Vikash
Polytechnic

LECTURE NOTE
ON
STRUCTURAL DSIGN -II

5th SEMESTER

DEPARTMENT
OF
CIVIL ENGINEERING

PREPARED BY
MRS. PRATIBHA PRADHAN
SENIOR LECTURE IN CIVIL ENGINEERING
VIKASH POLYTECHNIC, BARGARH

Structural Steel Fasteners & Connections: Bolted Connection.

→ Different elements or members of steel structures are required to be joined to one another either at their ends or at some intermediate length in order to facilitate the transmission of member forces which is known as connection.

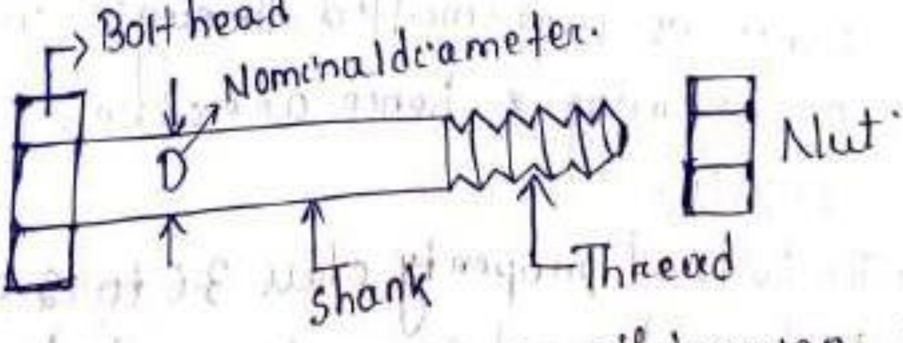
→ The devices required for preparing a connection are called connectors or fasteners. There are mainly four types of fasteners commonly used.

(a) Bolts (b) welds (c) Rivets (d) pins.

→ As a connection or a joint is the weakest part of a structure, which are to be properly designed.

Bolted Connections:-

A bolt is a metal pin with a head at one end & a shank threaded at other end to receive a nut.



Bolts are used for the purpose of joining together pieces of metals having holes through which these are inserted & the nuts are tightened at the threaded ends.

Types of bolts.

(i) unfinished bolts or black bolts.

(ii) finished bolts or turned bolt.

(iii) High strength friction grip bolt (HSFG).

(i) unfinished or black bolts:

- → They are also known as ordinary or common bolts.
- → These bolts are made from low carbon mild steel round rods with square or hexagonal head & the shank is left unfinished or rough.
- → These are used for light structures subjected to static loads as well as for secondary members such as purlins, roof trusses etc, but not recommended for structures subjected to vibration & fatigue.
- → As the bolt is unfinished, it may not establish perfect contact with structural member resulting in loose joints.
- → In the joints made with such bolts, the force is transmitted through interlocking or bearing & hence are called bearing type joints.

* The bolts of property class 3.6 to 12.9 are available, out of which most commonly used black bolt is property class 4.6.

leaning of property clas 4.6. :-

$\frac{1}{100}$ × nomonal ultimate strength = 4
of bolt (f_u).

$$\frac{\text{yield stress of bolt } (f_{yb})}{\text{ultimate strength } (f_{ub}) \text{ of bolt}} = 0.6.$$

(ii) finished & turned bolts :

-> These bolts are formed from mild steel hexagonal rods & are made by turning to circular shape.

-> As the connection is more tight, in this case it ensures better bearing contact between the bolts & holes. These are used where accurate alignment of components are necessary such as machine parts, structures subjected to dynamic loading etc.

(iii) High strength friction grip bolts :

-> These bolts are made from high strength steel rods like black bolts, but the surface of the shank of these bolts is kept unfinished & these bolts are tightened until very high tensile stresses are developed using calibrated wrenches so that the connected parts are clamped tightly together between bolts & nut heads.

-> This permits the loads to be transfered primarily by friction & not by shear.

→ This results into no slippage in the joints & the joints made with such bolts are known as slip-critical joint.

→ These are suitable for members subjected to dynamic load also.

Bolt holes:- table 19 (page 73):-

Nominal size of fastener (d)	bolt hole (mm)
12-14	+1
16-22	+2
24	+2
>24	+3.

Advantages of bolted connection:-

(i) use of simple tools & less skilled labour & working area.

(ii) speedy & noiseless erection.

(iii) Economical due to reduced labour & equipment costs.

(iv) Minimum strength reduction at joint due to less number of holes on bolts.

(v) Easy alternation or dismantling of connection.

Disadvantages of bolted connection:-

(i) high cost of material.

(ii) Reduced tensile strength due to area reduction.

(iii) Susceptibility to loosening of bolts under vibration & dynamic loads.

Classification of bolts based on load transfer Mechanism

-> divided into two groups.

(i) bearing type or slip type connection.

(ii) friction grip type or slip critical connection.

(i) **bearing type of bolt** :-

-> load transfer takes place by shearing / bearing of member.

ex finished bolt, unfinished bolt.

(ii) **friction grip type bolt** :-

-> load transfer takes place by friction between the members.

ex High strength friction grip (HSFG) bolt.

Advantages of HSFG bolts over bearing type bolt :-

(i) Rigidity of joint due to no slip condition.

(ii) No shearing or bearing stresses in members as the load transfer mechanism is mainly by friction.

(iii) Large clamping forces provide high static strength of joints.

Disadvantages of HSFG bolts over bearing type bolts.

(i) Material cost of HSFG bolts is greater than that of ordinary bolts.

(ii) special workmanship is required in installing & tightening of these bolts.

-> Additional labour cost is required for surface preparation of members to be joined.

Types of bolted connections / Joints :-

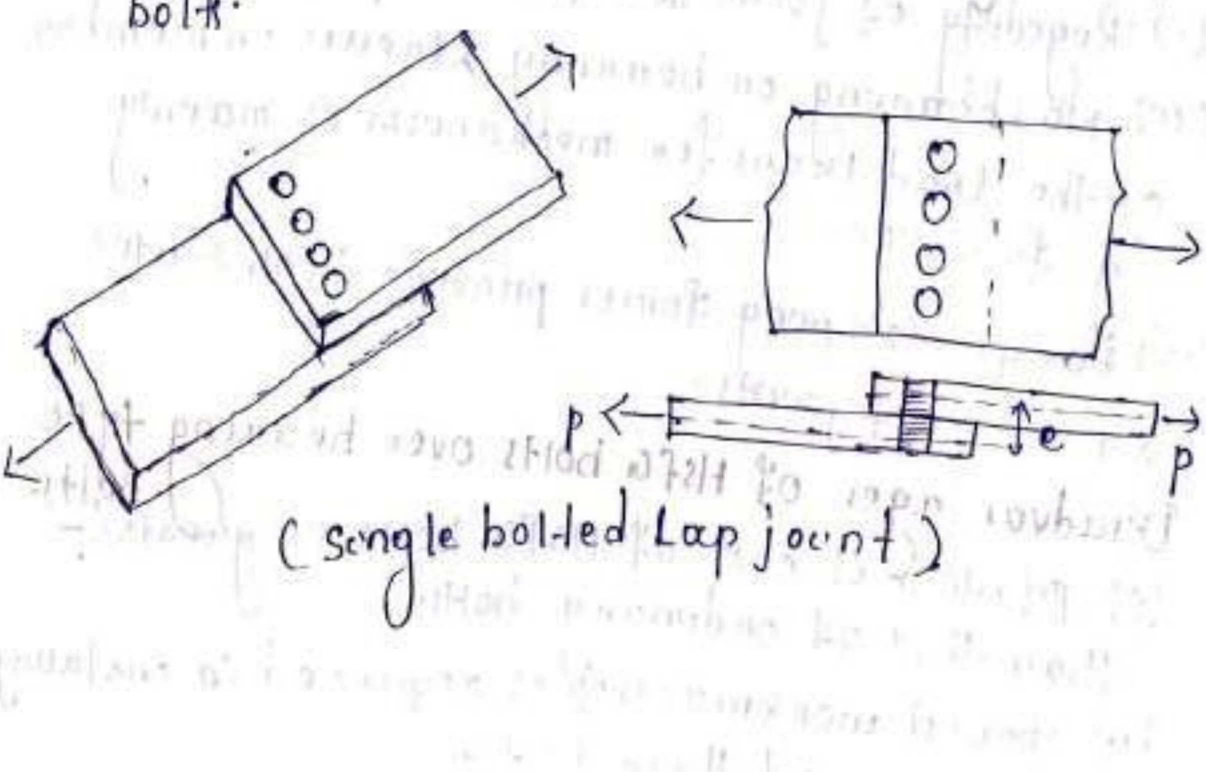
(a) Lap joint (b) butt joint.

(a) Lap joint

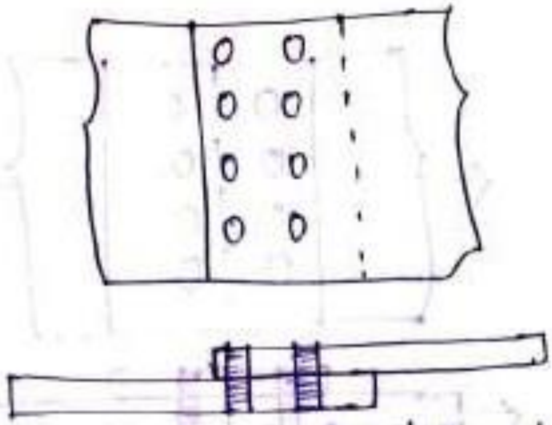
-> The two members to be connected over lap one another.

-> This constitutes the simplest type of joint requiring no extra cover plates.

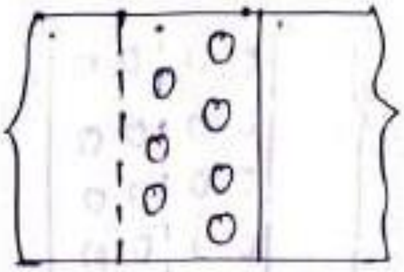
-> Since the centre of gravity of members joined in a lap joint are not collinear, the load in the lap joint has eccentricity, which may cause undesirable bending action. In order to minimize the effect of bending in lap joint, we have to increase no. of bolts.



(single bolted Lap joint)



(double bolted lap joint)
(chain bolting)

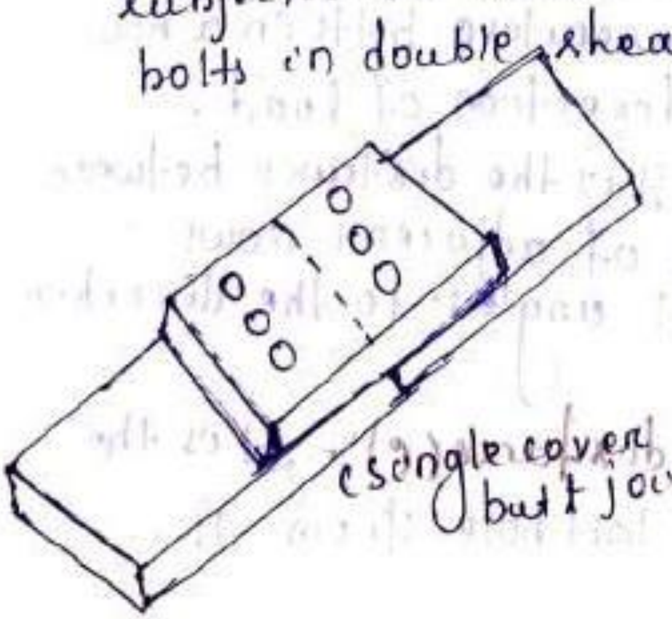


(zigzag bolting)

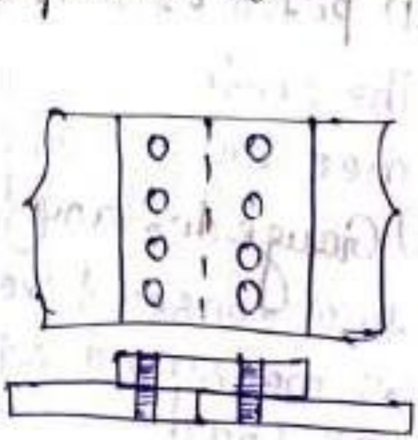
(b) Butt joint:-

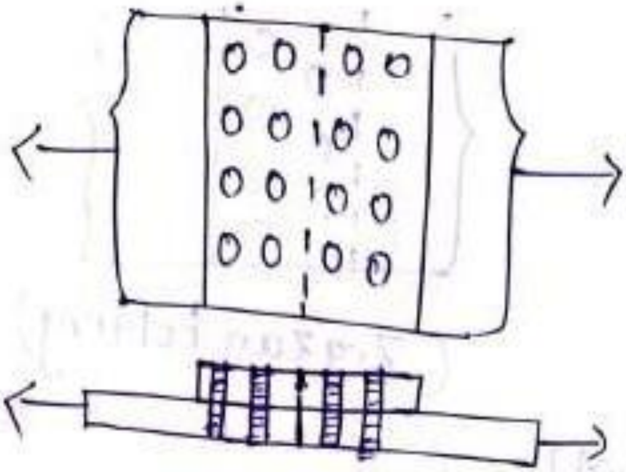
-> In this type of joint, the two members to be connected are placed end to end i.e. butt against each other & the connection is made by providing additional plates either on one side or on both sides. These additional plates are called cover plates where the members to be joined are called main plate.

-> bending is eliminated as there is no eccentricity of forces & the bolts are subjected to shear in two planes known as bolts in double shear.

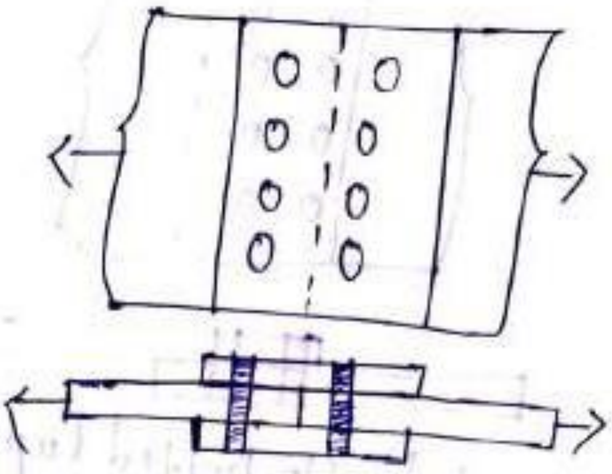


(single cover butt joint)

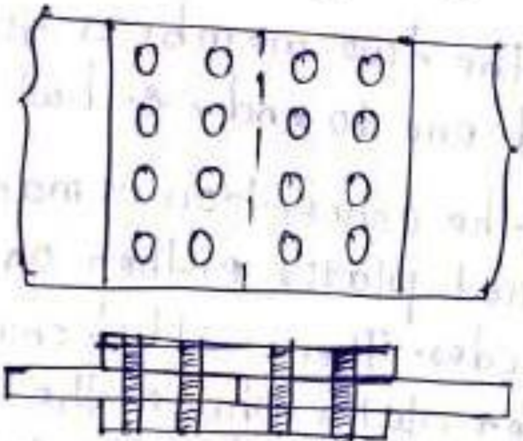




(single cover double bolted butt joint).



(double cover single bolted butt joint)



(double cover double bolted butt joint)

Terms used in bolted Connections:-

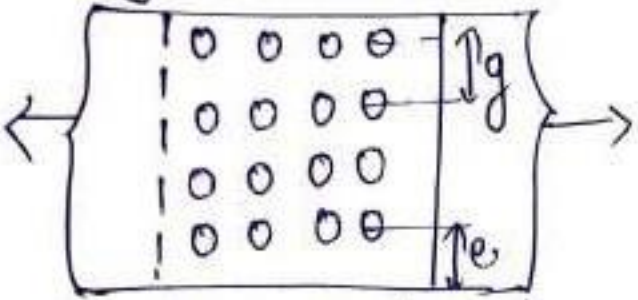
(1) pitch of bolt (p):- It is the distance between the centres of two consecutive bolts in a row measured along the direction of load.

(2) Gauge distance (g):- It is the distance between two consecutive bolts of adjacent rows & is measured at right angles to the direction of load.

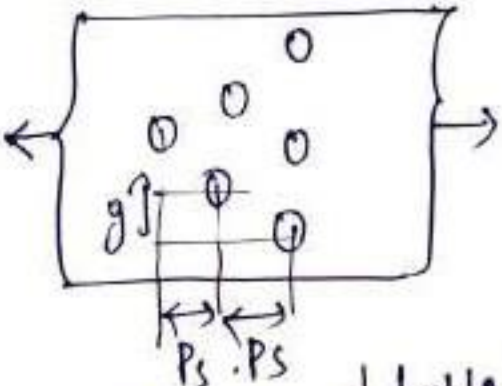
(3) Edge distance/end distance (e):- It is the distance of centre of bolt hole from the

adjacent edge or end of plate.

4. Staggered pitch :- (Ps) It is the centre to centre distance of staggered bolts measured along the direction of load.



(chain bolting).



(staggered bolted connection)

ssumptions in the analysis of bearing bolts:

following assumptions are made in the design of bearing type of bolted connection:-

1) The stress distribution on the plates between the bolt holes is uniform.

2) The friction between the plates is negligible.

3) The shearing stress is uniformly distributed over the cross section of bolt.

(4) The bolts in a group share the direct load equality.

(5) Bending stresses developed in bolts is neglected.

Coadal provisions for bolted joints:-

(i) Minimum pitch:- 2.5d, where d = nominal diameter of bolt. (cl:10.2.2)

(ii) Maximum pitch:(cl:10.2.3)

(a) 16t or 200mm whichever is less (for tensile member)

(b) 12t or 200mm, whichever is less, (for compression member)

t = thickness of thinner member.

(iii) Maximum gauge distance:- 100+4t or 200 mm } whichever is less. (cl:10.2.3.3)

(iv) Minimum edge distance:- (cl:10.2.4).

(a) 1.7do (for sheared or hand flame cut edges)

(b) 1.5do (rolled, machine flame cut, planed edge).

(v) Maximum edge distance:- (cl:10.2.4.3)

(a) $12t\varepsilon$ } ~~whichever is less~~.

or

(b) $40+4t$

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

Design strength of joint (bearing bolt)

Strength of bolt.

Strength of plate (Rupture strength)

Minimum will be the strength of joint.

bearing strength of bolt

Shearing strength of bolt.

Minimum will be the strength of bolt.

Design strength of bearing type of bolts in a joint:-

(A) shear strength of bolt
(B) Bearing strength of bolt. } Minimum will be taken

(A) Shear strength of bolt :- (cl: 10.3.3)

The design shear strength of the bolt (V_{dsb}) :

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

γ_{mb} = partial safety factor of material of bolt (table No-05)

V_{nsb} = nominal shear capacity of bolt:

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

f_{ub} = ultimate tensile strength of bolt

A_{sb} = nominal shank area of bolt = $\frac{\pi}{4} d^2$

A_{nb} = net area of bolt at threads = $0.78 \times \frac{\pi}{4} d^2$

n_n = number of shear planes with threads intercepting the shear planes.

($n_n = 1$ (for single shear)

$n_n = 2$ (for double shear).

n_s = number of shear planes without thread intercepting the shear plane.

($n_s = 0$ (for single shear)

($n_s = 1$ (for double shear))

Reduction factors for shear capacity of bolts: (cl: 10.3.3.1)

(i) Reduction factor for long joint:-

$\beta_{ij} = 1.075 - \frac{l_j}{200d}$ $(0.75 \leq \beta_{ij} \leq 1.0)$. (when $l_j > 15d$)

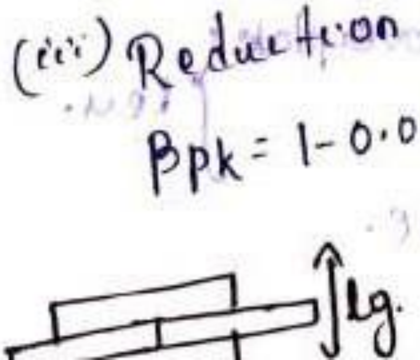
(ii) Reduction factor for large grip length:- (β_{lg})

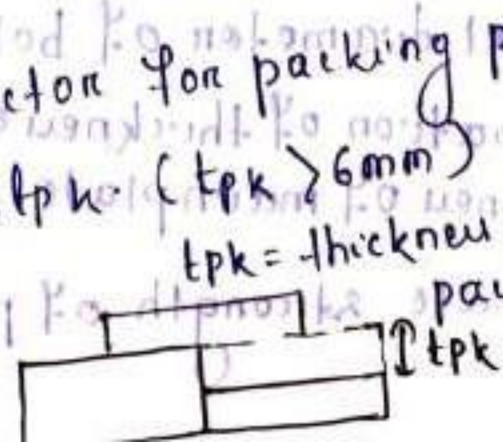
$\beta_{lg} = \frac{8d}{3d + l_g}$ ($l_g > 5d$) (cl: 10.3.3.2)

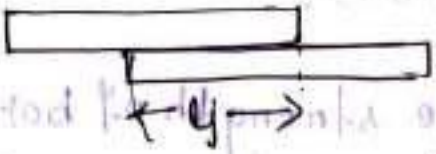
(iii) Reduction factor for packing plate (β_{pk}) (cl. 10.3.3.3)

$\beta_{pk} = 1 - 0.0125 t_{pk}$ ($t_{pk} > 6mm$)

t_{pk} = thickness of thicker packing in mm.







If $l_j > 15d$, $l_g > 5d$ & $t_{pk} > 6$:-
then the modified nomenal shearcapacity of bolt is:-

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

(B) **Bearing strength of bolt:-** (cl: 10.3.4).

The design strength of bolt, V_{dpb}:-

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

V_{npb} = nominal bearing strength of bolt
$= 2.5 k_b d t f_u$.

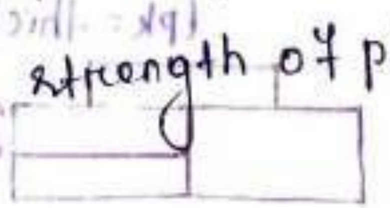
$$k_b = \left.\begin{array}{l} \frac{e}{3d_0} \\ \frac{p}{3d_0} - 0.25 \\ \frac{f_{ub}}{f_u} \\ 1 \end{array}\right\} \text{whichever is less.}$$

e = end distance
P = pitch distance
f_{ub} = ultimate strength of bolt
d_0 = diameter of bolt hole.

d = nominal diameter of bolt.

$t = \left.\begin{array}{l}\text{summation of thickness of coverplate} \\ \text{thickness of main plate}\end{array}\right\}$ less.

f_u = ultimate strength of plate.



Rupture strength of plate :- (Cl: 6.3)

The design tensile strength of a plate in the joint is the strength of thinner member against rupture which is given by:-

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

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

f_u = ultimate stress of material

A_n = net effective area of plate at critical section

$= \left(b - n d_0 + \sum \frac{p_{si}^2}{4 g_i}\right) t$ (for staggered bolted connection.

$= (b - n d_0) t$ (for chain bolted connection).

Efficiency of a joint:-

$$\eta (\text{efficiency}) = \frac{\text{Strength of joint}}{\text{Strength of Solid plate}} \times 100$$

Strength of solid plate:-

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

f_y = yield stress of material

A_g = gross area of cross section

γ_{m0} = partial safety factor for failure by yielding = 1.1 (from table 5)

Q-) find the maximum force that can be transmitted through a double bolted chain lap joint consisting of 6 bolts in 2 ~~rows~~ columns. Given that M16 bolts of grade 4.6 & plates of $f_e 410$ are used. Also find the efficiency of joint. (Given e = 30, p = 50).

Given data :- Double bolted lap joint

Thickness of plate $t_1 = 10$ mm, $t_2 = 12$ mm.

