10.2.17

chapter 7 Duggal book

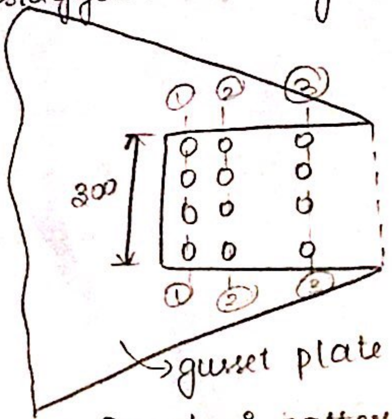
UNIT- II

TENSION MEMBERS

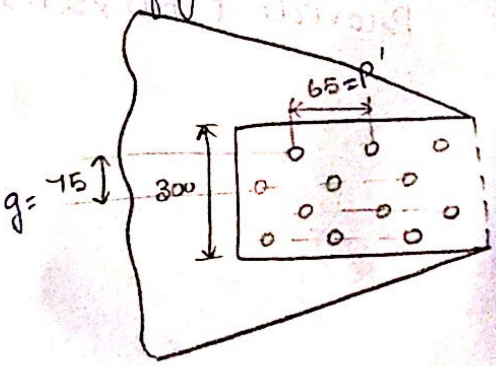
IS 800 : 2007 [CIS 6.2, 6.3, 6.3.1, 6.3.2, 6.3.3, 6.3.4, 6.4, 6.4.2]

Types of sections - net area - net effective sections for angles and Tee in tension - design of connection in tension member - design of lug angles - design of tension ~~splice~~ splice - concept of shear log.

1. A 300 mm ISF, 8mm grade FE410 is used as a tension member in a lattice girder is connected to a 12mm thick gusset plate by 18mm dia bolt. calculate effective area of the member in chain and staggered bolting as shown in figure① & ② respectively.



Fig①: chain pattern



Fig②: staggered bolting

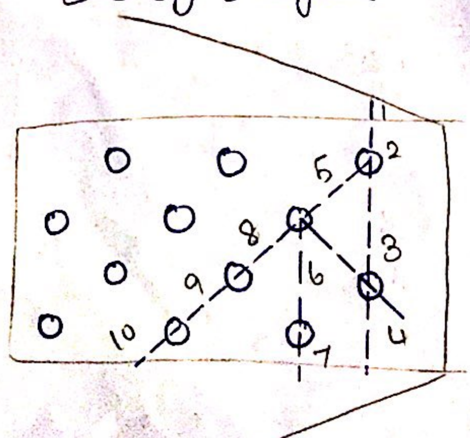
① For chain bolting (Fig①): → Pg: 33

$Eff\ A_n = (B - n d_n) t$ → n: no. of bolts in one critical section

$= (300 - 4 \times 20) 8$

$= 1760 mm^2$

② For staggered bolting (Fig②):-



In zig-zag bolting sections may fail along following section.

1-2-3-4

1-2-5-6-7

1-2-5-3-4

1-2-5-8-9-10

ⓐ **Net eff. area = 1-2-3-4**

$$A_n = (B - n d_n)t$$

$$= (300 - 2\times20)8$$

$$= 2080\,mm^2$$

ⓑ **Net eff. area 1-2-5-6-7**

5 is a ↓

n' ⇒ slanting portion

n' = 1

~~$A_n = (300 - 3\times2$~~

$$A_n = \left(B - n d_n + \frac{n'p'}{4g}\right)t$$

$$= \left(300 - 3\times20 + \frac{1\times65}{4\times75}\right)8$$

$$= \text{~~203266 mm}^2\text{~~}\ 1921.73\,mm^2$$

ⓒ **Net eff. area 1-2-5-3-4** n' = 3

$$A_n = \left(B - 3\times20 + \frac{3\times65}{4\times75}\right)8$$

$$= \text{~~2143.33 mm}^2\text{~~}\ 1925.2\,mm^2$$

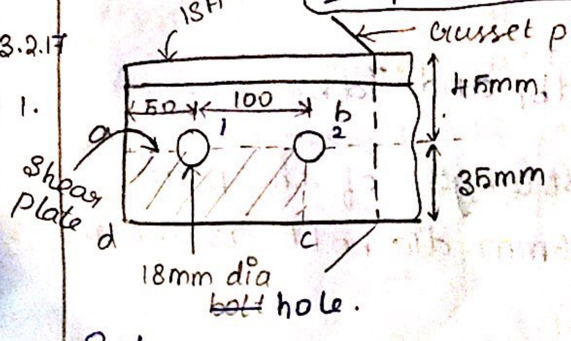
ⓓ **Net eff. area 1-2-5-8-9-10** n' = 2

$$A_n = \left(300 - 4\times20 + \frac{3\times65}{4\times75}\right)8$$

$$= 1765.2\,mm^2$$

13.2.17

1. Determine block shear strength of tension figure as shown in fig.



Sol: Pg:- 33 cls (6.4.1)

