Aug, TURDAY.

STEEL STRUCTURES (5±1 marks)

1. MATERIALS & SPECIFICATIONS OF STEEL

→ Advantages of Structural Steel vs. Concrete

(i) Higher strength

For M20 concrete, $f_{ck} = 20$ MPa.

For mild steel, $f_y = 250$ MPa. $\Rightarrow \frac{f_y}{f_{ck}} \approx 10$ to 12

Compressive & tensile strength of steel are equally good.

But for concrete, tensile strength is approx. $\frac{1}{10}$th compressive stren.

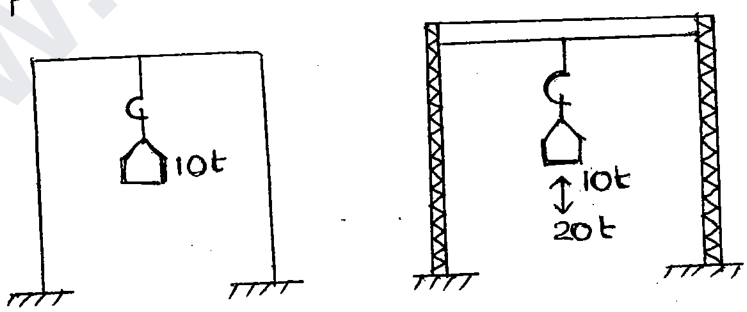
(ii) More economical.

Higher strength to weight ratio for steel compared to concrete. Tall buildings, large span buildings, bridges etc are therefore constructed with structural steel.

(iii) Rapid Construction.

Erection of structure can be completed quickly.

(iv) Easy repair or modification



(v) Ductile material.

$$\% \text{ elongation} = \frac{\delta l}{l} \times 100$$

% elongation > 15% → Ductile material.

5% – 15% → Intermediate material.

< 5% → Brittle material.

For steel, % elongation > 20%; shows signs of failure.

WWW.GATENOTES.IN

(vi) 100% scrap value (reusage value).

Existing steel members can be dismantled and reused for another application with 100% strength.

(vii) Overall construction cost is lesser.

Cost of material, cost of man power, cost of maintenance, dismantling cost etc are cheaper.

(viii) High quality & reliability.

	Safety norms
Working stress method	FOS (= 3 for concrete = 1.7–1.8 for steel)
Limit state Design	Partial safety factor (= 1.5 for concrete = 1.15 for steel)

Lesser values of FOS, P.S.F for steel are because its a factory product; quality control is proper and uncertainities are less.

* Corrosion and proneness to catch fire are the two disadvantages of steel compared to concrete. To make them corrosive and fire resistant, its an expensive process.

Iron (>98%) + Small % of Carbon (0.1 – 1.1% By weight) + [Sulphur, Phosphorous, Silicon, Manganese, Nickel, Chromium] → **STEEL**

(v)

* Nickel & Chromium content is more in Stainless steel giving it anti-corrosive properties

→ Mechanical Properties Imparted by Carbon

(i) Yield strength
(ii) Ultimate tensile strength
(iii) Hardness.
(iv) Ductility.
(v) Toughness.

Carbon content : 0.23 % ⟶ $f_y = 250$ MPa.
0.27 % ⟶ $f_y = 350$ MPa.

→ Classification of Steel (based on Carbon Content)

(i) Low Carbon steel. — 0.1 to 0.25%
(ii) Medium Carbon steel. — 0.25 to 0.6%
(iii) High Carbon steel. — 0.6 to 1.1%

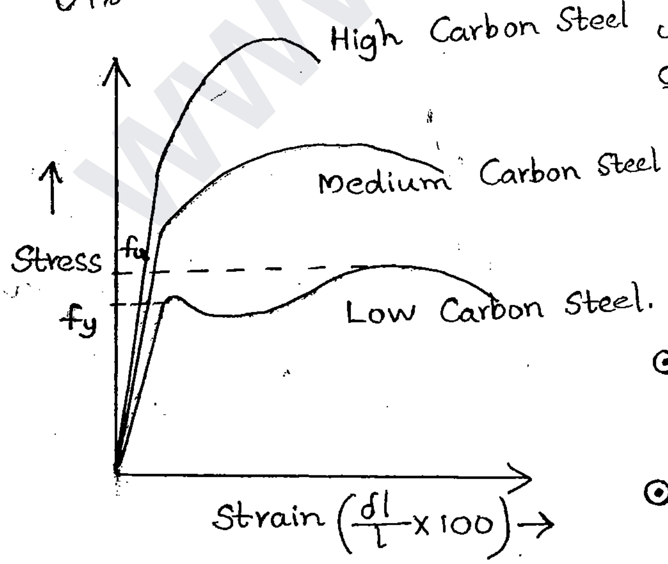
Mild steel used in RCC constructions as reinforcements (Fe 250, 0.25%)
Structural steel section used in steel building construction } Low Carbon

Rails, tyres, high tensile steels (Fe 415, 0.27%), hammers etc. → Medium Carbon

Stone masonry tools, drills, punches etc. → High Carbon

As carbon content ↑, yield strength (f_y), ultimate tensile strength (f_u), Hardness of steel } ↑

Ductility, Toughness } ↓



⊙ Hardness must be lesser to improve weldability.

