115 UNIT-II : DESIGN OF TENSION MEMBERS.

A Structural member subjected to two pulling forces applied at its end is called tension members.

Types of tension members:

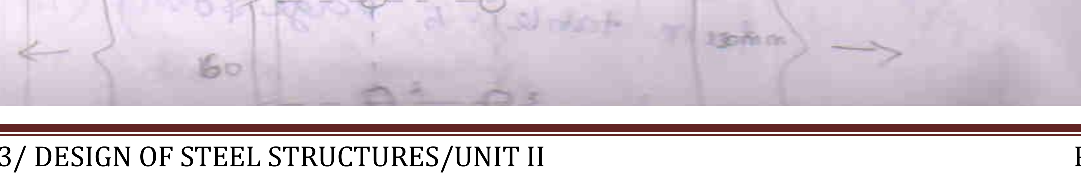
1. wires & cables
2. Rods and bars
3. Single structural shapes & plates.
4. build up member.

Design strength of tension Member:

1. Design strength due to yielding of Gross section (T_{dg})
2. Repture strength of critical section (T_{dn})
3. Design strength due to block shear (T_{db})

(from code pg: 32 Section-6 all classes)

1. Determine the design strength of the plate 130mm x 12mm with the holes for 16mm diameter for bolts as shwn in fig. the steel is Fe410 grade.



13/2/15 Stm:

Step 1: Strength due to yielding:

from code pg: 32 class 6.2

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

$$= \frac{(130 \times 12) \times 250}{1.1}$$

$\therefore T_{dg} = 354.545 \, KN$.

Step 2: Strength due to Rupture.

from code pg: 32 & 33 class 6.3.1

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

$$A_n = \left[b - nd_0 + \frac{\Sigma P_{si}^2}{4g_i}\right] t$$

$$= [130 - 2 \times 18 + 0] \, 12$$

$= 1128 \, mm^2$.

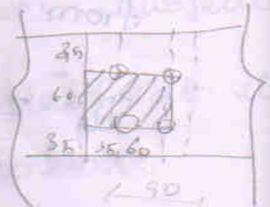
$$T_{dn} = \frac{0.9 \times 1128 \times 410}{1.25}$$

$= 332.985 \, kN$.

Step-3: strength due to block shear.
from code pg: 33 claus 6.4.1.

gross area in shear

$$A_{vg} = [(35+60)\times 12]\times 2$$



$$= 2280\ mm^2$$

critical section gross area

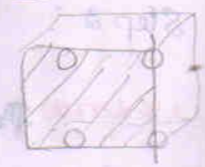
$$A_{tg} = 60\times 12$$

$$= 720\ mm^2$$

net area in shear

$$A_{vn} = 2\,[(35+60) - 1.5\times 18]\times 12$$

$$= 1632\ mm^2$$



critical section net area

$$A_{tn} = (60-18)\times 12$$

$$= 504\ mm^2$$

$$T_{db1} = \left[\frac{A_{vg} f_y}{\sqrt{3}\,\gamma_{m0}} + \frac{0.9 A_{tn} f_u}{\gamma_{m1}}\right]$$

$$T_{db2} = \left[\frac{0.9 A_{vn} f_u}{\sqrt{3}\,\gamma_{m1}} + \frac{A_{tg} f_y}{\gamma_{m0}}\right]$$

$$T_{db1} = \frac{2280\times 250}{\sqrt{3}\times 1.1} + \frac{0.9\times 504\times 410}{1.25}$$

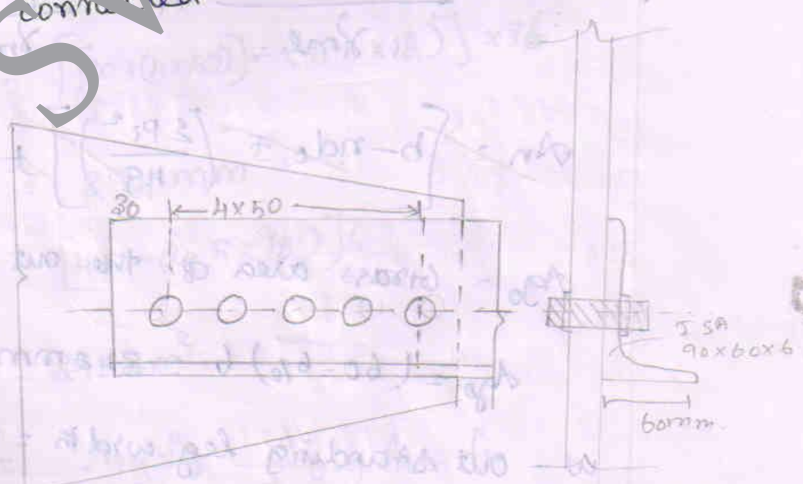
$$= 447.95\ kN$$

$$T_{db_2} = \left(\frac{0.9 \times 1632 \times 410}{\sqrt{3} \times 1.25} + \frac{720 \times 250}{1.1} \right)$$

$= 441.78$ kN

∴ Design tensile strength of the plate $= 332.985$ kN

2. A single unequal angle ISA 90×60×6 mm is connected to a 10mm gusset plate at the ends with 5 no.of 16mm bolt to transfer a tension as shown in fig. Determine the design strength of the angle, a) if the gusset is connected to 90mm leg. b) if the gusset is connected to 60 mm leg.



Soln: If the gusset is connected to 90mm leg.

Step-1 : Strength due to yielding :

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

$$A_g = \left[(90 - 6/2) + (60 - 6/2)\right] \times 6$$

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

$$T_{dg} = \frac{864 \times 250}{1.1}$$

$$= 196.364 \text{ kN}$$

Step 2 : strength due to rupture.
from code IS : 33 class 6.3.3.

$$T_{dn} = \frac{0.9 A_n f_u}{\gamma_{ml}} + B \frac{A_{go} f_y}{\gamma_{mo}}.$$

$$\cancel{A_n = \left[b - nd_0 + \left(\frac{\Sigma P_i^2}{4g}\right)\right] t}, \quad A_n = [(90 - 6/2) - 18] = 414 \text{mm}^2$$

A_{go} - Gross area of the out standing leg.

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

w - Out standing leg width = 60 mm.

$$w_i = \frac{90}{2} = 45 \text{ mm}.$$

$$b_s = w + w_i - t$$

$$= 60 + 45 - 6 = 99 \text{ mm}.$$

$$L_c = 4 \times 50 = 200 \text{ mm}$$

$$\beta = 1.4 - 0.076 (w/t) (f_y/f_u) (b_s/L_c)$$

$$= 1.4 - 0.076 \times \frac{60}{6} \times \frac{250}{410} \times \frac{99}{200}$$

$$= 1.17 \leq \frac{f_u \gamma_{m0}}{\gamma_{m1} f_y} \geq 0.7$$

$$\therefore T_{dn} = \frac{0.9 \times 414 \times 410}{1.25} + \frac{1.17 \times 342 \times 250}{1.1}$$

$$= 213.15 \text{ kN}$$

Step - 3 : Strength due to block shear.