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

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

A_{vg} = [(no. of pitch × pitch distance) + end distance] × t

A_{vg} → Gross area

t → thickness

$A_{vg} = [(1\times100)+50]\times8$

$= 1200\,mm^2$

$A_{vn} = [[(1\times100)+50] - [(\text{no. of hole} - \tfrac{1}{2})\times \text{dia of hole}]]\times t$

$= [[(1\times100)+50] - [(2-\tfrac{1}{2})\times18]]\,8$

$= 984\,mm^2$

A_{tg} = ((distance from bottom to centre of hole) × t)

$A_{tg} = (35\times8) = 280\,mm^2$

$A_{tn} = [(35 - (\tfrac{1}{2}\times18))]\times t$ (18 → dia of hole)

$= (35 - (\tfrac{1}{2}\times18))\times8$

$= 208\,mm^2$

$T_{db_1} = \left[\dfrac{1800\times250}{\sqrt{3}\times1.1}\right] + \left[\dfrac{0.9\times208\times410}{1.25}\right]$

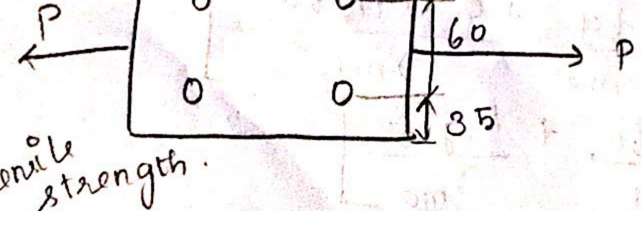
~~= 21915~~ 218860.76 N = 218.86 KN

$T_{db_2} = \left[\dfrac{0.9\times984\times410}{\sqrt{3}\times1.25}\right] + \left[\dfrac{280\times250}{1.1}\right]$

= 231.343 KN

Take minimum of T_{db_1} and T_{db_2}.

2. Determine the tensile strength of the plate 200×12mm with holes of 16mm dia bolt as shown in fig.



tensile strength.

4. A single unequal ISA 90x60x6 is connected to a 10mm gusset plate at the end with 5 nos of 16mm φ bolt to transfer tension. Determine the tensile strength of the angle if (i) The gusset plate is connected to 90mm leg (ii) The gusset plate is connected to 60mm leg. Take pitch as 40mm and edge distance 30mm.

Case (i): Sol: ① Design strength due to yielding:

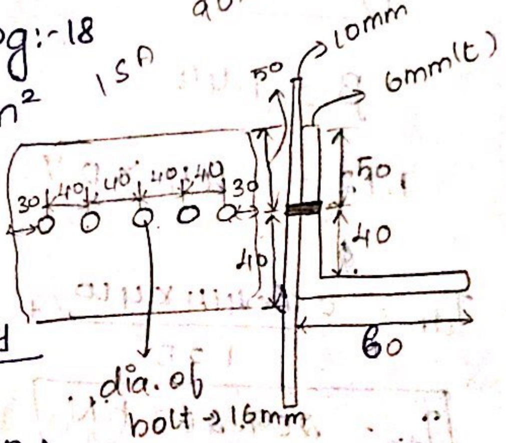
Pg: 33, cl: 6.2 (Yielding).

For ISA 90x60x6 Pg:-18

$A_g = 865 mm^2$

$F_y = 250$

$\gamma_{m0} = 1.1$



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

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

$$\boxed{T_{dg} = 196 KN}$$

90mm side

② Design strength due to rupture:

IS800 → cl:-6.3.3

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

A_{nc} = net area of connected leg.

$$= \left(h - \frac{\text{thickness of L angle}}{2} - \text{dia of hole}\right) \times t$$

$$= \left(90 - \frac{6}{2} - 18\right) \times 6$$

$$= 414 mm^2$$

A_{go} = gross area of out standing leg.

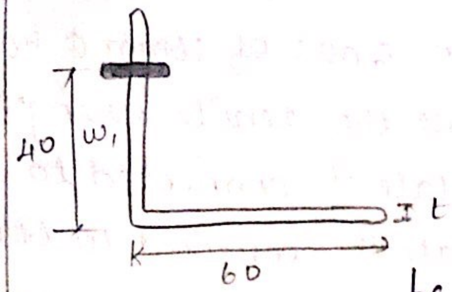
$$= \left(60 - 6/2\right) \times 6$$

$$= 342 mm^2.$$

cl-6.3.3

$$\beta = 1.4 - 0.076 (w/t)(f_y/f_u)(b_s/L_c) \leq \left(\frac{f_u \gamma_{m0}}{f_y \gamma_{m1}}\right)$$

$$\geq 0.7$$



$b_s = w + w_1 - t$
$= 60 + 40 - 6$
$b_s = 94 mm$

(pitch distance)
$L_c = 4 \times 40 = 160 mm.$

$\beta = 1.4 - 0.076 (60/6) \left(\frac{250}{410}\right)\left(\frac{94}{160}\right)$

