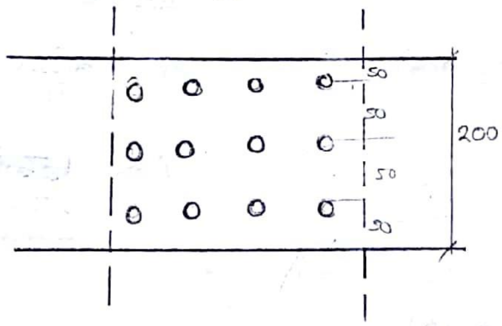
$= 10.04 \approx 12$

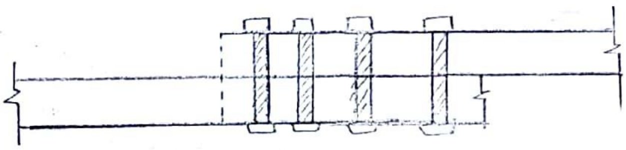
$\therefore$ Provide 12 bolts.



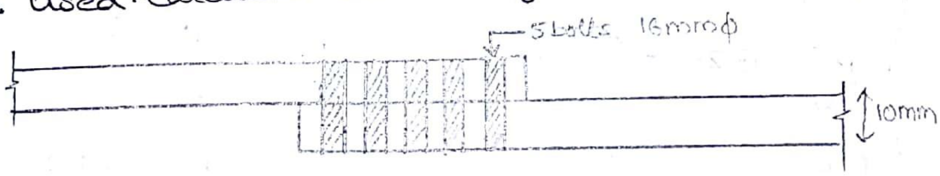
~~35+50+50+50+35 = 220 > 200 = Not possible~~

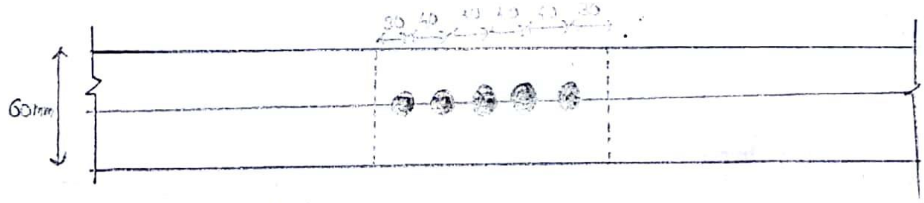
Step 4 : Detailing





Q Two plates 10mm X 60mm are connected in lap joint with 5 nos. 16 mm ϕ bolt of grade 4.6 & 410 grade plates are used. Calculate the strength & efficiency of joint.





Step 1 : Given data

Thickness of plate = 10 mm

width of plate = 60 mm

Bolt dia, d = 16 mm

No. of bolt = 5

$f_u = 410\ N/mm^2$

$x \cdot y = 4 \cdot 6$

$f_y = 250\ N/mm^2$

$x = \frac{1}{10} f_{ub}\ kgf/mm^2$

$f_{ub} = 400\ N/mm^2$

$f_{yb} = y\ f_{ub}$

$= 0.6 \times 400 = 240\ N/mm^2$

Step 2: Strength calculation of bolt

a) Design shear strength of bolt

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

$A_{sb} = \frac{\pi}{4} \times 16^2 = 201.06\ mm^2$

~~= 1005.3 mm²~~

$A_{nb} = \cancel{156.82}\ mm^2$

~~= 784.134 mm²~~

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

$$= \frac{400}{\sqrt{3} \times 1.25} (1 \times 156.82 + 0)$$

$$= \cancel{144.87}\ \cancel{28.97\ kN} = 28.97\ kN$$

b) Design of bearing strength bolt

$d_0 = 16 + 2$

$= 18\ mm$

$e = 30\ mm$

$p = 40\ mm$

$d = 16$

$t = 10$

$f_u = 410.$

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

$$k_b = \begin{cases} \frac{e}{3d_0} = 0.55 \\ \frac{p}{3d_0} - 0.25 = 0.49 \checkmark \\ \frac{f_{ub}}{f_u} = 0.975 \\ 1 \end{cases}$$

$V_{dpb} = 64.288\ kN$

∴ strength of 1 bolt $= 28.97\ kN$

∴ strength of 5 bolts $= 5 \times 28.974$

$= 144.87\ kN$

3/8/'18

Step 3: Strength of plate.
~~Strength of plate~~

(Pg: 32, Section 6, clause 6.2, IS 800:2007)

a) Strength due to yielding of gross section:

$$T_{dg} = \frac{A_g f_y}{\gamma_{mo}} \qquad \gamma_{mo} = 1.1$$

$$A_g = 600 \times 10 = \underline{6000\,mm^2}$$

$$T_{dg} = \frac{600 \times 250}{1.1} = \underline{136.36\,kN}$$

b) Strength due to rupture at critical section:

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

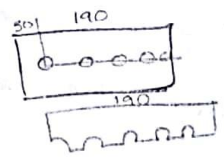
~~An = (18x8)xn~~ cross section:

$$A_n = (60 - 18) \times 10 = 420$$

$$= \frac{0.9 \times 420 \times 410}{1.25}$$

$$= \underline{123.984\,kN}$$

∴ strength of plate = $\underline{123.984\,kN}$ (least value)



Step 4: Block shear failure:

(Pg: 33, clause 6.4, IS 800:2007)

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

vertical:

$$A_{vg} = 190 \times 10 = \underline{1900\,mm^2}$$

No. of bolt holes

$$A_{vn} = (90 - (6.5 \times 18)) \times 10 = \underline{1090\,mm^2}$$

transverse:

$$A_{tg} = 30 \times 10 = \underline{300\,mm^2}$$

$$A_{tn} = \left(30 - \frac{18}{2}\right) 10 = \underline{210\,mm^2}$$

$$= \frac{1900 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 210 \times 410}{1.25}$$

$$= \underline{311.30\,kN}$$

or

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

$$= \frac{0.9 \times 1090 \times 410}{\sqrt{3} \times 1.25} + \frac{300 \times 250}{1.1}$$

$$= \underline{253.95\,kN}$$

Step 5: Strength of joint

Strength of joint = lose of strength of bolt & plate

$= 123.98\,kN$

step 6: Efficiency of joint

$$\eta = \frac{\text{Strength of joint}}{\text{Strength of solid plate}} \times 100$$

$$\text{Strength of solid plate} = \frac{0.9 A_g f_u}{\gamma_{m1}}$$

$$= \frac{0.9 \times 600 \times 410}{1.25}$$

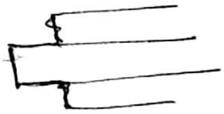
$$= 177.120\,kN$$

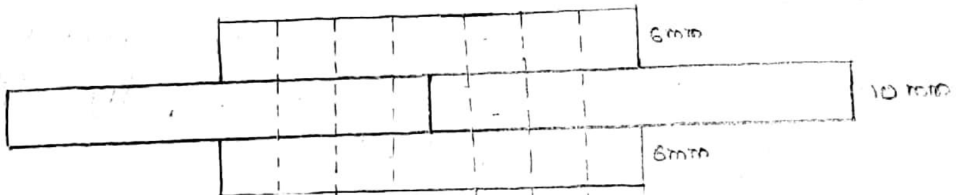
$$\eta = \frac{123.98}{177.120} \times 100$$

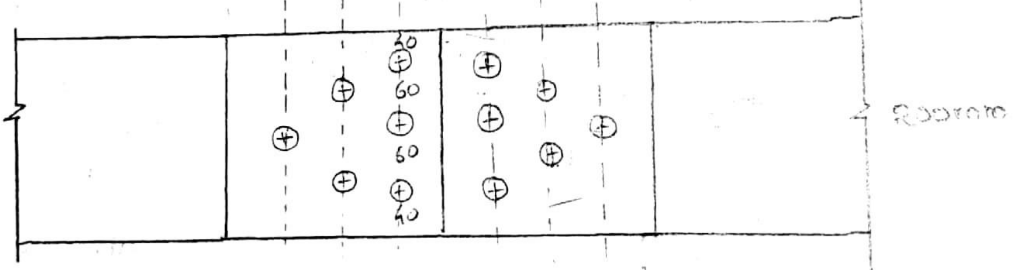
$$= 69.99 \approx 70\%$$

Butt joint:

Q Find the max. load this joint can resist provided that 4.6 grade 20mm dia bolt & Fe 410 steel is given. Assume the 2 cover plates act together.







Step 1: Given data

$d = 20\,mm$

$d_0 = 22\,mm$

$f_{ub} = 400\,N/mm^2$

$f_u = 410\,N/mm^2$

$f_{yb} = 240\,N/mm^2$

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

Step 2 : Strength of bolt

a) Double shear strength.

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

Assume that both shear plates passes through the thread position, $\therefore n_n = 2$

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

$$= \underline{90.54\ kN}$$

b) Design bearing strength:

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

$t = 10$ or $\underbrace{6+6}_{12}$ } least $= 10mm$

$$= \frac{2.5 \times 0.6 \times 20 \times 10 \times 410}{1.25}$$

$$= \underline{98.4\ kN}$$

$$k_b = \begin{cases} \frac{e}{3d_o} = \frac{40}{3 \times 22} = 0.60 \\ \frac{p}{3d_o} - 0.25 = \frac{60}{3 \times 22} - 0.25 = 0.659 \\ \frac{f_{ub}}{f_u} = \frac{400}{410} = 0.97 \\ 1 \end{cases}$$

Shear of 1 bolt $= 90.54\ kN$

Shear of 6 bolt $= \underline{543.24\ kN}$.

Step 3 : Strength of plate.

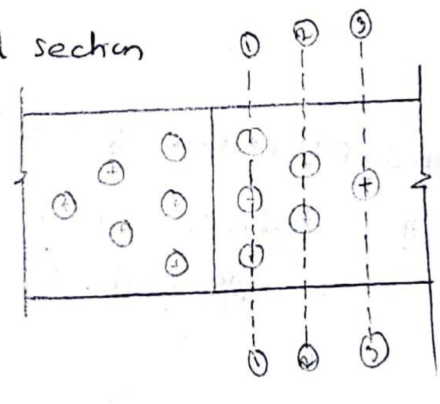
a) strength ~~due~~ due to yielding of cross section

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

$A_g = 200 \times 10$
$= \underline{2000 mm^2}$

$$= \underline{454.54\ kN}$$

4/8/18 b) Strength due to rupture of critical section



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

At section 3-3 $T_{dn} = \frac{0.9(200-22)10 \times 410}{1.25}$

$$= \underline{525.456\ kN}$$

At Section 2-2, $T_{dn} = \frac{0.9(200-(2\times22))\times10\times410}{1.25}$ + Strength of 1 bolt (balance)

$= 460.512 + 90.54$

$= 551.052$ kN

At Section 1-1, $T_{dn} = \frac{0.9(200-(3\times22))10\times410}{1.25}$ + Strength of 3 bolts (balance)

$= 395.568 + (90.54\times3)$

$= 667.188$ kN

Step 4 : Strength of cover plate

Only Section 1-1 is critical for cover plate, because even at Section 2-2 & 3-3 the other bolts hold the main plate. But at Section 1-1 the main plate separates.

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

$$= \frac{0.9(200-(3\times22))12\times410}{1.25}$$

$$= 474.6816 \text{ kN}$$

$\therefore$ Strength of joint = least of strength of bolt, strength of main plate & strength of cover plate.

