# ARYAN SCHOOL OF ENGINEERING & ECHNOLOGY

**BARAKUDA, PANCHAGAON, BHUBANESWAR, KHORDHA-752050**

# LECTURE NOTE

SUBJECT NAME- STRUCTURAL DESIGN-II

BRANCH-CIVIL ENGG.

SEMESTER-5TH SEM

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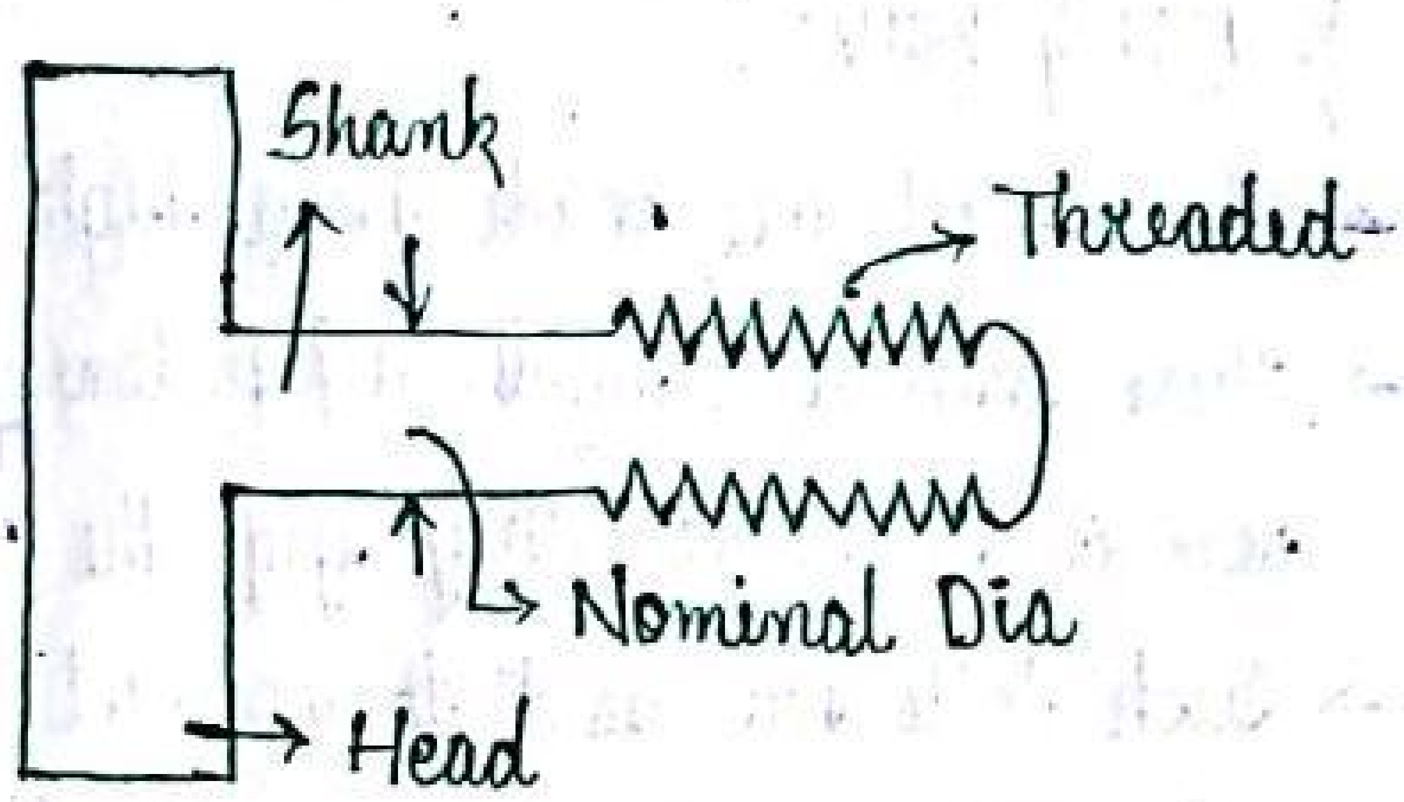
PREPARED BY- PRATIKSHYA BHUYAN

# Introduction To Design of Steel Structure

## # Bolted Connection :-

→ The figure shows a typical bolt which is a metal pin with a head at one end & shank threaded on the other end for connecting the metal plates with holes in which bolts are fixed and nuts are used for tightning purpose.




## # Classification of Bolts :-

1) Based on the Manufacturing :-
   a) Unfinished bolts.
   b) Finished bolts.
   c) High strength Friction Grip bolts. (HSFG)

1.a) Unfinished Bolts :-

→ These bolts are made up of steel rods with square or hexagonal heads.

→ The nominal diameter of these bolts of size 12, 16, 20, 24, 30, 36 mm are available & are commonly used.

→ These bolts are used for light structures under static load such as truss, bracings, temporary connections during constructions.

1.b) Finished Bolts :-

→ These bolts are also made from mild steel and are finished by turning to a circular shape because they are formed from horizontal rods.

→ Since the connection is more tight, it results into much better beari contact between the bolts & holes.

→ These bolts are used in connecting machine parts subjected to dynam loading.

1.c) <u>HSFG Bolts</u>:-

→ These bolts are made from high strength steel rods.

→ These bolts are tightened / fastened by using calibrated machine tools, i wrenches. Hence, they grip the member tightly.

→ Such bolts are used to connect members subjected to dynamic load Commonly available nominal diameter of HSFG bolts are 12, 16, 20, 24 30, 36, mm.

*** <u>Note</u>:-

→ M16, M20, M12 → The numbers denotes the diameter of the bolt & t alphabet 'M' stands for mild Steel.

2) <u>Based on the Load Transfer</u>:-

a) Bearing Bolts.

b) Friction Grip Bolts.

2.a) <u>Bearing Bolts</u>:-

→ Unfinished & Finished bolts belong to bearing type since they transfe the shear force or load from one member to another member by bearing of contact surface.

2b) <u>Friction Grip</u>:-

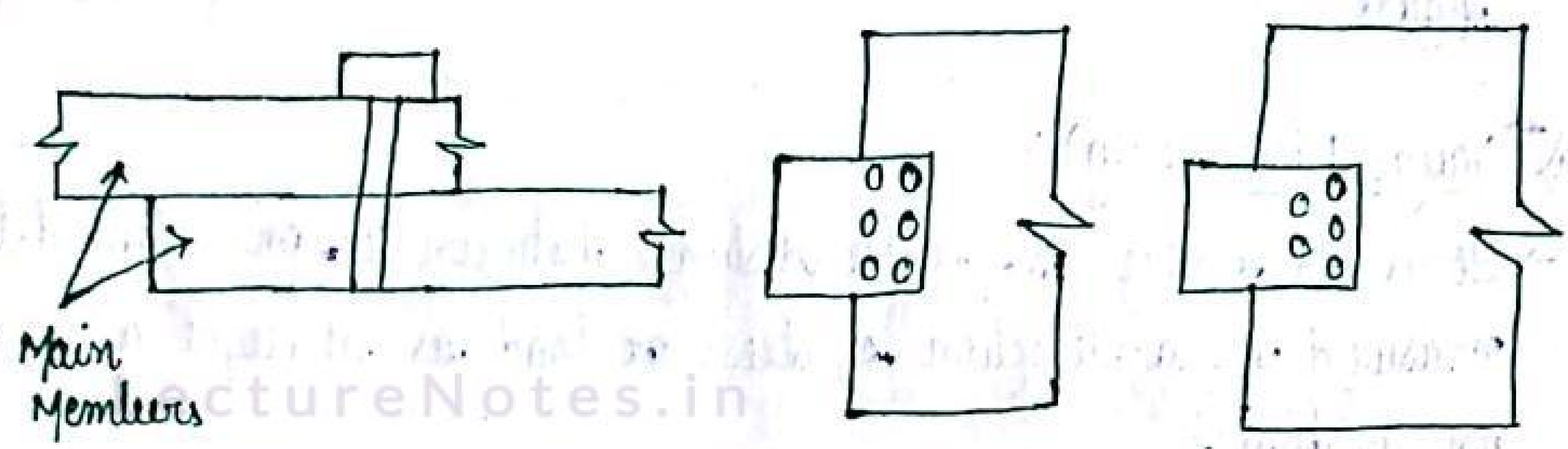
→ The high strength friction grip bolt belongs to friction grip type since, they transfer the load by friction.

# <u>Types of Bolted Connections</u>:-

→ There are 2 types of bolted connections : Lap joint & Butt Joint

1) Lap Joint :-

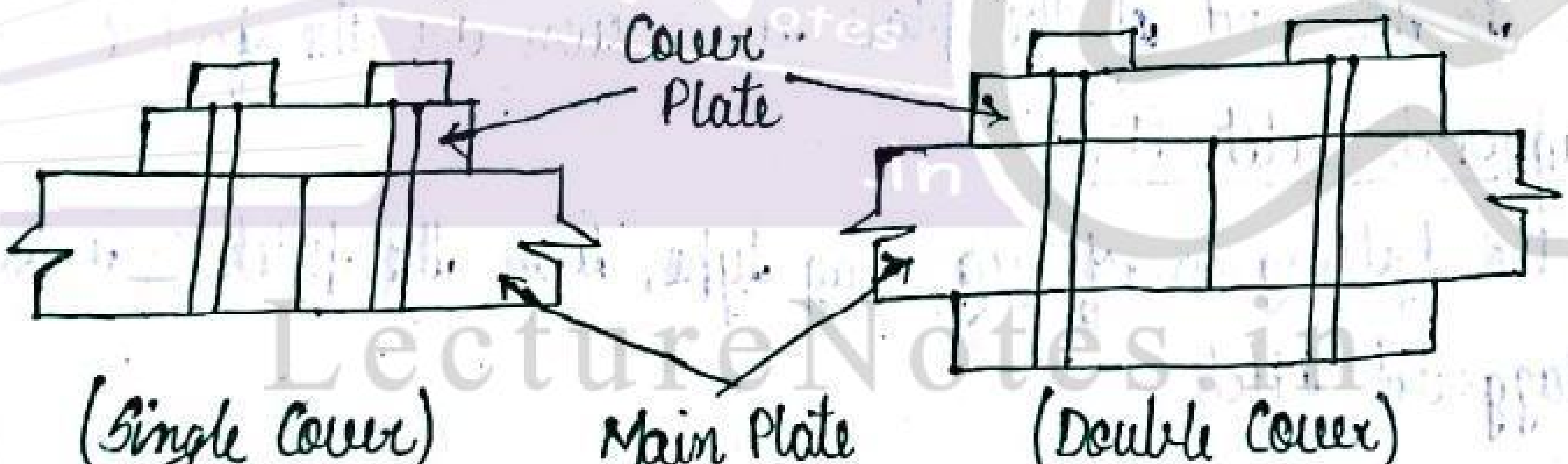
→ In this type of connection, one member plate overlaps on other member and then bolted.




(Chain Bolting) (Zig-Zag Bolting)

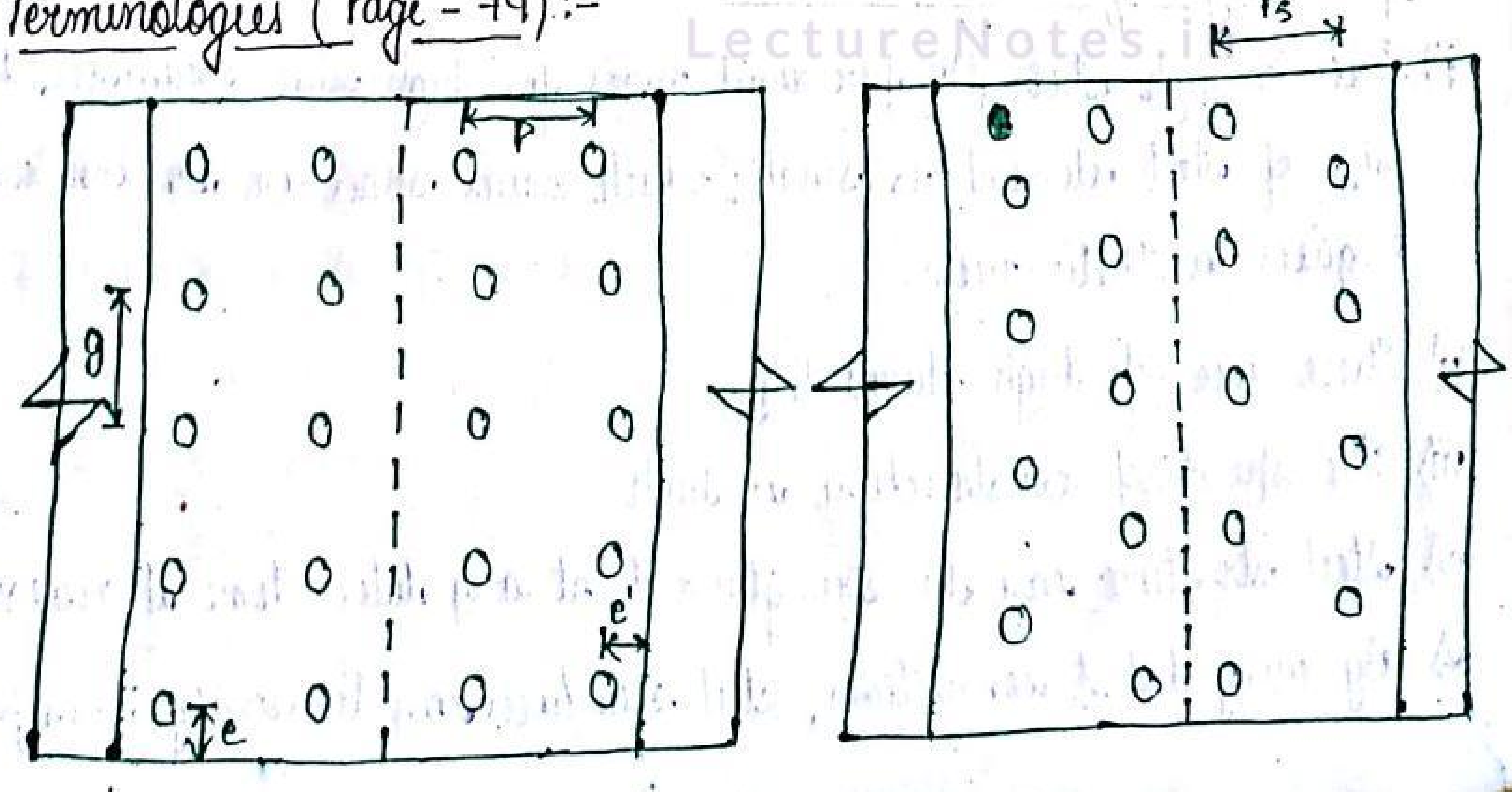
2) Butt Joint :-

→ In this type of connection, the two main plates touch each other head-to-head & connection is made by providing a cover plate on one-side (single cover butt joints) or 2 cover plates are used on the top & at the bottom (double cover butt joint).




(Single Cover) (Double Cover)

# Terminologies (Page - 74) :-



1) Pitch (P):-

→ It is the centre-to-centre distance between the 2 consecutive bolt holes in a line parallel to the direction of stress or load as shown in the figure.

2) Gauge Distance (g):-

→ It is the centre-to-centre distance between 2 consecutive bolt holes measured in a direction of stress or load at right angle as shown in the figure.

3) Edge Distance (e):-

→ It is the distance at right angle to the direction of stress from the centre of the hole to the adjacent edge of the plate as shown in the figure.

4) End Distance (e'):-

→ It is the distance in the direction of stress from the centre of the hole to the end of the plate as shown in the figure.

5) Staggered Pitch ($P_s$):-

→ If the bolting is of zig-zag type, then the pitch is known as Staggered Pitch.

# Advantages of Steel Structures:-

i) Due to high strength per unit mass for high rise structures, the size of steel element is small, which saves space in the construction & gives aesthetic view.

ii) These are of high durability.

iii) The speed of construction is high.

iv) Steel structure can be strengthened at any later-time if necessary.

v) By using bolted connections, steel structures can be easily transported to the site from the manufacturing point.

vi) The steel structure materials are reusable.

# Disadvantages of Steel Structures :-

i) It is more vulnerable / susceptible to corrosion.

ii) The maintanance cost is very much high since painting is required to prevent corrosion.

iii) Steel members are more costly than the concrete structures.

# Design Philosophy :-

→ The structure should be designed in such a way that it should :

i) sustain all the loads expected on it.

ii) should have adequate durability.

iii) sustain deformation during & after construction.

iv) should have resistance to misuse and fire.

→ There are 3 methods of design :

i) Working stress method.

ii) Limit state method.

iii) Ultimate load method.

1.) Limit State Method :-

→ Limit states are the state where beyond which the structure no longer satisfied the specified performance requirements.

→ All the relavant limit states as follows should be considered but usually it will be appropriate to design on the basis of strain and stability at the ultimate load & then checking for deflection under serviceability load.

→ Two types of limit states are mainly used, i.e; limit state of strength & limit state of serviceability.

1.a) Limit State of Strength :-

→ It is associated with failures under the action of probable & most unfavourable combination of loads on the structure using the appropriate partial safety factor, which may endanger the safety of life and properties.

→ It includes :-

i) Loss of equilibrium of structure as a whole or any of its parts or components.

ii) Loss of stability of the structure or any of its components.

iii) Fracture due to fatigue.

iv) Brittle fracture.

1.b) Limit State of Serviceability :-

→ It is the limit state beyond which the specified service criteria is not fulfilled.

→ It includes :-

i) Deformation or Deflection.

ii) Vibration.

iii) Corrosion & Durability.

iv) Repairable damage due to fatigue.

* Load Combinations :-

i) Dead Load + Live Load (or Imposed Load) = DL + LL (or IL)

ii) Dead Load + Live Load (or Imposed Load) + Wind Load + Seismic Load = DL + LL (or IL) + WL + SL.

iii) Dead load + Seismic load (or wind load) = DL + SL (or WL)

# Rolled Steel Section :-

1) Rolled Steel I - Section :-

i) Indian Standard Junior Beam → ISJB

ii) Indian Standard Light Beam → ISLB

iii) Indian Standard Medium Beam → ISMB

iv) Indian Standard Wide - Flange Beam → ISWB.

v) Indian Standard Heavy Beam → ISHB.

2) Rolled Steel Channel Sections :-

i) Indian Standard Junior Channel → ISJC

ii) Indian Standard Light Channel → ISLC

iii) Indian Standard Medium Channel → ISMC

iv) Indian Standard Special Channel → ISSC.

3) Rolled Steel Angle Section :-

i) Indian standard Angle (ISA).

↳ Unequal

↳ Equal

4.) Rolled Steel T- Section :-

i) Indian Standard Normal T-Section → ISNT

ii) Indian Standard Heavy T- Section → ISHT

iii) Indian Standard Special Legged Section → ISLLT.

iv) Indian Standard Light Section → ISLT

v) Indian Standard Junior T- Section → ISJT.

5) Rolled Steel Bars:-

i) Indian Standard Round Bars → ISRO.

ii) Indian Standard Square Bars → ISSQ.

6) Rolled Steel Plate (ISPL)

7) Rolled Steel Strip (ISST)

8) Rolled Steel Flats (ISF).



# Code Provision For Connections (Indian Standard):-

i) Clearance for the bottom holes (P-73, T-19)

ii) Minimum Pitch (P-73, 10.2.2)

iii) Maximum Pitch (P-74, 10.2.3.1, 10.2.3.2)

iv) Edge and End directions (P-74, 10.2.4.1, 10.2.4.2, 10.2.4.3)

v) Design strength of Bearing Type bolts:

→ Shear strength capacity of bolt (P-75, 10.3.3)

→ Bearing strength of bolt (P-75, 10.3.4)

→ Tension strength of bolt (P-76, 10.3.5)

→ Bolt subjected to combine shear & tension (P-76, 10.3.6)

vi) Design strength of plate at a joint:

→ Due to rupture of critical section (P-32, 6.3.1)

→ Due to block shear failure (P-33, 6.4.1)

→ Tensile property of structural steel product (P-13, T-1).



# Note:-

→ Efficiency of a Joint:- It is the ratio of strength of the joint to the strength of solid plate in tension.

Mathematically, efficiency is expressed as:

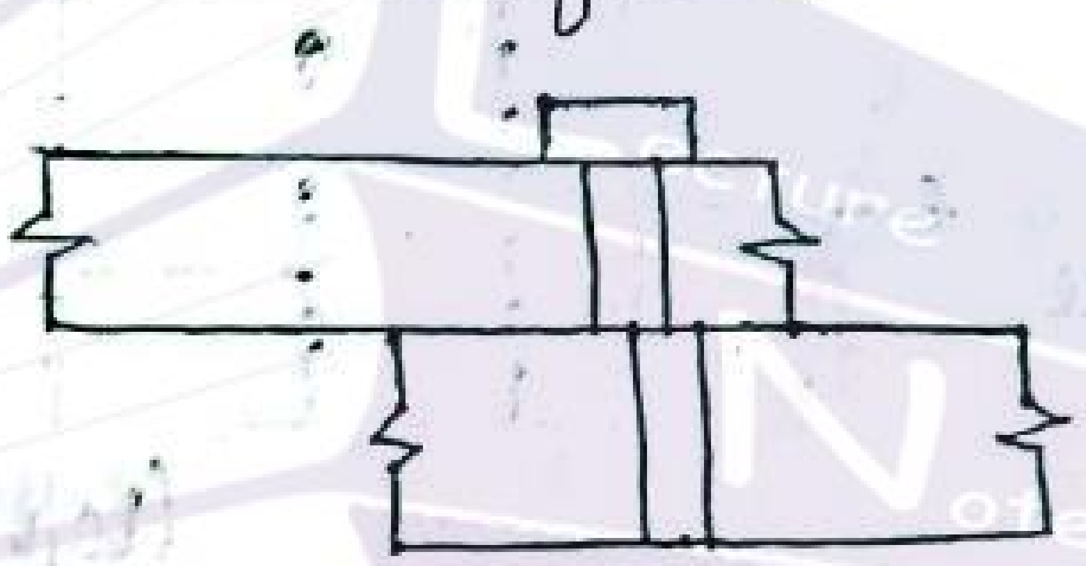
$$\boxed{\eta = \frac{\text{strength of joint}}{\text{strength of steel plate in tension}} \times 100}$$

# Types of Failure :-

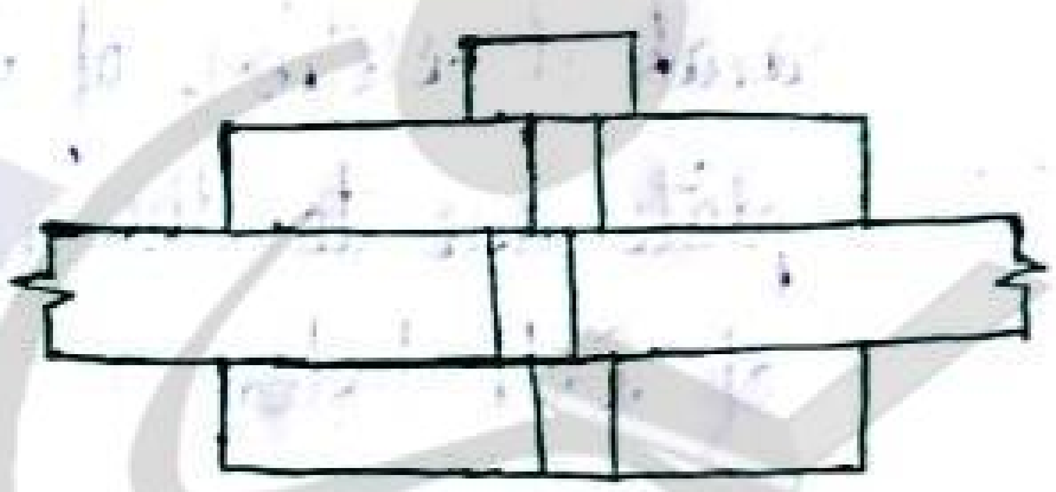
→ The failure of the joint occurs due to the failure of Bolt or due to the failure of plate.

1) Failure of Bolt :-

a) Shear Failure of Bolt :-



[Single Shear Failure of Bolt in case of Lap Joint]



[Double Shear Failure of Bolt in case of Double Cover Butt Joint.]

→ Bolt may fail due to shearing. Shearing may occur at one section known as single shear failure or at two sections known as Double shear failure which occurs only in case of Double cover Butt joint.

b) Crushing / Bearing Failure :-

→ The bolt may get crushed if plate material is stronger than the bolt material which is known as Crushing / Bearing failure of the bolt. Ex: Bolt of plastic fitted in Iron plate.

c) Tension Failure :-

→ When the bolt is subjected to tension, fracture may take place at the thread portion. Since, thread is the weakest section of the bolt.

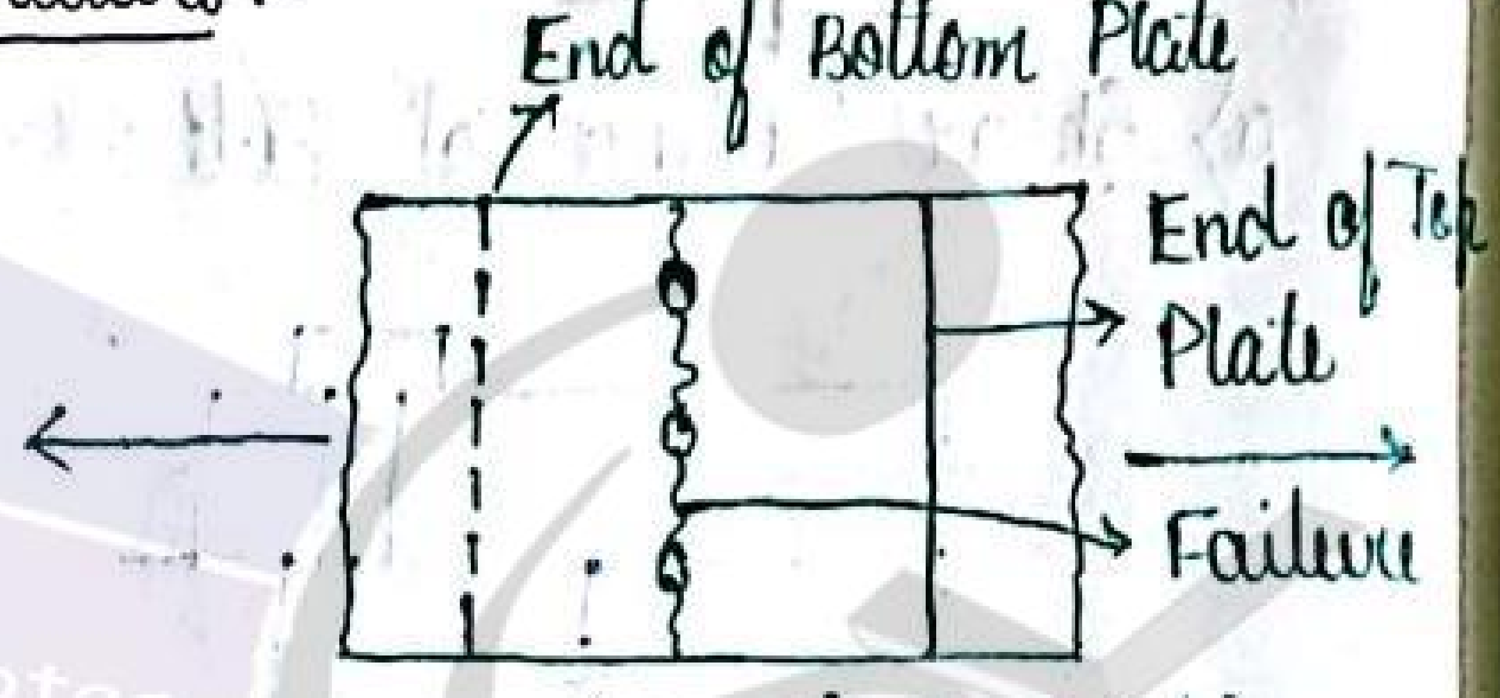
2) Failure of The Plate :-

a) Bearing / Crushing Plate :-

→ The plate may get crushed if the plate material is weaker than the bolt material.

b) Tension / Rupture / Tearing Failure :-

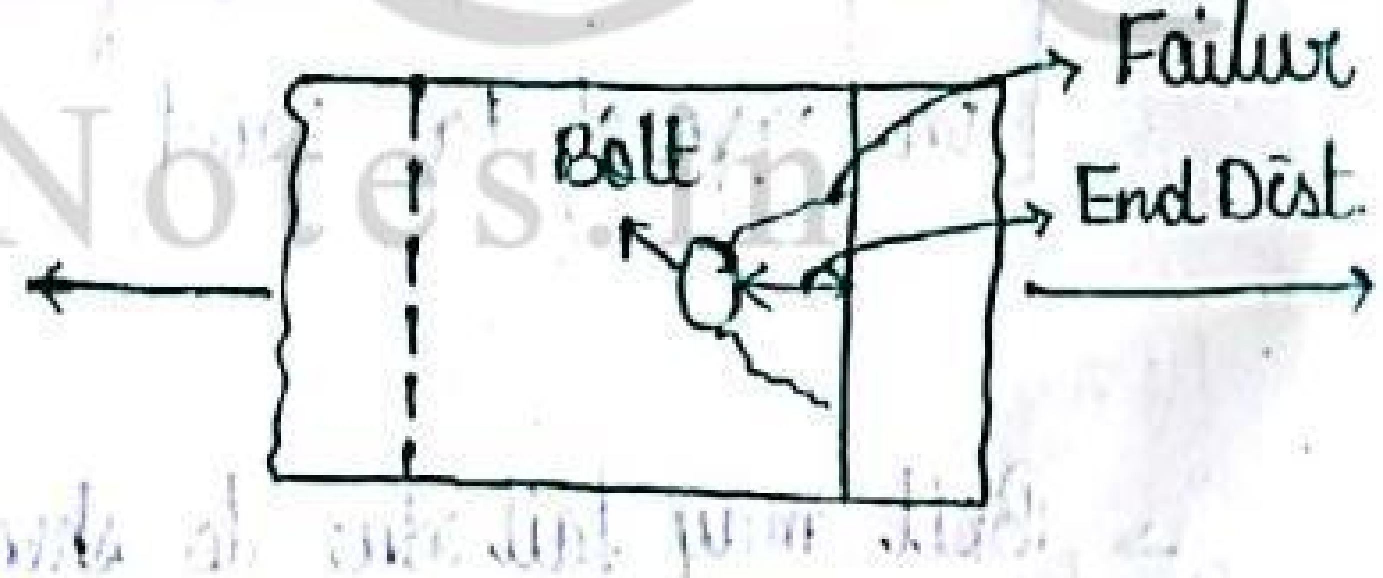
→ It takes place along the weakest section of the plate due to the presence of Bolt holes:




(Lap Joint)

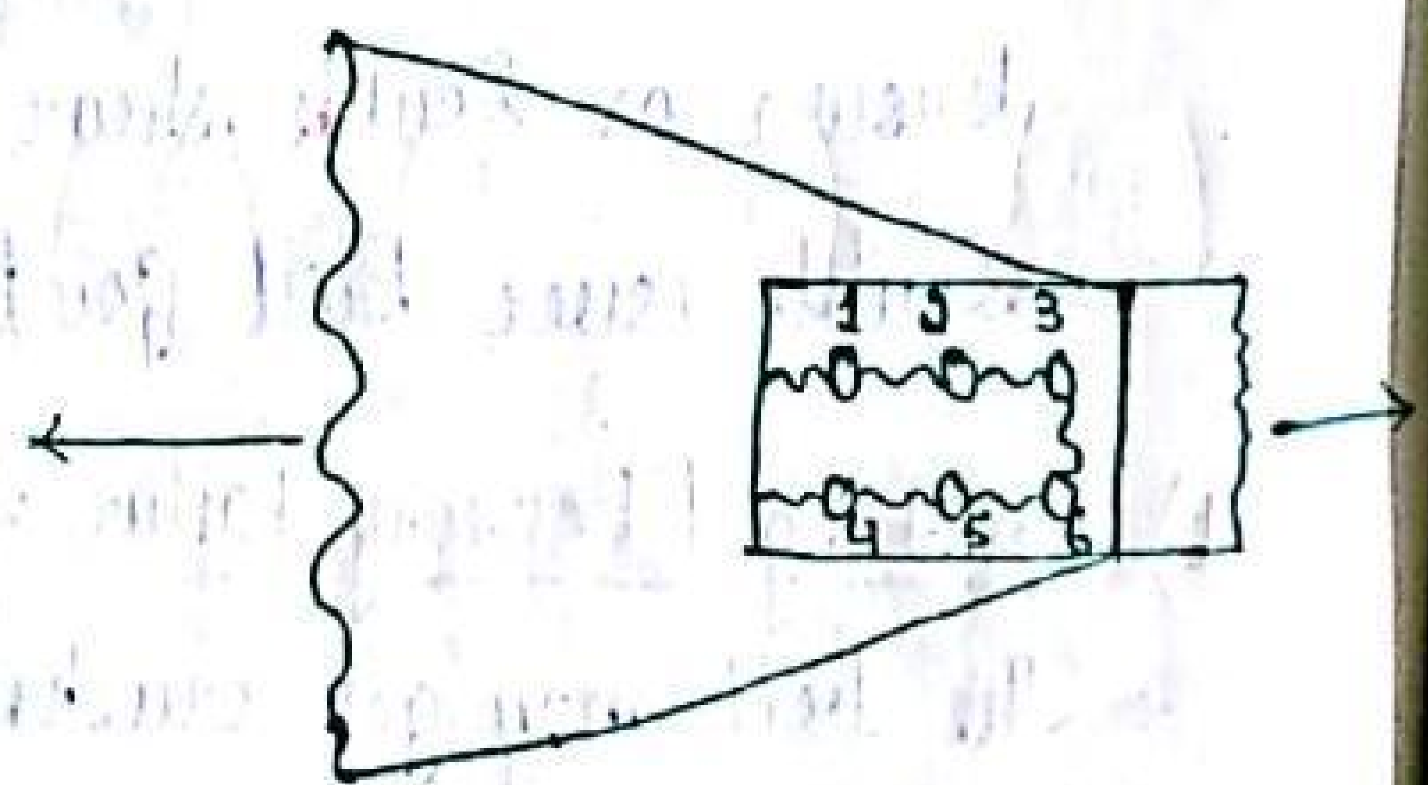
c) Shear Failure :-

→ This type of failure occurs when there is insufficient end directions.




d) Block Shear Failure :-

→ This shearing failure will occur in the direction of 1-2-3, 4-5-6 and the rupture failure will occur in the direction of 3-6.




→ A portion of the plate may fail by rupture & shearing failure & this type of failure is known as Block shear failure.

Q.) Calculate the strength of 20mm dia bolt of grade 4.6 for the following cases. The main plates to be connected are of 12mm thick.

i.) Lap joint.

ii.) Single cover Butt joint.

iii.) Double cover Butt joint.

The plate which are used of heavy $Fe_{410}$ grade steel.

A.) The grade of the bolt is 4.6

The ultimate strength of the bolt is:- $f_{ub} = 400 N/mm^2$ (P-13, T-1)

The grade of the plate is Fe 410, hence the ultimate strength of the plate, $f_u = 410 N/mm^2$ and the yield strength of the plate is $240 N/mm^2$. (P-14)

The diameter of the bolt is 20mm, hence minimum pitch.

$P = 2.5 \times d = 2.5 \times 20 = 50mm.$ (P-73)

The diameter of hole, $d_0 = 20 + 2 = 22.$ (P-73, T-19)

Minimum edge distance (P-74, 10, 2.3)

$\Rightarrow e = 1.5 \times d_0 = 1.5 \times 22 = 33$

i.) Lap joint :-

For shearing strength :

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

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

where, $\gamma_{mb} = 1.25.$

$A_{sb}$ = Area of shank

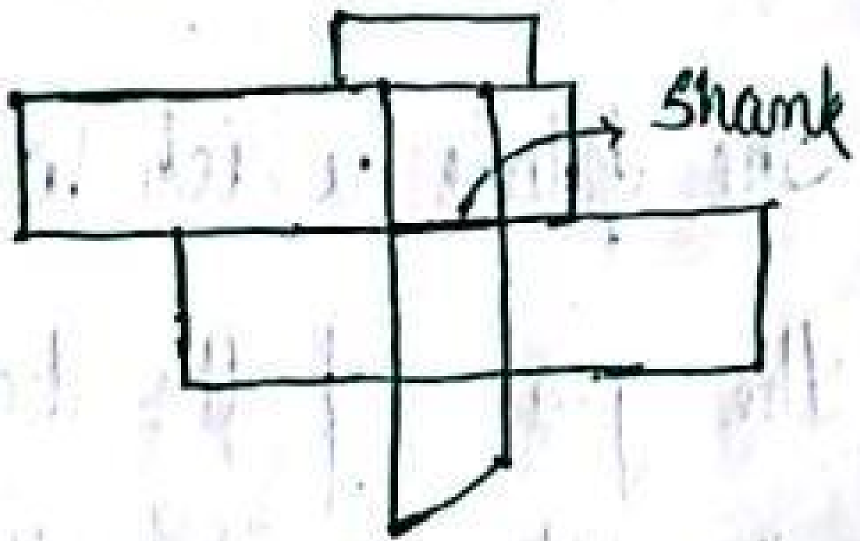
$A_{nb}$ = Area of thread

$n_n$ = no. of planes in thread in shear.

$n_s$ = no. of planes in shank in shear.

As thread is the weakest section of the bolt, hence assume that no shank is in the shear area. Hence, $n_n = 1$ & $n_s = 0$.

$$A_{nb} = 0.78 \times \frac{\pi}{4} \times d^2 = 245.05\,mm^2$$




$$A_{sb} = \frac{\pi}{4} \times d^2 = 314.16\,mm^2.$$

$$\therefore V_{nsb} = \frac{f_{ub}}{\sqrt{3}}\left(n_n A_{nb} + \cancel{n_s A_{sb}}^{\,0}\right)$$

$$= \frac{400}{\sqrt{3}}\left(1 \times 245.05\right)$$

$$= 56.59\ kN.$$

$$\therefore V_{dsb} = \frac{56.59}{1.25} = 45.272\ kN.$$

For bearing strength :-

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

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

$$k_b = \text{constant} = \left(\frac{e}{3d_0}\right) \text{ or } \left(\frac{P}{3d_0} - 0.25\right) \text{ or } \left(\frac{f_{ub}}{f_u}\right) \text{ or } (1.0)$$

$$= \left(\frac{33}{3\times 22}\right) \text{ or } \left(\frac{50}{3\times 22} - 0.25\right) \text{ or } \left(\frac{400}{410}\right) \text{ or } 1.0$$

$$= (0.0450) \text{ or } (0.5013) \text{ or } (0.975) \text{ or } (1.0)$$

$$= 0.50.$$

$$\Rightarrow V_{npb} = 2.5 \times 0.50 \times 20 \times 12 \times 410 = 123 \text{ kN}.$$

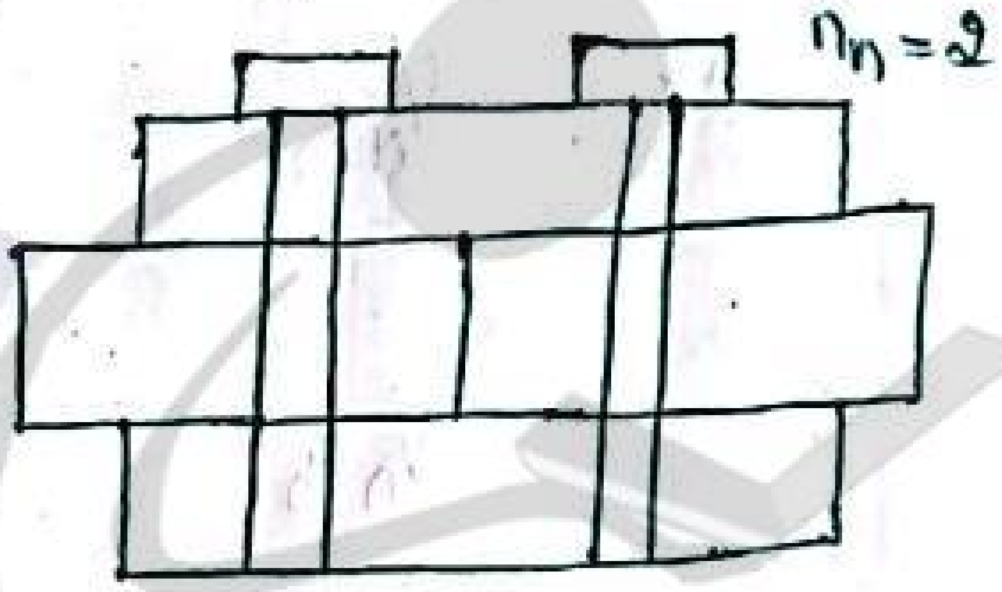
$$\Rightarrow V_{dpb} = \frac{V_{npb}}{\gamma_{mb}} = \frac{123}{1.25} = 98.4 \text{ kN}.$$

∴ The strength of the bolt is 45.272 kN.

ii) Single Cover Butt Joint :-

In the single cover butt joint, as the thread in the shear plane, hence the strength of the bolt in shearing & bearing will be same as in case of lap joint.

iii) Double cover Butt Joint :-




Strength of the bolt in shearing :-

Assume that no shank is in the shear area, hence $n_s = 0$

& $n_n = 2$.

$$\therefore V_{nsb} = \frac{f_{ub}}{\sqrt{3}}(n_n A_{ns}) = \frac{400}{\sqrt{3}} \times (2 \times 245) = 113.16 \text{ kN}.$$

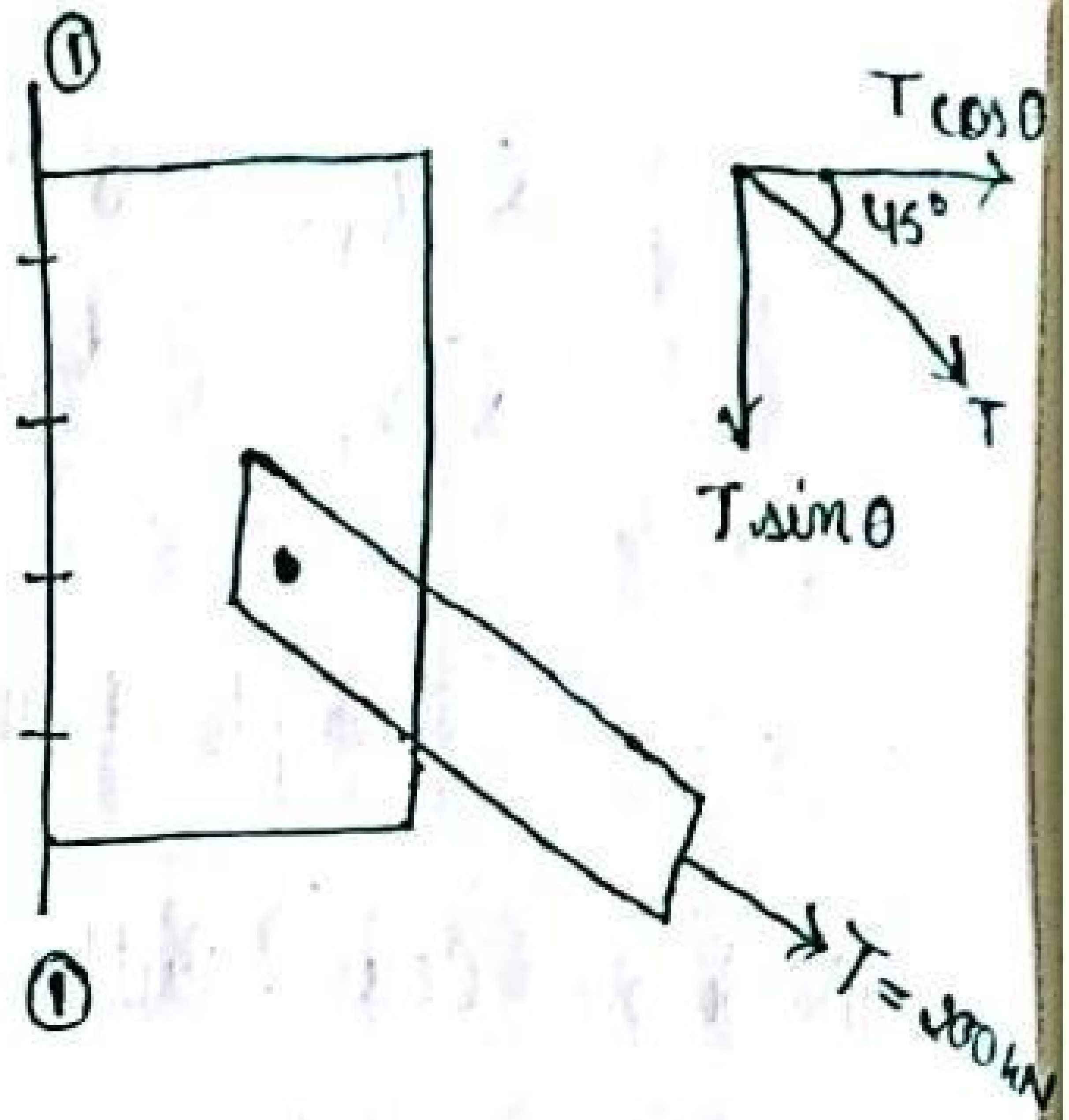
$$V_{dsb} = \frac{113.16}{1.25} = 90.528 \text{ kN}.$$

The bearing strength of the bolt as in case of lap joint. Hence, bearing strength = 98.4 kN

The strength of the bolt in double cover butt joint = 90.528 kN.