Total no. of bolts = 6

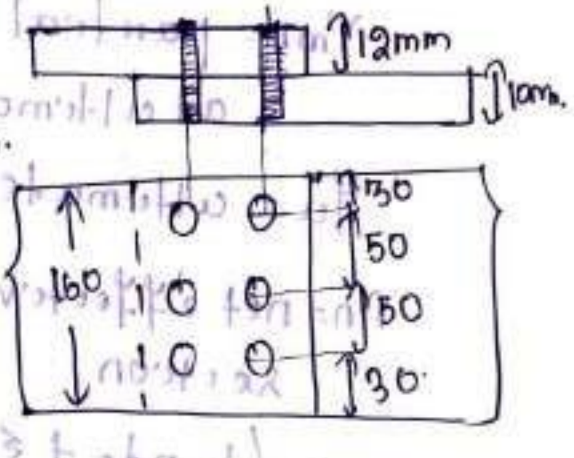
Dia of bolt d = 16 mm.

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

end distance (e) = 30 mm.

pitch (p) = 50 mm.

Grade of bolt 4.6



$f_{ub} = 400$ Mpa, $f_{yb} = 240$ MPa

$f_y = 250$ Mpa

ultimate strength of plate $f_u = 410$ Mpa.

Required: efficiency of joint $\eta = ??$

Solution :-

(A) Strength of bolt :-

(a) shear strength of a bolt :-

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

$f_{ub} = 400$, $A_{nb} = 0.78 \times \frac{\pi}{4} \times (16)^2$

$A_{sb} = \frac{\pi}{4} \times (16)^2$

for single shear (Lap joint) :-

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

$$V_{nsb} = \frac{400}{\sqrt{3}}\left(1\times 0.78\times \frac{\pi}{4}\times 16^2 + 0\times n_{sb}\right).$$

$$= 36.217 \text{ kN}$$

$$V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}} = \frac{36.216}{1.25} = 28.974 \text{ kN}.$$

Design shear strength of a bolt:- 28.974 kN

Design shear strength of 6 no. of bolt:-

$$28.974\times 6 = \boxed{173.846 \text{ kN.}}$$

(b) Bearing strength of bolt:-

The bearing strength of bolt against the thinner plate will be critical

Nominal strength/bolt (Vnpb):-

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

$d = 16$mm

$f_u = 410$

t = thickness of thinner plate = 10mm.

$$k_b = \begin{cases} \frac{e}{3d_0} = \frac{30}{3\times 18} = 0.56 \\ \frac{p}{3d_0} - 0.25 = \frac{50}{3\times 18} - 0.25 = 0.676 \\ \frac{f_{ub}}{f_u} = \frac{400}{410} = 0.9756 \\ 1 \end{cases} \text{ whichever is less}$$

$k_b = 0.56$.

$V_{npb} = 2.5 \times 0.56 \times 16 \times 10 \times 410$

$= 91.840$ kN per bolt.

Design bearing strength per bolt

$$V_{dpb} = \frac{V_{npb}}{\gamma_{mb}} = \frac{91.840}{1.25}$$

$= 73.47$ kN.

Design bearing strength of 6 no. of bolt,

$73.47 \times 6 = \boxed{440.832 \text{ kN.}}$

Design strength of joint bolt in joint:

minimum of $\begin{cases} V_{dsb} = 173.846 \\ V_{dpb} = 440.832 \end{cases}$

$= \boxed{173.846}$

(B) Rupture strength of plate (T_{dn}):-

$$T_{dn} = \frac{0.9 f_u A_n}{\gamma_{ml}} \qquad f_u = 410, \ \gamma_{ml} = 1.25$$

$A_n = (b - n d_0) t$

$= (160 - 3 \times 18) \times 10$

$= 1060 \text{ mm}^2$

$$T_{dn} = \frac{0.9 \times 410 \times 1060}{1.25} = \boxed{312.912 \text{ kN}}$$

Strength of joint:- minimum of $\begin{cases} \text{strength of bolt} = 173.846 \text{ kN} \\ \text{rupture strength of plate} = 312.912 \text{ kN} \end{cases}$

$= \boxed{173.846 \text{ kN.}}$

Strength of solid plate (Tdg):- $\frac{A_g f_y}{\gamma_{m0}}$

$f_y = 250 \text{ MPa}, \ \gamma_{m0} = 1.1$

$A_g = b \times t = 160 \times 10$
$= 1600 \text{ mm}^2$

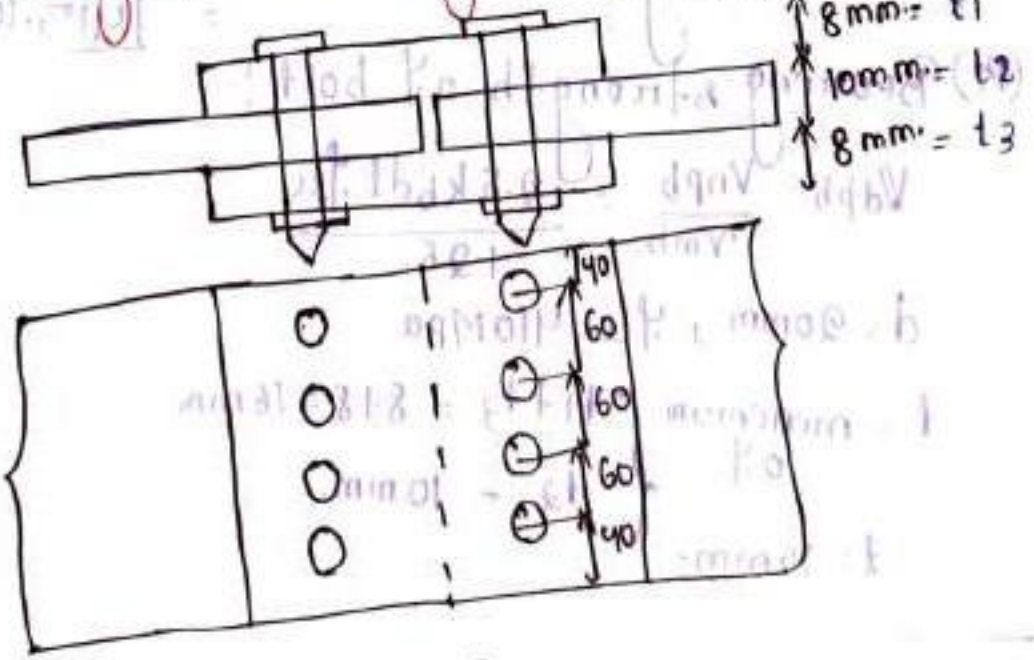
$T_{dg} = \frac{1600 \times 250}{1.1} = \boxed{363.636 \text{ kN}}$

$\therefore$ efficiency of joint $\eta = \frac{\text{strength of joint}}{\text{strength of solid plate}} \times 100$

$= \frac{173.846}{363.636} \times 100$

$\boxed{\eta = 47.8\%}$

Q-02 A single bolted double cover butt joint is used to connect two plates each 10mm thick. The bolts used were 20mm dia of grade 4.6 & cover plates were of 8mm thick. If 4 bolts were provided in a bolt line a pitch of 60mm with edge/end distance 40mm, Calculate the strength & efficiency of the joint.



Given data:

$f_u = 410$ Mpa, $f_{ub} = 400$ Mpa (for bolt grade 4.6)

$f_y = 250$ Mpa. $f_{yb} = 240$ Mpa

$d = 20$mm, $d_o = 22$mm.

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

Solution:

(A) Strength of a bolt in joint :-

(a) Shear strength of bolt :

$$A_{sb} = \frac{\pi}{4} \times d^2 = \frac{\pi}{4} \times (20)^2 = 314 mm^2$$

$$A_{nb} = 0.78 \times \frac{\pi}{4} d^2 = 0.78 \times \frac{\pi}{4} \times (20)^2 = 245 mm^2$$

for butt joint $n_n = 1$, $n_s = 1$.

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

$$= \frac{400}{\sqrt{3} \times 1.25} (1 \times 245 + 1 \times 314)$$

$$= 103.276 \text{ kN}$$

Shear strength of one bolt = 103.276 kN.

Shear strength of 4 no's bolt = 103.276 × 4

$$= \boxed{413.105 \text{ kN}}$$

(B) Bearing strength of bolt:

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

$d = 20$mm, $f_u = 410$ Mpa

$$t = \text{minimum of} \begin{cases} t_1 + t_3 = 8 + 8 = 16 mm \\ t_2 = 10 mm \end{cases}$$

$t = 10$mm.

$$k_b = \begin{cases} \frac{e}{3d_0} = \frac{40}{3\times22} = 0.606 \\ \frac{p}{3d_0} - 0.25 = \frac{60}{3\times22} - 0.25 = 0.659 \\ \frac{f_{ub}}{f_u} = \frac{400}{410} = 0.976 \\ 1 \end{cases}$$ whichever is minimum

$k_b = 0.606$

$$V_{dpb} = \frac{2.5 \times 0.606 \times 20 \times 10 \times 410}{1.25}$$

$= 99.384\text{ kN}$

bearing strength of one bolt = 99.384 kN.

bearing strength of 4 no's bolt = 99.384×4

$= \boxed{397.54\text{ kN}}$

(A) Design strength of bolt in joint :- minimum of $\begin{cases} V_{dsb} = 413.105 \\ V_{dpb} = 397.54 \end{cases}$

$= \boxed{397.54\text{ kN}}$

(B) Rupture strength of plate :-

$f_u = 410$, $\gamma_{m1} = 1.25$

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

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

$= (260 - 4\times22)\times10 = 1720\text{ mm}^2$

$$T_{dn} = \frac{0.9\times1720\times410}{1.25} = \boxed{507.74\text{ kN.}}$$

Design strength of joint:-

$$\text{minimum of}\begin{cases}\text{Design strength of } (397.54\text{kN}) \text{ bolt}\\ \text{Rupture strength of plate } \cancel{(174.036\text{kN})}\ (507.74\text{kN})\end{cases}$$

$= 397.54\text{ kN}.$

Strength of solid plate:-

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

$A_g = b \times t = 260 \times 10 = 2600\text{mm}^2$

$f_y = 250\text{ MPa}$

$\gamma_{m0} = 1.1$

$$T_{dg} = \frac{2600 \times 250}{1.1} = 590.91\text{kN}.$$

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

$$= \frac{397.54}{590.91} \times 100$$

$$\boxed{\eta = 67.28\%.}$$

Design procedure for bearing bolted joint:-

For the design of a lap or butt joint, when the thickness of plates & force to be transmitted is known, the following are steps for design:-

(1) The size of bolt is determined from Unwin's formula as $d = 6\sqrt{t}$

t = thickness of plate in mm.

The diameter of bolts so computed is rounded off to available size of bolts.

(2) The strength of bolts in shear & bearing are computed assuming suitable value of pitch, edge distance. The minimum of the above is taken as bolt value & the number of bolts required is obtained by dividing the applied force by bolt value.

(iii) The bolts are suitably arranged to produce a convenient & efficient joint.

(4) The joint is checked for rupture strength of plate with the assumed arrangement of bolts which should be more than the applied load.

Q-03 Two steel plates of 10mm & 12 mm thick are to be joined by a lap joint so as to transmit a load of 120kN using 20mm dia bearing bolts of property class 4.6 & plates of grade Fe410. Find the number & arrangement of bolts, if each of the plates are (i) 100mm wide (ii) 200mm wide.

Given data: dia of bolt (d) = 20mm.

$d_o = 22mm$

$P_u = 120kN$

$f_{ub} = 400$ Mpa

$f_u = 410$ Mpa.

(i) for 100 mm wide plate

$A_{nb} = 0.78 \times \frac{\pi}{4} d^2 = 0.78 \times \frac{\pi}{4} \times (20)^2 = 245 mm^2$

$A_{sb} = \frac{\pi}{4} \times d^2 = \frac{\pi}{4} \times (20)^2 = 314 mm^2$

$\gamma_{mb} = 1.25$

$$V_{dsb} = \frac{1}{\gamma_{mb}} \frac{f_{ub}}{\sqrt{3}} (n_n A_{nb} + n_s A_{sb})$$

$$= \frac{1}{1.25} \times \frac{400}{\sqrt{3}} (1 \times 245 + 0 \times 314)$$

$$= 45.26 \text{ kN}.$$

Design strength of a bolt in bearing:-

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

$f_u = 410 \text{Mpa}$, $\gamma_{mb} = 1.25$

$d = 20 \text{mm}.$

$t = 10 \text{mm}.$

$$k_b = \begin{cases} \frac{e}{3d_0} = \frac{40}{3 \times 22} = 0.606 \\ \frac{p}{3d_0} - 0.25 = \frac{60}{3 \times 22} - 0.25 = 0.66 \\ \frac{f_{ub}}{f_u} = \frac{400}{410} = 0.976 \\ 1 \end{cases} \text{ whichever is less}$$

$k_b = 0.606$

$$V_{dpb} = 2.5 \times 0.606 \times 20 \times 10 \times \frac{410}{1.25}$$

$$= 99.38 \text{ kN}.$$

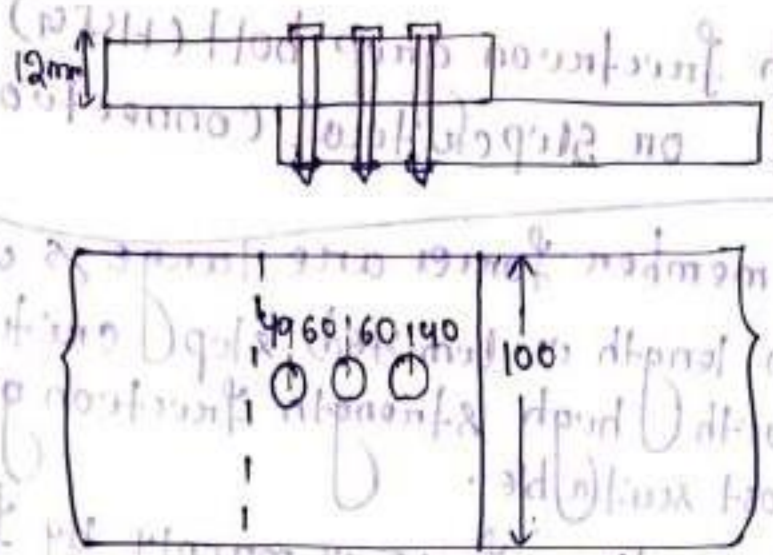
Design strength of a bolt = minimum of $\begin{cases} V_{dsb} = 45.26 \\ V_{dpb} = 99.38 \end{cases}$

= 45.26.

Number of bolts required to transmit a load of 120 kN:-

$$n = \frac{120}{45.26} = 2.65 \simeq 3 \text{ nos}.$$

(i) The bolts are to be arranged along the length in a row because width is not sufficient to accomodate them in a row along the width.



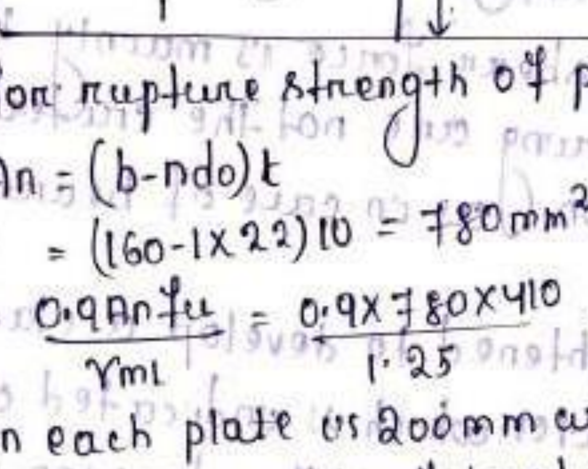
Check for rupture strength of plate :-

$A_n = (b - nd_0)t$

$= (160 - 1 \times 22)10 = 780 mm^2$

$T_{dn} = \frac{0.9 A_n f_u}{\gamma_{m1}} = \frac{0.9 \times 780 \times 410}{1.25} = 230.260 kN > 120 kN$ (ok)

(ii) When each plate is 200mm wide :-
To reduce the length of joint, bolts may be arranged along the width in a row :



Check for rupture strength of plate :-

$A_n = (b - nd_0) t$

$= (200 - 3 \times 22) \times 10 = 1340 mm^2$

$T_{dn} = \frac{0.9 A_n f_u}{\gamma_{m1}} = \frac{0.9 \times 1340 \times 410}{1.25}$

$= 395.57 kN > 120 kN$ (ok).

High Strength friction grip bolt (HSFG) or slip critical connection.

-> When the member forces are large & where the connection length is limited, slip critical connection with high strength friction grip bolts are most suitable.

-> Resistance to shear force is mainly by friction & shear or bearing are not the criteria for load transmission as in case of bearing type bolts.

-> The nut is tightened to develop a clamping force on the plates which is indicated as the tensile force in the bolt. This tension is about 90% of proof load.

-> When a shear load is applied to the joint, no slip will occur until the shear load exceeds the frictional resistance between the elements jointed. When the shear load exceeds the frictional resistance, slip occurs, then it acts as bearing bolt.

-> HSFG bolts are of two types.

(i) parallel shank type.

(ii) Waisted shank type.

(i) parallel shank type :-

-> parallel shank type HSFG bolts are designed for no slip at serviceability loads. Hence they slip at higher loads & slip into bearing at ultimate load. therefore such bolts should be checked for their bearing strength at ultimate load.

(ii) Waisted shank type :-

-> Waisted shank HSFG bolts are designed for no slip even at ultimate load & hence there is no need to check for their bearing strength.

Shear Capacity of HSFG bolts :- (cl: 10.4. IS 800)

The design slip resistance = $\frac{\text{nominal shear capacity of bolt}}{\gamma_{mf}}$

$$\boxed{V_{dsf} = \frac{\mu_f n_e K_h F_0}{\gamma_{mf}}}$$

μ_f = coefficient of friction (slip factor) (table 20).

n_e = number of effective interfaces offering frictional resistance to slip.

for Lap joint, $n_e = 1$

for butt joint, $n_e = 2$.

K_h = 1 for fasteners in clearance holes

= 0.85 for fasteners in oversized & short slotted holes & for long slotted holes loaded perpendicular to the slot

= 0.70 for fasteners in long slotted holes loaded parallel to the slot.

F_o = minimum bolt tension (proof load) at installation = $A_{nb} f_o$

A_{nb} = Net area of bolt at heads = $0.78 \times \frac{\pi}{4} d^2$.

f_o = proof stress = $0.70 f_{ub}$.

γ_{mf} = 1.1 (if slip resistance is designed at service load)

1.25 (if slip resistance is designed at ultimate load).

Reduction factor for shear capacity of HSFG bolt :- The provision for long joints in 10.3.3.1 shall apply to friction grip connections also.

Q. Two plates of 12mm thick are joined by double cover butt joint with 20mm HSFG bolts of property class 8.8 & cover plates of 8mm thick. Assuming that the fasteners are in clearance holes & slip factor as 0.30, determine the shear capacity of a bolt if (i) slip resistance is designed at service load (ii) slip resistance is designed at ultimate load.

For HSFG bolt of grade 8.8 :-

$f_{ub} = 800\ N/mm^2$.

For fasteners in clearance holes $K_h = 1$

slip factor (μ_f) = .3 (given)

Nominal shear capacity of bolt (V_{nsf}) ⇒

$$V_{nsf} = \mu_f n_e K_h F_0$$

For butt joint $n_e = 2$,

$$F_0 = 0.7 f_{ub} A_{nb}.$$

$$= 0.7 \times 800 \times 245.04 \quad \left(A_{nb} = 0.78 \times \frac{\pi}{4} \times d^2 \right)$$

$$= 137.22\ kN \quad = \left(0.78 \times \frac{\pi}{4} \times 20^2\right)$$

$$= 245.04$$

$$V_{nsf} = \mu_f n_e k_h F_0$$

$$= 0.3 \times 2 \times 1 \times 137.22$$

$$= \cancel{823.32}\ 82.332\ kN$$

(i) If slip resistance is designated at service load:

Design shear capacity of bolt (V_{dsf}) = $\frac{V_{nsf}}{\gamma_{mf}}$

$$V_{dsf} = \frac{\cancel{823.32}\ (82.332}{1.1} = \cancel{748.50\ kN}\ 74.85\ kN.$$

(ii) If slip resistance is designated at ultimate load $\gamma_{mf} = 1.25$

$$V_{dsf} = \frac{V_{nsf}}{\gamma_{mf}} = \frac{\cancel{823.32}}{\cancel{1.25}} = \frac{82.332}{1.25}$$

$$= 65.86\ kN$$

Q An ISA 110×110×8 mm carries a factored tensile force of 60kN. It is connected to a 12 mm thick gusset plate. Design a high strength bolted joint when (a) no slip is permitted (b) slip is permitted. Steel is of grade Fe410. Assume bolts in clearance holes & slip factor as 0.3.

Solution:- Dia of bolt, $d = 6\sqrt{t}$

$= 6\sqrt{8}$

$= 16.97$ mm

$\simeq 16$ mm

For HSFG bolt of property class 8.8:-

$f_{ub} = 800$ Mpa, $A_{nb} = 0.78 \times \frac{\pi}{4} \times d^2$

$= 0.78 \times \frac{\pi}{4} \times (16)^2$

$= 156.83 \text{ mm}^2$

For Fe410 grade of steel, $f_u = 410$ Mpa

(a) When slip is not permitted (slip critical connection):-

proof load $F_o = A_{nb} \times 0.7 f_{ub}$

$= 156.83 \times 0.7 \times 800$

$= 87.824$ kN

design shear capacity of a bolt,

$$V_{sf} = \frac{\mu_f n_e k_h F_o}{\gamma_{mf}}$$

For lap joint $n_e = 1$

For bolts in clearance hole $k_h = 1$

$$V_{sf} = \frac{0.3 \times 1 \times 1 \times 87.824}{1.25} = 21.10 \text{ kN}$$

($\gamma_{mf} = 1.25$ for slip resistance at ultimate load)

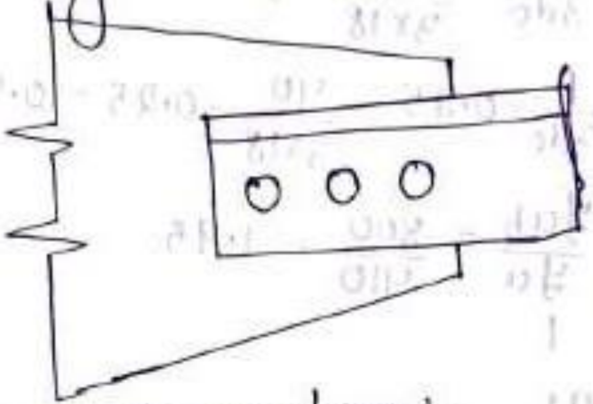
No. of bolts required =

$$\frac{\text{external load}}{\text{shear capacity of a bolt}} = \frac{60}{21.16}$$

$= 2.84 \simeq 3$ nos.

Hence provide 3 nos of 16mm dia 8.8 grade HSFG bolts for making the connection.

