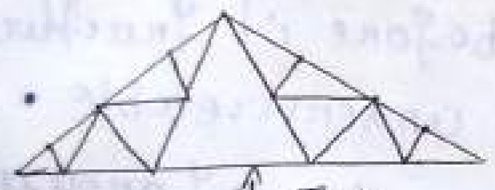
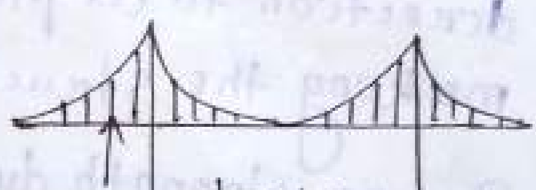
Tension Member

Tension members are linear members predominatly subjected to pulling (direct axial tensile load) which tend to stretch or elongate the member.

Ex Rope, tie of roof truss, suspenders of suspension bridge

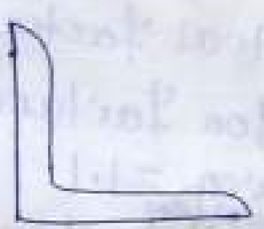


(Roof truss)

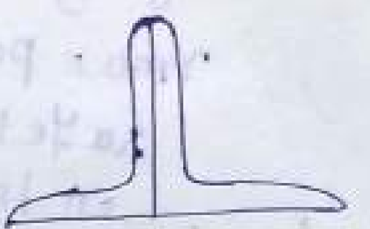


(Suspension bridge).

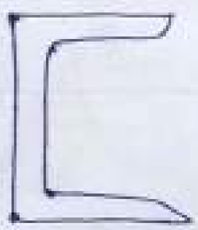
Common Shapes of Tension Member:-



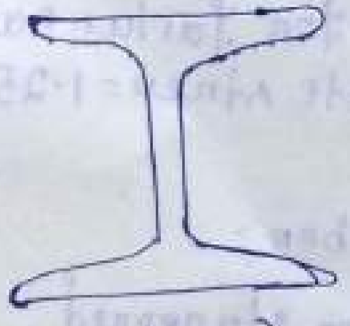
(angle section)



(double angle section)



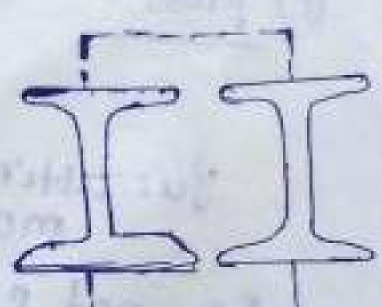
(channel section)



(I section)

(circular section)

(Steel flat)



(builtup section).

Types of failures of tension Members:-

(a) yielding of gross section

(b) Rupture of net critical section.

(c) block shear failure.

(a) Yielding of gross section:- (Cl: 6.2, page No 32)

Yielding of gross section occurs when considerable deformation of member in longitudinal direction takes place before it fractures, making the structure unserviceable.

Design strength due to yielding of gross section

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

f_y = yield stress of material

A_g = gross area of section.

γ_{m0} = partial factor of safety for failure in tension by yielding = 1.1

b) Design Strength due to rupture of critical section (cl: 6.3)

(i) plates

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

where γ_{m1} = partial safety factor for failure at ultimate stress = 1.25

f_u = ultimate stress of material

A_n = net effective area of member.

$$= \left(b - n d_0 + \sum_{i=1}^{2} \frac{p_{si}^2}{4 g_i}\right) t \quad \text{(for staggered bolted connection)}$$

$$= (b - n d_0) t \quad \text{(for chain bolted connection)}.$$

b = width of plate

d_0 = diameter of bolt hole.

g = gauge distance between bolt holes.

P_s = staggered pitch distance between the line of bolt holes.

t = thickness of plate.

n = number of bolt holes in the critical section.

(b) Threaded rods:-

$$T_{dn} = \frac{0.9 A_n f_u}{\gamma_{m1}} \quad (\text{cl: } 6.3.2)$$

A_n = net root area at the threaded section

$= 0.78 \times \frac{\pi}{4} \times d^2$.

(c) Single angle :- (cl: 6.3.3).

$$\text{Rupture strength } (T_{dn}) = \frac{0.9 A_{nc} f_u}{\gamma_{m1}} + \frac{\beta A_{go} f_y}{\gamma_{m0}}$$

A_{nc} = Net area of connected leg.