Q) Determine whether the joint shown in figure is same or not. 8–16 mm dia bolt of grade 4.6 have been used for making the connection at the given section. The bolt is machine fabricated & the material is bronze steel. The plate is of grade Fe 410.

A) The horizontal component of the coarse T cos θ will give tension in the bolt & the vertical component of the bolts T sin θ will give shear in the bolts.




For shear strength of the bolt :-

The shear force acting on the bolt $= T \sin\theta$

$= 200 \times \sin 45^\circ$

$= 141.4$ kN

Now, $V_{dsb} = \dfrac{V_{nsb}}{\gamma_{mb}}$

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

Assuming no shank portion is in the shear, $n_s = 0, n_n = 1$.

$= \dfrac{400}{\sqrt{3}} \times \left(1 \times 0.78 \times \dfrac{\pi}{4} \times 16^2\right)$

$= 36.21$ kN

$\Rightarrow V_{dsb} = \dfrac{36.21}{1.25} = 28.97$ kN

Force on 1 bolt $= \dfrac{141.4}{8} = 17.67$ kN. $= V_{sb}$

Hence, as the provided load is less than the design strength of 1 bolt in shear. So, the connection is shaped against shear.

For tension capacity of the bolt :- (P-76-10.3.5)

The tension force on the bolts = $T \cos\theta$

$= 200 \times \cos 45^\circ$

$= 141.4$ kN

∴ The tension force on 1 bolt $= \frac{141.4}{8} = 17.67$ kN. $= T_b$

$T_b \leq T_{db}$

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

$T_{nb} = 0.9 \times f_{ub} \times A_n$

$= 0.9 \times 400 \times 0.78 \times \frac{\pi}{4} \times 16^2$

$= 56.46$ kN.

$\Rightarrow T_{db} = \frac{56.46}{1.25} = 45.168$ kN

As, the tension capacity of bolt is more than the provided tension load, hence the connection is safe against tension.

For combined shear & tension strength in the bolt :-

(P-76-10.3.6)

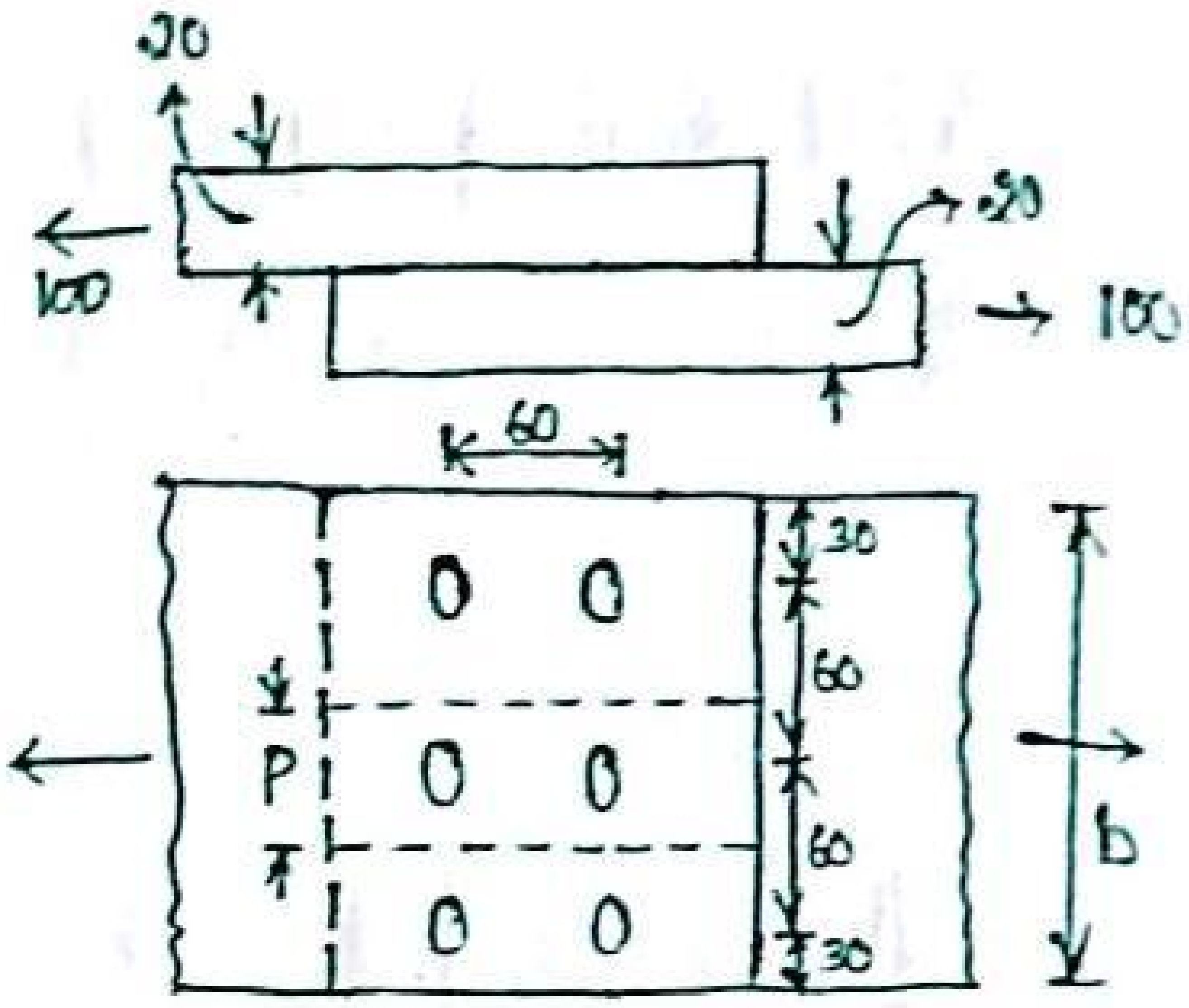
$$\left(\frac{V_{sb}}{V_{db}}\right)^2 + \left(\frac{T_b}{T_{db}}\right)^2 \leq 1.0$$

$$\Rightarrow \left(\frac{17.67}{28.97}\right)^2 + \left(\frac{17.67}{45.168}\right)^2 \leq 1.0$$

$$\Rightarrow 0.525 \leq 1.0$$

Hence, the connection is safe against combined shear & tension.

Q) Find the strength & efficiency of the joint as shown in the fig. In the joint, bolt M-20 @ 4.6 grade is used & the plate is of grade Fe410.




A) Given :-

$d = 20 mm$.

$d_0 = 20 + 2 = 22 mm$.

$f_{ub} = 400$ ~~N/mm²~~ MPa

$f_u = 410$ ~~N/mm²~~ MPa

$f_y = 250$ MPa

$A_{nb} = 245.05 mm^2$.



The strength of the joint:

Taking the small portion of the joint of height = pitch, we have to calculate the strength of the joint.

The strength of the joint includes strength of the plate, strength of the bolt in shear, strength of the bolt in bearing.

Strength of the plate (for each width) small section:

(P-32 - 6.3.1)

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

$$A_n = \left[ b - n d_h + \cancel{\Sigma \frac{s_i^2}{4 g_i}}^{\,0} \right] t$$

$b = P = 60$, $n = 1$.

As, for the top plate, only 1 bolt is there in the critical section per pitch width, hence $n = 1$.

$$\Rightarrow A_n = (60 - 22) \times 20 = 760 \text{ mm}^2$$

$$\Rightarrow T_{dn} = \frac{0.9 \times 760 \times 410}{1.25} = 224.35 \text{ kN}.$$

<u>Strength of the bolt in shear</u>:

The shear strength of the bolt = 45.26 kN (from previous problem)

The shear strength of the joint for pitch width = 45.26 × 2

= 90.54 kN.

As 2 no. of bolts are there in the small for pitch width, the shear strength is 90.54 kN.

<u>Strength of the bolt in ~~shear~~ bearing</u>:

$$K_b = \frac{e}{3d_0} \text{ or } \frac{P}{3d_0} - 0.25 \text{ or } \frac{f_{ub}}{f_u} \text{ or } 1.0$$

$$= \frac{30}{3 \times 22} \text{ or } \frac{60}{3 \times 22} - 0.25 \text{ or } \frac{400}{410} \text{ or } 1.0$$

$$= 0.45 \text{ or } 0.659 \text{ or } 0.975 \text{ or } 1.0$$

$$= 0.45.$$

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

$$= 2.5 \times 0.45 \times 20 \times 20 \times 410$$

$$= 184.5 \text{ kN}$$

$$\Rightarrow V_{dpb} = \frac{V_{npb}}{\gamma_{mb}} = \frac{184.5}{1.25} = 147.6 \text{ kN}$$

The strength of the joint in bearing for small section for pitch width = 295.2 kN.

The strength of the joint is lesser of : bearing strength or shearing strength or strength of the plate = 90.54 kN.

Strength of the plate in the small section per pitch width:

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

$$= \frac{0.9 \times (60 \times 20) \times 410}{1.25}$$

$$= 354.24 \text{ kN}$$

$$\eta = \frac{90.5}{354.24} \times 100$$

$$= 25.54 \%.$$

Q) Two plates of size 210 x 8mm are to be connected using 20mm dia bolt of grade 4.6 to form a lap joint. The joint is supposed to carry a factored load of 250 kN. Design the joint & determine the suitable pitch. The grade of the plate is Fe410.

A) Assume that, the shearing strength of the bolt is the lesser one. Hence the joint we have to design, by taking shearing strength of the bolt into consideration.

For shearing strength of the bolt:

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

$$\text{where } V_{nsb} = \frac{f_{ub}}{\sqrt{3}} (n_n A_{ns})$$

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

$$= 56.59 \text{ kN.}$$

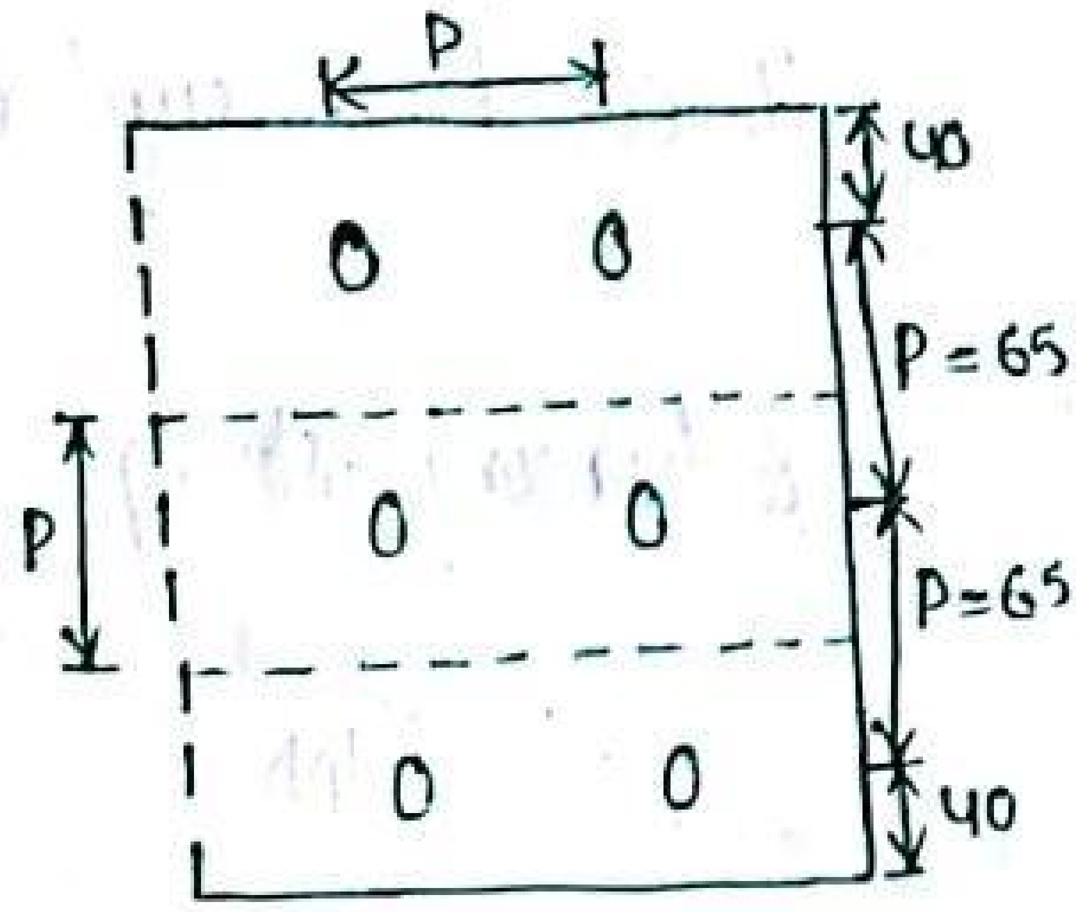
$$\therefore V_{dsb} = \frac{56.59}{1.25} = 45.27 \text{ kN.}$$

Factored load = 230 kN.

$\therefore$ No. of bolts $= \frac{250}{45.27} = 5.52 \simeq 6$ bolts.

Provide 6 bolts of 20mm dia with pitch = p. (assume).

Equating the tension capacity of the plate per pitch width with shearing strength of the bolt, we can find the required pitch.




Tension capacity of plate per pitch width:

$$\Rightarrow T_{dn} = \frac{0.9(p - nd_h)t \times f_{us}}{\gamma_{mb}}$$

$$= \frac{0.9(p - 1\times 22) \times 8 \times 410}{1.25}$$

$$= \frac{(0.9p - 19.8) \times 3280}{1.25}$$

$$= (0.9p - 19.8) \times 2624$$

Equating, we have:

$$(0.9p - 19.8) \times 2624 = 2 \times 45.27$$

$$\Rightarrow 2361.6p - 51955.2 = 90.54 \times 10^3$$

$$\Rightarrow p = \frac{10^3 \times 90.54 + 51955.2}{2361.6} = 60.33 \text{ mm}.$$

$\therefore$ Minimum pitch, $P_{min} = 2.5 \times 20 = 50\text{mm} < 60.33\text{mm}$.

Hence the required pitch is shaped.

Providing pitch, $P = 65$ ; Edge end distance $= \frac{210 - (2\times 65)}{2}$

$= 40\text{mm}$.

Minimum edge end distance, $e = 1.5 \times d_0 = 1.5 \times 22$

$= 33mm < 40mm.$

Hence, the provided edge end distance is shaped.

For bearing strength:



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

where. $V_{npb} = f_{ub} \times 2.5\ k_b\ dt.$

$$k_b = \frac{e}{3d_0} \text{ (or) } \frac{p}{3d_0} - 0.25 \text{ (or) } \frac{f_{ub}}{f_u} \text{ (or) } 1.0$$

$$= \frac{40}{3\times 22} \text{ (or) } \frac{65}{3\times 22} - 0.25 \text{ (or) } \frac{400}{410} \text{ (or) } 1.0$$

$$= 0.60 \text{ (or) } 0.74 \text{ (or) or } \cancel{1.025}\ 0.9 \text{ (or) } 1.0$$

$$= 0.60.$$



$$\Rightarrow V_{npb} = 410 \times 2.5 \times 0.60 \times 20 \times 8$$

$$= 99.385\ kN.$$

$$V_{dpb} = \frac{99.385}{1.25} = 79.5kN > 45.27\ kN.$$



Hence, the design is shaped.

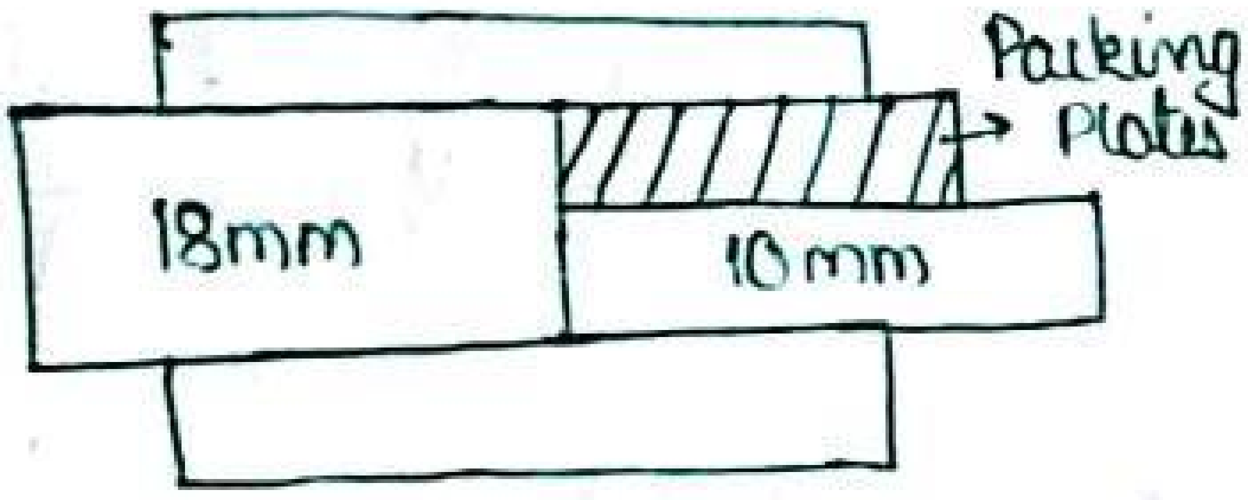
# Vacuum Plate:-

→ Two plates of thickness 10mm & 18mm.

→ The packing plates are provided in the double cover butt joint when the thickness of two main plates are not equal. [P-75, 10.3.3]

Q) Two plates of thickness 10mm & 18mm are to be connected by double cover butt joint. Design the joint for the following data : i) Factored design load = 750 kN, ii) 20mm dia bolt, iii) Grade of the bolt is 4.6 & plate is Fe410, iv) Two cover plates of thickness ~~are~~ 8mm are to be provided on both the side of the joint.

A) The design shear capacity of bolts carrying shear through a packing plate in excess of 6mm shall be decreased by a factor, $\beta_{pk}$ given by :-




$$\beta_{pk} = (1 - 0.0125\, t_{pk})$$

where, $t_{pk}$ = thickness of the thicker packing, in mm.

If the thickness of the packing plate is $\leq 6$mm, then no need to multiply any factor with the shearing strength.

Now, t of packing plate = 18 - 10 = 8mm.

$$\Rightarrow t_{pk} = 8\text{mm}.$$

$$\therefore \beta_{pk} = (1 - 0.0125 \times 8)$$

$$= 0.9.$$

Assuming shearing strength is the lesser one, we have to design the joint.

For shearing strength:

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

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

Assuming the shank & the thread both are in the shear plane,

$n_n = n_s = 1$.

$$\Rightarrow V_{nsb} = \frac{400}{\sqrt{3}}(1 \times 245.05 + 1 \times 314.15)$$

$$= 129.14 \text{ kN}.$$

$$\Rightarrow V_{dsb} = \frac{V_{nsb}}{1.25}$$

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

$$= 103.312 \text{ kN}$$

Due to the packing plate, the shearing strength is to be reduced by a factor $\beta_{pk}$.

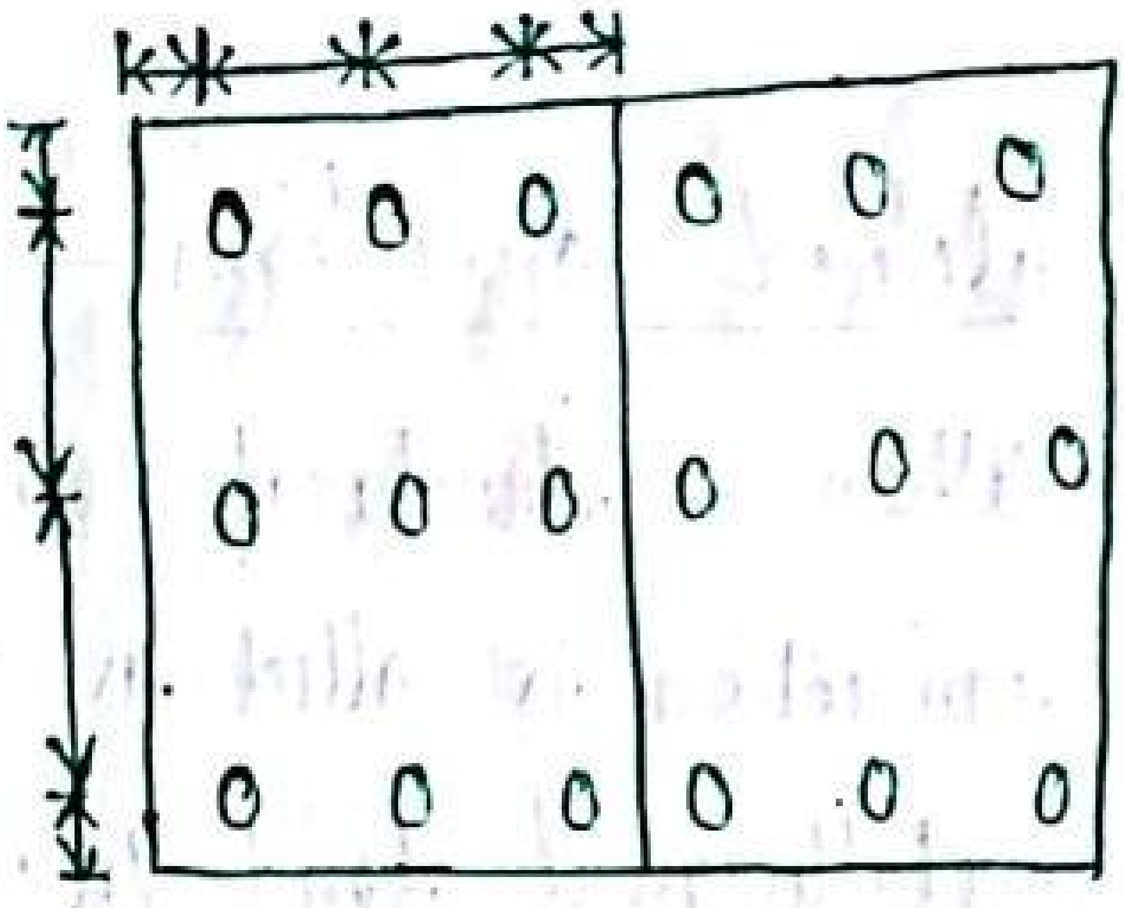
$$\text{Reduced shearing stress} = \beta_{pk} \times V_{dsb}$$

$$= 0.9 \times 103.312$$

$$= 92.98 \text{ kN}.$$

$$\therefore \text{No. of bolts} = \frac{750}{92.98} = 8.066 \simeq 9 \text{ bolts}.$$

Equating the tension capacity of the plate per pitch width with the shearing strength of the bolt, we have:-



$$\Rightarrow \frac{0.9\,(p - nd_h)\,t \times f_u}{\gamma_{mb}} = 92.98 \times 3 \times 10^3$$

$$\Rightarrow 0.9\,(p - 1 \times 22) = \frac{3 \times 92.98 \times 10^3 \times 1.25}{10 \times 410} = \cancel{0.029}\,85.04$$

$$\Rightarrow P = \frac{\overset{85.04}{\cancel{0.029}}}{0.9} + 22 = 116.49\ \text{mm.}$$

Minimum pitch, $P_{min} = 50\text{mm} < 120\text{mm}$.

Provide pitch = 120mm, which is shaped.

min. edge end distance = 33mm < 40mm.

For bearing strength:-

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

$$V_{npb} = f_u \times 2.5 \times k_b \times dt$$

$$k_b = 0.60.$$

$$\Rightarrow V_{npb} = \cancel{40\times}\ 2.5 \times 0.60 \times 20 \times 10 \times 410$$

$$= 123\ \text{kN.}$$

$$V_{dpb}' = \frac{123}{1.25} = 98.4\ \text{kN.} > 92.98\ \text{kN.}$$

Hence, the design is shaped.

# Welded Connections :-

→ When 2 structural members are connected by means of weld, the connection is called as welded connection, which develops a metallurgical bond between them.

→ The members or elements to be connected are brought closer and the metal is melted by means of electric arc or oxyacetylene flame alongwith weld rod, which adds metal to the joint. After cooling the bond is established between the 2 elements.

# Types of Welded Joints :-

→ Basically, 3 types of welded joints are available :

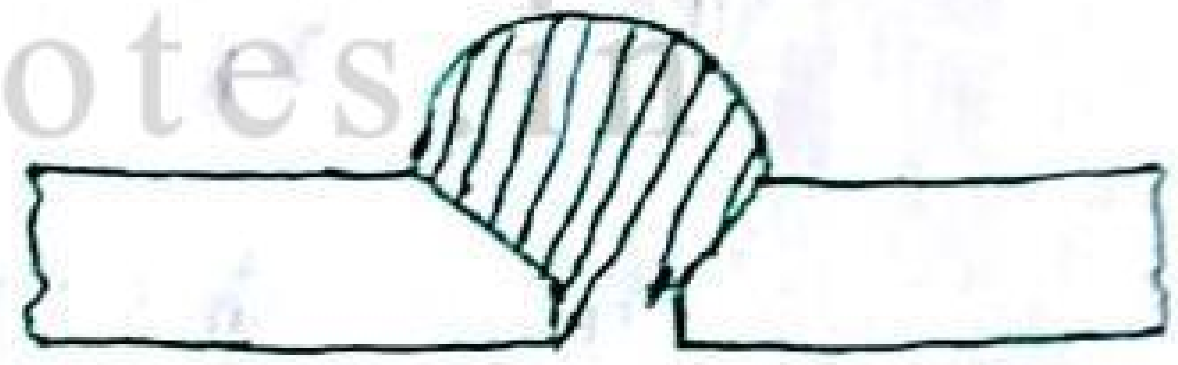
i) Butt / Grove weld

ii) Fillet weld

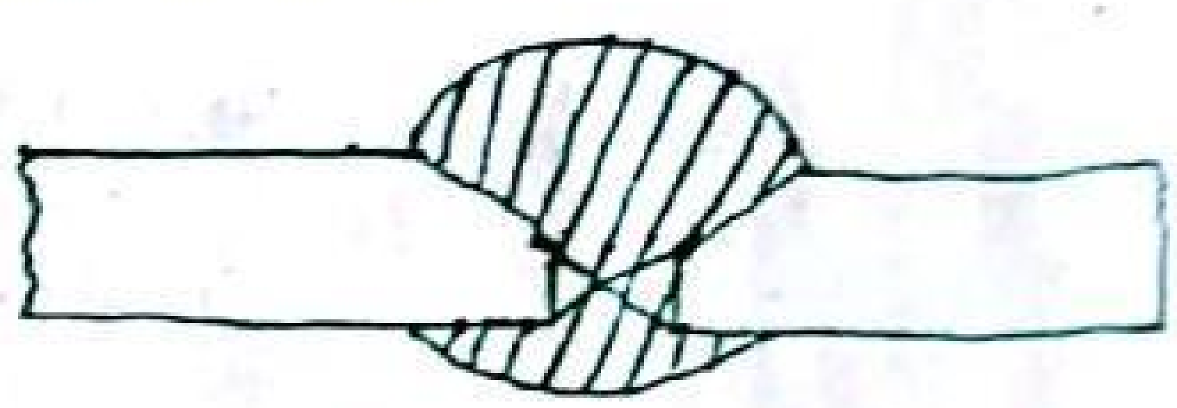
iii) Slot weld.

i) Butt / Grove Weld :-

→ Butt weld is also known as grove weld. Depending upon the shape of the weld, it can be classified as : Single-V, Double-V, Square, Single-U, Single-J



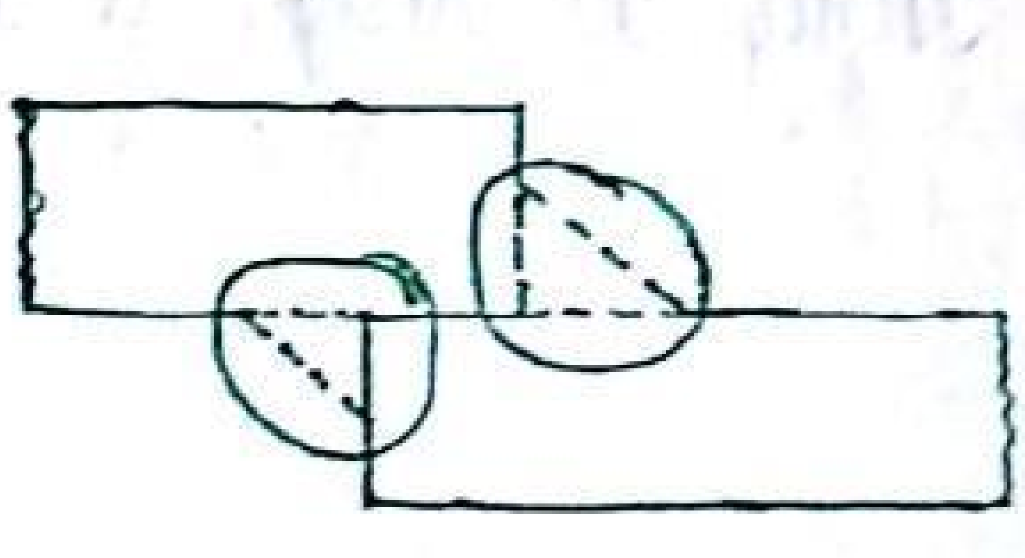
(Single - V)



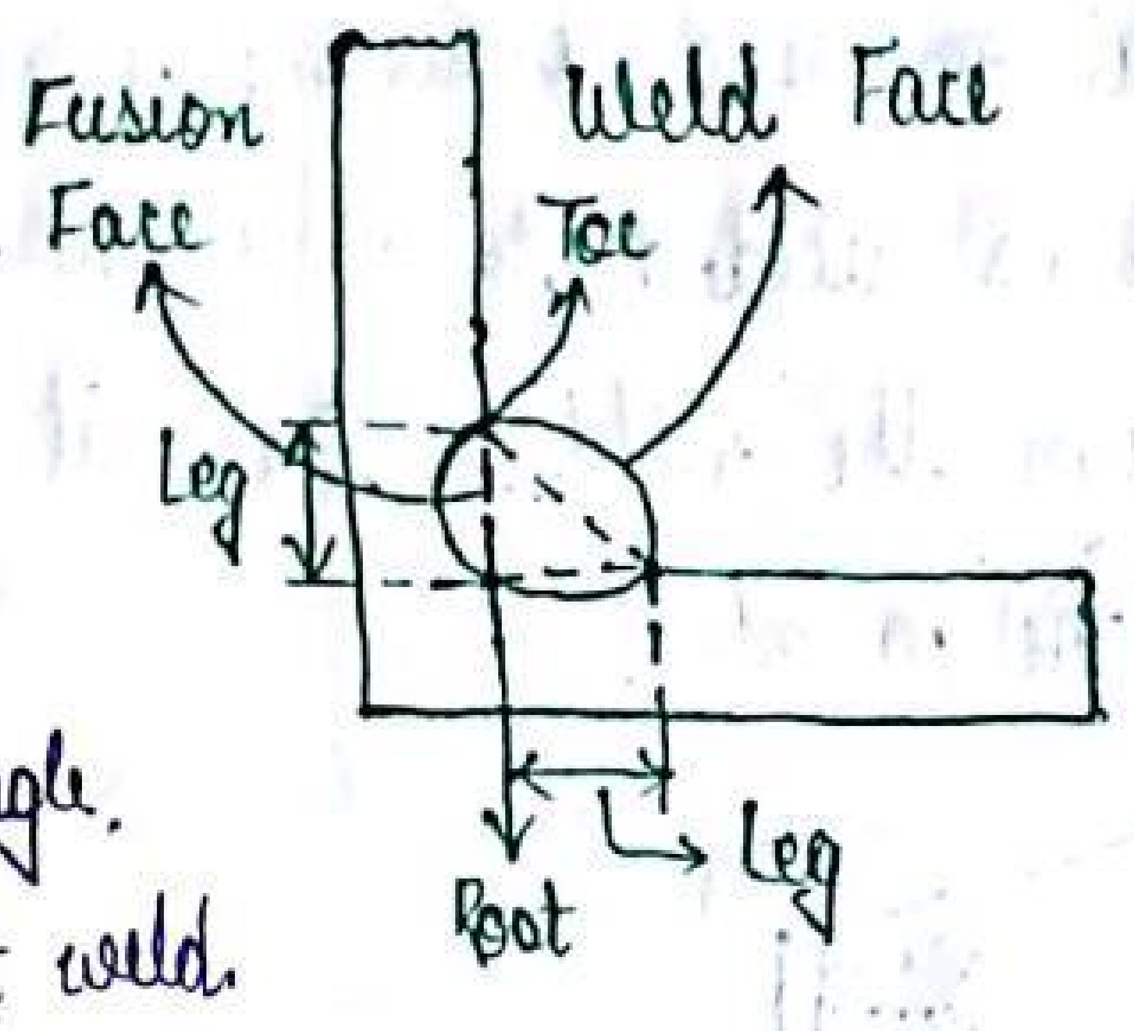
(Double - V)

ii) Fillet Weld :-

→ Fillet weld is the weld of approximately triangular cross-sectional joining 2 surfaces



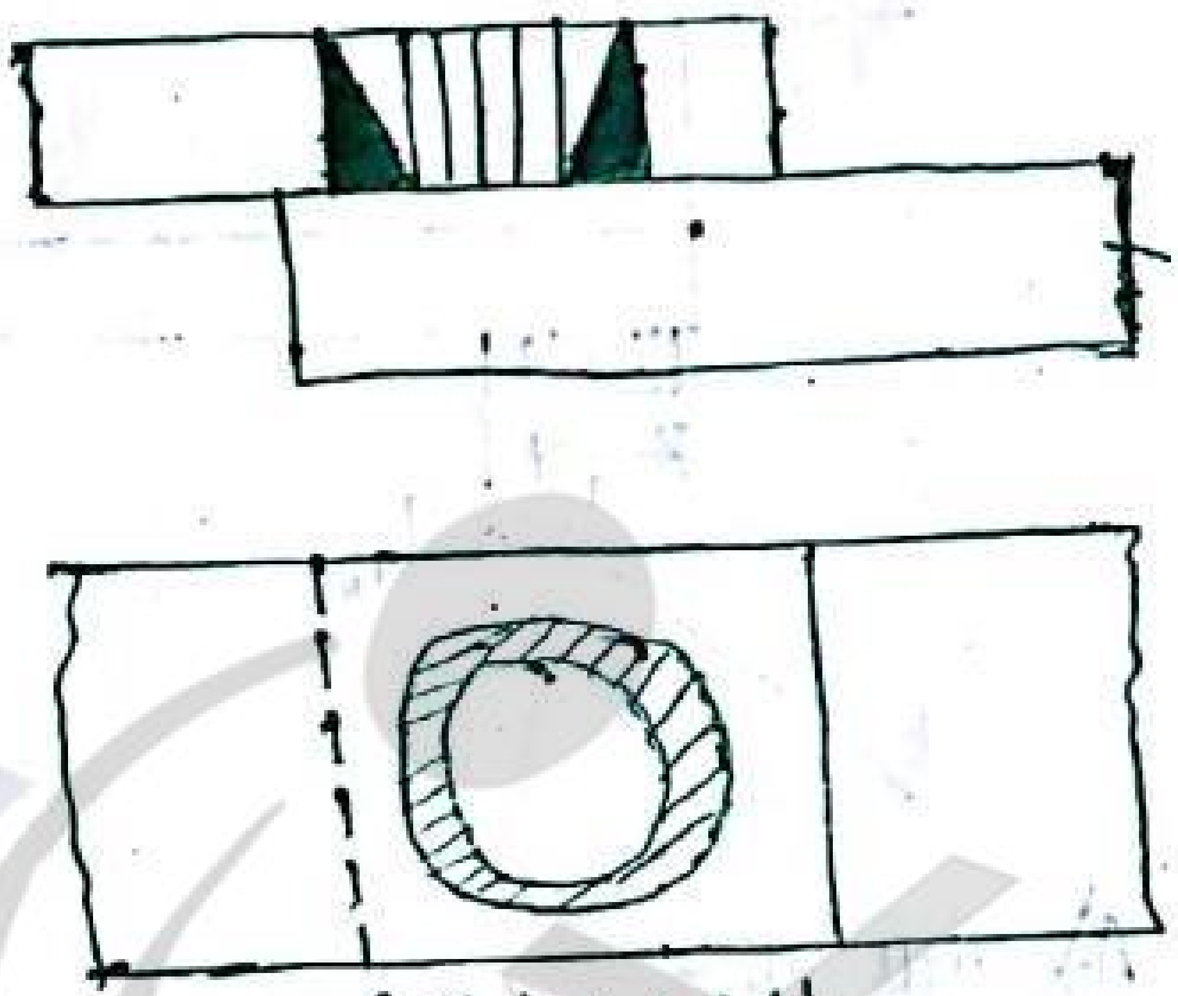
approximately at right angle with each other in lap joint (or) in T-joint.




→ When the cross-section of the fillet weld is an isosceles triangle, it is known as standard fillet weld.

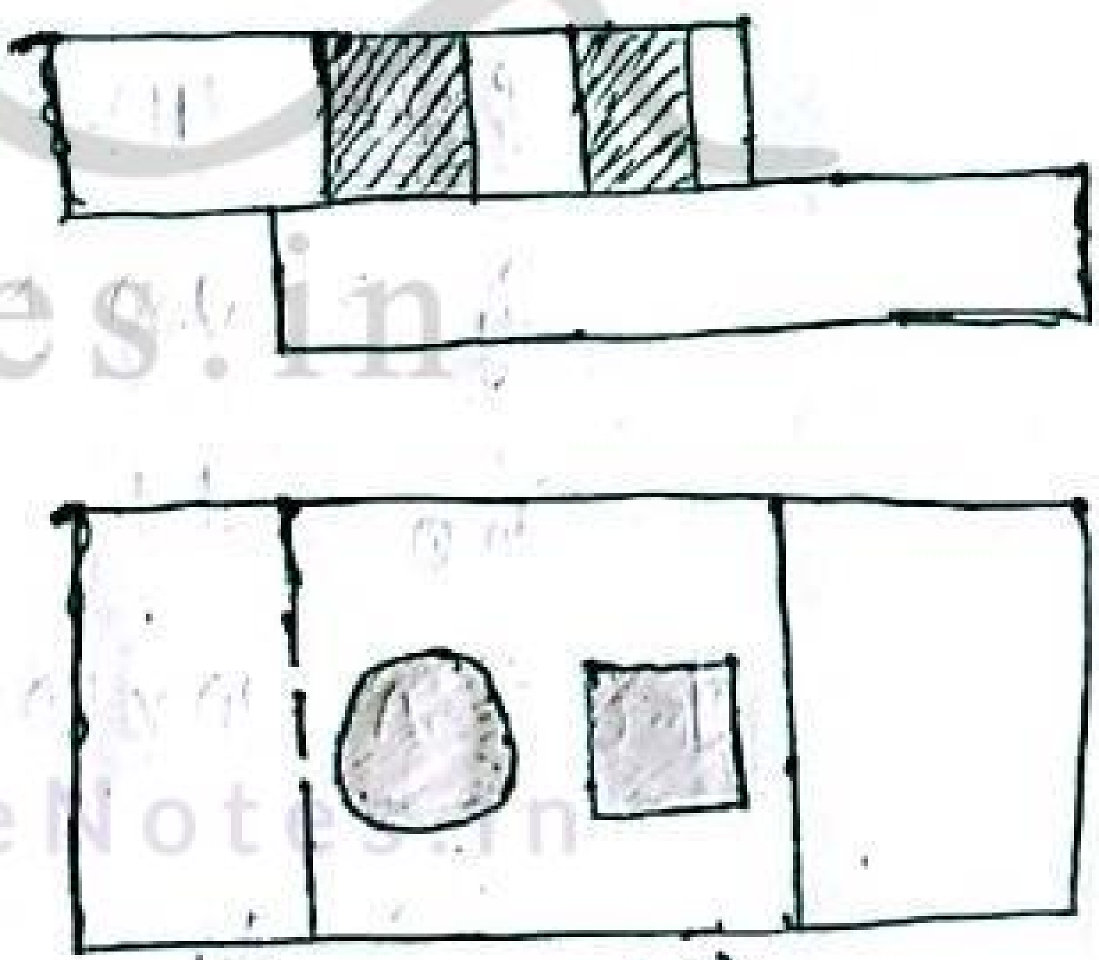
iii) Slot / Plug Weld :-

→ The figure-1 shows a typical slot weld, in which a plate with circular hole is kept with another plate to be connected & then fillet welding is made along the periphery of the hole.



(Slot Weld)

→ The figure-2 shows the plug weld, in which small holes are made in 1 plate & is kept on another plate to be connected & then the entire hole is filled with the filler material or weld material.



(Plug Weld)

# Strength of The Weld :-

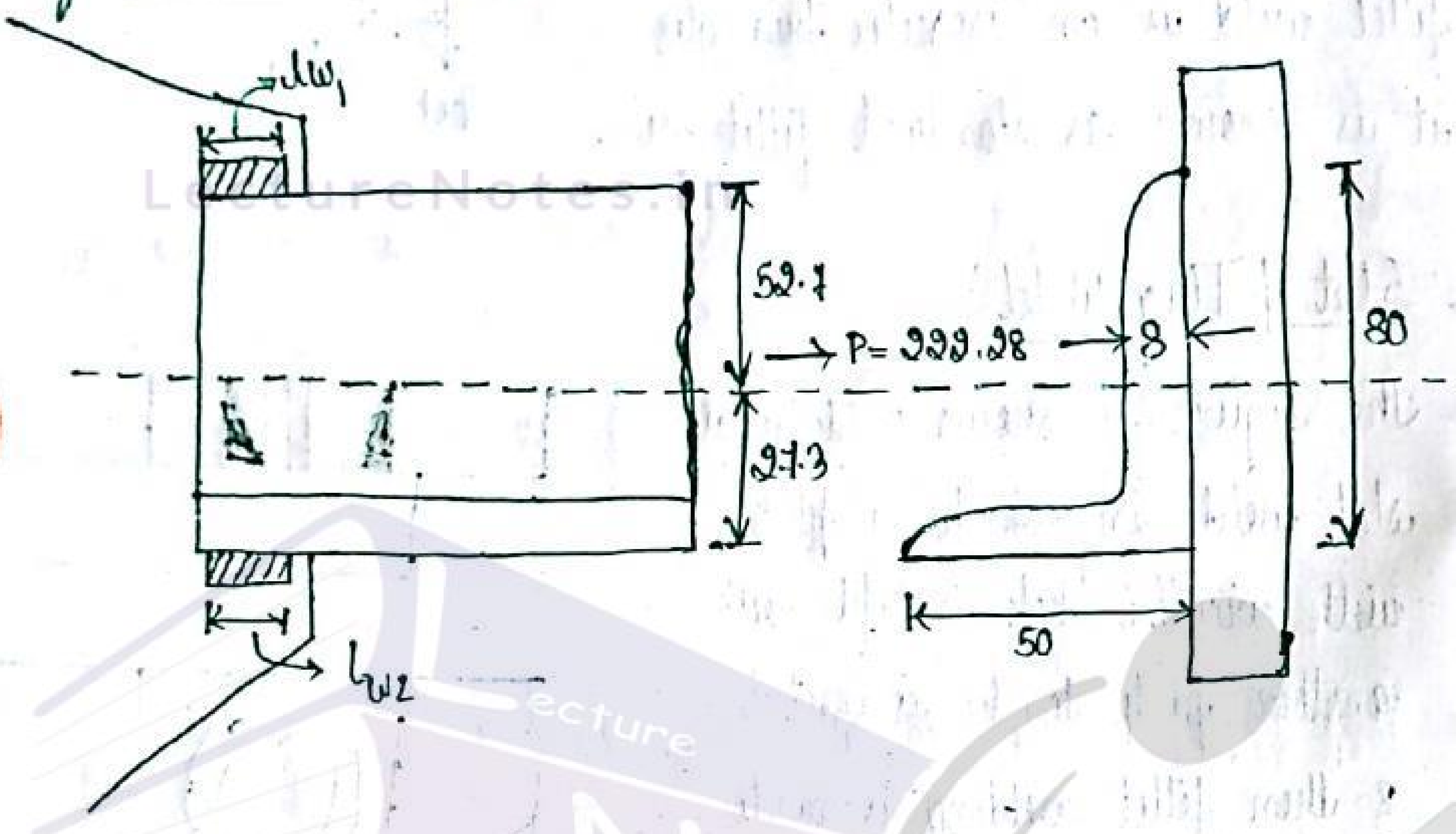
$$\boxed{\text{Strength of the weld} = \frac{l_w \, t_t \, f_u}{\sqrt{3}\, \gamma_{mw}}}$$

where, $l_w$ = length of the weld.