⊙ Ductility & toughness are very important for structural steel subj to dynamic & impact loads.

→ General Classification:

(i) Reinforcing Steels. :– used as reinforcement in RCC constructions.

Fe 250, Fe 415, Fe 500 etc.

Steel. ↗ ↖ Yield strength (MPa)

(ii) Structural Steels. :– used in steel building and bridge constructions.

Fe 410, Fe 540, Fe 570 etc.

Steel. ↗ ↖ Ultimate tensile strength (MPa).

For mild steel, $f_y = 250$ MPa.

$f_u = 415$ MPa.

→ Codes & Standards.

Failures of old constructions.
Performance of existing buildings.
Researches in various publications
} basis for CODE formulation.

- IS 800 —
  - 1956 – 1962 (1^{st})
  - 1984 (2^{nd}) WSM
  - 2007 (3^{rd}) LSD

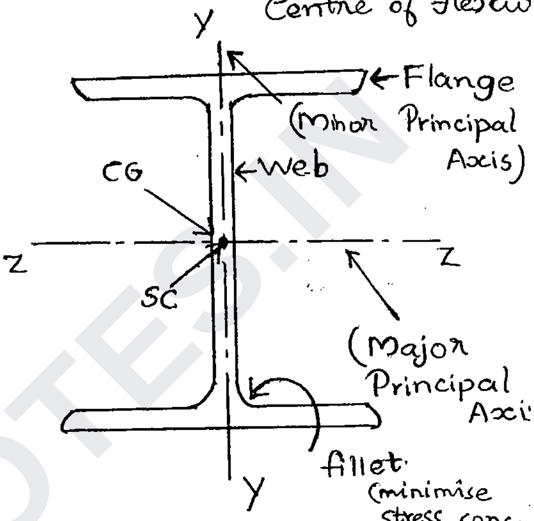
  } code of practise for use of structural steel in general building construct

- IS 875 : 1987 (Five parts) : code of practise for design loads for buildings & structur
- IS Hand Book No. 1 (Steel tables) :
  - a) weight / running meter.
  - b) c/s area.
  - c) radius of gyration.
  - d) Section modulus.

→ Rolled Steel Sections

(i) Rolled steel I-sections or I beams.
(ii) Rolled steel channel sections.
(iii) Rolled steel Tee sections or T-bars.
(iv) Rolled steel angle sections.
(v) Indian Standard tube sections.
(vi) Rolled steel bars.
(vii) Rolled steel plates.
(viii) Rolled steel sheets.
(ix) Rolled steel flats.
(x) Rolled steel strips etc

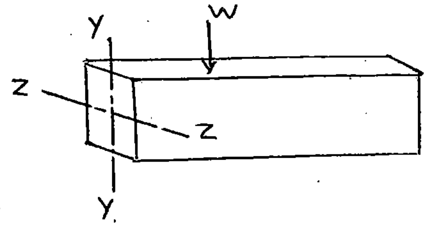
CG: Centre of Gravity.
SC: Shear centre (Centre of Flexure)



I-section or I-beams

* Z-Z axis → Major Principal Axis.
Moment of inertia wrt Z-Z axis is more. (bending axis).

$$I_{zz} = y^2\, dA$$



* When only tranverse loads acts on a rectangular beam, bending will be about Z-Z axis. ∴ I_{zz} should be more. For this, depth of beam is increased.

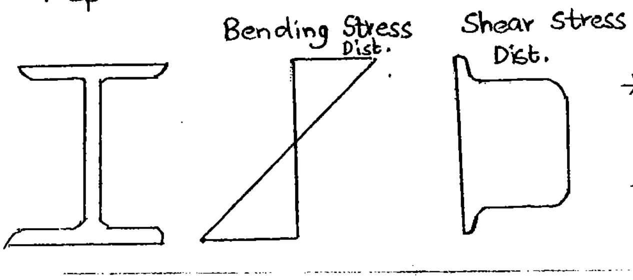
If lateral loads also act on the beam, there will be bending about y-y axis as well. In that case, width of beam is also increased.

* An I-section may be designated as:

ISLB 500 @ 735.6 N/m (≈ 75 kg/m)

Depth of I-section = 500 mm

Weight per running metre =



* 90–92% Bending stress will be taken by Flanges
* IIIy, shear by web.

→ Five Types of I-sections:

(i) ISJB — Indian Standard Junior Beams
(ii) ISLB — Indian Standard Light Weight Beams
(iii) ISMB — Indian Standard Medium Weight Beams
(iv) ISWB — Indian Standard Wide Flange Beams
(v) ISHB — Indian Standard Heavy ~~beams~~ Beams.