$\beta = 1.12$

$\beta \leq 1.44$

$1.12 \leq 1.44 \geq 0.7$

Check:
$\frac{f_u \gamma_{mo}}{\gamma_{mo}} / \frac{f_y \times \gamma_{m1}}{\gamma_{m1}}$
$= \frac{\left(\frac{410}{1.1}\right)}{\left(\frac{250}{1.25}\right)} = \frac{410 \times 1.1}{250 \times 1.25}$

$T_{dn} = \frac{0.9 \times 414 \times 410}{1.25} + \frac{(1.12 \times 342 \times 250)}{1.1}$

$T_{dn} = 209.267 KN$

③ Block shear: (Design strength):

14.2.17 Cl:- 6.4.1 Pg:- 33

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

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

$A_{vg} = [30 + (4 \times 40)] \times 6 = 1140 mm^2$

$A_{vn} = [[30 + (4 \times 40)] - ((5 \frac{1}{2}) \times 18)] \times 6 = 654 mm^2$

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

$A_{tn} = (40 - (\frac{1}{2} \times 18)) \times 6 = 186 mm^2$

$T_{db_1} = \frac{1140 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 186 \times 410}{1.25} = 204.49 KN$

$T_{db_2} = \frac{0.9 \times 654 \times 410}{\sqrt{3} \times 1.25} + \frac{240 \times 250}{1.1} = 166 KN$

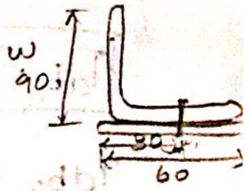
Taking least value $T_{db_2} = 166 KN$

Case(ii): If the gusset plate connected to 60mm leg
(1) Design strength due to yielding:

cl 6.2,
$$T_{dg} = \frac{A_g f_y}{\gamma_{m0}}$$
$$= \frac{865 \times 250}{1.1}$$
$$= 196.59 \text{ KN}$$

(2) Design strength due to rupture:

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



$$A_{nc} = (60 - \frac{6}{2} - 18) \times 6$$
$$\boxed{A_{nc} = 234 \text{ mm}^2}$$
$$A_{go} = (90 - 6/2) \times 6$$
$$\boxed{A_{go} = 522 \text{mm}^2}$$

$$\beta = 1.4 - 0.076\,(w/t)(f_y/f_u)(b_s/L_c) \leq (f_u \gamma_{m0} / f_y \gamma_{m1})$$
$$\geq 0.7$$

$$b_s = w + w_1 - t$$
$$= 90 + 30 - 6 = 114 \text{ mm}$$
$$l_c = 4 \times 40 = 160 \text{mm}$$

$$\beta = 1.4 - 0.076 \left(\frac{90}{6}\right)\left(\frac{250}{410}\right)\left(\frac{114}{160}\right) = 0.904$$

$$\frac{f_u \gamma_{m0}}{f_y \gamma_{m1}} = \frac{410 \times 1.1}{250 \times 1.25} = 1.44$$

$$\boxed{\beta = 0.9}$$

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

$$\boxed{T_{dn} = 175.8 \text{KN}}$$

(3) Design strength due to block shear:

cl: 6.4.1 Pg: 33

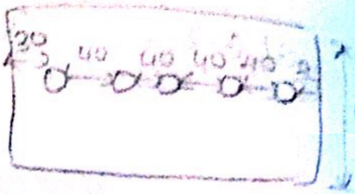
$$T_{db_1} = \frac{A_{vg} f_y}{\sqrt{3}\, \gamma_{m0}} + \frac{0.9 A_{tn} f_u}{\gamma_{m1}}$$

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

$A_{vg} = [30 + (4 \times 40)] \times 6 = 1140 mm^2$

$A_{vn} = [(30) + (4 \times 40) - (5 - 1/2) \times 18] \times 6.$

$= 654 mm^2$



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

$A_{tn} = (30 - (1/2 \times 18)) \times 6 = 126 mm^2$

$$T_{db_1} = \frac{1140 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 126 \times 410}{1.25}$$

$$\boxed{T_{db_1} = 186.78 KN}$$

$$T_{db_2} = \frac{0.9 \times 654 \times 410}{\sqrt{3} \times 1.25} + \frac{180 \times 250}{1.1}$$

$$\boxed{T_{db_2} = 152.37 KN}$$

Taking least value

$$\boxed{T_{db_2} = 152.37 KN}$$

11.5.2.17 1. ** Design a double angle tension member connected on each side of 18mm gusset plate to carry an axial factored load of 400KN. Use 20mm dia bolt.

① Design strength due to yielding:

cls (6.2) Pg: 30

[Design procedure for design of tension member:

Step ①: Find required factored gross area to carry the strength of yielding.

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

$A_g = ?$

Step ②: Select suitable section depending on the type of structure and location of the member and gross area = 25% to 40% of Ag calculated.