$t_t$ = throat thickness.

$f_u$ = ultimate strength of the plate.

Q) A tie member consisting of an ISA 80 x 50 x 8 mm of Fe410 grade steel is welded to a 12 mm thick gaussette plate at side. Design the weld to transmit a load equal to design strength of the member.




A) Given:-

For Fe410 grade steel:

$f_u$ = 410 N/mm²

$f_y$ = 250 N/mm²

$\gamma_{m0}$ = 1.1

For ISA 80 x 50 x 8, the cross area of the member, $A_g$ = 9.78 cm²

$\Rightarrow A_g$ = 978 mm².

The depth of the neutral axis from the bottom:

$C_{x\text{-}x}$ = 2.73 cm = 27.3 mm.

For design strength of the member: (P-32, C-6.2)

$$T_{dg} = \frac{A_g f_y}{\gamma_{m0}} = \frac{978 \times 250}{1.1} = 222.28 \text{ KN}$$

To avoid eccentricity, the C.G. of the weld should coincide with the centroidal axis of the member to give maximum strength of the weld. Provide the weld $l_{w_1}$ & $l_{w2}$ as shown in the figure.

Let $P_{w1}$ is the strength of the weld having length $l_{w1}$ & $P_{w2}$ is the strength of the weld having length $l_{w2}$.

By applying equilibrium equation, we have :

$$P_{w1} + P_{w2} = 222.28 \text{ kN}.$$

Taking moment at the bottom end corner = 0 :-

$$P_{w1} \times 80 = 222.28 \times 27.3 \times 10^3$$

$$\Rightarrow P_{w1} = 75.85 \cancel{kN} \times 10^3 \text{ N}$$

$$\therefore P_{w2} = 222.28 - 75.85 = 146.43 \cancel{kN} \times 10^3 \text{ N}$$

Now, $$\frac{l_{w1} \times t_t \times f_u}{\sqrt{3} \times \gamma_{mw}} = 75.85 \times 10^3 \text{ N}.$$

Throat ~~Hoor~~ thickness :

Min$^m$ size of the fillet weld (P-78, T-21) = 5mm

Max$^m$ size of the weld [P-80, F-17(a), 17(b)], taking round edge connection $= t - \frac{1}{4}t$

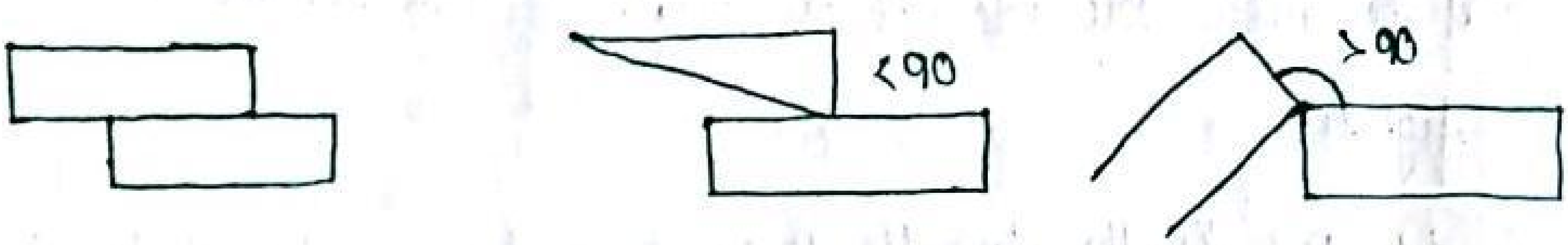
$$= 8 - \left(\frac{1}{4} \times 8\right).$$

$$= 6\text{mm}.$$

$t_t = K \times S$ , $S$ = size of the weld [P-78, T-21]

K = constant

To have better safety in the welded joint, the 2 fusion faces of the plate should be connected at 90°.




$k = 0.7$

$\Rightarrow t_t = k \times S = 0.7 \times 6 = 4.2$ mm.

Now, $\dfrac{l_{w_1} \times t_t \times f_u}{\sqrt{3} \times \gamma_{mw}} = 75.85 \times 10^3$

$$\Rightarrow l_{w_1} = \frac{75.85 \times 10^3 \times \sqrt{3} \times \gamma_{mw}}{4.2 \times 410} = 115 \text{ mm}.$$

By equating the rightward force with the leftward force, we have:

$$P_{w_1} + P_{w_2} = 222.27 \times 10^3$$

$$\Rightarrow 75.84 \times 10^3 + P_{w_2} = 222.27 \times 10^3$$

$$\Rightarrow P_{w_2} = 146.43 \times 10^3.$$

Now, $\dfrac{l_{w_2} \times t_t \times f_u}{\sqrt{3} \times \gamma_{mw}} = 146.43 \times 10^3$.

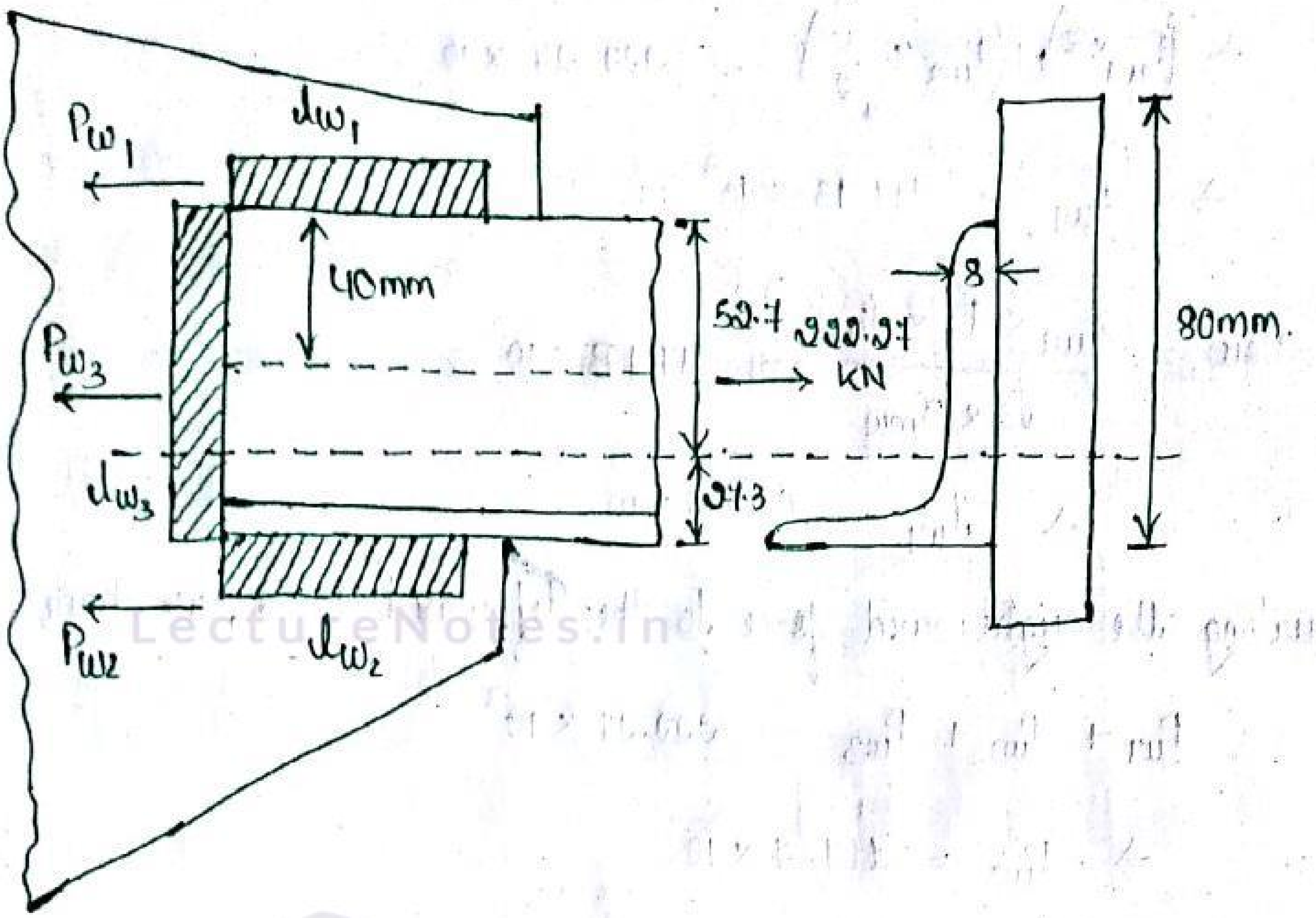
$$\Rightarrow l_{w_2} = \frac{146.43 \times 10^3 \times \sqrt{3} \times 1.5}{4.2 \times 410} = 221 \text{ mm}.$$

Q) A tie member consisting of an ISA 80×50×8 mm of Fe410 grade steel is welded to a 12mm thick gaussette plate at side but the face of connected members from square. Design the weld to transm a load equal to design strength of the member.

A) Max$^m$ size of the weld for square edge, $S = t - 1.5 = 6.5$ mm.

Min$^m$ size of the weld (S) = 5mm.

∴ Take size of the weld = 6.5mm = S.




Throat thickness :

$$t_t = k \times S = 0.7 \times 6.5 = 4.55\text{mm}.$$

{For the fusion angle - 90° [P-78, T-22]}

As we have to provide weld in the 3 sides of the member having length $l_{w_1}$, $l_{w2}$, $l_{w3}$ at the top, bottom & side face of the member respectively.

Assume, $P_{w1}$, $P_{w2}$, $P_{w3}$ are the strength of the weld for length $l_{w1}$, $l_{w2}$, $l_{w3}$ respectively. As the side face welding length is same as the length of 1 leg of member.

$$l_{w3} = 80\text{mm}$$

$$\text{So, } P_{w3} = \frac{l_{w3} \times t_t \times f_u}{\sqrt{3} \times \gamma_{mw}}$$

$$= \frac{80 \times 4.55 \times 410}{\sqrt{3} \times 1.5} = 57.45 \times 10^3 \text{ N}.$$

By applying the equilibrium eq$^n$ & taking the moment at the bottom lower left corner = 0.

$$\text{We have: } T_{dg} = \frac{A_g \times f_y}{\gamma_{m0}} = \frac{978 \times 250}{1.1} = 222.27 \text{ kN}.$$

$$\Rightarrow \left(P_{w1} \times 80\right) + \left(P_{w3} \times \frac{80}{2}\right) = 222.27 \times 10^3$$

$$\Rightarrow P_{w1} = 47.13 \times 10^3 \text{ N}.$$

Now, $$\frac{d_{w1} \times t_t \times f_u}{\sqrt{3} \times \gamma_{mw}} = 47.13 \times 10^3$$

$$\Rightarrow d_{w1} = 65.63 \text{ mm}.$$

Equating the rightward force to the leftward force, we have:

$$P_{w1} + P_{w2} + P_{w3} = 222.27 \times 10^3$$

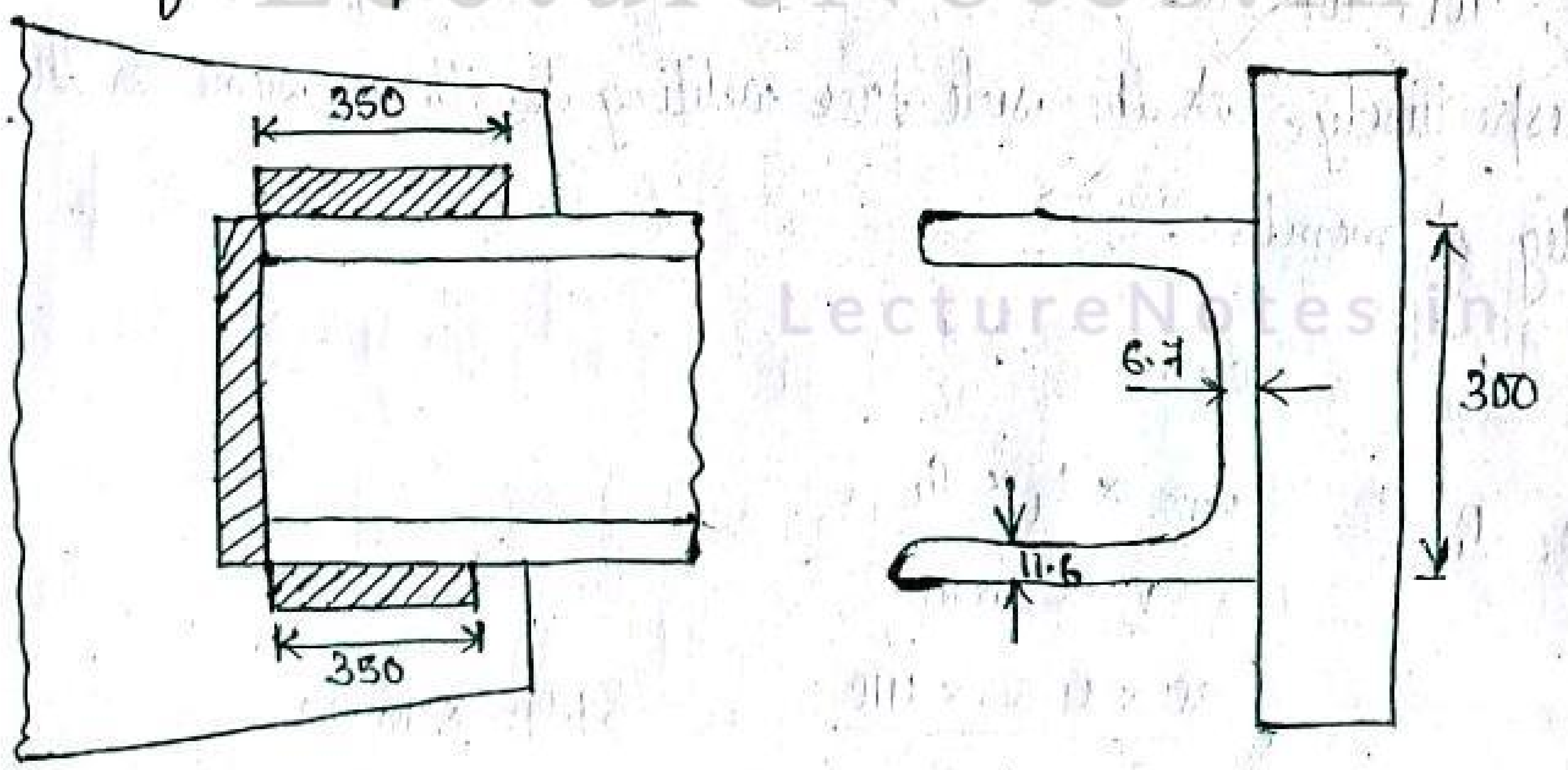
$$\Rightarrow P_{w2} = 117.7 \times 10^3$$

$$\Rightarrow \frac{d_{w2} \times t_t \times f_u}{\sqrt{3} \times \gamma_{mw}} = 117.7 \times 10^3$$

$$\Rightarrow d_{w2} = 163.93 \text{ mm}.$$

3.) An ISLC 300 @ 324.7 N/m of Fe410 grade steel is to carry a factored tensile load 900kN. The channel section is to be welded at the side to a gusset plate of 12mm thick. Design the fillet weld if overlap is limited to 350 mm.

A)




For ISLC 300 @ 324.7 N/m, we have gross sectional area:

$$A_g = 42.11 \text{ cm}^2 = 4211 \text{ mm}^2.$$

Thickness of the flange $(t_f)$ = 11.6mm.

Thickness of weld, $(t_w) = 6.7mm$

Height of section, $h = 300mm$.

The overlap limit = 350mm at top & bottom.

So, max$^m$ length of weld available = $350 + 350 + 300 = 1000 mm$.

min$^m$ size of weld = 5mm.

size (max$^m$ – square edge) = $t - 1.5$

$= 6.7 - 1.5 = 5.2mm$.

Provide size of weld $(S) = 5.5mm$.

Throat thickness for fusion angle of $90°$, $t_t = K \times S$

$= 0.7 \times 5.5$

$= 3.85mm$.

The strength of the weld for available max$^m$ length $= \dfrac{l_w \times t_t \times f_u}{\sqrt{3} \times \gamma_{mw}}$

$$= \frac{1000 \times 3.85 \times 410}{\sqrt{3} \times 1.5} = 607.56 \text{ KN}.$$

The extra load that cant be carried by the web $= 900 - 607.56$

$= 292.44 KN$.

To carry the extra amount of load, provide 2 slot weld by size & thickness same as provided weld.

Let the length of the weld to be provided in the 4 faces of the slot welds be '$L_w$'.

strength of slot weld $= \dfrac{l_w \times t_t \times f_u \times 4}{\sqrt{3} \times \gamma_{mw}}$

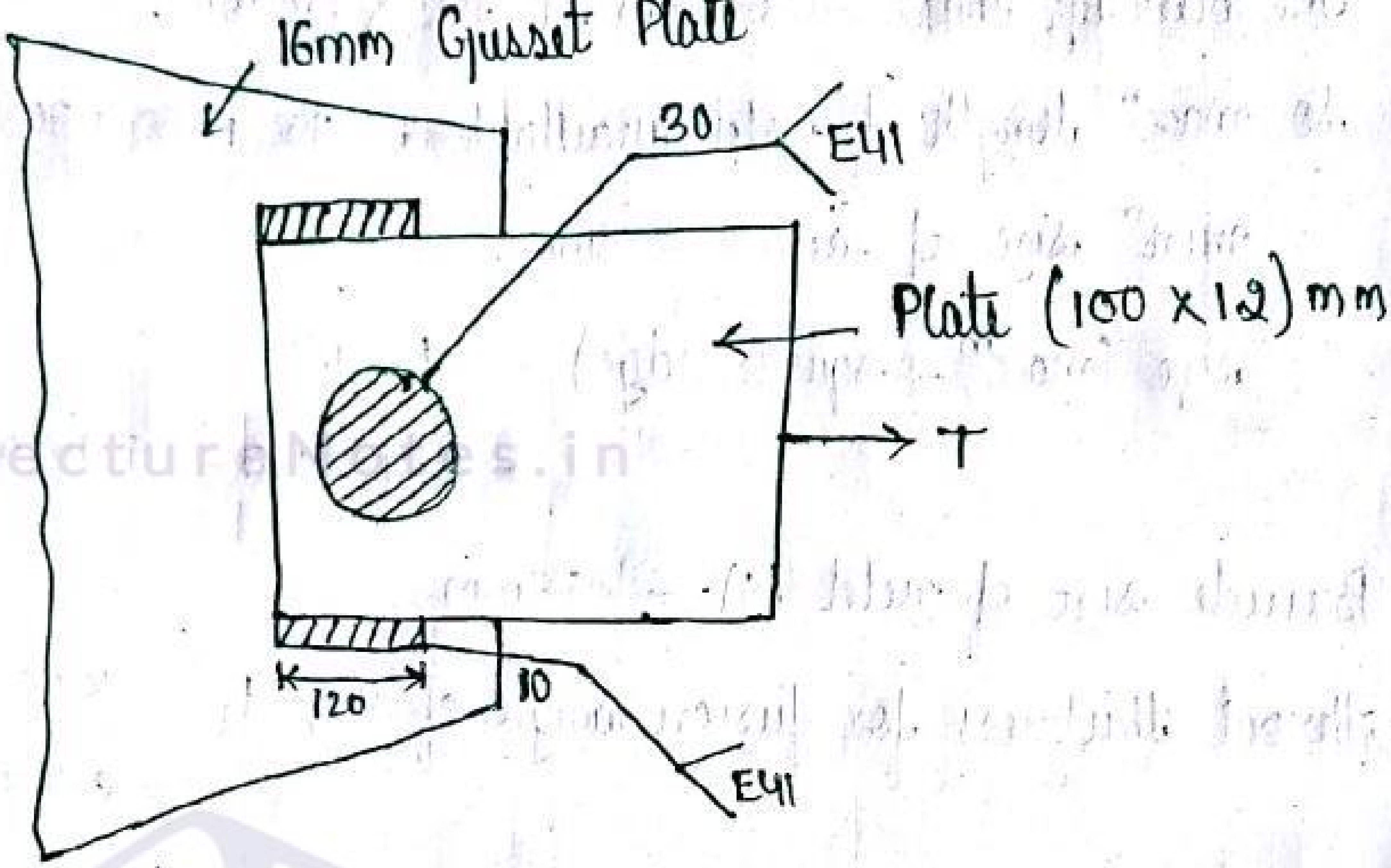
Equating the strength of weld with the available load, we have:

$$\frac{l_w \times t_t \times f_u}{\sqrt{3} \times \gamma_{mw}} \times 4 = 292.44 \times 10^3$$

$$\Rightarrow l_w = 120.33mm \simeq 125mm.$$

Provide 2 slot welds of length 125mm to carry the extra load.

Q.) Determine the service load permitted on the connection as shown in the figure. Assume grade of the steel Fe410 & the welding is done at side.




A) $f_u = 410 \text{ N/mm}^2$.

$f_y = 250 \text{ N/mm}^2$.

For strength of the weld :-

i) Fillet weld :-

$$\text{Strength of the weld} = \frac{l_w \, t_t \, f_u}{\sqrt{3}\, \gamma_{mw}}$$

$$l_w = 2 \times 120 = 240 \text{ mm}.$$

Size of the weld $= S = 10$ mm.

Throat thickness $= 0.7 \times 10 = 7$ mm (for 90° fusion face)

$$\Rightarrow \text{strength of the weld} = \frac{240 \times 7 \times 410}{\sqrt{3} \times 1.5} = 265.12 \text{ kN}.$$

ii) Plug weld :-

$$\text{Strength of the weld} = \frac{\left(\frac{\pi}{4} \times d^2\right) f_u}{\sqrt{3}\, \gamma_{mw}}$$

$$= \frac{\frac{\pi}{4} \times 30^2 \times 410}{\sqrt{3} \times 1.5} = 111.15 \text{ kN}$$

Strength of the total weld = (265.12 + 111.15) kN

= 376.62 kN.

For strength of the plate:-

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

$$= \frac{100 \times 12 \times 250}{1.10} = 272.73 \text{ kN}$$

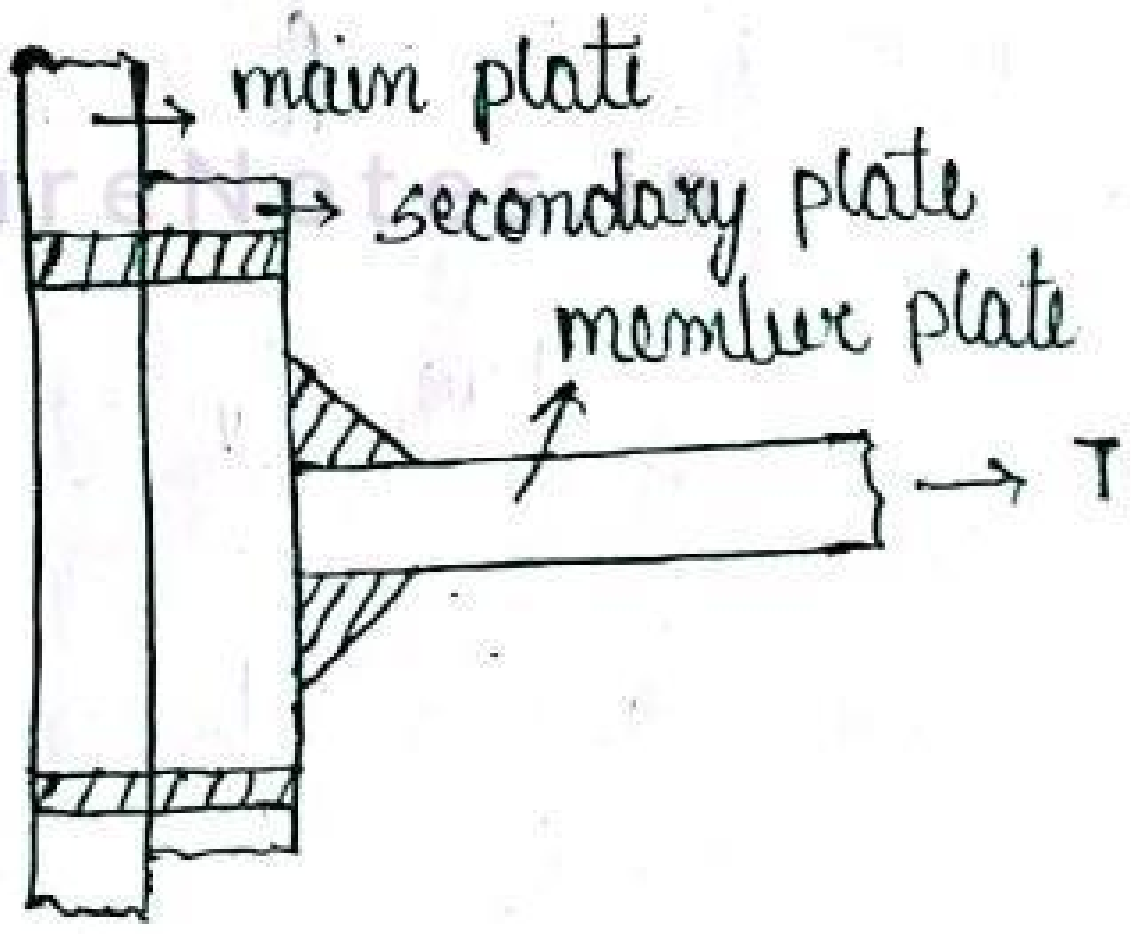
The strength of the connection is equal to lesser of strength of the weld & the strength of the plate.

∴ Strength of the connection = 272.73 kN.

∴ The service load the connection can resist = $\frac{272.73}{1.5}$ = 181.82 kN

where, Factor of safety = 1.5.

# Prying Force:- [P-77-10.4.7]

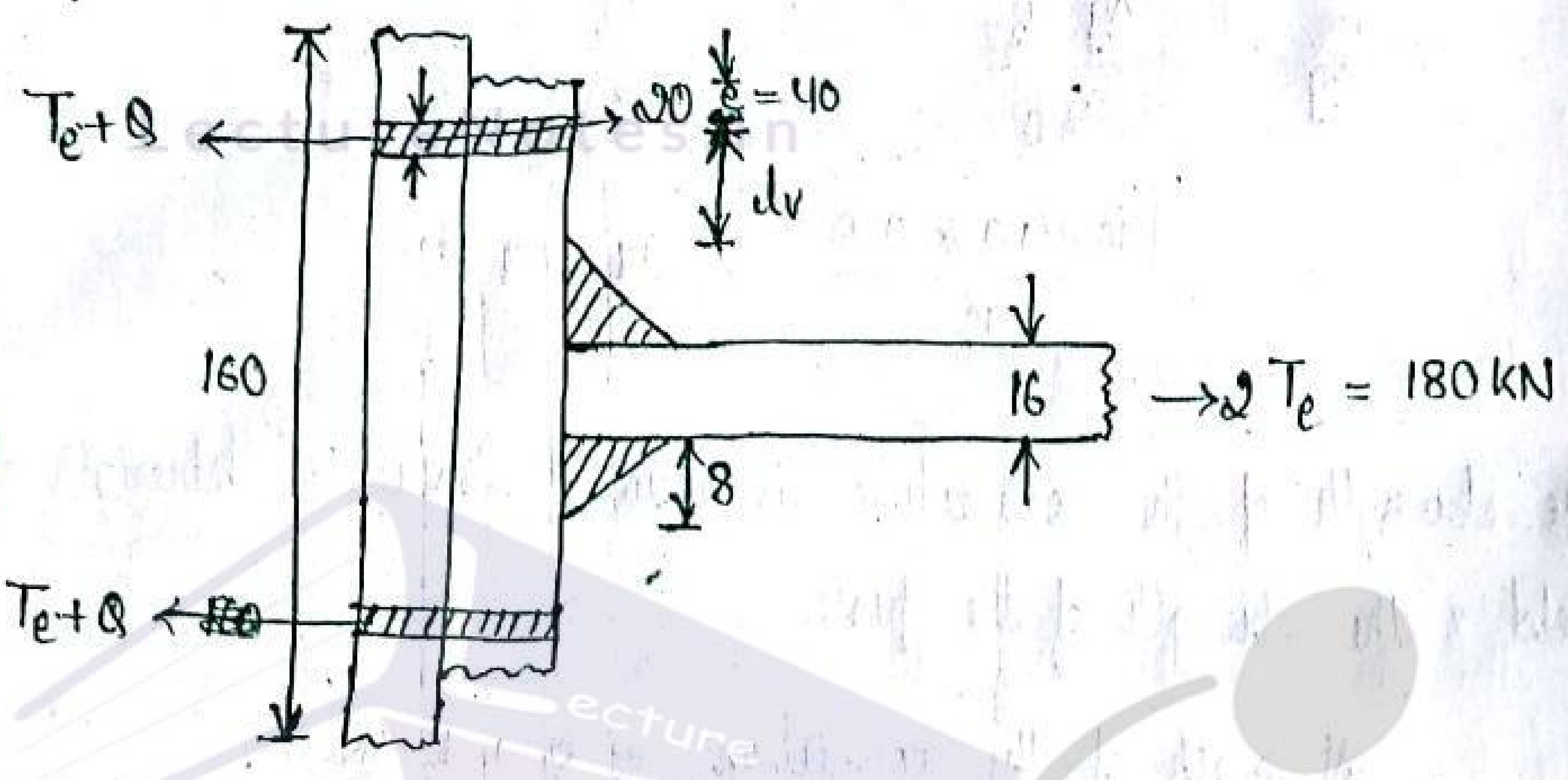



→ When the available members as per the steel table is not sufficient to carry the external load, then we have to design the member manually.

→ For that case, as shown in the figure, the connection will have an extra tension i.e., 'Q'. This extra tension will give pre-tensioning to the bolts or connection.

This extra tension is known as prying force.

Q) The joint shown in the figure has to carry a factored load of 180 kN. The end plate used is of size (160 × 140 × 16) mm. The bolts used are of M20 & HSFG 8.8. Check whether the connection is safe or not.




A) Grade of the bolt = HSFG 8.8, $f_{ub}$ = 830 N/mm²

Let's assume, grade of the plate is Fe410.

$\Rightarrow f_u = 410\ N/mm^2$ & $f_y = 250\ N/mm^2$.

Minimum bolt tension at the time of installation:

$$f_0 = 0.7 \times f_{ub} = 0.7 \times 830 = 581\ N/mm^2.$$

$$\text{Now, } l_v = \frac{160}{2} - 40 - 8 - \frac{16}{2} = 24\ mm.$$

$$l_e = 1.1 \times t \times \sqrt{\frac{\beta f_0}{f_y}}$$

$$= 1.1 \times 16 \times \sqrt{\frac{1 \times 581}{250}}$$

$$= 26.83\ mm.$$

$\beta = 1$, $\eta = 1.5$, $b_e = 140mm$

$$\therefore\ Q = \frac{l_v}{2l_e}\left[T_e - \frac{\beta \eta f_0 b_e t^4}{27 l_e l_v^2}\right]$$

$$= \frac{24}{2 \times 26.83}\left[90 - \frac{1 \times 1.5 \times 581 \times 140 \times 16^4}{27 \times 26.83 \times 24^2}\right]$$

$$= 0.44\ (90 - 19163.21)$$

$$= 0.44\ (90 - 19.163)$$

$$= 31.68 \text{ kN.}$$

As the strength of the bolt is lesser in shearing.

Hence, shearing strength of the bolt $= \dfrac{f_{ub}}{\sqrt{3}\,\gamma_{mw}}\left(n_n A_{nb} + \cancel{n_s A_{sb}}^{\,0}\right)$

$$= \frac{830}{\sqrt{3} \times 1.25} \times \left(1 \times 0.78 \times \frac{\pi}{4} d^2\right)$$

$$= 93.94 \text{ kN} \qquad (d = 20).$$

Hence, the design is safe.

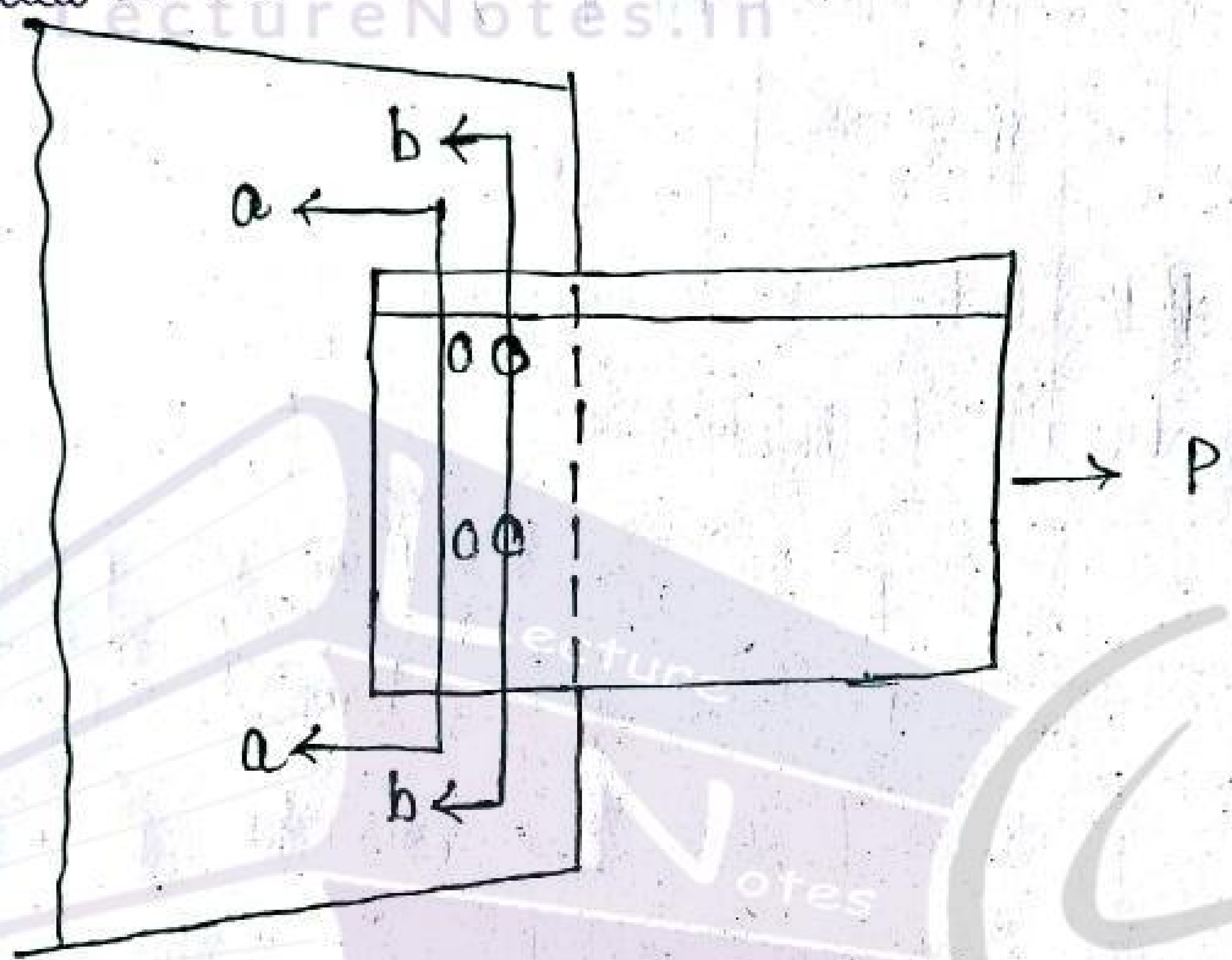


END OF MODULE - 1

# Tension Members

## # Tension Members:-

→ A structural member subjected to its pulling force at its end is called as the tension members, as shown in the figure.




→ It is also known as tie member.



## # Net Area or Net Section:-

→ The reduced area at the section (b-b) (figure) is referred as net area & the reduced section is called as net section.

## # Gross Area or Gross Section:-

→ The un-reduced area at the section (a-a) (figure) is the gross area of the section.

→ The net sectional area of a tension member is equal to gross sectional area of the member - the sectional area of maximum no. of bolt holes present at that section only.

# Net-Sectional Area of The Plate :-

→ The net sectional area of the plate in the tension member to a force, 'T' & provided with chain bolting is given as follows:

$$\boxed{A_n = (b - nd_h)t}$$ → [P-32, 6.3.1]

i) for threaded rods → [P-33, 6.3.2]

ii) angles & tees → [P-33, 6.3.3]

↳ for angle & Tee connected through one of its elements (leg), the net area so obtained will be reduced & is called "effective net area".

iii) for other sections → [P-33, 6.3.4]

# Types of Failure :-

→ A tension member may fail in any of the following reasons :-

i) Gross - section failure → [P-32, 6.2]

ii) Net - sectional rupture → [P-33, 6.3]

iii) Block - shear failure → [P-33, 6.4]

→ The factored design tension load in the member should be less than the design strength of the member.

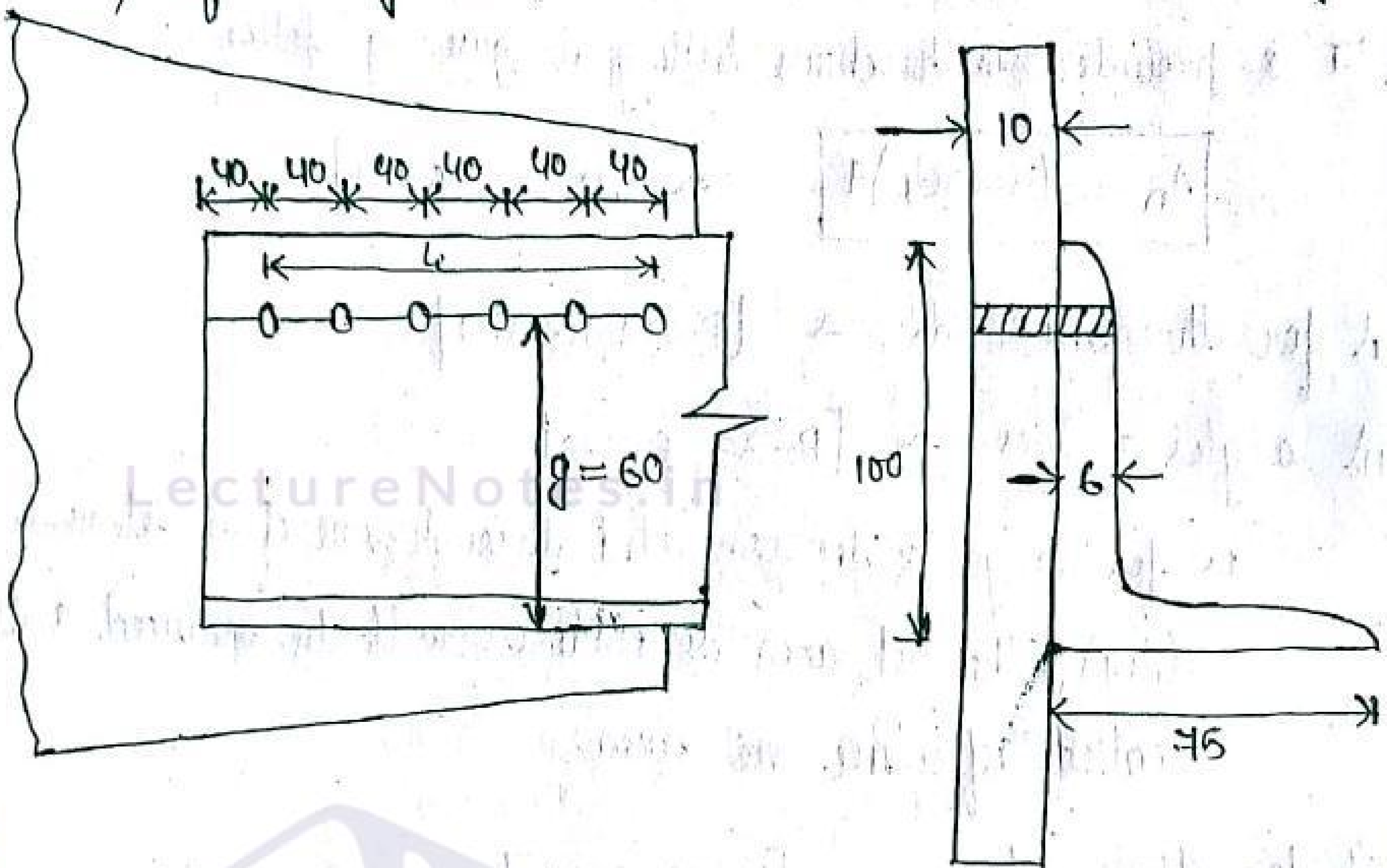
→ The design strength of the member under the axial tensile load is the lowest between the above 3.

# Analysis Problem :-

Q.) A single unequal angle of (100 × 75 × 6) mm is connected to a 10mm thick gusset plate, at the ends with (6 - 16φ mm) bolt to transfer tension as shown in the figure. Determine the design tensile strength of the angle. Assuming that the grade of steel is Fe410 & grade of the bolt is 4.6 for the following conditions :-

i) If the gusset plate is connected to 100mm leg.

ii) If the gusset plate is connected to 75mm leg.




Here, $g$ = 40mm for the 75mm leg connection.

$g$ = 60mm for the 100mm leg connection.

From the steel table, gross area of the angle section, $A_g$ = 1010 $mm^2$.

For Fe410 grade steel :-

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

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

[If no bolting is given, $\gamma_{m1}$ = 1.25 should be taken.]

d = 16mm.

$\Rightarrow d_0 = (16+2) = 18$ mm

$\gamma_{m0}$ = 1.10

$\gamma_{m1}$ = 1.25

The design strength of the member will be lesser of gross-sectional failure, net-sectional rupture or block shear failure.

i) Strength in the gross-sectional failure : (P-32, 6.2)

$$T_{dg} = \frac{A_g f_y}{\gamma_{m0}} = \frac{1010 \times 250}{1.1} = 229.54 \text{ kN}$$

ii) Strength in net-sectional rupture: (P-32, 6.3.3)

$A_{nc}$ = net area of the connected leg = $(100\times6) - (18\times6) = 492\ mm^2$.

$A_{go}$ = gross area of the outstanding leg = $(75\times6) = 450\ mm^2$.

$$B = 1.4 - \left[0.076\left(\frac{w}{t}\right)\times\left(\frac{f_y}{f_u}\right)\times\left(\frac{b_s}{L_c}\right)\right]$$

$$= 1.4 - \left(0.076\times\frac{75}{6}\times\frac{250}{410}\times\frac{129}{40\times5}\right)$$

$$= 1.026$$

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

$$= \frac{0.9\times492\times410}{1.25} + \frac{1.026\times450\times250}{1.1}$$

$$= 250.17\ kN$$

$\boxed{Or}$ $$T_{dn} = \frac{\alpha A_n f_u}{\gamma_{m1}} = \frac{0.8\times(A_{nc}+A_{go})\times410}{1.25}$$

$$= \frac{0.8\times(492+450)\times410}{1.25} = 247.18\ kN.$$

iii) Strength in block shear failure:

The strength due to block shear failure is as follows:-

$$T_{db} = \frac{A_{vg}\cdot f_y}{\sqrt{3}\ \gamma_{m0}} + \frac{0.9\ A_{tn}\cdot f_u}{\gamma_{m1}} \rightarrow (1)$$

$$T_{db} = \frac{0.9\ A_{vn}\cdot f_u}{\sqrt{3}\ \gamma_{m1}} + \frac{A_{tg}\cdot f_y}{\gamma_{m0}} \rightarrow (2)$$

→ Lesser will be considered

$A_{vg}$, $A_{vn}$ = minimum gross & net area along the bolt line ∥ to the external force.

→ $A_{vg} = [(40\times6)\times6] = 1440\ mm^2$

$A_{vn} = 5\times6\times18 + \frac{18}{2}\times6 = 144\ mm^2$

$A_{tg}, A_{tn}$ = minimum gross & net area in tension from the bolt hole to the toe of the angle, Ir to the line of the force.

$\Rightarrow A_{tg}$ = gross area from the bolt hole to the toe

$= (100 - 60) \times 6 = 240 \text{ mm}^2$

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

= net area from the bolt hole to the toe of angle.