$= 543.26$ or 454.54 or 474.6816

$= 454.54$ kN

Step 5 : Efficiency of joint

$$\eta = \frac{\text{Strength of joint}}{\text{Strength of solid main plate}}\times100$$

Strength of main solid plate $= \frac{0.9 A_g f_u}{\gamma_{ml}}$

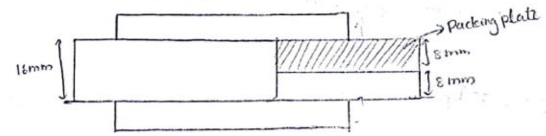
$= \frac{0.9 \times (200 \times 10) \times 410}{1.25}$

$= 590.4$ kN

$\eta = \frac{454.54}{590.4} \times 100$

$= 76.98\% \approx 77\%$

Q Design a double cover butt joint to connect 2 plates of 16 mm & 8 mm thickness to take a tensile force of 500kN.



Step1 : Designing the breadth of plate   Assume 22m ϕ bolt d:

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

$500 \times 10^3 = \frac{A_g \times 250}{1.1}$

$A_g = 2200 \text{ mm}^2$

To get max. breadth divide with least thickness.

$\therefore B = \frac{2200}{8} = 275$

≈ 300 mm

$\therefore$ 1st plate : 300×8

2nd plate : 300×16

Step 2: Strength of bolt

a) Shear strength capacity

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

Assume 22mmϕ bolts.

$n_n = 2$

$n_s = 0$

$$A_{nb} = 0.78 \frac{\pi}{4} \times 22^2$$

$$= \underline{\underline{296.503\ mm^2}}$$

$$V_{dsb} = \frac{400}{\sqrt{3} \times 1.25}(2 \times 296.503 + 0)$$

$$= \underline{\underline{109.559\ kN}}$$

~~cl 10.3.3~~ (β_{pk} : Pg 75, cl. 10.3.3.3, IS 800:2007)

$$\beta_{pk} = 1 - 0.0125\, t_{pk}^{\;8mm}$$

$$= \underline{\underline{0.9}}$$

$\therefore$ Actual $V_{dsb} = \underline{\underline{98.6031\ kN}}$ (0.9×109.559)

b) Design of bearing strength of bolt

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

$d = 2\cancel{0}2$ mm (assume)

$e = 1.5 d_0^{\;24}$

$= 3\cancel{8} \approx \underline{\underline{\cancel{4}40\ mm}}$

$$k_b = \begin{cases} \dfrac{e}{3d_0} = \cancel{0.60}0.555 \\ \dfrac{P}{3d_0} - 0.25 = 0.5\cancel{0}3 \\ \dfrac{f_{ub}}{f_u} = \dfrac{400}{410} = 0.97 \\ 1 \end{cases}$$

$P = 2.5d$

$= 2.5 \times 2\cancel{2} = \underline{55mm}$

$\approx \underline{\underline{60\,mm}}$

$$V_{dpb} = \frac{2.5 \times \cancel{0.583}^{\,0.555} \times 22 \times 8 \times 410}{1.25}$$

$$= \cancel{74.036\,kN} \quad \cancel{84.186\,kN}$$

$$= 80.097\,kN$$

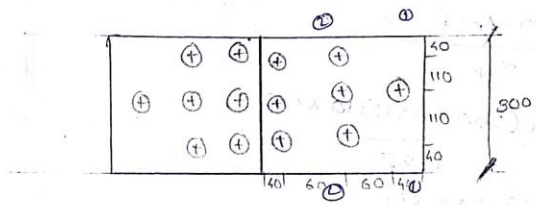
$\therefore$ Strength of ~~the~~ 1 bolt $= 80.097\,kN$

$$\therefore \boxed{\text{no. of bolts} = \frac{\text{Factored load}}{\text{Strength of 1 bolt}}}$$

$$= \frac{500}{80.097} = 6.24$$

$\approx$ ~~8~~ 7 bolts

Step 3 :



Step 4 : Design of cover plate

Let t be the thickness of 1 cover plate.

$$T_{dn} = \frac{0.9 \times A_n \times F_u}{\gamma_{mb}}$$

$$500 \times 10^3 = \frac{0.9(300-(3 \times 24))2t \times 410}{1.25}$$

~~t = 10.37~~

$t = 3.71 \approx$ provide 8mm plate

Step 5 : Check

a) Strength due to yielding of cross section $\qquad A_g = 300 \times 8$

$$T_{dg} = \frac{A_g f_y}{\gamma_{m0}} = \frac{2400 \times 250}{1.1}$$

$$= 545.45\,kN$$

~~Boif~~ ~~Subj~~

b) Strength due to rupture of critical section

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

$$①-① = \frac{0.9(300-24)8 \times 410}{1.25} = \underline{\underline{651.8016 \text{ kN}}}$$

$$②-② = \frac{0.9(300-24\times3)8 \times 410}{1.25} + \text{balance of bolt } (4\times80.097)$$

$$= \underline{\underline{858.8328 \text{ kN}}}$$

c) Strength of cover plate

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

$$= \frac{0.9(300-24\times3)10 \times 410}{1.25}$$

$$= \underline{\underline{673.056 \text{ kN}}}$$

$\therefore$ Strength of Joint = least of Strength of bolt, strength of main plate & strength of cover plate

$$= 560.679 \text{ or } 545.45 \text{ or } 673.056$$

$$= \underline{\underline{545.45 \text{ kN}}}.$$

$$\text{Efficiency}, \eta = \frac{\text{Strength of joint}}{\text{strength of solid main plate}} \times 100$$

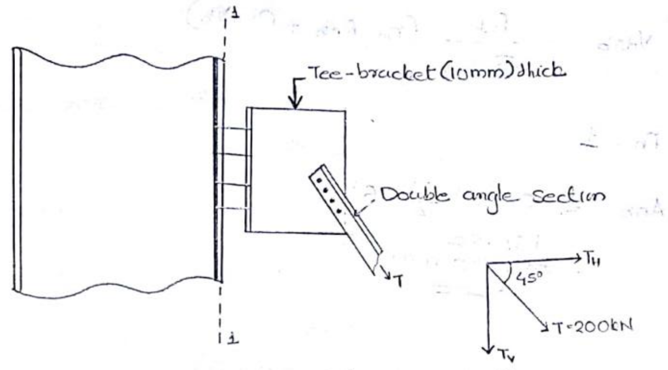
$$\text{Strength of solid main plate} = \frac{0.9 A_g f_u}{\gamma_{ml}}$$

$$= \frac{0.9(300\times8)410}{1.25} = \underline{\underline{708.48 \text{ kN}}}$$

$$\eta = \underline{\underline{77\%}}$$

6/8/'18 BOLT SUBJECTED TO COMBINED SHEAR & TENSION

Q Determine whether the joint shown in the fig. is safe or not. 8 No. of 16mm dia bolt of grade 4.6 have been used for making the connection at section 1-1. Neglect the effect of prying action. Also find the number of 16mm dia bolt of grade 4.6 to connect the double angle section (8mm thick each) member with the web of the T-bracket



solution:
Step 1: Given data

$d = 16mm \Rightarrow d_0 = 18mm$

$f_u = 410\ N/mm^2$

$f_{ub} = 400\ N/mm^2$

$f_{yb} = 240\ N/mm^2$

$f_y = 250\ N/mm^2$

$T = 200kN$

$T_H \rightarrow$ causes tension in bolt $= T\cos\theta = 141.42\ kN$

$T_V \rightarrow$ causes shear in bolt $= T\sin\theta = 141.42\ kN$

Tension in 1 bolt $= \frac{141.42}{8}$

$T_b = 17.68\ kN$

Shear in 1 bolt $= \frac{141.42}{8}$

$V_{sb} = 17.68\ kN$

(a) Step 2 : Bolt subjected to combined shear & tension

(cl. 3.6, Pg 76, IS 800 : 2007)

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

a) strength of bolt :

→ Shear strength capacity

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

$n_n = 1$

$$A_{nb} = 0.78 \times \frac{\pi}{4} \times 18^2$$

$$= \cancel{39.20}\ 156.82\ mm^2$$

$$V_{dsb} = \frac{400}{\sqrt{3} \times 1.25} (1 \times 39.20 + 0)$$

$$= \cancel{7.243}\ 29\ kN$$

Shear for 1 bolt = $\cancel{7.243}$ 29 kN   29 kN = V_{db}

~~Shear for 8 bolt = 57.94 kN~~ 232

∴ ~~V_{db} = 57.94 kN~~

b) Tension capacity

$$T_{db} = \frac{T_{nb}}{\gamma_{mb}}$$

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

$$= 56.4552 < 54.83$$

∠

$$T_{db} = \frac{54.83}{1.25}$$

$$= 43.86 \text{ kN}$$

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

$$\left(\frac{17.68}{29}\right)^2 + \left(\frac{17.68}{43.86}\right)^2 \le 1.0$$

$$0.534 < 1.0$$

⇒ Hence the joint is safe at section ①-①.

(b) Strength of bolt:

a) Shear strength capacity

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

$$= \frac{400}{\sqrt{3} \times 1.25}\left(2 \times \left(0.78 \times \frac{\pi}{4} \times 16^2\right) + 0\right)$$

$$= 58 \text{ kN}$$

b) Design of bearing strength of bolt

$e = 1.5 d_0$
$= 27 \approx 30$ mm

$P = 2.5d = 50$ mm

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

$$k_b = \begin{cases} \dfrac{e}{3d_0} = 0.\cancel{556} \\ \dfrac{P}{3d_0} - 0.25 = 0.67 \\ \dfrac{f_{ub}}{f_u} = 0.97 \\ 1 \end{cases}$$

$$= \frac{2.5 \times 0.67 \times 16 \times 8 \times 410}{1.25}$$

$$\approx \cancel{51.4804 \text{ kN}}$$

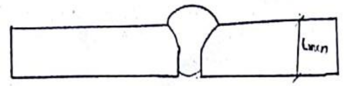
$$= 87.9 \text{ kN}$$

$$\text{No. of bolt} = \frac{200}{58} = 3.44$$

$$\approx 4 \text{ bolts}$$

Design of Welded Connection

Fillet:



$t = \frac{5}{8} t_{min}$, for incomplete

$t = t_{min}$, for complete.

Strength of 1mm weld = Stress x area

$= \frac{f_u \times (t \times 1)}{\gamma_{mw}}$

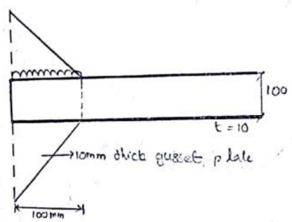
$\gamma_{mw} = 1.25$, for shop weld.

$= 1.5$, for field weld

throught thickness, $t = 0.7S$.

Q A plate 100mm X 10mm size has to be welded to a gusset plate using 8mm size fillet well. The length of the gusset plate thickness is connection is limited to 100mm. The gusset plate thickness is 10mm.

Solution.



Step 1 : Data Given

Step 2: Maximum force that can be resisted by the plate.

Max. force that can be resisted by plate due to yielding, T_{dg} $= \frac{f_y A_g}{\gamma_{m0}}$

$= \frac{250 (100 \times 10)}{1.1}$

$= 227.27$ kN

Step 3: Design of weld.

Consider 1mm length of the weld.

Size of weld, $s = 8$mm

$\therefore$ throught thickness, $t = 0.7 s$

$= 0.7 \times 8 = 5.6$mm.

~~design of~~ design strength of 1mm length of weld

$= \frac{f_u (t \times 1)}{\sqrt{3} \times \gamma_{mw}}$

Assume shop weld, $\gamma_{mw} = 1.25$

$\therefore$ design strength $= \frac{410 (5.6 \times 1)}{\sqrt{3} \times 1.25}$

$= 1.06$ kN

$\therefore$ Total length of weld required $= \frac{227.27}{1.06}$

$= 214.40$ mm

~~Here~~ It is given the question the length of the lap should be limited to 100mm, so providing weld on both sides 200mm can be given lap connection. Remaing 14.4 mm should be provided as plug weld.