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

$$A_{tg} = 45 \times 6 = 270 \text{ mm}^2$$

$$A_{vn} = [[30 + (4 \times 50)] - (4.5 \times 18)] \times 6$$

$$= 894 \text{ mm}^2$$

$$A_{tn} = [45 - (0.5 \times 18)] 6$$

$$= 216 \text{ mm}^2$$

$$T_{db1} = \frac{A_{vg} f_y}{\sqrt{3} \gamma_{m0}} + \frac{0.9 A_{tn} f_u}{\gamma_{m1}}$$

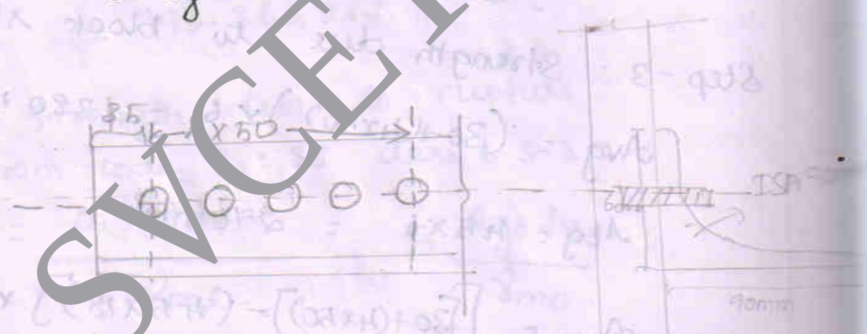
$$= \frac{1380 \times 250}{\sqrt{3} \times 1.10} + \frac{0.9 \times 216 \times 410}{1.25}$$

$$T_{db2} = \frac{0.9 A_{vn} f_u}{\sqrt{3}\,\gamma_{m1}} + \frac{A_{vg} f_y}{\gamma_{m0}}$$

$$= \frac{0.9 \times 894 \times 410}{\sqrt{3} \times 1.25} + \frac{270 \times 250}{1.1}$$

$$= 213.73\ kN$$

∴ Design of the tensile strength of angle = 190.36 kN.



If the gusset is connected to 60mm gusset plate.

Step-1: Strength due to yielding.

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

$$= \frac{864 \times 250}{1.1}$$

$$= 196.364\ kN$$

Step-2: Strength due to rupture

$$T_{dn} = \frac{0.9 A_n f_u}{\gamma_{ml}} + \frac{\beta A_{go} f_y}{\gamma_{mo}}$$

$A_n = [(60 - 6/2) - 18] \times 6 = 234 mm^2$

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

$w = 90 mm$

$w_i = \frac{60}{2} = 30 mm$

$b_s = w + w_i - t = 90 + 30 - 6 = 114 mm$

$L_c = 4 \times 50 = 200 mm$

$\beta = 1.4 - 0.076 (w/t)(f_y/f_u)(b_s/L_c)$

$= 1.4 - 0.076 \times \frac{90}{6} \times \frac{250}{410} \times \frac{114}{200}$

$= 1.00 \leq \frac{f_u \gamma_{mo}}{\gamma_{ml} f_y} \geq 0.7$

$$T_{dn} = \frac{0.9 \times \cancel{414}\,234 \times 410}{1.25} + \frac{1 \times 522 \times 250}{1.1}$$

$= \cancel{240.84 kN}$ $187.713 kN$

Step-3: Strength due to block shear.

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

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

$A_{vn} = [30 + (4 \times 50) - (4.5 \times 18)] \times 6$

$= 894 mm^2$

$A_{tn} = [30 - (0.5 \times 18)] 6$

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

$$= \frac{1380 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 126 \times 410}{1.25}$$

$$= 218.279 \text{ kN}$$

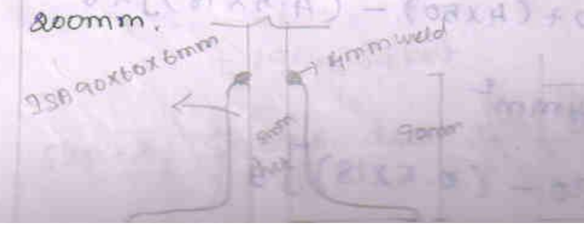
$$T_{db_2} = \frac{0.9 A_{vn} f_u}{\sqrt{3}\, \gamma_{m1}} + \frac{A_{tg} f_y}{\gamma_{m0}}$$

$$= \frac{0.9 \times 894 \times 410}{\sqrt{3} \times 1.25} + \frac{180 \times 250}{1.1}$$

$$= 193.2[illegible] \text{ kN}$$

Design tensile strength of angle = 187.793 kN.

16/2/15 3. Determine the design tensile strength of a roof truss member 2ISA 90x60x6 mm connected to the gusset plate of 8mm thick. The connection of plate and angle by 4mm weld. The effective length of the weld is shown in fig. The effective length of the weld is 200mm.



Soln:

Step 1: Design strength due to yielding

form code pg: 32 clause 6.2

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

from Steel table

$$A_g = 865 \text{ mm}^2.$$

$$= \frac{865 \times 250}{1.1}$$

$$= 196.59 \text{ kN}$$

Yield strength of the member $= 2 \times 196.59 = 393.18$ kN

Step 2: Strength due to rupture.

from code pg: 33 clause 6.3.1

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

$$A_{nc} = \left(\left(90 - 6/2\right) \cancel{\ } \right) 6$$

$$= \cancel{414 \text{ mm}^2} \; 522 \text{ mm}^2$$

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

$w = 60$ mm

$w_1 = \frac{90}{2} = 45$ mm.

$b_s = w$ ~~+w1-t~~ $=$ ~~60+45-6 = 99 mm.~~ $= 60$ mm

$L_c = 200$ mm

$$\beta = 1.4 - 0.076 \left(w/t\right) \left(f_y/f_u\right) \left(\frac{b_s}{L_c}\right)$$

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

$$\beta = 1.26 \le \frac{f_u \gamma_{m0}}{f_y \gamma_{ml}} \ge 0.7.$$

$$T_{dn} = \frac{0.9 \times 522 \times 410}{1.25} + \frac{1.26 \times 342 \times 250}{1.1}$$

$$T_{dn} = 252.84 \text{ kN}.$$

$\cancel{T_{dn}}$

Rupture strength $= 2 \times 252.84 = 504$ kN

Step 3: Design strength of block shear.

$$A_{vg} = A_{vn} = [2 \times 200 \times 6] = 2400 \text{ mm}^2$$

$$A_{tg} = A_{tn} = 90 \times 6 = 540 \text{ mm}^2.$$

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

$$= \frac{2400 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 540 \times 410}{1.25}$$

$$= \cancel{632.43 \text{ kN}} \quad 474.32 \text{ kN}$$

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

$$= \frac{0.9 \times \cancel{24}00 \times 410}{\sqrt{3} \times 1.25} + \frac{\cancel{7}20 \times \cancel{2}50}{1.1}$$

$$= \frac{0.9 \times 2400 \times 410}{\sqrt{3} \times 1.25} + \frac{540 \times 250}{1.1}$$

$= 531.76$ kN.