Step ③: Determine the no. of bolt (or) size of weld required.

Step ④: Determine the strength considering yielding, rupture, block shear.

Prob:

① Calculation of Ag:

cl (6.2) Pg: 30

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

$$400 \times 10^3 = \frac{A_g \times 250}{1.1}$$

$$A_g = 1760 \, mm^2$$

② Selection of section:

Assume 30% Gross area

$$= 1760 \times \frac{30}{100} = 528 \, mm^2$$

$$A_g = 1760 + 528$$

Area of 2 angles $= 2288 \, mm^2$

∴ Area of single angle $\simeq \frac{2288}{2}$

$$= 1144 \, mm^2.$$

ISA (90×60×8) mm → Pg: 17 suitable.

③ No. of bolts:

(a) shear capacity of bolt:

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

$$= \frac{400 \times 10^{-3}}{\sqrt{3} \times 1.25} \left(1 \times 0.78 \times \pi/4 \times 20^2 + 1 \times \pi/4 \times 20^2\right)$$

$$V_{dsb} = 103.314 \, kN$$

(b) Bearing capacity of bolt :

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

K_b is smaller of

(i) $\frac{e}{3d_0} = \frac{1.5 d_0}{3d_0} = 0.5$

(ii) $\frac{p}{3d_0} - 0.25 = \frac{2.5d}{3d_0} - 0.25 = 0.583$

(iii) $\frac{f_{ub}}{f_u} = 0.975$ (iv) 1

$K_b = 0.5$

$$V_{dpb} = \frac{2.5 \times 0.5 \times 20 \times 8 \times 410 \times 10^{-3}}{1.25}$$

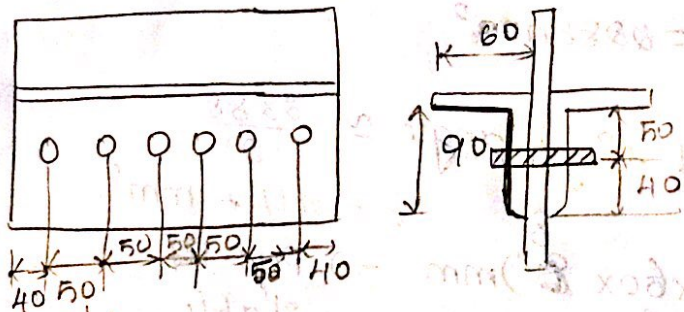
$= 66$

∴ strength of bolt = 82 kN

No. of bolt = $\frac{\text{Load}}{\text{strength}}$

$= \frac{400}{66} = 6.06$

$\simeq$ 6 nos.



$e = 1.5 d_0$
$= 33$
$\simeq 40$

$p = 2.5d$
$= 50$

④ Design strength due to yielding :

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

90x60x8 → $A_g = 1137$ mm

$= \frac{2 \times 1137 \times 250 \times 10^{-3}}{1.1}$

$T_{dg} = 258.4$ kN × 2 → double angle

$T_{dg} = 516$ kN > 400 kN

Hence safe

16.2.17 (5) Design strength due to rupture.

Pg: 33 cl: 6.3.3

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

$\alpha = 0.8$ (because 6 no. of bolts)

$A_n = A_{go} + A_{nc}$

$A_{nc} = (90 - \frac{8}{2} - 22) \times 8 \times 2$ → given prob & double angle.

$= 1024 mm^2$

$A_{go} = (60 - 8/2) \times 8 \times 2$

$= 896 mm^2$

$A_n = 1024 + 896$

$A_n = 1920 mm^2$

$T_{dn} = \frac{0.8 \times 1920 \times 410 \times 10^{-3}}{1.25}$

$= 503 KN > 400 KN$

Hence safe.

(6) Design strength due to block shear:

Pg: 33 cls (6.4)

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

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

$A_{vg} = (40 + (5 \times 50)) \times 8 = 2320 mm^2$

$A_{vn} = [(40 + (5 \times 50)) - (6 - ½) \times 22] \times 8 = 1352 mm^2$

$A_{tg} = (40 \times 8) = 320 mm^2$

$A_{tn} = [40 - (½ \times 22)] \times t$

$= (40 - (11)) \times 8 = 232 mm^2$

$T_{db} = \frac{2320 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 232 \times 410}{1.25}$

$T_{db_1} = 373\,KN \times 2 = 746\,KN > 400\,KN$ (double angle)

$$T_{db_2} = \frac{0.9 \times 1352 \times 410}{\sqrt{3} \times 1.25} + \frac{320 \times 250}{1.1}$$

$T_{db_2} = 303\,KN \times 2 = 606\,KN > 400\,KN$ (double angle)