(b)



(b) Bearing type Connection:-

for bearing type connection we will design for bearing strength & shearing strength.

for Lap joint $n_n = 1$, $n_s = 0$,

(i) Strength of the bolt in single shear:-

$$V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}} = \frac{1}{\gamma_{mb}}\left\{\frac{f_{ub}}{\sqrt{3}}(n_n A_{nb} + n_s A_{sb})\right\}$$

$$= \frac{1}{1.25}\left\{\frac{800}{\sqrt{3}}(1\times157+0)\right\}$$

$$\boxed{V_{dsb} = 58.01\,kN}$$

(ii) strength of bolt in bearing:-

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

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

Assume $e = 1.5d_0 = 1.5 \times 18$
$= 27mm$
$\approx 30mm.$

$p = 2.5d = 2.5 \times 16$
$= 40mm.$

$$K_b = \text{least of} \begin{cases} \frac{e}{3d_0} = \frac{30}{3\times18} = 0.55 \\ \frac{p}{3d_0} - 0.25 = \frac{40}{3\times18} - 0.25 = 0.491 \\ \frac{f_{ub}}{f_u} = \frac{800}{410} = 1.95 \\ 1 \end{cases}$$

$K_b = 0.491$

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

$= 2.5 \times 0.491 \times 16 \times 8 \times \frac{410}{1.25}$

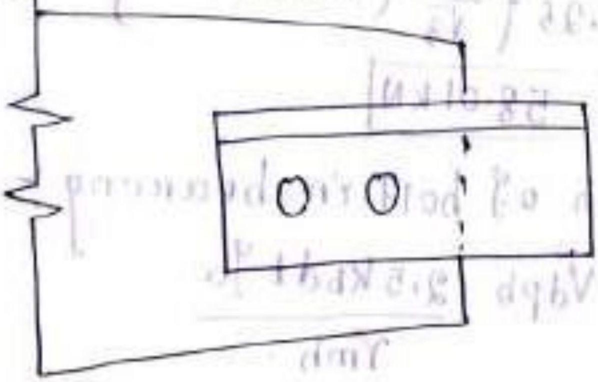
$\boxed{V_{dpb} = 51.54\,kN.}$

Strength of bolt = minimum of $\begin{cases} V_{dsb} = 58.01kN \\ V_{dpb} = 51.54kN \end{cases}$

$= 51.54\,kN.$

No. of bolts required $= \frac{60}{51.54} = 1.16 \approx 2$ Nos.

Hence provide 2 nos of 16mm dia HSFG bolts.



Welded Connection.

Welding is a method of connecting two pieces of metal by heating to a plastic or fluid state.

-> The elements to be connected are brought closer & the metal is melted by means of electric arc along with weld rod which adds metal to the joint. The bond is established between the two elements after cooling.

Types of welded joint:-

(i) butt weld (iii) slot weld / plug weld.

(ii) fillet weld

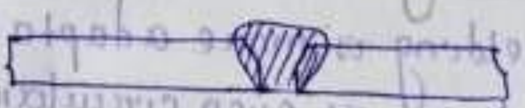
(1) **Butt weld**:- This is also known as groove weld.

-> Butt welds are provided when the members to be joined are placed end to end or aligned in the same plane. (used for plates upto 8 mm thickness).

-> Square butt weld on one side.

-> Square buttweld on both side:

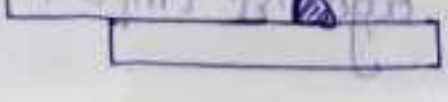
-> Single V butt joint:-

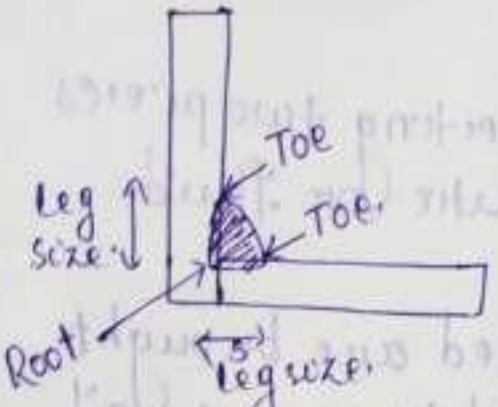


-> Double V-butt joint:-

-> Single U butt joint:

(2) **fillet weld**:- fillet welds are provided when two members to be joined are in different planes.

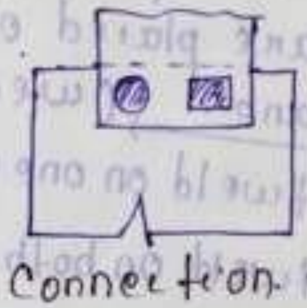




(iii) Slot & plug welds :- Slot & plug welds are used to supplement the fillet welds, when the required length of fillet weld can not be provided.

-> In slot weld, slots are made on one of the plates, which is kept with another plate & then fillet welding is made along the periphery of hole.

-> In case of plug weld, small holes are made in one plate & is kept over another plate to be connected & then entire hole is filled with fillet material.



Advantages of welded connection :-

(i) welding is more adaptable than bolting or riveting, as even circular sections can be easily connected by welding.

(ii). 100% efficiency can be achieved in contrast to bolted connection.

(iii) Since there is no ~~deduction~~ deduction for holes, the gross section is effective in carrying loads.

(iv) Better resistant against impact or vibrating loads.

(v) Results in light structure due to absence of connecting plates, gusset plates etc.

(vi) Noise pollution is nearly eliminated.

(vii) presents good asthetic appearance.

(viii) Connections are water & air tight.

Disadvantages of welded connection:-

(i) Skilled labour & electricity is neccessary for welding.

(ii) Welded joints are more brittle.

(iii) Difficult to detect defects like internal air pockets.

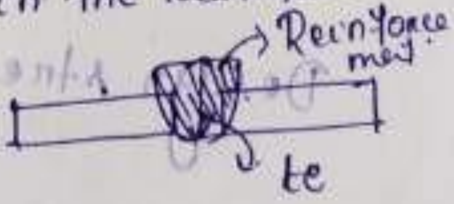
(iv) Welded joints are over rigid & proper welding in field condition is difficult task.

IS code provisions for welding:

(1) Butt weld.:-

(i) Reinforcement:- It is the extra material deposit above the surface level of parent metal to be connected. (It increases efficiency of joint).

-> but in case of repeated or vibrating loads stress concentration develops in the reinforcement leading to early failure.



-> Maxm reinforcement = 3mm

(ii) Size of weld

size of weld (S) = effective throat thickness (te)

for complete penetration:- size of weld = effective throat thickness = thickness of thinner member (tmin).

for incomplete penetration:-

size of weld(s) = effective throat thickness (te) = $\frac{5}{8} \times t_{min}$.

(iii) effective length (Leff) = Length of full size weld

(iv) effective area of weld = effective throat thickness (te) × effective Length of butt weld (Leff).

* Minimum Length of weld = 4 × size of weld.

Design strength of butt weld:-

The stress in weld shall not exceed those permitted stress in the parent metal

-> It can resist external load by tension or compression / shear action.

Design stress of butt/groove weld in tension or compression $f_{dw} = \frac{f_y}{\gamma_{mw}}$

f_y = yield stress of material

γ_{mw} = partial safety factor for weld material (1.25). (from table 5)

Design strength of butt weld = Design stress of weld × Area of weld.

$$= \boxed{\frac{f_{yw}}{\gamma_{mw}} \times A_w} \quad (A_w = t_e \times L_e)$$

Design strength of weld in shear.

$$\text{Design strength} = \boxed{\frac{f_{yw}}{\gamma_{mw} \times \sqrt{3}} \times A_w}$$

Reduction of design stress for Long joints:-

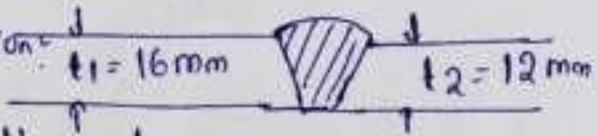
when the length of joint (L_j) > 150 t_t. (t_t = throat thickness) the design capacity of weld (f_{wd}) shall be reduced by a factor:-

$$\beta_{tw} = 1.2 - \frac{0.2L_j}{150t_t} \le 1$$

Q-1 Two steel plates of 16mm & 12mm thick are to be joined by butt welding. If effective length of weld is 180mm, determine the strength of joint for following cases (i) single V groove weld joint (ii) Double V groove weld joint.

(i) Single V butt weld joint:

As for incomplete penetration:



Size of weld = effective throat thickness $= \frac{5}{8} t_{min}$

$= \frac{5}{8} \times 12$

$= 7.5$mm.

effective Length of weld (L_e) = 180mm.

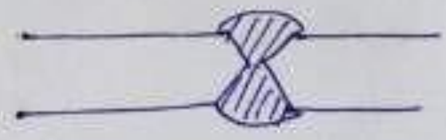
Design strength of welded joint =

$$\frac{f_{yw}}{\gamma_{mw}} \times (L_e \times t_e)$$

$$= \frac{250}{1.25} \times 180 \times 7.5$$

= 270kN

(ii) Double V butt weld joint:-



For complete penetration:

Size of weld = effective throat thickness = t_{min} = 12mm.

Design strength of weld:-

$$\frac{fy}{\gamma_{mw}}(le \times te)$$

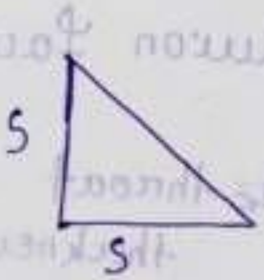
$$= \frac{250}{1.25}(180 \times 12)$$

$$= 432\ kN$$

Fillet weld

IS code provisions for welding:-

(i) Size of weld:-



Generally the size of weld is taken as the minimum weld leg size.

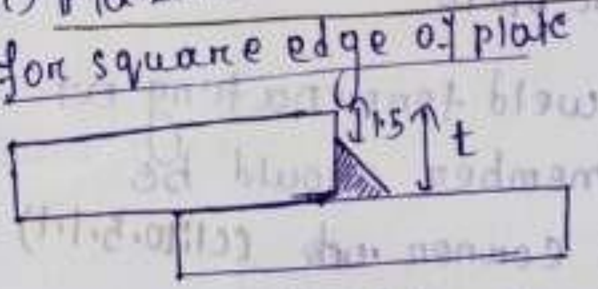
Minimum size of weld:- (table No 21)

Minimum size of weld depends upon thickness of thicker member.

Thickness of thicker part (mm)	Minimum size of weld (mm).
1-10	3
11-20	5
21-32	6
33-50	8 of first run, 10 for minimum size of weld.

(ii) Maximum size of weld:-

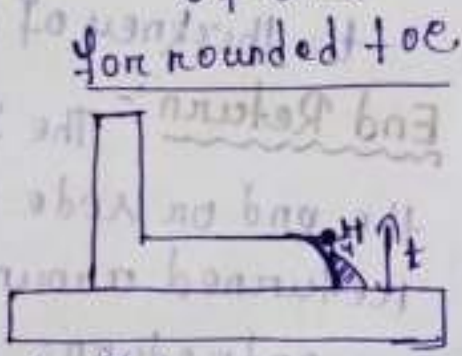
for square edge of plate



$S_{max} = t - 1.5$

(cl: 10.5.8.1 & 10.5.8.2)

for rounded toe



$S_{max} = \frac{3}{4}t$

Throat Thickness:- The throat of a fillet is the length of perpendicular from the right angle corner to hypotenuse. The effective throat thickness is calculated as:-

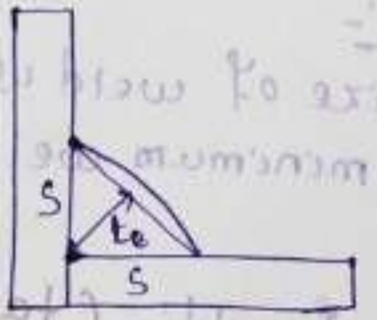
$$\boxed{\text{Throat thickness } (t_e) = K \times \text{size of fillet weld}}$$

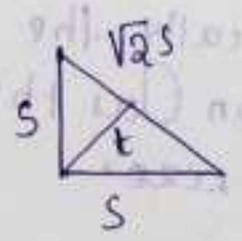
(Cl NO: 10.5.3.2)

Throat thickness = K × ~~fillet~~ size of fillet weld

The value of K depends upon the angle between fusion faces.

t_e = throat thickness





$$t = \frac{\sqrt{2}s}{2} = \frac{s}{\sqrt{2}}$$
$$= 0.7s.$$

K value (table NO-22).

Angle between fusion faces	60°-90°	91°-100°	101°-106°	107°-113°	114°-120°
constant(k).	0-7	0.65	0.60	0.55	0.50.

* In most cases, a right angled fillet is used, for which $k = 0.7$.

* Minimum value of throat thickness ($t_{e\,min}$):- 3 mm
Maximum value of throat thickness:- 0.7t ($t_{e\,max}$)
t = thickness of thinner plate

End Return:- The fillet weld terminating at the end or side of a member should be returned around the corner ~~is~~ (Cl: 10.5.1.1)
end return = 2× size of weld is.

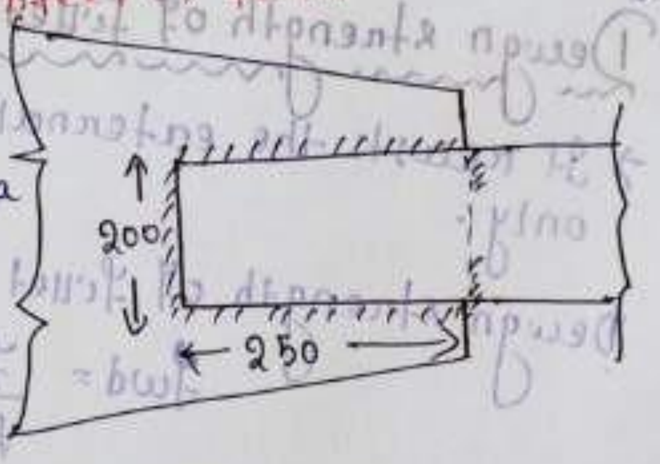
Assumptions in the analysis of welded joint:-

a) Welds connecting various parts are homogenous isotropic & elastic.

(b) The parts connected by the welds being rigid their deformation is usually neglected.

(c) Only stresses due to external loads are considered & effects of residual stresses, stress concentration & shape of welds are neglected.

Q A steel plate 200mm × 12mm is welded to a 10mm thick gusset plate such that the overlap of member is 250mm. If fillet weld of size 6mm is used for the connection, determine the design strength of the joint. Given that shop welding is to be done on three sides & grade of steel is Fe 410.



Given data :-

$f_u = 410\,\text{Mpa}$, $f_y = 250\,\text{Mpa}$

for shop welding,

$\gamma_{mw} = 1.25$ (Table no-05)

$l_w = 2\times250 + 200 = 700\,\text{mm}$ (Assuming .

$l_e = l_w - 2\times2\times s$

$= 700 - 4\times6$

$\Rightarrow 700 - 24 = 676\,\text{mm}$.

Effective throat thickness $(t_e) = k\times s$ (for 90° angle $k = 0.7$)

$= 0.7\times6 = 4.2$

Design strength of weld :-

$$f_{dw} = L_w t_e \times \frac{f_u}{\sqrt{3}\times\gamma_{mw}}$$

$$= 676 \times 4.2 \times \frac{410}{\sqrt{3}\times 1.25}$$

$$= 537.661\ kN$$

Design strength of plate :-

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

$A_g = 200 \times 12$
$= 2400$
$f_y = 250\ Mpa$
$\gamma_{m0} = 1.1$

$$= \frac{2400 \times 250}{1.1}$$

$$T_{dg} = 545.45\ kN$$

Design strength of joint :- minimum of $\begin{cases} f_{dw} = 537.661 \\ T_{dg} = 545.45 \end{cases}$

$= 537.661\ kN.$

Design procedure for fillet weld :-

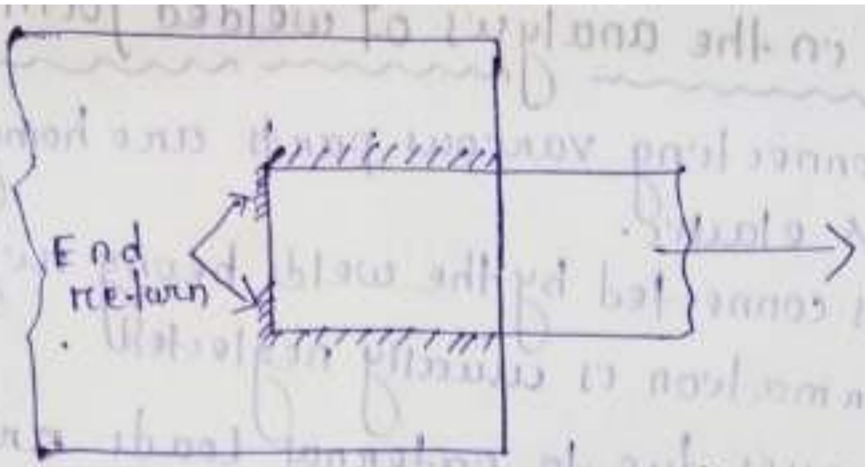
(1) The size of weld is selected based on the thickness of members to be joined.

(2) Depending on the angle between fusion faces, effective throat thickness is calculated.

(3) If force to be transmitted is not given, gross design strength should be taken as the ~~rupture~~ strength of plate to develop maximum force.

(4) strength of weld per mm length is calculated.

(5) effective length of weld required is calculated by dividing the factored load or design strength



Effective Length :- (cl; 10.5.4)

effective length (Le) = overall length of weld − end return (2 × S)

→ effective length of weld is designed to transmit load.

→ Minimum effective length = 4 × size of weld (S).

Lap Length: Minimum lap Length = 4 × tmin or 40mm } whichever is more.

Design strength of fillet weld :- (cl; 10.5.7).

→ It resists the external load by shear action only.

Design strength of fillet weld:-

$$f_{wd} = \frac{f_u}{\sqrt{3} \times \gamma_{mw}} \times \text{Area of weld}$$

$$\boxed{f_{wd} = \frac{f_u}{\sqrt{3} \times \gamma_{mw}} \times (L_e \times t_e)}$$

f_u = ultimate stress of weld

γ_{mw} = partial safety factor (table 5).

$L_e = L - 2 \times S$, $t_e = k \times S$.

by the strength of weld per mm length.

(6) Total length of weld is suitably arranged.

(7) If the length of weld in the direction of load exceeds 150t, the design capacity of weld is reduced by reduction factor for long joints.

(8) check for minimum lap joint.

(9) End returns of length equal to twice the size of weld at each end of fillet weld are provided.

Q. Design a suitable fillet weld to connect a tie bar 60mm x 8mm to a 12mm thick gusset plate so as to develop maximum force of

(i) shop welding is done on two sides.

(ii) field welding is done on three sides.

Assuming the grade of steel for tie bar as fe410

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

for 12mm thick gusset plate,

Minimum size of weld = 5mm

for 8mm thick tie bar,

Maximum size of weld = 8-1.5

= 6.5mm

Hence, let us provide a weld size of (S) = 6mm.

effective throat thickness (t_e) = 0.7×S

= 0.7×6

= 4.2mm.

To develop maximum force, the design strength of weld should be equal to strength of the plate strength of plate in yielding

$$T_{dg} = \frac{A_g f_y}{\gamma_{m0}} = \frac{(b \times t) f_y}{\gamma_{m0}} = \frac{60 \times 8 \times 250}{1.1}$$

$$= 109.09 \text{ kN.}$$

(c) for shop welding on two sides, $\gamma_{mw} = 1.25$ (table-5).

Strength of weld per mm length:

$$f_{dw}: L_w t_e \frac{f_u}{\sqrt{3}\, \gamma_{mw}} = 6 \times 4.2 \times \frac{410}{\sqrt{3} \times 1.25}$$

$$= 0.795 \text{ kN/mm } l_w$$

for maximum efficiency:

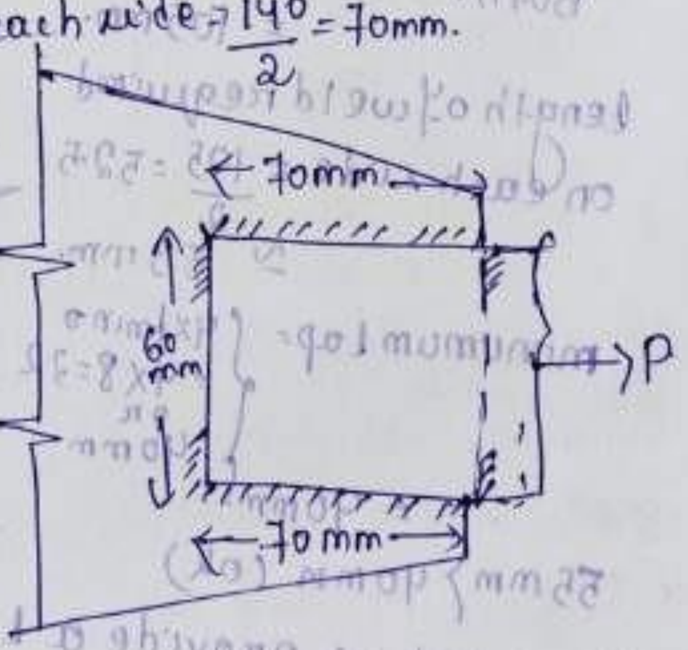
$$T_{dg} = f_{dw}$$

$$\Rightarrow 109.09 = 0.795\, L_w$$

$$\Rightarrow L_w = 137.22 \text{ mm}$$

$$\simeq 140 \text{ mm.}$$

length of weld on each side $= \frac{140}{2} = 70$mm.



$l_w = 140\text{mm} > 4 \times s$ (ok)
$= 4 \times 6$
$= 24$mm

minimum Lap length
$= 4 \times t_{min}$
$= 4 \times 8$
$= 32$mm
or
40mm

whichever is more

$= 40$mm. ∴ $70 > 40$mm (ok).

length on each side = 70+2×2×6
= 94mm.

Total length of weld = 94×2 = 188mm.

(ii) For field welding on three sides:-

$\gamma_{mw} = 1.5$ (table-5).

strength of weld per mm length:

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

$$= l_w \times 4.2 \times \frac{410}{\sqrt{3} \times 1.5} = 0.663\, l_w$$

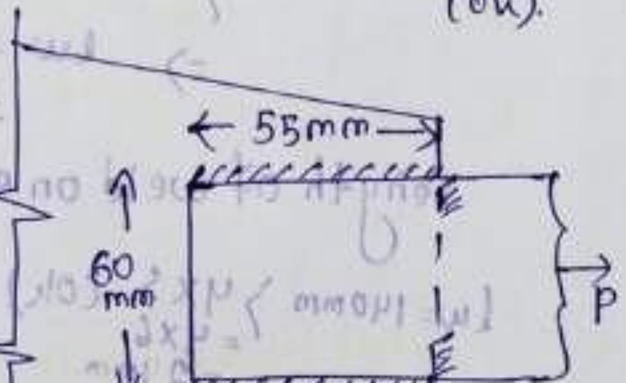
For maximum efficiency:

$T_{dg} = 0.663\, l_w \Rightarrow 109.09 = 0.663\, l_w$

$\Rightarrow l_w = 164.54$ mm

$l_w \simeq 165$ mm. $\nless 4 \times s = 4 \times 6 = 24$mm (ok).

length of weld required on both sides => 165-60
= 105 mm.



length of weld required on each side = $\frac{105}{2}$ = 52.5
≃ 55 mm.

minimum lap = $\left\{ \begin{array}{l} 4 \times t_{min} \\ = 4 \times 8 = 32 \\ \text{or} \\ 40\text{mm} \end{array} \right.$

= 40mm

55mm > 40mm (ok)

Hence let us provide a lap of 55mm & 6mm size lap.

welds on three sides with end return-

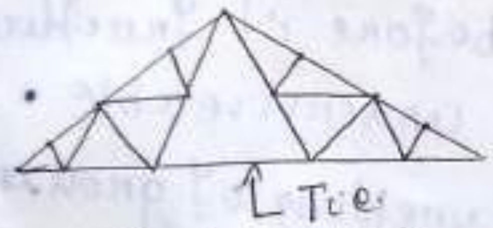
Total length of weld = 60+2×55+24 = 194mm

2×2×6 = 24mm.