Yield strength of the member $= 2 \times 531.77$
$= 1063.54$ kN

∴ Design yield strength of the member $= 393.2$ kN//.

Design of Tension Members

Design Procedure :-

Step-1 :- Find the required gross area to carry the factored load considering the strength in yielding.

$$A_g = \frac{T_{dg} \gamma_{m0}}{f_y} \quad \text{(From code Pg-32)}$$

2. Select suitable shape of the section depending upon the type of

more than A_g Calculated.

3. Determine the no. of bolts / length of welding required & arrange.

4. Find the strength considering

i. Strength in yielding of cross area.

ii. Strength in rupture of critical section

iii. Strength in block shear.

5. If the min. edge & pitch distance are maintained.

1. Design a single angle section for a tension member of a roof truss to carry a factored tensile force of 225 kN. The member is subjected to the possible reversle of stress due to the action of wind. The length of the member is 3m. Use 20mm shop bolt of grade 4.6 for the connection

Solution:

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

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

$$\Rightarrow A_g = 990\ mm^2$$

Step 2:- Select suitable shape

Try ISA $100 \times 75 \times 8$ mm

from steel table $A_g = 1336\ mm^2$

Assume thickness of gusset plate = 10mm

Step 3: No. of bolts calculation

$$\text{No. of bolts} = \frac{\text{Design load}}{\text{Strength of bolt}}$$

a. Strength of bolt in shear :

Strength of bolt in shear, $V_{dsb} = \dfrac{V_{nsb}}{\gamma_{mb}}$

$d = 20mm$, $d_o = 22$ mm., $\gamma_{mb} = 1.25$,

$f_{ub} = 400$ MPa, $f_u = 410$ MPa

Nominal Shear stress, $V_{nsb} = \dfrac{f_{ub}}{\sqrt{3}}(n_n A_{nb} + n_s A_{sb})$

$n_n = 1$, $n_s = 0$ ($\because$ single shear)

$V_{nsb} = \dfrac{400}{\ }\left(1 \times 0.78 \times \dfrac{\pi}{4} \times 20^2\right)$

$V_{nsb} = 56.59$ kN

$\Rightarrow V_{dsb} = \frac{56.59 \text{ kN}}{1.25} = 45.27$ kN

b. Strength of bolt in bearing:-

Strength of bolt in bearing,

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

Nominal strength in bearing

$$V_{npb} = 2.5 k_b \, d \, t \, f_u$$

i. Pitch distance:

From code Pg-75, class 10.2.2

$P \not< 2.5 d = 2.5 \times 20 = 50$ mm $\simeq 60$ mm

ii. Edge distance:

From code Pg-74, class 10.2.4.2

$e \not< 1.5 d_o = 1.5 \times 22 = 33$ mm $\simeq 40$ mm

k_b is the least of the following

i. $\frac{e}{3 d_o} = \frac{40}{3 \times 22} = 0.606$

$\frac{60}{} - 0.25 = 0.659$

iii. $\frac{f_{ub}}{f_u} = \frac{400}{410} = 0.975$

iv. $1.0 = 1$

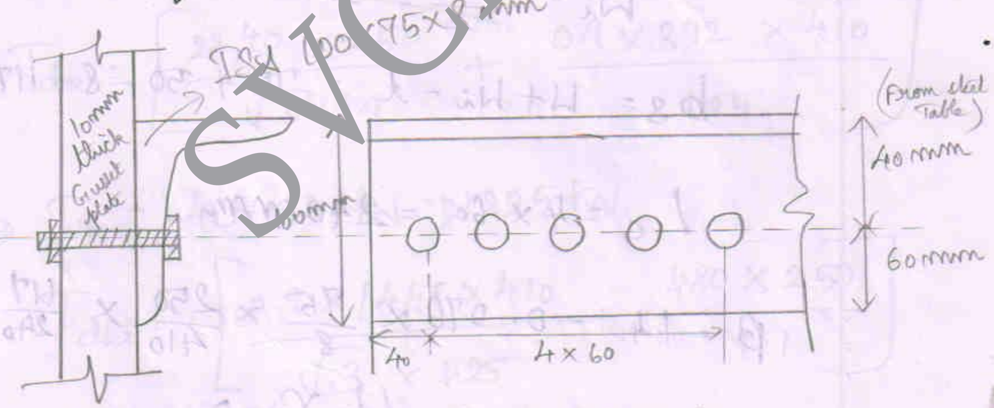
$\Rightarrow K_b = 0.606$

$V_{npb} = 2.5 \times 0.606 \times 20 \times 8 \times 410 = 99.384$

$V_{dpb} = \frac{99.384}{1.25} = 79.51$ kN

∴ Strength of bolt $= 45.27$ kN

no. of bolts $= \frac{225}{45.27} = 4.97 \simeq 5$ bolts



Step-4: Check the design

i. Strength due to yielding:

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

$= \frac{1336 \times 250}{1.1} = 303.63$ kN > 225 kN

ii. Strength due to rupture :-

$$T_{dn} = \left[\frac{0.9 \times A_{nc} f_u}{\gamma_{m1}} + \frac{\beta A_{go} f_y}{\gamma_{m0}}\right]$$

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

$$A_{go} = \left(75 - \frac{8}{2}\right) \times 8 = 568 \text{ mm}^2$$

$$\beta = 1.4 - 0.076 \times \frac{w}{t} \times \frac{f_y}{f_u} \times \frac{b_s}{L_c}$$

$w = 75$ mm

$w_1 = \cancel{\frac{100}{2} = 50 \text{ mm}}$ $\quad 0.6 g = 40$ mm

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

$L_c = 4 \times 60 = 240$ mm

$$\beta = 1.4 - 0.076 \times \frac{75}{8} \times \frac{250}{410} \times \frac{117}{240}$$

$$\beta = 1.18 \leq \left(\frac{f_u \gamma_{m0}}{f_y \gamma_{m1}}\right) \geq 0.7$$

$$T_{dn} = \left[\frac{0.9 \times 592 \times 410}{1.25} + \frac{1.18 \times 568 \times 250}{1.1}\right]$$

$T_{dn} = 327.09$ kN > 225 kN

Strength due to block shear :-

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

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

$$A_{vg} = \left[40 + (4 \times 60)\right] \times 8 = 2240\ mm^2$$

$$A_{tg} = 60 \times 8 = 480\ mm^2$$

$$A_{vn} = \left[(40 + (4 \times 60)) - (4.5 \times 22)\right] \times 8 = 1448\ mm^2$$

$$A_{tn} = \left[60 - (0.5 \times 22)\right] \times 8 = 392\ mm^2$$

$$T_{db_1} = \left[\frac{2240 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 392 \times 410}{1.25}\right]$$

$$\Rightarrow T_{db_1} = 409.61\ kN > 225\ kN$$

$$T_{db_2} = \left[\frac{0.9 \times 1448 \times 410}{\sqrt{3} \times 1.25} + \frac{480 \times 250}{1.1}\right]$$

$$\Rightarrow T_{db_2} = 355.88\ kN > 225\ kN$$

Hence the design is safe//.