T_{db_1} & $T_{db_2} > 400\,KN$

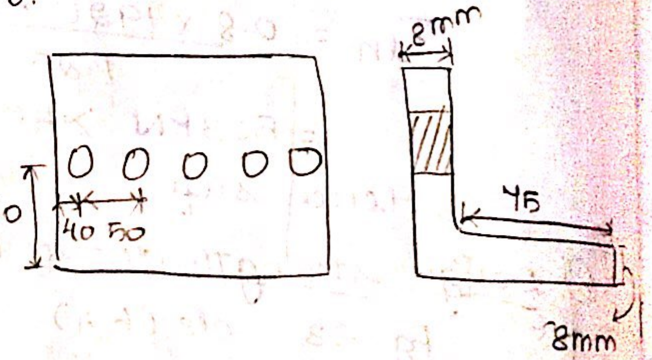
$\therefore$ Safe

Assignment :-

IA. Design a single angle section for a tension member of a roof truss to carry a factored load of 225 KN. The member is subjected to a possible reversal of stress due to action of beam. Effective length of the member is 3m. Use 20mm shop bolt of grade 4.6.

(35% Gross area)

20.2.17 **Step 1: Find gross Area**



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

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

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

$$= 990\,mm^2.$$

(2) Selection of section:

Assume 35% increase in gross area.

$$990 \times \frac{35}{100} = 346.5\,mm^2$$

$$A = 990 + 346.5 = 1336.5\,mm^2.$$

From steel table (5). select ISA unequal section of $100 \times 45 \times 8$.

Step 3: No. of bolts:

Shear capacity:

a) $V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}}$ $(n_s = 0)$ since single angle.

$= \frac{f_{ub}(n_n A_n + n_s A_s)}{\sqrt{3}\,\gamma_{mb}}$

$= 45.27\,kN$

Bearing capacity:

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

K_b:

e = 40, P =

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

(ii) $\frac{P}{3d_0} - 0.25 = \frac{50}{3 \times 22} - 0.25 = 0.507$

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

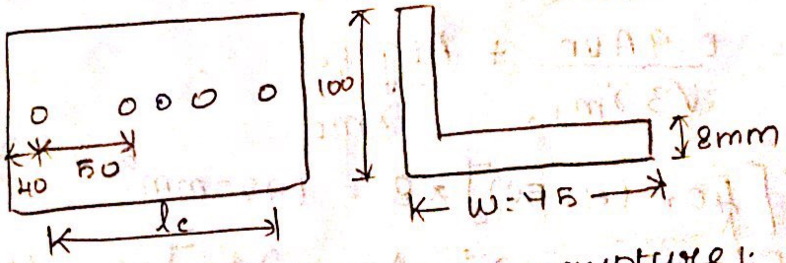
(iv) 1

$K_b = 0.507$

$V_{dpb} = \frac{2.5 \times 0.507 \times 20 \times 8 \times 410}{1.25}$

$= 66.58\,kN$

No. of bolts $= \frac{225}{45.27} = 4.97 = 5$ Nos.



④. Design strength due to rupture:

For single angle connection.

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

$A_{nc} = (100 - 8/2 - 22) \times 8 = 592 mm^2$

$A_{go} = (75 - 8/2) \times 8 = 568 mm^2$

$$\beta = 1.4 - 0.076\left(\frac{w}{t}\right)\left(\frac{f_y}{f_u}\right)\left(\frac{b_s}{l_c}\right) \le \frac{f_u \times \gamma_{mo}}{f_y \times \gamma_{m1}} \ge 0.7$$

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

$l_c = 4 \times 50 = 200 mm$

$\beta = 1.146$

$$\frac{f_u \times \gamma_{mo}}{f_y \times \gamma_{m1}} = 1.44.$$

$\therefore T_{dn} = 322.69 < 225 KN$

⑤ Design of strength due to yielding:

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

$A_g = (100 - 8/2) + (75 - 8/2) \times 8$

$= 1336 mm^2$

$$T_{dg} = \frac{1336 \times 250}{1.1} = 303.6 KN > 225 KN.$$

Hence safe.

⑥ Design strength due to block shear:

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

$$T_{db2} = \frac{0.9 A_{vn}}{\sqrt{3}\gamma_{m1}} + \frac{A_{tg} f_u}{\gamma_{mo}}$$

$A_{vg} = [40 + (4 \times 50)] \times 8 = 1920 mm^2.$

$A_{vn} = [(40 + 4(50)] - (5 - 1/2) 22] \times 8 = 1128 mm^2$

$A_{tg} = 50 \times 8 = 400 mm^2$

$A_{tn} = (50 - 1/2 \times 22) \times 8 = 312 mm^2$

Sub these in T_{db1} & T_{db2}

$T_{db1} = 344.037 KN$

$T_{db_2} = 288.16\,KN$

$\therefore$ Design strength due to Block shear $= 288.16\,KN$
$> 285\,KN$

$\therefore$ Safe

Step 4: check for slenderness ratio:

Cl: 3.8 Pg: 20

$\frac{K_L}{r}$ = slenderness ratio.