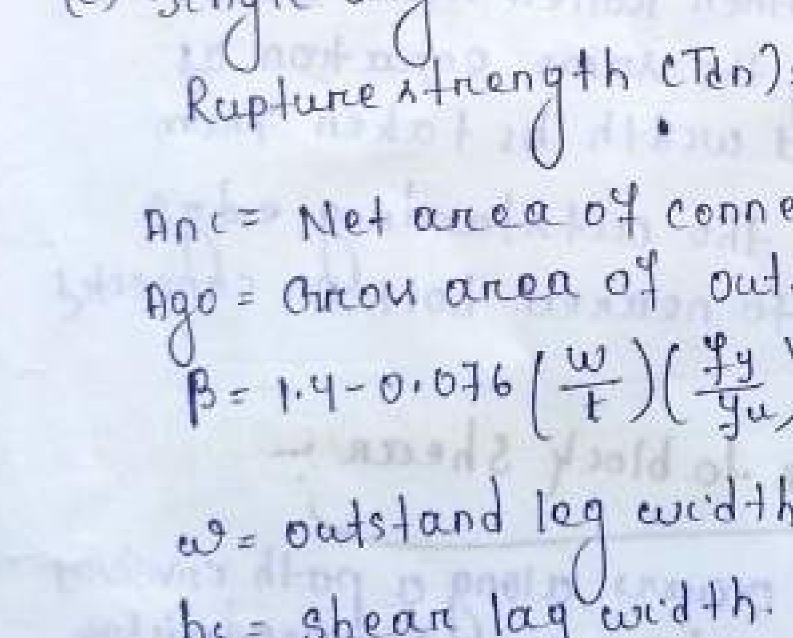
A_{go} = Gross area of outstanding leg.

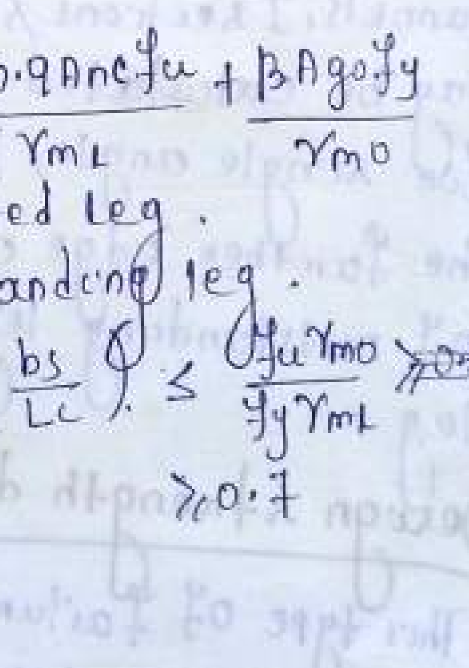
$$\beta = 1.4 - 0.076\left(\frac{w}{t}\right)\left(\frac{f_y}{f_u}\right)\left(\frac{b_s}{L_c}\right) \leq \frac{f_u \gamma_{m0}}{f_y \gamma_{m1}} \geq \cancel{0.7}$$

≥ 0.7

w = outstand leg width

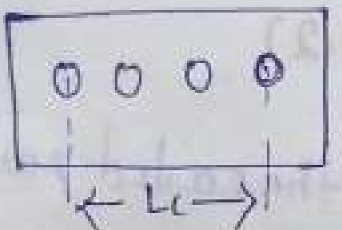
b_s = shear lag width.





t = thickness of the leg.

L_c = length of end connection, that is the distance between the outermost bolts in the end joint measured along the load direction.
or when the segments of welds in the end connections are of different length in the direction of load, the length of longest segment will be taken as L_c.



* The rupture strength T_{dn} of double angles, channels, I sections & other rolled steel sections may be calculated by the same equation as for single angle, but width b_s taken from the farther edge of the outstanding edge of outstanding leg to nearest bolt in connected leg.

Design strength due to block shear :-

This type of failure occurs along a path involving tension on one plane & shear on a perpendicular plane.

-> When applied tensile load is increased, the fracture strength of the weaker plane is approached. However, this plane does not fail as it is restrained by stronger plane & the load can still be increased until the fracture strength of stronger plane is reached. By this time the weaker plane would have yielded. Thus the total strength equals fracture strength of stronger plane plus yield strength of weaker plane (cl: 6.4).

(a) for Bolted connection:-

The block shear strength T_{db} at bolted connection may be taken as smaller of the following:-

(i) yielding of shear section + fracture of tensile section

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

(ii) yielding of tensile section + fracture of shear section

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

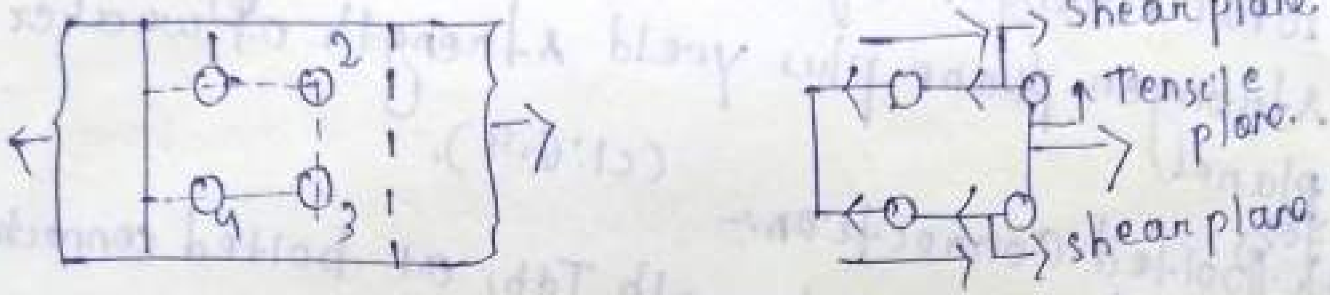
A_{tg} = ~~shear~~ gross area in ~~shear~~ tensile section