$\therefore$ force to be resisted by the plug weld

$= 14.4 \times 1.06$

$= \underline{15.264 \text{ kN}}$

Let d be the diameter of the plug weld,

$$\frac{f_u \text{ (area)}}{\sqrt{3}\, \gamma_{mw}} = 15.26 \times 10^3$$

$$\frac{410 \times \frac{\pi}{4} \times d^2}{\sqrt{3} \times 1.25} = 15.26 \times 10^3$$

$\underline{d = 10.3 \text{ mm}}$.

Let us provide $\underline{12 \text{ mm}}$. dia plug weld.

8/8/18

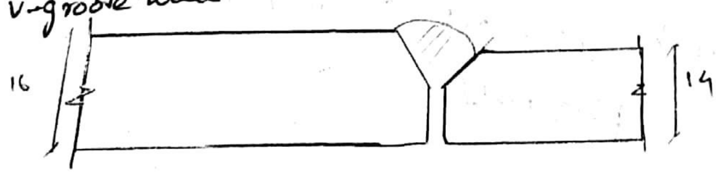
Q Two plates of 16mm & 14mm thickness are to be joined by a groove weld (butt weld) as shown in the figure. The joint is subjected to a factored tensile force of 430 kN. Due to some reasons the effective length of the weld that could be provided was 175mm only. check the safety of the joint if:

a) single V-groove weld is provided.

b) Double V-groove weld is provide.

Assume the plates to be shop welded

a) Single V-groove weld



a) Single V-groove weld.

$F_t = 430\,kN$

$l_w = 175\,mm$

$t_e = \frac{5}{8} \times t_{min}$

$= \frac{5}{8} \times 14 = \underline{8.75\,mm}$

Strength of 1 mm weld $= \frac{\text{stress} \times \text{area}}{\gamma_{mw}}$

$= \frac{f_y \times (t \times 1)}{\gamma_{mw}}$

$= \frac{250\,(8.75 \times 1)}{1.25}$

$= 1750\,N = \underline{1.75\,kN}$

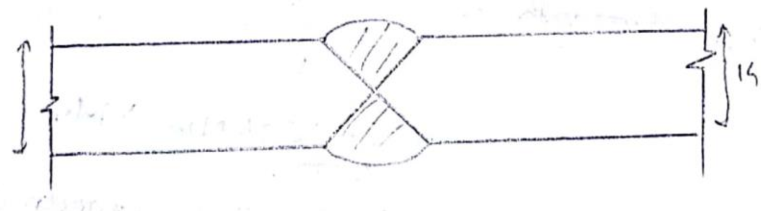
∴ Effective length of weld,

$l_w = \frac{430}{1.75}$

$= \underline{245.71\,mm}$

So the weld is $\underline{\text{unsafe}}$ & inadequate

b) Double V-groove weld.



$t_e = t_{min} = \underline{14\,mm}$

Strength of 1 mm weld $= \frac{f_y\,(t \times 1)}{\gamma_{mw}}$

$= \frac{250\,(14 \times 1)}{1.25}$

$= \underline{2.8\,kN}$

∴ Effective length of weld,

$l_w = \frac{430}{2.8} = \underline{153.57\,mm}$

So the weld is safe & adequate

Q A groove weld is to connect two plates $\overset{b}{180\,mm} \times \overset{t}{18\,mm}$ each. Determine the design bending strength of the joint if it is subjected to a moment of 13 kN m. Also determine the adequacy of the joint if the shear force at the joint is 200kN. Assume the welds to be double u-shop welded.

(a) 1) Data presentation:

$M = 13\,kNm \Rightarrow M_u = 19.5\,kNm$

$V = 200\,kN \Rightarrow V_u = 300\,kNm$

$\gamma_{mw} = 1.25$

2) Design bending strength $= \dfrac{1.2\,Z_e f_y}{\gamma_{mo}}$

Z_e: section modulus.

$Z_e = \dfrac{bd^2}{6} = \dfrac{18 \times 180^2}{6}$

$= 97200\,mm^3$

Design bending strength $= \dfrac{1.2 \times 97200 \times 250}{1.1} \times 10^{-6}$

$= 26.509\,kNm > M_u$

⇒ Hence the weld is safe against bending.

(b) Design shear strength for 1mm weld = Stress x area

$= \dfrac{\dfrac{f_u}{\sqrt{3}}(t \times 1)}{\gamma_{mw}} = \dfrac{\dfrac{410}{\sqrt{3}}(18 \times 1)}{1.25}$ double U.

$= \cancel{10.72}\ 3.4\cancel{08}\,N$

∴ Effective length of weld $= \frac{300}{3.408}$

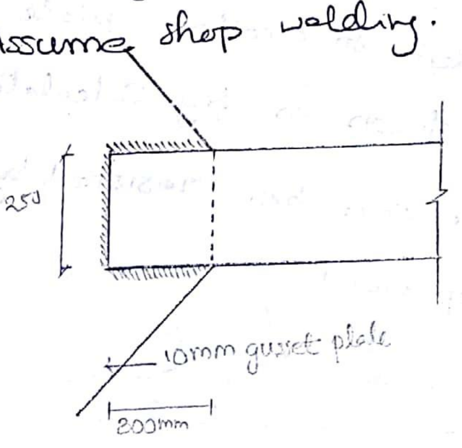
$= 88.03$ mm < 180 mm

⇒ So provide throughout the breadth for 180mm.

Q. A tie member in a truss girder is 250mm x 14 mm in size. It is welded to a 10mm thick gusset plate by fillet weld. The overlap of the member is 300mm, & the weld size is 6mm. a) Determine the design strength of joint if the weld is done as shown in fig.

b) what is the increase in strength of the joint if welding is done all around.

Assume shop welding.



Size : 250 mm x 15mm

10mm gusset plate

Overlap = 300mm

S = 6mm

a) S = 6mm

Throat thickness, $t = 0.7 S$

$= 4.2$ mm

Effective length of ~~throat~~ weld, $l_w = 300 + 250 + 300$

$= 850$mm

∴ Design strength of weld, $l_w = 300 + 250 + 300$

$= 850$mm

Design strength of weld = $\dfrac{(\text{stress})\ \text{area}}{\gamma_{mw}}$

$$= \frac{\frac{f_y}{\sqrt{3}} \times (l_w \times t_e)}{\gamma_{mw}}$$

$$= \frac{\frac{410}{\sqrt{3}} \times 850 \times 4.2}{1.25} = 676.054\,kN$$

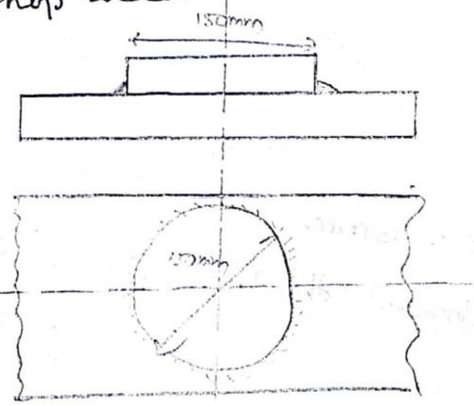
b) $l_w = 2(300+250) = 1100mm$

design strength of weld = $\dfrac{\frac{f_y}{\sqrt{3}} \times l_w \times t_e}{\gamma_{mw}}$

$$= \frac{410 \times 1100 \times 4.2}{\sqrt{3} \times 1.25} = 874.893\,kN$$

⇒ Increase in strength = 199.84 kN

Q A circular plate 150mm dia is welded to another plate by means of 6mm fillet weld as shown in fig. Calculate the ultimate twisting moment that can be resisted by the weld use Fe410 steel & shop weld.



$S = 6mm$

circular plate = 150mmϕ

Throat thickness = $0.7S$

$= 0.7 \times 6 = 4.2mm$

Effective length of plate = l_w

$= \pi d$ (Perimeter of circle)

$= 150\pi = 471.23mm$

Design shear strength of weld = $\dfrac{\frac{f_y}{\sqrt{3}} \times (l_w \times t_e)}{\gamma_{mw}}$

$$= \frac{410 \times 471.23 \times 4.2}{\sqrt{3} \times 1.25}$$

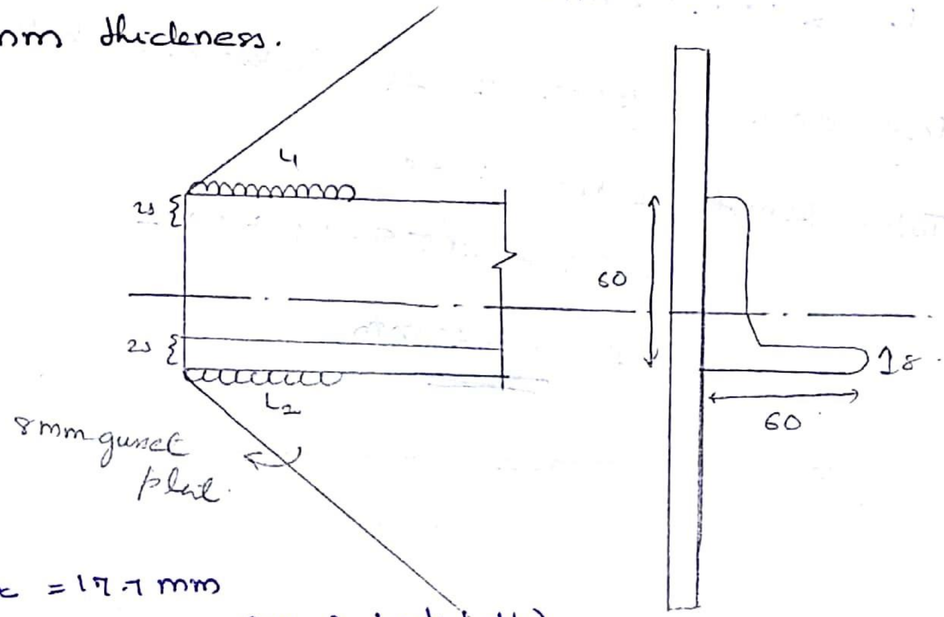
$$= 374.79 \text{ kN}$$

force x distance

Ultimate twisting moment $= 374.79 \times \left(\frac{150}{2}\right)$

$= 28.11 \text{ kNm}$

Q A tension member consist of a single angle ISA 60X60X8 mm has to carry a factored load of 150kN. Design a moment resistant welded connection with a gusset plate of 8mm thickness.



$C_{xx} = 17.7$ mm

$C_{yy} = 17.7$mm (steel book table).

$\Sigma M = 0$

Size of weld (max) $= t - 1.5$

$= 8 - 1.5 = 6.5 \approx 6$mm

Throat thickness, $t = 0.75 = 4.2$mm.

Strength of 1mm weld $= \dfrac{\frac{f_u}{\sqrt{3}} \times (t \times 1)}{\gamma_{mw}}$

$= \dfrac{410\,(4.2 \times 1)}{\sqrt{3} \times 1.5}$ Sight weld = 1.5

$= 0.662$ kN

Eff. length of weld $= l_w = \frac{150}{0.662} = 226.58$ mm

$L_1 + L_2 = 226.58$ —— (1)

$\Sigma M = 0$

$(0.662\ l_1)(60 - 17.7) = (0.662 \times l_2) \times 17.7$

$60 - 17.7 l_1 = 17.7 l_2$ —— (2)

$42.3 l_1 - 17.7 l_2 = 0$.

$l_1 = 66.84$ mm

$l_2 = 159.74$ mm

Provided end returns of 2s distance on both sides.