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

$= \dfrac{1440 \times 250}{\sqrt{3} \times 1.1} + \left(\dfrac{0.9 \times 186 \times 410}{1.25}\right)$

$= 243.85 \text{ kN}$

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

$= \dfrac{0.9 \times 846 \times 410}{\sqrt{3} \times 1.25} + \dfrac{240 \times 250}{\sqrt{3} \times 1.1}$

$= 198.73 \text{ kN}.$

$T_{db_1} > T_{db_2}$ (lesser one)

strength of member in tension lesser of:

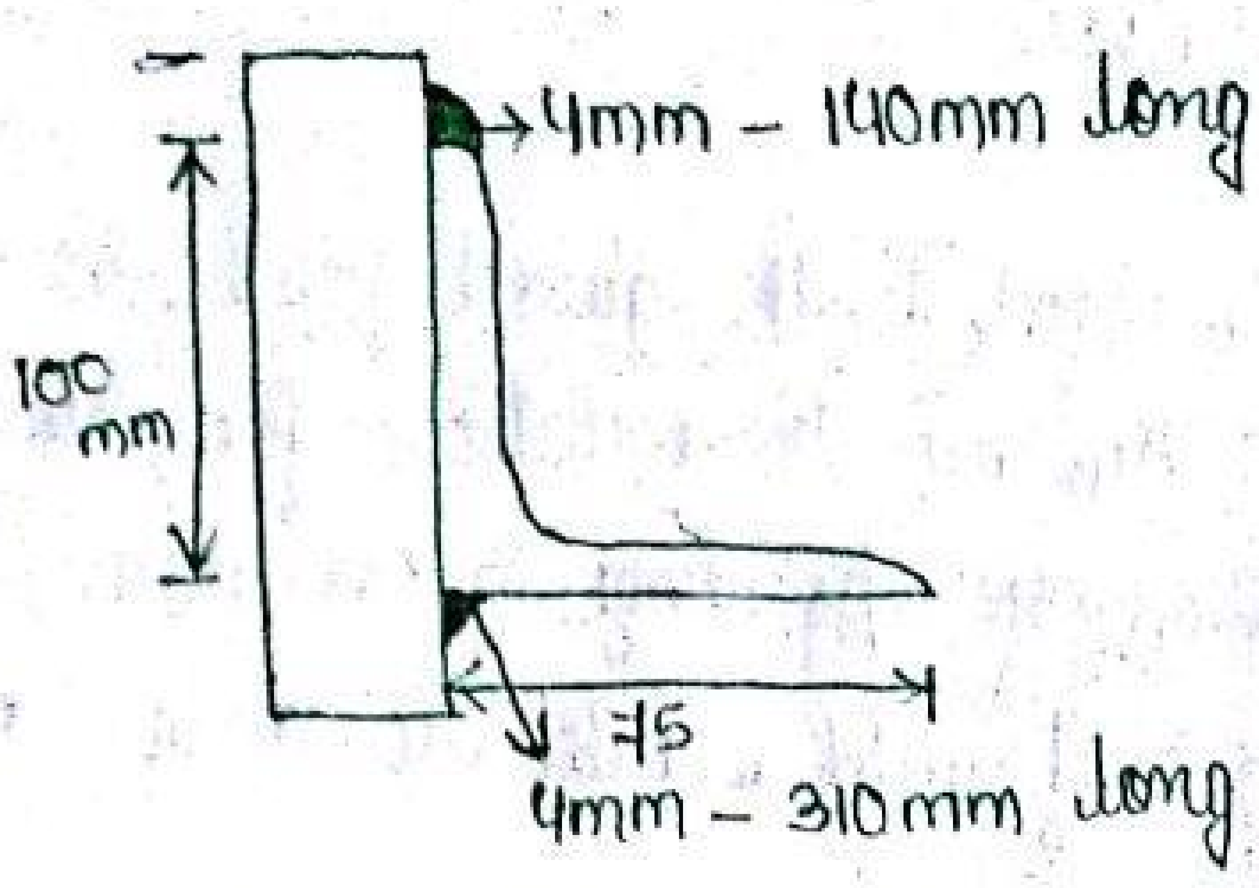
gross sectional failure, $T_{dg} = 229.54$

net cross-section rupture $T_{dn} = 250.17$

Block shear, $T_{db} = 198.73$

$\therefore$ strength $= 198.73 \text{ kN}.$

Q.) Determine the tensile strength of roof truss having a member of $(100 \times 75 \times 6)$ mm connected by a gusset plate by 4mm size weld as shown in figure. Grade of steel is Fe410.




A) for Fe410, $f_y = 250\ N/mm^2$

$f_u = 410\ N/mm^2$

$\gamma_{m0} = 1.1$

$\gamma_{m1} = 1.25$

Gross area of that member, $A_g = 1010\ mm^2$.

Strength of the member is checked as per the code provisions :-

Strength of the member in gross sectional failure :

$$T_{dg} = \frac{A_g f_y}{\gamma_{m0}} = \frac{1010 \times 250}{1.1} = 229.54\ kN$$

Strength of the member in net sectional rupture :

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

$$A_{nc} = \left(100 - \frac{6}{2}\right) \times 6 = 582\ mm^2$$

$$A_{go} = \left(75 - \frac{6}{2}\right) \times 6 = 432\ mm^2$$

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

$w = 75mm$, $t = 6mm$, $f_y = 250\ N/mm^2$

$f_u = 410\ N/mm^2$, $b_s = w = 75mm$, $L_c = 140 + 310 = 450mm$.

$$\Rightarrow \beta = 1.4 - 0.076 \left(\frac{75}{6}\right)\left(\frac{250}{410}\right)\left(\frac{75}{450}\right) = 1.303 > 0.7$$

$$\therefore T_{dn} = \frac{0.9 \times 582 \times 410}{1.25} + \frac{1.303 \times 432 \times 250}{1.1} = 299.73\ mm^2.$$

Alternatively, $T_{dn} = \dfrac{\alpha A_n f_u}{\gamma_{m1}} = \dfrac{0.8 \times 1014 \times 410}{1.25} = 266.07\ kN$

So, take $T_{dn} = 266.07$ kN.

*** Since the member is welded to the gusset plate, no net area is involved, hence $A_{vn}$ & $A_{tn}$ in the equation for block shear failure should be taken as the corresponding gross area. In the calculation of this, the avg. length of weld should be taken. ***

Strength of member in block shear failure:

$A_{vg} = A_{vn} = (140 + 310) \times 4 = 1800 \text{ mm}^2.$

$A_{tg} = A_{tn} = 100 \times 6 = 600 \text{ mm}^2.$ (distance between 2 welds].

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

$$= \frac{1800 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 600 \times 410}{1.25} = 413.3 \text{ kN}$$

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

$$= \frac{0.9 \times 1800 \times 410}{\sqrt{3} \times 1.25} + \frac{600 \times 250}{1.1} = 443.14$$

$$\Rightarrow T_{db_1} < T_{db_2}$$

$\therefore T_{db} = 413.3$ kN

$\therefore$ Strength of member in tension $= 229.54$ kN.

# Design Problem of Tension Member:-

→ Following are the design steps to be followed in the design of tension member:

i) The net area required ($A_n$) to carry the factored load (T) is obtained by the following equation:

For plate, $$\boxed{A_n = \frac{T \times \gamma_{m1}}{0.9 \times f_u}}$$

For single angle, $\boxed{A_n = \dfrac{T \times \gamma_{m1}}{\alpha \times f_u}}$

ii) The net area calculated in step 2 is to be increased by 25% to 40% to find out the gross sectional area.

iii) The required gross area is also obtained from the following eq$^n$:-

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

iv) From the steel table a suitable rolled steel section / built up section providing a gross-section area matching with the computed gross area is to be selected.

v) The no. of bolts required to make the connection is then calculated.

vi) The design strength of member in tension is lesser of $T_{dg}$, $T_{dn}$, $T_{db}$.

vii) The design strength should be more than the factored load.

viii) The slenderness ratio of the member should be checked.

Q) Design a bridge truss diagonal subjected to a factored load of 300 kN. The length of diagonal is 3m. The tension member is connected to a gusset plate of 16mm thick with one line of 20mm dia of bolt of grade 8.8. Assume the grade of plate as per your choice.

A) The grade of the steel is Fe410.

$$f_y = 250 \text{ N/mm}^2$$

$$f_u = 410 \text{ N/mm}^2.$$

The grade of the bolt is 8.8, i.e., $f_{ub} = 830 \text{ N/mm}^2$.

$\gamma_{m0} = 1.1$

$\gamma_{mb} = 1.25$

$\gamma_{ml} = 1.25$

$d = 20mm$, $d_0 = 20 + 2 = 22mm.$

calculation for no. of bolts :-

strength of bolt in shear $= V_{dsb} = \frac{f_{ub}}{\sqrt{3}\gamma_{mw}}(n_n A_{nb} + \cancel{n_n A_{sb}}^{0})$

$$\Rightarrow V_{dsb} = \frac{830}{\sqrt{3} \times 1.25}\left(1 \times \frac{\pi}{4} \times 20^2 \times 0.78\right)$$

$$= 93.9 \text{ kN}.$$

strength of the bolt in bearing $= V_{dpb} = \frac{2.5 K_b d t f_u}{\gamma_{mw}}$

$$\Rightarrow V_{dpb} = \frac{2.5 \times 1 \times 20 \times 16 \times 410}{1.25} \quad (\text{assume } K_b = 1)$$

$$= 262.4 \text{ kN}$$

$\therefore$ The strength of the bolt is 93.9 kN.

$\therefore$ The no. of bolts required for designing the connection $= \frac{300}{93.9}$

$= 3.19$

$\simeq 4$ bolts

Minimum pitch, $P_{min} = 2.5 \times d = 2.5 \times 20 = 50$ mm.

minimum edge-end distance, $e_{min} = 1.5 \times d_0 = 33 mm. \simeq 35mm$

Design of the tension member :-

Provide an angle section as a tension member & equating the factored load with the strength of the member, we have:

net area to be provided, $A_{net} = \dfrac{T \cdot \gamma_{m1}}{\alpha \cdot f_u}$.

$$= \frac{300 \times 10^3 \times 1.25}{0.8 \times 410} = 1143.29 \text{ mm}^2$$

Increasing the net area by 25% we have:

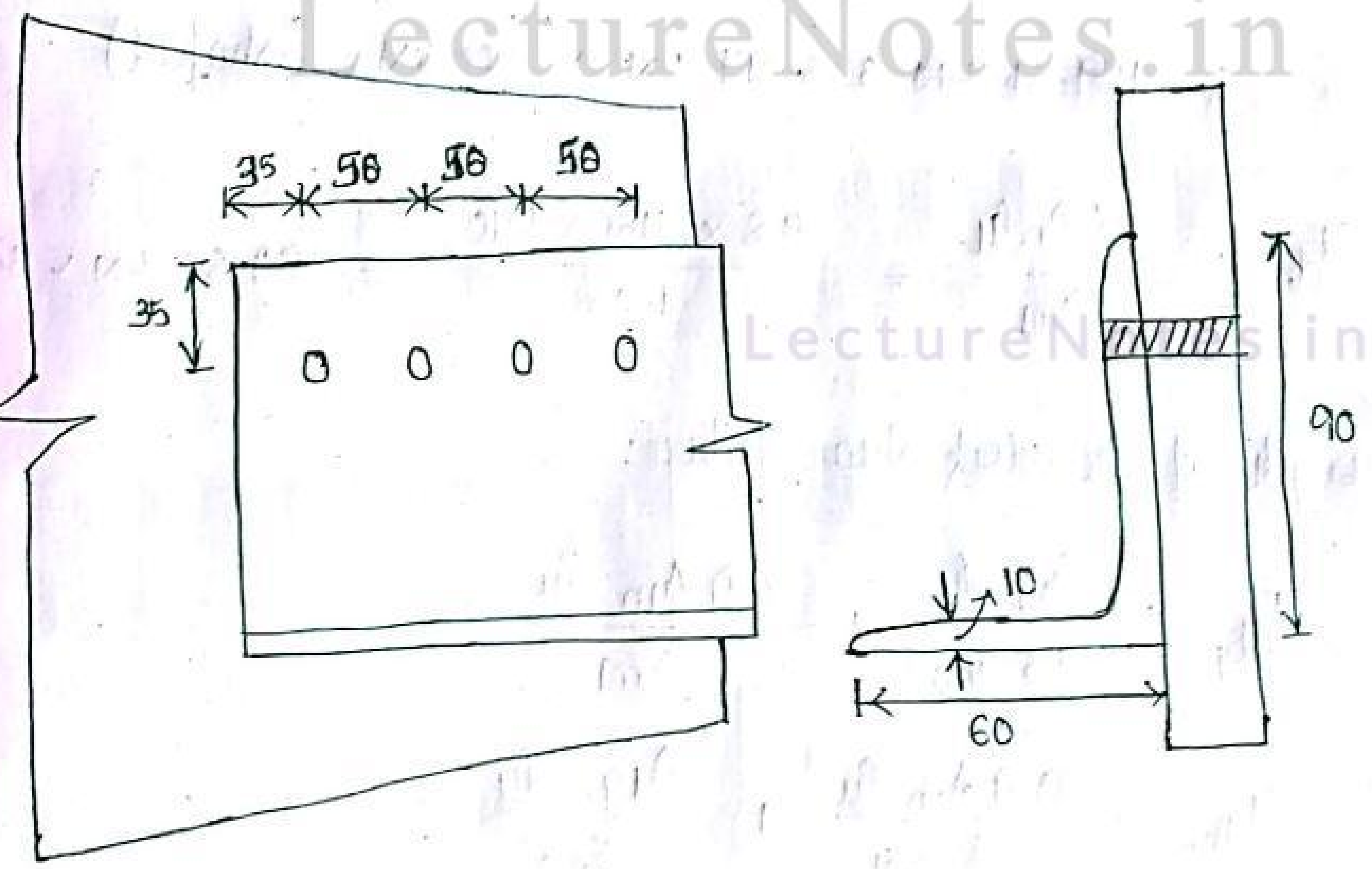
$$A_n' = A_n + 25\% (A_n) = 1429.11 \text{ mm}^2$$

The gross area to be provided, $A_g = \dfrac{T \gamma_{m0}}{f_y}$

$$= \frac{300 \times 10^3 \times 1.1}{250} = 1320 \text{ mm}^2.$$

*** The area to be selected from the steel table should have higher value of gross area then the required gross area ***.

From the steel table, choosing the section ISA $(90 \times 60 \times 10)$ mm, we have $A_g = 1401 \text{ mm}^2$.




Connecting the 90mm leg of the angle section to the gusset plate as shown in the above figure.

Check for failure criteria :-

i) strength of the member gross sectional failure :

$$T_{dg} = \frac{A_g \cdot f_y}{\gamma_{m0}} = \frac{1401 \times 250}{1.1} = 318.40 \text{ kN} > 300 \text{ kN}$$

(safe)

ii) strength of the member in net sectional rupture :

$$A_{nc} = \left(90 - 22 - \frac{10}{2}\right) \times 10 = 630 \text{ mm}^2$$

$$A_{go} = \left(60 - \frac{10}{2}\right) \times 10 = 550 \text{ mm}^2.$$

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

$$= 1.4 - \left(0.076 \times \frac{60}{10} \times \frac{250}{410} \times \frac{105}{150}\right)$$

$$= 1.20.$$

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

$$= \frac{0.9 \times 630 \times 410}{1.25} + \frac{1.20 \times 550 \times 250}{1.1}$$

$$= \cancel{199.61 \text{ kN}}\ 335.97 \text{ kN} > 300 \text{ kN}$$ (safe)

$$\text{or } T_{dn} = \frac{\alpha A_n f_u}{\gamma_{m1}} = \frac{0.8 \times 1180 \times 410}{1.25} = 309.63 \text{ kN} > 300 \text{ kN}$$

iii) strength of the block shear failure:

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

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

$$\Rightarrow A_{vg} = 185 \times 10 = 1850 \text{ mm}^2.$$

$$A_{vn} = (185 - 3 \times 22 + 11) \times 10 = 1080 \text{ mm}^2.$$

$\Rightarrow A_{tg} = 35 \times 10 = 350 \text{ mm}^2.$

$A_{tn} = \left(35 - \frac{22}{2}\right) \times 10 = 240 \text{ mm}^2.$

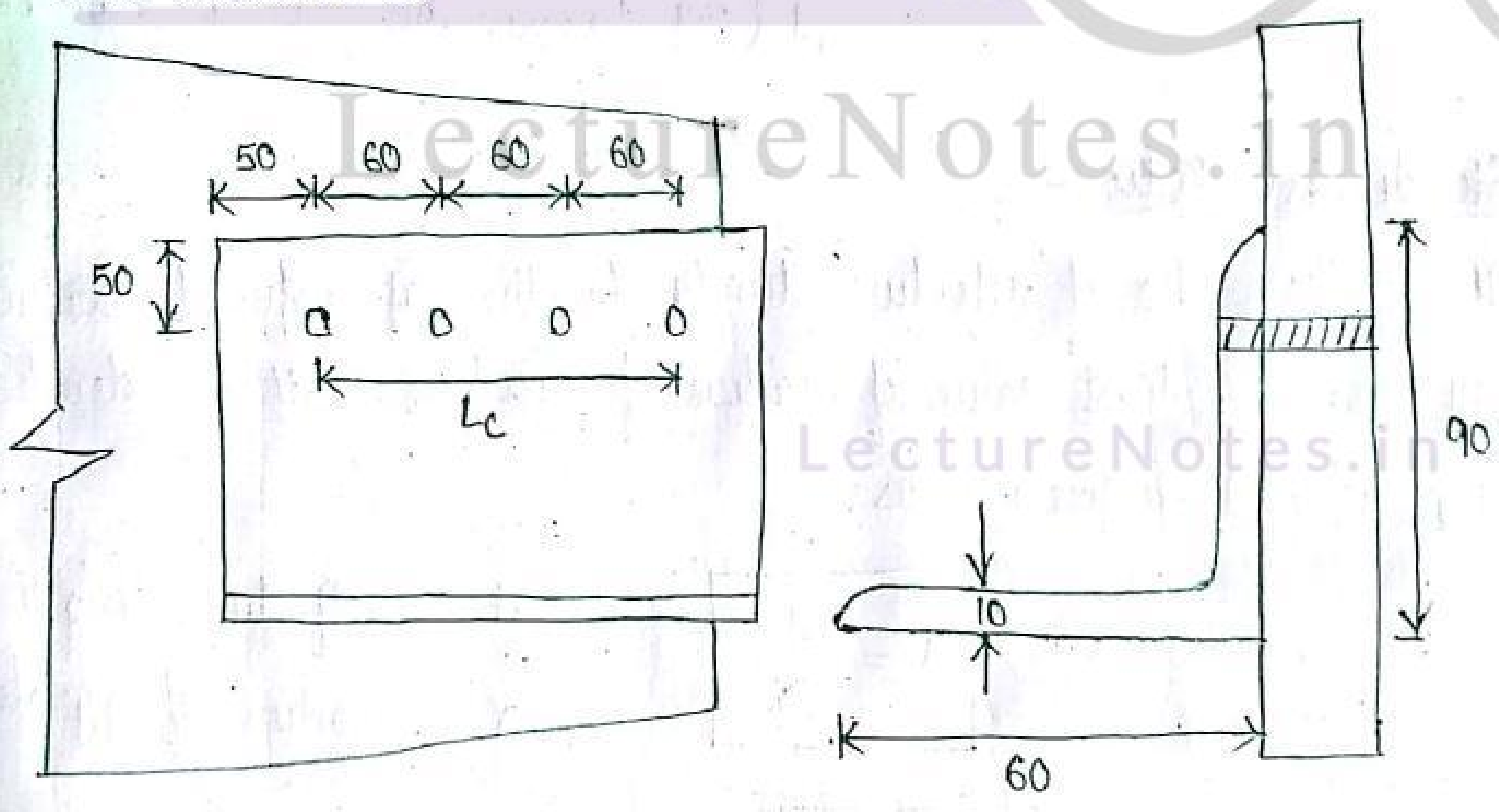
$\therefore T_{db_1} = \frac{1850 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 240 \times 410}{1.25} = 313.59 \text{ kN} > 300 \text{ kN}$

$T_{db_2} = \frac{0.9 \times 1080 \times 410}{\sqrt{3} \times 1.25} + \frac{350 \times \overset{250}{\cancel{410}}}{1.1} = \overset{263.2}{\cancel{314.52}} \text{ kN} \not> 300 \text{ kN}.$

$\Rightarrow T_{db} = 313.6 \text{ kN}.$ (lesser of $T_{db_1}$ & $T_{db_2}$)

$\therefore$ The design strength of the member is

As the section is not strong enough against the block shear failure hence by increasing the pitch & edge-end distance, we will achieve our required target strength. Increase the pitch to 60mm & edge-end distance to 50 mm.



# Compression Members

## # Compression Members :-

→ The structural member subjected to 2 equal & opposite pushing force at which both the ends is called as compression members. Eg: columns, stachian (I - section), etc.

* Terms :-

a) Effective Length :-

→ It is the product of effective length factor (K) & actual length (L) of the member considering rotational & relative translational boundary conditions at the end of the section. mathematically.

$$\boxed{l = K \times L}$$

The effective factor $K = 0.65$ (member is fixed at both ends)
$= 1$ (member is hinged at both ends)

b) Slenderness Ratio :-

→ It is the ratio of effective length to the appropriate radius of gyration or least value of radius of gyration. It is denoted by $\lambda$ and expressed as:

$$\boxed{\lambda = \frac{l}{r}}$$

$l$ = effective length
$r$ = radius of gyration

$$\boxed{r = \sqrt{\frac{I}{A}}}$$

→ For the calculation of min. radius of gyration, the min. value of M.O.I through both the axis will be taken.

→ For min. radius of gyration through an axis will give the buckling of the column through that axis as the radius of gyration is minimum there.

* <u>Code Provisions</u> :-

→ The design compressive strength : page - 34, 7.1.2.

→ The design compressive stress : page - 40, T - 9(a), 9(b), 9(c), 9(d)

→ Buckling class : page - 44, Table - 10.

→ Effective length of the member : page 45, Table - 11

* <u>Analysis Problem</u> :-

Q) Calculate the design compression load for a stachion 350 @ 710.2 kN/m of 3.5m height. The column is restrained in direction & positioned at both the ends. It is to be used as an column in a single - storied building. Use steel of grade Fe410.

A) From the steel table, for ISHB 350 @ 710.2 kN/m, the sectional properties are :-

depth of the section, $h = 350$ mm.

width of the flange, $b_f = 250$ mm.

thickness of the flange, $t_f = 11.6$ mm

thickness of the web, $t_w = 10.1$ mm.

gross - sectional area, $A = 9221\ mm^2$.

radius of gyration through y - axis, $r_y = 52.2$ mm.

radius of gyration through x - axis, $r_x = 146.5$ mm. $= r_z$.

The grade of the steel is Fe410.

$\Rightarrow f_u = 410 \; N/mm^2$ & $f_y = 250 \; N/mm^2$.

From, the code, the design compressive strength,

$$P_d = A_e f_{cd}$$

where, $A_e = A$ = gross-sectional area.

$f_{cd}$ = design compressive stress

The effective length of the member, $l = K \times L = 0.65 \times 3.5$
$= 2.275 \text{ m}$.

The min. radius of gyration, $r_{min} = r_y = 52.2 \text{ mm}$.

$\Rightarrow$ Slenderness ratio, $\lambda = \frac{KL}{r} = \frac{l}{r} = \frac{2275}{52.2} = 43.582$

From code, page-44, Table-10, we have.

$\frac{h}{b_f} > 1.2 \rightarrow z\text{-}z \rightarrow a. = \frac{350}{250} = 1.4.$

$t_f \leq 40 \text{ mm} \rightarrow y\text{-}y \rightarrow b = 11.6 \text{ mm}.$

It is satisfying both the conditions, hence the column may buckle through axis z-z for buckling class a & through the axis y-y for buckling class b.

As the min. $r_{min} = r_y = 52.2 \text{ mm}$, hence the buckling axis will be y-y & the corresponding buckling class b.

Hence, we have to choose table - 9(b) to calculate $f_{cd}$ :-

$\Rightarrow \lambda = 43.6, \; f_{cd} = 201.68$

$\lambda = 40, \; f_{cd} = 206$

$\lambda = 50, \; f_{cd} = 194.$

The design compressive strength, $P_d = A_e \times f_{cd}$

$= 9221 \times 201.68$

$= 1859.6$ kN.

*** Design Problem:-**

Q) Design a column to support load of (factored) 1050 kN. The column has an effective length of 7m w.r.t. z-axis & x-axis & 5m w.r.t. y-axis. Use steel of grade Fe410 & both the ends of the section are hinged.

A) Given:-

$P_e = 1050$ kN.

For the Fe410 grade steel, $f_u = 410\ N/mm^2$ & $f_y = 250\ N/mm^2$.

Factored load to be provided, $P_e = 1050$ kN.

Taking $\lambda = 150$, lets provide an I-section column.

From the code & steel table, we know that $r_y < r_x (r_z)$.

Hence, for the buckling class will be b because the column will buckle through y-y axis.

For the value of design compressive stress, we have to choose table - 9(b).

$\Rightarrow \lambda = 150, \ f_{cd} = 64.0$

$\Rightarrow P_e = A_e \times f_{cd}$

$$\Rightarrow A_e = \frac{P_e}{f_{cd}} = \frac{1050 \times 10^3}{64.0} = 16406.25\ mm^2.$$

From, steel table, the required section is ISWB 600 @ 133.7 kg/m

Provided sectional area, $A_{provided} = 17038\ mm^2$.

Radius of gyration, $r_{xx} = r_{zz} = 24.97\ cm = 249.7\ mm$

$r_{yy} = 5.25\ cm = 52.5\ mm$

Height (depth) of the section = 600 mm = h.

Width of the flange, $b_f = 250\ mm$

Thickness of the flange, $t_f = 23.6$

Thickness of the web, $t_w = 11.8$

From IS code, we have, $\frac{h}{b_f} = \frac{600}{250} = 2.4 > 1.2$.

$t_f = 21.3 < 40$.

Hence, the column will buckle through z-z axis having buckling class 'a' & through y-y axis having buckling class 'b'.

The slenderness ratio through the x-x axis is:

$$\lambda_x = \frac{7000 \times 1}{249.7} = 28.03$$

Similarly, $\lambda_y = \frac{5000 \times 1}{52.5} = 95.23$

∴ Take min. slenderness ratio for stability, i.e., $\lambda = 28.03$

*** While taking the $f_{cd}$ value from the table - 9, we have to take the minimum slenderness ratio irrespective of the buckling axis & buckling class. ***

$\lambda = 28.03$, $f_{cd} = ?$

$\lambda_1 = 20$, $f_{cd_1} = 225$

$\lambda_2 = 30$, $f_{cd_2} = 216$

$\Rightarrow f_{cd} = 217.773$.

$\therefore$ The design compressive strength, $P_e = A_p \times f_{cd}$

$= 17038 \times 217.773$

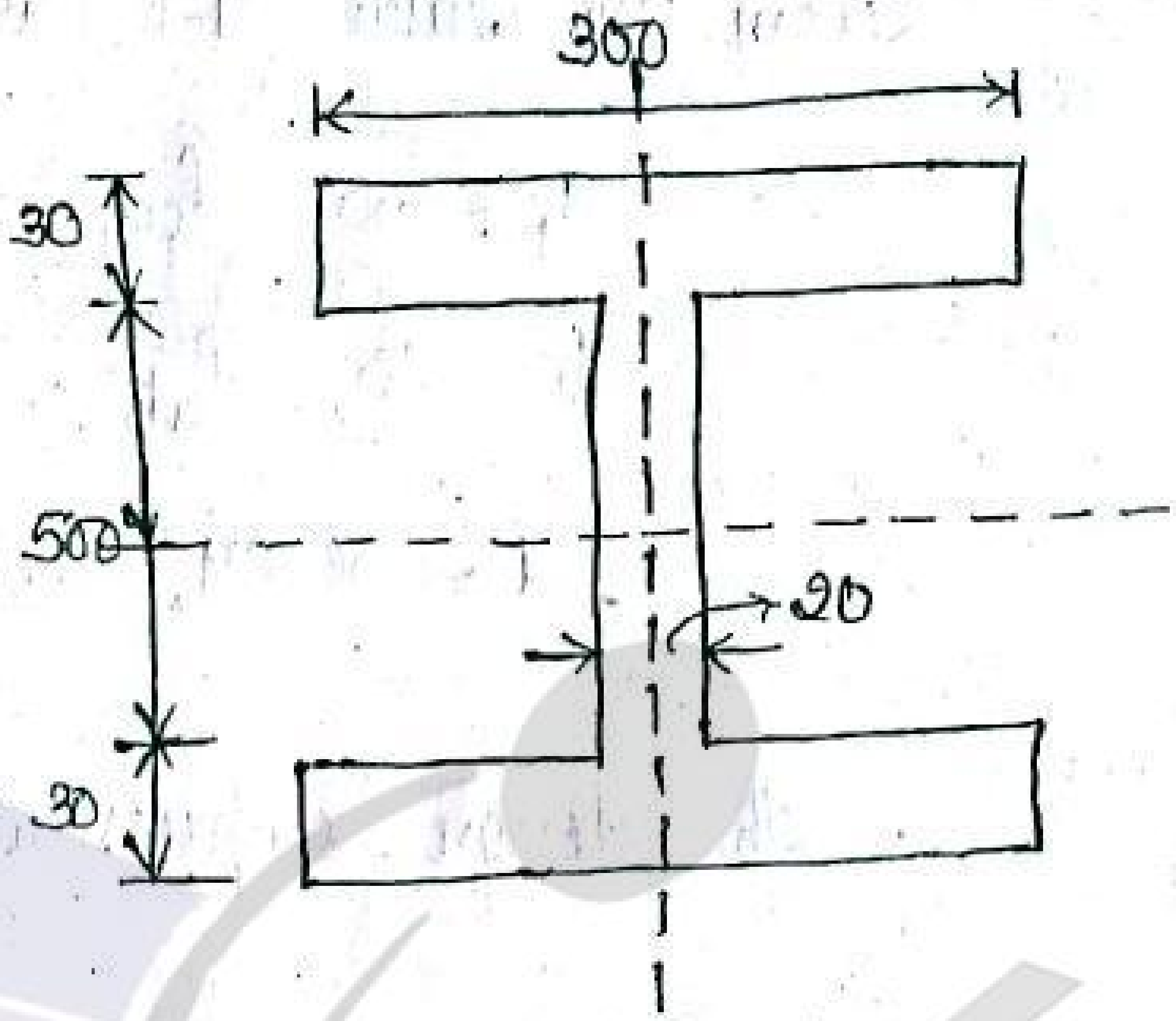
$= 3710.416$ kN.

$\therefore (3710.416 > 1050)$ kN

Hence, the design is safe.



Q) For a column section built-up as shown in the figure, determine the axial load carrying capacity in compression for the data shown in the figure. Use steel of grade Fe 410 & the length of the column is 6m. The ends of the column are restrained both against direction & position. All the dimensions given in figure are in mm.






A) $I_x = 2 \times \frac{bd^3}{12} + \frac{bd^3}{12}$

$= 2 \times \frac{300 \times 30^3}{12} + \frac{20 \times 500^3}{12}$

$I_x = 1.473 \times 10^9$ mm$^4$

$I_y = 1.35 \times 10^8$ mm$^4$

$A = 28000$ mm$^2$.

$r_x = 229.41$ , $r_y = 69.52$.

$30 < t_f < 40$

z – z axis = class b
y – y axis = class c
} from IS code. for welded I-sec$^n$.

$\because r_x > r_y \Rightarrow r_y = 69.52$ should be considered.

Hence, the column will buckle through y-y axis & buckling class is 'c'.

The slenderness ratio for y-y axis is :-

$$\lambda_y = \frac{0.65 \times 6000}{69.52} = 56.09$$

From the table 9-c, we have

$\lambda_1 = 50, \quad f_{cd_1} = 183$

$\lambda_2 = 60, \quad f_{cd_2} = 168.$

$\lambda = 56.09, \quad f_{cd} = 173.865.$

$\therefore$ The design compressive strength is $P = A \times f_{cd}$

$= 28000 \times 173.865$

$= 4868.22$ kN

Q.) An ISA (100 x 100 x 6) mm of Fe410 grade steel is used as a compression member in a truss. The length of the member between the intersection of each end is 3m. Calculate the strength of the member if :-

i.) it is connected by two bolt at each end.

ii.) it is connected by one bolt at each end.

iii.) it is connected by welded connection at each end.

A.) Code provisions for angle sections of compression members :-

i.) Pg - 35 → 7.2.4

ii.) Pg - 47 → 7.5

iii.) Pg - 34 → 7.1.2.1

iv.) Pg - 48 → 7.5.1.2

From the steel table, for ISA (100x100x6) mm :-

$$a = 11.67 \text{ cm}^2 = 1167 \text{ mm}^2.$$

$$r_{xx} = r_{yy} = 3.09 \text{ mm}$$

$$r_{vv} = 19.2 \text{ mm}$$

$$r_{uu} =$$



The design compressive strength of the angle section :

$$P_d = A_e f_{cd}$$

$$\Rightarrow A_e = A_g.$$

$$\Rightarrow f_{cd} = \text{designed compressive stress}$$

$$\Rightarrow f_{cd} = \frac{f_y/\gamma_{m0}}{\phi + (\phi^2 - \lambda^2)^{0.5}} \quad [\text{page - 34, 7.1.2}]$$

$$\phi = 0.5 \times [1 + \alpha(\lambda - 0.2) + \lambda^2]$$

From IS code (page - 44, table - 10), for angle section, we have buckling class as 'c'. so, $\alpha = 0.49$.

where, $\lambda$ = slenderness ratio [page - 48, 7.5.1.2]

$$\lambda_e = \sqrt{k_1 + k_2 \lambda_{vv}^2 + k_3 \lambda_\phi^2}$$

where, $k_1, k_2, k_3$ = constants depending upon the end conditions.

Assuming both the ends as fixed.

i) For 2 bolts connections, $k_1 = 0.20$, $k_2 = 0.35$, $k_3 = 20$

ii) For 1 bolt connection, $k_1 = 0.75$, $k_2 = 0.35$, $k_3 = 20$.

ii) $$\lambda_{vv} = \frac{l/r_{vv}}{\epsilon\sqrt{\frac{\pi^2 E}{250}}} = \frac{3000/19.5}{1\sqrt{\frac{\pi^2 \times 2\times10^5}{250}}} = 1.731$$

$l = 3000\ mm$

$r_{vv} = 19.2\ mm$

$E = 2\times10^5\ N/mm^2$

$\epsilon = \left(\frac{250}{f_y}\right)^{0.5} = 1.$

$$\lambda_\phi = \frac{(b_1+b_2)/2t}{\epsilon\sqrt{\frac{\pi^2 E}{250}}} = \frac{(100+100)/2\times6}{\epsilon\sqrt{\left(\frac{\pi^2 E}{250}\right)}} = 0.1875$$

$b_1, b_2$ = width of the 2 legs of the angle section.

$t$ = thickness of the leg = 6mm.

$$\therefore \lambda_e = \sqrt{0.20 + (0.35\times(1.73)^2) + 20\times(0.18)^2}$$

$= 1.376$

$$\therefore \phi = 0.5\times\left[1 + 0.49(1.376-0.2) + 1.376^2\right]$$

$= 1.73$

$$\Rightarrow f_{cd} = \frac{f_y/\gamma_{m0}}{\phi + \left[\phi^2 - \lambda^2\right]^{0.5}}$$

$$= \frac{250/1.1}{1.73 + (1.73^2 - 1.37^2)^{0.5}}$$

$$= \frac{227.28}{2.79} = 81.46$$

$\therefore P_d = A_g \times f_{cd} = 1167 \times 81.46 = 95.06\ kN$

ii) $\lambda_e = \sqrt{0.75 + \left(0.35 \times (1.73)^2\right) + \left(20 \times (0.18)^2\right)} = 1.5638.$

$$\phi = 0.5 \times \left[1 + 0.49 \times (1.5638 - 0.2) + 1.5638^2\right]$$

$$= 2.056.$$

$$f_{cd} = \frac{250/1.1}{2.056 + \left(2.056^2 - 1.5638^2\right)^{0.5}}$$

$$\doteq 67.026$$

$\therefore\ P_d = A_g \times f_{cd} = 1167 \times 67.026 = 78.21\ \text{kN}$

iii) Now, for the welded connection:-

As the $k_1$, $k_2$, $k_3$ are not specified in the IS code, hence the strength of the member in welded connection will be equal to or same as the strength of member as in case of 2 bolt connection.

| | |
|---|---|
| LACING<br>BATTENING | } → AT LAST |

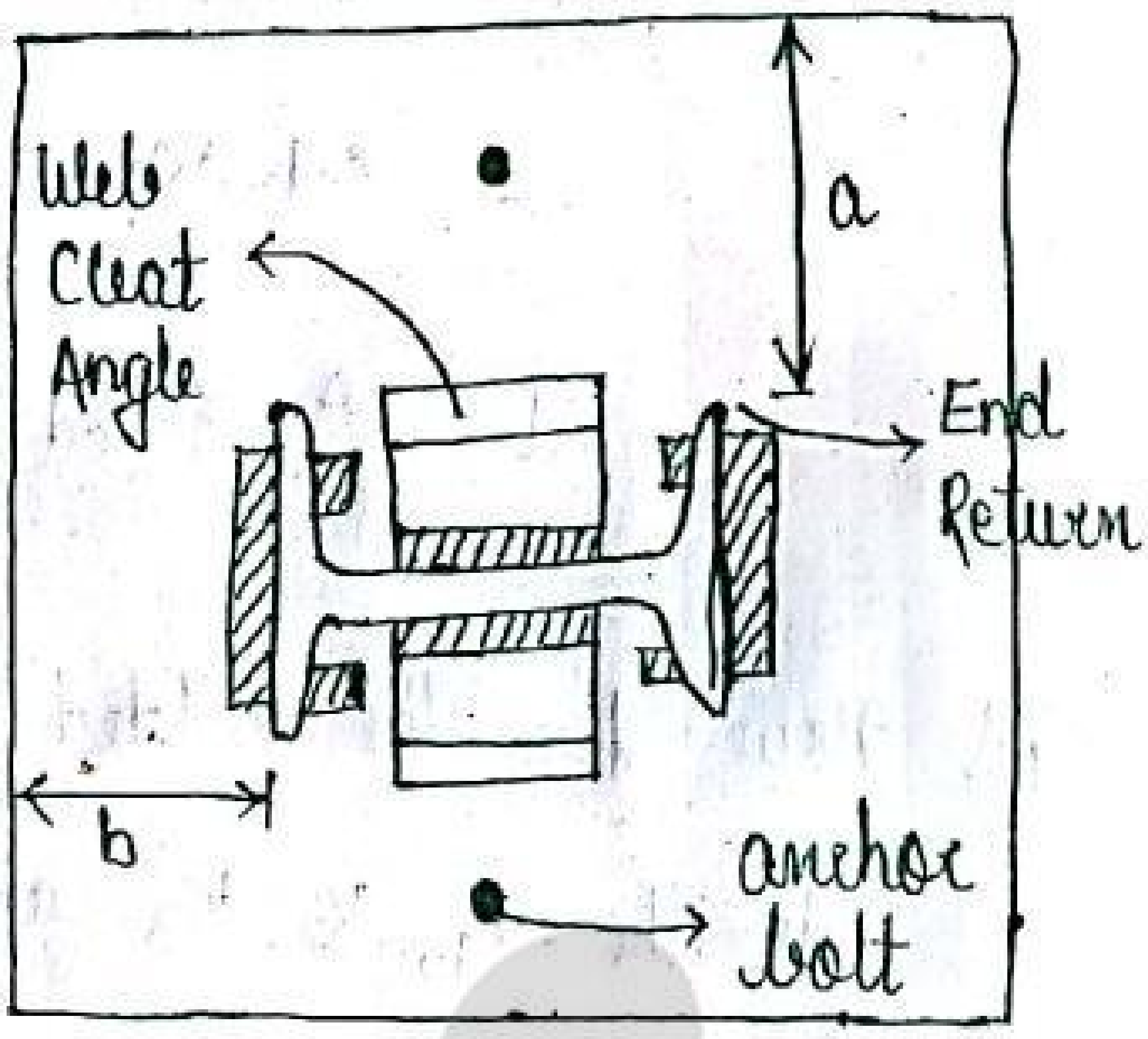
END OF MODULE - 2

# COLUMN BASE

## # Column Base :-

i) The design of column base
↳ Pg - 46, 7.4

ii) Minimum thickness of the base.
↳ Pg - 47, 7.4.3.1




a → larger projection.
b → short term projection.

## # Terminologies :-

1) End Return :-
→ In case of welded connection, it is difficult to provide weld at the end of the section & the intermediate joint, which is known as 'end return'.

$$\boxed{\text{End return} = 2 \times s}$$

where, s = size of the weld.

2) Webcleat Angle :-
→ To make the column base fixed to its position with the column base, 2 angle sections are provided at 2 sides of the column web, which is known as web cleat angle.

3) Anchor Bolts :-
→ To make the column base fixed at its position, 2 anchor bolts are provided at the top & at the bottom of the column base.
→ It also helps from the sliding failure of the column base.

Q) Design a slab base for a column ISHB 350 @ 710.2 N/m subjected to an factored axial compressive load of 1500 kN for the following conditions :-

a) The load is transfered to the base plate by direct bearing of the column flanges.

b) The load is transfered to the column base by the welded connection.

The column end & base plate are not machined for bearing. whether the anchor force required ? The column base is resting on a concrete pedestrial of grade M20.

A) For ISHB 350 @ 710.2 N/m :-

$A = 9221\ mm^2$

$h = 350\ mm.$

$b_f = 250\ mm$

$t_f = 11.6\ mm$

$t_w = 10.1\ mm$

Assuming, grade of steel as Fe410 : $f_y = 250\ N/mm^2$, $f_u = 410\ N/mm^2$

a) Bearing strength of concrete $= 0.45 \times f_{ck} = 0.45 \times 20 = 9N.$

$\gamma_{m0} = 1.1$ ; $\gamma_{mw} = 1.25$

Area of the base plate required $= \dfrac{\text{load on the column}}{\text{bearing strength of concrete}}$

$$= \frac{1500 \times 10^3}{9} = 166.67 \times 10^3\ mm^2.$$

Provide a square column base having dimension $(D \times D)$ mm

$\Rightarrow$ Area of the column $= D^2 = 166.67 \times 10^3\ mm^2.$

$\Rightarrow D = 408.248\ mm$

Providing a clear space of 10mm, to resist corrosion,

$$D = 408.248 + 10 = 418.248 \simeq 420 \text{ mm}$$

Area provided $= D^2 = 420^2 = 176400 \text{ mm}^2$.