A_{vg} = gross area in shear section

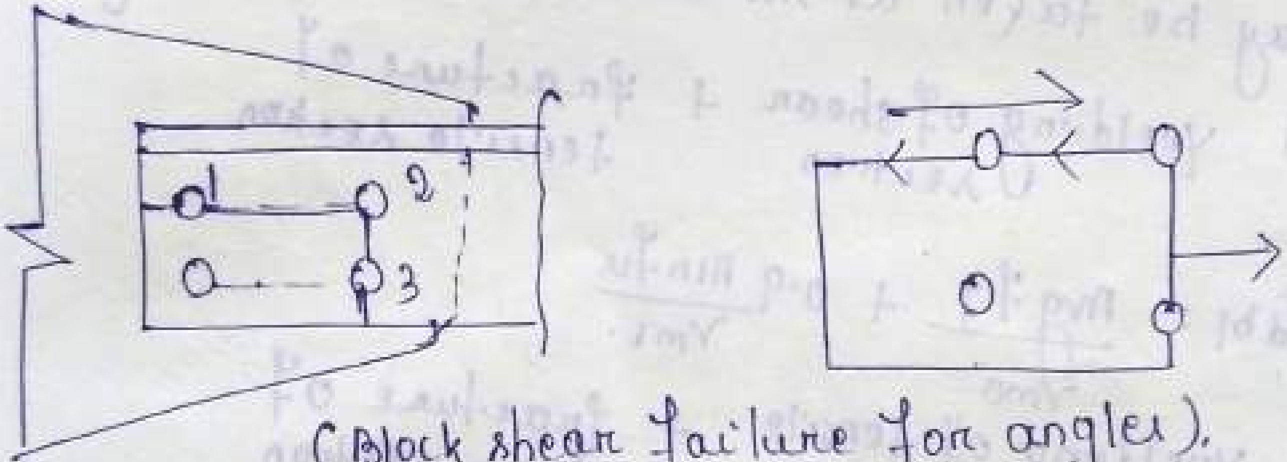
A_{tn} = net area in tensile section

A_{vn} = net area in shear section.

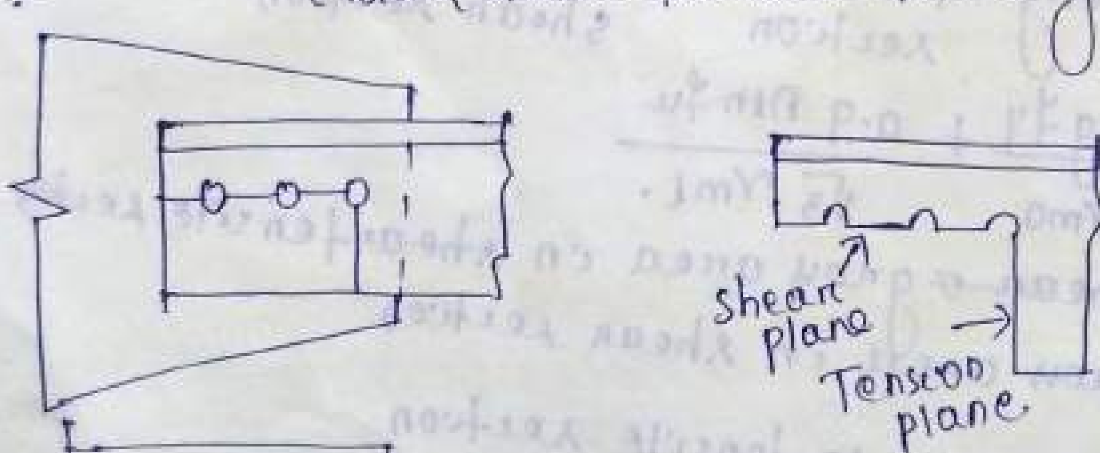
(b) Welded connection:- The block shear strength, T_{db} for welded connection shall be calculated by taking an appropriate section in member around the end weld. However for welded angle tension members, hence it is the gross area only that is involved. A_{tn} & A_{vn} is to be replaced by A_{tg} & A_{vg} respectively in above equation.