2. Design a double angle tension member connected on each side of a 10mm thick gusset plate to carry an axial factored load of 375 kN. Use 20mm black holes. Assume shop connections.

18/2/15

Soln:

Step 1: Gross area of the section:

from code pg: 32.

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

$$= \frac{375 \times 10^3 \times 1.1}{250}$$

$$= 1650\,mm^2$$

Step 2: Select suitable shape.

Try 2 ISA 75 x 50 x 8mm

from steel table $A_g = 938mm^2$.

Area of 2 angle = $1876mm^2$

Gross area of the section = $1876mm^2$.

Step 3: No. of bold calculation

$d = 20mm$, $d_0 = 22mm$, $f_{ub} = 400$ Mpa, $f_u = 410$ Mpa

a) Strength of bolt in shear.

$$V_{nsb} = \frac{f_{ub}}{\sqrt{3}} (n_n A$$

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

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

$n_n = 1$, $n_s = 1$ (due to double shear)

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

$= 129.14$ kN.

$$V_{dsb} = \frac{129.14}{1.25}$$

$= 103.31$ kN.

b) Strength of bolt in bearing :

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

$V_{npb} = 2.5\, k_b \times d \times t\, f_u$

pitch distance :

from code pg: 73 class 10.2.2.

$p \nless 2.5\, d = 2.5 \times 20 = 50$ mm $\simeq 60$ mm

Edge distance :

from code pg: 74 class 10.2.4.2

$e \nless 1.5 d_0 = 1.5 \times 22 = 33$ mm $\simeq 40$ mm.

k_b is least

i) $\frac{e}{3d_0} = \frac{40}{3 \times 22} = 0.606$

ii) $\frac{p}{} - 0.25 = \frac{60}{} - 0.25 = 0.659$

iii) $\frac{f_{ub}}{f_u} = \frac{400}{410} = 0.975$.

iv) $1.0 = 1.0$

$\therefore k_b = 0.606$

$V_{npb} = 2.5 \times 0.606 \times 20 \times 8 \times 410$

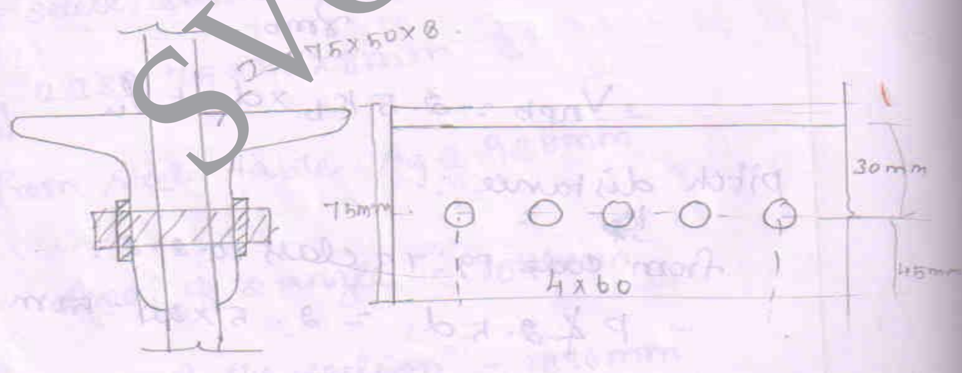
$V_{dpb} = \frac{99.384 \text{ kN}}{1.25}$

$= 79.5 \text{ kN}$.

strength of bolt $= 79.5$ kN.

no. of bolts $= \frac{375}{79.55} = 4.72 \simeq 5$ bolts

Step 5:



Step 4: check the design:

i) strength due to yielding

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

$$= \frac{1876 \times 250}{1.1}$$

$\ldots$ kN > 375 kN

ii) Strength due to rupture:

from code pg: 33, class 6.3.3

$$T_{dn} = \frac{0.9 A_n f_u}{\gamma_{mL}} + \beta \frac{A_{go} f_y}{\gamma_{mo}}$$

$$A_{nc} = \left(\left(75 - 8/2\right) - 22\right) \times 8 = 392\,mm^2 \times 2 = 784\,mm^2$$

$$A_{go} = (50 - 8/2) \times 8 = 368\,mm^2 \times 2 = 736\,mm^2$$

$w = 50\,mm$

$$w_i = \frac{75}{2} = 37.5\,mm$$

$$b_s = w + w_i - t = 50 + 37.5 - 8 = 79.5\,mm$$

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

$$\beta = 1.4 - 0.076 \left(w/t\right)\left(\frac{f_y}{f_u}\right)\left(b_s/L_c\right)$$

$$= 1.4 - 0.076\left(\frac{50}{8}\right)\left(\frac{250}{410}\right)\left(\frac{79.5}{240}\right)$$

$$= 1.3 \leq \frac{f_u \gamma_{mo}}{\gamma_{mL} f_y} \geq 0.7$$

$$T_{dn} = \frac{0.9 \times 784 \times 410}{1.25} + \frac{1.3 \times 736 \times 250}{1.1}$$

$$= 448.89\,kN$$

iii) Strength due to block shear:

$$T_{db1} = \left(\frac{A_{vg} f_y}{} + \frac{0.9 A_{tn} f_u}{}\right)$$

$$T_{db_2} = \left(\frac{0.9 A_{vn} f_u}{\sqrt{3}\,\gamma_{m1}} + \frac{A_{tg} f_y}{\gamma_{mo}}\right)$$

$A_{vg} = (30 + (4 \times 60)) \times 8 = 2160\,mm^2 \times 2 = 4320\,mm^2$

$A_{tg} = 45 \times 8 = 360\,mm^2 \times 2 = 720\,mm^2$

$A_{vn} = [30 + (4 \times 60) - (4.5 \times 22)] \times 8$

$= 1368\,mm^2$

$A_{tn} = [45 - (0.5 \times 22)] \times 8 = 272\,mm^2$

$$T_{db_1} = \frac{4320 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 272 \times 410}{1.25}$$

$= 647.14 > 375$

$$T_{db_2} = \frac{0.9 \times 1368 \times 410}{\sqrt{3} \times 1.25} + \frac{720 \times 250}{1.1}$$

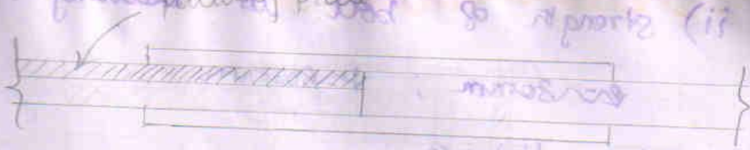
$= 396.78 > 375$

Hence the design is safe.