K_L = eff. length

r = radius of gyration

from steel table for ISA (100×75×8) m.

$r_x = 31.4$, $r_y = 21.8$, $r_u = 34.8$

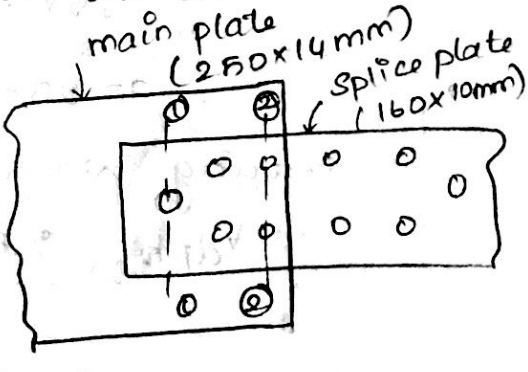
$r_v = 15.9$

Take least, $r = 15.9$

$\therefore \frac{K_L}{r} = \frac{3000}{15.9} = 188.69 < 350$ (Table 3, Pg 20) case (v).

Tension splice:

1. Design a splice for joining tension member sections, 160×10 mm and 250×14mm. The member is subjected to factored tensile load of 300 KN. Use 20mm φ bolt of grade 4.6



Design:

Design tensile strength of members.

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

$= \frac{(160 \times 10) \times 250 \times 10^{-3}}{1.1}$

$= 363.63\,KN$

(ii) Design strength due to rupture:-

$$T_{dn} = \frac{0.9 A_n f_u}{\gamma_{m1}}$$ (for plates) (6.2)

$A_n = (b - d_n) \cdot t = (160 - 22) \times 100 = 1380 mm^2$

$$T_{dn} = \frac{0.9 \times 1380 \times 410 \times 10^{-3}}{1.25} = 407.37 \text{ kN}.$$

∴ Design strength of member = 363.63 kN

(Take least of T_{dg} & T_{dn}) < 300 kN.

Hence take factored load = 300 kN

Since two plates are of different thickness.

Thickness of packing plate $t_{pbs} = 14 - 10$

$= 4 mm < 6 mm$

∴ Provide 6mm packing plate.

(iii) Shear strength of bolt:-

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

$$= \frac{400 \times 10^{-3}}{\sqrt{3} \times 1.25} (1 \times 0.78 \times \pi/4 \times 20^2) + (1 \times \pi/4 \times 20^2)$$

$= 90.25 kN$

Bearing strength of bolt

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

$$= \frac{2.5 \times 1 \times 20 \times 10 \times 410 \times 10^{-3}}{1.25}$$

Assume $k_b = 1$

$= 164 kN$

∴ No. of bolt $= \frac{300}{90.25} = 3.32 = 5$

Size of splice plate

width of splice plate = width of main plate = 160mm

1–1 → critical section for main plate.

2–2 → critical section for splice plate.

To get thickness of splice equate design strength at section 2-2 = Design tensile strength

$$300 = \frac{0.9 A_n f_u}{\gamma_{mb}}$$

front & back (two splice)

$$= \frac{0.9 \times (160 - 2\times 22) \times 2\, t_{sp} \times 410 \times 10^{-3}}{1.25}$$

$t_{sp} = 4.38 \simeq 6$ mm

Take

$p = 2.5d = 50$mm

$e = 1.5d_0 = 33 \simeq 35$mm

length of splice $= 4 \times (50+35)$

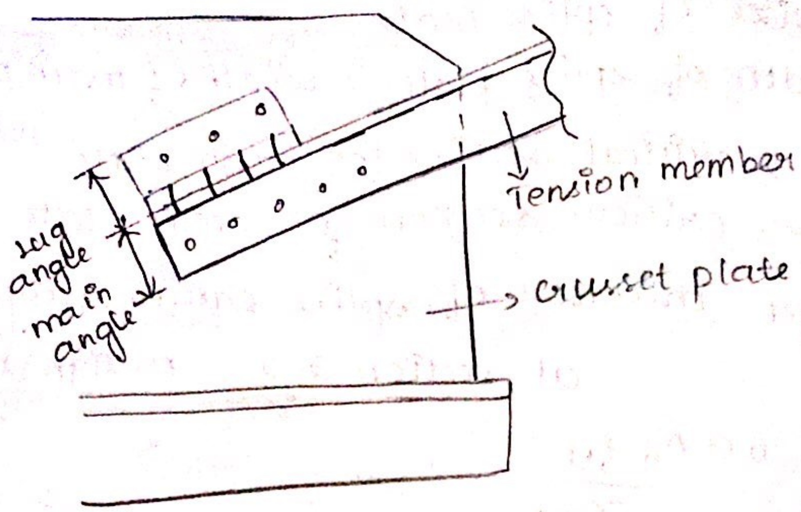
$= 340$mm

∴ Provide splice plate of size (340 × 160 × 6) mm.