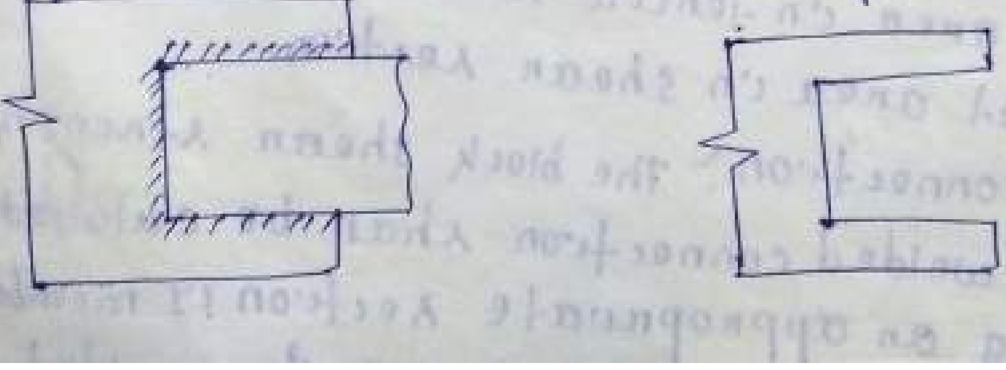


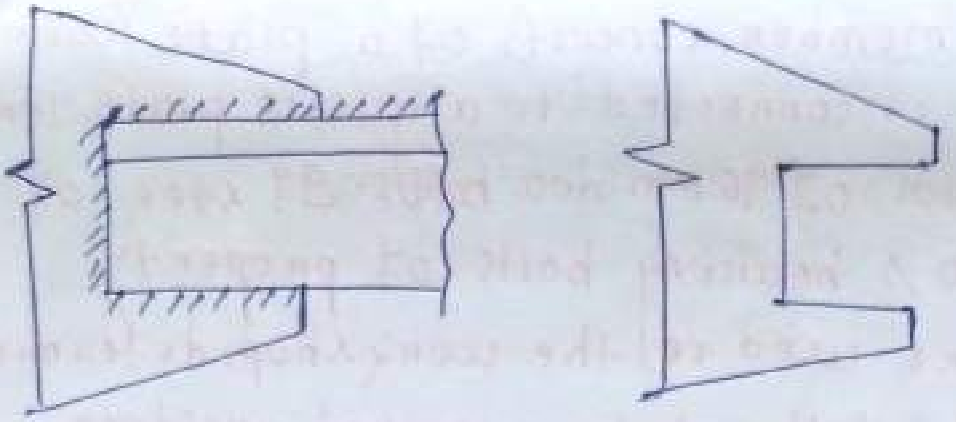
(Block shear failure for plates).



(Block shear failure for angles).







Slenderness Ratio (λ).

The slenderness ratio of a tension member is the ratio of effective length (KL) to its least radius of gyration r.

where K = coefficient depending on the end condition. (table No 11)

-> Theoretically, there should be no limitation on the slenderness ratio of tension member since stability or buckling is of little concern. But they may be subjected to stress reversals during erection, wind or earthquake load etc. From these point of view, the design specifications usually limit the slenderness ratio of tension member.

-> Value of λ (table No-3 IS 800).

Q A tension member consists of a plate 100mm x8mm which is connected to a gusset plate 10mm thick by 2nos of 16 mm dia bolts. If steel of grade Fe410 & bearing bolts of property class 4.6 are used in the workshop, determine the strength of the plate against yielding, rupture & block shear.

Solution:- For Fe410 grade of steel:-

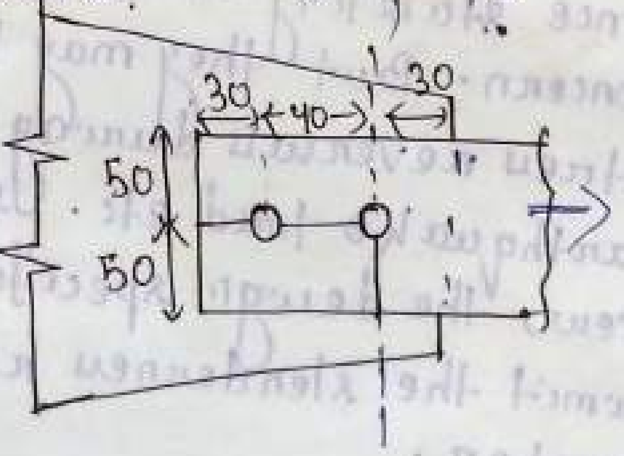
$f_u = 410$ Mpa, $f_y = 250$ Mpa, $\gamma_{m0} = 1.1$, $\gamma_{m1} = 1.25$

For 16mm dia shop bolts of property class 4.6:-

$d = 16$mm, $d_0 = 16+2 = 18$ mm, $\gamma_{mb} = 1.25$

Strength of the plate:-

(i) Strength of plate against yielding of gross cross-section:-



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

$A_g = 100 \times 8 = 800 mm^2$

$$T_{dg} = \frac{800 \times 250}{1.1}$$

$= 181.82$ kN.

(ii) Strength of plate against rupture of critical section

In this case section 1-1 is critical.