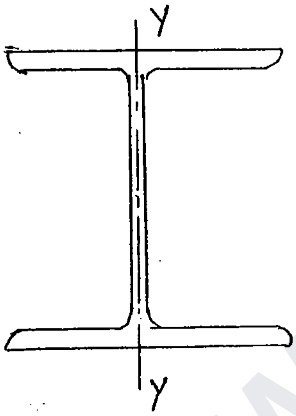
(Max. depth :- 600 mm)

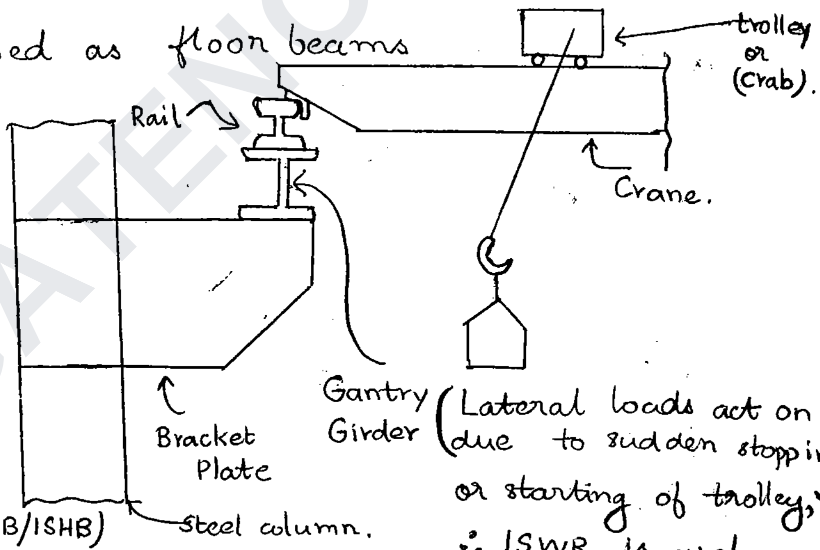
* Special I-section:

ISSC : Indian Standard Column Section

(Max. depth :- 250 mm)

* ISLB & ISMB are used as floor beams

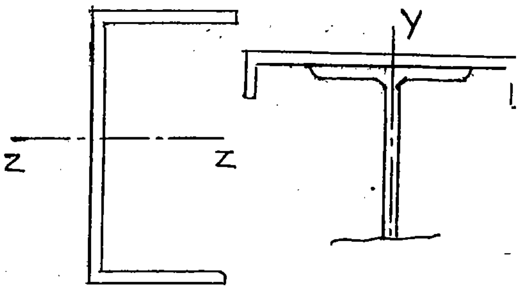




(Lateral loads act on it due to sudden stopping or starting of trolley, ∴ ISWB is used)

$I_{YY}(ISWB) > I_{YY}(ISLB/ISMB/ISHB)$

To increase I_{yy}, sometimes channel sections are also added to the flanges. ∴ I_{zz} of channel section will add to the I_{yy} of I section.



* ISHB is used for columns.

Sectional area & radius of gyration for ISHB are more compared to its counterparts

Critical load carrying capacity, $P_{cr} = \frac{\pi^2 EI}{l^2} = \frac{\pi^2 E A r^2}{l^2}$

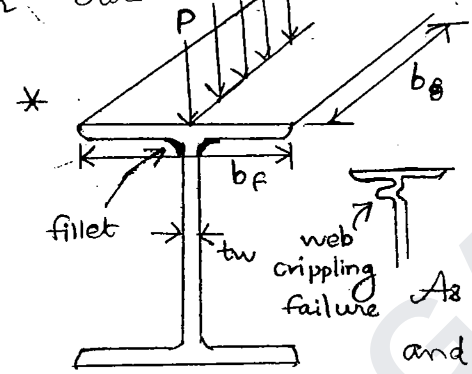
$$P_{cr} = \frac{\pi^2 EA}{(l/r)^2} = \frac{\pi^2 EA}{\lambda^2} \qquad \lambda = \frac{L}{r}$$

$$(P_{cr})_{zz} = \frac{\pi^2 EA}{(l/r_{zz})^2} \quad ; \quad (P_{cr})_{yy} = \frac{\pi^2 EA}{(l/r_{yy})^2}$$

$$r_{zz} = \sqrt{\frac{I_{zz}}{A}} \quad ; \quad r_{yy} = \sqrt{\frac{I_{yy}}{A}} \Rightarrow r_{zz} > r_{yy}.$$

$$(P_{cr})_{zz} > (P_{cr})_{yy}.$$

Columns are designed for $(P_{cr})_{yy}$ and failure will be along y-y axis, However, solid circular sections or hollow circular sections are most efficient column sections. So ISHB used in older constructions.



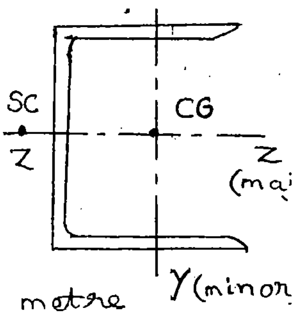
For the flange, stress $= \dfrac{P}{b \times b_f}$.

For the web, stress $= \dfrac{P}{b \times t_w}$

As $b_f > t_w$, stress conc. occurs in web. and ∴ web crippling failure may occur.

∴ area is increased in fillets to avoid stress concentration.