24.2.17 **Lug angle:**

To make the connections at joints a certain length of tension member and that of gusset plate is utilized. If the load is heavy and no. of bolts or length of weld required for making connection is more (large), the size of gusset plate required will be uneconomical.

An alternative to this, an additional angle is used along with the tension member to reduce the length of the joint and size of gusset plate. Such angle is called lug angles.



Disadvantages of using lug angles:

Cost is more for connection and fabrication and providing lug angles can be eliminated by providing an unequal angle section with wider connected leg and two rows of staggered bolting.

1. A diagonal member of a roof truss carries a maximum pull of 300KN. Design the section and its connection with a 16mm thick gusset plate. Length of the connection is limited to 340mm. Design the lug angles if required. Use Fe410 grade steel and 4.6 grade bolt.

Sol: ① The ~~required area of~~ section:

Pg: 33 IS 800

$$T_{dn} = \frac{\alpha A_n f_u}{\gamma_{m_1}}$$

Assume no. of bolts 7 4 ∴ $\alpha = 0.8$

$$(1.5 \times 300 \times 10^3) \; T_{dn} = \frac{0.8 \times A_n \times 410}{1.25}$$

(considering as working load)

$A_n = 1714.93 mm^2$

② selection of angle section:-

Assume the area to be increased by 25%.

$= \left(1714.93 \times \frac{25}{100}\right) + 1714.93$

$A_g = 2143.6 mm^2$

Ref. steel book, Pg: 18 take ISA 150×75×10mm.

For ISA 150×75×10

$A_g = 2156 mm^2$

$r_v = 15.9mm$

③ No. of bolts:-

Assume the strength of the bolt in the shear is minimum.

Single shear

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

Use 20mm ϕ bolt

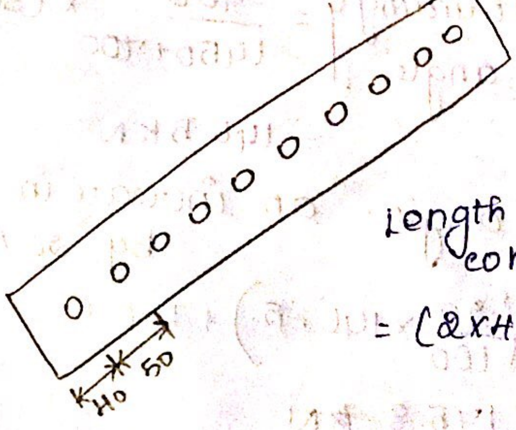
$= \frac{400 \times 10^{-3} \times 1 \times 0.78 \times \pi/4 \times 20^2}{\sqrt{3} \times 1.25}$

$= 45.27 KN$.

~~Assuming st~~ No. of bolts $= \frac{300 \times 1.5}{45.27}$

$= 9.94 \simeq 10$ NO.S

Provide 20mm ϕ bolt with 50mm pitch & 40mm edge distance.



Length of connection

$= (2 \times 40) + (9 \times 50) = 530mm$.

$530mm > 340mm$.

Hence lug angle is provided.

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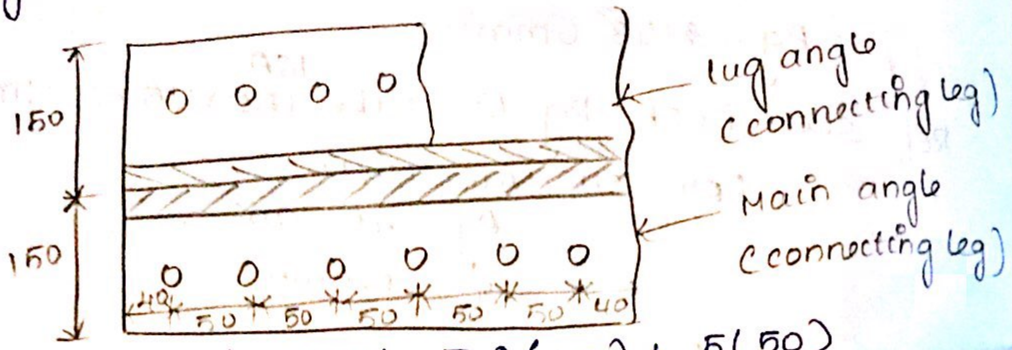
Provide 6 bolts for connecting main angle leg with gusset plate and 4 bolts for connecting lug angle leg with gusset plate as shown in figure.



Length of gusset plate required $= 2(40) + 5(50)$
$= 330 < 340\,mm$ (given)

Design of lug angle: for connection

For design of lug angle load in the outstanding leg of main angle is calculated by distributing the loads in the leg in the proportion of the areas shared by them.