Net area $A_n = (b - n d_0) t$

$= (100 - 1 \times 18) \times 8 = 656 mm^2$

$$T_{dn} = \frac{0.9 A_n f_u}{\gamma_{m1}} = \frac{0.9 \times 410 \times 656}{1.25} = 193.65 \text{ kN}.$$

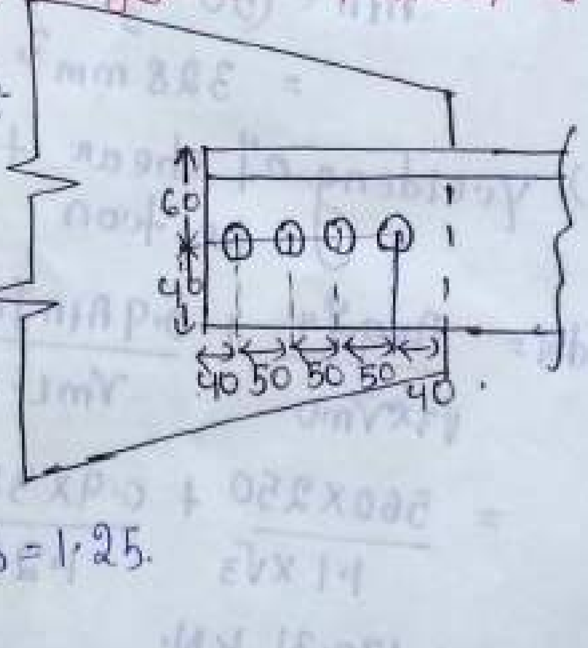
Block shear strength = minimum of T_{db1} & T_{db2}

$T_{db} = 149.54\,kN$.

Tensile strength of plate = minimum of $\begin{cases} T_{dg} = 181.82 \\ T_{dn} = 193.65 \\ T_{db} = 149.54 \end{cases}$

$= 149.54\,kN$.

Q A single unequal angle ISA 100×75×8 mm is connected by longer leg to a 12 mm gusset plate at the ends with 4 nos of 20 mm dia field bearing bolts of property class 4.6 (in a line) to transfer tension as shown in figure. Determine the design tensile strength of the angle. Take fe 410 grade of steel.



For fe 410 grade of steel:-

$f_u = 410\,Mpa$

$f_y = 250\,Mpa$.

For 20 mm dia. field bolts of property class 4.6:-

$d = 20\,mm$, $d_0 = 22\,mm$, $\gamma_{mb} = 1.25$.

~~$A_{nb} = 0.78 \times \frac{\pi}{4}$~~

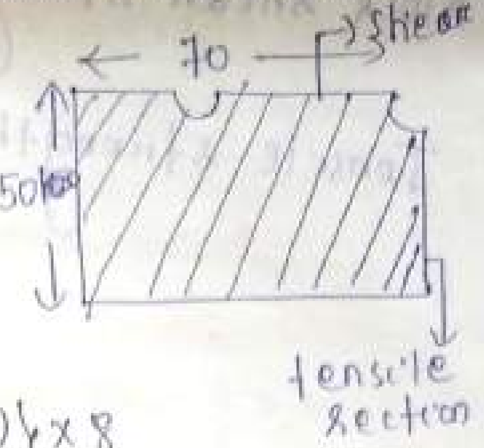
Strength of angle section:-

(i) Strength of angle against yielding of gross section.

For ISA 100×75×8,

$A_g = 1336\,mm^2$ (from steel table).

(ii) Strength of the plate against block shear-



gross area at shear section:-

$$A_{vg} = 70 \times 8 = 560\,mm^2$$

net area at shear section:-

$$\cancel{A_{tn}}\ A_{vn} = \{70 - (1.5 \times 18)\} \times 8 = 344\,mm^2$$

gross area at tensile section:-

$$A_{tg} = \cancel{100 \times 8}\ 50 \times 8 = 400\,mm^2$$

net area at tensile section:-

$$A_{tn} = \left(50 - \frac{1}{2} \times 18\right) \times 8 = 328\,mm^2$$

(i) Yielding of shear section + Rupture of tensile section

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

$$= \frac{560 \times 250}{1.1 \times \sqrt{3}} + \frac{0.9 \times 328 \times 410}{1.25}$$

$$= 170.31\ KN.$$

(ii) Rupture of shear section + yielding of tensile section

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

$$= \frac{0.9 \times \overset{344}{\cancel{328}} \times 410}{1.25 \times \sqrt{3}} + \frac{400 \times 250}{1.1} = 149.54\ KN.$$

$T_{dg} = \frac{A_g f_y}{\gamma_{m0}} = \frac{1336 \times 250}{1.1} = 303.64 \text{ kN}$