24-2-2015 Tension Member Splice:

If the tension members are unequal thickness then packing plates are used to surface of the tension member splice.

packing plate



Tension splice

24/2/15

1. Design a splice to connect a 300 mm x 20 mm plate with a 300 mm x 10 mm plate. The design load is 500 kN. Use 20 mm black holes, fabricated in the shop.

Soln:

Let double cover butt joint with 6 mm cover plate be used.

$d = 20\text{mm}$; $d_0 = 22\text{mm}$

$f_u = 410\text{ N/mm}^2$; $f_{ub} = 400\text{ N/mm}^2$

i) Strength of Bolt in shear:

Use packing plate condition.

$$\beta_{pk} = 1 - 0.0125 \times t = 1 - 0.0125 \times 10$$

$$= 0.875$$

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

$$= \frac{0.875 \times 400 \left[1 \times 0.78 \times \frac{\pi}{4} \times (20)^2 + 1 \times \frac{\pi}{4} \times (20)^2\right]}{\sqrt{3} \times 1.25}$$

$$= 90.39\text{ kN}$$

ii) strength of bolt in bearing :

e = 30mm ; p = 60mm.

i) Edge distance :

$e \nless 1.5 d_0 = 1.5 \times 22 = 33 \simeq 40 mm$.

ii) pitch distance :

$p \nless 2.5 d = 2.5 \times 20 = 50 mm \simeq 60mm$

k_b is least

i) $\frac{e}{3d_0} = \frac{40}{3 \times 22} = 0.606$.

ii) $\frac{p}{3d_0} - 0.25 = \frac{60}{3 \times 22} - 0.25 = 0.659$

iii) $\frac{f_{ub}}{f_u} = \frac{400}{410} = 0.975$.

iv) $1.0 = 1.0$

$\therefore k_b = 0.606$

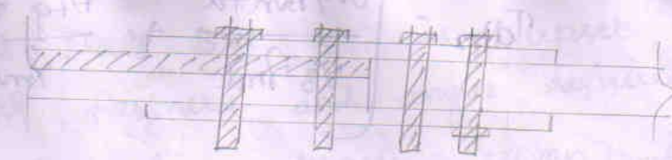
$V_{dpb} = \frac{2.5 k_b d t f_u}{\gamma_{mb}}$

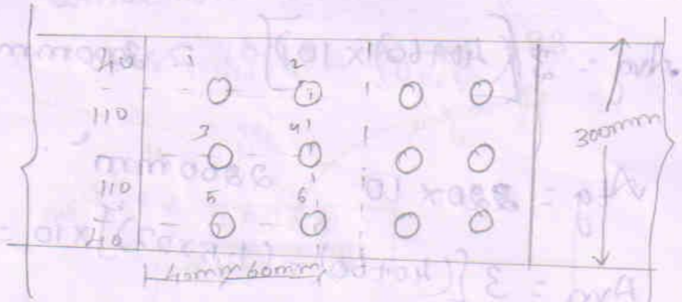
$= \frac{2.5 \times 0.606 \times 20 \times 10 \times 400}{1.25}$

$V_{dpb} = 99.384 kN$

$\therefore$ strength of the bolt $= 90.39 kN$

$\therefore$ No of bolts $= \frac{500}{90.39} = 5.53 \simeq 6$ bolts.





Check the design:

i) strength against yielding

$$T_{dg} = \frac{A_g f_y}{\gamma_{m0}} = \frac{(300\times10)\times250}{1.1}$$

$$= 681.81\ kN > 500\ kN$$

ii) Strength of Rapture:

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

$$A_n = [300 - (3\times22)]\times10 = 2340\ mm^2$$

$$= \frac{0.9\times2340\times410}{1.25}$$

$$= 690.76\ kN > 500\ kN$$

iii) Strength due to block shear:

$$T_{db1} = \left(\frac{A_{vg} f_y}{\sqrt{3}\,\gamma_{m0}} + \frac{0.9 A_{tn} f_u}{\ldots}\right)$$

$$T_{db2} = \left[\frac{0.9 A_{vn} f_u}{\sqrt{3}\,\gamma_{m1}} + \frac{A_{tg} f_y}{\gamma_{m0}}\right]$$

$$A_{vg} = 3\left[(40+60)\times 10\right] = 3000\,mm^2$$

$$A_{tg} = 220 \times 10 = 2200\,mm^2$$

$$A_{vn} = 3\left[(40+60) - (1.5 \times 22)\right] \times 10 = 2010\,mm^2$$

$$A_{tn} = \left[220 - 2 \times 22\right] \times 10 = 1760\,mm^2$$

$$T_{db1} = \frac{3000 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 1760 \times 410}{1.25}$$

$$= 913.7\,kN > 500\,kN$$

$$T_{db2} = \frac{0.9 \times 2010 \times 410}{\sqrt{3} \times 1.25} + \frac{2200 \times 250}{1.1}$$

$$= 842.57\,kN > 500\,kN$$

Hence the design is safe.

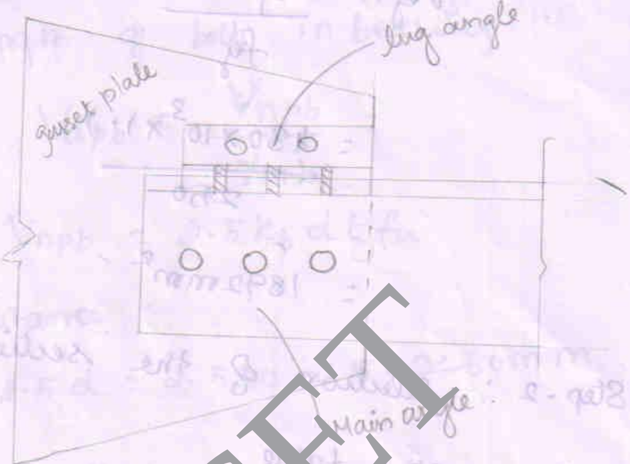
Lug Angles.