Check for the bearing strength of the concrete :-

$$\text{Bearing strength} = \frac{P}{A} = \frac{1500 \times 10^3}{176400} = 8.503 \text{N} < 9\text{N}.$$

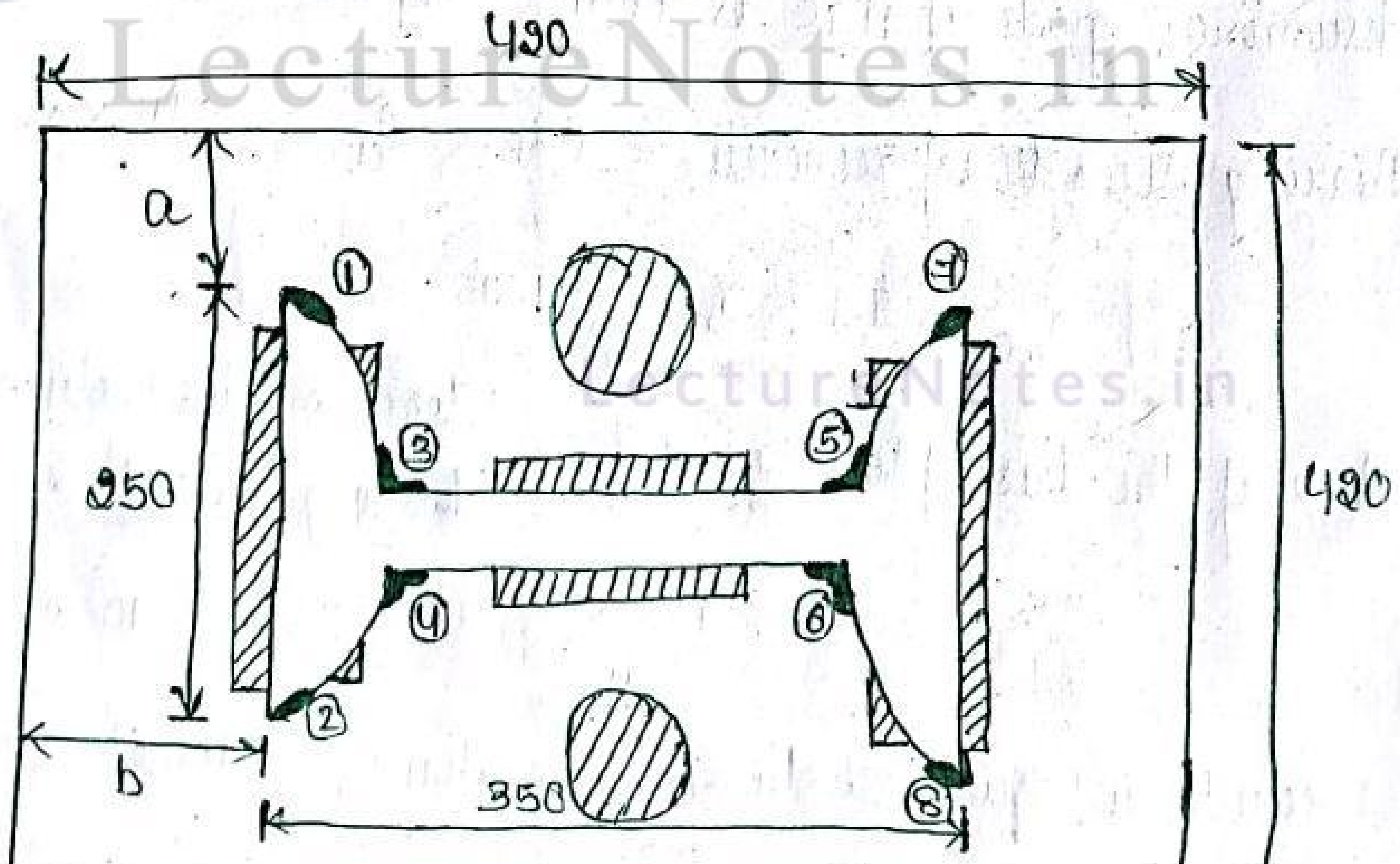
∴ Hence the dimension of column base provided is 420mm.

*** Always provide the column at the centre of the column base to make it safe against eccentric moment & eccentric loading. ***

Now, $w$ = provided bearing strength $= 8.503 \text{ N/mm}^2$.

$\Rightarrow$ shorter projection, $b = \frac{420 - 350}{2} = 35 \text{ mm}$.

Larger projection, $a = \frac{420 - 250}{2} = 85\text{mm}$.




Thickness of the column base :- (P-47, 7.4.3.1)

$$t_s = \sqrt{2.5\, w\,(a^2 - 0.3\, b^2) \times \frac{\gamma_{m0}}{f_y}}$$

$$\Rightarrow t_s = \sqrt{2.5 \times 8.503\left(85^2 - 0.3 \times 35^2\right) \times \frac{1.1}{250}}$$

$$= 25.321\text{ mm} \simeq 30\text{mm} \geq t_f.$$

The dimension of the column base provided = (420 x 420 x 30) mm

To keep the column in position with the column base, provide 2 nos cleat angle of dimension (60 x 60 x 10) mm, provided angle is safe against tension & compression.

b.) <u>Provision of welded connections</u> :-

Length of the total weld available =

$$= (2 \times 250) + \left[2 \times (250 - 10.1)\right] + \left[2 \times \left(350 - 2 \times 11.6\right)\right]$$

$$= 1633.4\text{ mm}.$$

The no. of end return available = 12

Assume size of the weld = 8mm

$\Rightarrow$ size of the return = $2 \times S$ = $2 \times 8$ = 16mm.

Effective length available for welding = $1633.4 - (16 \times 12)$

$= 1505.4$ mm

$\Rightarrow$ The strength of the welding provided = $\dfrac{l_w \times t_t \times f_u}{\sqrt{3} \times \gamma_{mw}}$

$$= \frac{1505.4 \times (0.7 \times 8) \times 410}{\sqrt{3} \times 1.25}$$

$$= 1596.44\text{ kN} > 1500\text{ kN}$$

$\therefore$ Hence, provide the weld of length 1505.5 mm with 8 no. of end returns as specified above & provide (2 – 24mm) $\phi$ bolts as anchor bolts at the top & bottom of the column base to resist the sliding failure.

# Design of the Gusseted Base :-

Q) A column ISHB 350 @ 661.2 N/m carries an axial compressive factored load of 1400 kN. Design a suitable bolted gusseted base. The base is resting on the concrete pedestrial of grade M15. Use 24mm dia bolts of grade 4.6 to make the connection

A) For ISHB 350 @ 661.2 N/m :-

$A = 8591 \text{ mm}^2$

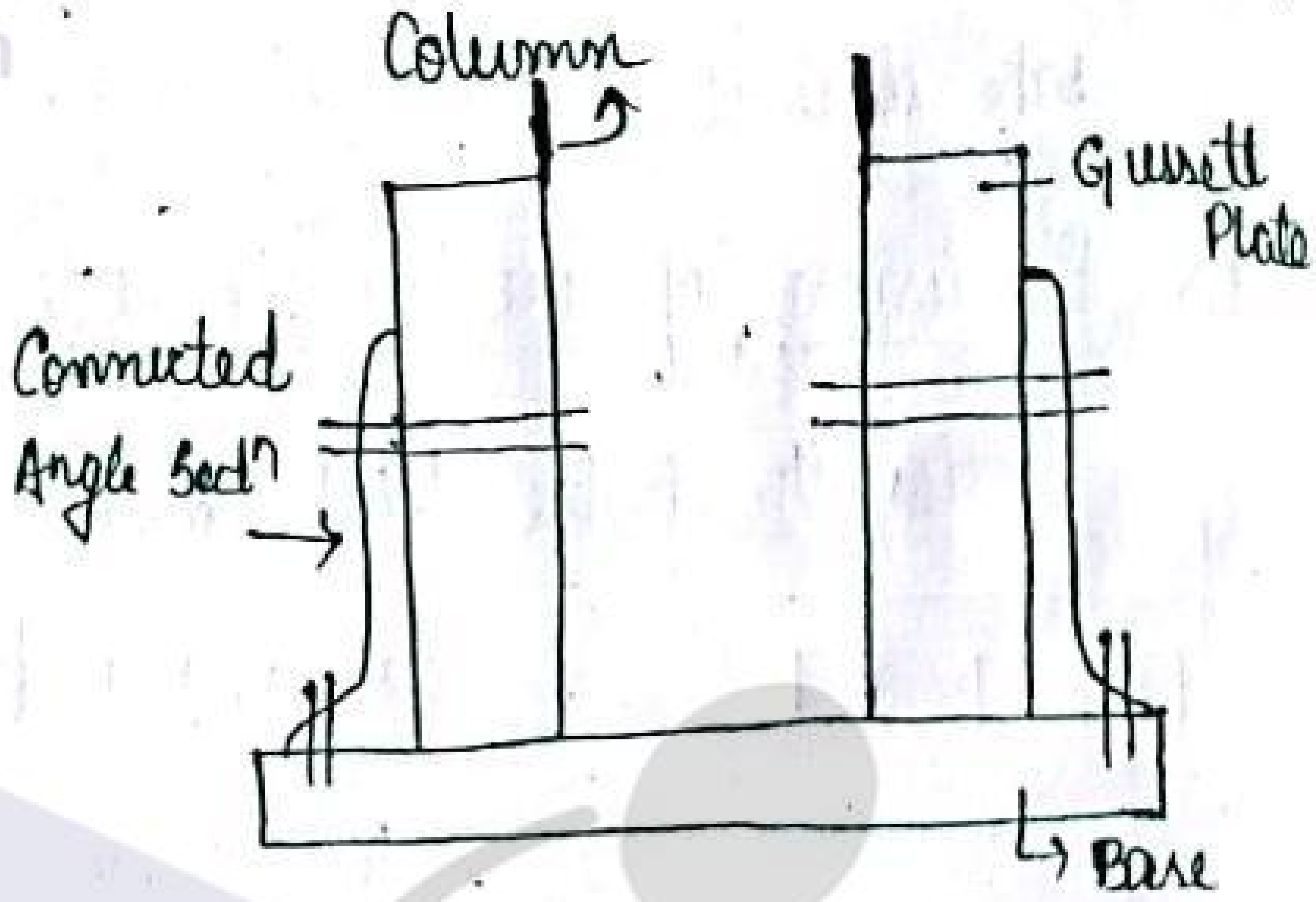
$h = 350 \text{ mm}$

$b_f = 250 \text{ mm}$

$t_f = 11.6 \text{ mm}$

$t_w = 8.3 \text{ mm}$

$z_e = 1.094 \times 10^6 \text{ mm}^3$.




For the strength of the bolt in shearing :-

$$V_{dsb} = \frac{f_{ub}}{\sqrt{3}\gamma_{mb}} \left( n_n A_{nb} + n_s \cancel{A_{sb}}^{\,0} \right)$$

$$= \frac{400}{\sqrt{3} \times 1.25} \left( 1 \times 0.78 \times \frac{\pi}{4} \times 24^2 \right)$$

$$= 65.192 \text{ kN}.$$

For the strength of the bolt in bearing :-

$P_{min} = 2.5d = 60 \text{mm}$.

$e_{min} = 1.5 \times d_0 = 39 \text{mm}$

$$\Rightarrow V_{dpb} = \frac{2.5\ K_b\ dt\ f_u}{\gamma_{mb}}$$ (assuming the thickness of gusset plate as 16mm).

$$= \frac{2.5 \times 0.5 \times 24 \times 16 \times 410}{1.25} = 157.44 \text{ kN}.$$

Hence, the strength of the bolt is 65.192 kN.

For the dimension of the base :-

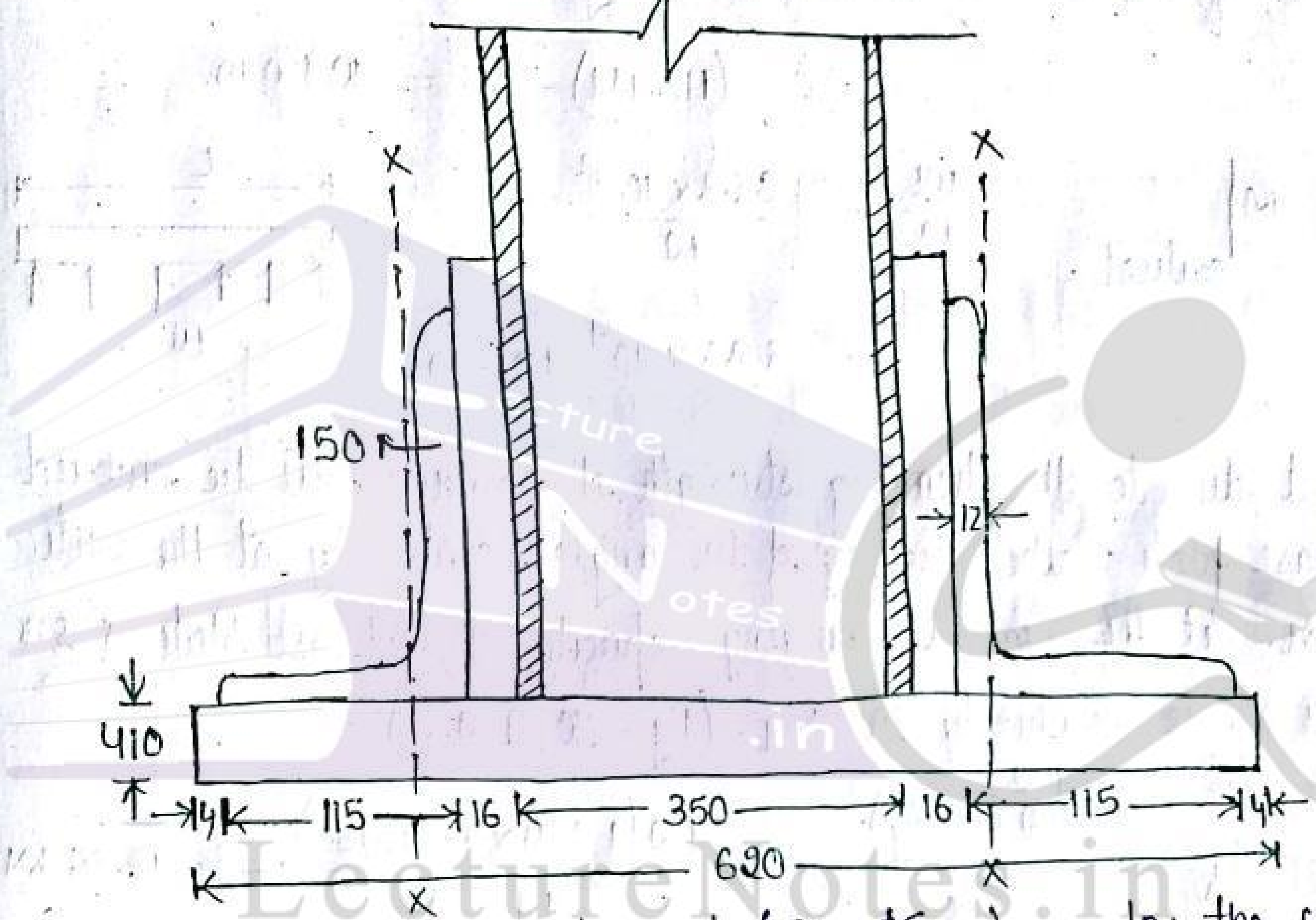
Grade of the concrete = M15.

Bearing strength, $f_{ck} = 0.45 \times 15 = 6.75\ N/mm^2$.

area of base plate $= \frac{P_u}{f_{ck}} = \frac{1700 \times 10^3}{6.75} = \cancel{251.85}$

$= 251851.85$

$= 2.518 \times 10^5\ mm^2$.




Providing an angle section of $(150 \times 1\cancel{5}5 \times 12)$ mm for the connection of column to the column base :

A =

width of the column base, $b = 620$ mm.

Extra space of 4mm is provided in both side of the connection to resist or to be safe against corrosion.

depth of the column base $= \frac{A}{b} = \frac{2.518 \times 10^5}{620}$

$= 406.129 \simeq 410$ mm.

$\therefore$ Provided area, $A_p = 2.542 \times 10^5\ mm^2$.

Provided bearing strength, $f_{ck_p} = \dfrac{1700 \times 10^3}{2.542 \times 10^5} = 6.687 < 6.75$

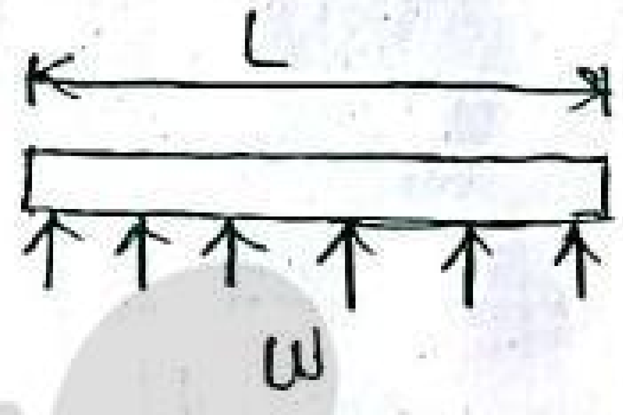
Hence, provide the area of column base as $(620 \times 410)$ mm

The critical section 'x-x' will fail under the intensity of load provided by the concrete. If the bearing strength provided, the load on the critical section is $w = 6.68$.

Distance of the critical section from right hand side end, $L =$

$$(115+4) - 12 = 107 \text{ mm}.$$

$$M\Big|_{critical} = \frac{wL^2}{2} = \frac{6.68 \times 107^2}{2}$$




$$= 3.83 \times 10^4 \text{ kN-m}.$$

Moment due to the bearing strength of concrete will be carried by the base plate & the one leg of the angle combinely at the critical section. Let the moment carrying capacity of the base plate & one leg of the angle combinely be '$M_d$'. (Pg - 70, 9.2.2)

$$M_{dv} = \frac{1.2 (Ze \cdot f_y)}{\gamma_{m0}} = \frac{1.2 (1.094 \times 10^6 \times 250)}{1.1} = 298.36 \text{ kN-m}$$

where, $Z_e$ = elastic sec$^n$ modulus of the column base plate.

$$= \frac{bd^2}{6}$$

for 1mm run, $b = 1$mm, $d$ = thickness of plate $= t$.

$$\Rightarrow Z_e = \frac{t^2}{6}.$$

Equating the moment carrying capacity of the base plate & angle section combinely with the applied moment, we have:

$$\frac{1.2 Z_e f_y}{\gamma_{m0}} = 3.23 \times 10^4$$

$$\Rightarrow \quad \frac{1.2 \times \frac{t^2}{6} \times 250}{1.1} = 3.23 \times 10^4$$

$$\Rightarrow \quad t = 26.55\,mm \simeq 30\,mm.$$

Hence, provide the dimension (620 x 410 x 30) mm. Assuming 50% of the load directly carried by the column & 50% of the load will be carried by the connection (bolted), the no. of bolts to be provided $= \frac{0.5 \times 1700}{65.19} = 13.038 \simeq 14$ nos.

END OF MODULE - 2

# Design of Beams

## # Terminologies :-

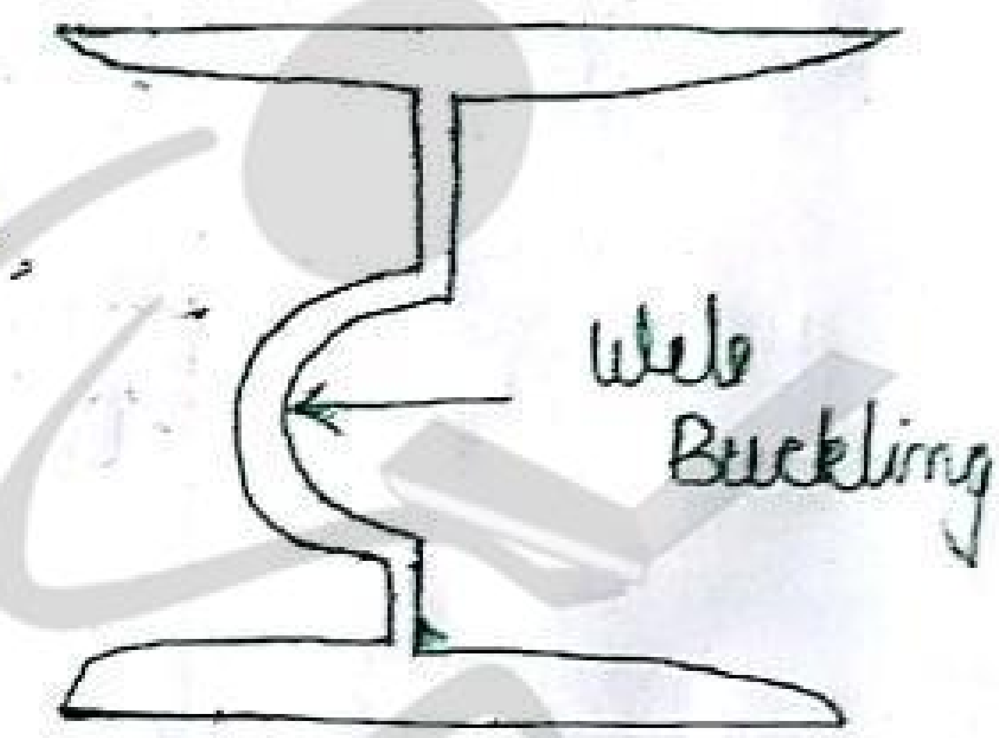
### 1) Shape Factor :-

→ It is the ratio between the plastic section modulus to the elastic section modulus. It is denoted by 'S'.

i.e.; $\boxed{S = \dfrac{\text{plastic section modulus } (Z_p)}{\text{elastic section modulus } (Z_e)}}$

### 2) Web Buckling :-

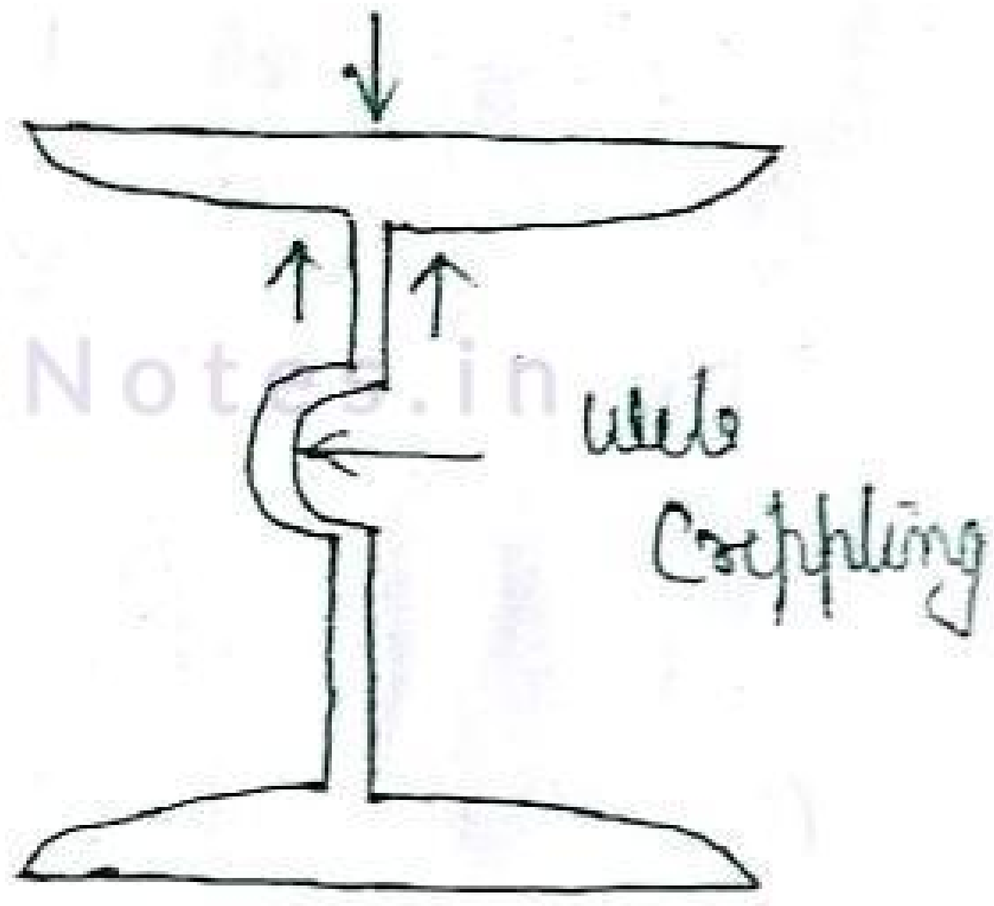
→ When the applied load on the beam section is more than the design strength the web portion of the section behaves like a compression member & due to the load, buckling will occur at the web portion.




### 3) Web Crippling :-

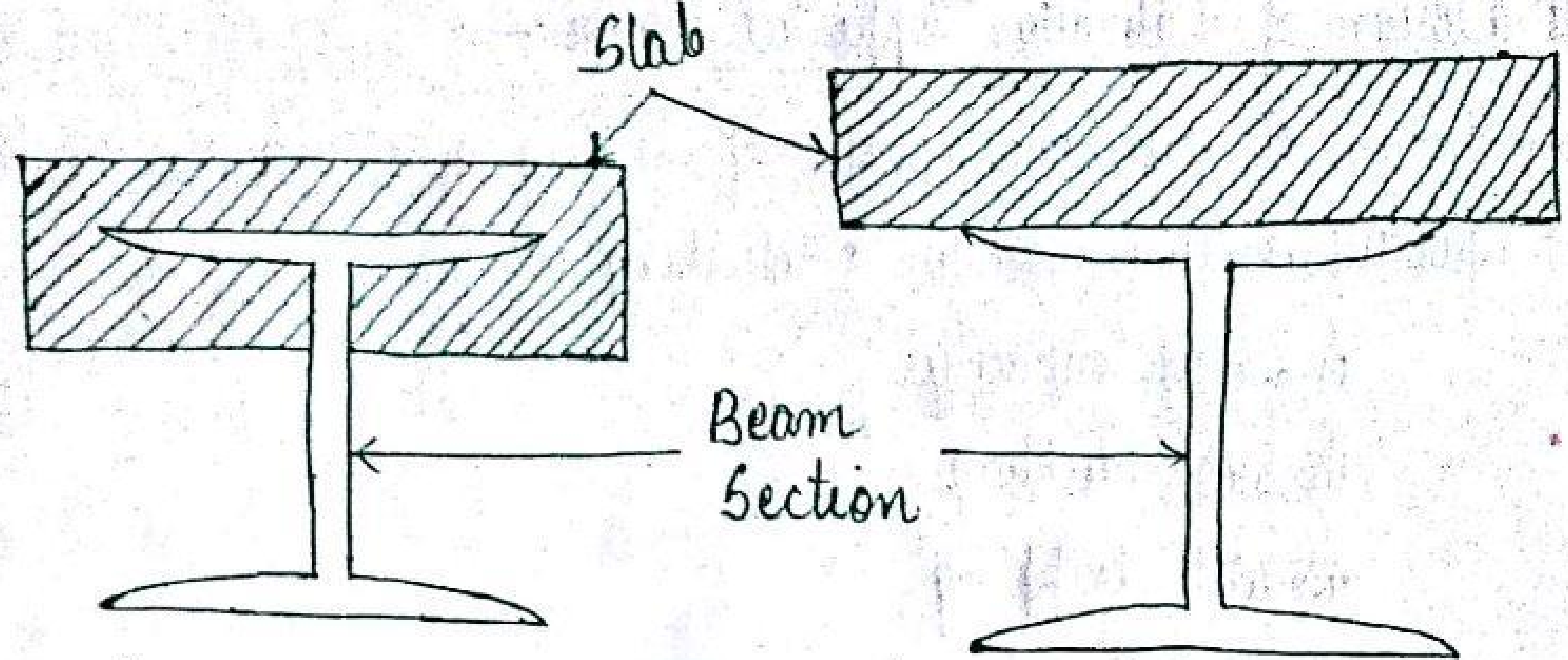
→ It is the buckling of the web occurs just at the starting of the web or near the connection of web & flange is known as web crippling.

→ This occurs due to the reaction force of the web against the applied load.




## # Laterally Supported And Unsupported Beam :-

→ When the flange of the beam section is fixed into the slab as shown in the figure - 1, the beam is known as "laterally supported beam".




(Figure - 1) (Figure - 2)

→ When the slab is just resting on the beam as shown in the figure - 2, then the beam is known as "laterally unsupported beam".

→ In case of laterally supported beams, the slab help the flange not to move in lateral direction.

# Classification of Sections :-

→ For Indian standard rolled steel section, maximum of the section are of compact & semi-compact type.

→ Based on the plastic moment capacity & ductility, the section is divided into 4 groups :-

| Section | Plastic Hinge | Ductility |
|---|---|---|
| Plastic | ✓ | ✓ |
| Compact | ✓ | ✗ |
| Semi- Compact | ✗ | ✓ |
| Slender | ✗ | ✗ |

# Design of Laterally Supported Beam :-

→ Design of this beam consist of selecting a section on the basis of plastic section modulus & checking it for :

i) shear capacity.

ii) web buckling.

iii) Web crippling.

iv) Deflection

* Design Steps :-

i) Factored load is determined by multiplying factor of safety ($\gamma_f = 1.5$) with the service (or) working load.

ii) The maximum BM 'M' & shear force 'V' are to be calculated for the beam.

iii) A trial section based upon the plastic section modulus for the beam is defined as follows :-

$$\boxed{Z_p = \frac{M \cdot \gamma_{mo}}{f_y}} \quad [Pg - 53, 8.2.1.2]$$

iv) Looking at the value of $Z_p$ required, a suitable section is selected. The trial section may be ISLB, ISMB, ISWB, etc.

v) The classification of the section is checked.

vi) The trial section is checked for shear.

vii) The trial section is checked for design bending strength.

viii) The trial section is checked for buckling. (P-59, 8.4.2.1)

ix) It is checked for bearing / web crippling.

x) The trial section is checked for deflection.

## * Code Provision For Laterally Supported Beam:-

i) Laterally supported beam → P-52; 8.2.1

ii) '$Z_p$' required → P-53; 8.2.1.2

iii) Selection of section → P-138; Annexure-H

iv) check for shear capacity → P-59; 8.4

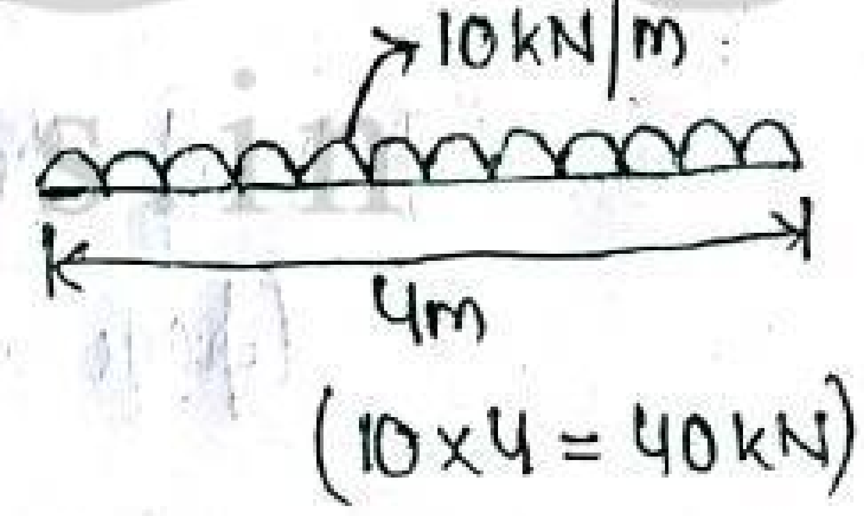
v) check for high/low shear → P-53; 8.2.1.2

vi) Check for design bending strength → P-53; 8.2.1.2

vii) Check for deflection → P-31; T-6.

viii) Check for web crippling → P-67; 8.7.4

Q) A simply supported steel joist (beam) of 4m effective length is laterally supported throughout. It carries a total UDL of 40kN (including self weight). Design an appropriate section using steel of grade Fe410. Assume any data if required.




A) For Fe410 grade steel:-

$f_u = 410\ N/mm^2$

$f_y = 250\ N/mm^2$

$\gamma_{m0} = 1.1$

Partial safety factor, $\gamma_f = 1.5$

Total UDL value = 40 kN.

Density of the UDL = $w = \frac{40}{4} = 10 kN/m$.

Factored UDL density, $w' = \gamma_f \times w = 1.5 \times 10 = 15 kN/m$.

Maximum factored moment, $M = \frac{w'l^2}{8} = \frac{15 \times 4^2}{8} = 30\ kN\text{-}m$.

Maximum shear force, $V = \frac{w l}{2} = \frac{15 \times 4}{2} = 30\ kN.$

Selection of section :-

$$(Z_P)_{req.} = \frac{M \cdot \gamma_{mo}}{f_y} = \frac{30 \times 10^6 \times 1.1}{250} = 1.32 \times 10^5\ mm^3$$

$$= 132.00\ cm^3.$$

From page - 139, let us select ISLB 200 @ 19.8 kg/m.

$$Z_P = 184.34\ cm^3 = 1.84 \times 10^5\ mm^3.$$

$$\because\ Z_P > (Z_P)_{req.}$$

Sectional properties (from steel table) :-

$A = 2527\ mm^2$ $\quad b_f = 100\ mm$

$H = 200\ mm$ $\quad t_f = 7.3\ mm$

$r_1 = 9.5\ mm$ $\quad t_w = 5.4\ mm.$

$r_2 = 3.0\ mm$

$H - 2 \times (t_f + r_1)$

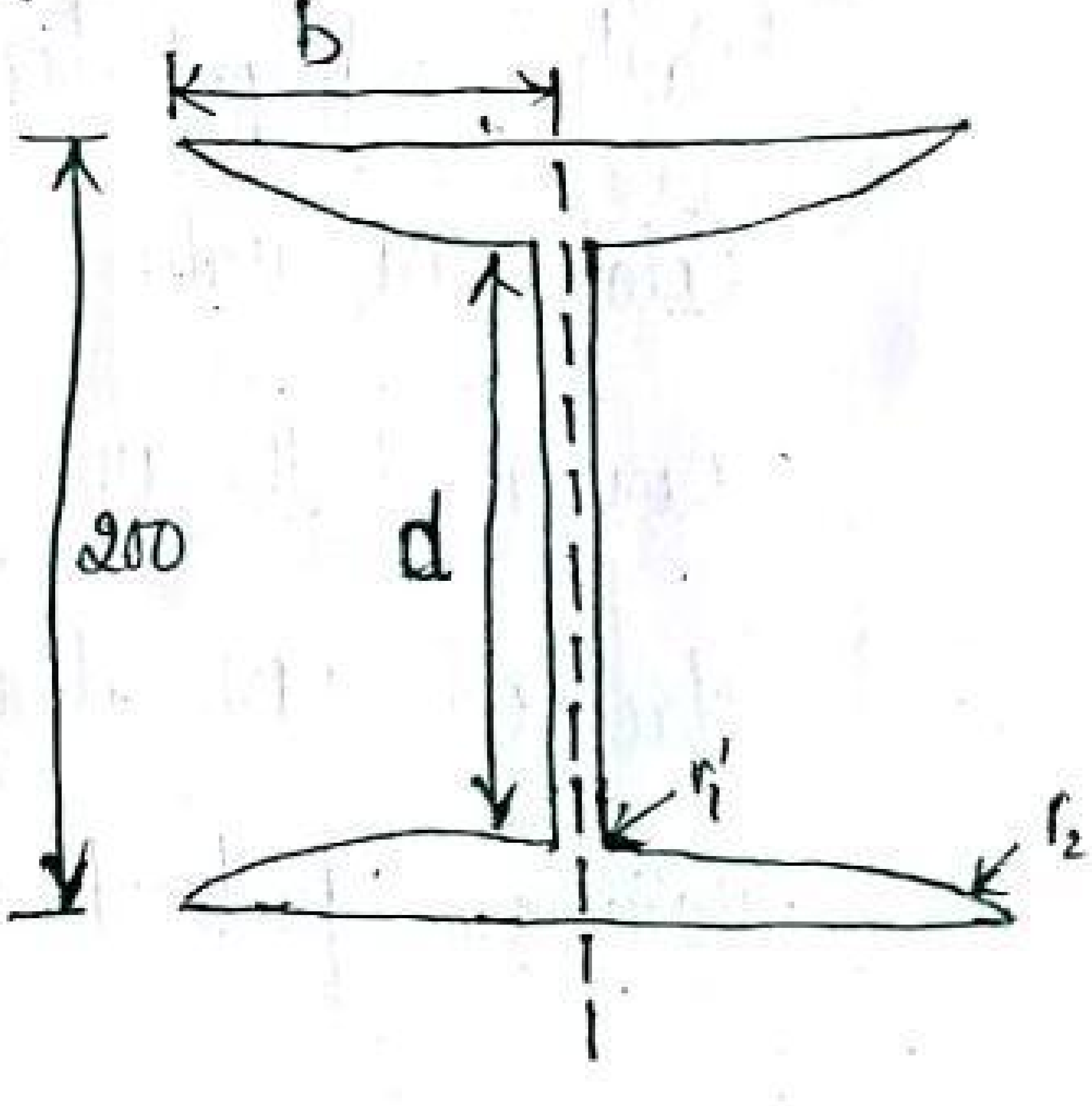
Depth of the weld, $d = 200 - 2 \times (t_f + r_1)$
(page - 19)

$$= 200 - 2 \times (7.3 + 9.5)$$

$$= 166.4\ mm$$

Now, for classification from page - 18, table - 2 :-

$b = \frac{b_f}{2} = \frac{100}{2} = 50\ mm$
(for flange)




for rolled section :

$$\frac{b}{t_f} = \frac{50}{7.3} = 6.849 < 9.4\,\epsilon$$

where $\epsilon = \sqrt{\frac{250}{f_y}} = 1.$

∴ Section → class-1, plastic.

For web, $\frac{d}{t_w} = \frac{166.4}{54} = 30.81 < 84\epsilon$ (NA for mid span)

∴ Section → class-1, plastic.

∴ The section ISLB 200 is classified as plastic section.

ii) Check for the shear capacity:-

$$V \leq V_d$$

$$\Rightarrow V_d = \frac{V_n}{\gamma_{m0}}$$

$$V_n = V_p$$

$$V_p = \frac{A_v f_{yw}}{\sqrt{3}}$$

where, $A_v$ = shear area (8.4.1.1, pg-59)

$f_{yw}$ = yield strength of the web.

For major axis bending & hot-rolled section:

$$A_v = h \times t_w = 200 \times 5.4 = 1080\ mm^2.$$

$f_{yw}$ = yield strength of the plate material

$= 250\ N/mm^2$.

$$\Rightarrow V_p = \frac{1080 \times 250}{\sqrt{3}} = 155.88\ kN = V_n.$$

$$\Rightarrow V_d = \frac{V_n}{\gamma_{m0}} = \frac{155.88}{1.1} = 141.70\ kN. > 30kN.$$

Hence, the section is safe against shear.

Check for high / low shear:

For the check of shear, we have $0.6 V_d = 0.6 \times 141.7 = 85.02$ kN $> 30kN$.

Hence, it is a case of low shear.

ii) Check for design bending strength :-

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

$$= \frac{1 \times 250 \times 1.84 \times 10^5}{1.1} = 41.81 \text{ kN-m} > 30 \text{ kN-m}.$$



Hence, it is safe against applied bending strength.

iii) Check for web buckling :-

$$\frac{d}{t_w} > 67\epsilon \quad \text{(for a web without stiffness)}$$

$$\text{(or)} \quad \frac{d}{t_w} > 67\epsilon\sqrt{\frac{k_v}{5.35}} \quad \text{(for a web with stiffness)}$$

$$\Rightarrow \frac{d}{t_w} = \frac{166.4}{5.4} = 30.81 < 67\epsilon \quad \left[\epsilon = \sqrt{\frac{250}{f_y}} = 1\right]$$

Hence, the section is safe against web shear buckling. So, no check for web buckling is required.



iv) Check for web ~~crippling~~ deflection :-



Maximum deflection, $\Delta = \frac{5wl^4}{384EI}$

$$= \frac{5 \times 10 \times 10^3 \times (4 \times 10^3)^4}{384 \times 2 \times 10^5 \times 1.696 \times 10^7}$$

$$= 9.827 \text{ mm}.$$

$I$ = higher value of moment of inertia between $I_{xx}$ & $I_{yy}$.

Maximum allowable deflection = 9.827 mm.

Now from page - 31, table - 6:

Industrial building - vertical - live load - for simple span - brittle cladding = $\frac{l}{300} = \frac{4000}{300} = 13.34$ mm.

$\therefore (9.827 < 13.34)$ mm.

Hence, the section is safe against deflection.

vi) <u>Check for web crippling</u>:-

$$F_w = \frac{(b_1 + n_2)\, t_w\, f_{yw}}{\gamma_{m0}}$$

where, $n_2$ = length obtained by dispersion through the flange to the web.

$$= (t_f + r_1)\, 2.5 = (7.3 + 9.5) \times 2.5$$

$$= 42 \text{ mm}.$$

$b_1$ = stiff bearing length (8.7.1.3)

[*** $b_1$ is the length to be provided by the designer to provide stiffners to resist lateral movement of the beam. ***]

Assuming an higher value of $b_1$ = 100 mm.

$$\therefore F_w = \frac{(100 + 42) \times 5.4 \times 250}{1.1} = 174.27 \text{ kN}.$$

$\therefore$ 174.27 kN > 30 kN.

Hence, the section is safe against web crippling.

$\therefore$ Provide ISLB 200 @ 194.2 which will be used as a beam as the steel joist.

# Design of Laterally Unsupported Beam :- (P-54, 8.2.2)

Q.) Design a laterally unsupported beam for the following data: effective span = 4m, Maximum BM = 550 kN-m, Maximum shear force = 200 kN, the grade of the steel to be used is Fe410.

A) Design of laterally unsupported beam is similar to the laterally supported beam w.r.t. the design steps. The $(Z_p)_{req.}$ has to be increased by 30% (min.) in case of laterally unsupported beam.

For Fe410 grade steel:-

$f_u = 410 \ N/mm^2$

$f_y = 250 \ N/mm^2$

$\gamma_{m0} = 1.1$.

Plastic section modulus, $Z_p = \frac{1.3 M . \gamma_{m0}}{f_y} = \frac{550 \times 1.1 \times 1.3 \times 10^6}{250}$

$\Rightarrow (Z_p)_{req.} = 3.146 \times 10^6 \ mm^3$.

From page - 139, let us select a beam section ISMB 600 @ 1222.71 N/m for design of laterally unsupported beam sec$^n$.

$h = 600mm$    $t_w = 12mm$    $I_y = 2.65 \times 10^7 \ mm^4$

$b_f = 210mm$    $r_1 = 20mm$    $I_x = 9.1813 \times 10^8 \ mm^4$

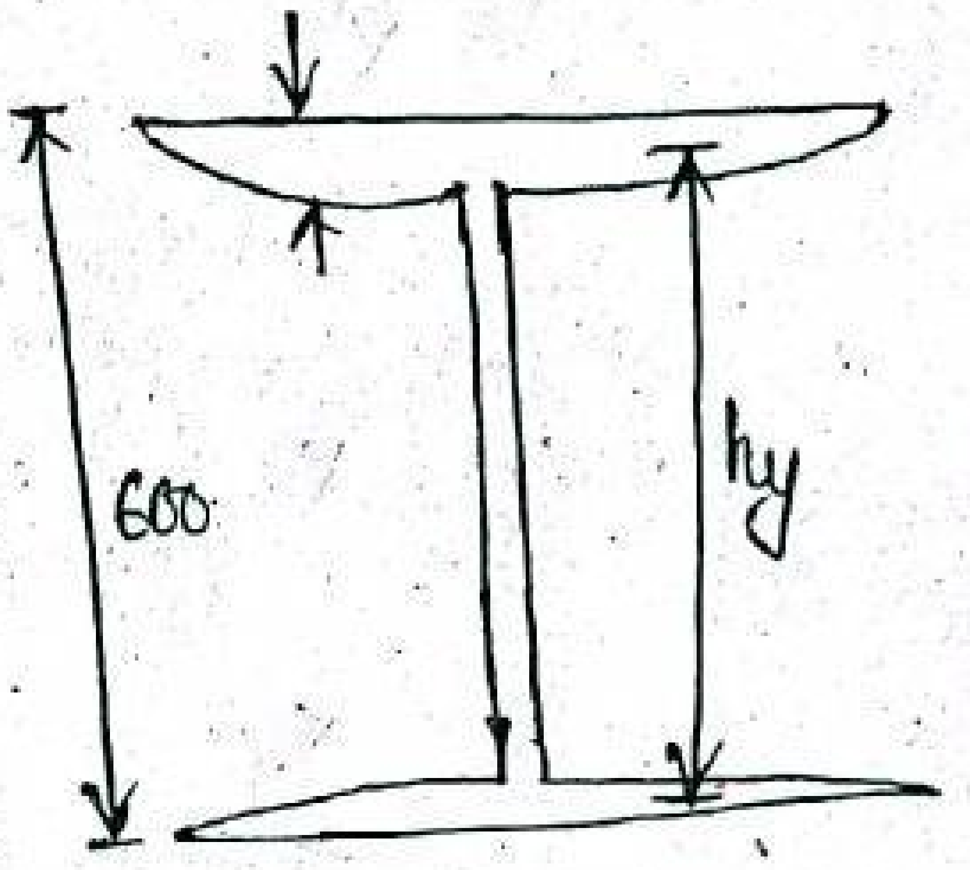
$t_f = 20.8mm$    $r_2 = 10mm$

$Z_e = 3.06 \times 10^6 \; mm^3$

$Z_p = 3.51 \times 10^6 \; mm^3$.

Sectional classification :- (page - 18, table - 2)

$\epsilon = \sqrt{\frac{250}{f_y}} = 1$.



$\frac{b}{t_f}$ ; $b = \frac{b_f}{2} = \frac{210}{2} = 105mm$

$\frac{d}{t_w}$ ; $d = 600 - 2 \times (10.8 + 20)$

$= 518.4 \; mm$

$\boxed{d = h - 2(t_f + r_1)}$

$\frac{b}{t_f} = \frac{105}{20.8} = 5.05 < 9.84 \epsilon$

$\frac{d}{t_w} = \frac{518.4}{12} = 43.2 < 84\epsilon$.

Hence, the section is classified as plastic section



i) Check for shear :-

$V_d = \frac{V_n = V_p}{\gamma_{m0}}$

$V_p = \frac{A_v f_{yw}}{\sqrt{3}}$

$\Rightarrow A_v = h \times t_w = 600 \times 12 = 7200 mm^2$.