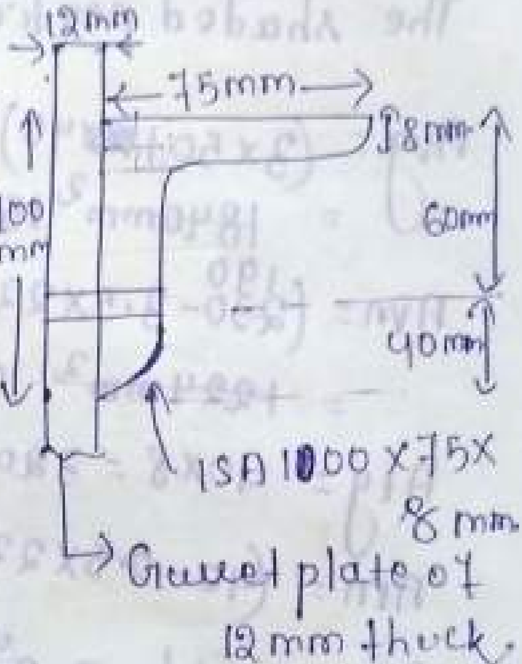
(ii) Strength of the angle against rupture of critical section:

Net area of connected leg,

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

Gross area of outstanding leg,

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



w = outstand leg width = 75 mm

$w_1 = w - g$.
$= 100 - 40 = 60$ mm.

Shear lag width (bs) =
$w + w_1 - t$
$= 75 + 60 - 8 = 127$ mm.

Length of connection (L_c) = 3×50
$= 150$ mm.

$$\beta = 1.4 - 0.076\left(\frac{w}{t}\right)\left(\frac{f_y}{f_u}\right)\left(\frac{b_s}{L_c}\right) \le \frac{f_u \gamma_{m0}}{f_y \gamma_{m1}} \ge 0.7$$

$$= 1.4 - 0.076\left(\frac{75}{8}\right)\left(\frac{250}{410}\right)\left(\frac{127}{150}\right) \le \frac{410 \times 1.1}{250 \times 1.25}$$

$= 1.032 \le 1.4432$
≥ 0.7 (ok).

hence $\beta = 1.032$.

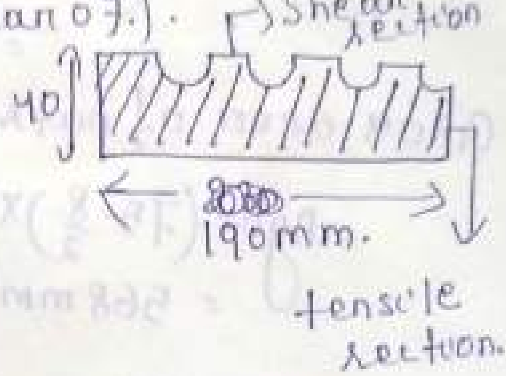
$$T_{dn} = \frac{0.9 A_{nc} f_u}{\gamma_{m1}} + \frac{\beta A_{go} f_y}{\gamma_{m0}} \quad (\text{cl } 6.3.3)$$

$$= \frac{0.9 \times 542 \times 410}{1.25} + \frac{1.032 \times 568 \times 250}{1.1}$$

$$= 307.980 \text{ kN}$$

(iii) Strength of angle section against block shear:

The shaded portion may shear off.



$$A_{vg} = (3 \times 50 + \cancel{}40) 8$$

$$= 1840 \text{mm}^2 \; 1520 \text{mm}^2$$

$$A_{vn} = (\overset{190}{230} - 3.5 \times 22) \times 8$$

$$= \cancel{1224 \text{mm}^2}, \; 904 \text{mm}^2$$

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

$$A_{tn} = (40 - 0.5 \times 22) \times 8 = 232 \text{mm}^2.$$

(i) yielding of shear section + rupture of tensile section

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

$$= \frac{1520 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 232 \times 410}{1.25}$$

$$= 267.934 \text{ kN}$$

T_{db2} = (yielding of tensile section) + (rupture of shear section)

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

$$= \frac{320 \times 250}{1.1} + \frac{0.9 \times 904 \times 410}{\sqrt{3} \times 1.25} = 226.799 \text{ kN}.$$

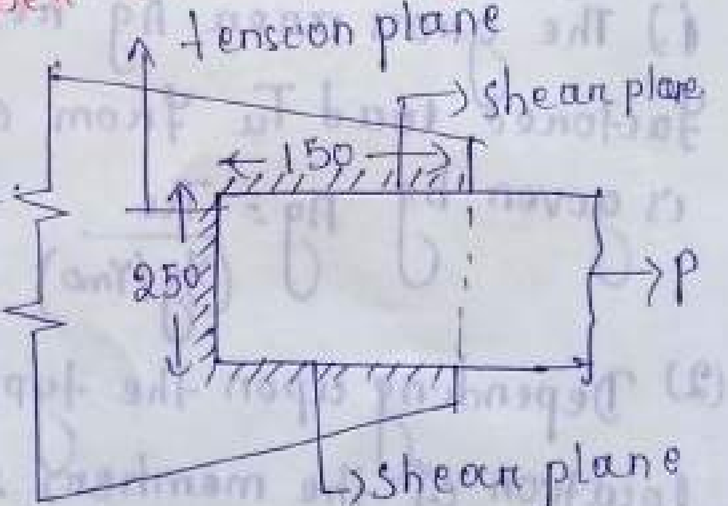
block shear strength = minimum of $\begin{cases} T_{db1} = 267.934\,kN \\ T_{db2} = 226.799\,kN \end{cases}$

$T_{db} = 226.799\,kN$.