→ Channel or C – Sections



⊙ A channel section may be designated as

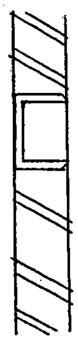
ISLC 350 @ 380.6 N/m

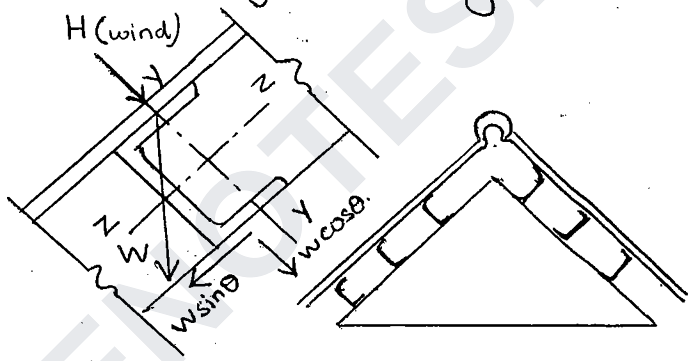
Depth of channel = 350 mm

Weight per running metre = 380.6 N/m (≈ 39.5 k

* $(I_{zz})_{channel} < (I_{zz})_{I\text{-}section}$

∴ channel sections are used for light loadings like lintels, roof purlins.



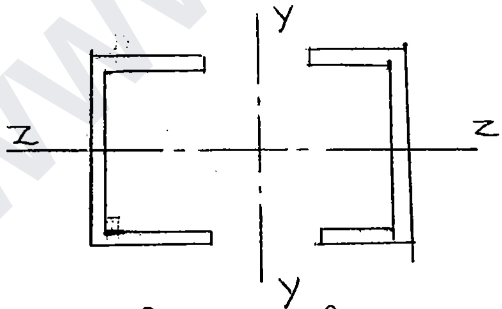


(H + Wcosθ) cause moment about major axis (Z-

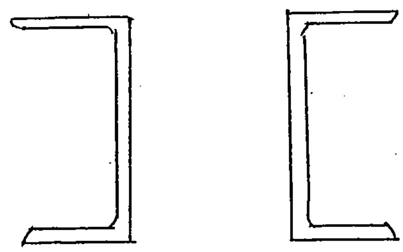
Wsinθ cause moment about minor axis (y-y)

∴ channel sections are used for light loads as its more economical than I-sections.

* Channel sections are also used as Built-up Column sectio



face-to-face



back-to-back

* Types of Channel Sections.

(i) ISJC – Indian Standard Junior Channels.

(ii) ISLC – Indian Standard Light Channels.

(iii) ISMC Medium wt. channel with sloping fl

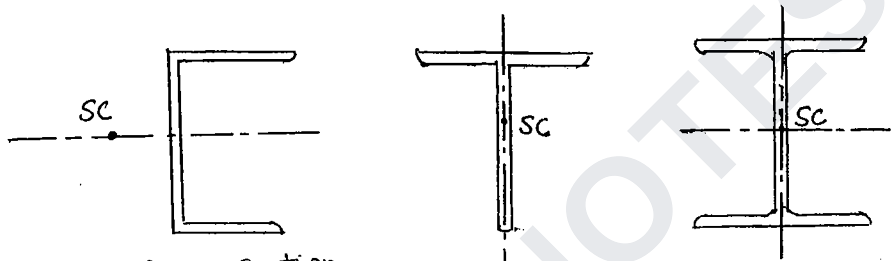
(iv) ISMCP Medium wt. channel with parallel fla

(v) ISGC. Gate Channels.

* SC is a point through which resultant transfer shear load must act if there is to be no twisting or torsion (section is subjected to BM only).

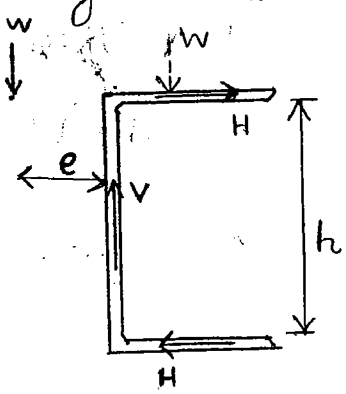
* If given member or section has axis of symmetry about both the principal axes, SC is coincident with CG of a section or member.

* If given section is symmetric about one axis, SC will be lying on axis of symmetry.



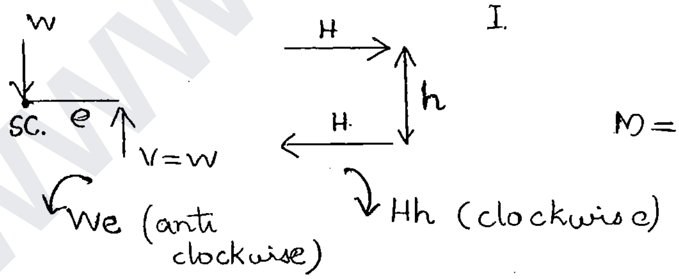
open section   open sections

If load acts at a point different from SC, twisting moment occurs.



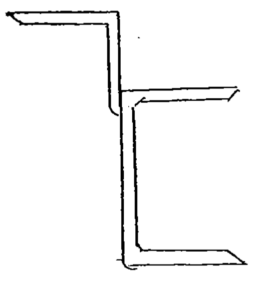
Shear stress, $q = \frac{VA\bar{y}}{I.b}$

Shear flow, $Q = q.b = \frac{VA\bar{y}}{I}$



$M = We - Hh$

* If W acts through SC, the net moment will reduced.
* If W acts at a different point from SC, 'We' will be clockwise and Hh will also be. This will increase the net moment.