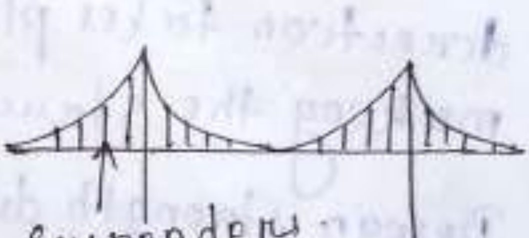
Tension Member

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



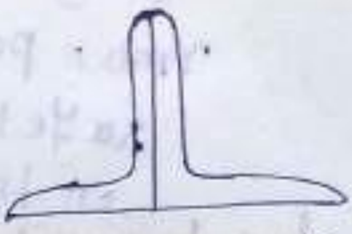
(Roof truss)



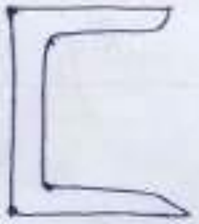
(Suspension bridge).

Common Shapes of Tension Member :-

(angle section)



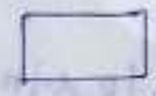
(double angle section)



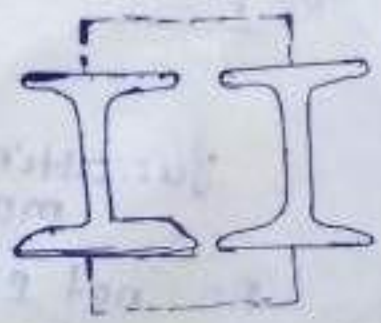
(channel section)

(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:-

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

$= (b - n d_0) t$ (for chain bolted connection).

b = width of plate

d_o = 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.

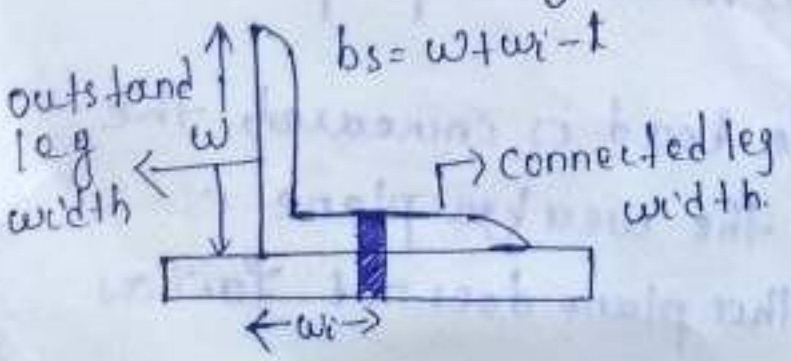
A_{go} = Gross area of outstanding leg.

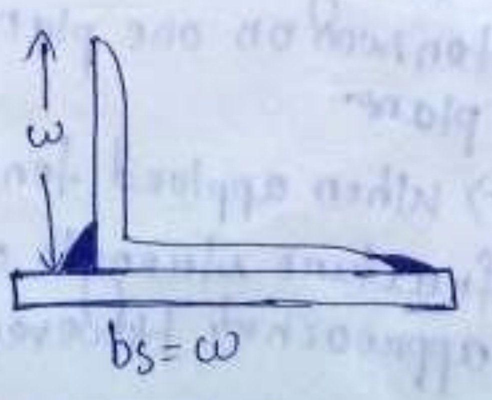
$$\beta = 1.4 - 0.076\left(\frac{w}{t}\right)\left(\frac{f_y}{f_u}\right)\left(\frac{b_s}{L_c}\right) \le \frac{f_u \gamma_{m0}}{f_y \gamma_{m1}} \cancel{\ge 0.7}.$$

$$\ge 0.7$$

w = outstand leg width

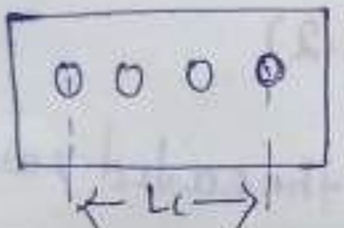
b_s = shear lag width.





t= thickness of the leg.

Lc= 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 Lc.



* The rupture strength Tdn of double angles, channels, I sections & other rolled steel sections may be calculated by the same equation as for single angle, but width bs 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 Tdb, 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

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

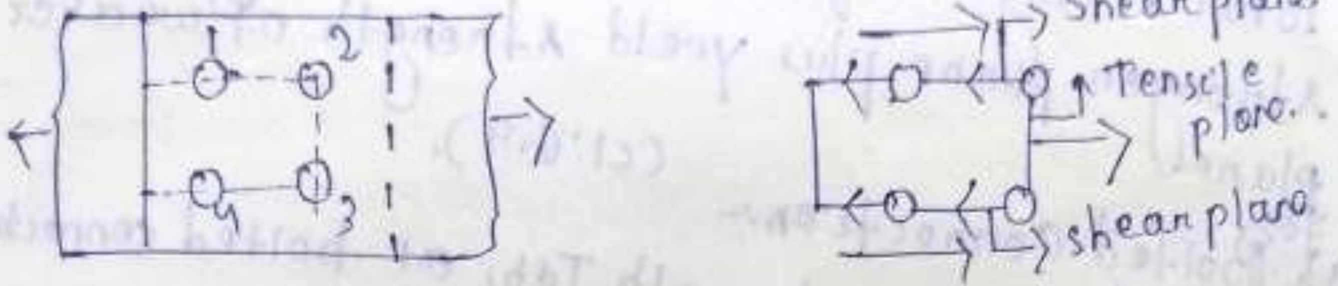
A_{tg} = ~~shear o~~ gross area in ~~shea~~ tensile section

A_{vg} = gross area in shear section

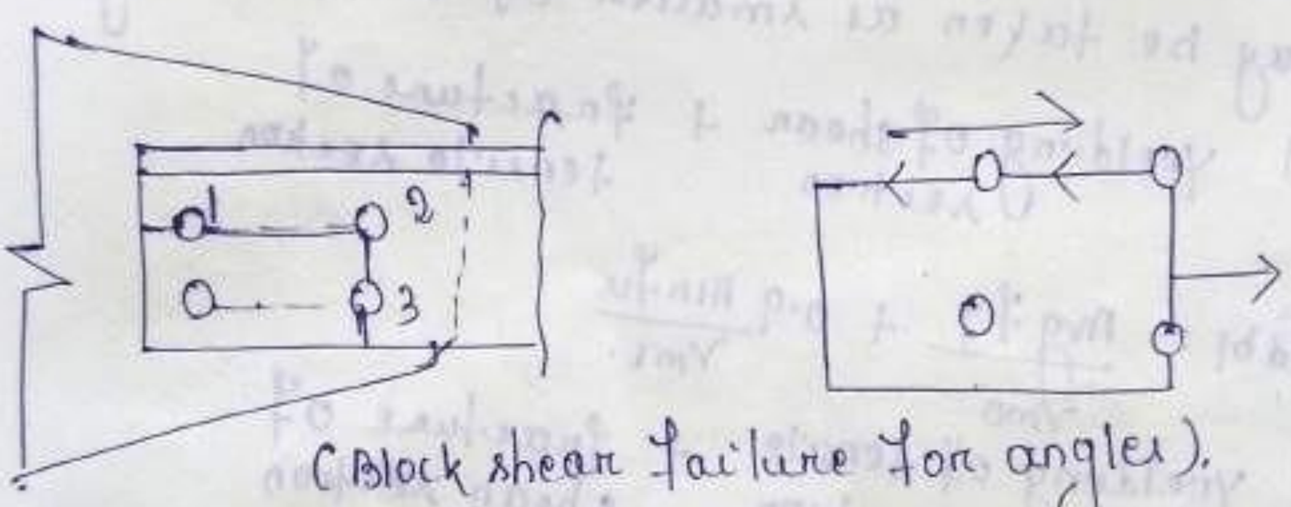
A_{tn} = net area in tensile section

A_{vn} = net area in shear section.

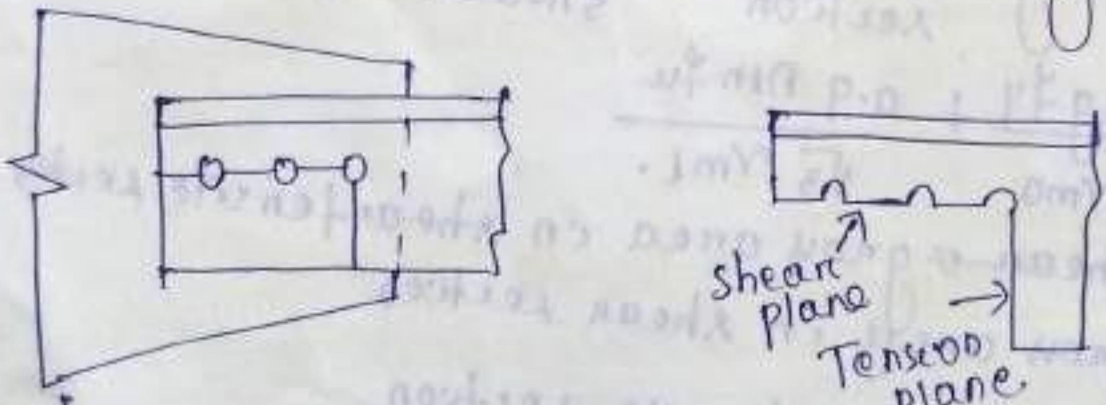
(b) Welded connection:- The block shear strength, Tdb, for welded connection shall be calculated by taking an appropriate section is member around the end weld. However for welded angle tension members, since it is the gross area only that is involved. Atn & Avn is to be replaced by Atg & Avg respectively in above equation:

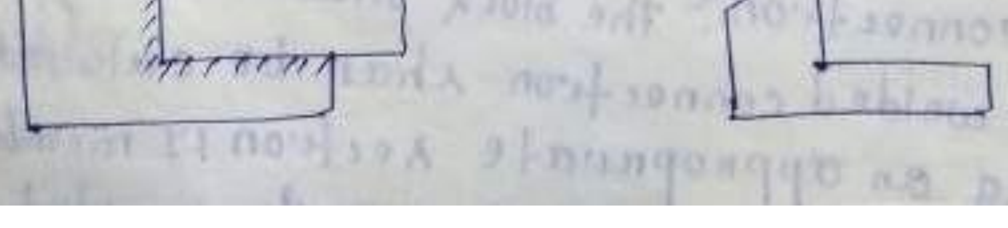


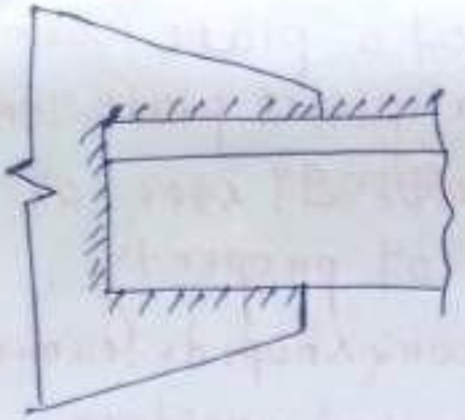
(Block shear failure for plates).

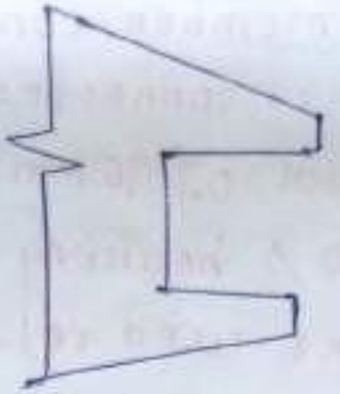


(Block shear failure for angles).









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 errection, wind or earthquake load etc. From there 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 x 8mm 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:-

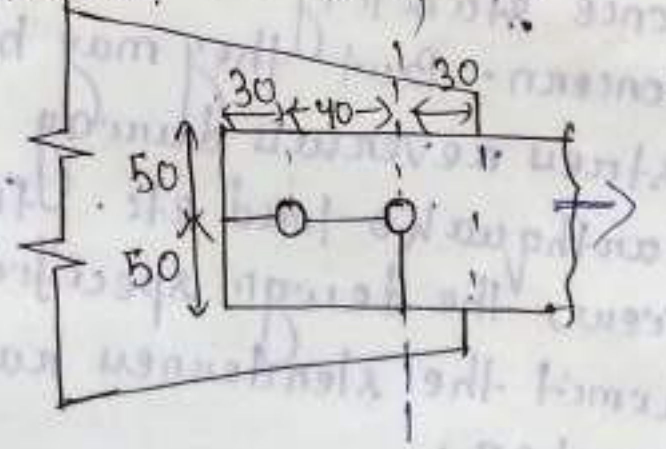
$f_u = 410 Mpa$, $f_y = 250 Mpa$, $\gamma_{mo} = 1.1$, $\gamma_{m1} = 1.25$

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

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

Strength of the plate:-

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



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

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

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

$$= 181.82 kN$$

(ii) Strength of plate against rupture of critical section

In this case section 1-1 is critical.

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

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

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

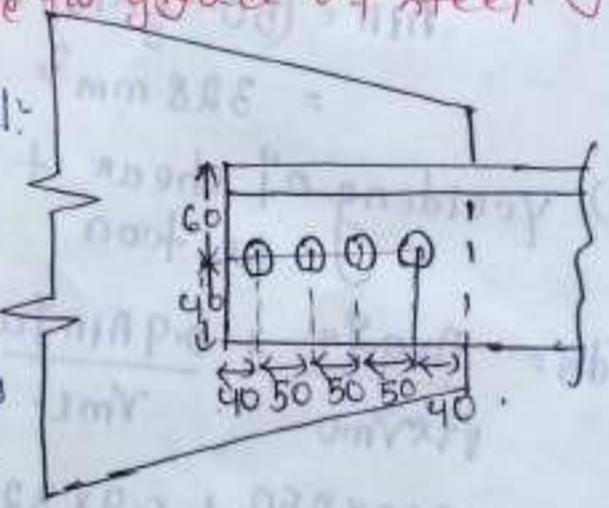
Block shear strength = minimum of T_{db1} & T_{db2}

T_{db} = 149.54kN.

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

= 149.54kN.

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 20mm dia yield 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 fe410 grade of steel:-

f_u = 410Mpa

f_y = 250 Mpa.

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

d = 20mm, d_o = 22mm, γ_{mb} = 1.25.

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

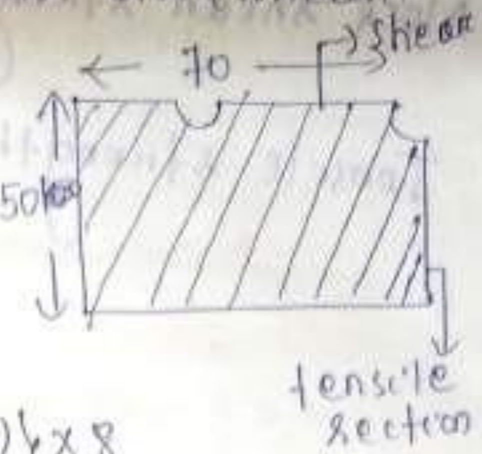
Strength of angle section:-

(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:-

$A_{vg} = 70\times8$

$= 560\text{mm}^2$

net area at shear section:-

~~Atn~~ $A_{vn} = \{70-(1.5\times18)\}\times8$

$= 344\text{mm}^2$

gross area at tensile section:-

$A_{tg} = \cancel{100\times8}\ 50\times8$

$= 400\text{mm}^2$

net area at tensile section:-

$A_{tn} = (50-\frac{1}{2}\times18)\times8$

$= 328\text{ mm}^2$

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

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

$$= \frac{560\times250}{1.1\times\sqrt{3}} + \frac{0.9\times328\times410}{1.25}$$

$= 170.31$ KN.

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

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

$$= \frac{0.9\times\overset{344}{\cancel{328}}\times410}{1.25\times\sqrt{3}} + \frac{400\times250}{1.1} = 149.54\text{ KN}.$$

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

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

Net area of connected leg,

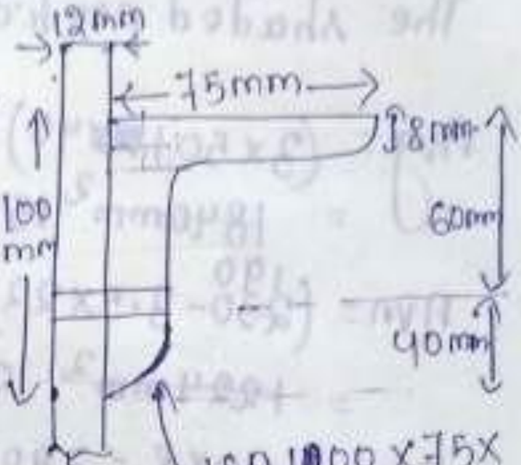
$$A_{nc} = \left(100 - 22 - \frac{8}{2}\right) \times 8$$

$$= 592 \text{ mm}^2$$

Gross area of outstanding leg,

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

$$= 568 \text{ mm}^2$$



ISA $1\cancel{0}00 \times 75 \times$ 8 mm

Gusset plate of 12 mm thick.

w = outstand leg width = 75 mm.

$w_1 = w - g$.

$= 100 - 40 = 60$ mm.

Shear lag width (bs) =

$$w + w_1 - t$$

$$= \cancel{100}\ 75 + 60 - 8 = 127 \text{ mm}.$$

Length of connection (Lc) = 3×50

= 150mm.

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

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

$$= 1.032 \leq 1.4432$$

$$\geq 0.7 \text{ (ok)}.$$

hence $\beta = 1.032$.

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

$$= \frac{0.9 \times 592 \times 410}{1.25} + \frac{1.032 \times 568 \times 250}{1.1}$$

$$= 307.980 kN.$$

(iii) Strength of angle section against block shear:-

The shaded portion may shear off.

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

$$= \cancel{1840 mm^2}\ 1520 mm^2$$

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

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

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

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

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

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

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

$$= 267.934 kN$$

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

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

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

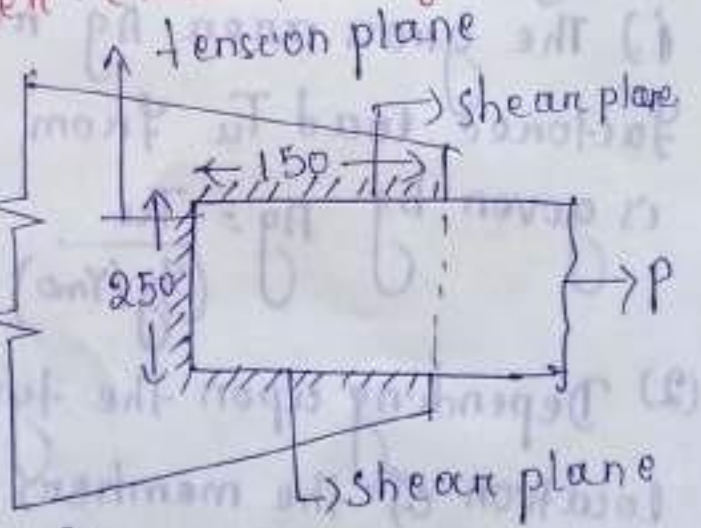
block shear strength = minimum of $\begin{cases} T_{db1} = 267.934\text{ kN} \\ T_{db2} = 226.799\text{ kN} \end{cases}$

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

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

$= 226.79\text{ kN}$.

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

$f_u = 410\text{ Mpa}, f_y = 250\text{ Mpa}$

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

$A_{vg} = A_{vn} = 150 \times 6 \times 2$

$= 1800\text{ mm}^2$

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

block shear strength:-

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

$$= \frac{1800 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 1\overset{5}{8}00 \times 410}{1.25}$$

$= 678.98\text{ kN}$.

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

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

$$= 647.689 \text{ kN}$$

block shear strength T_{db} = minimum of $\begin{cases} T_{db_1} = 678.95 \text{ kN} \\ T_{db_2} = 647.68 \text{ kN} \end{cases}$

$T_{db} = 647.689$ kN.

Design of tension members subjected to axial load :-

Design steps :-

(i) The gross area 'Ag' required to carry the factored load Tu 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 T_d of trial section is calculated considering minimum of T_{dg}, T_{dn}, & T_{db} which should be more than the factored Load.

(5.) If ~~Td ≮ P~~ $T_d < P$, 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 300kN. 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:-

$$A_g = \frac{T_u \gamma_{m0}}{f_y} = \frac{300\times10^3\times1.1}{250} = 1320 mm^2$$

Let us adopt an angle section whose area > $1320 m^2$ from steel table.

So choose an angle section 90x60x10mm having gross area $1401 mm^2$ connected by long 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

$d_0 = 20+2 = 22mm$, $\gamma_{mb} = 1.25$.

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

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

$f_{ub} = 400 Mpa.$

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 $e = 1.5 \times 22 = 33mm$, $p = 2.5 \times 20 = 50mm$.

$e \simeq 40mm.$

$$k_b = \text{minimum of} \begin{cases} \frac{e}{3d_0} = \frac{40}{3 \times 22} = 0.606 \\ \frac{p}{3d_0} - 0.25 = \frac{50}{3 \times 22} - 0.25 = 0.51 \\ \frac{f_{ub}}{f_u} = \frac{400}{410} = 0.975 \\ 1 \end{cases}$$

$k_b = 0.51$

Design bearing strength of bolt (Vdpb):-

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

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

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

Strength of one bolt =

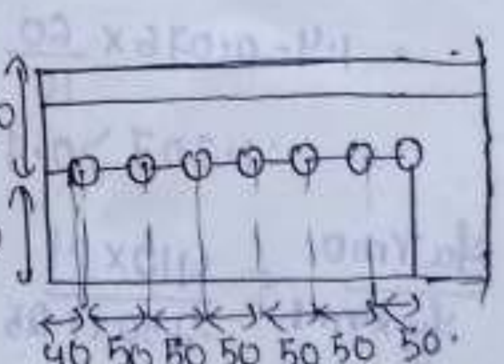
minimum of $\begin{cases} \text{(i) } V_{dsb} = 45.27 \text{ kN.} \\ \text{(ii) } V_{dpb} = 83.64 \text{ kN} \end{cases}$

$= 45.27$ kN.

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

$= \frac{300}{45.27} = 6.63 \approx 7$ 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)

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

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

$= 300$ mm

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

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

(i) Strength against yielding of gross section:

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

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

$A_g = 1401$ from steel table

$f_y = 250$ Mpa.

(c) 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 = 100 mm.

length of connection (L_c) = 6×50 = 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).

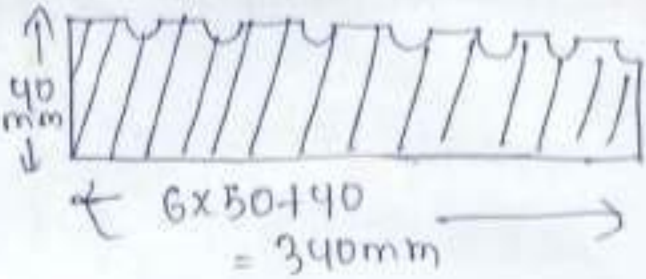
$$T_{dn} = \frac{0.9 A_{nc} f_u}{\gamma_{m1}} + \frac{\beta A_{go} f_y}{\gamma_{m0}}$$

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

= 349.35kN > 300 kN (ok).

(iii) Strength of plate in block shear :-

(cl:6.4)



$A_{vg} = 340\times10$
$= 3400\,mm^2$

$A_{vn} = (340-6.5\times22)\times10$
$= 1970\,mm^2$

$A_{tg} = 40\times10 = 400\,mm^2$

$A_{tn} = (40-0.5\times22)\times10 = 290\,mm^2$

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

$$= \frac{3400\times250}{\sqrt{3}\times1.1} + \frac{0.9\times290\times410}{1.25} = 531.742\,kN$$

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

$$= \frac{400\times250}{1.1} + \frac{0.9\times1970\times410}{\sqrt{3}\times1.25} = 426.67\,kN$$

block shear strength (T_{db}) =

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

$= 426.67\,kN > 300\,kN$ (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)

Compression Member

-> pure compression members are structural elements subjected to axial compressive force only.