$f_{yw} = f_y = 250 \; N/mm^2$.

$\therefore V_d = \frac{7200 \times 250}{\sqrt{3} \times 1.1} = 944.75 \; kN > 200 \; kN$.

Hence, the section is safe against shear.

ii) Check for design bending strength:-

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

$$X = \frac{1 \times 250 \times [3.51 \times 10^6 + \cancel{(30\% \times 3.51 \times 10^6)}]}{1.1}$$

$$X = \frac{1 \times 250 \times 3.51 \times 10^6}{1.1}$$

$$X =$$

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

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

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

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

$\alpha_{LT} = 0.21$ (for rolled steel sections)

$$\lambda_{LT} = \sqrt{\frac{\beta_b Z_p f_y}{M_{cr}}}, \quad G = \text{page-12, 2.2.4.1} = 0.769 \times 10^5 \ N/mm^2$$

$M_{cr}$ = elastic critical moment

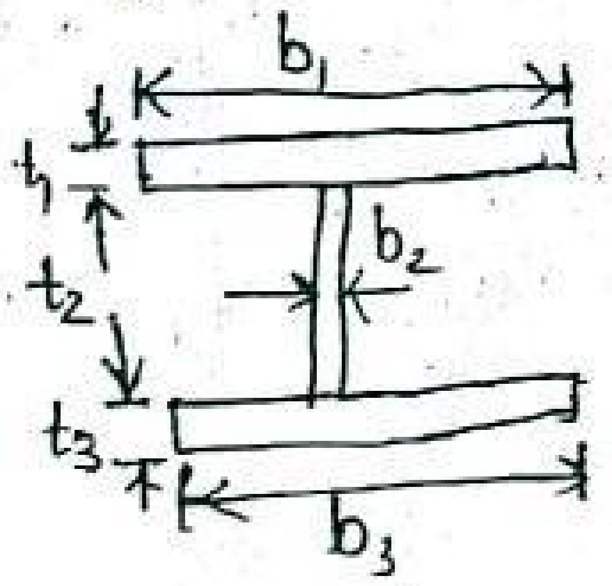
$$= \sqrt{\left\{\left(\frac{\pi^2 E I_y}{(L_{LT})^2}\right)\left[G I_t + \frac{\pi^2 E I_w}{(L_{LT})^2}\right]\right\}}$$

$L_{LT}$ = effective length for lateral torsional buckling.

$$M_{cr} = \sqrt{\left\{\left(\frac{\pi^2 \times 2\times 10^5 \times 2.65\times 10^7}{4000}\right) \times \left[G I_t + \frac{\pi^2 E I_w}{(L_{LT})^2}\right]\right\}}$$

$$I_t = \text{torsional constant} = \frac{\Sigma\, b_i t_i^3}{3}$$

$$= \left(\frac{210 \times 20.8^3}{3}\right) \times 2 + \frac{[600 - (2\times 20.8)] \times 12^3}{3}$$




*** While calculating '$b_2$', we dont have to substract the radius at the root, i.e., '$r_1$'. ***

$$I_t = 1.259\times 10^6 + 321.638\times 10^3$$
$$= 1.58 \times 10^6.$$

$I_w$ = warping constant (page - 129)

$$= (1-\beta_f)\,\beta_f\, I_y\, h_y^2$$

$$\beta_f = \frac{I_{fc}}{I_{fc}+I_{ft}} = \frac{I}{2I} = 0.5.$$

$I_{fc}$, $I_{ft}$ = MOI of the compression & tension flanges

As the dimension of compression & tension flanges, hence, the moment of inertia at the compression & tension flange will be equal, i.e., $I_{fc} = I_{ft} = I$.

$h_y$ = distance between shear centre of the two flanges.

$$= 600 - \left(2\times \frac{20.8}{2}\right) = 579.2 \text{ mm}.$$

Now, $I_w = (1-0.5) \times 0.5 \times 2.65 \times 10^7 \times 579.2^2$

$= 2.222 \times 10^{12}$.

$$\Rightarrow M_{cr} = \sqrt{\frac{3.2693 \times 10^6}{\cancel{1.3077 \times 10^{10}}} \times \left[ (0.769 \times 10^5 \times 1.58 \times 10^6) + \left( \frac{\pi^2 \times 2 \times 10^5 \times 2.222 \times 10^{12}}{4000^2} \right) \right]}$$

$$= \sqrt{(3.2693 \times 10^6) \times (1.21 \times 10^{11} + 2.741 \times 10^{11})}$$

$= \cancel{1138.25 \text{ kN-m}}\ 1138.25 \times 10^6$ kN-m.

Now, $\lambda_{LT} = \sqrt{\dfrac{1 \times 3.51 \times 10^6 \times 250}{1138.25 \times 10^6}} = 0.878$.

$\phi_{LT} = 0.5 \left[ 1 + 0.21 (0.878 - 0.2) + 0.878^2 \right]$

$= 0.5 \times (1 + 0.14238 + 0.770884)$

$= 0.956632 \doteq 0.9567$.

$$\chi_{LT} = \frac{1}{\{0.9567 + (0.9567^2 - 0.878^2)^{0.5}\}}$$

$= 0.748$

$f_{bd} = \dfrac{0.748 \times 250}{1.1} = 170$

$$M_d = 1 \times 3.51 \times 10^6 \times \cancel{...} 170 = 596.7 \; kN\text{-}m$$

$$> 550 \; kN.$$

Hence, the section is safe against bending.

iii) Check for ~~web~~ bending strength (shortcut) :-

In this method :

$$M_{cr} = \beta_b Z_p f_{crb}$$

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

$$= \frac{1.1 \times \pi^2 \times 2 \times 10^5}{\left(\frac{4000}{41.2}\right)^2} \left[1 + \frac{1}{20}\left(\frac{\left(\frac{4000}{41.2}\right)}{\left(\frac{579.2}{20.8}\right)}\right)^2\right]^{0.5}$$

$$= 230.354 \left[1 + (0.05 \times 12.156)\right]$$

$$= 230.354 \times 1.60$$

$$= 291.37$$

From page - 55, table 13-a, corresponding to the value of $\alpha_{LT} = 0.21$, the design bending compressive stress for lateral buckling is given as:-

$$250 = 152.3$$

$$291.37 = x$$

$$300 = 163.6$$

By interpolation, $x = f_{bd} = 161.81$

The design bending strength, $M_d = \beta_b Z_p f_{bd}$

$$= 1 \times 3.51 \times 10^6 \times 161.81$$

$$= 567.95 \text{ kN/m.}$$

$$> 550 \text{ kN/m.}$$

iv.) Check for web buckling :-

$$\frac{d}{t_w} > 67\,\epsilon \quad \text{(for a web without stiffness)}$$

$$\frac{d}{t_w} > 67\,\epsilon \sqrt{\frac{K_v}{5.35}} \quad \text{(for a web with stiffness)}$$

$$\therefore \frac{d}{t_w} = 43.2 < 67\,\epsilon.$$

Hence, the section is safe against web buckling.

v.) Check for web deflection :-

Maximum deflection, $\Delta = \dfrac{5wl^4}{384EI}$

$= 5\times$

v.) Check for web crippling :-

$$F_w = \frac{(b_1 + n_2)\, t_w\, f_{yw}}{\gamma_{m0}}$$

$b_1 = 100$ mm, $n_2 = 2.5(t_f + r_1) = 102$ mm.

vii) $$F_w = \frac{(100 + 102) \times 12 \times 250}{1.1} = 550.9 \text{ kN} > 220 \text{ kN}$$

Hence, the section is safe against web crippling.

vi) Check for web deflection:-

The check for deflection for laterally unsupported beam is same as the check for laterally supported beam.

END OF MODULE - 3

# Eccentric Moment Connection

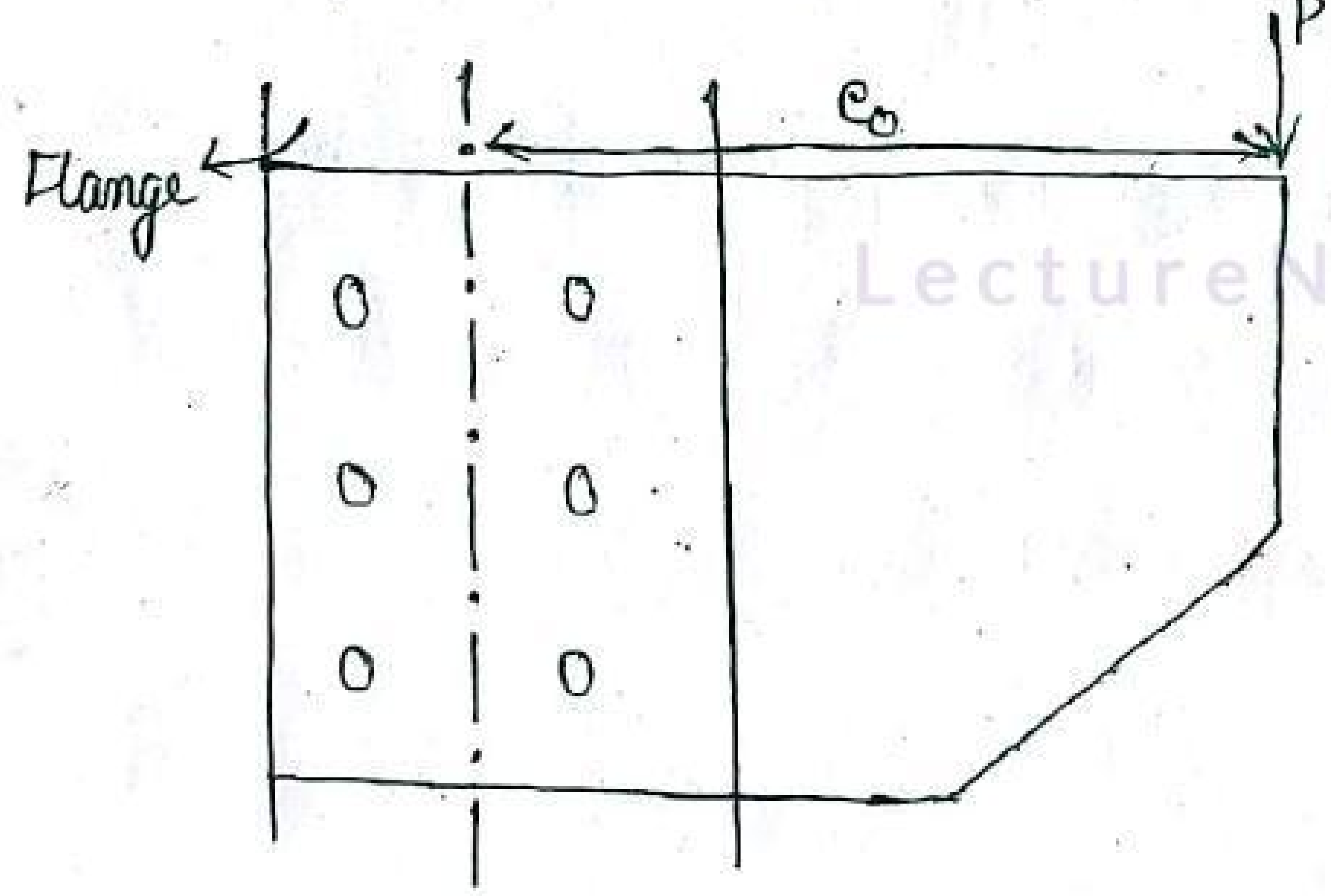
→ These are of 2 types :-

i) Bolted bracket connection type - 1 (for L & I sec^n).

ii) Bolted bracket connection type - 2 (for T - section).

## # Bolted Bracket Connection Type - I :-

→ When twisting moment is in the plane of connection, the connection may be termed as bracket connection type-I. This situation may arrive when the line of action of load is in the plane of bolted connection & the centre of gravity of connection (elastic method) is the centre of rotation. The bolt group is subjected to shear as well as torsion.




(when connected to flange, it is type-1 & when it is connected to web it is type-2.)

* Design Steps :-

i) Force in the bolts due to the direct load P.

$$\boxed{f_1 = \frac{P}{n}}$$ (direct shear)

where, $n$ = no. of bolts

$P$ = applied load

2) The torque, $\boxed{M = Pe_0}$, where $e_0$ = distance of the load from the centre of bolt arrangement.

3) Force on the extreme bolts due to the torque, i.e.:-

$$\boxed{f_2 = \frac{Pe_0 r_n}{\Sigma r^2} = \frac{M r_n}{\Sigma r^2}}$$

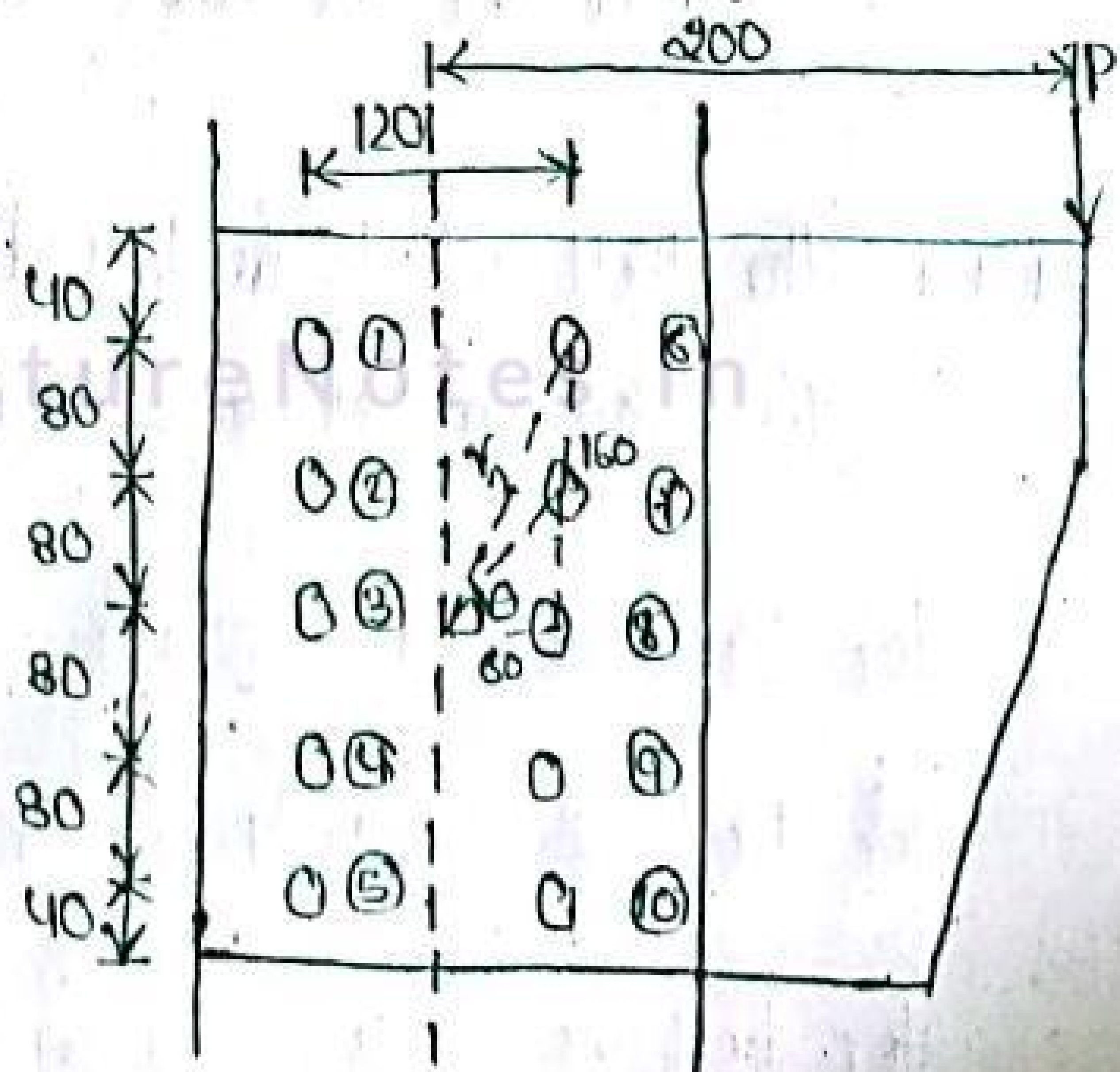
where, $r_n$ = distance of the extreme bolt.

$r$ = shortest distance from the centre of the bolt arrangement.

4) The required force, $\boxed{F = f_1^2 + f_2^2 + 2 f_1 f_2 \cos\theta}$

where, $\theta$ = angle between $f_1$ & $f_2$.

Q) Determine the safe load 'P', that can be carried by the joint as shown in the figure. The bolts used are of 20mm dia & of grade 4.6. The thickness of the flange is 2.1mm & thickness of the bracket plate is 10mm.




A) Assuming the grade of plate as Fe410.

$f_{ub}$ = 400 N/mm².

$f_u$ = 410 N/mm².

$f_y$ = 250 N/mm².

$\gamma_{mb}$ = 1.25

dia, $d$ = 20 mm.

$d_0$ = 20 + 2 = 22mm

minimum pitch, $P_{min} = 2.5 \times d = 2.5 \times 20 = 50$ mm.

minimum edge distance, $e_{min} = 1.5 \times d_0 = 1.5 \times 22 = 33$mm

Strength of bolt in shear:- 

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

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

$$= 45.272 \text{ kN}$$

Assuming shearing strength of the bolt is more than the bearing strength, hence the strength of the bolt is 45.27 kN.

Let '$P_1$' = factored load that can be carried by the bolts.

Service load / working load, $P = \frac{P_1}{FOS}$

Taking Factor of safety, FOS = 1.5.

$$\Rightarrow P = \frac{P_1}{1.5}$$

The direct force, $f_1 = \frac{P}{n} = \frac{P}{10} = 0.1 P.$

Torque, $M = P \times e_0 = 200 P.$

Force on the bolt due to torque, $f_2 = \frac{M r_n}{\Sigma r^2}$

*** The bolt which will carry the load or face the load first will be taken as extreme bolt. ***

For bolt nos. 1-5-6-10: $r_1 = 170.88$ mm, $r_1^2 = 29199.9744$

For bolt nos. 2-4-7-9: $r_2 = \sqrt{60^2 + 80^2} = 100$ mm, $r_2^2 = 10000$.

For bolt no. 3-8: $r_3 = 60$, $r_3^2 = 3600$

$$\Sigma r^2 = 4r_1^2 + 4r_2^2 + 2r_3^2$$

$$= 4(29199.9744) + 4(10000) + 2(3600) = 163999.8976.$$

$$\Rightarrow f_2 = \frac{Mr_n}{\Sigma r^2} = \frac{200P \times 170.88}{163999.8976}$$

$$= 0.208390374 \times P.$$

The resultant force on the bolt is $F = \sqrt{f_1^2 + f_2^2 + 2 f_1 f_2 \cos\theta}$

$$\cos\theta = \frac{60}{170.88} = 0.3511.$$

$$\Rightarrow F = \sqrt{(0.1P)^2 + (0.2084P)^2 + (2 \times 0.1 \times 0.2084 \times 0.36)P^2}$$

$$= \sqrt{0.01P^2 + 0.0434P^2 + 0.0150P^2}$$

$$= 0.2616\ P.$$

By equating the resultant force with the strength of the bolt, we have:

$$0.2616\ P = 45.272 \times 10^3$$

$$\Rightarrow P = \frac{45.272 \times 10^3}{0.2616} = 173.05\ kN.$$

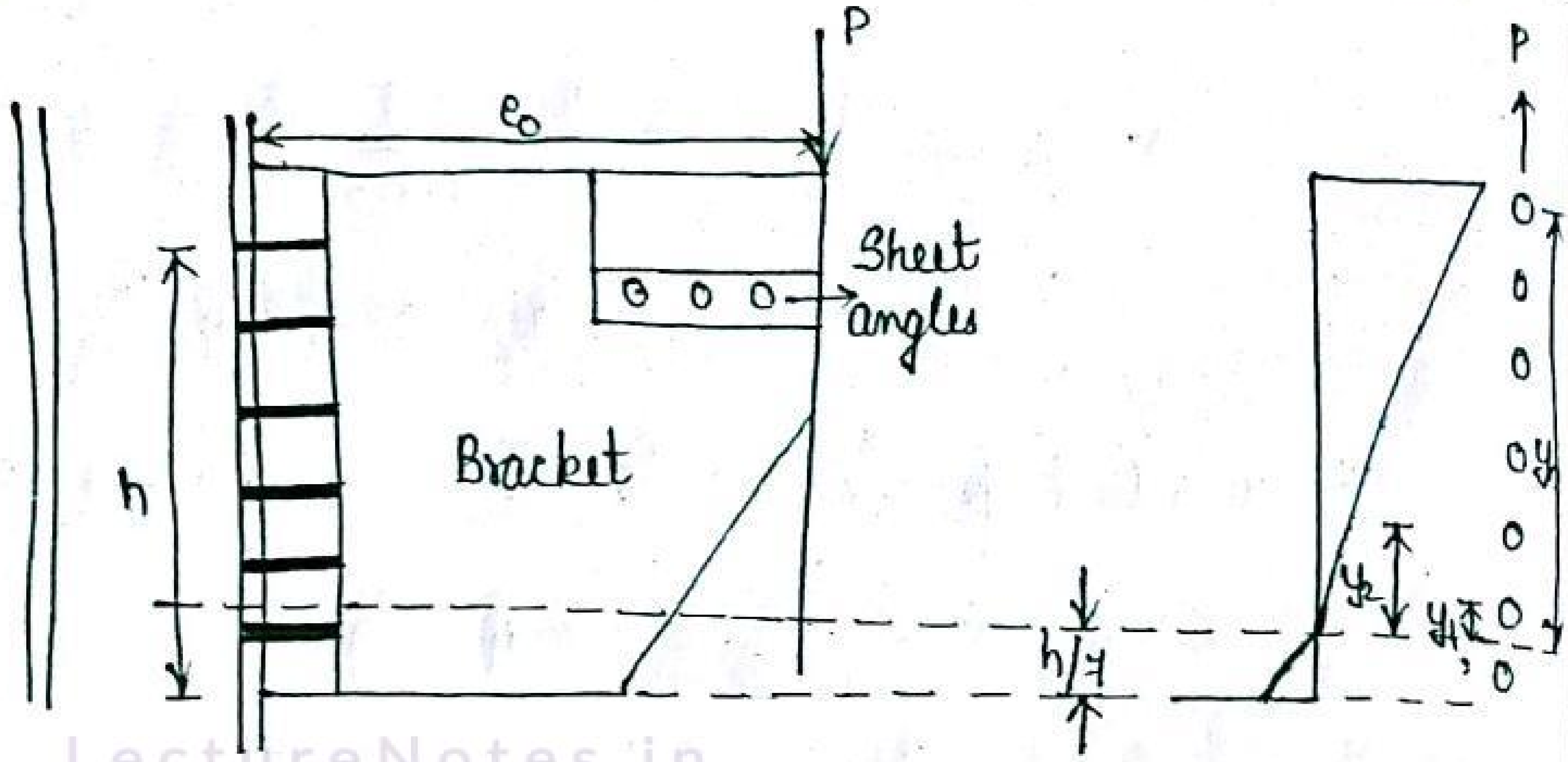
The factor load that can be carried, $P_1 = P \times 1.5$

$$= 173.05 \times 1.5$$

$$= 260\ kN.$$

# Bolted Bracket Connection Type - II :-

→ When the moment is in a plane ⊥r to the plane of connection, the bracket connection is termed as bracket connection type - 2. The line of action of load does not lie in the plane of group of bolts (or) bolt arrangement and the line of rotation does not pass through the centre of gravity of the bolt holes. The bolts are subjected to direct shear along with tension due to moment.






* **Design Steps:-**

1) Force in a bolt due to direct shear or loading, $V = \frac{P}{n}$.

2) The tensile force in the extreme critical bolt due to bending:

$$\boxed{T_e = \frac{M' y_n}{\Sigma y_i^2}}$$

where, M' = moment of resistance provided by bolts in tension.

$$\boxed{M' = \frac{M}{1 + \frac{2h}{21}\left(\frac{\Sigma y_i}{\Sigma y_i^2}\right)}} \qquad (M = Pe_0)$$



$e_0$ = eccentricity of the load 'P' from the bolt plane to the line of action of load.

$y_1, y_2, y_3, \ldots, y_n$ = distance of bolts in tension from the axis of rotation.



*** The axis of rotation is assumed to be at a distance of $\frac{h}{7}$ from the bottom of the plate & where h = height of the plate. ***

3) No. of bolts required in one row = n.

$$\boxed{n = \sqrt{\frac{6M}{p n' V_{sd}}}}$$

where $M$ = applied maximum / factored moment.

$n'$ = no. of rows of bolt line.

$p$ = pitch

$V_{sd}$ = strength of the bolt in shearing or bearing (whichever is lesser).

* Code Provisions :-

→ Strength of the bolt in tension → P-76, C-10.3.5.

→ Bolt subjected to combined shear & tension → P-76, C-10.3.6.

Q) Design a bracket connection to transfer a maximum shear force (end reaction) of 225 kN due to the factored loads. The end reaction from the girder is at eccentricity of 300 mm from the face of the column flange. Design the bolted joint connecting the flanges. Use steel of grade Fe410 & bolt of grade 4.6.

A) Given data:

$f_u = 410\ N/mm^2$

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

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

The end reaction = maximum shear force = 225 kN.

Corresponding, BM due to the end reaction = $(225 \times 10^3) \times$ eccentricity

$= 225 \times 10^3 \times 300 = 67.5$ kN-m.

Assume, dia of bolt = 24mm = d.

$\Rightarrow d_0 = (24+2)$ mm = 26 mm.

$P_{min} = 2.5 \times d = 2.5 \times 24 = 60$ mm

$e_{min} = 1.5 \times d_0 = 1.5 \times 26 = 39$ mm.

Providing, $P = 65mm$ & $e = 40mm$.

Strength of bolt in shearing:-

$$V_{dsb} = \frac{f_{ub}}{\sqrt{3}\,\gamma_{mb}}\left(A_{nb}\, n_n + n_s \cancel{A_{sb}}^{\,0}\right)$$

$$= \frac{400}{\sqrt{3} \times 1.25}\left(1 \times 0.78 \times \frac{\pi}{4} \times 24^2\right) = 65.192 \text{ kN}$$

As bearing strength of the bolt is higher than the shearing strength, hence bearing strength calculation is not required.

Strength of bolt in tension:-

$$T_{db} = \frac{T_{nb}}{\gamma_{mb}} = \frac{0.9 \times 400 \times 0.78 \times \frac{\pi}{4} \times 24^2}{1.25} = 101.624 \text{ kN}$$

So, the strength of the bolt is 65.192 kN.

Let us provide the no. of bolts in 2 vertical rows:

$$n = \sqrt{\frac{6M}{p n' V_{sd}}} \quad (\because n' = 2)$$

$$= \sqrt{\frac{6 \times 67.5 \times 10^6}{65 \times 2 \times 65.192}}$$

$$= 6.91 \simeq 7 \text{ nos. (1 row)}$$

for 2 rows, $n = 7 \times 2 = 14$ nos.

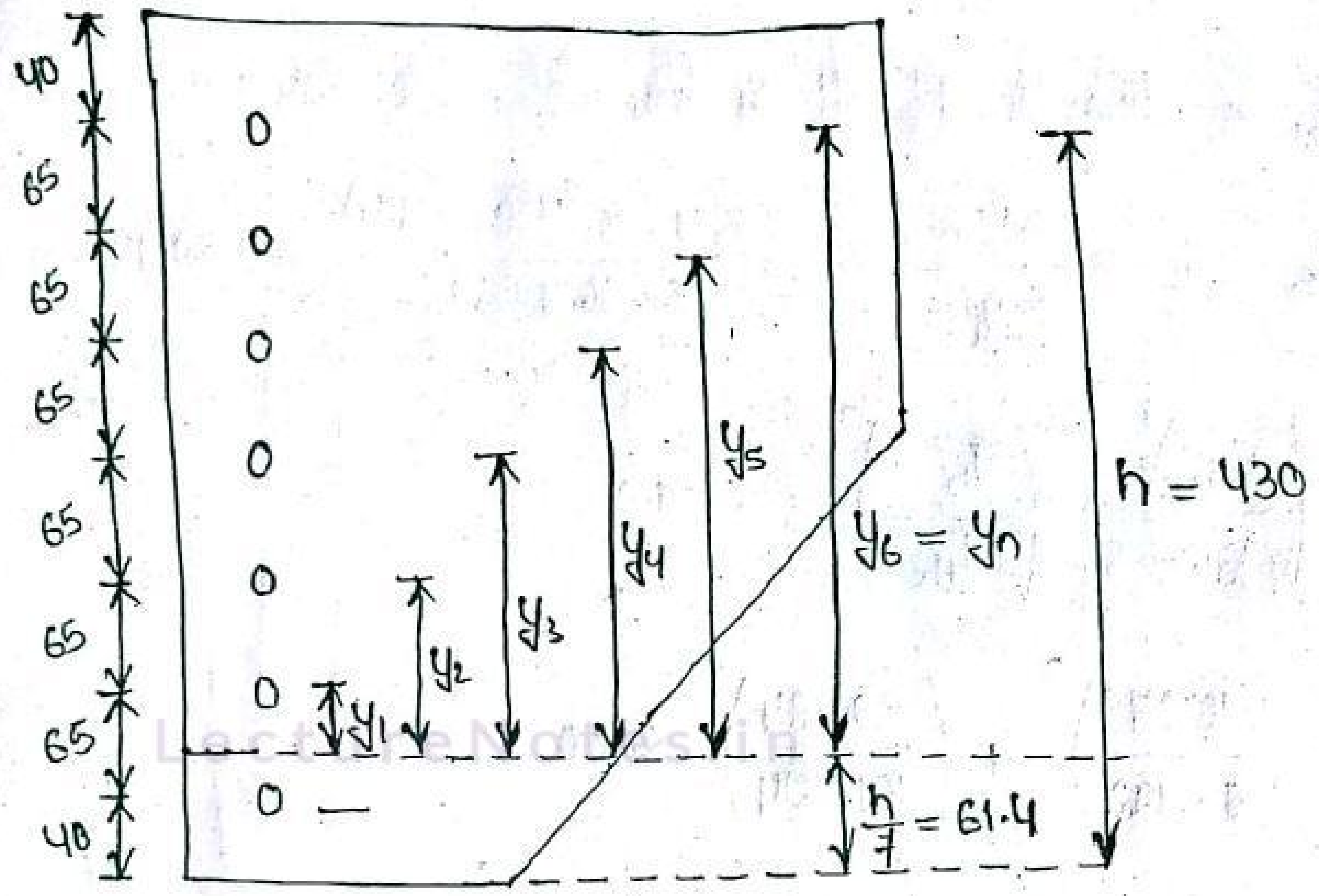
Direct shear, $V = \frac{P}{n} = 16.07$ kN

Effective depth of the total bracket plate, $h = (65 \times 6) + 140 = 430$ mm

NA is assumed to lie at the height of $\frac{h}{7} = \frac{430}{7} = 61.42$ from the bottom of the bracket plate.

$$\Sigma y_i = 2 \times (y_1 + y_2 + y_3 + y_4 + y_5 + y_6)$$

$$\Sigma y_i^2 = 2 \times (y_1^2 + y_2^2 + y_3^2 + y_4^2 + y_5^2 + y_6^2)$$




$$\Rightarrow \Sigma y_i = 2 \times \big[(65+40-61.4) + (65+65+40-61.4) + (65+65+65+40 - 61.4) + (4\times65+40-61.4) + (5\times65+40-61.4) + (6\times65+40-61.4)\big]$$

$$= 2 \times (43.6 + 108.6 + 173.6 + 238.6 + 303.6 + 368.6)$$

$$= 2473.20 \text{ mm.}$$

$$\Rightarrow \Sigma y_i^2 = 2 \times \big[(43.6)^2 + (108.6)^2 + (173.6)^2 + (238.6)^2 + (303.6)^2 + (368.6)^2\big]$$

$$= 657601.52 \text{ mm.}$$

Moment of resistance, $M' = \dfrac{M}{1 + \dfrac{2h}{21}\left(\dfrac{\Sigma y_i}{\Sigma y_i^2}\right)}$

$$= \frac{67.5}{1 + \left(\dfrac{2\times430\times2473.20}{21\times657601.52}\right)}$$

$$= 58.49 \text{ kN-m.}$$

<u>Bolt subjected to combined shear and tension:-</u>

$V_{sb} = V = 16.07$ kN.

$V_{db}$ = shearing strength = 65.192 kN

$T_{db}$ = strength of bolt in tension = 101.624 kN.

$$T_b = T_e = \frac{M' y_n}{\Sigma y_i^2} = \frac{58.49 \times (430 - 61.4)}{654601.52} = 32.18 \text{ kN.}$$

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

$$\Rightarrow \left(\frac{16.07}{65.192}\right)^2 + \left(\frac{32.184}{101.624}\right)^2 \leq 1.0$$

$$\Rightarrow 0.1648 \leq 1.0$$

Hence, the design is safe for bracket connection type - 2.

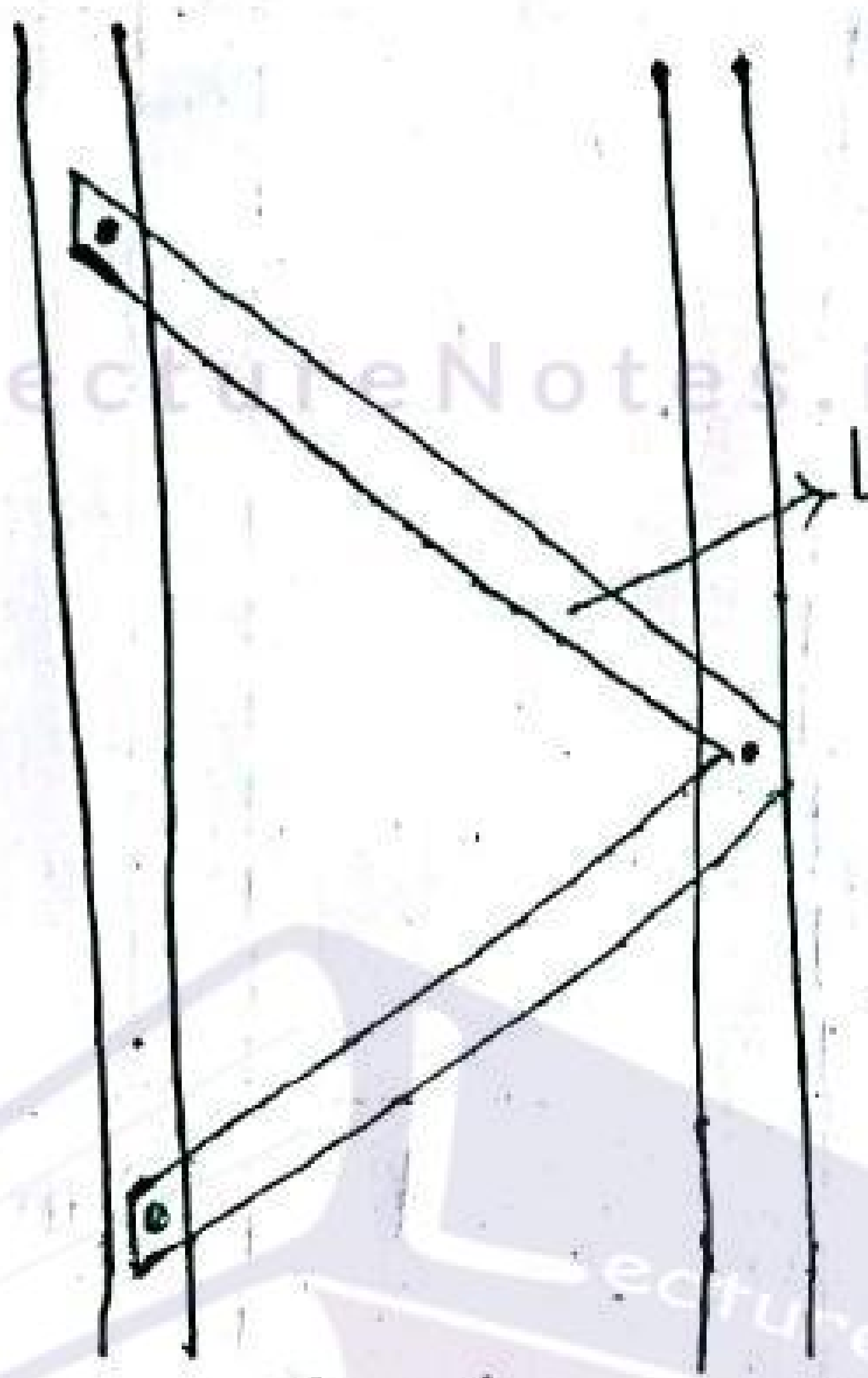
MODULE 4.2 → after MODULE - 3.3



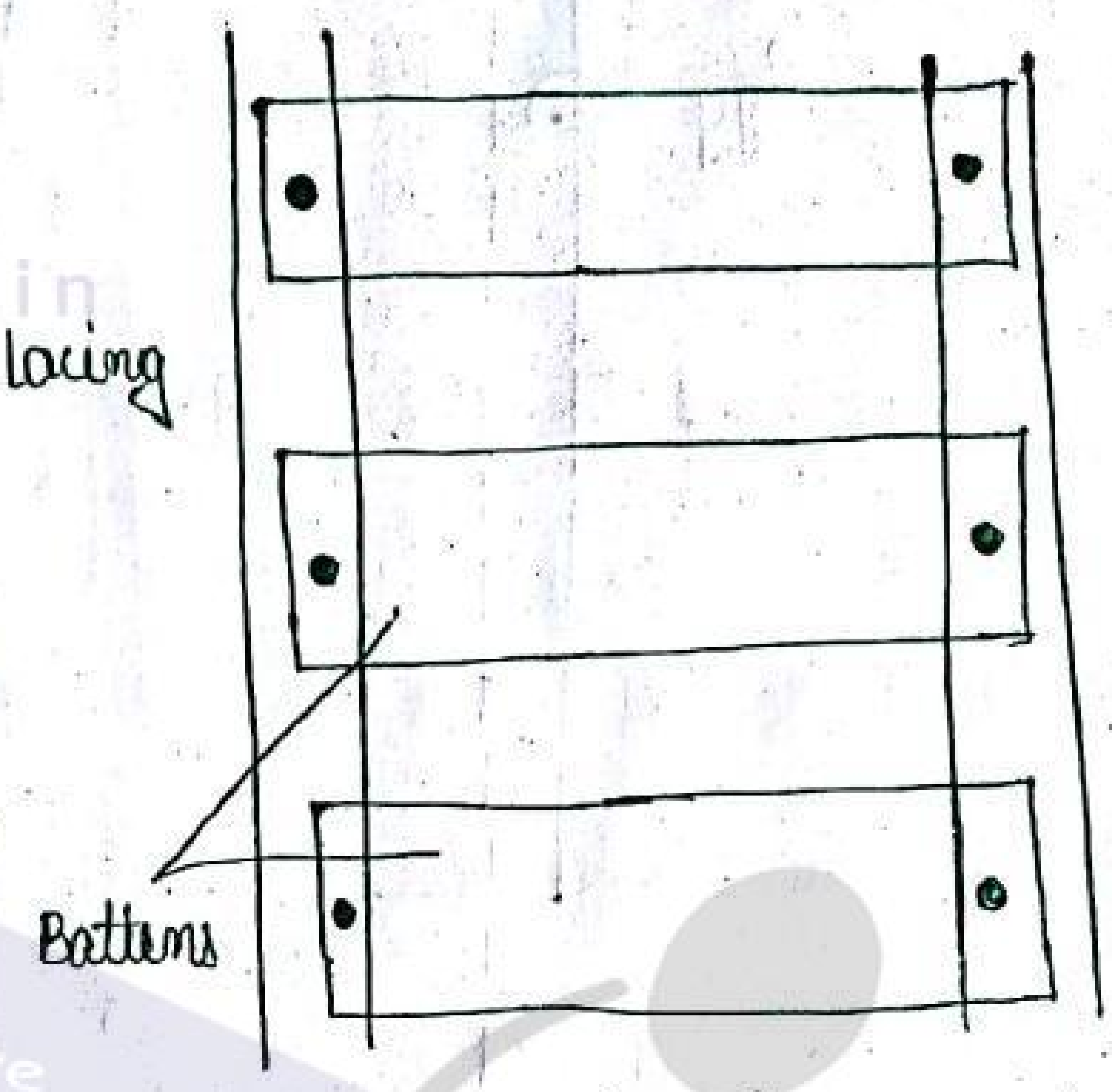


END OF MODULE 4.1

# LACING AND BATTENING




(F - 1)




(F - 2)

## # Requirement of Lacing & Battening:-

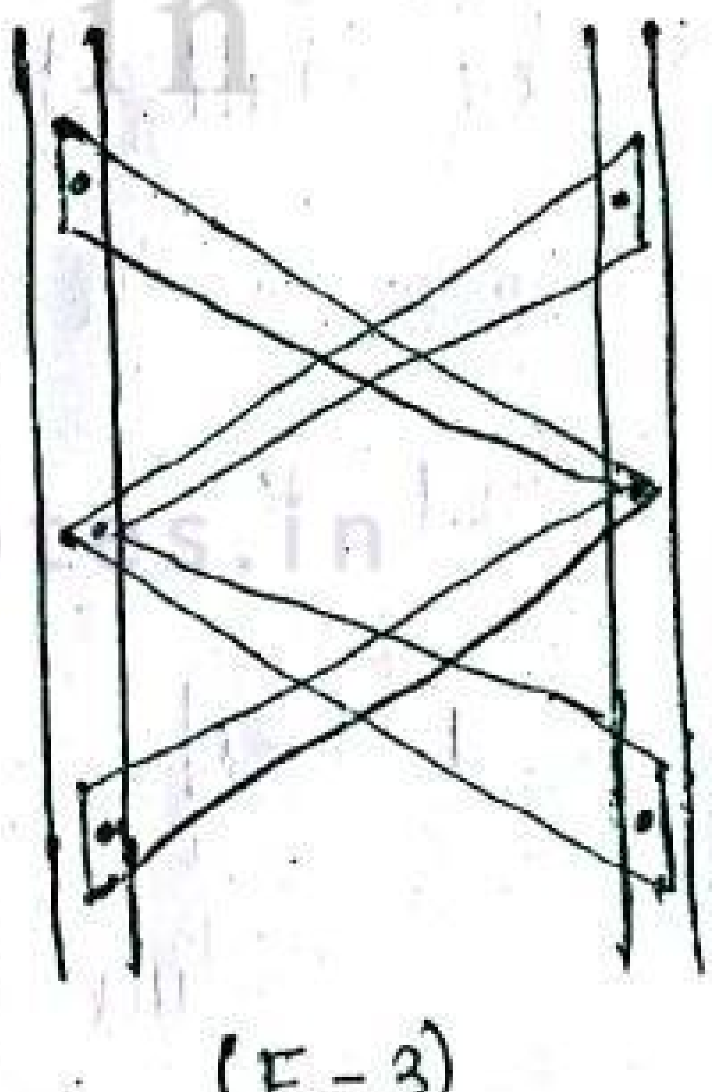
→ To maintain the spacing constant through out the span.

→ It helps in load transfer.