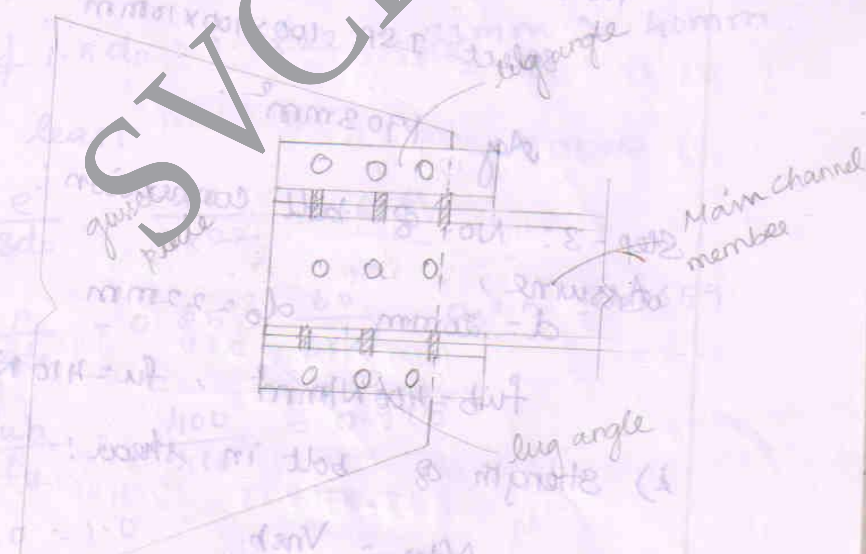
25/2/15

An additional angle is used along with a tension members for the length required to accomodate the connectors to make the

By using lug angles there will be saving in gusset plate. But it is upset by additional fastners (bolts) and angles required.

The connection details given in code IS 800:2007 class 10.12 pg.NO - 83.





1. A tension member of roof truss carries a factor axial tension of 430kN. Design the section and its connection a) without using lug angle b) using lug angle

Soln:

a) without using lug angle:

Step 1: Cross Area of the section

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

$$= \frac{430 \times 10^3 \times 1.1}{250}$$

$$= 1892\ mm^2$$

Step - 2: Selection of the section.

from steel table

select ISA 100 × 100 × 10mm.

$A_g = 1$[illegible]$3\ mm^2$.

Step - 3: No. of bolt connection:

Assume,

$d = 20mm$, $d_0 = 22mm$

$f_{ub} = 400\ N/mm^2$, $f_u = 410\ N/mm^2$

i) Strength of bolt in shear:

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

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

$n_n = 1$, $n_s = 0$ (due to single ~~double~~ shear)

$$= 400\left(1 \times 0.78 \times \frac{\pi}{4} \times (20)^2 + \cancel{1 \times \frac{\pi}{4} \times (20)^2}\right)$$

$= \cancel{129.14\,kN}\ 56.59\,kN$

$$V_{dsb} = \frac{\cancel{129.14}}{\cancel{1.25}}\ \frac{56.59\,kN}{1.25}$$

$= \cancel{103.31\,kN}\ 45.27\,kN.$

b) strength of bolt in bearing:

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

$$V_{npb} = 2.5\, k_b\, d\, t\, f_u$$

pitch distance :

$p \nless 2.5d = 2.5 \times 20 = 50 \simeq 60\,mm.$

edge distance :

$e \nless 1.5 d_0 = 1.5 \times 22 = 33\,mm \simeq 40\,mm.$

k_b is least

i) $\frac{e}{3d_0} = \frac{40}{3\times22} = 0.606$

ii) $\frac{p}{3d_0} - 0.25 = \frac{60}{3\times22} - 0.25 = 0.659$

iii) $\frac{f_{ub}}{f_u} = \frac{400}{410} = 0.975$

iv) $1.0 = 1.0$

$\therefore k_b = 0.606.$

$V_{npb} = 2.5 \times 0.606 \times 20 \times 10 \times 410$

$= 124.23\,kN.$

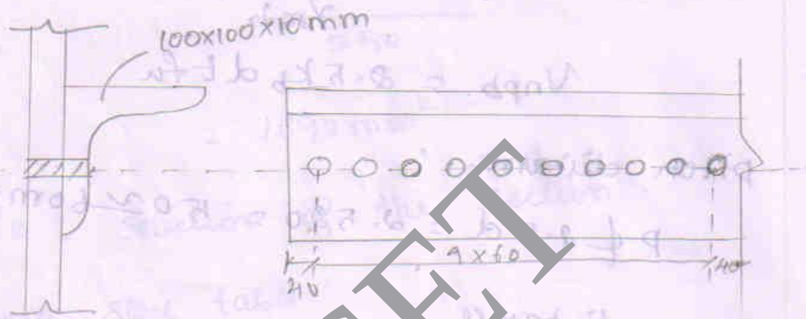
$V_{dpb} = \frac{124.23}{1.25} = 99.38\,kN.$

$\therefore$ No. of bolts $= \dfrac{430}{45.27}$

$= 9.49 \simeq 10$ bolts.

$\therefore$ length of the connection $= 40 + (9 \times 60)$

$= 580mm$.



Check the design:

1) Strength against yielding:

$T_{dg} = \dfrac{A_g f_y}{\gamma_{m0}}$

$= \dfrac{\cancel{(100 \times 10) \times 250}}{\cancel{1.1}}$ $\quad \dfrac{1903 \times 250}{1.1}$

$T_{dg} = 432.51$ kN > 430 kN

ii) Strength of Rapture:

from code Pg-33 class 6.3.3.

$T_{dn} = \left[\dfrac{0.9 \times A_{mc} \times f_u}{\gamma_{ml}} + \dfrac{\beta A_{go} f_y}{\gamma_{m0}}\right]$

$A_{nc} = \left(\left(100 - \frac{10}{2}\right) - 22\right) \times 10 = 730 mm^2$

$A_{go} = \left(100 - \frac{10}{2}\right) \times 10 = 950 mm^2$

from code pg-33.

$\beta = 1.4 - 0.076 \times w/t \times \frac{f_y}{f_u} \times \frac{b_s}{L_c}$

$w = 100 mm$

from steel table.

$w_i = C_{xx} = 28.4 mm$

$b_s = w + w_i - t = 100 + 28.4 - 10 = 118.4 mm.$

$L_c = 9 \times 60 = 540 mm$

$\beta = 1.4 - 0.076 \times \frac{100}{10} \times \frac{250}{410} \times \frac{118.4}{540}$

$\beta = 1.29 \le \frac{f_u \gamma_{m0}}{\gamma_{m1} f_y} \ge 0.7$

$T_{dn} = \left[\frac{0.9 \times 730 \times 410}{1.25} + \frac{1.29 \times 950 \times 250}{1.1}\right]$

$T_{dn} = 495.02 kN > 430 kN$

26|2|15 b) Using lug angle:

Gross Area of connected leg = Gross area of out standing leg

∴ load is shared equally.