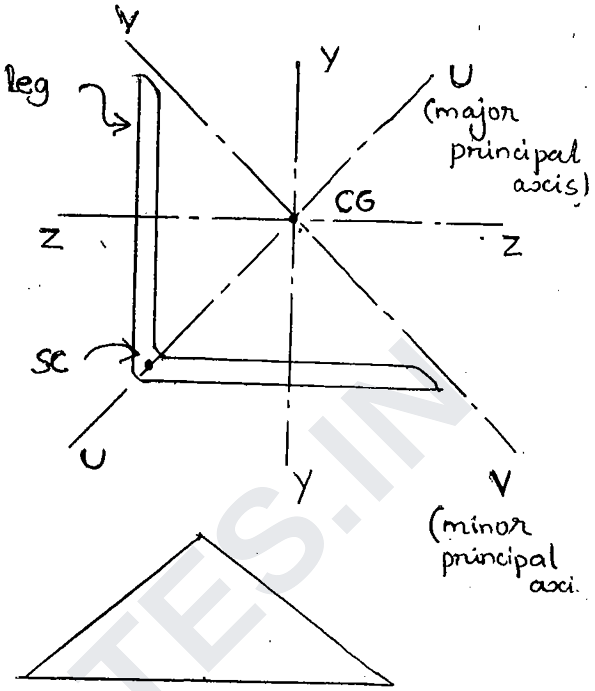


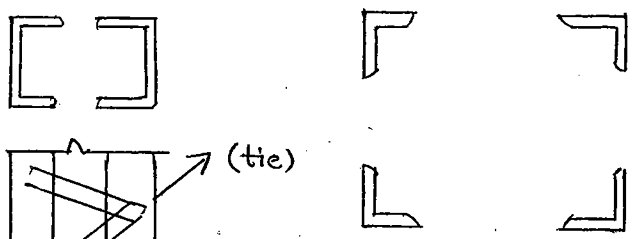
The purpose of angle section:

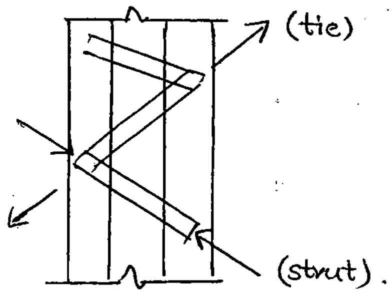
a) I_{zz} increases
b) I_{yy} increases. $\Delta \propto \frac{1}{I}$
c). Deflection decreases
d) makes load pass through SC.

→ Angle Sections (or) L Section

* Angle sections are used as. strut (compression) members or tie (tension) members in roof truss, built up colums, bracings etc.







* Three types of Angles.

(i) Equal Angles.

> Designated as. ISEA or ISA.

> Eg: ISEA 100X100X10.

> exccellent strut members

ISEA 100X100X10

$A = 1902\ mm^2$

$r_{vv} = 19.4\ mm.$

ISA 125X75X10.

$A = 1903\ mm^2$

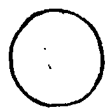
$r_{vv} =$

rd Aug, TURDAY

→ Indian Standard Bars

(i) Indian Standard Round Bars.

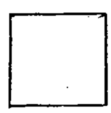
Designated as ISRO 20



> rivets, bolts, welding rods.

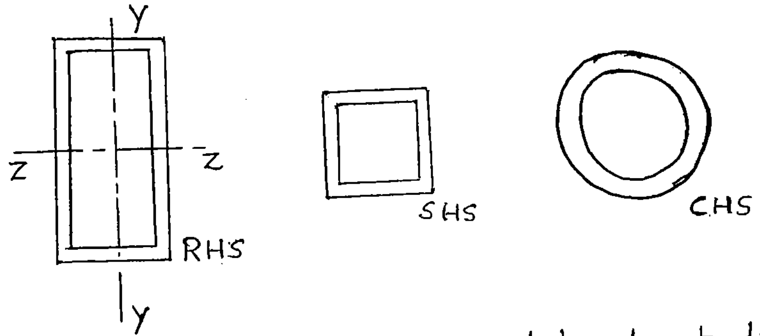
(ii) Indian Standard Square Bars.

Designated as ISSQ 50



→ Indian Standard Tube Section

(i) Rectangular Hollow Sections. (RHS)
(ii) Square Hollow Sections (SHS)
(iii) Circular Hollow Sections (CHS)



To withstand torsional loads, tubular sections are most efficient.

Polar moment of inertia, $I_p = I_{xx} = I_{zz} + I_{yy}$.

$$\frac{T}{J} = \frac{f_s}{r}$$

$$T = \frac{f_s}{r} \times J \qquad (J = I_p).$$

(I_{zz} & I_{yy} are very large for tubular section)

NOTE:

(i) The best section or economical section for column is Solid Circular section or hollow circular section.

(ii) For same weight (c/s area), hollow circular section will provide higher radius of gyration than solid circular section. Hence H.C.S will provide higher compressive strength than S.C.S.