Total length $= l_1 + l_2 + 2s$

$= 66.84 + 159.74 + 2 \times 12$

$= 250.3467$ mm

Q/ A column 4m long has to support a factored load of 6000kN. The column is effectively held at both ends & restrained in direction at one of the end. Design Column using beam section & plate.

Ans) Given,

Factored load, $P = 6000$ kN

$L = 4$ m

The column is held @ both ends & restrained in dir @ 1 end

$\therefore KL = 0.8L$

(from IS 800, pg 45, Table 11)

$\therefore K = 0.8$

~~we have to design~~

To design beam section & plate

(∴ Built up section)

Step 1: Assume f_{cd} value

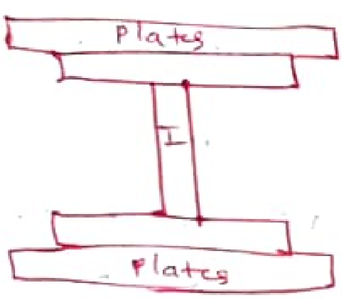
$$f_{cd} = 200\ N/mm^2 \quad (\text{heavy loads})$$

Step 2: Effective Sectional area (A_e)

$$A_e = \frac{P}{f_{cd}} = \frac{6000 \times 10^3}{200}$$

$$= 30 \times 10^3\ mm^2$$

Step 3: built-up section



Built up section

I section — 2 plates

So I section edukumbol thanirikuna area ns $\frac{1}{3}^{rd}$ ~~to~~ $\frac{1}{2}$ portion edukuka. [Here 10,000 to 15,000]

balance given to plate.

~~Area of I section~~

Choose ISHB 450 @ 92.5 kg/m (from SP6)

$A = 117.89\ cm = 11789\ mm^2$

$b_f = 250\ mm$ — from SP6.

$t_f =$

Area of 2 plates $= 30\times10^3$ − area of I section

$= 30\times10^3 - 11789$

$= 18211\ mm^2$ (for 2 plates)

Area of 1 plate $= \frac{18211}{2} = 9105.5\ mm^2$

(width & t of plate namuk venam)

Assume 20mm thick plate,

∴ width of plate $= \frac{\text{Area of 1 plate}}{\text{thickness}}$

$= \frac{9105.5}{20} = 455.275\ mm$

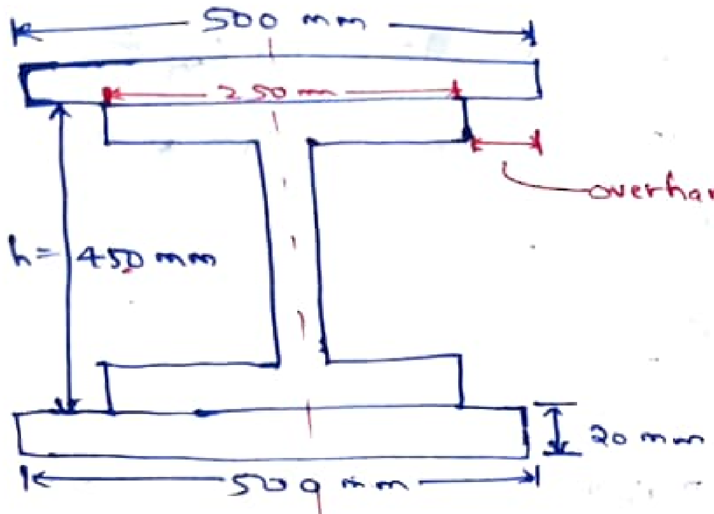
$\approx 500\ mm$

∴ Provide $\overset{b}{500}\times\overset{t}{20}$ mm plate on either side of I section.

∴ Total area of section = Area of I section + 2 (area of 1 plate)

$= 11789 + 2\times(\quad) = 31789\ mm^2$ ($A_{g\,prov}$)

∴ $A_{pro} > A_{req}$ (Hence ok)



overhang $= \frac{500}{2} - \frac{250}{2}$

$= 125$ mm

As per Cl 10.2.3.2, Pg 74, IS 800;

Overhang ≯ { 12 times thickness of plate $= 12 \times 20 = 240$ mm (there in this qu) ; 200 mm }

Overhang $= 125$ mm { < 240 mm ; < 200 mm }

So built-up section case and overhang kodi check Cheyuka....

built-up ah anu overhang varuka

Hence safe.

Step 4: Eff. length

$KL = 0.8L$

$\therefore \underline{\underline{k = 0.8}}$

Step 5: Slenderness ratio

Built up section $\Rightarrow$ Buckling class C (always)

IS 800 Pg. 44

Assume

$f_y = 250\ N/mm^2$. (Fe410 plate)

$$\lambda = \frac{KL}{r_{min}}$$

Builtup section of r_{min} Calculate

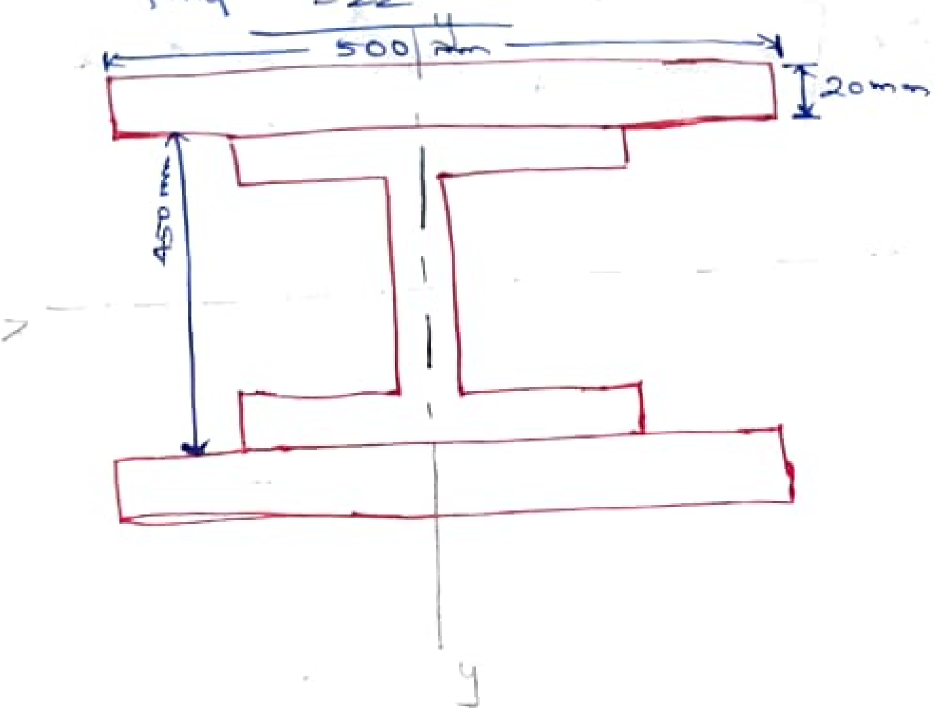
Channel I_{zz} & I_{yy}

Calculate

ie built up section

I_{zz} & I_{yy}

find I_{zz} & I_{yy}

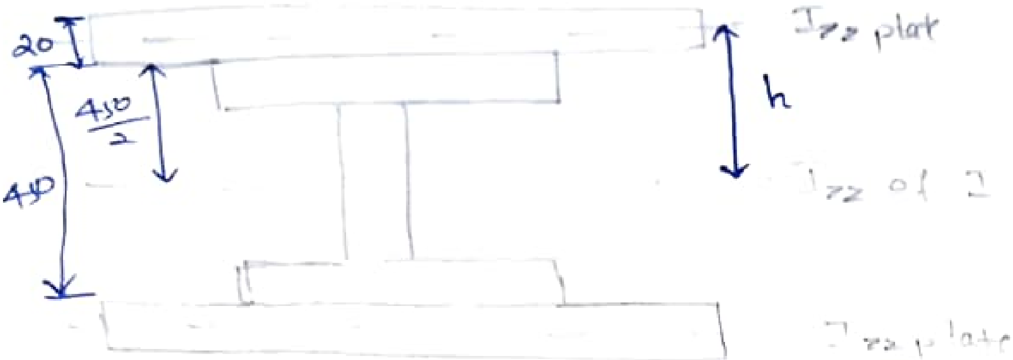


zz axis is same as xx axis.

I section ... buckling axis is ...

I_{zz} of built up Section = I_{zz} of I section +

$2 \times [I_{zz}$ of 1 plate$]$



So I_{zz} I section will get from SP 6.

$\parallel^{l}$ axis theorem vechi tan I_{zz} plate calculate
Cheyuka . .

$$I_{zz} + Ah^2 .$$

I_{zz} built up section = 40349.9×10^4 +

$$2 \times \left[\overset{\frac{bd^3}{12}}{\frac{500 \times 20^3}{12}} + \overset{b \times d}{500 \times 20} \times \left(\frac{450}{2} + \frac{20}{2} \right)^2 \right]$$

$$= 1508.6 \times 10^4 \text{ mm}^4$$

I_{yy} of builtup Section = I_{yy} of I Section +

$2 \times [I_{yy}$ of 1 plate$]$

$$= 3045 \times 10^4 + \overset{db^3/12}{\frac{2 \times 20 \times 500^3}{12}}$$

$$= 447.11 \times 10^6 \text{ mm}^4$$

$$r_{min} = \sqrt{\frac{I_{yy}}{A}} = \sqrt{\frac{447.11 \times 10^6}{31789}}$$

min MI

veram edukan..

Here min is I_{yy}.

Total area of builtup section

$$= 118.59 \text{ mm}$$

$$\therefore \lambda = \frac{KL}{r_{min}} = \frac{0.8 \times 4000}{118.59}$$

$$= 26.89$$

IS 800, pg 42, Table 9(c).

$20 \longrightarrow 224$

$26.98 \longrightarrow ?$

$30 \longrightarrow 211$

$$f_{cd} = 224 + \left(\frac{211 - 224}{30 - 20}\right)(26.98 - 20) = 214.92 \text{ N/mm}^2$$

Step 6 :- Actual load

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

$$= 31789 \times 214.92$$

$$= 6832.28 \text{ kN} > 6000 \text{ kN}$$

Factored load

P

Hence Safe

MOD 5

Purlins

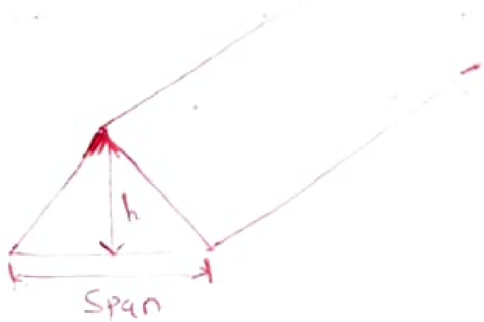
Long horizontal structural member in a roof

Design of purlin (I Section & Channel section)

(Cl. 8.2.1 & 8.2.2, Cl 5.6.1, Section 9)