-> Axial compressive force means force applied along the centroid of longitudinal axis of the cross-section. A column is defined as a structural member whose longitudinal dimension is comparatively more than its lateral dimension & subjected to compressive force in a direction parallel to its longitudinal axis.

Common Shapes of compression members:

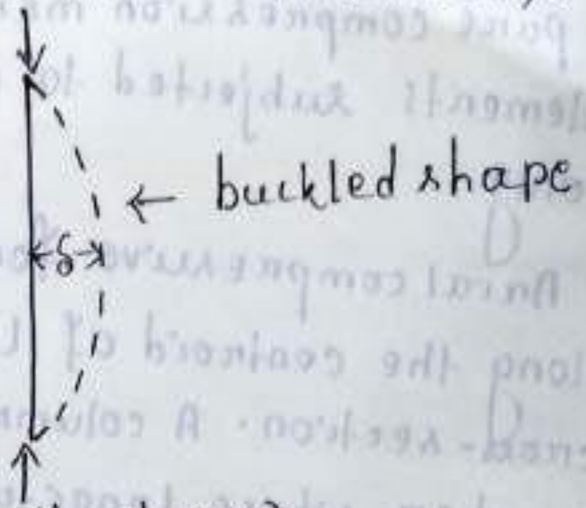
(1) Single angle section
(2) Double angles back to back
(3) T sections.
(4) Single channels.
(5) Circular hollow sections.
(6) Square and rectangular hollow section
(7) I section
(8) Builtup section.

Buckling of Columns.

-> Buckling is defined as the sudden bending of the compression members under compression.

-> Due to buckling, deformation developed in a column occurs in a direction or plane normal to the direction of loading.

→ Buckling resistance depends on magnitude of the applied load, stiffness of member & length of member.



(buckling of column).

Effective length of Column :-

Effective length (le) of a column is defined in terms of equivalent length of column hinged at both ends for various end condition.

Effective length (le) = KL

L = actual length of column.

K = constant depends upon end support condition.

value of k → table No-11 of IS 800:2007

* Where the boundary conditions in the plane of buckling can be assessed, the effective length 'KL' can be calculated on the basis of Table No-11 of IS 800:2007.

* In case of bolted or welded trusses, the effective length KL shall be taken as 0.7 to 1 times the actual length.

Appropriate radius of gyration:-

The radius of gyration may be different about different transverse axes (YY, ZZ, UU, VV etc).

-> However the radius of gyration of compression member about the axis of buckling is known as appropriate radius of gyration

Slenderness Ratio

It is defined as the ratio of effective length to the corresponding radius of gyration of section. $\boxed{\lambda = \frac{le}{r_{min}} = \frac{KL}{r_{min}}}$

-> Maximum value of slenderness ratio:- from table NO 3 of IS 800: 2007.

* Also the compression members are required to satisfy the limiting width to thickness ratio depending upon class of section as per table-2 of IS 800: 2007.

Buckling class of cross sections (cl: 7.1.2.2).

(Table NO-10 of IS 800: 2007)

-> The minimum load, that causes collapse of column section, is to be determined by checking safety of column about all axis.

-> However columns have tendency to buckle about Z, Y, V or U axis.

-> Depending upon geometric dimensions of individual structural elements & their corresponding limits, buckling classes are of

Four types:

(i) buckling class a (iii) buckling class c

(ii) buckling class b (iv) buckling class d.

Design Compressive Stress & Strength of compression member:-

The design compressive strength P_d of a member is given by:

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

A_e = effective sectional area. (from steel table)

f_{cd} = design compressive stress (by interpolation from table 9-(a),b),c),d))

$$\boxed{P < P_d}$$

→ P = axial compressive Load (load applied).

→ To calculate value of f_{cd} from Table 9(a), 9(b), 9(c), 9(d), we require λ, f_y & buckling class.

Q Calculated factored axial load on the column section ISHB 400 @ 806.38 N/m. The height of column is 3m & it is pin-ended. Use steel of Fe410 grade.

(1) For steel grade Fe 410,

f_u = 410 Mpa, f_y = 250 Mpa.

γ_{m0} = 1.1, E = 2×10^5 Mpa.

(2) For ISHB 400 @ 806.38 N/m:

section properties (from steel table):-

t_w = 10.6 mm.

h = 400mm, b_f = 250mm, t_f = 12.7mm,

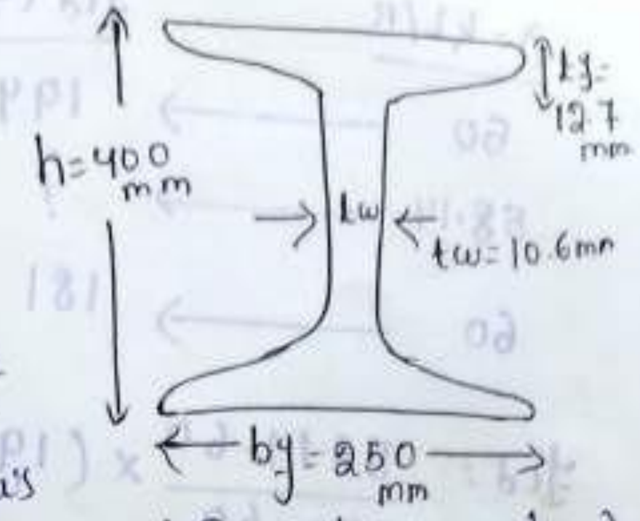
A = 104.66 cm^2 = ~~104.66~~ 10466 mm^2, r_{zz} = 166.1 mm

r_{yy} = 51.6 mm

(3) buckling class (cl: 7.1.2.2.1) (Table 10).

$\frac{h}{b_f} = \frac{400}{250} = 1.6 > 1.2$

$t_f = 12.7mm \leq 40mm.$



for buckling about z-z axis

= buckling class a

for buckling about y-y axis

= buckling class b (Rolled I section)

(ISHB 400 @ 806.38 N/m)

(4) effective sectional area & effective length of column:-

Ae = 10466 mm²

Le = KL

for columns pinned at both ends value of k = 1 (from table NO-11)

L = Length of column given = 3 m.

Le = 1×3 = 3 m. = 3000mm

(5) slenderness ratio (λ) :-

$\lambda_{yy} = \frac{kL}{r_{yy}} = \frac{3\times1000 \text{ mm.}}{51.6 \text{ mm.}} = 58.14$

$\lambda_{zz} = \frac{Le}{r_{zz}} = \frac{kL}{r_{zz}} = \frac{3000}{166.1} = 18.06$

(6) Design compressive strength :-

(i) About buckling axis YY & buckling class b :-

$\lambda_{yy} = 58.14$, $f_y = 250 Mpa.$

We can get value of fcd by interpolation from table No 9(b) of IS 800:

$\lambda = KL/r$		f_{cd} (for f_y 250 Mpa)
50	$\longrightarrow$	194
58.14	$\longrightarrow$	?
60	$\longrightarrow$	181

$$f_{cd} = \frac{58.14-60}{50-60} \times (194-181) + 181$$

$$= 183.42 \text{ Mpa}$$

factored axial load or design compressive strength of column $(P_d) = A_e f_{cd}$

$$= 10466 \times 183.42$$

$$= 1919 \text{ kN}$$

(ii) About buckling axis zz, buckling class a:

$\lambda_{zz} = 18.06$, $f_y = 250$ Mpa

by interpolating from table No 9(a):-

$\lambda = KL/r$		f_{cd} (for $f_y = 250$ Mpa)
10	$\longrightarrow$	227
18.06	$\longrightarrow$	?
20	$\longrightarrow$	226

$$f_{cd} = \frac{18.06-20}{10-20} \times (227-226) + 226$$

$$= 226.194 \text{ Mpa}$$

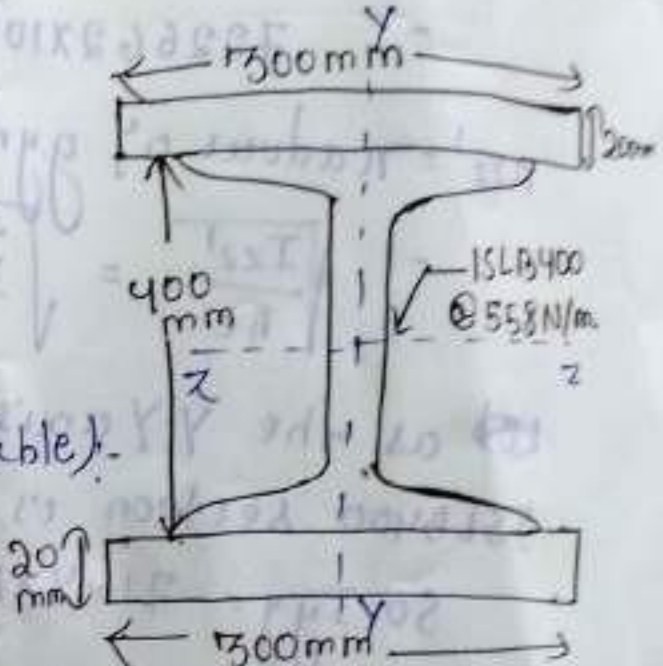
factored axial load $P_d = A_e f_{cd}$

$$= 10466 \times 226.194$$

$$= 2367.35 \text{ kN}$$

∴ Design factored axial load = minimum of two = 1919 kN (Ans)

Q A compound column consists of ISLB 400 @ 558 N/m with one cover plate 300 mm × 20 mm in each flange. The actual length of column is 4.2 m & it is fixed at one end and hinged at other end, using the grade of steel Fe410, determine the working load that the column can carry.



(1) For Fe410 grade steel

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

$\gamma_{m0} = 1.1$,

(2) For ISLB 400 @ 558 N/m

Section properties (from steel table):-

$h = 400 mm$ $I_{zz} = 19306.2 \times 10^4 mm^4$

$b_f = 165 mm$

$t_f = 12.5 mm$ $I_{yy} = 716.4 \times 10^4 mm^4$

$t_w = 8 mm$, $r_{zz} = 163.3 mm$, $r_{yy} = 31.5 mm$.

$A = 7243 mm^2$

(3) buckling class:-

As it is a builtup section, so it belongs to buckling class 'c' about any axis. (table-10)

(4) effective area ~~& effective length~~:-

A_e = (Area of I section) + 2×(area of one cover plate)

= 7243 + 2 × (300 × 20)

= 19243 mm²

(5) Moment of Inertia of whole section about Y-Y & Z-Z axis,

230	29.2	29.4	29.5	29.6	29.7	29.8	29.9	30.0	30.1	30.3	30.4
240	27.1	27.2	27.3	27.3	27.4	27.5	27.6	27.7	27.8	27.9	28.0
			25.3	25.3	25.4	25.5	25.6	25.7	25.8	25.9	26.0

Let I_{zz}^{1} = moment of Inertia of whole section about zz^{1} axis.

I_{zz}^{1} = (Moment of Inertia of ISLB400 section about zz axis i.e. I_{zz}) + 2x (moment of Inertia of one cover plate about z-z axis).

$= 19306.2 \times 10^4\, mm^4 + 2 \times \left(\frac{300 \times (20)^3}{12} + 300 \times 20 \times (210)^2 \right)$

$= 722662 \times 10^3\, mm^4$

r_{zz}^{1} = radius of gyration of whole section

$= \sqrt{\frac{I_{zz}^{1}}{Ae}} = \sqrt{\frac{722662 \times 10^3}{19243}} = 193.78\, mm$

as the Y-Y axis of both cover plates & ISLB400 section is same, so I_{yy} will not be modified.

so $I_{yy} = 716.4 \times 10^4\, mm^4$

$r_{yy} = 31.5\, mm$

(6) effective length of column & slenderness ratio-

for one end fixed & other end hinged-

effective length $L_e = kL = 0.8 \times 4.2 \times 1000$

$= 3360\, mm$

(table No-11)

slenderness ratio:-

$\lambda_{zz} = \sqrt{\ } = \frac{KL}{r_{zz}^{1}} = \frac{3360}{193.78} = 17.34$

$\lambda_{yy} = \frac{KL}{r_{yy}} = \frac{3360}{31.5} = 106.67$

(7) Design compressive strength (P_d):-

$(P_d)_{z-z}$ axis :-

We can get value of f_{cd} by interpolating from table 9(c) :-

λ_{zz}	f_{cd} (for f_y = 250 Mpa)
10	227
17.34	?
20	224

$$f_{cd} = \frac{17.34-20}{10-20} \times (227-224)+224$$

$= 224.798$ Mpa.

Factored axial load $(P_d)_{zz} = A_e f_{cd}$

$= 19243 \times 224.798$

$= 4325.787$ kN

$(P_d)_{y-y}$ axis :-

λ_{yy}	f_{cd} (for f_y = 250 Mpa)
100	107
106.67	?
110	94.6

$$f_{cd} = \frac{106.67-110}{100-110} \times (107-94.6)+94.6$$

$= 98.7292$ Mpa

Factored axial load $(P_d)_{YY} = A_e f_{cd}$

$= 19243 \times 98.73$

$= 1899.86$ kN

∴ Factored axial load that the column can carry = minimum of $(P_d)_{zz}$ & $(P_d)_{YY}$.

$P_d = 1899.86$ kN.

working load capacity of column $= \frac{P_d}{1.5}$

$= \frac{1899.86}{1.5}$

$= 1266.57$ kN

Design of axially Loaded compression member

Design steps:-

(1)(i) Assume slenderness ratio & determine design compressive stress (f_{cd}) considering grade of steel & assuming buckling class.

we can ~~consider~~ assume slenderness ratio as:

-> 70 to 90 for rolled steel beam

-> 110 to 130 for angle strut

-> 40 for member carrying large load.

(ii) Alternatively the design stress in compression (f_{cd}), we can ~~consider~~ assume directly as:-

-> 130 Mpa to 140 Mpa for rolled steel I section

-> 80 to 100 Mpa for angle strut & channel.

-> 190 to 200 Mpa for heavy / builtup section.

(2) calculate effective sectional area required.

$$(A_e)_{req} = \frac{\text{applied factored axial load}}{\text{compressive stress}}$$

$$(A_e)_{req} = \frac{P_d}{f_{cd}}.$$

Choose a trial section from steel table whose area should be greater than area required. Write down the section properties of that section from steel table & find rmin. of the trial section.

(3) find effective length & maximum slenderness ratio $\lambda max = \frac{le}{rmin}$ considering end conditions & type of connection.

(4) Then find out buckling class of the provided section

(5) Determine design compressive stress (fcd) considering grade of steel & actual buckling class. Compute design compressive strength (Pd) of member.

$$Pd = Ae\, fcd.$$

(6) check $P < Pd$., if ~~Pd~~ [illegible] $P > Pd$, then redesign by choosing another section.

(7) The section may be checked for limiting thickness also.

Q. Design a column section (using channel section only) to carry a factored axial load of 400kN. The column is 4m long & is effectively held in position at both ends but restrained against rotation at one end only. Consider fy = 250Mpa. Assume wind/earthquake actions.

(1) let us assume permissible design compressive stress (f_{cd}) = 80Mpa (for channel section).

$$(A_e)_{reqd} = \frac{P_d}{f_{cd}} = \frac{400\times10^3 N}{80} = 5000 mm^2$$

(2) choose a section from steel table whose area should be greater than Area required.

SO Try ISMC 350 @ 413 N/m, having $A = 5366 mm^2$ (from steel table).

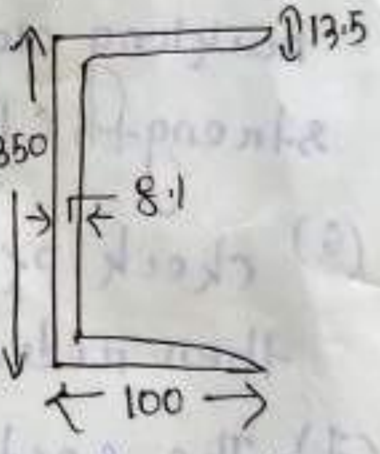
$h = 350mm$ $t_f = 13.5mm$

$b_f = 100mm$ $t_w = 8.1mm$

$r_{zz} = 136.6mm$, $r_{yy} = 28.3mm$.

$r_{min} = 28.3 mm$

root of radius $r_1 = 14mm$



(3) L = 4m.

for one end fixed & other end pinned:

k = 0.8 (table 11).

$l_e = kL = 0.8 \times 4 \times 1000$

$= 3200mm$.

$\lambda_{max} = \frac{kL}{r_{min}} = \frac{3200}{28.3} = 113.07 < 250$ → for wind/earthquake condition (from table 3).

(4) The buckling class is 'c' for channel section. We can get value of f_{cd} for ISMC 350 section by interpolating from table 9(c):

λ	f_{cd} (for f_y = 250Mpa)
110	94.6
113.07	?
120	83.7

$f_{cd} = \frac{113.07-120}{110-120} \times (94.6-83.7)+83.7$.

$= 91.254$ Mpa

(5) Design compressive strength (Pd):-

$P_d = A_e . f_{cd}$

$= 5366 \times 91.254$

$= 489.67$ kN $>$ 400 kN(P) (ok)

hence safe.

(c) Check for limiting thickness:-

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

$d_w = h - 2(t_f + r_1)$

$= 350 - 2(13.5+14) = 295$ mm.

$b = \frac{b_f}{2} \approx 100$ (for channel section).

$\frac{b}{t_f} = \frac{100}{\cancel{13.5}\ 8.1} = \cancel{7.41}\ 12.34 < 15.7\varepsilon = 15.7$, $\frac{d}{t_w} = \frac{295}{8.1} = 36.42 < 42\varepsilon = 42$

$\frac{b}{t_f} < 15.7$, $\frac{d}{t_w} < 42$ (ok)
(table 2)

Tubular Steel Structure.

Tubular steel structures are used in truss members, scaffolding of building, stadium, exhibition halls, transmission towers.

codes Required:- IS 1161.1998, IS 806.1968.

Advantages:-

(1) These have small self weights. Also because of direct connections, gusset plates are eliminated further reducing dead loads.

(2) Torsional strength of these structure is more than any other rolled section.

(3) For the same load, the surface area of a tube is about 60 to 70% of that for other rolled sections. Because of less area economy is achieved in maintenance, painting & fire proofing.

(4) Due to smooth finished surface, dirt & moisture do not collect over the surface, reducing the possibility of corrosion.

(5) ~~Due to~~ The change in load with the floor levels can be accomodated by varying the tube thickness & the external tube dimension may be maintained.

(6) The internal hollow space of tubular columns may be used for carrying drain pipes, wires cable etc. Also these spaces may

be filled with concrete to increase the load carrying capacity & to improve fire resistance.

Disadvantage:

(1) They pose difficulty in connection among themselves or to any plate element due to their shape problems.

(2) Bolting & riveting on those sections are not convenient.

(3) Their light weight some times become responsible for the structural instability.

(4) Highly skilled man power & special welding techniques are required for their connection.

Designation of steel tubes:

Steel tubes are designated by their nominal bore size.

classification of steel tube →
- → based on wall thickness
  - light
  - medium
  - heavy.
- → based on yield stress
  - YSt 22 (YSt 210) ↳ MPa
  - YSt 25, YSt 240 ↳ MPa
  - YSt 32. YSt 310 ↳ MPa

Ys → Yield stress
t → tube.

22, 25, 32 are in kgf/mm²,

Note

(i)* 1Mpa = $1 N/mm^2$ = 0.102 kgf/mm^2

Tensile properties of steel tubes for Structural purposes :-

(table-2, clause-11.2.1., IS code 1161).

Grade	tensile strength (Min)	Yield stress (Min).
Yst 210	330	210
Yst 240	410	240
Yst 310	450	310.

* The standard sizes of tubular sections, their mass/weight, relevant geometrical properties are given in table 1 as per IS:1161:1998.

Behaviour of tubular Sections :

(1) compression member :-

depends upon slenderness ratio.

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

$L_{effective}$ → depends of end conditions.
= kL (table-7, cl: 6.4, IS 806

r = we will get from section properties given is IS 1161:1998.

maximum limit of slenderness ratio:

type of member.	$\lambda = \frac{l}{r}$
(a) carrying loads resulting from dead load & super-imposed load.	180
(b) carrying loads resulting from wind or seismic forces only provided the deformation of such members does not adversely, affect the stress in any part of the structure.	250
(c) Normally acting as a tie in a roof truss but subject to possible reversal or stress resulting from the action of wind	350.

Axial stress in compression: The direct stress in compression on the cross-sectional area of axially loaded steel tubes shall not exceed the value of Fc given in table-2) of IS 806:1968.

Permissible axial stress in compression → (fc).
(table 2, clause 5.2)
(IS 806)

the cross-sectional area of axially loaded steel tubes shall not exceed the values of F_c given in Table 2 in which l/r is equal to the effective length of the member divided by the radius of gyration.

---

*Specification for steel tubes for structural purposes (*revised*) (*Second revision* in 1968).

†Specification for covered electrodes for metal arc welding of mild steel (*revised*) (Third revision in 1970).

‡Code of practice for structural safety of buildings: Loading standards (*revised*).

TABLE 2 PERMISSIBLE AXIAL STRESS IN COMPRESSION

(*Clause* 5.2)

l/r	F_c		
	GRADE YSt 22 kgf/cm²	GRADE YSt 25 kgf/cm²	GRADE YSt 32 kgf/cm²
(1)	(2)	(3)	(4)
0	1 250	1 500	1 900
10	1 217	1 448	1 821
20	1 175	1 400	1 760
30	1 131	1 352	1 679
40	1 088	1 303	1 610
50	1 046	1 256	1 539
60	1 002	1 207	1 465
70	970	1 155	1 375
80	929	1 088	1 283
90	876	1 003	1 128
100	814	910	989
110	745	813	869
120	674	721	758
130	603	638	665
140	540	565	564
150	490	503	517
160	432	443	450
170	381	392	396
180	339	348	352
190	304	311	316
200	271	278	280
210	243	249	250
220	219	225	227
230	198	204	203
240	180	185	187
250	162	167	167
300	106	106	106
350	71	71	72

NOTE 1 — Intermediate values may be obtained by linear interpolation.

NOTE 2 — The formula, from which these values have been derived, is given in Appendix A.

5.3 Bending Stresses — In tubes, the tensile bending stress and the compressive bending stress in the extreme fibres shall not exceed the values of F_b given in Table 3.

TABLE 3 PERMISSIBLE BENDING STRESS IN EXTREME FIBRES IN

6.4 Compression Members

6.4.1 *Effective Length of Compression Members* — Effective length (l) of a compression member for the purpose of determining allowable axial stresses shall be assumed in accordance with Table 7, where L is the actual length of the strut, measured between the centres of lateral supports. In the case of a compression member provided with a cap or base, the point of lateral support at the end shall be assumed to be in the plane of the top of the cap or bottom of the base.