Gross area connected leg of main angle

$= \left(150 - \frac{10}{2}\right) \times 10$

$= 1450\,mm^2.$

Gross area of outstanding leg of main angle

$= (75 - 10/2) \times 10$

$= 700\,mm^2$

Load shared by outstanding leg of main angle $= \frac{700}{1450+700} \times (300 \times 1.5)$

$= 146.5\,KN.$

Design load on lug angle = an increase in load by 20%

$= \left(\frac{20}{100} \times 146.5\right) + 146.5$

$= 175.8\ KN$

Area required for lug angle

$$T_1 = \frac{\alpha A_{n2} f_u}{\gamma_{m1}}$$

$$A_{n2} = \frac{T_1 \times \gamma_{m1}}{\alpha f_u}$$

$$= \frac{175.8 \times 10^3 \times 1.25}{0.8 \times 410}$$

$$= 669.96 mm^2$$

$(80 - 5/2) \times 5 = 387.5$

$\left(\frac{237.5}{387.5 + 237.5}\right) \times \left(\frac{300}{\times 1.1}\right) = 171$

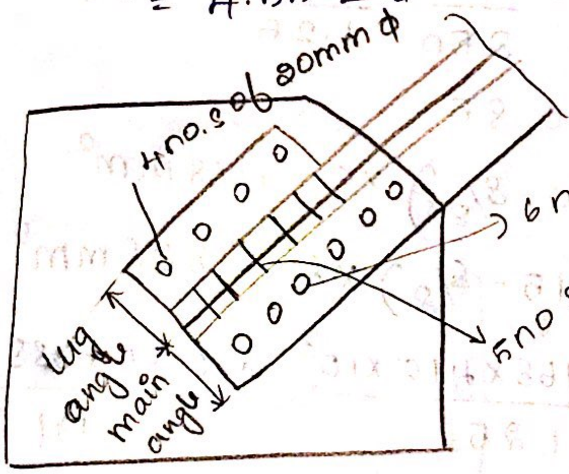
Refer, steel table IS ~~150×75×8~~ 80×50×5.

No. of bolts required to connect the outstanding leg of main angle and lug angle

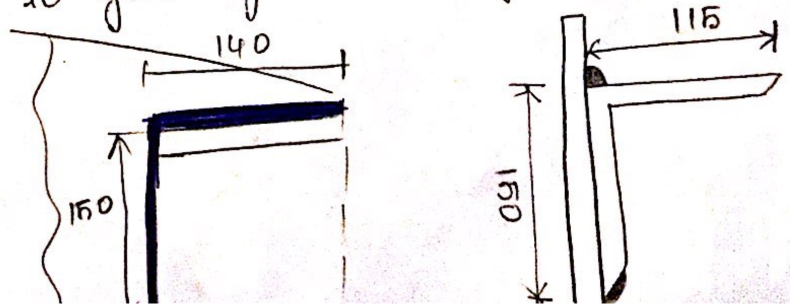
$$= \frac{146.5 + \left(\frac{40}{100} \times 146.5\right)}{45.26}$$

$$= 4.55 \simeq 5$$

$$= \frac{171 + \left(\frac{40}{100} \times 171\right)}{45.27} = 5.2 = 5$$



2. Compute the tensile strength of an angle section 150×115×8. Using weld connected with the gusset plate as shown in figure. Find (i) strength due to yielding. (ii) Strength due to rupture.



(i) Tensile strength due to yielding: IS 800.

For ISA 150×115×8 mm $\quad A_g = 2058\ mm^2$

$$T_{dg} = \frac{A_g f_y}{\gamma_{m0}} = \frac{2058 \times 250 \times 10^{-3}}{1.1}$$

$$= 467.7\ KN$$

(ii) Tensile strength due to rupture:

$$T_{dn} = \frac{0.9 A_{nc} f_u}{\gamma_{m1}} + \frac{\beta A_g f_y}{\gamma_{m0}} \rightarrow \text{IS 800} \quad Pg:- 33, 6.3.3$$

$$\beta = 1.4 - 0.076 \left(\frac{w}{t}\right)\left(\frac{f_y}{f_u}\right)\left(\frac{b_s}{L}\right) \le \frac{f_u \times \gamma_{m0}}{f_y \times \gamma_{m1}} \geq 0.$$

$b_s = w = 115$

$L_s = 140$

Fig 6 → Pg: 33

$$\beta = 1.4 - 0.076 \left(\frac{115}{8}\right)\left(\frac{250}{416}\right)\left(\frac{115}{140}\right) = 0.85$$

$$\frac{f_u \gamma_{m0}}{f_y \gamma_{m1}} = \frac{410}{250} \times \frac{1.1}{1.25} = 1.44$$

$$\beta = 0.85$$

$$A_{nc} = \left(150 - 8/2\right) \times 8 = 1168\ mm^2$$

$$A_{go} = \left(115 - 8/2\right) \times 8 = 888\ mm^2$$

$$T_{dn} = \frac{0.9 \times 1168 \times 410 \times 10^{-3}}{1.25} + \frac{0.85 \times 888 \times 250 \times 10^{-3}}{1.1}$$

$$= 516.34\ KN$$