Step 1:- Determination of slope (of roof truss)

$$\theta = \tan^{-1}\left(\frac{h}{\left(\frac{Span}{2}\right)}\right)$$



h = ht of truss

Span = Span of truss

Step 2:- Determination of Loads

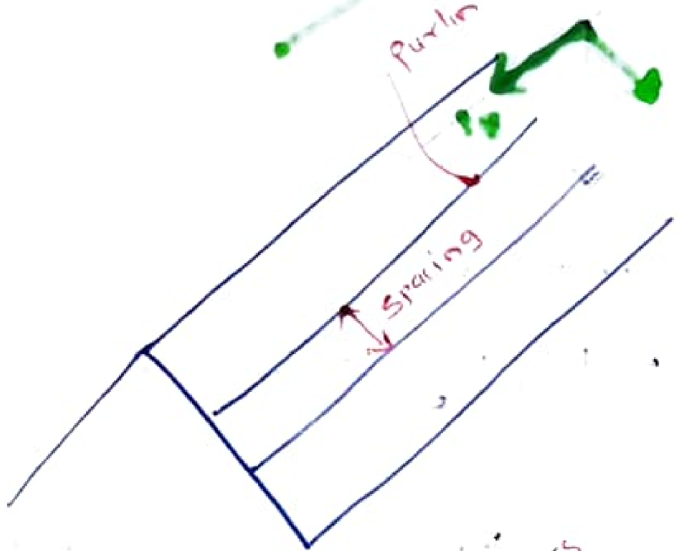
factored load (kN/m²) $= 1.5\left[\begin{array}{c}\text{DL from sheet} + \\ \text{SW of purlin} + \\ \text{LL}\end{array}\right]$

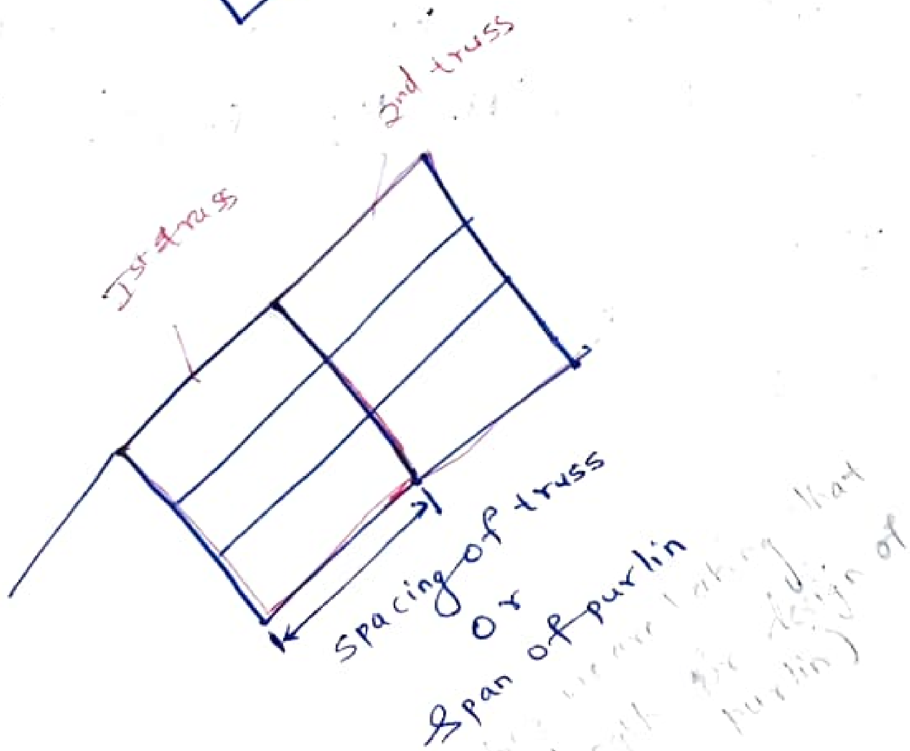
Loads
Sheet load
Purlin load
LL

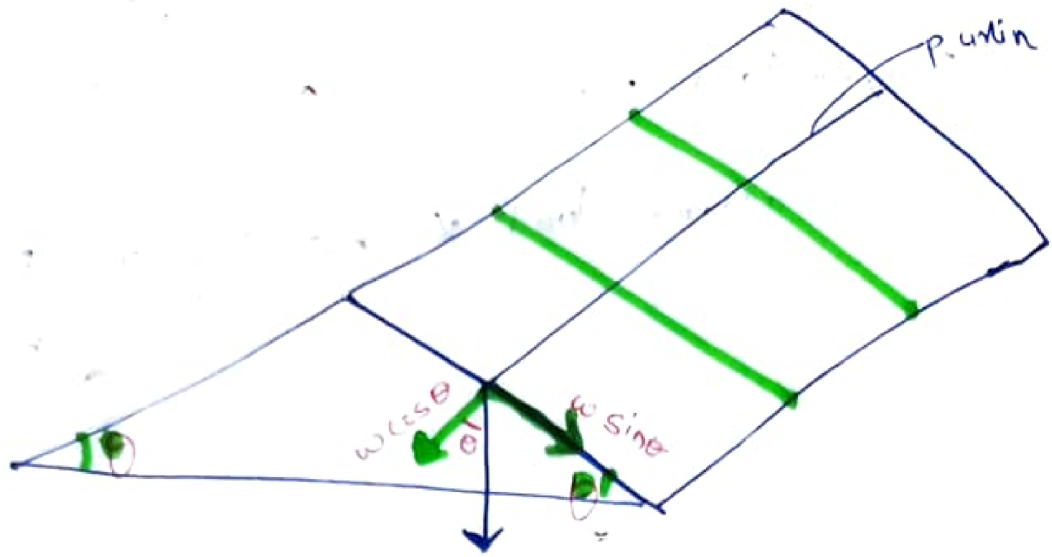
if LL is given in que only, you have to take ..

Total factored load per on run (w)

= factored load x Spacing of purlin (kN/m)







Load normal to sheet, $w_z = w \cos\theta$

Load ‖ʟ to sheet, $w_y = w \sin\theta$

Step 3 :- Determination of BM & SF abt Major & Minor axis (M_z, M_y, F_z, F_y) :-

$$M_z = \frac{w_z l^2}{8}$$

$$M_y = \frac{w_y l^2}{8}$$

for Simply Supported

$$M_z = \frac{w_z l^2}{10}$$

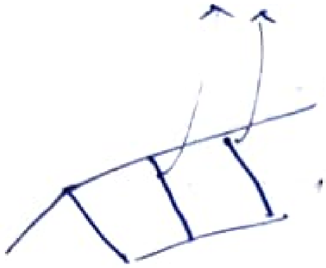
$$M_y = \frac{w_y l^2}{10}$$

for Continuous beam

$$F_z = \frac{w_z l}{2}$$

$$F_y = \frac{w_y l}{2}$$

we know roof inclined ayitula member any



so ithil act chyuna load will be downward.
ithil namal roofin/
Sheetin ∥l & ⊥r ayit
force consider cheyum
load an split to 2 components

w cos θ &
w sin θ

3rd stepil ⇒ for findg BM & SF.

Step 4 nd ⇒ ≥ ISMC 125 means min 125mm
depth ulla ~~ISM~~ chanel secⁿ. veno

& for I section, ≥ ISMB 150 means min
150 mm depth ulla section venam.

V_{dz} = Major axis bending

V_{dy} = Mino axis bending .. ithil flange ant
area anu edukendath

So .. I means 2 flange & 1 web ..
So V_{dy} nd $2 \times (\quad)$ ena equ varum

Step 4 :- Selection of Section

- for channel section, use section greater than or equal to ISMC 125 ..
- for I section, use section greater than or equal to ISMB 150

Step 5 :- Check for Shear

(Cl 8.4, pg 59)

(Major axis) $V_{dz} = \dfrac{h t_w f_y}{\sqrt{3}\, \gamma_{mo}} > F_x$.   (Shear area $= h t_w$)

(Minor axis) $V_{dy} = \dfrac{2 \times b \times t_f\, f_y}{\sqrt{3}\, \gamma_{mo}} > F_y$.

(Shear area $= 2 b t_f$)

$\therefore$ Section will be safe.

Step 6 :- Section Classification

(Pg 18, IS 800)
Table 2

for flange & for web section's.

d/t_w & b/t_f

Choose section

Step 7 :- Check for moment

Z_{pz} = (from IS800, Annex, pg 138 of section chosen)

$$Z_{py} = (b_f t_f) \times \frac{b_f}{2} = \frac{(b_f)^2 t_f}{2}$$

$$M_{dz} = \frac{\beta_b Z_{pz} f_y}{\gamma_{m0}}$$

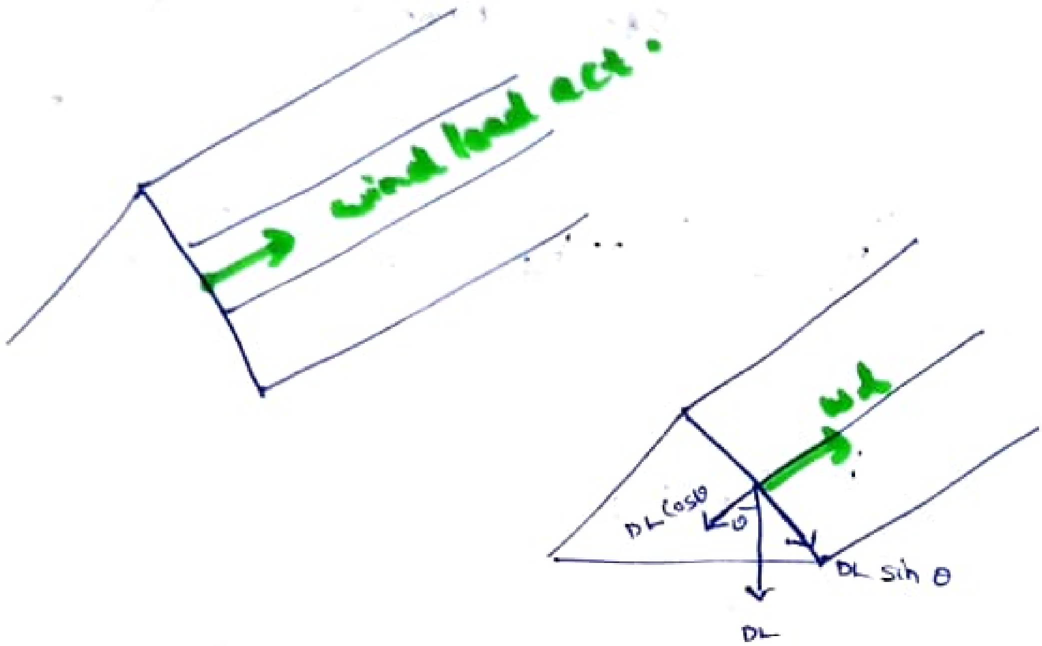
$$M_{dy} = \frac{\beta_b Z_{py} f_y}{\gamma_{m0}}$$

$\left[\begin{array}{l} \beta_b = 1 \text{ (plastic \& compact)} \\ \beta_b = \frac{Z_e}{Z_p} \text{ (semicompact)} \end{array}\right]$

from section 9,

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

Step 8 :- **Check for wind Section**



oppo to DL ⟹ wind load will act (-ve)

Load normal to Sheeting

$$= -WL + DL\cos\theta \quad (w_z)$$

Load $\parallel^{l}$ to Sheeting

$$= DL\sin\theta \quad (w_y)$$

$$M_z = \frac{w_z l^2}{8}$$

$$M_y = \frac{w_y l^2}{8}$$

$$M_{dz} = \beta_b Z_{pz} f_{bd}$$

(pg 54, Cl. 8.2.2 IS 800)

f_{bd} ⇒ laterally unsupported beam (4 Steps)

$\parallel^{ly}$, for $M_{dy} = \beta_b Z_{py} f_{bd}$

Prove :-

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

Step 9 :- <u>Check for</u> deflection

$$\text{Max deflection} = \frac{5}{384} \frac{wl^4}{EI}$$

$w = w_z$ (2^{nd} step)

$I = I_{zz}$ (Chosen Section)

$$\text{Permissible deflection} = \frac{L}{150}$$

Permissible deflection > Max deflection

(Hence safe)

((l = span of purlin or spacing of truss)) specialist literature, or conservatively taken as the value satisfying the limit given above.