→ The lacing can be provided as follows:-

a) Single lacing system (F-1)

b) Double lacing system. (F-3)

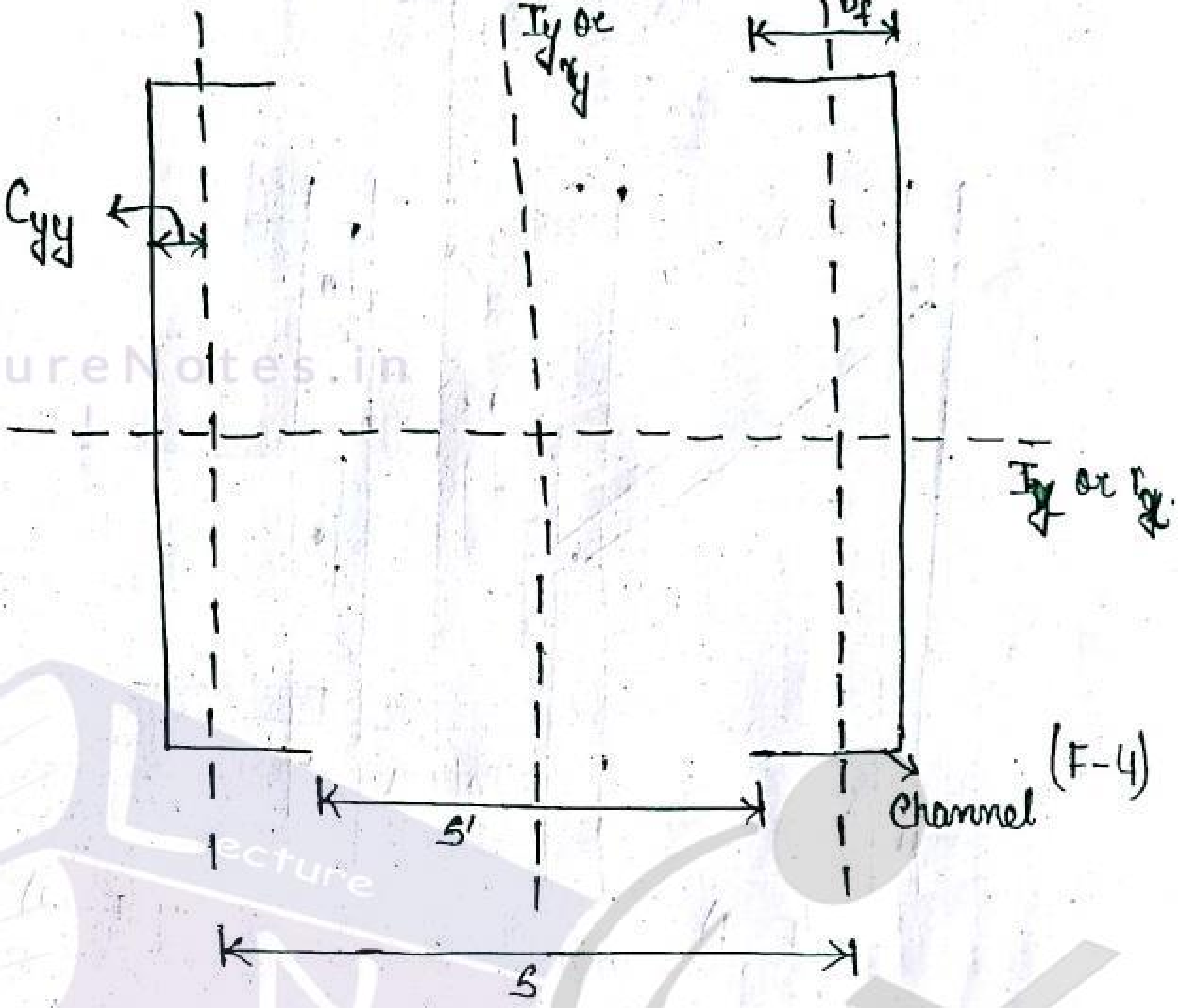


(F - 3)

## # Difference of Lacing & Battening:-

→ Lacing are the diagonal or inclined manner. The angle of inclination varies between 40° to 60° & the standard one is 45°. Battening are the horizontal members.

→ Lacing ~~stay~~ only take axial force (as in case of truss). Battening takes the moment created by the applied force & resist it by bending & shear (as in case of beams).



(F-4)

# * Lacing :-

## # Radius of Gyration of the Composite Section :-

→ For built-up or composite section, the radius of gyration is same as the radius of gyration of single rolled steel section w.r.t x or z axis.

* Proof :-

Radius of the gyration of the single rolled section = $r_x$

" " " " " " composite section = $r_{xx}$

$$\text{where, } r_x = \sqrt{\frac{I_x}{A}} \quad \& \quad r_{xx} = \sqrt{\frac{I_{x_1} + I_{x_2}}{A_1 + A_2}}$$

$$\Rightarrow r_{xx} = \sqrt{\frac{2 I_{x_1}}{2A}} = \sqrt{\frac{I_x}{A}} = r_x$$

As the two channel sections which are connected are same w.r.t. their sectional properties.

→ For the built-up section or composite section the z-z or x-x axis is the $\overset{\text{weaker}}{\cancel{\text{greater}}}$ axis

# Spacing Between Lacings :-

$$\boxed{I_{yy} = I_y + A\left(\frac{S}{2} + C_{yy}\right)^2}$$

where, S = spacing.

$I_{yy}$ = M.O.I for composite section.

$I_y$ = M.O.I of the single rolled steel section.

A = area of the section.

$C_{yy}$ = centre of gravity of the section.

# Code Provisions :-

a) Lacing member (P-48, 7.6)

b) Force carried by lacing member (P-50, 7.6.6.1)

c) Spacing (P-50, 7.6.5)

d) Slenderness ratio of lacing member (P-48, 7.6.1.5)

e) For design of lacing (P-50, 7.6.6)

f) Width of the lacing bar (P-50, 7.6.2)

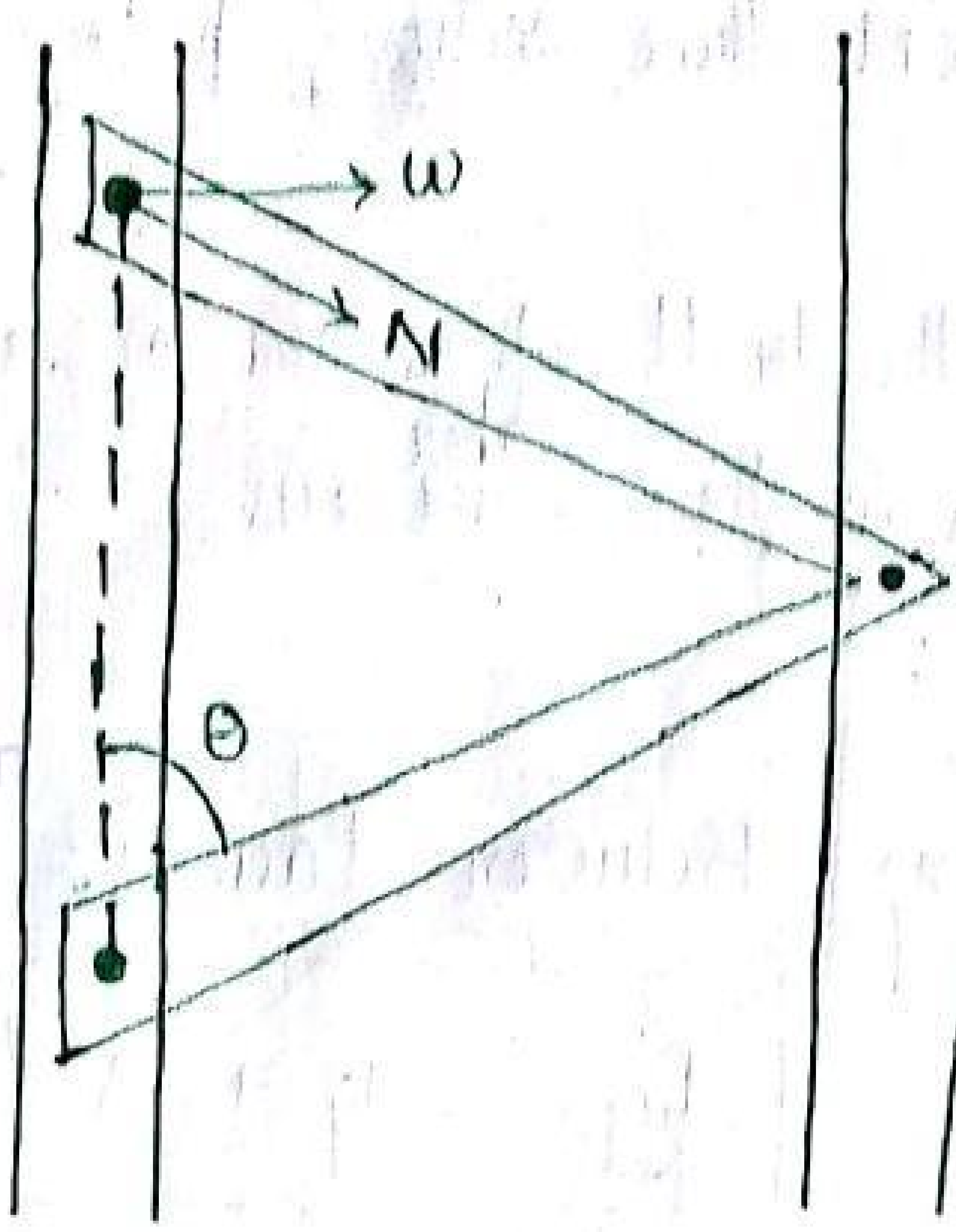
g) Thickness of the lacing bar (P-50, 7.6.3)

h) Angle of the inclination of lacing (P-50, 7.6.4)

# Load Carrying Capacity of a Lacing Member :-

→ The load carrying capacity of the lacing member = V.

$$\boxed{V = \frac{V_t}{N} \operatorname{cosec} \theta}$$




(F-5)

where $V_t$ is the shear capacity of the lacing, N is the no. of parallel plates to be connected.

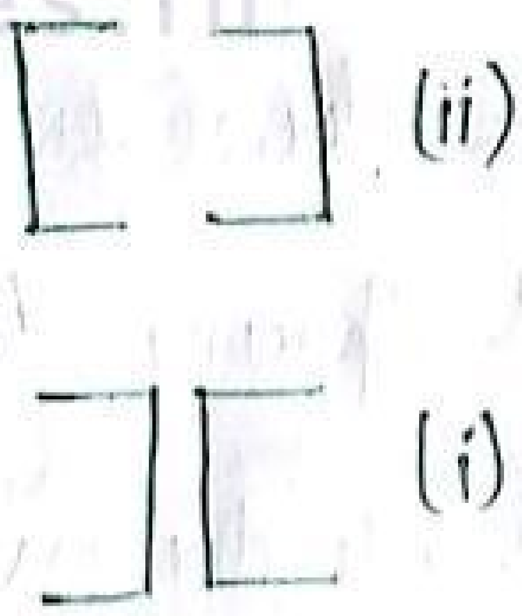
i.e., N is always equal to 2.

Q) Design a built-up column of 10m long to carry a factored axial load of 1080 kN. The column is restrained in position for not in direction at both the ends. Provide single lacing system with bolted connection for the following cases:

i.) Design the column with 2 channels placed back to back.

ii) Design the column with 2 channels placed toe to toe.

The grade of the steel is Fe410.




A) back to back - hill to hill.

toe to toe - face to face.

The factored load to be carried by the composite section is 1080 kN. Hence, the factored load to be carried for single section is 540 kN.

i) Case - 1 :-

When the members are placed back to back :-

For the compression member, the load carrying capacity :

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

assume $f_{cd} = 250 \ N/mm^2$

$$\Rightarrow A_e = \frac{P_d}{f_{cd}} = \frac{540 \times 10^3}{250} = 2160 \ mm^2$$

Increasing the area by 25% , $(A_e)_{req} = 2700 \ mm^2$.

Taking the section ISMC 200 @ 216.8 N, from steel table:

$A = 2821 \ mm^2$ $\quad t_w = 6.1 \ mm$.

$h = 200 \ mm$ $\quad C_{yy} = 2.17 \ cm = 21.7 \ mm$

$b_f = 75 \ mm$ $\quad I_{xx} = 1.81 \times 10^7 \ mm^4$

$t_f = 11.4 \ mm$ $\quad I_{yy} = 1.40 \times 10^7 \ mm^4$

$r_x = 80.3 \ mm$ $\quad r_y = 22.3 \ mm$.

Design strength of the selected channel section :

$$\lambda_z = \frac{KL}{r_z} = \frac{1 \times 10 \times 10^3}{80.3} = 124.5$$

$$f_{cc} = \frac{\pi^2 E}{\lambda_z^2} = \frac{\pi^2 \times 2 \times 10^5}{(124.5)^2} = 1.27 \times 10^2$$

(P-34, 7.1.2.1)

$$\lambda = \sqrt{\frac{f_y}{f_{cc}}} = \sqrt{\frac{250}{127}} = 1.40$$

For buckling class - c, $\alpha = 0.49$

$$\phi = 0.5 \left(1 + \alpha(\lambda - 0.2) + \lambda^2\right)$$

$$= 1.774$$

$\gamma_{m0} = 1.1$

$$f_{cd} = \frac{f_y / \gamma_{m0}}{\phi + (\phi^2 - \lambda^2)^{0.5}} = 79.36 \simeq 80 \text{ kN}$$

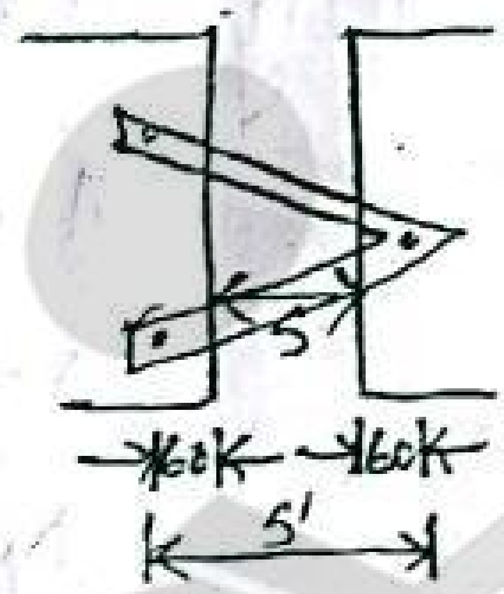
The design strength, $P_d = 2821 \times 80 = 225.68$ kN

i.e., < 540 kN.

As the strength of the column is less than the applied load, hence we have to redesign by taking ISLC 300 @ 324.7 N.

<u>Design for the lacing beam:-</u>

i) Spacing between two columns:




$$I_{yy} = I_y + A\left(\frac{S}{2} + C_{yy}\right)^2$$

By equating the MOI or radius of gyration to the weaker axis (x-x or z-z), we have:

For ISLC 300 @ 324.7 N/m:

| | |
|---|---|
| $A = 4211 \text{ mm}^2$ | $C_{yy} = 2.55 \text{ cm} = 25.5 \text{ mm}$ |
| $h = 300 \text{ mm}$ | $I_x = 6.047 \times 10^7 \text{ mm}^4$ |
| $b_f = 100 \text{ mm}$ | $I_y = 3.46 \times 10^6 \text{ mm}^4$ |
| $t_f = 11.6 \text{ mm}$ | $r_{xx} = 119.8 \text{ mm} = r_{zz}$ |
| $t_w = 6.7 \text{ mm}$ | $r_{yy} = 28.7 \text{ mm}.$ |

$$\Rightarrow \sqrt{\frac{I}{A}} = 119.8$$

$$\Rightarrow \sqrt{I_{yy}} = 119.8 \times \sqrt{A_2} = \cancel{7.77 \times 10^3}\ 10.99 \times 10^3$$

$$\Rightarrow I_{yy} = \cancel{104.811 \text{ mm}^4}\ 1.20 \times 10^8 \text{ mm}^4.$$

By increasing the MOI by 30%, we have:

$$I_{yy} = 1.56 \times 10^8 \text{ mm}^4.$$

Now, putting all the values in the above equation, we have:

$$1.56 \times 10^8 = 3.46 \times 10^6 + 4211\left(\frac{S}{2} + 25.5\right)^2$$

$$\Rightarrow 1.56 \times 10^8 - 3.46 \times 10^6 = \left(\frac{S^2}{4} + 650.25 + 25.5S\right)4211$$

$$\Rightarrow 1.53 \times 10^8 = 1052.75\,S^2 + (2601 + 102.5)\,4211$$

$$\Rightarrow 1052.75\,S^2 + 4.29 \times 10^5\,S = (1.53 \times 10^8 - 1.09 \times 10^7)$$

$$\Rightarrow S = 335 \text{ mm}.$$

Providing 20 mm dia bolt we have $P_{min} = 50$ mm, $e_{min} = 33$ mm

dia of the hole, $d_0 = 22$ mm.

So, taking pitch, $P = 60$ mm, $e = 40$ mm.

ii) Width of the lacing:- (page - 50, 7.6.2)

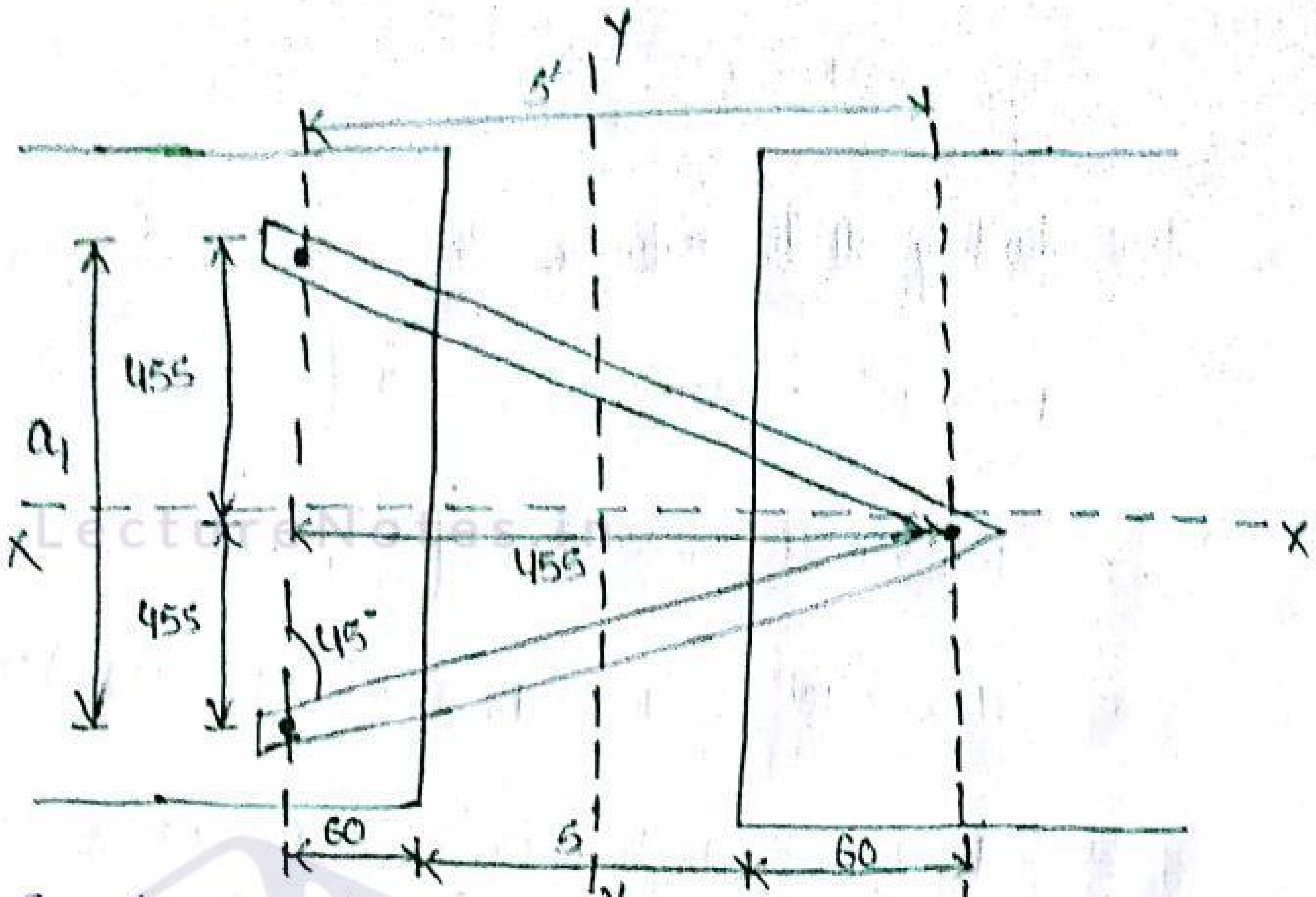
The width of the lacing $= 3 \times d = 60$ mm.

Hence, provide 2 channel sections ISLC 300 at a spacing of 335 mm with 60mm lacing bars.

iii) Effective length of the lacing:-

The min. space which is to be provided between the edge of the column & the centre of the bolt should be same as the width of the lacing plate.

$\Rightarrow S' = 335 + 60 + 60 = 455$ mm.




Providing the lacing bars at an angle of 45° towards the vertical, the length of the lacing bar is:

$$\sin 45° = \frac{P}{h}$$

$$\Rightarrow h = \frac{P}{\sin 45°} = \frac{455}{\sin 45°} = 643.46$$

$$\simeq 645 \text{ mm.} = \text{length.}$$

iv) Thickness of the lacing bar:

Thickness $= \frac{1}{40} \times$ effective length (length of lacing bar)

$$= \frac{1}{40} \times 645 = 16.125 \simeq 20 \text{ mm}$$

v) Spacing between 2 joints of the lacing bar: (page - 50, -16.5.1

From, the right angled triangle, we have:

$$\tan 45° = \frac{P}{b}$$

$$\Rightarrow b = \frac{P}{\tan 45°} = \frac{455}{1} = 455 \text{ mm.}$$

Hence, the distance between the 2 lacing connection:

$$= 455 + 455 = 910 \text{ mm.}$$

Check for the distance between the connection:

$$\frac{a_1}{r_1} = \frac{910}{28.7} = 31.707 < 50$$

$r_1$ = lowest radius of gyration.

$$\lambda_{unfavourable} = \frac{KL}{r_1} = \frac{1 \times 10 \times 10^3}{28.7} = 348.43 > 31.7$$

Hence, provide a single lacing system of lacing bar size of (645 x 60 x 20) mm at a spacing of 910 mm between the 2 connection of the member.

Load carrying capacity of the lacing member :-

$$V = \frac{V_t}{N} \operatorname{cosec} \theta.$$

$\Rightarrow V_t$ = shear capacity of the lacing member
= 2.5 % of the factored load. (page - 50, 7.6.6.1)
$= 2.5\% \times 1080 = 27$

Applied load on the lacing member $= \frac{27}{2} \operatorname{cosec} 45^\circ$
$= 19.09$ kN

Check for the design strength of lacing member:

i) Compression check :-

The radius of the gyration of the lacing bar, $r = \sqrt{\frac{I}{A}}$

where, $I$ = MOI of the lacing bar.

$$= \frac{60 \times 20^3}{12} = 4 \times 10^4 \ mm^4.$$

$A$ = area of the lacing member.

$$= 60 \times 20 = 1200 \ mm^2.$$

$$\Rightarrow r = \sqrt{\frac{4 \times 10^4}{1200}} = 5.77 \ mm.$$

$$\lambda = \frac{KL}{r} = \frac{1 \times 645}{5.77} = 111.785$$

For the lacing bar of solid plate, the buckling class is class c. From the IS code, P-42, T- 9(C), $f_y = 250 \ N/mm^2$:-

| $\lambda$ | $f_{cd}$ |
|---|---|
| 110 | 94.6 |
| 111.785 | 92.65 = $f_{cd}$ |
| 120 | 83.7 |

The design compressive strength of the lacing bar:-

$$P_d = A_e \times f_{cd} = 1200 \times 92.65$$

$$= 111.18 \ kN > 19.09 \ kN.$$

Hence, it is safe in compression.

ii) Tension check:-

a) Gross-sectional failure:

$$T_{dg} = \frac{A_g \cdot f_y}{\gamma_{m0}} = \frac{1200 \times 250}{1.1} = 272.73 \ kN > 19.09 \ kN$$

b) Net-sectional rupture :

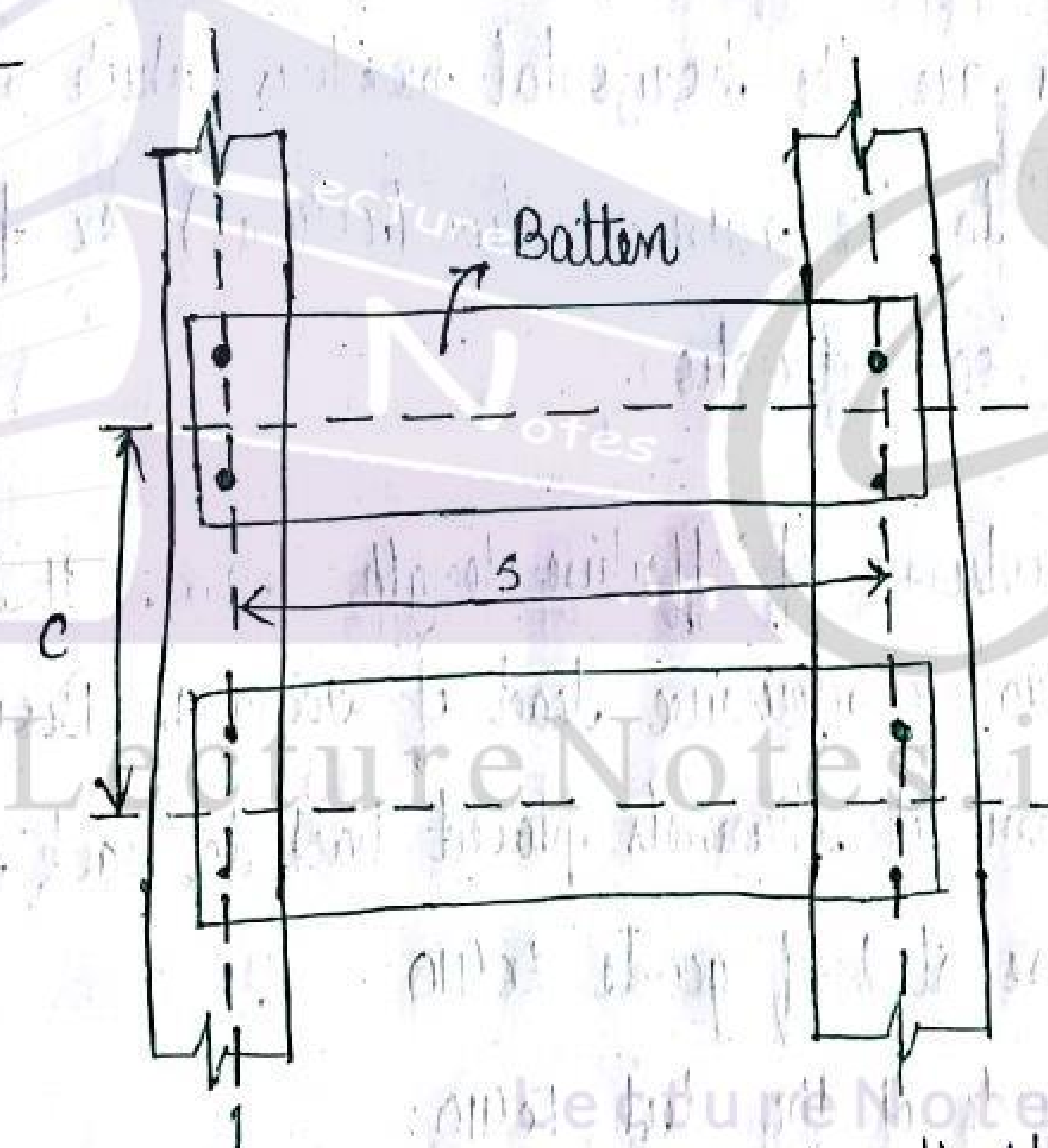
$$T_{dn} = \frac{0.9 A_n f_u}{\gamma_{m1}}$$

$$= \frac{0.9 \times [1200 - (22 \times 20)] \times 410}{1.25}$$

$$= \frac{0.9 \times \cancel{760}^{320} \times 410}{1.25} = \cancel{224.35kN}\ 94.464\text{ kN} > 19.09\text{ kN}$$

Hence, it is safe against the tension.

# Battening :-




When 2 vertical members are connected with plates which are perfectly vertical to the load direction, this member is known as batten member. In the above diagram, 's' is the spacing between 2 plates to be connected, 'c' is the spacing between 2 consecutive batten members.

* Code Provisions :-

a) Batten member – P-50, 7.7.

b) Size of the batten member – P-51, 7.7.2.3

c) Design of batten member – P-51, 7.7.2.1

d) Tie plate – P-51, 7.7.2.2.



* Note :-

→ The lacing members are lateral members with some angle to the main plates (main member) whereas the batten members are the horizontal members which are perfectly vertical to the main member (column) or perfectly vertical to the load direction.

Q) Design a column of effective length 5.9m. It is subjected to factored axial compressive load of 2000 kN. Provide 2 column in the form of channels placed back to back connected with batens. Use steel of grade Fe410.



A) For the grade of the steel Fe410 :-

$f_y = 250\ N/mm^2$; $f_u = 410\ N/mm^2$.



The factored load is to be carried by 2 columns. Hence, load on each column is 1000 kN.

Design of the column section :-

By selecting an section ISLC 400 @ 448.3 to carry the factored load, we have :

$A = 5825\ mm^2$ $\quad C_{yy} = 23.6\ mm$

$h = 400\ mm$ $\quad I_{xx} = 1.39 \times 10^8\ mm^4$

$b_f = 100\ mm$ $\quad I_{yy} = 4.60 \times 10^6\ mm^4$

$t_f = 14.0\ mm$ $\quad r_x = 155\ mm$

$t_w = 80mm$ $\quad r_y = 28.1\ mm.$

The design compressive strength of the member:

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

For the channel section, the buckling class is 'c'.

$$\lambda = \frac{KL}{r} = \frac{1 \times 5.9 \times 10^3}{28.1} = 209.964$$

From the IS code, page - 42, table - 9(c):-

$200 = 36.3$

$209.964 = 33.31$

$210 = 33.3$

$\Rightarrow f_{cd} = 33.31.$

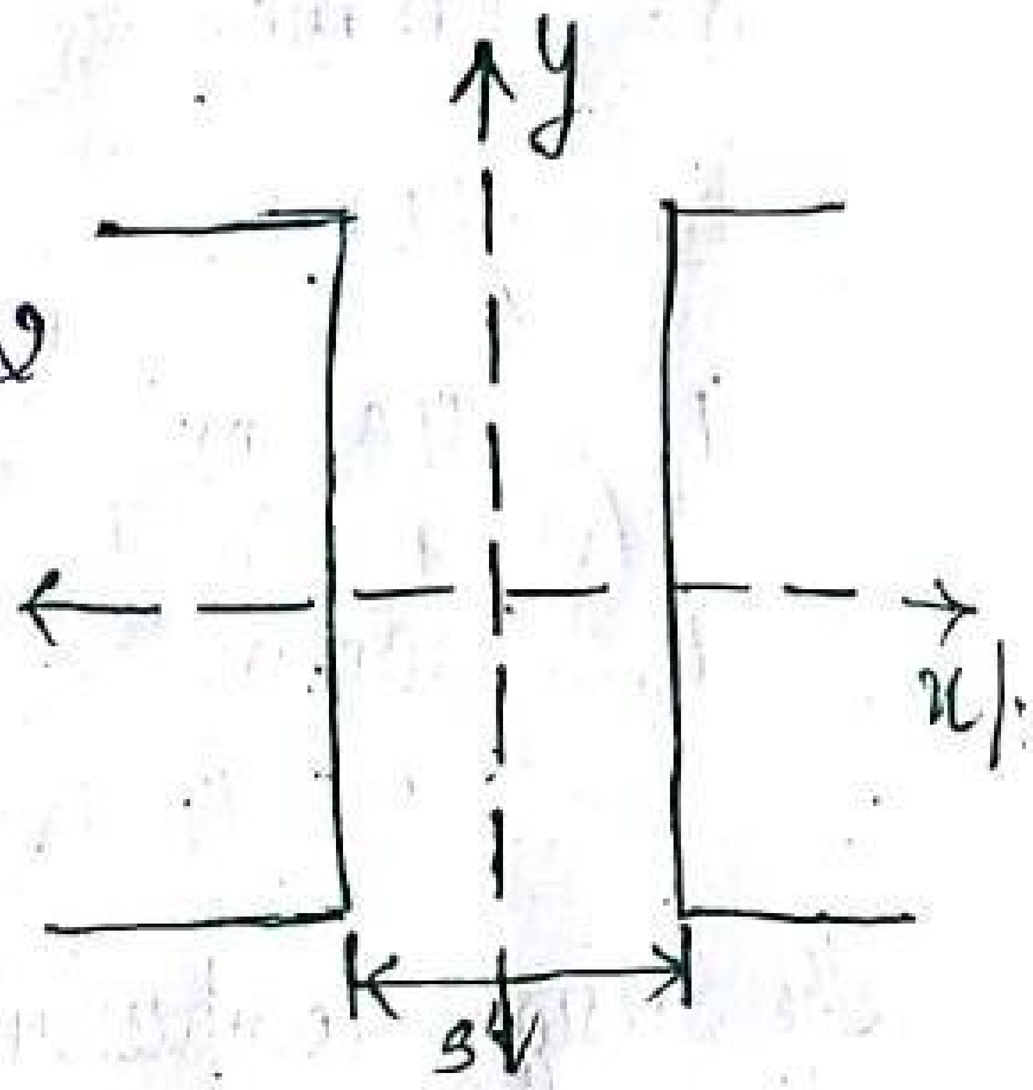
$\Rightarrow P_d = 5825 \times 33.31 = 194.03kN. < 1000\ kN.$

Hence, we have to redign the section by taking a higher sectional area.

Design for the batten member:-

In the figure, 'S' is the spacing between the two main members, so that the total composite section should be safe.

To find the spacing, we have to equate the moment of inertia of the component section through the y-axis with that of x-axis (z-axis).




$$I_{yy} = \left[I_y + A\left(s' + \frac{C_{yy}}{2}\right)^2\right] \times 2$$

$$I_{zz} = 2 I_z$$

By equating, we have:

$$2\left[I_y + A\left(s' + \frac{C_{yy}}{2}\right)^2\right] = 2 I_z$$

For ISLC 400 :-

$$(4.60 \times 10^6) + 5825\left(s' + \frac{23.6}{2}\right)^2 = 1.39 \times 10^8$$

$$\Rightarrow 5825\left(s'^2 + 139.24 + 278.48\, s'\right) = 134.4 \times 10^6$$

$$\Rightarrow s'^2 + 139.24 + 278.48\, s' = 23.072 \times 10^3$$

$$\Rightarrow s'^2 + 278.48\, s' - 22932.76 = 0.$$

$$\Rightarrow s' = 140.097 \simeq 145 \text{ mm}.$$

Effective depth of the batten member (size):-

The distance between the CG of the two members is equal to

$$= s' + 2C_{yy} = 145 + 47.2 = 192.2 \text{ mm}$$

$$\simeq 195 \text{ mm}.$$

The effective depth of the batten member is equal to the c-c distance between the main members = 195 mm. (and battens).

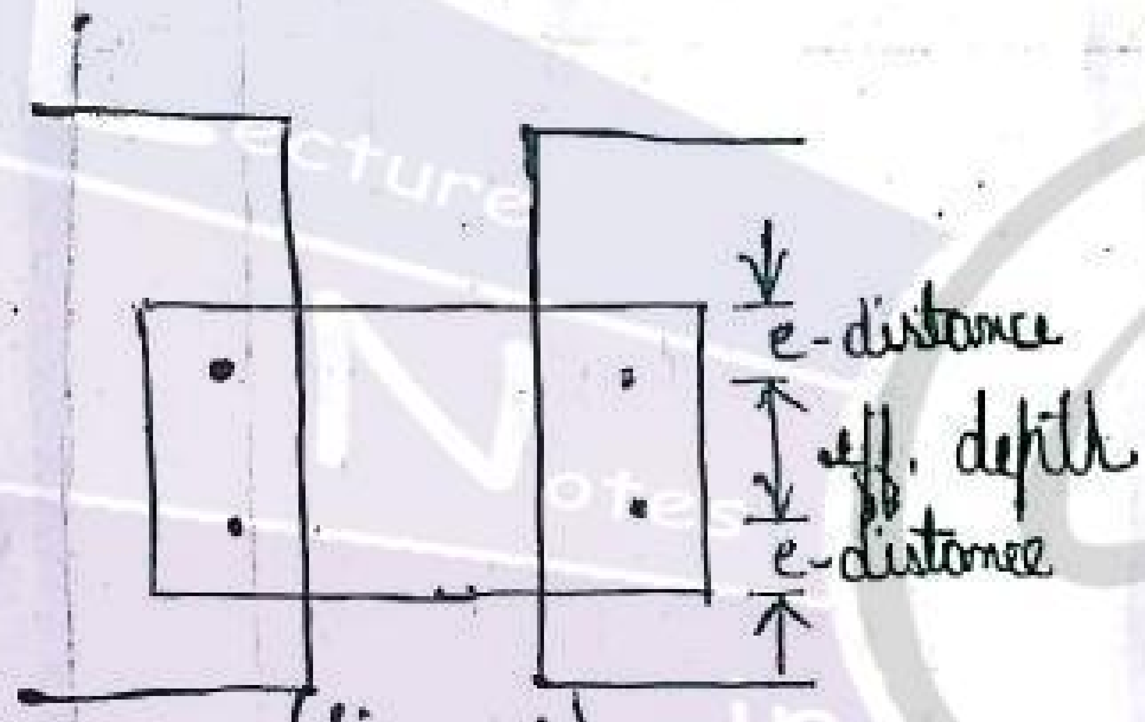
The effective depth of the intermediate batten members

$$= \frac{3}{4} \times \text{effective depth of end battens}$$

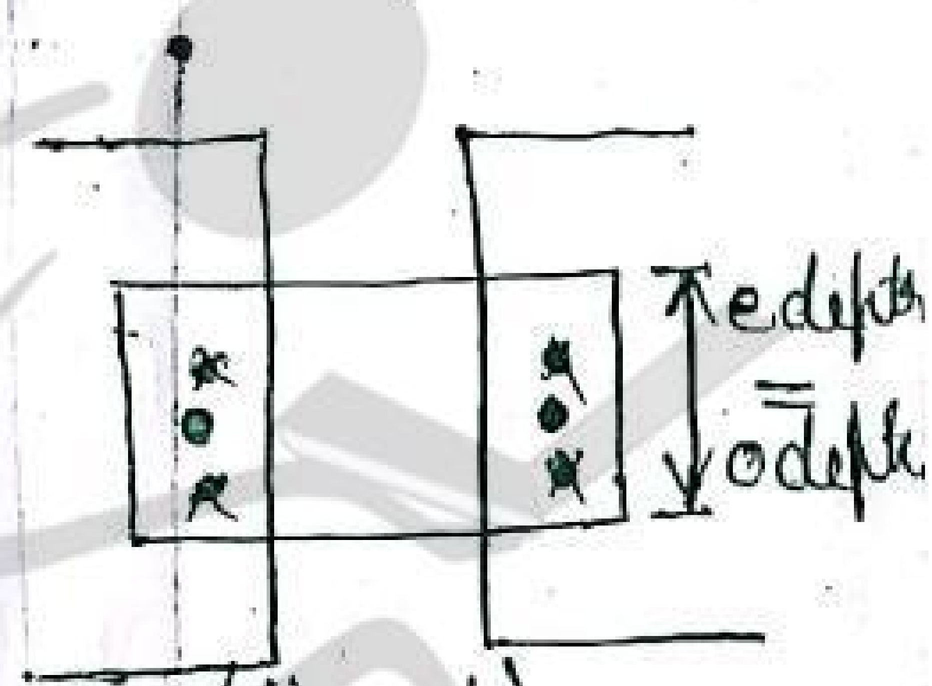
$$= \frac{3}{4} \times 195 = 146.25 \text{ mm} \simeq 150 \text{ mm}.$$

The overall depth of the batten member = effective depth + edge (end) distance.

[*** For 2 or more no. of bolts in connection, the effective depth is the distance between the end bolts. (figure - a).



(fig. - a)



(fig. - b)

For single bolt connection of batten member, the effective depth = overall depth. (figure - b). ***]

Overall depth of the end batten, taking 16 mm dia bolts is equal to $= 195 + (2 \times e)$

$$= 195 + [2 \times (1.5 \times 18)] = 249 \simeq 250 \text{ mm}.$$

Similarly overall depth of the intermediate battens is:

$$= 150 + 2 \times e = 150 + [2 \times (1.5 \times 18)]$$

$$= 204 \simeq 205 \text{ mm}.$$

<u>Thickness of the batten member</u>:-

If the spacing between the bolts is not more than the pitch value, one line of bolt with higher diameter is to be provided in place 2- line of bolts of lower diameter

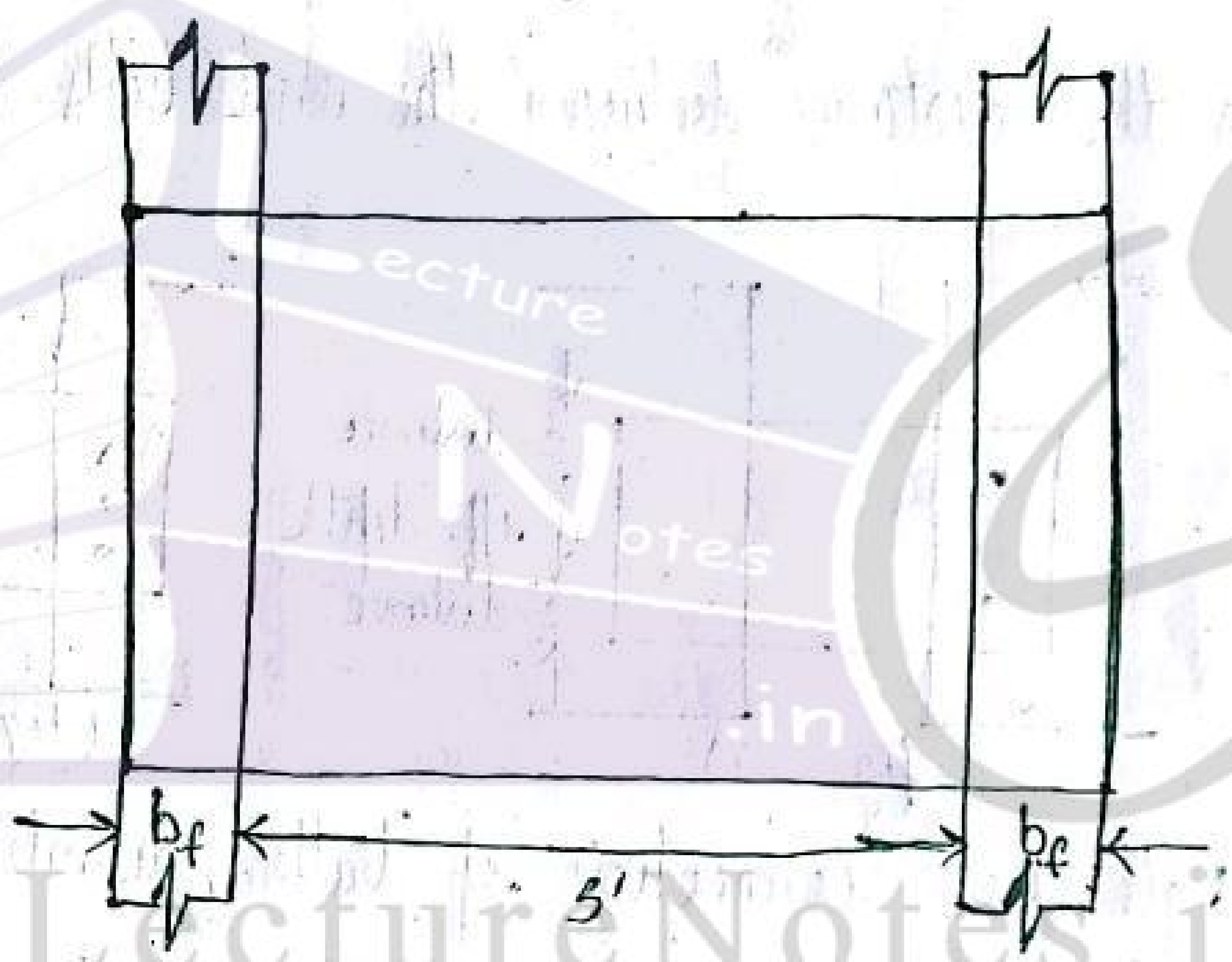
Providing 1-row of bolts, from the steel table, $g = 60\,mm$.