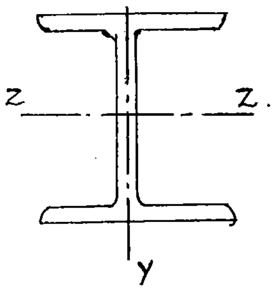
$r_{SCS} < r_{HCS}$.

$$P_{cr} = \frac{\pi^2 E I}{l^2} = \frac{\pi^2 E A r^2}{l^2}$$

$$\Rightarrow \underline{\underline{P_{cr} \propto r^2}}$$

⊙ For sections having symmetry about both the axes, bending will be about the axis of symmetry.

⊙ For sections which are symmetric about only one axis, bending will be about the axis of symmetry or an axis normal to the axis of symmetry.



→ Indian Standard Plates (when $t \geq 5$ mm).

Designated as ISPL

Eg:- ISPL $2000 \times 1000 \times 10$ mm.

Length $L = 2000$

Breadth $B = 1000$

Thickness $t = 10$

→ Indian Standard Sheets (when $t < 5$ mm)

Designated as ISSH

Eg: ISSH $1000 \times 600 \times 2$ mm.

$L = 1000$

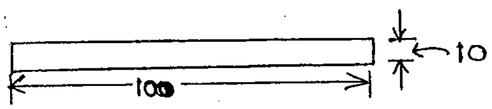
$B = 600$

$t = 2$

→ Indian Standard Flats. (when $t \geq 5$ mm)

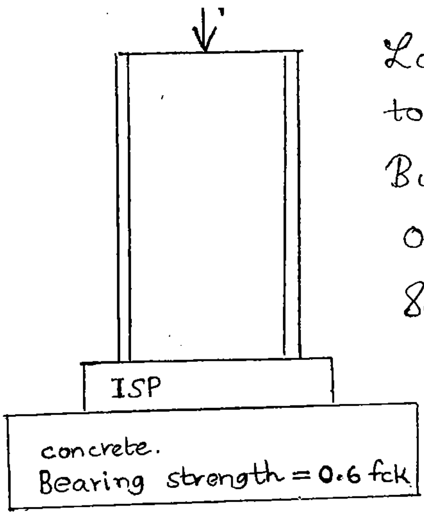
Designated as. ISF.

Eg: 100 ISF 10



$B = 100$ mm

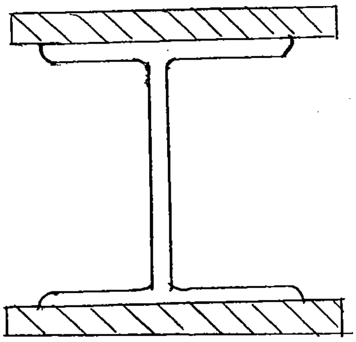
$t = 10$ mm



Load transferred by steel sections to the concrete foundation will be high. But bearing strength of concrete is only 0.6 fck.

So plates are used.

Bearing strength on plates $= \frac{P}{L \times B} < 0.6\ f_{c}$



$M \leq M_d$

$M_d = f(z)$.

Design moment should be greater than the ultimate bending moment acting on th the beams. To increase the M_d, I_{zz} value is increased making use of built up I-sections with flanged plates.

- Sheets are used as coverings.
- Flats are used as lacings and battens for built-up colum

→ Structural Steel vs. Aluminium.

Physical Properties	Steel	Aluminium
Density. ($\rho_a \approx \frac{1}{3}\rho_s$)	$\rho_s = 7850\ kg/m^3$.	$\rho_a = 2700\ kg/m^3$.
Youngs modulus. ($E_a \approx \frac{1}{3}E_s$)	$E_s = 2 \times 10^5$ MPa	$E_a = 0.7 \times 10^5$ MPa.
Coeffecient of Thermal expansion. ($\alpha_a \approx 2\alpha_s$)	$\alpha_s = 12 \times 10^{-6}/°c$.	$\alpha_a = 23 \times 10^{-6}/°c$.

(i) $\left(\frac{\text{Strength}}{\text{weight}}\right)_{\text{Al. sections}} > \left(\frac{\text{Strength}}{\text{weight}}\right)_{\text{structural steel sections}}$

But $E_a \approx \frac{1}{3} E_s$.

$P_{cr} = \frac{\pi^2 E A}{(l/r)^2}$ ⟹ For lighter compressive loads, Al section have more chances of buckling.

∴ Al is not used in civil construction applications. But for aeroplane constructions, Al is used as weight is more important parameter there. Aerospace constructions like missiles, space structures make use of Al.

(ii) Al sections are more corrosive resistant than structural steel sections. Hence maintenance may be lesser for Al sections

→ Method of Design.