TABLE 7 EFFECTIVE LENGTH OF COMPRESSION MEMBERS

TYPE	EFFECTIVE LENGTH
Effectively held in position and restrained in direction at both ends	0·67 L
Effectively held in position at both ends and restrained in direction at one end	0·85 L
Effectively held in position at both ends but not restrained in direction	L
Effectively held in position and restrained in direction at one end, and at the other end effectively restrained in direction but not held in position	L
Effectively held in position and restrained in direction at one end, and at the other end partially restrained in direction but not held in position	1·5 L
Effectively held in position and restrained in direction at one end but not held in position or restrained in direction at the other end	2·0 L

Table 1 Sizes and Properties of Steel Tubes for Structural Purposes

(*Clauses* 3.1, 6.1, 6.1.1 *and* 6.1.2)

Nominal Bore	Outside Diameter	Class	Thickness	Weight	Area of Cross Section	Internal Volume	Surface		Moment of Inertia	Modulus of Section	Radius of Gyration	Square of Radius of Gyration
							External	Internal				
mm	mm		mm	kg/m	cm^2	cm^3/m	cm^2/m	cm^2/m	cm^4	cm^3	cm	cm^2
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
15	21.3	Light	2.0	0.947	1.21	235	669	543	0.57	0.54	0.69	0.47
		Medium	2.6	1.21	1.53	203		506	0.69	0.64	0.66	0.44
		Heavy	3.2	1.44	1.82	174		468	0.75	0.70	0.55	0.42
20	26.9	Light	2.3	1.38	1.78	390	845	700	1.36	1.01	0.87	0.76
		Medium	2.6	1.56	1.98	370		681	1.48	1.10	0.86	0.74
		Heavy	3.2	1.87	2.38	330		644	1.70	1.26	0.84	0.71
25	33.7	Light	2.6	1.98	2.54	638	1 059	895	3.09	1.83	1.10	1.21
		Medium	3.2	2.41	3.06	585		857	3.61	2.14	1.08	1.17
		Heavy	4.0	2.93	3.73	518		807	4.19	2.48	1.05	1.11
32	42.4	Light	2.6	2.54	3.25	1 086	1 332	1 168	6.47	3.05	1.41	1.98
		Medium	3,2	3.10	3.94	1 017		1 130	7.62	3.59	1.39	1.93
		Heavy	4.0	3.79	4.82	929		1 080	8.99	4.24	1.36	1.86
40	48.3	Light	2.9	3.23	4.13	1 418	1 517	1 335	10.70	4.43	1.61	2.59
		Medium	3.2	3.56	4.53	1 378		1 316	11.59	4.80	1.59	2.54
		Heavy	4.0	4.37	5.56	1 275		1 265	13.77	5.70	1.57	2.47
50	60.3	Light	2.9	4.08	5.23	2 332		1 711	21.59	7.16	2.03	4.13
		Medium	3.6	5.03	6.41	2 213		1 667	25.88	8.58	2.00	4.02
		Heavy	4.5	6.19	7.88	2 066		1 611	30.90	10.2	1.98	3.92
65	76.1	Light	3.2	5.71	7.32	3 814	2 391	2 189	48.79	12.82	2.58	6.66
		Medium	3.6	6.42	8.20	3 727		2 163	54.02	14.20	2.57	6.60
		Heavy	4.5	7.93	10.1	3 534		2 107	63.12	17.1	2.54	6.43
80	88.9	Light	3.2	6.72	8.61	5 343	2 793	2 591	79.23	17.82	3.03	9.19
		Medium	4.0	8.36	10.7	5 138		2 540	96.36	21.68	3.00	9.00
		Heavy	4.8	9.90	12.7	4 936		2 490	112.52	25.31	2.98	8.88
90	101.6	Light	3.6	8.70	11.1	6 993	3 192	2 964	133.27	26.23	3.47	12.03
		Medium	4.0	9.63	12.3	6 877		2 939	146.32	28.80	3.45	11.91
		Heavy	4.8	11.5	14.6	6 644		2 889	171.44	33.75	3.43	11.76
100	114.3	Light	3.6	9.75	12.5	9 004	3 591	3 363	192.03	33.60	3.92	15.36
		Medium	4.5	12.2	15.5	8 704		3 306	234.3	41.0	3.89	15.10
		Heavy	5.4	14.5	18.5	8 409		3 250	274.5	48.0	3.85	14.86
110	127.0	Light	4.5	13.6	17.3	10 930	3 990	3 705	325.3	51.2	4.33	18.78
		Medium	4.8	14.5	18.4	10 819		3 686	344.58	54.27	4.32	18.69
		Heavy	5.4	16.2	20.0	10 599		3 649	382.0	60.2	4.30	18.32
125	139.7	Light	4.5	15.0	19.1	13 410	4 389	4 104	437.2	62.6	4.78	22.87
		Medium	4.8	15.9	20.3	13 287		4 085	463.44	66.35	4.77	22.76
		Heavy	5.4	17.9	22.8	13 063		4 047	514.5	73.7	4.75	22.58
135	152.4	Light	4.5	16.4	20.9	16 142	4 788	4 503	572.2	75.1	5.23	27.37
		Medium	4.8	17.5	22.2	16 008		4 484	606.92	79.65	5.22	27.25
		Heavy	5.4	19.6	25.0	15 740		4 446	674.3	88.5	5.20	27.03
150	165.1	Light	4.5	17.8	22.7	19 128	5 187	4 902	732.6	88.7	5.68	32.27
		Medium	4.8	18.9	24.2	18 981		4 883	777.32	94.16	5.67	32.14
		Heavy	5.4	21.3	27.1	18 690		4 843	864.7	103.0	5.65	31.92
150	168.3	Light	4.5	18.2	23.1	19 921	5 287	5 002	777.2	92.4	5.79	33.56
		Medium	4.8	19.4	24.7	19 771		4 983	824.78	98.01	5.78	33.42
		Heavy 1	5.4	21.7	27.6	19 473		4 946	917.7	109.0	5.76	33.21
		Heavy 2	6.3	25.2	32.0	19 030		4 889	1 053	125.0	5.73	32.85
175	193.7	Light	4.8	22.4	28.5	26 606	6 085	5 781	1 271.71	131.31	6.68	44.63
		Medium	5.4	25.1	32.0	26 260		5 743	1 417	146	6.66	44.36
		Heavy	5.9	27.3	34.8	25 974		5 712	1 535.2	158.65	6.64	41.11
200	219.1	Light	4.8	25.4	32.3	34 454	6 883	6 578	1 856.51	169.47	7.58	57.45
		Medium	5.6	29.5	37.5	33 930		6 528	2 141	195	7.55	57.02
		Heavy	5.9	31.0	39.5	33 734		6 509	2 247	205	7.54	56.86
225	244.5	Heavy	5.9	34.7	44.2	42 507	7 681	7 307	3 149	258	8.44	71.21
250	273.0	Heavy	5.9	38.9	49.5	53 557	8 578	8 202	4 412	323	9.45	89.30
300	323.9	Heavy	6.3	49.3	62.8	76 073	10 177	9 775	7 992	493	11.2	125.44
350	355.6	Heavy	8.0	68.6	87.3	90 533	11 173	10 663	13 111	737	12.3	151.29

Q-1 A tubular steel column of 4.8m Length is hinged at both ends. It has nominal dia of 225mm & of Yst 25 grade. Determine the safe load carrying capacity of the column.

solution:-

step-1 Given data & section properties:-

thickness = 5.9mm, weight = 34.7 kg/m.

Area of cross section = 44.2 cm^2

radius of gyration (r) = 8.44 cm.

nominal dia = 225mm. Yst = 25

outside dia = 244.5mm.

length of column (L) = 4.8m.

as both the ends are hinged, effective length (l_e) = KL

= 1×L = 4.8m.

(table-7, IS806).

Step(2)

calculation of slenderness ratio:-

$\lambda = \frac{l_{eff}}{r}$, $\lambda = \frac{4.8 \times 10^3}{8.44 \times 10} = 56.87 < 180$ (cl-6.4.2) (IS 806) (ok)

Step-(3) permissible stress in compression.

(for Yst = 25) f_c (kgf/cm^2) (from table-2) (IS 806)

$\lambda_1 = 50 \longrightarrow 1255$

$\lambda = 56.87 \longrightarrow ?$

$\lambda_2 = 60 \longrightarrow 1207$

by interpolating we will get value of permissible compressive stress.

$$\frac{56.87-60}{50-60} \times (1255-1207)+1207$$

$\Rightarrow f_c = 1222.024\ kg/cm^2$ ($1kgf = 9.81N \approx 10N$)

$= \cancel{122.2024\ kN/cm^2}$

$12220.24\ N/cm^2$

$1cm^2 = 10mm$
$100mm$

Safe load carrying capacity :-

$A \times f_c$

$P = 44.2 \times \cancel{122.2024}\ 12220.24$

$= \cancel{5401.35\ N}$

$= 540{;}134\ N \cong 540.134\ kN.$

Assignment :-

Q.2 A tubular column consists of IS1161 grad YSt 32 steel. The column is hinged in both the ends. The outside diameter of tube is 219.1mm. The weight of 1m length of tube is 310N. The length of column is 4.5m. Determine the safe load carrying capacity of column.

Minimum thickness of material:

For tubes painted with one prime coat of red oxides then periodically painted, the thickness should not be less than:-

(i) for construction exposed to weather → ~~3.2mm~~ 4mm

(ii) for construction not exposed to weather → 3.2 mm.

(iii) for members not readily accessible for maintenance = 5mm.

For tubes painted with one coat of zinc primer followed by two coats of paint, the thickness should not be less than:-

(i) for construction exposed to weather = 3.2 mm

(ii) for construction not exposed to weather = 2.6mm.

Permissible axial stress in tension (table-1, IS 806)

Grade	Ft (kgf/cm²)
Yst 22	1250
Yst 25	1500
Yst 32	1900

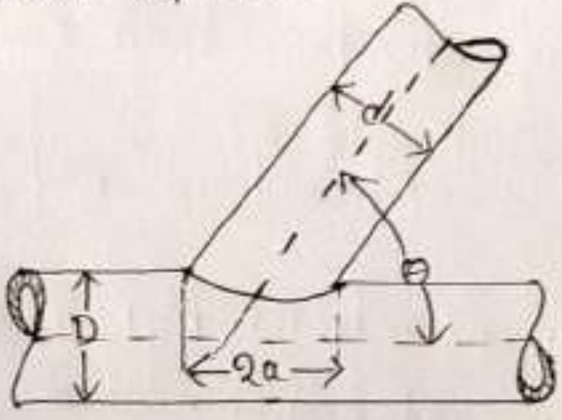
permissible stresses as per IS 806 (table-3, 4, 5)

Grade	Permissible bending stress (fb) (kgf/cm²)	permissible maxm shear stress (fs)	permissible maximum bearing stress (fp)
Yst 22	1400	900	1700
Yst 25	1655	1100	1900
Yst 32	2050	1350	2500

Connection:- cl-5.7.2, appendix B, IS806)

Determination of the Length of the curve of intersection of a tube with another tube or with a flat plate :-

The Length of the curve of intersection may be taken as :-



$$P = a+b+3\sqrt{a^2+b^2}$$

$$a = \frac{d}{2}\operatorname{cosec}\theta$$

$$b = \frac{d}{3}\times\frac{3-(d/D)^2}{2-(d/D)^2}$$

↳ for intersection with a tube.

$= \frac{d}{2}$ for intersection with a flat plate.

d = outside diameter of branch tube

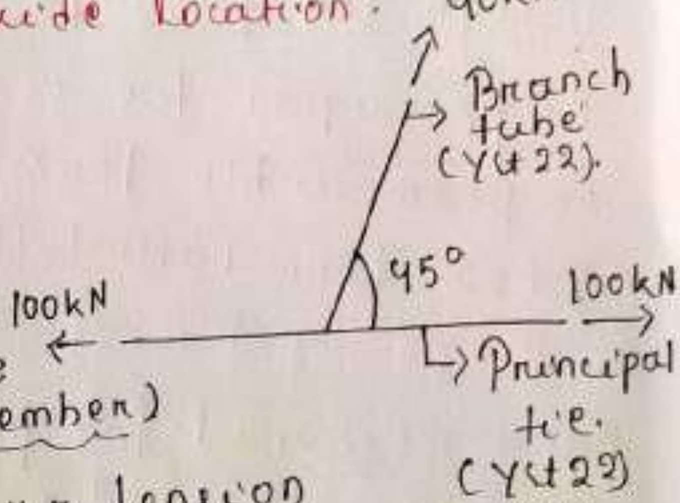
θ = angle between branch & main tube.

D = outside diameter of main tube.

Permissible stresses in welds :- (cl :- 5.7).

* for butt weld, the allowable tensile, compressive & shear stresses shall not exceed the stresses respectively permissible in the parent metal, whichever Yst 25 tubes or in the parent metal, whichever is less.
* In a fillet weld or in a fillet butt weld, the permissible stress shall not exceed the permissible in Yst 25 tubes the shear stress permissible metal, whichever is less. or in the parent

Q A tension member carrying a force of 40kN meets the principal tie of tubular truss at an angle of 45°. If the force in principal tie is 100kN, Design the members & welded joint between the two tube. Use YSt 22 grade of steel. Assume these member are placed around inside location.



Design of Branch tube (40kN member)

As the branch tube is a tension member, so permissible tensile stress on main tube. for YSt 22 (f_t) = 1250kgf/cm² (table-1 IS806)
= 125 N/mm²

→ Load coming on the branch tube (P) = 40kN = 40×10^3N.

→ Area of the branch tube = $\frac{P}{f_t} = \frac{40 \times 10^3}{125}$
= 320mm²

let choose cross-sectional area = 373 mm²

So let us provide steel tube of nominal bore size 25mm (class heavy) & outside diameter of 33.7mm & area of cross section 373mm².

check the minimum thickness from durability consideration ->

we provide thickness of member = 4mm > 3.2mm.

(as this member is located inside, minimum thickness should be 3.2 mm as per cl 6.3.1).

Design for principal tie (100kN tie member)

As principal tie is a tension member so permissible tensile stress (ft) for Yst 22 from table-1, IS 806 = 1250 kgf/cm²

= 125 N/mm²

Area required for principal tie :-

$$= \frac{\text{load on principal tie}}{f_t}$$

$$= \frac{100\times10^3}{125} = 800\text{mm}^2$$

Let provided area = 820mm² (choose from table-1, IS1161).

So let us provide a steel tube of nominal bore of 65mm (medium class) & outside dia of 76.1 mm & provide thickness of 3.6 mm.

check for thickness.

provided thickness (3.6mm) > 3.2mm.

↳ minimum thickness required as per cl 6.3.1 for durability consideration.

so it is ok.

Design of connection :- (referen appendix B of IS 806).

Length of connection =

$$p = a + b + 3\sqrt{a^2 + b^2}$$

$a = \frac{d}{2} \operatorname{cosec} \theta$

d = outside dia of branch tube = 33.7 mm.

$= \frac{33.7}{2} \operatorname{cosec} 45$

$= 23.83$

D = outside dia of principal tube = 76.1 mm.

$$b = \frac{d}{3} \times \frac{3 - (d/D)^2}{2 - (d/D)^2}$$

$$\Rightarrow \frac{33.7}{3} \times \frac{3 - \left(\frac{33.7}{76.1}\right)^2}{2 - \left(\frac{33.7}{76.1}\right)^2} = 17.46$$

$$p = a + b + 3\sqrt{a^2 + b^2}$$

$$= 23.83 + 17.46 + 3\sqrt{(23.83)^2 + (17.46)^2}$$

$$= 129.91 \text{ mm}$$

let us assume fillet weld :-

permissible shear stress in weld :-

permissible shear stress in

(i) permissible shear stress in parent material (Yst 22) = 90 N/mm² (table-4)

(ii) for Yst 25, permissible shear stress = 110 N/mm²

} minimum will be taken.

so permissible shear stress in weld :- 90 N/mm²

Required area of weld = $\frac{40 \times 10^3}{90} = 444.44 \text{ mm}^2$

Area of weld = effective throat thickness × length of weld

$444.44 = t_e \times 129.91$

$\Rightarrow t_e = 3.42$ mm.

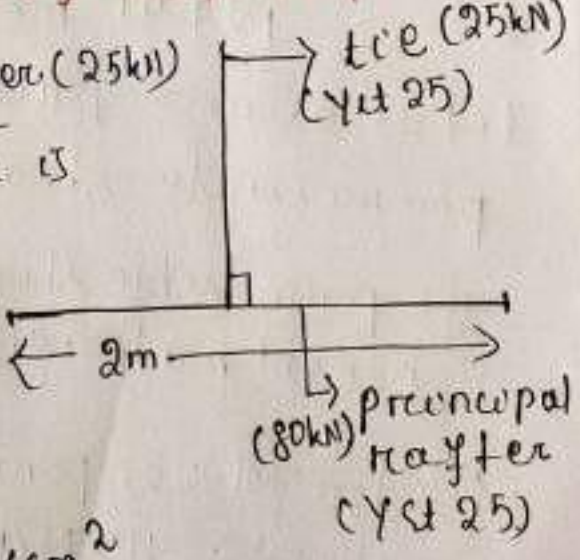
effective throat thickness (t_e) = 0.7 × size of weld

$3.42 = 0.7 \times S$

$\Rightarrow S = \frac{3.42}{0.7} = 4.88$ mm.

So provide size of weld = 4.88 mm.

Q. The principal rafter (compression member) in round tubular truss carries maximum force of 80kN. A tie member meeting at a joint at right angle to carry a force of 25 kN. If the panel length of principal rafter is 2m. Design the member as well as welded joint. Assume outside location & Yst 25 grade of material.



Design of branch tie member (25kN)

As the branch tie member is tension member, so permissible tensile stress for Yst 25 from table 1 (IS 806) :-

$f_t = 1500$ kgf/cm^2
$= 150$ N/mm^2

Area required for branch tie :-

$$= \frac{\text{Load on branch tie}}{f_t}$$

$$= \frac{25\times10^3}{150} = 166.67\ mm^2.$$

So provide a section of 373 mm². (class heavy) of nominal bore size 25mm & of outside diameter of 33.7 mm & thickness 4mm to satisfy durability requirements.

◎ <u>Design of principal rafter.</u>

let assume slenderness ratio $\lambda = 100$

so for $\lambda=100$, the safe compressive stress (f_c) = 91N/mm²

(from table-2)

$$\text{Required area} = \frac{\text{load on principal rafter}}{f_c}$$

$$= \frac{80\times10^3}{91} = 879.12\ mm^2$$

letus provide a steel tube of nominal bore of 76.1mm of heavy class having cross section area 1010 mm², outside dia 76.1mm & the thickness 4.5 mm which is more than 4mm. as per 3.6.7.1. (hence ok).

as nothing is given about end support condition, let assume effective length of principal rafter (L_{eff}) = 0.85L.

Scanned with CamScanner

so, $L_e = 0.85 \times 2$
$= 1.7 m.$

radius of gyration of choosen section(r) = 2.54 cm.

slenderness ratio(λ) $= \frac{1.7 \times 100}{2.54}$
$= 66.92$

permissible stress according to λ = 66.92. (from table-2 IS 806).

$\lambda = \frac{l}{r}$	f_c (kgf/cm²) (for Yst 25)
60	1207
λ = 66.92 ⟶	?
70	1155

by interpolating,

$$f_c = \frac{66.92-70}{60-70} \times (1207-1155) + 1155$$
$$= 1171.016 \text{ kgf/cm}^2 = 117.1016 \text{ N/mm}^2$$

Load = $f_c \times A_{provided}$
~~= 1171.016 × 1010~~
$= 117.1016 \times 1010$
$= 118273 N = 118.273 kN > 80 kN$ (ok).

Design of connection:- (appendix B of IS 806).

Length of connection:-

$P = a + b + 3\sqrt{a^2+b^2}$

d = 33.7 mm

D = 76.1 mm.

$a = \frac{d}{2} \operatorname{cosec} \theta$
$= \frac{33.7}{2} \operatorname{cosec} 90°$
$= 16.85$

$$b = \frac{d}{3} \times \frac{3-(d/D)^2}{2-(d/D)^2}$$

$$= \cancel{\frac{76.1}{3}} \frac{33.7}{3} \times \frac{3-\left(\frac{33.7}{76.1}\right)^2}{2-\left(\frac{33.7}{76.1}\right)^2} = 17.46$$

Length of weld $P = a + b + 3\sqrt{a^2+b^2}$

$$P = 16.85 + 17.46 + 3\sqrt{(16.85)^2 + (17.46)^2}$$

$$= 107.104 \text{ mm}$$

Letus assume fillet weld:

permissible shear stress in weld:-

(i) permissible shear stress in parent material (Yst 25) = 110 N/mm^2 (table-4)

(ii) for Yst 25, permissible shear stress = 110 N/mm^2

} minimum will be taken.

So permissible shear stress in weld = 110 N/mm^2

Required area of weld = $\dfrac{\text{load on branch tube}}{\text{permissible shear stress in weld}}$

$$= \frac{25 \times 10^3}{110} = 227.27 \text{ mm}^2$$

Area of weld = effective throat thickness × Length of weld

$227.27 = te \times 107.14$

$\Rightarrow te = 2.12$ mm.

$te = 0.7 \times$ size of weld(s)

$\Rightarrow 2.12 = 0.7 \times s \Rightarrow s = 3.028$ mm.

So provide size of weld = 3.028mm

⊙ Tubular beam :-

limiting deflection of beam :-

The maximum deflection should not exceed 1/325 of the span for simply supported members. This requirements may be satisfied if the bending stress in compression or tension does not exceed $31500\frac{D}{L}$ kgf/cm², where 'D' is the outer diameter of the tube in cm & 'L' is the effective Length of beam in cm.

Q A medium steel tubular section of 200mm nominal diameter is simply supported as a flexural member over effective span of 4.5m. Determine the safe uniformly distributed super imposed load which can be placed over it? Assume Yst 25 grade of steel).

Section properties of 200mm nominal dia medium class steel tube :-

(from table-I)

thickness (t) = 5.6mm , γ = 29.5 kg/m.

= 289.395 N/m.

E = 2×10^5 Mpa.

Area of cross section (A) = 37.5cm²

= 3750 mm²

modulus of section (Z) = 195cm³

= 195000mm³.

moment of Inertia (I) = 2141 cm⁴

= 2141×10^4 mm⁴

$$b = \frac{d}{3} \times \frac{3-(d/D)^2}{2-(d/D)^2}$$

$$= \cancel{\frac{76.1}{3}}\ \frac{33.7}{3} \times \frac{3-\left(\frac{33.7}{76.1}\right)^2}{2-\left(\frac{33.7}{76.1}\right)^2} = 17.46$$

Length of weld $P = a+b+3\sqrt{a^2+b^2}$

$$P = 16.85+17.46+3\sqrt{(16.85)^2+(17.46)^2}$$

$$= 107.104 \text{ mm}$$

Let us assume fillet weld:

permissible shear stress in weld:-

(i) permissible shear stress in parent material (YSt 25) = 110 N/mm² (table-4)

(ii) for YSt 25, permissible shear stress = 110 N/mm²

} minimum will be taken.

So permissible shear stress in weld = 110 N/mm²

Required area of weld = $\dfrac{\text{load on branch tube}}{\text{permissible shear stress in weld}}$

$$= \frac{25\times10^3}{110} = 227.27 \text{ mm}^2$$

Area of weld = effective throat thickness × Length of weld

$227.27 = te \times 107.14$

$\Rightarrow te = 2.12$ mm.

$te = 0.7 \times$ size of weld(s)

$\Rightarrow 2.12 = 0.7 \times S \Rightarrow S = 3.028$ mm.

(i) load carrying capacity based on bending stress:-

permissible bending stress for Yst 25 (f_b) = 1655 kgf/cm^2

= 165.5 N/mm^2

allowable bending stress due to load applied (f_{ab}) = $\frac{M}{Z}$.

for simply supported beam, bending moment due to uniformly distributed load (M) = $\frac{w l_{eff}^2}{8}$ (l_{eff} given = 4.5m)

$= \frac{w(4.5)^2}{8} = 2.53w$ N m.

section modulus (Z) from section properties = 19500mm^3,

$f_{ab} = f_b$

$\frac{M}{Z} = f_b \Rightarrow \frac{2.53w \times 10^3}{195000} = 165.5$

$\Rightarrow w = 12755.928$ N/m.