∴ load in out standing leg = load in connected leg.

$= \frac{430}{2} = 215 KN$

(from code pg: 83, Clause 10.12.2)

The lug angle connected to the gusset to take a load of $\}$: $1.2 \times 215 = 258 kN$

$\therefore$ Gross area of lug angle Required $\Big\} = \dfrac{258 \times 10^3 \times 1.1}{250}$ $\quad A_g = \dfrac{T_{dn}\, \gamma_{m0}}{f_y}$

$= 1135\ mm^2$.

$\therefore$ provide ISA $100 \times 100 \times 6$ mm.

$\therefore A_g = 1167\ mm^2$

Bolt value.

i) In shear $= 45.27\ kN$.

ii) In bearing $= \dfrac{2.5 \times 0.606 \times 20 \times 6 \times 410}{1.25}$

$= 59.63\ kN$

$\therefore$ No. of bolts required for connection of lug angle with gusset $= \dfrac{258}{45.27} = 5.69 \simeq 6$ bolts

from code pg: 83 to 84 class 10.12.2

lug angle connected to the main angle $= 215 + \left(\dfrac{40}{100} \times 215\right)$

$= 301\ kN$

No. of bolts required to connect lug angle with main angle $\Bigg\} = \dfrac{301}{45.27} = 6.648 \simeq 7$

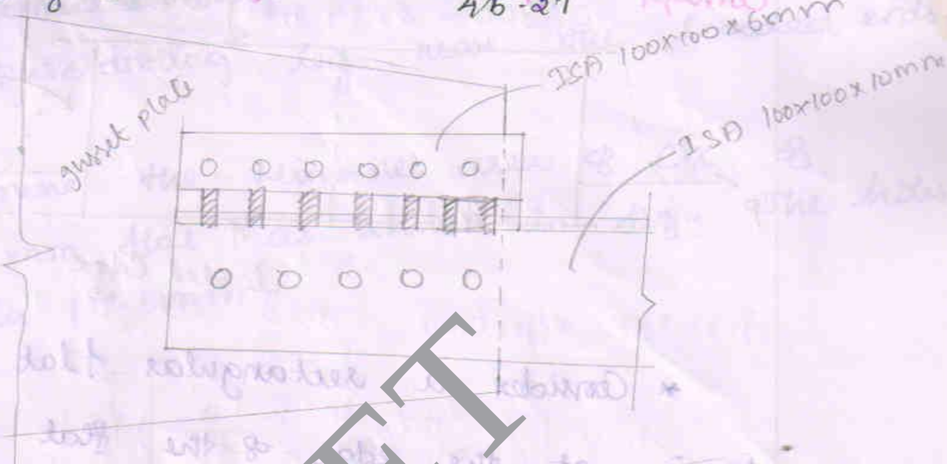
$= 7$ bolts.

Connection of Main angle to gusset plate;

force to be transfered = 215 kN

Bolt value = 45.27 kN.

No. of bolts required = $\frac{215}{45.27}$ = 4.74 ≈ 5 bolts.



26/2/15 : Required length of the connection } = 40 + 6 x 60

= 400mm.

Result :

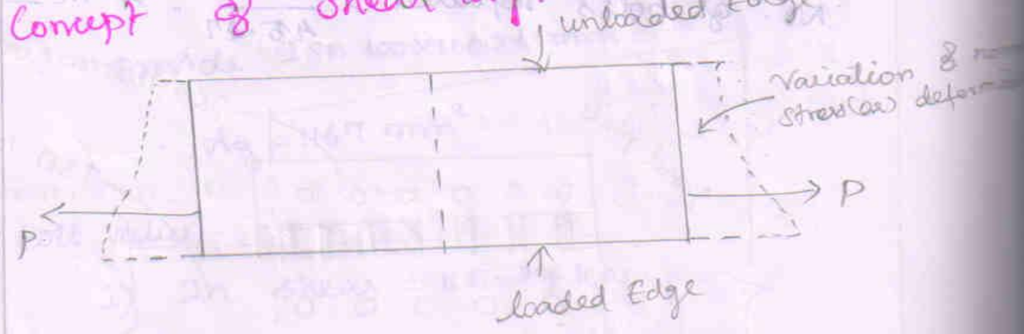
a) length of the connection (without using lug angle) } = 580mm.

b) length of the connection (using lug angle) } = 400mm.

Shear lag

In any section, when the forces are applied is non-uniform, the more normal stress (or) elongation along the loaded edge and less normal stress as phenomenon which is due to shear stress and shear deformation is known as shear lag.

Concept of shear lag:



* Consider a rectangular flat subjected to tension at the edge of the flat as shown in fig.

* The normal stress distribution and deformation in the flat near the ends, where the forces are applied is non-uniform, with more normal stress along the loaded edge & the less normal stress along the unloaded edge.

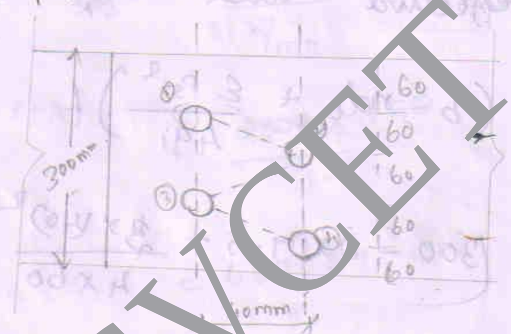
* The distribution of normal stress and deformation of the flat becomes uniform at the section away from the ends. That is in the middle of the flat.

* The design of steel section such as angles, channels and Tees as they are connected to a gusset through one leg or ... when used as tension or compression

* The connecting elements of the section are more stressed than the unconnected elements of the section near the loaded ends.

* for eg; in the case of single angle the connected leg is more stressed than the outstanding leg near the loaded ends.

1. Determine the Min. net area of C/s of 300x12 mm flat as shown in fig. The holes dia is 17.5mm.



Soln:

Pitch (p) = 40mm

gauge (g) = 60mm.

Edge (e) = 60mm.

hole's diameter = 17.5 mm.

∴ Dia of bolt = 15.5 mm.

i) Net area of the section at 1-2

code pg: 33.

$$A_n = \left[b_e - n d_0 + \Sigma \frac{P_{si}^2}{4 g_i} \right] \times t$$

$$= (300 - 1 \times 17.5) \times 12$$

ii) Net effective area at section ①-②-③

$$A_n = \left(b - nd_0 + \Sigma \frac{P_{si}^2}{4g_i}\right) \times t$$

$$= \left(300 - 2 \times 17.5 + \frac{2 \times (40)^2}{4 \times 60}\right) \times 12$$

$A_n = 3340 \text{ mm}^2$.

iii) Net effective area at section ①-④

$$A_n = \left(b - nd_0 + \frac{\Sigma P_{si}}{4g_i}\right) t$$

$$= \left(300 - 2 \times 17.5 + \frac{3 \times (40)^2}{4 \times 60}\right) \times 12$$

$= 3420 \text{ mm}^2$.