8.2.2 *Laterally Unsupported Beams*

Resistance to lateral torsional buckling need not be checked separately (member may be treated as laterally supported, *see* 8.2.1) in the following cases:

a) Bending is about the minor axis of the section,
b) Section is hollow (rectangular/ tubular) or solid bars, and
c) In case of major axis bending, λ_{LT} (as defined herein) is less than 0.4.

The design bending strength of laterally unsupported beam as governed by lateral torsional buckling is given by:

$$M_d = \beta_b Z_p f_{bd}$$

where

β_b = 1.0 for plastic and compact sections.
= Z_e/Z_p for semi-compact sections.

Z_p, Z_e = plastic section modulus and elastic section modulus with respect to extreme compression fibre.

f_{bd} = design bending compressive stress, obtained as given below [*see* Tables 13(a) and 13(b)]

$f_{bd} = \chi_{LT} f_y / \gamma_{m0}$

χ_{LT} = bending stress reduction factor to account for lateral torisonal buckling, given by:

$$\chi_{LT} = \frac{1}{\left\{\phi_{LT} + \left[\phi_{LT}^2 - \lambda_{LT}^2\right]^{0.5}\right\}} \le 1.0$$

$$\phi_{LT} = 0.5\left[1 + \alpha_{LT}(\lambda_{LT} - 0.2) + \lambda_{LT}^2\right]$$

α_{LT}, the imperfection parameter is given by:

α_{LT} = 0.21 for rolled steel section

α_{LT} = 0.49 for welded steel section

The non-dimensional slenderness ratio, λ_{LT}, is given by

$$\lambda_{LT} = \sqrt{\beta_b Z_p f_y / M_{cr}} \le \sqrt{1.2\ Z_e f_y / M_{cr}}$$

$$= \sqrt{\frac{f_y}{f_{cr,b}}}$$

where

M_{cr} = elastic critical moment calculated in accordance with 8.2.2.1, and

$f_{cr,b}$ = extreme fibre bending compressive stress corresponding to elastic lateral buckling moment (*see* 8.2.2.1 and Table 14).

8.2.2.1 *Elastic lateral torsional buckling moment*

In case of simply supported, prismatic members with symmetric cross-section, the elastic lateral buckling moment, M_{cr}, can be determined from:

$$M_{cr} = \sqrt{\left\{\left(\frac{\pi^2 EI_y}{(L_{LT})^2}\right)\left[GI_t + \frac{\pi^2 EI_w}{(L_{LT})^2}\right]\right\}} = \beta_b Z_p f_{cr,b}$$

$f_{cr,b}$ of non-slender rolled steel sections in the above equation may be approximately calculated from the values given in Table 14, which has been prepared using the following equation:

$$f_{cr,b} = \frac{1.1\ \pi^2 E}{(L_{LT}/r_y)^2}\left[1 + \frac{1}{20}\left(\frac{L_{LT}/r_y}{h_f/t_f}\right)^2\right]^{0.5}$$

The following simplified equation may be used in the case of prismatic members made of standard rolled I-sections and welded doubly symmetric I-sections, for calculating the elastic lateral buckling moment, M_{cr} (*see* Table 14):

$$M_{cr} = \frac{\pi^2 EI_y h_f}{2L_{LT}^2}\left[1 + \frac{1}{20}\left(\frac{L_{LT}/r_y}{h_f/t_f}\right)^2\right]^{0.5}$$

where

I_t = torsional constant $= \sum b_i t_i^3/3$ for open section;

I_w = warping constant;

I_y, r_y = moment of inertia and radius of gyration, respectively about the weaker axis;

L_{LT} = effective length for lateral torsional buckling (*see* 8.3);

h_f = centre-to-centre distance between flanges; and

t_f = thickness of the flange.

M_{cr} for different beam sections, considering loading, support condition, and non-symmetric section, shall be more accurately calculated using the method given in Annex E.

8.3 Effective Length for Lateral Torsional Buckling

8.3.1 For simply supported beams and girders of span length, L, where no lateral restraint to the compression flanges is provided, but where each end of the beam is restrained against torsion, the effective length L_{LT} of the lateral buckling to be used in 8.2.2.1 shall be taken as in Table 15.

assumed to exist if the frictional or other positive restraint of a floor connection to the compression flange of the member is capable of resisting a lateral force not less than 2.5 percent of the maximum force in the compression flange of the member. This may be considered to be uniformly distributed along the flange, provided gravity loads constitute the dominant loading on the member and the floor construction is capable of resisting this lateral force.

The design bending strength of a section which is not susceptible to web buckling under shear before yielding (where $d/t_w \leq 67\varepsilon$) shall be determined according to 8.2.1.2.

8.2.1.1 *Section with webs susceptible to shear buckling before yielding*

When the flanges are plastic, compact or semi-compact but the web is susceptible to shear buckling before yielding ($d/t_w \leq 67\varepsilon$), the design bending strength shall be calculated using one of the following methods:

a) The bending moment and axial force acting on the section may be assumed to be resisted by flanges only and the web is designed only to resist shear (*see* 8.4).

b) The bending moment and axial force acting on the section may be assumed to be resisted by the whole section. In such a case, the web shall be designed for combined shear and normal stresses using simple elastic theory in case of semi-compact webs and simple plastic theory in the case of compact and plastic webs.

$V < 0.6\,V_d$

8.2.1.2 When the factored design shear force does not exceed $0.6\,V_d$, where V_d is the design shear strength of the cross-section (*see* 8.4), the design bending strength, M_d shall be taken as:

$$M_d = \beta_b Z_p f_y / \gamma_{m0}$$

To avoid irreversible deformation under serviceability loads, M_d shall be less than $1.2\, Z_e f_y / \gamma_{m0}$ incase of simply supported and $1.5\, Z_e f_y / \gamma_{m0}$ in cantilever beams;

where

β_b = 1.0 for plastic and compact sections;

β_b = Z_e / Z_p for semi-compact sections;

Z_p, Z_e = plastic and elastic section modulii of the cross-section, respectively;

f_y = yield stress of the material; and

γ_{m0} = partial safety factor (*see* 5.4.1).

$V > 0.6\,V_d$

8.2.1.3 When the design shear force (factored), V exceeds $0.6 V_d$, where V_d is the design shear strength of the cross-section (*see* 8.4) the design bending strength, M_d shall be taken

$$M_d = M_{dv}$$

where

M_{dv} = design bending strength under high shear as defined in 9.2.

8.2.1.4 *Holes in the tension zone*

a) The effect of holes in the tension flange, on the design bending strength need not be considered if

$$(A_{nf} / A_{gf}) \geq (f_y/f_u)\,(\gamma_{m1}/\gamma_{m0}) / 0.9$$

where

A_{nf} / A_{gf} = ratio of net to gross area of the flange in tension,

f_y/f_u = ratio of yield and ultimate stress of the material, and

γ_{m1}/γ_{m0} = ratio of partial safety factors against ultimate to yield stress (*see* 5.4.1).

When the A_{nf}/A_{gf} does not satisfy the above requirement, the reduced effective flange area, A_{nf} satisfying the above equation may be taken as the effective flange area in tension, instead of A_{gf}.

b) The effect of holes in the tension region of the web on the design flexural strength need not be considered, if the limit given in (a) above is satisfied for the complete tension zone of the cross-section, comprising the tension flange and tension region of the web.

c) Fastener holes in the compression zone of the cross-section need not be considered in design bending strength calculation, except for oversize and slotted holes or holes without any fastener.

8.2.1.5 *Shear lag effects*

The shear lag effects in flanges may be disregarded provided:

a) For outstand elements (supported along one edge), $b_o \leq L_o / 20$; and

b) For internal elements (supported along two edges), $b_i \leq L_o / 10$.

where

L_o = length between points of zero moment (inflection) in the span,

b_o = width of the flange with outstand, and

b_i = width of the flange as an internal element.

Where these limits are exceeded, the effective width of flange for design strength may be calculated using

Table 10 Buckling Class of Cross-Sections

(*Clause* 7.1.2.2)

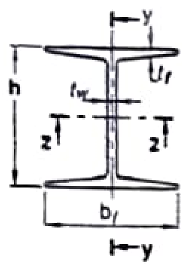
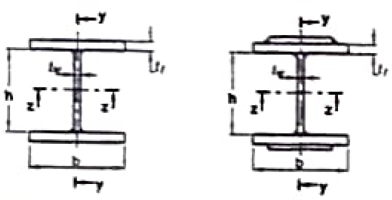
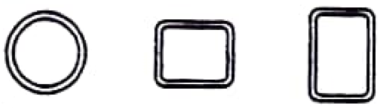
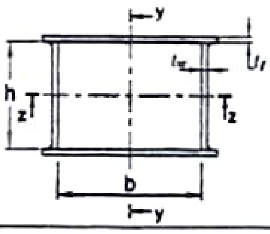
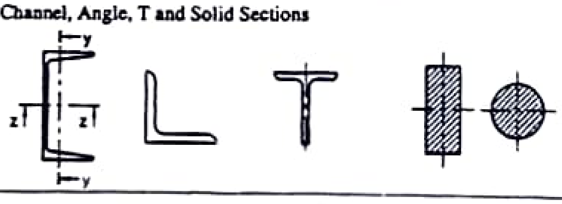
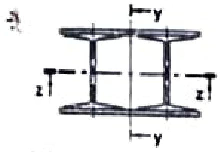
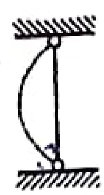
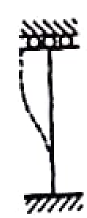
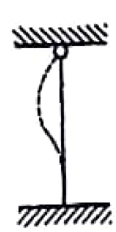
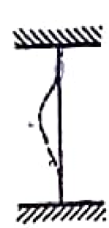
Cross-Section (1)	Limits (2)	Buckling About Axis (3)	Buckling Class (4)
Rolled I-Sections 	$h/b_f > 1.2$: $t_f \le 40$ mm	z-z y-y	a b
	$40 \le$ mm $< t_f \le 100$ mm	z-z y-y	b c
	$h/b_f \le 1.2$: $t_f \le 100$ mm	z-z y-y	b c
	$t_f > 100$ mm	z-z y-y	d d
Welded I-Section 	$t_f \le 40$ mm	z-z y-y	b c
	$t_f > 40$ mm	z-z y-y	c d
Hollow Section 	Hot rolled	Any	a
	Cold formed	Any	b
Welded Box Section 	Generally (except as below)	Any	b
	Thick welds and $b/t_f < 30$ $h/t_w < 30$	z-z y-y	c c
Channel, Angle, T and Solid Sections 		Any	c
Built-up Member 		Any	c

Table 11 Effective Length of Prismatic Compression Members

(*Clause* 7.2.2)

Boundary Conditions				Schematic Representation	Effective Length
At One End		At the Other End			
Translation	Rotation	Translation	Rotation		
(1)	(2)	(3)	(4)	(5)	(6)
Restrained	Restrained	Free	Free		2.0*L*
Free	Restrained	Free	Restrained		
Restrained	Free	Restrained	Free		1.0*L*
Restrained	Restrained	Free	Restrained		1.2*L*
Restrained	Restrained	Restrained	Free		0.8*L*
Restrained	Restrained	Restrained	Restrained		0.65*L*

NOTE — *L* is the unsupported length of the compression member (*see* 7.2.1).