$= \boxed{12.756 \text{ kN/m.}}$

(2) based on shear stress:-

for Yst 25, permissible maximum shear stress (f_s) = 110 N/mm^2

allowable shear stress due to applied load (f_{as}) = $2 \times \frac{\text{maximum shear force (V)}}{\text{Area of section}}$

maximum shear force (V) due to applied load = $\frac{w l_{eff}}{2} = \frac{w \times 4.5}{2} = 2.25w$ N (Let w is in N/m)

Area of section from section properties:-

$A = 3750 \text{ mm}^2$

Now, $f_{as} = f_s$

$\Rightarrow 2\times\frac{V}{A} = f_s \Rightarrow 2\times\frac{2.25w}{3750} = 110$

$\Rightarrow w = 91667 \text{ N/m}$

$= \boxed{91.667 \text{ kN/m.}}$

(B) Load carrying capacity in view of deflection:-

Maximum permissible deflection for simple supported beam δ_{max} =

$\frac{1}{325}\times l_{eff}$

$= \frac{1}{325}\times 4.5\times10^3$

$= 13.85$ mm.

(w is in N/mm)

Deflection due to applied load:-

$\delta = \frac{5}{384}\cdot\frac{w(l_{eff})^4}{EI}$

$= \frac{5}{384}\times\frac{w\times(4.5\times10^3)^4}{2\times10^5\times2141\times10^4\times10^3}$

$= 1246.93w$

Now $\delta = \delta_{max}$

$\Rightarrow 1.25w = 13.85 \Rightarrow w = 11.08 \text{ N/mm.}$

$= \boxed{11.08 \text{ kN/m.}}$

load carrying capacity = minimum of above 3 loads calulated

$w = 11.08$ kN/m.

self wt. of member (dead load) or = 289.395 N/m.
= 0.289 kN/m.

w = super imposed load (live load) + dead load.

=> super imposed load = w - dead load
= 11.08 - 0.289
= 10.79 kN/m.

Design Of Steel Beam:-

-> Flexural members or bending members are called beams.

-> A beam is a structural member subjected to transverse load i.e. load perpendicular to the longitudinal axis.

-> The load produces bending moment & shear forces.

Ex Joist, girder, purlin.

Nature of forces acting on beam:-

-> It is assumed that the beam is subjected to only transverse loading.

-> All the Loads & sections lie in the plane of symmetry. So It follows that such a beam will be primarily subjected accompanied by shear in the loading plane with no external torsion & axial force.

-> but the problem of torsion can not completed be avoided in a beam even if the beam shape is symmetrical & loads are in the plane of symmetry. The reason is the instability caused by compressive stresses, such instability is defined as lateral buckling, when it is involving only local components of a beam it is called Local buckling.

Modes of failure:-

(i) bending failure:- due to crushing of compression flange or fracture of tension flange.

(ii) Shear failure:- when shear stress value exceeds the limiting value. Thus occurs due to buckling of web of beam near location of high shear forces. The beam can fail locally due to crushing or buckling of web near the reaction of concentrated loads.

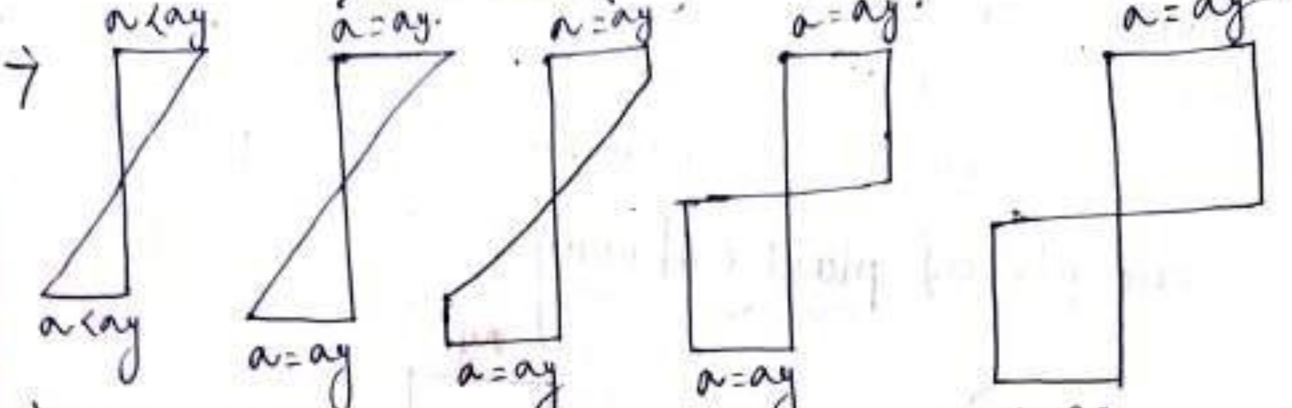
(iii) deflection failure:- A floor having large deflection not only produces a feeling of insecurity but also damages the non structural elements attached to it. Hence deflection in beam should be restricted by specifying deflection limitation imposed by code which is usually in terms of deflection ~~deflection~~ ~~depth~~ to span ratio

Sections available for beam member:-

→ Angles } weaken bending } built up sections → when stress exceeding limiting value.
→ T-section
→ channel } used for light load.
→ I section → most effective as a beam member.
(economical, bending strength is more as I is more $M = \frac{\sigma}{y} I$

Common cross sections & their classification:

Slender cross section | Semi-compact cross section | compact cross section | plastic cross-section.



→ $M < M_y$ $M = M_y$ $M_y < M < M_p$ $M = M_p$ $M = M_p$.

→ ~~elevents~~

M_y → yield moment

M_p → plastic moment

(i) plastic cross-section → plastic cross-sections are those which can develop plastic hinges & have the rotation Capacity required for failure. of structure by formation of plastic mechanism.

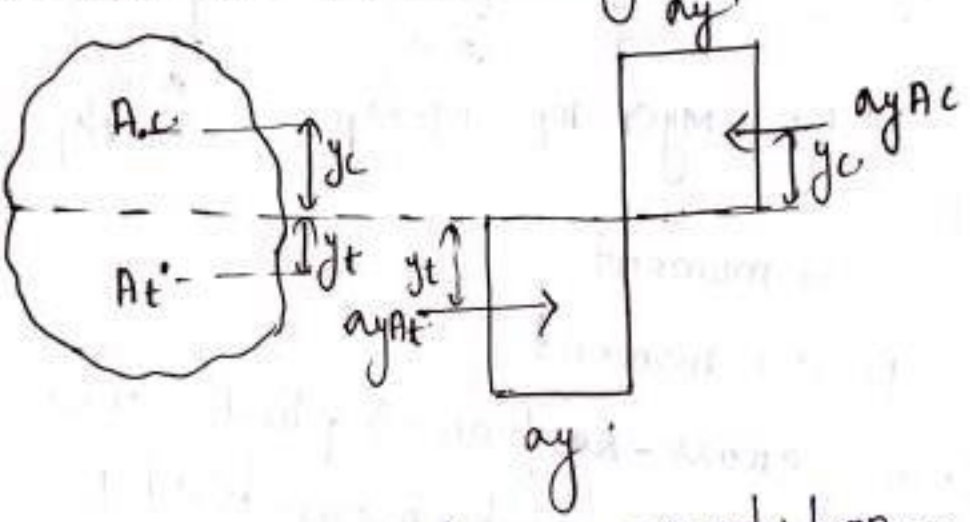
(2) compact section → compact cross-section are those which can develop plastic moment of resistance, but have inadequate plastic hinge rotation capacity for formation of plastic mechanism due to local buckling.

(3) semi compact cross-section → semi compact cross sections are those in which the stress in the extreme fibres in compression should be limited

to yield stress. These sections can not develop plastic moment of resistance due to local buckling.

Slender cross-section:- Slender cross-section are those in which the elements buckle locally even before reaching yield stress.

Concepts of plastic theory:-



Considering equilibrium condition:-

$$\Sigma H = 0$$

Total force in compression = Total force in tension.

$$\sigma_y A_c = \sigma_y A_t$$

$$\Rightarrow \boxed{A_c = A_t}$$

So the neutral axis that divides the cross-section into two equal haves is known as plastic neutral axis.

$$\boxed{A_t = A_c = \frac{A}{2}}$$

A_t = area in tension.

A_c = area in compression

A = total area.

Here,

plastic moment capacity (Mp) :-

$$(a_y A_c) y_c + (a_y A_t) y_t$$

$$\Rightarrow a_y (A_c y_c + A_t y_t)$$

$$\boxed{M_p = a_y Z_p}$$

Z_p = plastic section modulus.

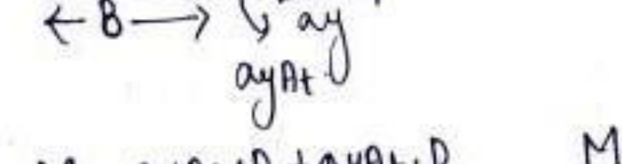
$= A_c y_c + A_t y_t$.

Shape factor. Shape factor of a cross section is defined as the ratio between plastic moment to yield moment.

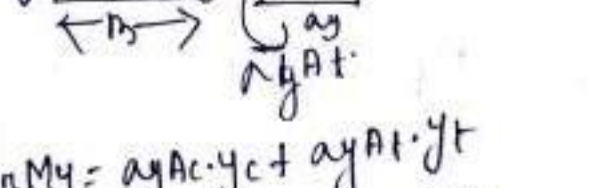
$$\boxed{\text{Shape factor} = \frac{M_p}{M_y} = \frac{a_y Z_p}{a_y Z_e} = \frac{Z_p}{Z_e}}$$

Shape factor of a rectangular section :-

plastic moment (Mp).



yielding moment (My)



$$M_p = a_y A_c \cdot \frac{D}{4} + a_y A_t \cdot \frac{D}{4}$$

$$= a_y B \cdot \frac{D}{2} \cdot \frac{D}{4} + a_y \cdot B \cdot \frac{D}{2} \cdot \frac{D}{4}$$

$$\boxed{M_p = a_y \frac{BD^2}{4}}$$

Here $Z_p = \frac{BD^2}{4}$

$$M_e \text{ or } M_y = a_y A_c \cdot y_c + a_y A_t \cdot y_t$$

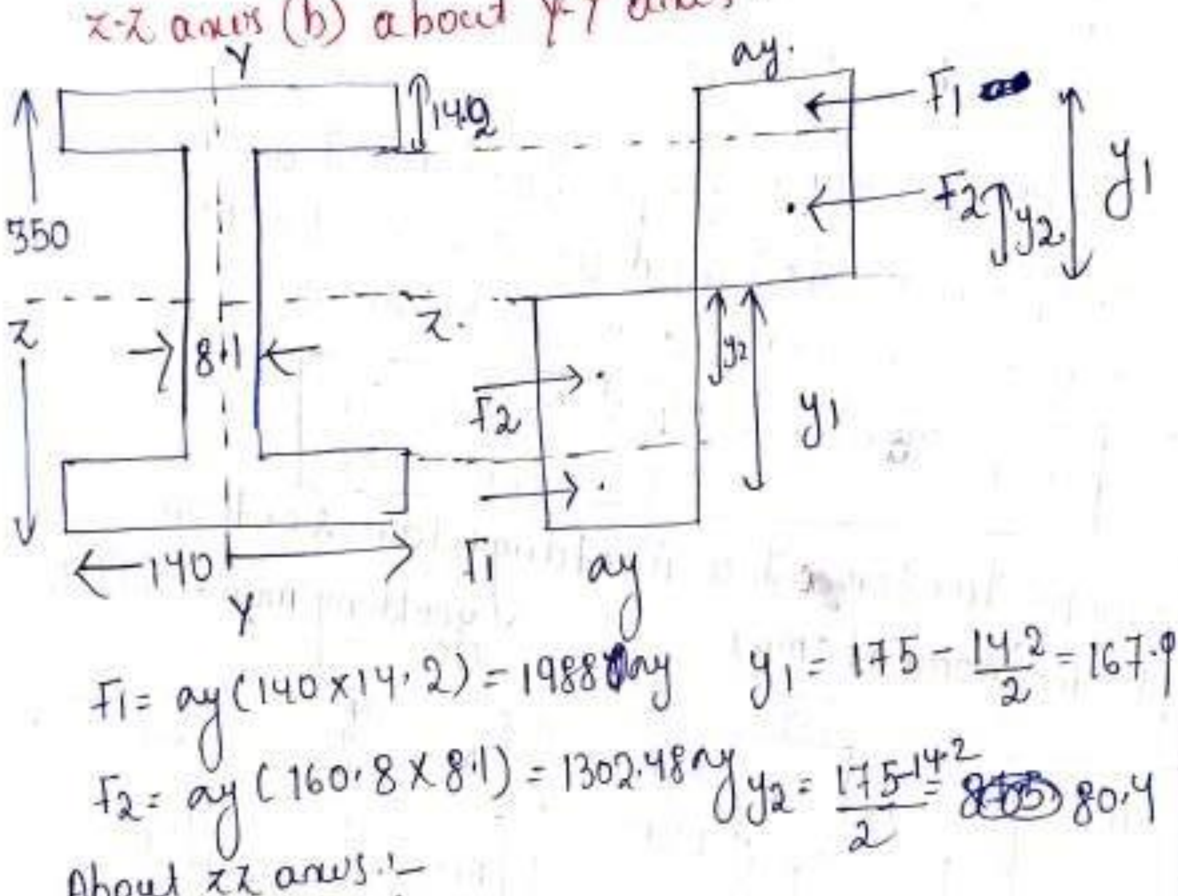
$$= \frac{a_y}{2} B \cdot \frac{D}{2} \cdot \frac{2}{3} \cdot \frac{D}{2} + \frac{a_y}{2} B \cdot \frac{D}{2} \cdot \frac{2}{3} \frac{D}{2}$$

$$\boxed{M_y = a_y \cdot \frac{BD^2}{6}}$$

Here $Z_e = \frac{BD^2}{6}$

Shape factor $= \dfrac{M_p}{M_y} = \dfrac{\sigma_y \cdot \frac{bd^2}{4}}{\sigma_y \frac{bd^2}{6}} = 1.5.$

Q. Determine the plastic moment capacity & plastic section modulus of a symmetrical I section having depth of section 350 mm, width of flange 140mm, thickness of flange 14.2mm & thickness of web 8.1mm (a) about z-z axis (b) about y-y axis.



$F_1 = \sigma_y(140 \times 14.2) = 1988\sigma_y$ $y_1 = 175 - \frac{14.2}{2} = 167.9$

$F_2 = \sigma_y(160.8 \times 8.1) = 1302.48\sigma_y$ $y_2 = \frac{175 - 14.2}{2} = 80.4$

About zz axis:-

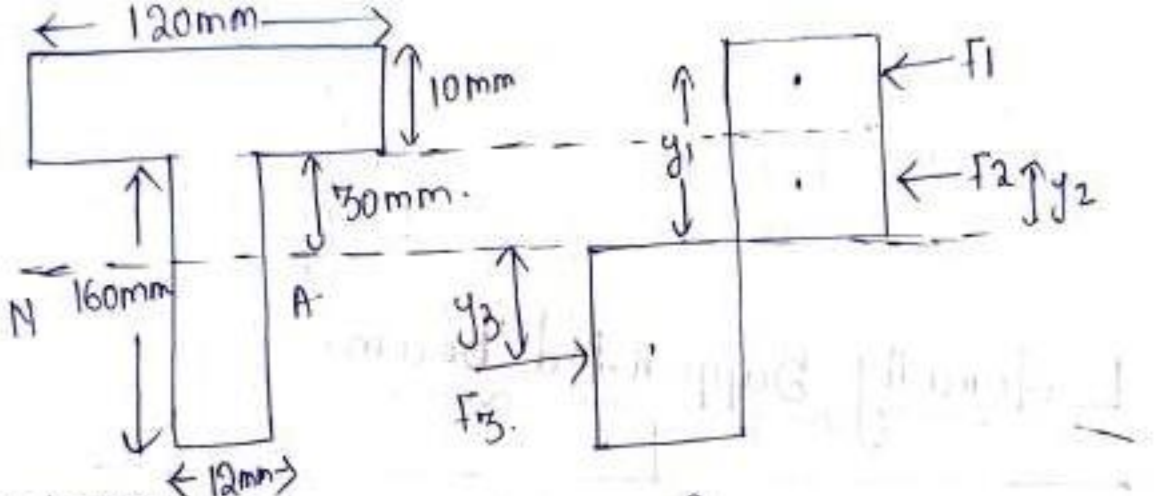
$$M_p = (F_1 y_1 + F_2 y_2)_{\text{from compression region}} + (F_1 y_1 + F_2 y_2)_{\text{from tension region}}$$

$$= (1988\sigma_y \times 167.9 + 1302.48\sigma_y \times 80.4) + (1988\sigma_y \times 167.9 + 1302.48\sigma_y \times 80.4).$$

$$= 877.009 \times 10^3 \times \sigma_y.$$

$$Zp = \frac{Mp}{\sigma y} = \frac{877.009\times10^3 \times \sigma y}{\sigma y} = 877.009\times10^3 mm^3$$

Q Determine the plastic section modulus of a 'T' section having flange width 12cm, flange thickness 1cm, depth of web 160mm & width of web 12mm.



Total area

$A = 120\times10 + 160\times12 = 3120mm^2$

Plastic N.A. divides the total section into two part of same area.

$\frac{A}{2} = \frac{3120}{2} = 1560\ mm^2$

$120\times10 + 12\times y = 1560$

$\Rightarrow \boxed{y = 30mm}$ → Location of plastic N.A. from the top of ~~flange~~ web.

$Mp = F_1 y_1 + F_2 y_2 + F_3 y_3$

$= \sigma y\times120\times10\times35 + \sigma y\times12\times30\times15 + \sigma y\times12\times130\times65$

$= 148.800\ \sigma y$

$Zp = \frac{Mp}{\sigma y} = \frac{148.800\sigma y}{\sigma y} = 148.800\ mm^3$

Beam (based on lateral supports to compression flange)

- laterally supported beam.
  - If compression flanges are laterally supported
    - subjected to bending & shear stress.
- Laterally unsupported beam.
  - If compression flange is not laterally supported.
    - Lateral buckling occurs.

Laterally Supported beam.

- low shear. $V \leq 0.6 V_d$ (page no 53)
- high shear. $V > 0.6 V_d$ (page No- 70)

V = factored design shear force due to external load

V_d = design shear strength of cross section (page No 59)

Design shear strength: (V_d) (Go to page No 59)

from clause No 8.4:-

$$\text{design shear strength } (V_d) = \frac{A_v f_{yw}}{\sqrt{3}\, \gamma_{m0}}$$

f_{yw} = yield strength of web

γ_{m0} = partial safety factor against shear failure (refer cl: 5.41)

= 1.1

A_v = Shear area.

For I & channel section:- $\underline{A_v}$ (for major axis bending).

Hot rolled section → $\left.\begin{matrix} ht_w \\ dt_w \end{matrix}\right\}$

welded → dt_w

Hot rolled or welded → $2b_ft_f$ (Minor axis bending).

circular hollow tube of uniform thickness → $2A/\pi$

plates & solid bars → A

A = cross section area.

h = overall depth of section

t_f = thickness of flange.

t_w = thickness of web.

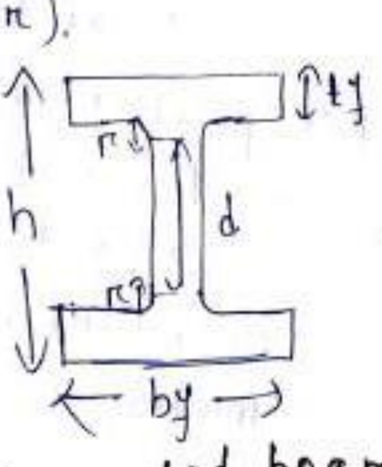
d = clear depth of web between flanges.

b_f = width of flange.

* For classification of section, go to page No 18 (table 2):-

Here $b = \frac{b_f}{2}$, $d = h - 2(t_f + r)$.

r = root of radius (from steel table we can get this value)



Design bending strength of laterally supported beam.

$$\boxed{\left(\frac{d}{t_w} \le 67\varepsilon\right). \quad \varepsilon = \sqrt{\frac{250}{f_y}}}$$

bending strength for low shear ($V \le 0.6 V_d$). (Page 53)

$M \le M_d$

M = factored bending moment due to applied load.

M_d = design bending strength.

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

$\le \frac{1.2 Z_e f_y}{\gamma_{m0}}$ (for simply supported)

$\le \frac{1.5 Z_e f_y}{\gamma_{m0}}$ (for cantilever beam)

where,

$\beta_b = 1$ for plastic & compact section

$= \frac{Z_e}{Z_p}$ for semicompact section.

Z_p, Z_e = plastic & elastic section modulus of cross section

f_y = yield stress of material.

$\gamma_{m0} = 1.1$

bending strength for high shear ($V > 0.6 V_d$). (page 70)

$M \le M_{dv}$.

M_{dv} = design moment capacity under high shear.

(a) for plastic & compact section) $M_{dv} = M_d - \beta(M_d - M_{fd}) \le \frac{1.2 Z_e f_y}{\gamma_{m0}}$

where,

$$\beta = \left(\frac{2V}{V_d} - 1\right)^2$$

V = factored applied shear force.

V_d = design shear strength

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

Z_e = elastic section modulus of whole section.

(b) for semicompact section:

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

Steps for analysis problem.

(1) write down the section properties of choosen section. ~~ISMB 600 @ 1200 [illegible]~~ given in question.

(2) classify the section whether it is compact section, plastic section, semicompact section, or slender section.

(3) Assume the section is low shear case. i.e. $V \leq 0.6 V_d$. for low shear case calculate the design bending strength (M_d).

(4) calculate the load by taking ⇒ $M = M_d$.

(5) calculate the maximum shear force (V). (by taking the load that is calculated from (bending strength).

(6) Then calculate design shear strength (V_d). then check whether the section is low shear case or high shear case. if it is low shear case, then our assumption is ok but if it is high shear case, then solve according to high shear case.

(7.) calculate Load carrying capacity from deflection criteria by taking

$$(\delta_{max})_{beam} = \frac{span}{240.}$$ (for simple supported beam) (from table - 06) code IS 800.

Q A laterally supported beam ISMB 600 @ 1202.71 N/m is placed between two supports. Determine the safe uniformly distributed load per meter length which can be placed over the beam for an effective span of 12m. Take $f_y = 250 N/mm^2$. Neglect web buckling & web crippling.

(1) Section properties of ISMB 600 @ 1202.71 N/m :- (from steel table).

plastic section modulus $Z_p = 3510630 mm^3$

depth of section h = 600 mm.

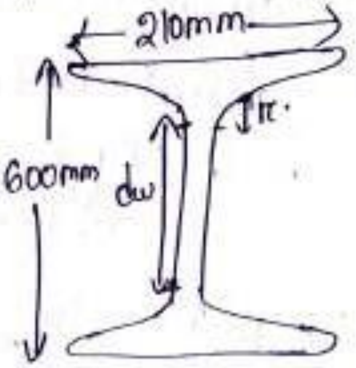
$b_f = 210mm$, $A = 15621 mm^2$, root of radius $(r_1) = 20 mm$.

$t_f = 20.8 mm$, $t_w = 12mm$.

depth of web $d_w = h - 2(t_f + r_1)$

$= 600 - 2(20.8 + 20)$

$= 518.4mm$



$I_{zz} = 91813 \times 10^4 mm^4$.

elastic section modulus $(Z_e) = 3060.4 \times 10^3 mm^3$.