Strength of angle = minimum of $\begin{cases} (i)\ T_{dg} = 303.64\,kN \\ (ii)\ T_{dn} = 307.98\,kN \\ (iii)\ T_{db} = 226.79\,kN \end{cases}$

$= 226.79\,kN$.

Q A steel plate 250mm × 6mm is connected to a 10mm thick gusset plate as a tension member by fillet welding. Determine the block shear strength of member. (Take fe410 grade steel)



For fe410 grade steel

$f_u = 410\,Mpa$, $f_y = 250\,Mpa$

$\gamma_{m0} = 1.1$, $\gamma_{ml} = 1.25$

$A_{vg} = A_{vn} = 150 \times 6 \times 2$
$= 1800\,mm^2$

$A_{tg} = A_{tn} = 250 \times 6 = 1500\,mm^2$

block shear strength:

(i) $T_{db1} = \dfrac{A_{vg} f_y}{\sqrt{3}\,\gamma_{m0}} + \dfrac{0.9 \times A_{tn} \times f_u}{\gamma_{ml}}$

$= \dfrac{1800 \times 250}{\sqrt{3} \times 1.1} + \dfrac{0.9 \times 1500 \times 410}{1.25}$

$= 678.98\,kN$.

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

$$= \frac{1500 \times 250}{1.1} + \frac{0.9 \times 1800 \times 410}{1.25 \times \sqrt{3}}$$

$$= 647.689 \text{ kN}$$

block shear strength T_{db} = minimum of $\begin{cases} T_{db_1} = 675.95 \text{ kN} \\ T_{db_2} = 647.68 \text{ kN} \end{cases}$

$T_{db} = 647.689$ kN.

Design of tension members subjected to axial load :-

Design steps :-

(1) The gross area 'A_g' required to carry the factored load T_u from consideration of yielding is given by $A_g = \frac{T_u}{(f_y / \gamma_{m0})}$

(2) Depending upon the type of structures & the location of the members suitable shape & area of section is selected from steel table.

(3) The connection is designed by calculating the number of bolts or the length of weld required, which is suitably arranged as per requirement.

$$\text{no. of bolts} = \frac{\text{external load}}{\text{strength of a bolt}}$$

↳ minimum of bearing strength & shearing capacity of bolt.

(4) The design strength Td of trial section is calculated considering minimum of Tdg, Tdn, & Tdb which should be more than the factored load.

(5.) If ~~Td < P~~ Td < P, then the section is suitably revised.

(6.) The effective slenderness ratio of member is checked which should satisfy I.S. specification.

Q Design a single angle section for a tie of a roof truss to carry a factored tensile force of 300 kN. The member is subjected to possible reversal of stress due to action of wind. The effective length of member is 2.5m. Given that bearing bolts of property class 4.6 & steel of grade Fe410 are used.

For steel grade Fe410:-

$f_u = 410 Mpa$, $f_y = 250 Mpa$.

$\gamma_{m0} = 1.1$, $\gamma_{m1} = 1.25$

(i) Calculation of sectional area required:-

$$A_g = \frac{T_u \gamma_{m0}}{f_y} = \frac{300 \times 10^3 \times 1.1}{250} = 1320 mm^2$$

Let us adopt an angle section whose area > 1320 mm² from steel table.

So choose an angle section 90x60x10mm having gross area 1401 mm² connected by long ~~leg~~ leg.

(ii) Design of connection:-

from unwin's formula, $d = 6\sqrt{t}$

$= 6\sqrt{10} = 18.97$

$\simeq 20mm.$

Let us adopt 20mm bearing type bolts

$d_o = 20+2 = 22mm$, $\gamma_{mb} = 1.25$.

$A_{nb} = 0.78 \times \frac{\pi}{4} \times (20)^2 = 245.04 mm^2$

$A_{sb} = \frac{\pi}{4} \times (20)^2 = 314.16 mm^2$

$f_{ub} = 400 Mpa$.

for lap joint $n_n = 1$, $n_s = 0$.

(a) Design strength of bolt in single shear:

(Cl: 6.2).

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

$$= \frac{400}{\sqrt{3} \times 1.25} (1 \times 245.04)$$

$$= 45.27 kN$$

(b) assume $e = 1.5 \times 22 = 33mm$, $p = 2.5 \times 20 = 50mm$.