Thickness of the batten member $= \frac{1}{50} \times (S' + 2g)$

$$= \left(\frac{1}{50} \times \left(145 + (2 \times 60)\right)\right)$$

$$= \cancel{122.9\,mm}\ 5.3\,mm \simeq 6\,mm.$$

Length of the batten member :-




$\Rightarrow$ Length $= 2b_f + S' = (2 \times 100) + 145 = 345\,mm.$

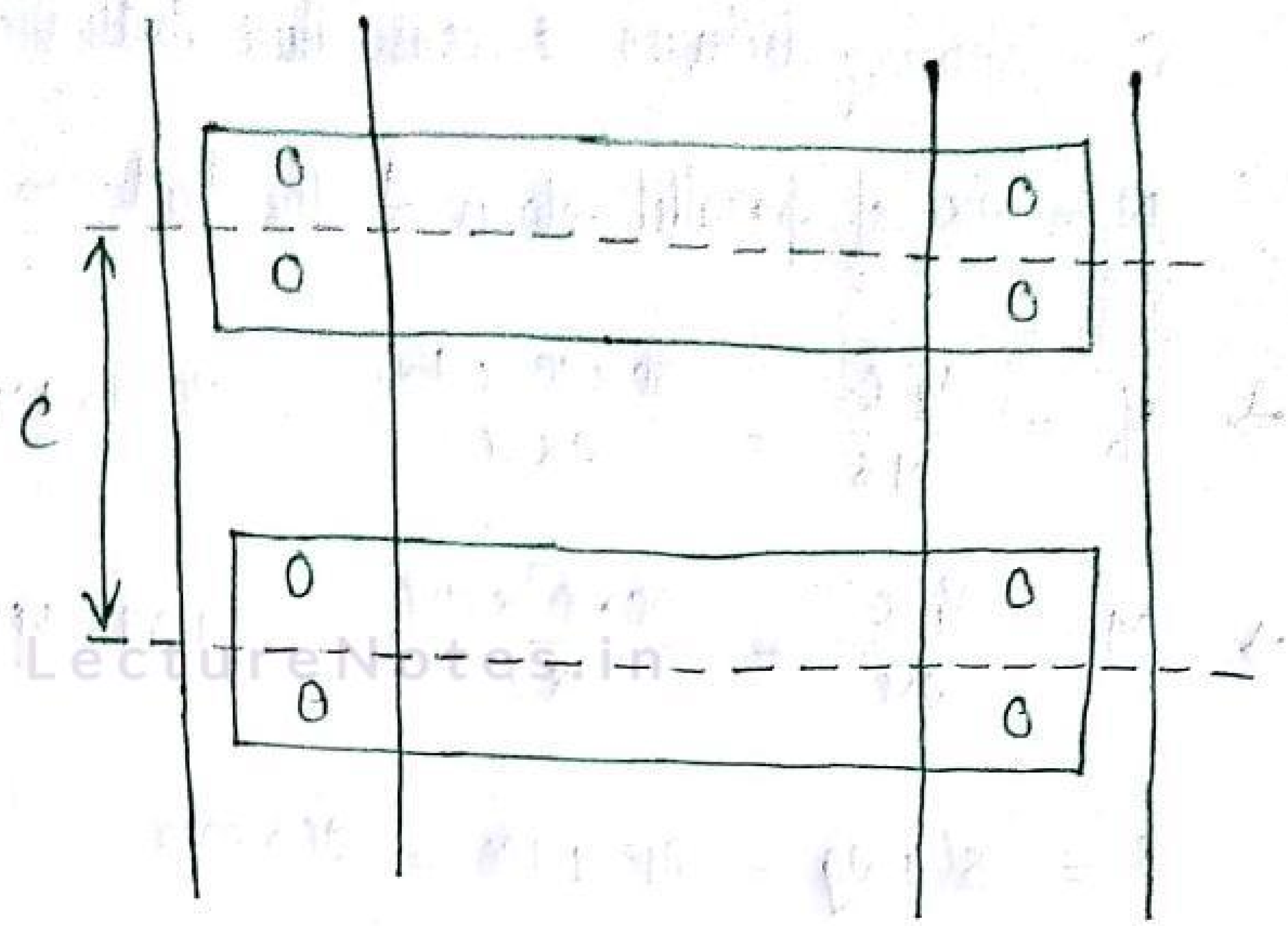
Provide the end batten of size $(345 \times 195 \times 6)\,mm$ & intermediate batten of size $(345 \times 150 \times 6)\,mm$.

~~# Spacing Between~~

Spacing between the two consecutive battens :- (P-51, 7.7.3)

The spacing between the two consecutive battens $= \frac{C}{r_y}$

where, $C =$ ~~c~~ C-C distance between the batten members.



$$\Rightarrow \frac{C}{r_y} = \text{lesser of} \begin{cases} 50 \\ 0.7 \times \frac{KL}{r_z} \end{cases} = \begin{cases} 50 \\ 0.7 \times \frac{1 \times 5.9 \times 10^3}{155} \end{cases}$$

$$\Rightarrow 26.64 < \frac{C}{r_y} < 50.$$

$$\Rightarrow \frac{C}{r_y} = 26.64.$$

$$\Rightarrow C = 26.64 \times r_y = 748.58 \simeq 750 \text{ mm}.$$

Hence, provide the batten members with a minimum spacing of 750 mm.

<u>Check for the strength of the batten</u>:- (P-51, 7.7.2.1)

$$V_b = \frac{V_t \times C}{N \cdot S}$$

$$M = \frac{V_t \cdot C}{2N}$$

$V_t$ = design shear capacity = 2.5% of factored load = (2.5% × 2000) = 50 kN.

$C_1$ = spacing between 2 consecutive batten members.

N = no. of parallel planes of the batten = 2.

$$\Rightarrow V_b = \frac{V_t C}{N \cdot S} = \frac{50 \times 10^3 \times 750}{2 \times 265} = 70.75 \text{ kN.}$$

$$\Rightarrow M = \frac{V_t \cdot C}{2N} = \frac{50 \times 10^3 \times 750}{2 \times 2} = 9.375 \text{ kN-m}$$

$$S = S' + 2g = 145 + 120 = 265 \text{ mm}$$

<u>Shear stress of the batten member</u>:-

$$\text{shear stress} = \frac{\text{shear force}}{\text{area of batten}} = \frac{70.75 \times 10^3}{1170}$$

$$= 60.47 \text{ N/mm}^2.$$

Permissible shear stress (P-59, 84) $= V_d = \frac{A_v f_{yw}}{\sqrt{3}\,\gamma_{m0}}$

$$\Rightarrow \frac{V_d}{A_v} = \frac{f_{yw}}{\sqrt{3}\,\gamma_{m0}} = \frac{250}{\sqrt{3} \times 1.1}$$

$$= 131.21 > 60.47 \text{ (N/mm}^2\text{)}$$

~~Hence, provide~~ As the member is safe against the design shear stress, hence no need to check for the design BM.

Hence, provide the batten members as per the specified size with centre-centre spacing of 750mm with one line of 16mm dia bolts.

# Diamond Bolted Connection

## # Diamond Arrangement of Bolted Connection :-

Q) Find the maximum force which can be transfered through the double cover butt joint as shown in figure. Find the efficiency of the joint. Given, the grade of the bolt is 4.6 of M20 type. The grade of the steel is Fe410.

A) For the grade of the plate,

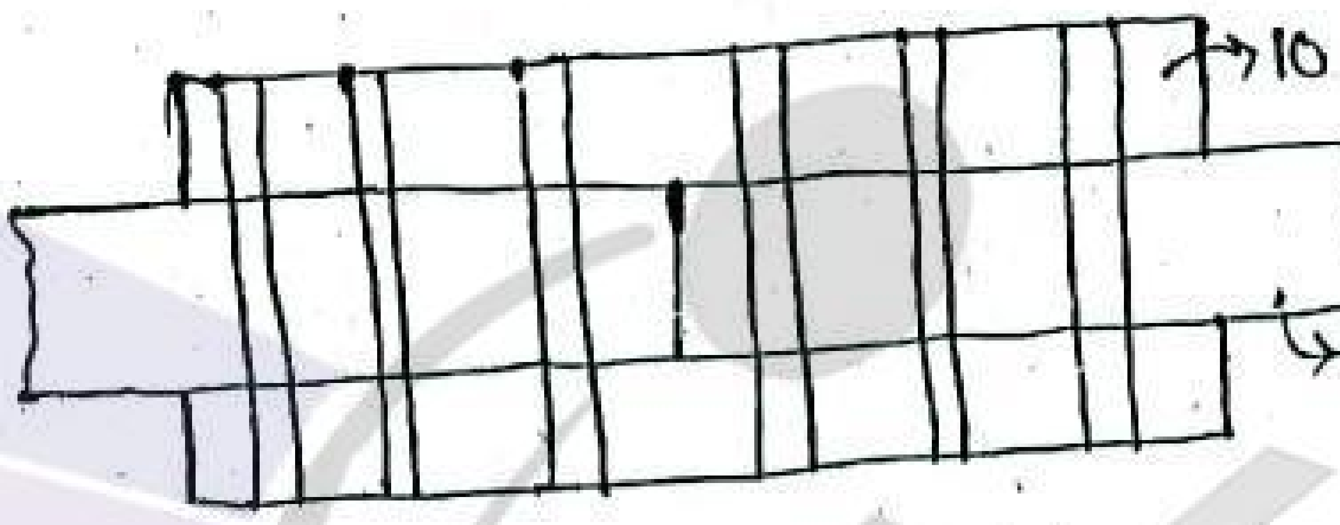
Fe410 :-

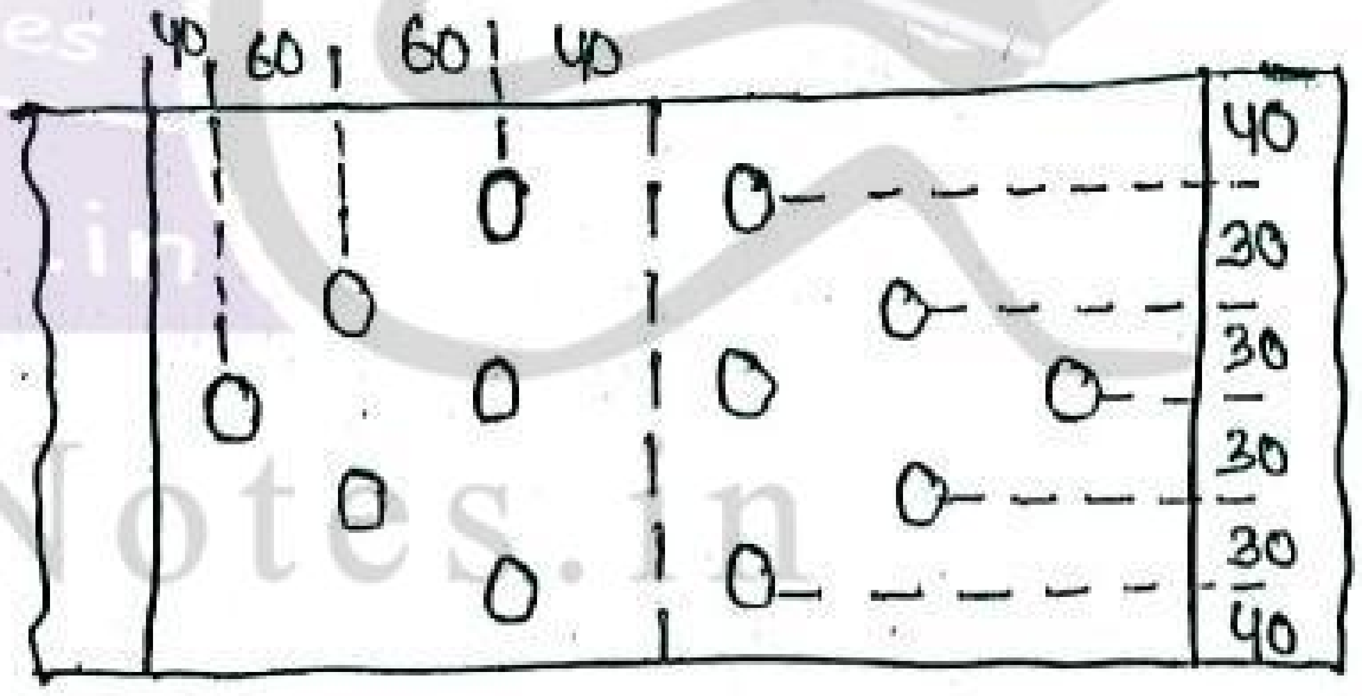
$f_{ub} = 400\ N/mm^2$

$f_u = 410\ N/mm^2$

$f_y = 250\ N/mm^2$

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




ALL DIMENSIONS ARE IN 'MM'.

[*** The maximum force which can be carried by the joint is the lowest between the strength of the plate or strength of the bolt ***]

$P = 60mm,\ e = 40mm,\ g = 30\,mm.$

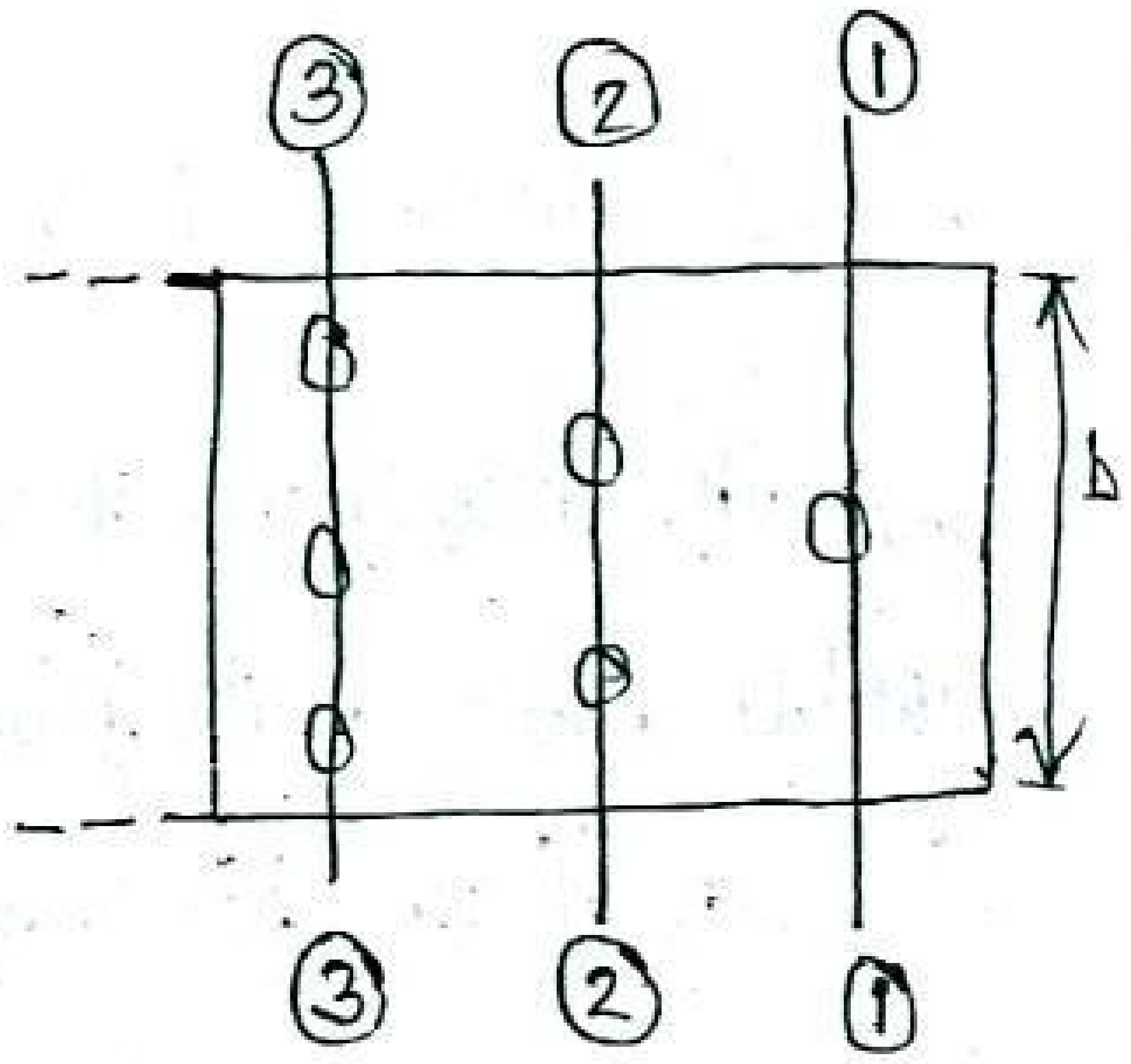
As, the main plate & cover plate are connected together, hence either of these 2 may fail.

$t_m = 16mm, \quad t_c = 10+10 = 20\,mm.$

$\gamma_{ml} = 1.25 = \gamma_{mb}$

$A_{nb} = 0.78 \times \frac{\pi}{4} \times d^2 = 245 \text{ mm}^2$.

Strength of the plate :-



→ For the main plate, section ①-① is the critical section as the applied load is directly in contact with the first bolt.

→ For the cover plate, section - ③-③ is the critical section as the stress concentration is more in case of 3 bolt holes.

Strength of the main plate, $(T_{dn})_m = \frac{0.9 A_n f_u}{\gamma_{ml}}$

$A_n = (b - n d_h) t$

$= (200 - 1 \times 22) 16 = 2848 \text{ mm}^2$.

$\Rightarrow (T_{dn})_m = \frac{0.9 \times 2848 \times 410}{1.25} = 840.72 \text{ kN}$

Strength of the cover plate, $(T_{dn})_c = \frac{0.9 A_n f_u}{\gamma_{ml}}$

$A_n = (200 - 3 \times 22) \times 20 = 2680 \text{ mm}^2$.

$\Rightarrow (T_{dn})_c = \frac{0.9 \times 2680 \times 410}{1.25} = 791.136 \text{ kN}$.

The strength of the plate = lower of $(T_{dn})_c$ and $(T_{dn})_m$, i.e.,

$T_{dn}$ = 790.136 kN.

<u>Strength of the bolt</u>:-

i) Shearing strength of the bolt:-

The strength of the bolt in shearing : $V_{dsb} = \frac{f_{ub}}{\sqrt{3}\,\gamma_{mb}} n_n A_{nb}$

$$= \frac{400}{\sqrt{3} \times 1.25} (2 \times 245) = 90.528 \text{ kN.}$$

strength of 6 bolts = 543.168 kN.

ii) Bearing strength of the bolt:-

The bearing strength of the bolt will be calculated as section wise, i.e., separately for each section 1, 2 & 3.

Section 1-1:-

$$(V_{dpb})_1 = \frac{2.5\, k_b\, d\, t\, f_u}{\gamma_{mb}}$$

$k_b = \frac{e}{3d_0} \Big| \frac{p}{3d_0} - 0.25$

$= 1.52 | 0.65$

$$= \frac{2.5 \times 0.65 \times 20 \times 410 \times 16}{1.25}$$

$$= 170.56 \text{ kN}$$

Section - 2-2 :

$$k_b = \frac{70}{66} = 1.06 \;|\; 0.65.$$

$$(V_{dpb})_2 = 170.56 \text{ kN.}$$

$$= 2 \times 170.56 = 341.12 \text{ kN.}$$

Section - ③ - ③ :-

$$k_b = \frac{40}{66} = 0.60 < 0.65 < 1.025 < 1.0.$$

$$(V_{dpb})_3 = 157.44 \text{ kN} = 3 \times 157.44 = 472.32 \text{ kN.}$$

Bearing strength of 6 bolts $= (2 \times 170.56) + (3 \times 157.44) + 170.5$

$$V_{dpb} = 984 \text{ kN.}$$

$\therefore$ Strength of the bolt $= 543.168$ kN.

Strength of the joint $=$ lesser of $= (T_{dn} \text{ \& } V_{dsb})$

$$= 543.168 \text{ kN.}$$

$$\text{Efficiency of the joint} = \frac{\text{strength of the joint} \times 100}{\text{strength of solid plate in tension}}$$

$$= \frac{543.168 \times 10^3 \times 100}{\frac{0.9 A_n f_u}{\gamma_{m1}}}$$

$$= \frac{543.168 \times 10^3 \times 100}{\frac{0.9 \times 3200 \times 410}{1.25}}$$

$$= 57.5\%.$$

$$\boxed{** \; A_n = b \times t = 200 \times 16 = 3200 \text{ mm}^2 \; **}$$

# Plate Girders

## # Plate Girders :-

→ when the applied load is more, than the design strength of the rolled steel section which are available in the steel table, then we have to choose built up sections. These built up sections is known as plate girders.

→ Such situations are common in the following cases:-

i) Larger column.

ii) In a workshop where the beams are not sufficient to carry the crane load.

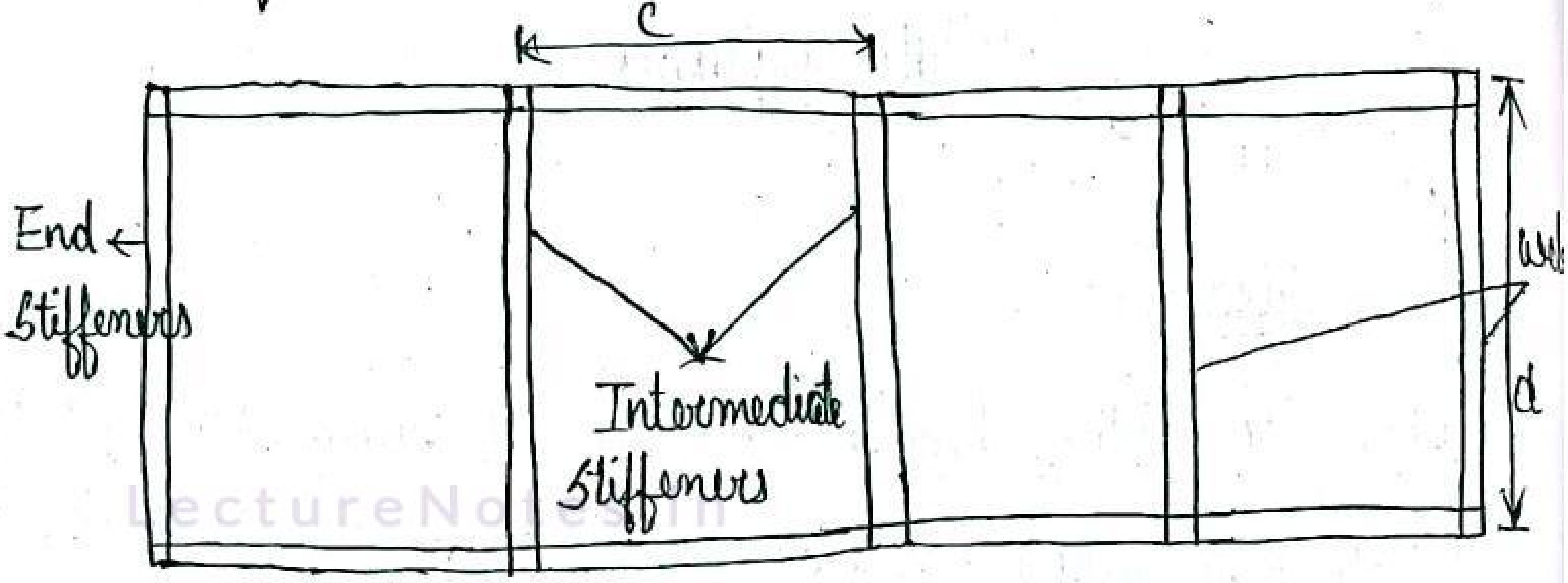
iii) Railway bridges.

→ The built-up I-section is prefered for the plate girder design.

→ For the plate girder design, the web & flange are to be designed separately. The flange & web are to be connected either by welded connection or bolted connection which to be designed also.

→ Most preferably welded connection is used for built up section in economic point of view.

# Elements of Plate Girder :-




Following are the elements of the plate girders :-

1) Web :- Webs of required depth & thickness are provided to:

→ to keep the flanges at required distance.

→ To resist the shear in the beam.

2) Flange :- Flanges of required width & thickness are provided to resist the BM acting on the beam by developing compressive force in one flange & tensile force on the other flange.

3) Stiffeners :- Stiffeners are provided to check the web against local buckling failure. The stiffeners provided may be of the following type :

a) Transverse or Vertical Stiffeners.

→ Bearing Stiffeners

→ Intermediate stiffners.

b) Longitudinal stiffners - These are provided to increase the buckling strength of the web. When the length of the span of the beam is more, then intermediate stiffeners are provided.

# ~~Longitudinal Girder~~ :-

Note:-

* For efficient design, use thick flange and thin webs.
* Note that, the web should not be so thin that serviceability & flange buckling are affected.

# Modes of Failure in the Limit State :-

i) Yielding of tension flange.

ii) Buckling of compression flange (lateral torsional buckling)

iii) Buckling of the web in shear.

iv) Flange yielding & buckling similar to the beam.

# Role And Rigidity of the Stiffners:-

→ Transverse stiffeners play important role:

i) by increasing web buckling stress.

ii) by supporting tension field after web buckling.

iii) by preventing tendency of flanges to get cooled towards each other.

# Code Provision :-

i) Resistance to shear buckling has been verified when:

a) $\frac{d}{t_w} > 67\epsilon$ where, unstiffened webs are used.

b) $\frac{d}{t_w} > 67\epsilon\sqrt{\frac{k_v}{5.35}}$ where, stiffened webs are provided.

$k_v$ = shear buckling coefficien

ii) minimum web thickness for serviciability :- (C - 8.6.1.1)

a) when transverse stiffners are not provided

→ $\frac{d}{t_w} \leq 200\epsilon$ when web connection to flange along both longitudinal edges

→ $\frac{d}{t_w} \leq 90\epsilon$ when web connection to flanges along 1 longitudinal axis.

b) When only transverse stiffeners are provided :-

→ $3d \geq c \geq d$ ; $\frac{d}{t_w} \leq 200\epsilon$

→ $0.74d \leq c \leq d$ ; $\frac{d}{t_w} \leq 200\epsilon$

→ $c < d$ ; $\frac{d}{t_w} \leq 270\epsilon$

→ $c > d$ ; the web is considered as unstiffened.

c) when transverse & longitudinal stiffeners are provided :-

→ $2.4d \geq c \geq d$ ; $\frac{d}{t_w} \leq 250\epsilon$ (one level only)

→ $0.74d \leq c \leq d$ ; $\frac{d}{t_w} \leq 250\epsilon$

→ $c < 0.74d$ ; $\frac{d}{t_w} \leq 340\epsilon$

d) When longitudinal stiffeners are provided along the neutral axis :

$$\frac{d}{t_w} \leq 400\epsilon$$

# Types of Stiffeners :-

→ The bearing stiffeners prevents local crushing of the web, flange in pairs on the web at unframed girder end and at the location of concentrated load provided.

→ The load carrying stiffeners prevents local buckling of the web due to any concentrated load.

→ An intermediate transverse web stiffeners mainly includes the buckling strength of the web and prevents the flanges from moving towards each other.

→ The torsional stiffeners are provided at the supports to restrain the girder against torsional effect.

→ Local strengthening of the web under the combination of shear & bending is provided by diagonal stiffeners.

→ The tensile force from the flange are transmitted to the web through the tension stiffeners.

→ A longitudinal stiffener increases the buckling resistance of the web.

# Code Provision :-

→ Compression flange buckling requirement

# Design Steps :-

i) Taking $\boxed{\frac{L}{d} = 15}$ calculate the minimum depth required and ~~add up~~ adopt a suitable depth.

ii) The required area of the flange :

$$\boxed{A_f = \frac{M \cdot \gamma_{m0}}{f_y d}}$$

M = factored BM.

Use, $b_f = 0.3d$ to calculate the thickness of the flange plate. Also, calculate the axial force in the plane.

iii) Check whether the flange is plastic.

iv) Based on the web thickness for serviceability, choose appropri web thickness.

v) Check for flange buckling into web [P-64, 8.6.1.2]

vi) Design of the girder :-

a) Check for the moment capacity [8.2.1 / 8.2.2 – P-52, 54]

b) Check for plastic shear resistance of the web [P-59, 8.4.1]

c) Check for resistance to shear buckling [P-59, 8.4.2.2]

d) If $\boxed{V_e < \frac{V_{cr}}{\gamma_{m0}}}$ then the section is safe, else check by the tension field method. [P-59, 8.4.2(b)]

$V$ - applied shear

$V_{cr}$ - shear strength of the member

e) Design the connection between flange & web plate by welding or bolting.

f) Design the end canal or end post :-

→ Not utilising tension field action - design the end post as load bearing stiffeners.

→ Utilising tension field action - in addition to carry the reaction, the end post should be designed as a beam spanning between the flanges.

g) Shear capacity of the end canal [P-62, 8.5.3]

h) Design for stiffeners : [P-65, 66 ; 8.7.2, 8.7.2.2]

→ Decide whether to provide the stiffener both the side of the web or one side of the web.

→ Check for the minimum stiffness [P-66, 8.7.2.4]

→ Check for the buckling of the stiffeners [P-66, 67 ; 8.7.1.5, 8.7.2.5]

i) Connection to the web [P-67, 8.7.2.6]

j) Check for intermediate stiffeners under the load :-

$$\boxed{\frac{F_q - F_x}{F_{qd}} + \frac{F_x}{F_{xd}} + \frac{M_q}{M_{yq}} \leq 1} \quad [P-67, 8.7.2.5]$$

k) Design for bearing stiffeners by considering local buckling of the web :-

$$\boxed{F_w = \frac{(b_1 + n_2)\, t_w\, f_{yw}}{\gamma_{m0}} > P}$$

LectureNotes.in



END OF MODULE - 4

UNIT — I - CONNECTIONS — RIVETED, WELDED & BOLTED

1. Mention the advantages and disadvantages of steel structures?

Advantages: Ability to resist high loads, due to its high density, steel is completely non-porous Durability Easy to disassembling or replacing some steel members of a structure

Disadvantages: Corrosion At high temperature steel loses most of its strength, leading to deformation or failure

2. What is meant by Girder?

Girder means a major beam frequently at wide spacing that supports small beams.

3. What is meant by joists?

It is a closely spaced beam supporting the floors and roofs of buildings

4. What is meant by Purlins?

It is a. roof beam usually supported by truss

5. What is meant by Rafters?

It is a roof beam usually supported by purlin.

6. What is Girts?

It is horizontal wall beams used to support wall covering on the side of an industrial building 8. 7. What is meant by Spandrel beam?

It is beam around the outside perimeter of a. floor that support the exterior walls and the outside edge of the floor

8. Name the different types of connections?

Riveted connections Welded connections Bolted connections Pinned connection

9. Name the types of riveted connections?

Lap Joint - single riveted and double riveted Butt joint — single cover and double cover

10. What is meant by rivet value?

The least of the strengths in shearing and bearing is the rivet value

11. Name the different modes of failure of a riveted joint?

Tearing failure of the plate, Shear failure of the plate, Shear failure of the rivet, Bearing failure of the rivet, Splitting failure of plate

12. As per the American practice where the neutral axis lie in the rivet group?

It is assumed that the line of rotation lies at a distance of 1/7 th of the effective bracket depth from the bottom of the bracket

13. What are the load combinations for the design purposes?

Dead load + Imposed Load (Live load)

Dead Load + Imposed Load + Wind Load or earthquake load

Dead Load + Wind Load or Earthquake load

UNIT — II - TENSION MEMBERS

1. Tie member — Explain.

Tie member or a tension member is a structural element carrying an axial tensile force. For the tensile force to be axial it is necessary that the load be applied through centroid of the section of the member. But under axial tension the member gets straightened and eccentricity of the force decreases. The member is a hoist straight at the yield point and the distribution of the stress over the section becomes uniform.

2. How the tension members are classified?

It is classified according to its shape and size and it depends upon the type of structures. Wires and cables — Used in hoists, derricks, suspenders in suspension bridges Rods and bars — Used in radio tower, small spanned roof trusses with different cross-sections such as round, rectangular or square

3. What is meant by single section member?

structural sections such as I-section, T-section, angle, and channel are used as tension members. As the structural shapes provide more rigidity than cables or rods, their buckling tendency under compression load is reduced and so can be used where reversal of stress takes place

4. Under what circumstances you would go for Built-up members?

When single structural sections fail to provide required strength and stiffness to carry tension as well as compression in case of reversal of stresses, built-up members are used.

5. How the tension members are selected?

It depends upon the various factors such as type of fabrication, type of structure, type of loading, i.e. whether the member undergoes reversal of stresses, and the maximum tension to be carried by the member.

6. What is net sectional area of a tension member? How it is calculated in chain riveting?

The gross sectional area of the tension member minus the sectional area of the maximum number of rivet/bolt holes is known as net sectional area. In case of chain riveting.

$a_{net} = (b - nd)\, t$

7. What is Lug angle?

A larger length of the tension member and the gusset plate may be required sometimes to accommodate the required number of connection rivets. But this may not be feasible and economical. To overcome this difficulty lug angles are used in conjunction with main tension members at the ends. It provides extra gauge lines for accommodating the rivets and thus enables to reduce the length of the connection. They are generally used when the members are of single angle, double angle or channel sections.

8. What are the main objectives of the lug angles?

They produce eccentric connections, due to rivets placed along lug angle. The centroid of the rivet system of the connection shifts, causing eccentric connection and bending moments. Stress distribution in the rivets connecting lug angles is not uniform. It is preferred to put a lug angle at the beginning of the connection where they are more effective and not at the middle or at the end of the connection. Rivets on the lug angles are not as efficient as those on the main member. The out-standing leg of the lug angle usually gets deformed and so the load shared by the rivets on the lug angles is proportionately less.

9. What is the permissible stress in axial tension? ISAI

As per IS: 800 — 1984, the permissible stress in axial tension = $0.6\, f_y$ N/mm²

$f_y$ = minimum yield stress in steel in N/mm²

10. How will you join the member of different thickness in a tension member?

When tension member of different thickness are to be jointed, filler plates may be used to bring the member in level.

11. What happens when a single angle with one leg is connected to a gusset plate, which is subjected to an eccentric load?

The rivets connecting the angle to the gusset plate does not lie on the line of action of load. This gives rise to an eccentric connection due to which the stress distribution becomes non-uniform. The net cross-sectional area of such a section is reduced to account for this non-uniform stress distribution resulting from eccentricity.

COMPRESSION MEMBERS

1. What do you mean by compression members?

Compression members are the most common structural elements and it is termed as columns, struts, posts or stanchions. They are designed to resist axial compression.

2. Name the modes of failures in a column.

Failure of the cross-section due to crushing or yielding. Failure by buckling, due to elastic instability. Mixed mode of failure due to crushing and buckling.

3. Define slenderness ratio

It is defined as the ratio of effective length l of the column to the least radius of gyration r of the column section.

4. Distinguish column and strut.

Columns are the vertical members which carry the loads to the beams, slabs etc. generally they are used in ordinary buildings.

Struts are commonly used for compression members in a roof truss; it may either be in vertical position or in an inclined position

5. What is meant by stanchions?

These are the steel columns made of steel sections, commonly used in buildings.

6. What is Post?

It is loosely used for a column, but in truss bridge girders, end compression members are called end posts.

7. State the assumptions that made in Euler's theory.

The axis of the column is perfectly straight when unloaded. The line of thrust coincides exactly with the unstrained axis of the strut. The flexural rigidity EI is uniform The material is isotropic 8. Why the lateral systems are provided in compound columns?

If the plates are not connected throughout their length of the Built up sections, lateral systems may be provided, which act as a composite section. In such cases the load carrying elements of the built-up compression member in the relative position, without sharing any axial load. However when the column deflects, the lateral system carries the transverse shear force.

9. Name the lateral systems that are used in compound columns and which is the mostly used one?

Lacing or latticing. Battening or batten plates, perforated cover plates.

Lacing or latticing is the most common used lateral system and the sections are flats, angles and channels.

10. What will be the thickness for the single and double lacing bars?

The thickness of flat lacing bars shall not be less than one-fortieth of the length between the inner end rivets or welds for single lacing, and one-sixtieth of the length for double lacing.

11. What is the purpose of providing battens in compound steel columns?

Batten plates consist of flats or plates, connecting the components of the built-up columns in two parallel planes. These are used only for axial loading. Battening of the composite column should not do if it is subjected to eccentric loading or a applied moment in the plane of battens.

12. What is the thickness of a batten plate?

The thickness of batten plate shall not be less than one fiftieth of the distance between the inner most connecting lines of rivets or welds. This requirement eliminates lateral buckling of the batten.

13. Where the perforated cover plates are used and mention its advantages?

They are mostly used in the box sections, which consist of four angle sections so that the interior of column remains accessible for painting and inspection.

Advantages: They add to the sectional area of column and the portions beyond the perforation share axial load to the extent of their effective area. There is economy and fabrication and maintenance Perforations conveniently allow the riveting and painting work on the inside portion.

14. Name the types of column base?

Slab Base, which is a pinned base. Gusseted base, which is a rigid base.

15. State the purpose of column base?

The base of the column is designed in such a way to distribute the concentrated column load over a definite area and to ensure connection of the lower column end to the foundation. It should be in adequate strength, stiffness and area to spread the load upon the concrete or other foundations without exceeding the allowable stress.

16. Give the difference between slab base and gusseted base for steel columns.

Slab base is a thick steel base plate placed over the concrete base and connected to it through anchor bolts. The steel base plate may either be shop-welded to the stanchion, or else can be connected at the site to the column through cleat angles.

17. What is slab base and for what purpose is it provided?

The base plate connected to the bottom of the column to transfer over wider area is known as slab base. Column end is machined to transfer the load by direct bearing. No gusset materials are required.

18 When the slenderness ratio of compression member increases, the permissible stress decreases. Why?

The section must be so proportioned that it has largest possible moment of inertia for the same cross-sectional area. Also the section has approximately the same radius of gyration about both the principal axes.

UNIT —IV BEAMS

1. What is a beam?

A beam is a structural member, which carries a. load normal to the axis. The load produces bending moment and shear force in the beam.

2. What is meant by castellated beam?

A rolled beam with increased depth is to be castellated. To obtain such sections, a zigzag line is cut along the beam by an automatic flame-cutting machine. The two halves thus produced are rearranged so that the teeth match up and the teeth are then welded together.

3. How the beams are failed?

Bending failure Shear failure Deflection failure

The designs are based on these three failures which are to be determined.

4. What do you mean by bending failure?

Bending failure may be due to crushing of compression flange or fracture of the tension flange of the beam. Instead of failure due to crushing, the compression flange may fail by a column like action with side ways or lateral buckling. Collapse would follow the lateral buckling.

5. What is the maximum deflection that to be allowed in steel beams?

The deflection of a member shall not be such as to impair the strength or efficiency of the structure and lead to finishing. The deflection is generally should not exceed 1/325 of the span. 6. What is web crippling?

Web crippling is the localized failure of a beam web due to introduction of an excessive load over a small length of the beam. It occurs at point of application of concentrated load and at point of support of a. beam. A load over a. short length of beam can cause failure due to crushing and due to compressive stress in the web of the beam below the load or above the reaction. This phenomenon is also known as web crippling or web crushing.



7. What are laterally supported beams?

The beams which are provided with the lateral supports either by embedding the compression flange in the concrete slab or by providing effective intermediate (support) restraints at a. number of points to restrain the lateral buckling is called laterally supported

8. Mention the advantages of using rolled steel wide flange section as beams.

More section modulus, Lesser area, Economical

9. Why does buckling of web occur in beams?

Diagonal compression due to shear Longitudinal compression due to bending Vertical compression due to concentrated loads

10. Under what situations the plated beams are used?

When a bending moment is large which cannot be resisted by the largest available rolled beam section. The depth of the beam is restricted due to headroom requirements.

11. What do you mean by curtailment of flanges?

The section of a plate girder is to be designed first at mid span. The bending moment will go on decreasing towards the supports. Hence the flange plates, provided at the maximum section can be curtailed.

12. What is the purpose of providing the bearing stiffener?

It prevents the web from crushing and buckling sideways under the action of concentrated loads

It relieves the rivets Connecting the Flange angles and web, from vertical shear.

Write note on tension member splice.

A tension member is spliced when the available length is less than the required length of the tension member. A tension member is also spliced when the members of different thickness are required to be connected. In such a case packing is required to fill up the gap.

What do you understand by Gross area?

Total area of cross section which can be taken as equal weight of the member per unit length divided by density of the material is called Gross area. The sectional area given by the manufacturer is taken as the gross area.

Explain shear lag effect.

The tensile force is transferred from gusset to the tension member (such as angles, channels or T- sections) through one leg by bolts or welds. In this process initially the connected leg may be subjected to more stress than the outstanding leg and finally the stress distribution becomes uniform over the section away from the connection. Thus one part lags behind the other; this is referred to as shear lag.

What is the purpose for providing anchors bolt in base plate?

Anchor bolts are provided to stabilize the column during erection and to prevent uplift for cases involving large moments. Anchor bolts can be cast-in place bolts or drilled-in bolts. The latter are placed after the concrete in set and are not too often used. Their design is governed by the manufacturer's specifications. Cast-in-place bolts are hooked bars, bolts, or threaded rods with nuts placed before the concrete is set.

What are the types of bases provided fro connecting the column to the base?

- Slab base
- Gusseted base
- Moment resisting base

Under what circumstances gusset base is used?

When the load on the column is large or when the column is subjected to moment along with axial load, gusseted base is provided. It consists of a base plate, gusset plate, connecting angles provided on either side of the column and web cleat angle.

Write about batten plates in compression member.

When compression members are required for large structures like bridges, it will be necessary to use built-up sections. They are particularly useful when loads are heavy and members are long (e.g. top chords of Bridge Trusses). The cross section consists of two channel sections connected on their open sides with some type of lacing or latticing (dotted lines) to hold the parts together and ensure that they act together as one unit. The ends of these members are connected with "batten plates" which tie the ends together.

What are the three classifications for determination of size of plate?

Class I- will pertain to all base plates the moment on which is so small in proportion to the direct load that there is compression over the entire area between the bottom of the base and its foundation.

Class II- will pertain a comparatively small range of base plates which have tension over a small portion - one - third or less of the area.

Class III- will include those which are exposed to a comparatively large moment and which therefore have tension over a large portion - more than one -third of the area between the bottom of the base plate and its concrete footing.

What are the functions of providing column bases?

The basic function of bases is to distribute the concentrated load from the column over a larger area. The column load is distributed over the base plate and then to supporting concrete and finally to the soil.

What is meant by slab base?

The slab base as shown in Figure consists of cleat angles and base plate. The column end is faced for bearing over the whole area. The gussets (gusset plates and gusset angles) are not provided with the column with the slab bases. The sufficient fastenings are used to retain the parts securely in plate and to resist all oments and forces, other than the direct compression. The forces and moments arising during transit, loading and erection are also considered.

hat is meant by column splice?

joint in the length of a column provided, when necessary, is known as column splice. It is also described s column joint

Write Short notes on compact sections

When the lateral support to the compression flange is adequate, the lateral buckling of the beam is prevented and the section flexural strength of the beam can be developed. The strength of I-sections depends upon the width to thickness ratio of the compression flange. When the width to thickness ratio is sufficiently small, the beam can be fully plastified and reach the plastic moment, such section are classified as compact sections.

What is meant by slenderness sections?

When the width to thickness ratio of the compression flange is sufficiently large, local buckling of compression flange may occur even before extreme fibre yields. Such sections are referred to as slender sections.

List the various factors affecting the lateral-torsional buckling strength

- Distance between lateral supports to the compression flange.
- Restraints at the ends and at intermediate support locations (boundary Conditions).
- Type and position of the loads.
- Moment gradient along the length.
- Type of cross-section.

How do you improve the shear resistance in plate girder?

i. Increasing in buckling resistance due to reduced c/d ratio,

ii. The web develops tension field action and thus resists considerably larger Stress than the elastic critical strength of web in shear

What are the classifications in Stiffeners?

a) Intermediate transverse web stiffeners

b) Load carrying stiffeners

c) Bearing stiffeners

d) Torsion stiffeners

e) Diagonal stiffeners and

f) Tension stiffeners

Under what circumstances web plates are stiffened and unstiffened?

A web plate is kept unstiffened when the ratio of clear depth to thickness of web is less than 85. It does not require stiffeners. A web plate is called stiffened, when the ratio of clear depth to thickness of web is greater than 85 and stiffeners are provided to contribute additional strength to web.

Define shape factor

The ratio of plastic moment to elastic moment Mp / My is the property of cross sectional area and is not dependent on material properties. This ratio is called as shape factor.

What is meant by plastic hinge?

Plastic hinge is the yield section of the beam, which acts as if it were hinged, except with a constant restraining plastic moment.

What is meant by lateral buckling of beam?

A long beam with laterally unrestrained compression flange when incrementally loaded, first deflects downwards and when load exceeds a particular value, it tilts sideways due to instability of compression flange and rotates about longitudinal axis. This phenomenon is known as laterally buckling or torsional buckling of beam.

How the laterally supported beam fails?

The laterally supported beam can fail by,

- Flexure
- Shear
- Bearing

What is web buckling and web crippling?

A heavy concentrated load produces a region of high compressive stresses in the web either at support or under the load. This causes the web either to buckle or to cripple.

Web buckling occurs when the intensity of compressive stress near the centre of the section exceeds the critical buckling stress of web acting as a strut. This type of failure is more in the case of built up sections having greater ratio of depth to thickness of the web.

What is the purpose of providing stiffener in plate girder?

In the plate girder the depth of the web is kept large for economy and hence it is made thin to reduce the self weight of the girder. A very thin web may buckle laterally or may cripple under the heavy concentrated load. In such a case the web is strengthened by providing stiffeners.

Under what circumstances load bearing stifferener are used in plate girder?

The load carrying stiffeners are attached with the web plate of the plate girder to avoid local bending failure of flanges, crushing of web and buckling of web plate. They are provided under the heavy concentrated loads and the reactions at supports.

What are the types of splices?

- Flange splice
- Web splice