(2) Section classification:-

$\varepsilon = \sqrt{\frac{250}{f_y}} = \sqrt{\frac{250}{250}} = 1$, $b = \frac{b_f}{2} = \frac{210}{2} = 105mm$.

for rolled section-

outstand of compression flange:-

$\frac{b}{t_f} = \frac{105}{20.8} = 5.05 < 9.4\varepsilon$. (table 2)

So flange is plastic section.

for web with N.A. at mid depth:- $\frac{d}{t_w} = \frac{518.4}{12}$

$= 43.2 < 84\varepsilon$. (table 2).

So web is plastic section.

Hence overall classification of section is plastic.

(3) $\frac{d}{t_w} = \frac{518.4}{12} = 43.2 < 67\varepsilon$.

Let assume it is a low shear case

i.e. $V \le 0.6 V_d$.

Design bending strength $M_d = \frac{\beta_b Z_p f_y}{\gamma_{m0}} \le \frac{1.2 Z_e f_y}{\gamma_{m0}}$ (for simply supported beam)

$\beta_b = 1$ (for plastic section).

$$M_d = \frac{1 \times 3510630 \times 250}{1.1}$$

$= 797.87 \times 10^6$ Nmm

$= 797.87$ kNm.

$$\frac{1.2 Z_e f_y}{\gamma_{m0}} = \frac{1.2 \times 3060.4 \times 10^3 \times 250}{1.1}$$

$= 834.65 \times 10^6$ Nmm

$= 834.65$ kN m.

So $M_d \le \frac{1.2 Z_e f_y}{\gamma_{m0}}$

(4) Load carrying capacity based on design bending strength.

$M = M_d$. (for simply supported beam maxm bending moment $= \frac{wl^2}{8}$

$$\frac{w \times 12^2}{8} = 797.87 \times 10^6$$

$\Rightarrow w = 44.326$ kN/m.

Safe uniformly distributed load $w' = \frac{w}{1.5}$

$= \frac{44.326}{1.5}$

Safe u.d.l. that the beam $w' = 29.551$ kN/m.

can carry $= w' -$ self wt. of beam

$= 29.551 - 1.202 = 28.349$ kN/m.

(5) Factored shear force $V = \frac{wl}{2}$

$(w = 1.5w')$

$$V = \frac{44.326 \times 12}{2}$$

$= 265.956$ kN. (for simply supported beam)

Design shear strength

$$(V_d) = \frac{A_v f_{yw}}{\sqrt{3}\,\gamma_{mo}}$$

(for rolled section, $A_v = h t_w$, $A_v = 600 \times 12$)

$$V_d = \frac{600 \times 12 \times 250}{\sqrt{3} \times 1.1}$$

$= 944.75$ kN

$0.6V_d = 0.6 \times 944.75$

$= 566.85$ kN.

$V < 0.6V_d$ (low shear case) so, our assumption is ok.

(6) Load carrying capacity from deflection criteria:

$$(\delta_{max})_{beam} \text{ for udl} = \frac{5wl^4}{384EI}$$

$$= \frac{5 \times w \times (12 \times 10^3)^4}{384 \times 2 \times 10^5 \times 91813 \times 10^4}$$

$= 1.47w$

$\delta_{max} = \frac{span}{240} \Rightarrow 1.47w = \frac{12 \times 10^3}{240} \Rightarrow w = 34.01$ kN/m

working load $w' = \frac{34.01}{1.5} = 22.67$ kN/m.

safe working load placed on beam =

$w' -$ self wt $= 22.67 - 1.207 = 21.47$ kN/m

Hence the safe working Load that the beam can carry is minimum of the two = 21.47 kN/m.

Design steps.

(1) (a) Determine the service Load on the beam & multiply with γ_f (1.5) to find the ultimate load or factored Load.

(b) Determine the effective span of the beam.

(c) Calculate the maximum bending moment M & maximum shear force V.

(2) Trial section & section classification:-

(a) Determine a trial plastic section using the formula:-

$$Z_{p(reqd)} = \frac{M\gamma_{m0}}{f_y}$$

M = maximum bending moment determined in step(1)(c).

γ_{m0} = partial factor of safety (1.1)

f_y = yield stress of material (250 Mpa).

(b) from steel table, choose a trial section having plastic section modulus more than that required.

(c) check the classification of section (see table-2, IS 800).

(3) check for bending & shear strength:-

(a) Determine shear strength of section (cl 8.4 of IS 800) & compare with maximum shear force V determined in step 1(c) & check (b) whether the ~~x~~ it is low shear case or high shear case.

(b) find design bending strength (depending on $V < 0.6V_d$ or $V > 0.6V_d$) as cl: 8.2.1.2 or cl: 9.2.2 of IS 800:2007.

check M (maximum bending moment) < Design bending strength

If not satisfied, repeat from step-2(b) or 3(b).

(4) check for deflection (cl: 5.6.1 & table-6). Calculate the maximum deflection in the beam considering effective span, loading & support condition. The maximum deflection shall be less than the permissible value given in table 6 of IS 800.

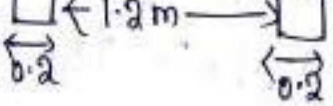
Q Design a simple supported beam of clear span 1.2m carrying a concentrated load of 260kN at mid span. Width of support is 200mm. Consider $f_y = 250 N/mm^2$.

(1) (a) assume self wt. of beam (w) = 1kN/m.

effective span = centre to centre of supports.

$$= 1.2 + 2 \times \frac{0.2}{2}$$

$$= 1.4 m.$$



Given point load (W) = 260kN.

~~self wt of beam = 1kN/m × 1.2m~~

~~= 1.2kN.~~

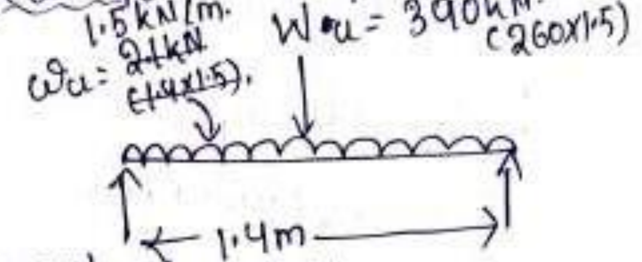
factored point Load (W_u) = 260×1.5

= 390 kN.

factored self wt (w_u) = ~~1.4×1.5~~ 1 kN/m × 1.5

= ~~2.1 kN~~ 1.5 kN/m.

(b) calculation of maximum bending moment (M) & maximum shear force (V):-



maximum bending moment at centre:-

$$(M) = \left(\frac{w_u l_{eff}^2}{8} + \frac{W l_{eff}}{4}\right)$$

$$M = \frac{1.5 \times 1.4^2}{8} + \frac{390 \times 1.4}{4}$$

$$= 136.87 \text{ kNm.}$$

$$= 136.87 \times 10^6 \text{ Nmm.}$$

maximum shear force at support (V) =

$$\left(\frac{W_u}{2} + \frac{w_u l_{eff}}{2}\right)$$

$$= \left(\frac{390}{2} + \frac{1.5 \times 1.4}{2}\right) = 196.05 \text{ kN}$$

$$= 196.05 \times 10^3 \text{ N.}$$

(2) Trial section & section classification:-

(a) $(Z_p)_{reqd} = \frac{M \gamma_{mo}}{f_y} = \frac{136.87 \times 1.1 \times 10^6}{250} = 602228 \text{ mm}^3$

(b) then choose a section from steel table having $Z_p > 602228 \text{ mm}^3$.

So try ISMB 350 @ 514 N/m having $Z_p = 889600 \text{ mm}^3$

Section propertees:

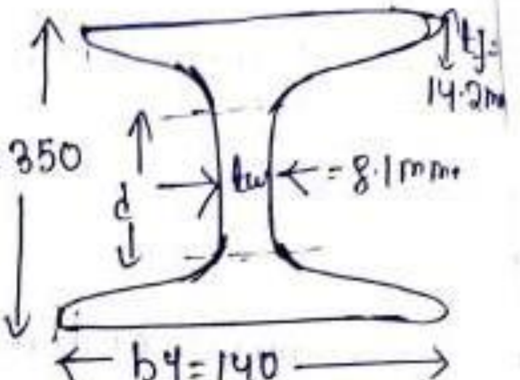
$h = 350 mm.$ $t_f = 14.2 mm$ $I_{zz} = 13630.3\times10^4 mm^4$ (a)

$b_f = 140 mm$ $t_w = 8.1 mm.$ $Z_p = 889600 mm^3$

$A = 6670 mm^2$ $Z_e = 778.9\times10^3 mm^3$. $r_1 = 14$

$\gamma = 0.514 kN/m.$

(c) Section classification:-



$$d = h - 2(t_f + r_1).$$
$$= 350 - 2(14.2 + 14)$$
$$= 293.6 mm.$$

$$b = \frac{b_f}{2} = \frac{140}{2} = 70 mm. \quad , \quad \epsilon = \sqrt{\frac{250}{f_y}} = \sqrt{\frac{250}{250}} = 1$$

for outstand of compression flange for rolled section, $\frac{b}{t_f} = \frac{70}{14.2} = 4.93 < 9.4\epsilon$ (plastic section).

Web of I-section with NA at mid depth:-

$$\frac{d}{t_w} = \frac{293.6}{8.1} = 36.25 < 84\epsilon$$

(plastic section)

Hence the section is classified as plastic section.

check for assumed self wt:-

wt. of section = 0.514 kN/m < 1 kN/m (assumed) (hence ok).

(3) check for bending & shear strength:-

Design shear strength of section:-

$$V_d = \frac{f_{yw} \times A_v}{\sqrt{3} \times \gamma_{mo}} \qquad A_v = h \times t_w$$
$$= 350 \times 8.1$$

$$= \frac{250 \times 350 \times 8.1}{\sqrt{3} \times 1.1}$$

$V_d = 371.997\,kN.$ $(V = 196.05 < V_d = 371.997)$

(hence safe).

$\Rightarrow 0.6 V_d = 0.6 \times 371.997$
$= 223.198\,kN.$

s $V = 196.05 < (0.6 V_d) = 223.198$ (low shear case).

(b) for low shear case $(V < 0.6 V_d)$:

design bending strength (M_d):-

$$M_d = \beta_b Z_p \frac{f_y}{\gamma_{mo}} < \frac{1.2 Z_e f_y}{\gamma_{mo}}$$ (for simply supported beam)

$$M_d = \frac{1 \times 889.6 \times 10^3 \times 250}{1.1}$$ ($\beta = 1$ (for plastic section)

$$< \frac{1.2 \times 778.9 \times 10^3 \times 250}{1.1}$$

$M_d = 202.18\,kNm < 212.43\,kNm$ (ok)

$M = 136.87 < M_d = 202.18$ (Hence the section is safe).

(4) check for deflection:-

permissible deflection for a beam in building assuming elastic cladding:- $\frac{l_e}{240}$

$$= \frac{1400}{240} = 5.83$$

maximum deflection corresponding to load:

$$\delta = \frac{5wl^4}{384EI} + \frac{Wl^3}{48EI}$$

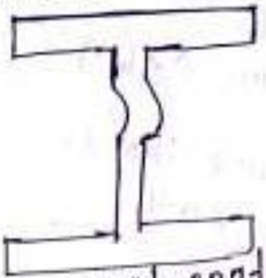
$$= \frac{5\times1.5\times10^3\times(1400)^4}{384\times2\times10^5\times13630.3\times10^4} + \frac{260\times10^3\times(1400)^3}{48\times2\times10^5\times13630.4\times10^4}$$

$$= 3.208 < \frac{Le}{240}$$ (so the section is safe).

Web crippling.

→ When a member is subjected to concentrated load, then this load is resisted by compressive stress of member. So stress concentration occur at the junction of web & flange.

→ Crippling occur near the stress concentration zone (local buckling occurs at perticular area).



→ so bearing strength should be greater than the concentrated load.

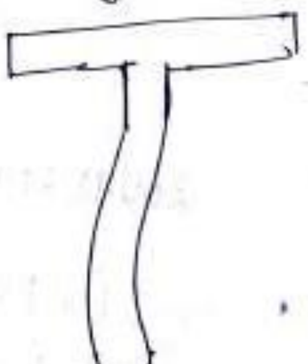
→ so to avoid web crippling bearing stiffner is provided or thickness of web is provided.

Web buckling.

→ When vertical compressive stress exceed critical buckling stress for web acting as column.

→ not occur in hot rolled beam section.

→ occurs in case of plate girder.



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

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: $V \leq 0.6 V_d$

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

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**).

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 $V > 0.6 V_d$

✓ $$M_d = M_{dv}$$

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_{ef} 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 forces required shall be taken as 2.5 percent of the maximum force in the compression flange plus 1.25 percent of this force for every member of the series other than the first, up to a maximum total of 7.5 percent.

8.3.5 Purlins adequately restrained by sheeting need not be normally checked for the restraining forces required by rafters, roof trusses or portal frames that carry predominately roof loads provided there is bracing of adequate stiffness in the plane of rafters or roof sheeting which is capable of acting as a stressed skin diaphragm.

8.3.6 In case of beams with double curvature bending, adequate direct lateral support to the compression flange in the hogging moment region may be provided as given above for simply supported beam. The effect of support to the tension (top) flange in the hogging moment region on lateral restraint to the compression flange may be considered as per specialist literature.

8.4 Shear

The factored design shear force, V, in a beam due to external actions shall satisfy

$$V \leq V_d$$

where

V_d = design strength

= V_n / γ_{m0}

where

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

The nominal shear strength of a cross-section, V_n may be governed by plastic shear resistance (*see* **8.4.1**) or strength of the web as governed by shear buckling (*see* **8.4.2**).

8.4.1 The nominal plastic shear resistance under pure shear is given by:

$$V_n = V_p$$

where

$$V_p = \frac{A_v f_{yw}}{\sqrt{3}}$$

V_d V_p γ_{m0}

A_v = shear area, and

f_{yw} = yield strength of the web.

8.4.1.1 The shear area may be calculated as given below:

I and channel sections:

Major Axis Bending:

Hot-Rolled — $h\,t_w$

Welded — $d\,t_w$

Minor Axis Bending:

Hot-Rolled or Welded — $2b_f t_f$ √y

Rectangular hollow sections of uniform thickness:

Loaded parallel to depth (h) — $A\,h\,/\,(b+h)$

Loaded parallel to width (b) — $A\,b\,/\,(b+h)$

Circular hollow tubes of uniform thickness — $2\,A\,/\,\pi$

Plates and solid bars — A

where

A = cross-section area,

b = overall breadth of tubular section, breadth of I-section flanges,

d = clear depth of the web between flanges,

h = overall depth of the section,

t_f = thickness of the flange, and

t_w = thickness of the web.

NOTE — Fastener holes need not be accounted for in plastic design shear strength calculation provided that:

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

If A_{vn} does not satisfy the above condition, the effective shear area may be taken as that satisfying the above limit. Block shear failure criteria may be verified at the end connections. Section 9 may be referred to for design strength under combined high shear and bending.

8.4.2 *Resistance to Shear Buckling*

8.4.2.1 Resistance to shear buckling shall be verified as specified, when

$\frac{d}{t_w} > 67\varepsilon$ for a web without stiffeners, and

$> 67\varepsilon\sqrt{\frac{K_v}{5.35}}$ for a web with stiffeners

where

K_v = shear buckling coefficient (*see* **8.4.2.2**), and

$\varepsilon = \sqrt{250 / f_y}$

8.4.2.2 *Shear buckling design methods*

The nominal shear strength, V_n, of webs with or without intermediate stiffeners as governed by buckling may be evaluated using one of the following methods:

a) *Simple post-critical method* — The simple post critical method, based on the shear buckling strength can be used for webs of I-section girders, with or without intermediate transverse stiffener, provided that the web has transverse stiffeners at the supports. The nominal shear strength is given by:

$$V_n = V_{cr}$$

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*

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

where

$\beta = (2V/V_d - 1)^2$ $Z_p f_y \frac{1}{\gamma_{m0}} \le 1.2 Z_e f_y/\gamma_{m0}$

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**).

double [illegible]

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:

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

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

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

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*

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

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

where

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

c) *For standard I or H sections*

for $n \le 0.2$ $M_{ndy} = M_{dy}$
for $n > 0.2$ $M_{ndy} = 1.56\, M_{dy}(1-n)(n+0.6)$
$M_{ndz} = 1.11\, M_{dz}(1-n) \le M_{dz}$

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

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

$$M_{ndy} = M_{dy}(1-n)/(1-0.5a_f) \le M_{dy}$$
$$M_{ndz} = M_{dz}(1-n)/(1-0.5a_w) \le M_{dz}$$

where

$a_w = (A - 2\,b\,t_f)/A \le 0.5$
$a_f = (A - 2\,h\,t_w)/A \le 0.5$

e) *Circular hollow tubes without bolt holes*

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

Table 2 Limiting Width to Thickness Ratio

(*Clauses* 3.7.2 *and* 3.7.4) $b = \frac{b_f}{2}$

Compression Element (1)			Ratio (2)	Class of Section		
				Class 1 Plastic (3)	Class 2 Compact (4)	Class 3 Semi-compact (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		
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	$\frac{84\varepsilon}{1+r_1}$ but $\le 42\varepsilon$	$\frac{105.0\,\varepsilon}{1+r_1}$	$\frac{126.0\,\varepsilon}{1+2r_2}$ but $\le 42\varepsilon$
		If r_1 is positive :	d/t_w		$\frac{105.0\,\varepsilon}{1+1.5r_1}$ 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:

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

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

IS1905:1987

Design of Masonry Structure

Introduction:-

- Masonary structure are those Structures which are built from individuals units laid & bound together by mortar.
- Masonry is commonly used for walls.
- Brick & concrete blocks are most commonly use materials.
- Masonry has high compressive strength under vertical loads but has low tensile strength against twisting or stretching unless reinforced.

Types of wall According to Structural Design consideration for masonry wall:-

1- Load bearing wall
2- Non-load bearing wall

Load bearing wall:-

A load bearing wall is part of the structure of the building used to support floors, ceiling, roof & other walls.

Non load bearing wall:-

- A non load bearing wall is used to divide rooms but doesn't hold anything up apart from its own weight.
- You can remove a non load bearing wall with no repercussion but a load bearing wall can be removed but you have to redistribute the load path.
- Types of ~~to~~ Non load bearing wall
  - a- Partition wall
  - b- Panel wall
  - c- Curtain wall

a- Partition Wall:-

It is an interior non load bearing wall to divide the larger space into smaller spaces. These walls are made up of glass, fiber boards or bricks masonry.

b. Panel wall :-

- It is generally made of wood & is an exterior non load wall in framed construction.
- It is used for aesthetic of the building both inside & outside.
- It remains totally supported at each storey but subjected to lateral loads.

c. Curtain wall :-

- It is an outer covering of a building in which the outer walls are non structural, but merely keep the weather out & the occupants.
- It is also known as skin of building.

MORTAR

Types of mortar based on application :- (Table-1, Pg-6)

1. Bricklaying or Stone laying mortar -

This type of mortar used to bind bricks and stones in masonary construction. The proportions of ingredients for bricklaying or stone laying mortar is decided based on kind of binding material used.

2. Finishing mortar :-

- Finishing mortar is used for pointing and plastering work.
- It is also used for architectural effects of building to give aesthetic appearances.
- The mortar used for ornamental finishing should have great strength, mobility and resistance against atmospheric action like rain, wind, etc.

3. Cement mortar :-

- Cement is used as a binding material in this type of mortar and sand is employeed as aggregate. The proportion of cement and sand is decided based on the specified durability and working conditions.
- Cement mortar will give high strength and resistance against water. The proportion of cement to sand may varies from 1:2 to 1:6.

1. Lime Mortar:-

- In this case, lime is used as binding material. There are two types of limes namely fat lime and hydraulic lime. Fat lime in lime mortar requires 2 to 3 times of sand and it is used for dry work.
- Hydraulic lime and sand in 1:2 ratios will give good results in damp conditions and also suitable for water logged areas.
- Finally, the lime mortar has a high plasticity so it can be placed easily.

5. Gypsum mortar:-

- Gypsum mortar consists of plaster and sand as binding material.
- Commonly, it has low durability in damp conditions.

Design Consideration of Load Bearing wall:-

- Masonry buildings are mainly constructed of load bearing wall where wall are used to transfer gravity as well as lateral loads to the foundation in addition to its common function of subdividing space providing thermal & acoustic isolation, providing fire resistance and providing weather protection.
- While transferring design loads, the masonry is subjected to mainly compressive, tensile and shear stress which should be well within the permissible limits and the wall should not buckle or overturn.
- Load bearing walls are structurally more efficient when the load is uniformly distributed and the structure is so planned that the eccentricity of loading on the member is as small as possible.
- Avoidance of eccentric loading by providing adequate bearing of roof/floor on the walls providing adequate stiffeners of slab and avoiding fixing at the support etc. is specially important in load bearing walls in masonary structure.

- In order to ensure uniformity of loading, openings in walls shouldnot be too large. ~~and the~~
- Bearings for lintels and bed blocks under beams should be liberal in size, heavy concentration of load should be avoided by planning and sections of load bearing members should be varied where feasible with the loading so as to obtain more or less uniform stress in adjoining parts of the members.

Design loads :- (Cl-5.2, Pg-15)

The load to be taken in consideration for design of masonry walls are, (i) Gravity loads - Vertical loads, such as dead load (DL), (LL) Liveload of the super structure.

(ii) Lateral loads - Horizontal loads like- accidental loads (AL) Wind Load (WL), earthquake loads (EL).

Permissible Stresses :- [Cl-5.4, Pg-15 to 5.4.3, to-17]

Design consideration for Non-load bearing walls :-

- A non-load bearing wall is designed to resist only lateral loads. It may be provided as an exterior wall to protect against weather and as an interior wall for the purpose of partitioning. Hence, a non-load bearing wall may be called a panel wall / Curtain wall / partition wall.
- Panel wall are non-load bearing exterior walls in framed construction wholly supported on ~~b~~ each storey and subject to lateral loads only.
- Curtain walls are supported by horizontal and vertical structural member where necessary and subjected to lateral loads only.

Effective height of masonry walls :- Table-4 of IS 1905 : 1987
Pg-11, Cl-2.6, Cl-4.3

Effective length of masonry walls :- Table-5 of IS 1905-1987
Pg-12, Cl-2.7)

Effective thickness :- (Cl-4.5 - 4.5.5, Pg-13-14) (Cl-2.9)

Slenderness Ratio :- (cl-4.6, pg-14-15)
to 4.6.2

Q1 A ground floor masonry wall is 4m. clear ht upto bottom of the roof slab. Ht of plinth above foundation footing = 0.8m. If the wall thickness is 30cm, calculate effective height & slenderness ratio for partial restraint on both end.

Solⁿ

Ht of wall measured from top of the footing = 4+0.8 = 4.8m
(cl-4.3.1)

From Table-4 of IS 1905:1987,

Effective ht of wall = 1.0H = 1 * 4.8 = 4.8m. (Ans)

Slenderness ratio = $\frac{h}{t} = \frac{4.8}{0.3} = 16$ (Ans)

Q2 A masonry wall is 4m. height and 6m length, calculate effective length of the wall for the support condition, wall is supported by a crosswall at one ~~one~~ end and continuous with crosswall at the other end.

Solⁿ For the case as given in question
length = 6m, height = 4m. (Table-5, Pg-12)
∴ Effective length = 0.9L = 0.9 × 6 = 5.4m (Ans)

Design of Masonry columns:-

Effective ht. of column – H is taken as actual height or clear distance between the supports for the direction, it is laterally supported and as twice the actual ht. for the direction, it is not laterally supported. (Fig-12)