∴ Result:

Min. net effective area = 3340 mm^2.

2. Determine the ultimate load carrying capacity of the section 2ISA 75×75×10mm connected back to back same side of the gusset. Assume M_{24} bolt of property class 4.6. The yield & ultimate strength of steel 250 & 410 Mpa respectively.

Soln:

Dia of the bolt hole

$d_0 = 24 + 2 = 26\,mm$.

$d = 24\,mm$.

Gross area of the section

$= 2 \times 1402 = 2804\,mm^2$.

(from steel table)

i) Design strength due to yielding

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

$$= \frac{2804 \times 250}{1.1}$$

$= 637.272\,kN$.

ii) Design of connections:

Strength of shear:

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

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

$n_n = 1$, $n_s = 0$.

$$= \frac{400}{\sqrt{3}} \left(1 \times 0.78 \times \frac{\pi}{4} \times (24)^2 + \cancel{1 \times \frac{\pi}{4} \times (24)^2} \right)$$

$= 81.49\,kN$.

$$V_{dsb} = \frac{81.49}{1.25} = 65.19\,kN$$

Strength of bearing :

i) pitch distance :

$p = 2.5 \times 24 = 60mm$.

ii) Edge distance .

$e = 1.5 \times 26 = 39 \simeq 40mm$.

Strength of bolt in bearing

$V_{npb} = 2.5 \times k_b d t f_u$

k_b is least of ~~bearing~~ following,

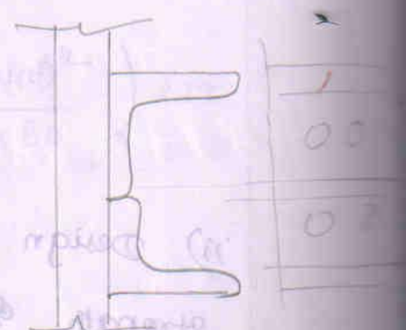
i) $\frac{e}{3d_0} = \frac{40}{3 \times 26} = 0.512$

ii) $\frac{p}{3d_0} - 0.25 = \frac{60}{3 \times 26} - 0.25 = 0.519$

iii) $\frac{f_{ub}}{f_u} = \frac{400}{410} = 0.975$

iv) $1.0 = 1.0$

$\therefore k_b = 0.512$.



Strength of bolt in bearing , face to face Connection

$V_{npb} = 2.5 \times 0.512 \times 24 \times 10 \times 410$

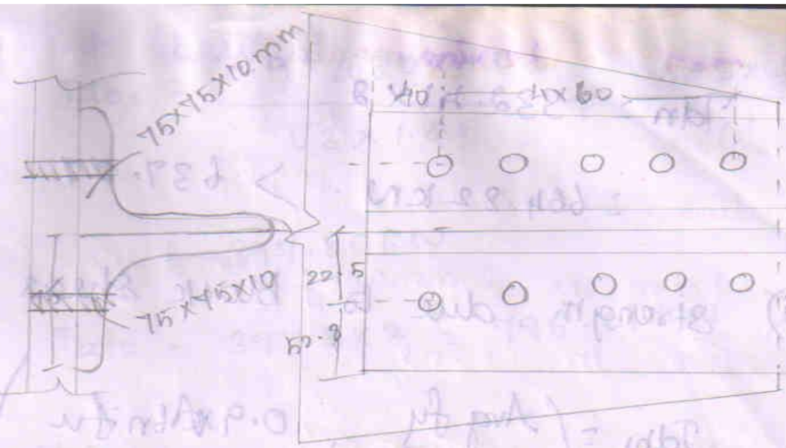
$= 125 kN$.

$V_{dpb} = \frac{125}{1.25} = 100.76 kN$.

Strength of the bolt $= 65.19 kN$.

No. of bolts $= \frac{637.272}{65.19} = 9.77 \simeq 10$ bolts

$= \frac{10}{2} = 5$ bolts



Check design:

i) ~~the~~ Strength due to Rapture:

$$T_{dn} = \left[\frac{0.9 A_n f_u}{\gamma_{m1}} + \frac{\beta A_{go} f_y}{\gamma_{m0}}\right]$$

$$A_n = \left[\left(75 - \frac{10}{2}\right) - 22\right) \times 10 = 440\,mm^2$$

$$A_{go} = \left(75 - \frac{10}{2}\right) \times 10 = 700\,mm^2$$

$$\beta = 1.4 - 0.076 \times \frac{w}{t} \times \frac{f_y}{f_u} \times \frac{b_s}{L_c}$$

$w = 75\,mm$

$w_i = C_{xx} = 22.2$

$b_s = w + w_i - t = 75 + 22.2 - 10 = 87.5\,mm$

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

$$\beta = 1.4 - 0.076 \times \frac{75}{10} \times \frac{250}{410} \times \frac{87.5}{240}$$

$$= 1.273 \le \frac{f_u \gamma_{m0}}{\gamma_{m1} f_y} \geq 0.7$$

$$T_{dn} = \left[\frac{0.9 \times 440 \times 410}{1.25} + \frac{1.273 \times 700 \times 250}{1.1}\right]$$

$T_{dn} = 332.41 \times 2$

$= 664.82\,kN > 637.27$

ii) Strength due to block shear.

$$T_{db1} = \left(\frac{A_{vg} f_y}{\sqrt{3}\,\gamma_{m0}} + \frac{0.9 \times A_{tn} f_u}{\gamma_{m1}}\right)$$

$$T_{db2} = \left(\frac{0.9 A_{vn} f_u}{\sqrt{3}\,\gamma_{m1}} + \frac{A_{tg} f_y}{\gamma_{m0}}\right)$$

$A_{vg} = (40 + (4 \times 60)) \times 10 = 2800\,mm^2$

$A_{tg} = \overset{52.8}{\cancel{35}} \times 10 \quad \cancel{350\,mm^2} \quad 528\,mm^2$

$A_{vn} = [40 + (4 \times 60) - (4.5 \times 26)] \times 10$

$= 1630\,mm^2$

$A_{tn} = [52.8 - (0.5 \times 26)] \times 10$

$= \cancel{220\,mm^2} \quad 398\,mm^2$

$$T_{db1} = \frac{2800 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 398 \times 410}{1.25}$$

$= 484.89\,kN$

$T_{db1} = 484.89 \times 2 = 969.78\,kN > 637.27$