Table 9(c) Design Compressive Stress, f_{cd} (MPa) for Column Buckling Class c
(*Clause* 7.1.2.1)

KL/r ↓	Yield Stress, f_y (MPa)																		
	200	210	220	230	240	250	260	280	300	320	340	360	380	400	420	450	480	510	540
10	182	191	200	209	218	227	236	255	273	291	309	327	345	364	382	409	436	464	491
20	182	190	199	207	216	224	233	250	266	283	299	316	332	348	364	388	412	435	458
30	172	180	188	196	204	211	219	234	249	264	278	293	307	321	335	355	376	395	415
40	163	170	177	184	191	198	205	218	231	244	256	268	280	292	304	320	337	352	367
50	153	159	165	172	178	183	189	201	212	222	232	242	252	261	270	282	295	306	317
60	142	148	153	158	163	168	173	182	191	199	207	215	222	228	235	244	252	260	267
70	131	136	140	144	148	152	156	163	170	176	182	187	192	197	202	208	213	218	223
80	120	123	127	130	133	136	139	145	149	154	158	162	165	169	172	176	180	183	186
90	108	111	114	116	119	121	123	127	131	134	137	140	142	144	146	149	152	154	156
100	97.5	100	102	104	105	107	109	112	114	116	119	120	122	124	125	127	129	131	132
110	87.3	89.0	90.5	92.0	93.3	94.6	95.7	97.9	100	102	103	104	106	107	108	110	111	112	113
120	78.2	79.4	80.6	81.7	82.7	83.7	84.6	86.2	87.6	88.9	90.1	91.1	92.1	93.0	93.8	94.9	95.9	96.8	97.6
130	70.0	71.0	71.9	72.8	73.5	74.3	75.0	76.2	77.3	78.3	79.2	80.0	80.7	81.4	82.0	82.9	83.6	84.3	84.9
140	62.9	63.6	64.4	65.0	65.6	66.2	66.7	67.7	68.6	69.3	70.0	70.7	71.2	71.8	72.3	72.9	73.5	74.1	74.6
150	56.6	57.2	57.8	58.3	58.8	59.2	59.7	60.4	61.1	61.7	62.3	62.8	63.3	63.7	64.1	64.6	65.1	65.5	65.9
160	51.1	51.6	52.1	52.5	52.9	53.3	53.6	54.2	54.8	55.3	55.7	56.1	56.5	56.9	57.2	57.6	58.0	58.4	58.7
170	46.4	46.8	47.1	47.5	47.8	48.1	48.4	48.9	49.3	49.8	50.1	50.5	50.8	51.1	51.3	51.7	52.0	52.3	52.6
180	42.2	42.5	42.8	43.1	43.4	43.6	43.9	44.3	44.7	45.0	45.3	45.6	45.8	46.1	46.3	46.6	46.9	47.1	47.3
190	38.5	38.8	39.0	39.3	39.5	39.7	39.9	40.3	40.6	40.9	41.1	41.4	41.6	41.8	42.0	42.2	42.5	42.7	42.9
200	35.3	35.5	35.7	35.9	36.1	36.3	36.5	36.8	37.0	37.3	37.5	37.7	37.9	38.1	38.2	38.4	38.6	38.8	39.0
210	32.4	32.6	32.8	33.0	33.1	33.3	33.4	33.7	33.9	34.1	34.3	34.5	34.7	34.8	34.9	35.1	35.3	35.4	35.6
220	29.9	30.1	30.2	30.4	30.5	30.6	30.8	31.0	31.2	31.4	31.5	31.7	31.8	31.9	32.1	32.2	32.4	32.5	32.6
230	27.6	27.8	27.9	28.0	28.2	28.3	28.4	28.6	28.8	28.9	29.1	29.2	29.3	29.4	29.5	29.7	29.8	29.9	30.0
240	25.6	25.7	25.9	26.0	26.1	26.2	26.3	26.4	26.6	26.7	26.9	27.0	27.1	27.2	27.3	27.4	27.5	27.6	27.7
250	23.8	23.9	24.0	24.1	24.2	24.3	24.4	24.5	24.7	24.8	24.9	25.0	25.1	25.2	25.3	25.4	25.5	25.6	25.7

Table 2 Limiting Width to Thickness Ratio

(Clauses 3.7.2 and 3.7.4)

Compression Element			Ratio	Class of Section		
				Class 1 Plastic	Class 2 Compact	Class 3 Semi-compact
(1)			(2)	(3)	(4)	(5)
Outstanding element of compression flange	Rolled section		b/t_f	9.4ε	10.5ε	15.7ε
	Welded section		b/t_f	8.4ε	9.4ε	13.6ε
Internal element of compression flange	Compression due to bending		b/t_f	29.3ε	$33.5\,\varepsilon$	42ε
	Axial compression		b/t_f	Not applicable		42ε
Web of an I, H or box section	Neutral axis at mid-depth		d/t_w	84ε	105ε	126ε
	Generally	If r_1 is negative:	d/t_w	$\dfrac{84\varepsilon}{1+r_1}$ but $\le 42\varepsilon$	$\dfrac{105.0\,\varepsilon}{1+r_1}$	$\dfrac{126.0\,\varepsilon}{1+2r_2}$ but $\le 42\varepsilon$
		If r_1 is positive :	d/t_w		$\dfrac{105.0\varepsilon}{1+1.5r_1}$ but $\le 42\varepsilon$	
	Axial compression		d/t_w	Not applicable		42ε
Web of a channel			d/t_w	42ε	42ε	42ε
Angle, compression due to bending (Both criteria should be satisfied)			b/t d/t	9.4ε $9.4\,\varepsilon$	10.5ε 10.5ε	15.7ε 15.7ε
Single angle, or double angles with the components separated, axial compression (All three criteria should be satisfied)			b/t d/t $(b+d)/t$	Not applicable		15.7ε 15.7ε 25ε
Outstanding leg of an angle in contact back-to-back in a double angle member			d/t	9.4ε	10.5ε	15.7ε
Outstanding leg of an angle with its back in continuous contact with another component			d/t	9.4ε	10.5ε	15.7ε
Stem of a T-section, rolled or cut from a rolled I-or H-section			D/t_f	8.4ε	9.4ε	18.9ε
Circular hollow tube, including welded tube subjected to: a) moment			D/t	$42\varepsilon^2$	$52\varepsilon^2$	$146\varepsilon^2$
b) axial compression			D/t	Not applicable		$88\varepsilon^2$

NOTES

1 Elements which exceed semi-compact limits are to be taken as of slender cross-section.

2 $\varepsilon = (250/f_y)^{1/2}$.

3 Webs shall be checked for shear buckling in accordance with 8.4.2 when $d/t > 67\varepsilon$, where, b is the width of the element (may be taken as clear distance between lateral supports or between lateral support and free edge, as appropriate), t is the thickness of element, d is the depth of the web, D is the outer diameter of the element (*see* Fig. 2, 3.7.3 and 3.7.4).

4 Different elements of a cross-section can be in different classes. In such cases the section is classified based on the least favourable classification.

5 The stress ratio r_1 and r_2 are defined as:

$$r_1 = \frac{\text{Actual average axial stress (negative if tensile)}}{\text{Design compressive stress of web alone}}$$

$$r_2 = \frac{\text{Actual average axial stress (negative if tensile)}}{\text{Design compressive stress of overll section}}$$

9.2.2 When the factored value of the applied shear force is high (exceeds the limit specified in **9.2.1**), the factored moment of the section should be less than the moment capacity of the section under higher shear force, M_{dv}, calculated as given below:

a) *Plastic or Compact Section*

$$M_{dv} = M_d - \beta(M_d - M_{fd}) \leq 1.2 Z_e f_y / \gamma_{m0}$$

where

β = $(2V/V_d - 1)^2$

M_d = plastic design moment of the whole section disregarding high shear force effect (*see* **8.2.1.2**) considering web buckling effects (*see* **8.2.1.1**),

V = factored applied shear force as governed by web yielding or web buckling,

V_d = design shear strength as governed by web yielding or web buckling (*see* **8.4.1** or **8.4.2**),

M_{fd} = plastic design strength of the area of the cross-section excluding the shear area, considering partial safety factor γ_{m0}, and

Z_e = elastic section modulus of the whole section.

b) *Semi-compact Section*

$$M_{dv} = Z_e f_y / \gamma_{m0}$$

9.3 Combined Axial Force and Bending Moment

Under combined axial force and bending moment, section strength as governed by material failure and member strength as governed by buckling failure shall be checked in accordance with **9.3.1** and **9.3.2** respectively.

9.3.1 *Section Strength*

9.3.1.1 *Plastic and compact sections*

In the design of members subjected to combined axial force (tension or compression) and bending moment, the following should be satisfied:

$$\left(\frac{M_y}{M_{ndy}}\right)^{\alpha_1} + \left(\frac{M_z}{M_{ndz}}\right)^{\alpha_2} \leq 1.0$$

Conservatively, the following equation may also be used under combined axial force and bending moment:

$$\frac{N}{N_d} + \frac{M_y}{M_{dy}} + \frac{M_z}{M_{dz}} \leq 1.0$$

where

M_y, M_z = factored applied moments about the minor and major axis of the cross-section, respectively;

M_{ndy}, M_{ndz} = design reduced flexural strength under combined axial force and the respective uniaxial moment acting alone (*see* **9.3.1.2**);

N = factored applied axial force (Tension, T or Compression, P);

N_d = design strength in tension, T_d as obtained from 6 or in compression due to yielding given by $N_d = A_g f_y / \gamma_{m0}$;

M_{dy}, M_{dz} = design strength under corresponding moment acting alone (*see* **8.2**);

A_g = gross area of the cross-section;

α_1, α_2 = constants as given in Table 17; and

γ_{m0} = partial factor of safety in yielding.

9.3.1.2 For plastic and compact sections without bolts holes, the following approximations may be used for evaluating M_{ndy} and M_{ndz}:

a) *Plates*

$$M_{nd} = M_d(1 - n^2)$$

b) *Welded I or H sections*

$$M_{ndy} = M_{dy}\left[1 - \left(\frac{n-a}{1-a}\right)^2\right] \leq M_{dy} \text{ where } n \geq a$$

$$M_{ndz} = M_{dz}(1-n)/(1-0.5a) \leq M_{dz}$$

where

$n = N/N_d$ and $a = (A - 2bt_f)/A \leq 0.5$

c) *For standard I or H sections*

for $n \leq 0.2$ $M_{ndy} = M_{dy}$

for $n > 0.2$ $M_{ndy} = 1.56\, M_{dy}(1-n)(n+0.6)$

$M_{ndz} = 1.11\, M_{dz}(1-n) \leq M_{dz}$

d) *For rectangular hollow sections and welded box sections*

When the section is symmetric about both axes and without bolt holes

$$M_{ndy} = M_{dy}(1-n)/(1-0.5a_f) \leq M_{dy}$$

$$M_{ndz} = M_{dz}(1-n)/(1-0.5a_w) \leq M_{dz}$$

where

$a_w = (A - 2\,b\,t_f)/A \leq 0.5$

$a_f = (A - 2\,h\,t_w)/A \leq 0.5$

e) *Circular hollow tubes without bolt holes*

$$M_{nd} = 1.04\, M_d(1 - n^{1.7}) \leq M_d$$

MOD 5

Purlins

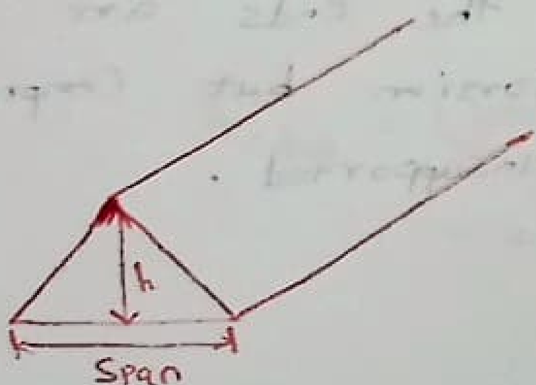
Long horizontal structural member in a roof