$e \simeq 40mm$.

$$k_b = \text{minimum of} \begin{cases} \frac{e}{3d_o} = \frac{40}{3 \times 22} = 0.606 \\ \frac{p}{3d_o} - 0.25 = \frac{50}{3 \times 22} - 0.25 = 0.51 \\ \frac{f_{ub}}{f_u} = \frac{400}{410} = 0.975 \\ 1 \end{cases}$$

$k_b = 0.51$

Design bearing strength of bolt (V_{dpb}):-

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

$$= \frac{2.5 \times 0.51 \times 20 \times 10 \times 410}{1.25}$$

$$= 83.640 \text{ kN}.$$

Strength of one bolt = minimum of
(i) $V_{dsb} = 45.27$ kN.
(ii) $V_{dpb} = 83.64$ kN

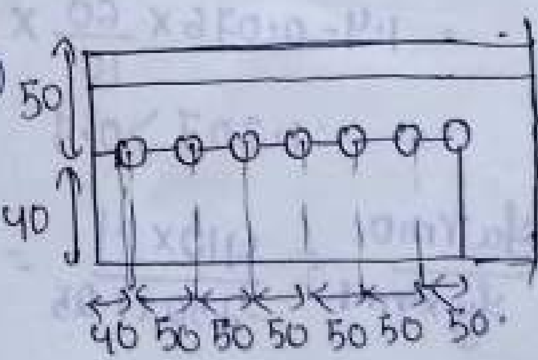
= 45.27 kN.

$$\text{number of bolts required} = \frac{T_u}{\text{strength of one bolt}}$$

$$= \frac{300}{45.27} = 6.63 \approx 7 \text{ no.}$$

Hence let us provide 7 nos of 20mm dia bolts in a single row.

<u>Check for long joint</u> (cl:10.3.3.1)



$l_j = 6 \times 50 = 300\text{mm} = 15 \times d$

$300\text{mm} = 15 \times 20 = 300\text{ mm}$

$(l_j = 15 \times d)$ (ok)

(3) <u>check for strength of plate</u>:-

(i) Strength against yielding of gross section:

$$T_{dg} = \frac{A_g f_y}{\gamma_{m0}} = \frac{1401 \times 250}{1.1}$$

$A_g = 1401$ from steel table

$f_y = 250$ Mpa.

$= 318.409 \text{ kN} > 300\text{kN}$ (ok).

(ii) Strength against rupture of section:- (Cl: 6.3.3).

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

Net area of connected leg (A_{nc}) =

$$= \left(90 - 22 - \frac{10}{2}\right) \times 10$$

$$= 630\,mm^2$$

Area of outstand leg (A_{go}) =

$$= \left(60 - \frac{10}{2}\right) \times 10$$

$$= 550\,mm^2$$

w = outstand leg width

$= 60mm$.

shear lag width (b_s) = $w + w_1 - t$

$= 60 + 50 - 10 = 100mm$.

length of connection (L_c) = $6 \times 50 = 300mm$.

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

$$= 1.4 - 0.076 \times \frac{60}{10} \times \frac{250}{410} \times \frac{100}{300}$$

$$= 1.307 > 0.7$$

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

$\beta = 1.307 > 0.7$, < 1.44 (ok).

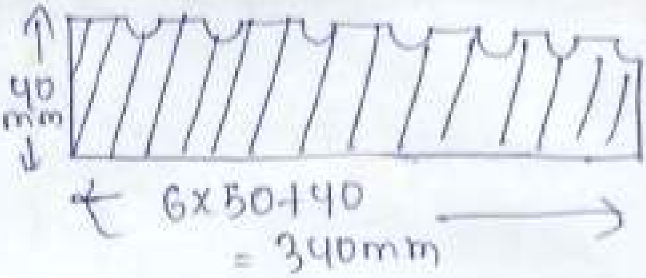
$$T_{dn} = \frac{0.9 A_{nc} f_u}{\gamma_{m1}} + \frac{\beta A_{go} f_y}{\gamma_{m0}}$$

$$= \frac{0.9 \times 630 \times 410}{1.25} + 1.307 \times \frac{550 \times 250}{1.1}$$

$= 349.35kN > 300kN$ (ok).

(iii) Strength of plate in block shear:-

(11.6.4)



$A_{vg} = 340 \times 10$
$= 3400 mm^2$

$A_{vn} = (340 - 6.5 \times 22) \times 10$
$= 1970 mm^2$

$A_{tg} = 40 \times 10 = 400 mm^2$

$A_{tn} = (40 - 0.5 \times 22) \times 10 = 290 mm^2$

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

$$= \frac{3400 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 290 \times 410}{1.25} = 531.742 kN$$

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

$$= \frac{400 \times 250}{1.1} + \frac{0.9 \times 1970 \times 410}{\sqrt{3} \times 1.25} = 426.67 kN$$

block shear strength (T_{db}) =

minimum of $\begin{cases} T_{db1} = 531.74 \\ T_{db2} = 426.67 \end{cases}$

$= 426.67 kN > 300 kN$ (ok)

(4) check for slenderness ratio:

for a member subjected to possible reversal of stress due to action of wind $\lambda = 350$ (from table 3)

minimum radius gyration $r = 12.7 mm$ (from steel table)

maximum slenderness ratio $\lambda = \frac{KL}{r} = \frac{2500}{12.7} = 196.8 < 350$ (ok)