Design of purlin (I section & Channel section)

(Cl. 8.2.1 & 8.2.2, Cl 5.6.1, Section 9)

Step 1 :- Determination of slope (of roof truss)

$$\theta = \tan^{-1}\left(\frac{h}{\left(\frac{span}{2}\right)}\right)$$



h = ht of truss

Span = Span of truss

Step 2 :- Determination of Loads

factored load (kN/m²) $= 1.5\left[\begin{array}{c} \text{DL from sheet +} \\ \text{SW of purlin +} \\ \text{LL} \end{array}\right]$

Loads

Sheet load

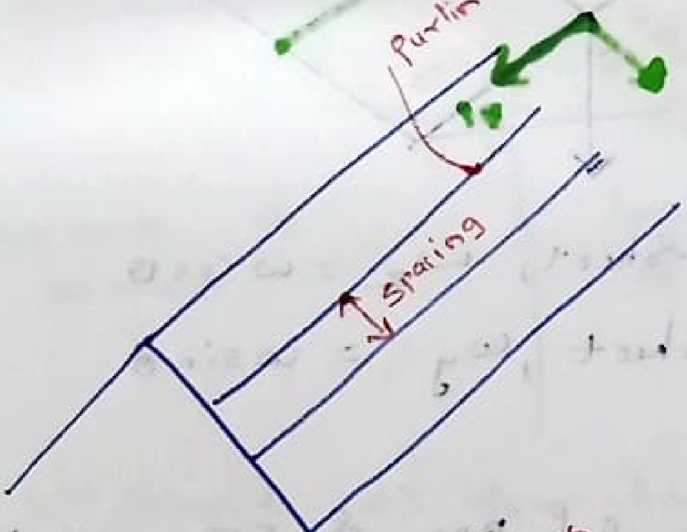
Purlin load

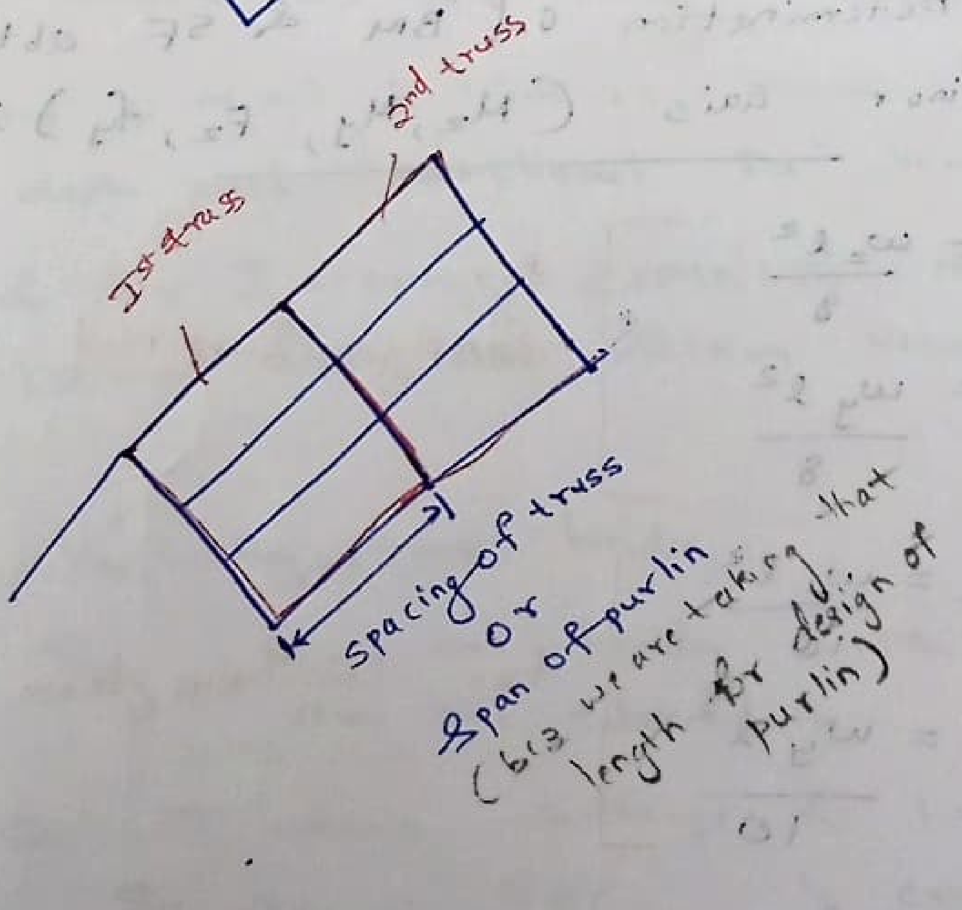
LL

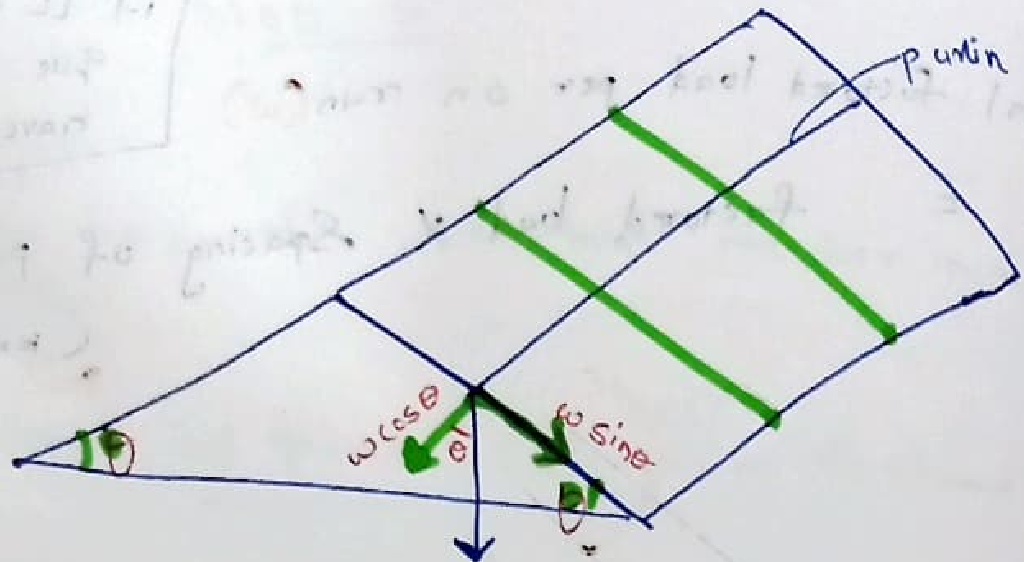
if LL is given in que only, you have to take..

Total factored load per on run (w)

= factored load x Spacing of purlin (kN/m)







Load normal to sheet, $w_z = w \cos\theta$

Load ∥ to sheet, $w_y = w \sin\theta$

Step 3 :- Determination of BM & SF abt Major & Minor axis (M_z, M_y, F_z, F_y) :-

$$M_z = \frac{w_z l^2}{8}$$

$$M_y = \frac{w_y l^2}{8}$$

for simply supported

$$M_z = \frac{w_z l^2}{10}$$

$$M_y = \frac{w_y l^2}{10}$$

for Continuous beam

$$F_z = \frac{w_z l}{2}$$

$$F_y = \frac{w_y l}{2}$$

Step 4 :- Selection of Section

- for channel section, use section greater than or equal to ISMC 125.
- for I section, use section greater than or equal to ISMB 150

Step 5 :- Check for Shear

(Cl. 8.4, pg 59)

(Major axis) $V_{dz} = \frac{h t_w f_y}{\sqrt{3} \gamma_{mo}} > F_x$ (Shear area = $h t_w$)

(Minor axis) $V_{dy} = \frac{2 \times b \times t_f f_y}{\sqrt{3} \gamma_{mo}} > F_y$.

(Shear area = $2bt_f$)

$\therefore$ Section will be safe.

Step 6 :- Section Classification

(Pg 18, IS. 800)

Table 2

for flange & for web section's.

d/t_w & b/t_f

Choose section

Step 7 :- Check for moment

Z_{pz} = (from IS800, Annex, pg 138 of section chosen)

$$Z_{py} = (b_f t_f) \times \frac{b_f}{2} = \frac{(b_f)^2 t_f}{2}$$

$$M_{dz} = \frac{\beta_b Z_{pz} f_y}{\gamma_{m0}}$$

$$M_{dy} = \frac{\beta_b Z_{py} f_y}{\gamma_{m0}}$$

$\beta_b = 1$ (plastic & compact)
$\beta_b = \frac{Z_e}{Z_p}$ (semicompact)

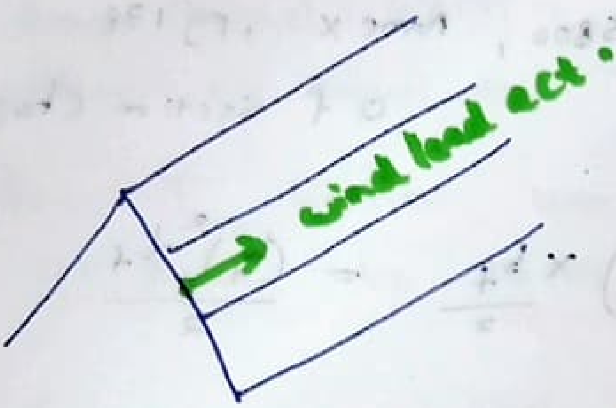
from section 9,

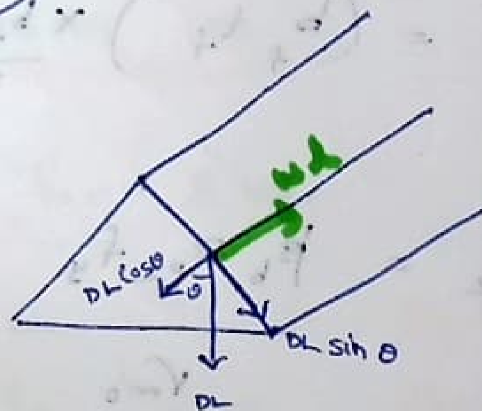
$$\frac{M_z}{M_{dz}} + \frac{M_y}{M_{dy}} \le 1 .$$

3.5.6 The crane ... shall be checked for bumper impact loads also, as

shall be less than that specified under Class 1 (Plastic), in Table 2.

Step 8 :- Check for wind Section





oppo to DL ⟹ wind load will act (-ve)

Load normal to Sheeting

$= -WL + DL \cos\theta$ (w_z)

Load ∥ to Sheeting

$= DL \sin\theta$ (w_y)

$M_z = \dfrac{w_z l^2}{8}$

$M_y = \dfrac{w_y l^2}{8}$

$M_{dz} = \beta_b Z_{pz} f_{bd}$ (pg 54, Cl.8.2.2 IS 800)

DL cos θ are not w cos θ. ... load combination ... DL, LL or DL + WL are ...

f_{bd} ⇒ laterally unsupported beam (4 Steps)

$\parallel ly$, for $M_{dy} = \beta_b Z_{py} f_{bd}$

Prove :-

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

Step 9 :- Check for deflection

$$\text{Max deflection} = \frac{5}{384}\frac{wl^4}{EI}$$

$w = w_z$ (2nd step)

$$\text{permissible deflection} = \frac{L}{150}$$

$I = I_{zz}$ (Chosen Section)

Permissible deflection > Max deflection
(Hence safe)

((l = span of purlin or spacing of truss))