(i) Working Stress Method (WSM) – FOS
(ii) Ultimate Load Design (ULD) – Load Factor
(iii) Limit State Design (LSD) – Partial Safety Factor.

$$FOS = \frac{\text{yield stress}}{\text{working stress}}$$

$$\text{Load Factor} = \frac{\text{ultimate load}}{\text{working load}}$$

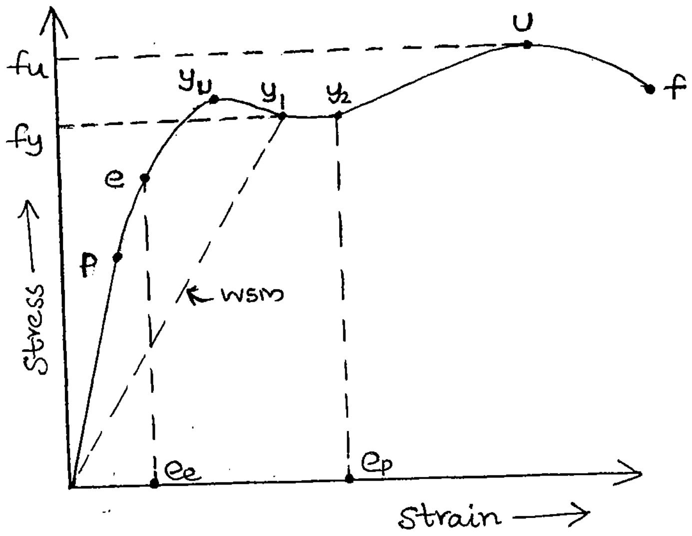
Partial Safety Factor – safety norm for load and material strength

⊙ Design is based on analysis of :-
(i) Design Force – shear force & axial force.
(ii) Design moments – twisting moments & bending mom..

⊙ Design is done to obtain :
(i) Shape (Eg: I section).
(ii) Size (Eg: ISMB 400).
(iii) Connection details.

⊙ Design requirements :
(i). Anticipated loads.
(ii) Deflection to be
(iii) Economical sections
(iv). Life span.

8th Aug, THURSDAY → Working Stress Method: (WSM)



P: Proportionality limit

e: elastic limit.

Y_U: upper yield point.

Y_1: lower yield point.

Y_1Y_2: plastic yielding

U: ultimate stress point.

f: Breaking stress point.

e_p : Plastic strain ≈ 10 to 15 times e_e

e_e : elastic strain.

NOTES:

- WSM of design is linear elastic method of design. In this method, it is assumed that the stress and strain varies linear upto yield point.
- In WSM of design, the stress in a member due to various working loads ~~are~~ working load combinations are to be evoluted, which are called 'Working Stress'.
- For safety of structural member, stresses due to various working load combinations must be less than or equal to permissible value of stresses. Permissible stress is a fraction of yield strength. ($K \cdot f_y$; $K < 1.0$)
- Design requirement for safety:
  - (i) Working stress due to DL + LL ≤ Permissible stress.
  - (ii) Working stress due to DL + WL ≤ Permissible stress.
  - (iii) Working stress due to DL + LL + WL ≤ 1.33 × Permissible stress
- Safety norm used in WSM of design is FOS.

$$FOS = \frac{\text{yield stress}}{\text{working stress}}$$

• WSM results in uneconomic sections as permissible stress is a fraction of yield stress. ∴ present code recommends Limit State Design.

→ Limit State Design.

• Types of Limit State.

(i) Limit State of strength.

(ii) Limit State of serviceability.

Limit State of strength	Limit State of serviceabil.
– strength (against yielding (tension) or buckling (column) or flexure (beam))	– deflection or deformations
– stability (against sliding or overturning) (retaining wall)	– vibrations
– fatigue strength (gantry girder)	– corrosions
– plastic strength.	– fire etc

• Design requirements for safety.

Design action (Sd) $\leq$ Design strength (Rd)

• Design action (Sd) : design values of internal forces (SF & AF), design moments (BM & TM) due to design loads (Fd).

• Design Load, Fd or Factored load = a partial safety factor or load factor × characteristic loads.

$$F_d = \gamma_L \times \text{characteristic loads.}$$

• Design strength (Rd) $= \dfrac{\text{characteristic strength}}{\text{partial safety factor or Resistance factor } (\gamma_m)}$

Resistance factor (or) partial safety factor against :

– yieldi strength, $\gamma_{m0} = 1.10$

– buckling strength, $\gamma_{m0} = 1.10$

– ultimate tensile strength, $\gamma_{m1} = 1.25$

- A load factor or a partial safety factor (γ_L)

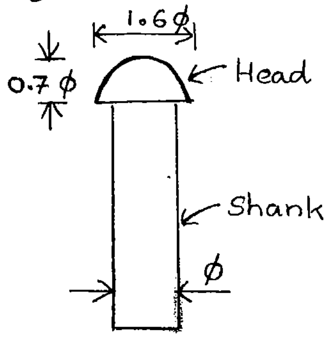
For strength limit state:

Load or load combination	γ_L
DL + LL	1.5
DL + WL	1.5
DL + LL + WL	1.2

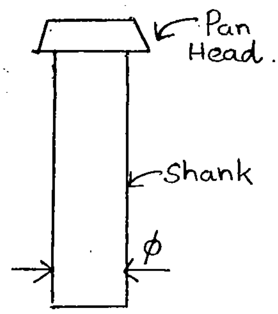
28th Aug,
THURSDAY

2. BOLTED CONNECTIONS

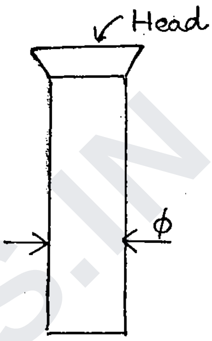
Rivet:



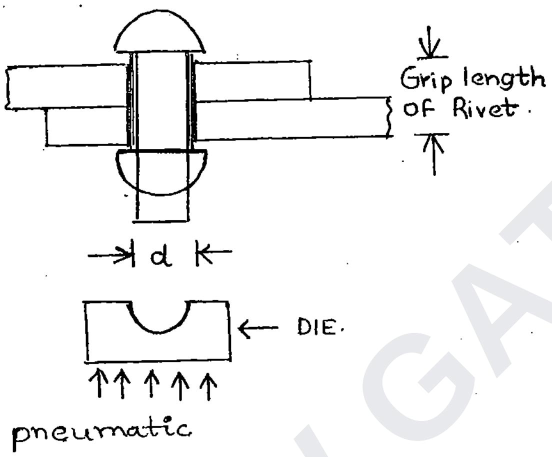
Snap Head or
Round Head Rivet.



Pan Head
Rivet



Flat counter
Shank rivet.



Nominal Diameter(φ) :

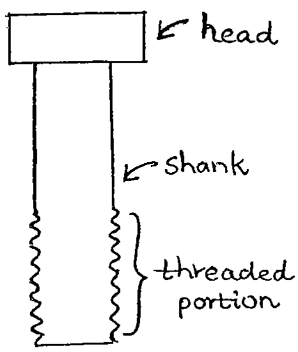
It is the diameter of rivet be rivetting process. It is same as sha diameter, φ

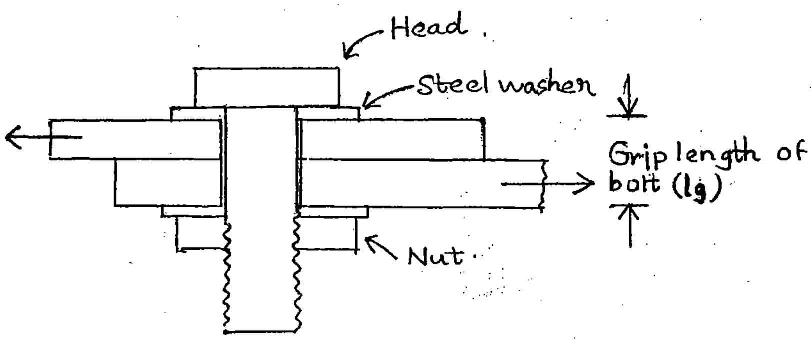
Effective Diameter or Gross Diameter(d

It diameter of rivet after rivet process. which is also same as dia. rivet hole (d).

$$d = \phi + 1.5\text{ mm} \quad (\text{when } \phi \leq 25\text{ mm})$$
$$= \phi + 2.0\text{ mm} \quad (\text{when } \phi > 25\text{ mm})$$
= diameter of rivet hole

Bolt & Bolting :





Hole diameter > 1 to 3mm than shank diameter

A bolt may be designated as :

M d x L

- d ← Shank diameter
- L ← length of bolt
- M ← metric size.

→ Types of Bolts :

(i) Unfinished Bolts / Ordinary / Rough / Black bolts.
(ii) High strength Friction grip bolts. (HSFG bolts)
(iii) Unfinished bolts / Black bolts / Ordinary bolts.

– Bolts are normally made from mild steel round bars. with square or hexagonal head shapes.

– Diameter of bolt available in market : 5 mm – 36 mm or designated as M5 to M36. However most commonly used ones a M16, M20, M24, M30.

– These bolts are recommended to use in connections which ar subj. to static loads and also used in secondary structures such as purlins, bracing members, tension or compression member in roof trusses.

– These are not recommended in structural connections which are subj. to dynamic loads and impact loads.

– Mechanical properties of these bolts are specified with grade property class.

Grade 4.6 to Grade 8.8 are available in market. However, most commonly used one is grade 4.6. Connection with the help of black bolt or unfinished bolt is eg. for bearing type connection or slip type connection or non-frictional connection.

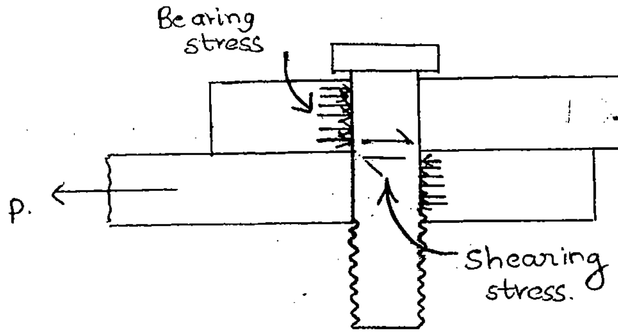
Grade 4 . 6

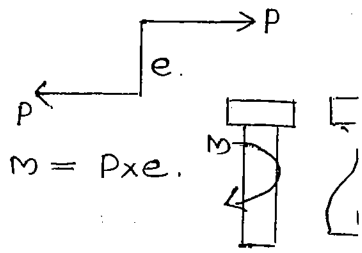
$4 \leftarrow \frac{1}{100}$ UTS

$f_{ub} = 400$ MPa

$6 \leftarrow 0.6 = \frac{f_{yb}}{f_{ub}}$

$f_{yb} = 0.6 \times 400$
$= 240$ MPa.





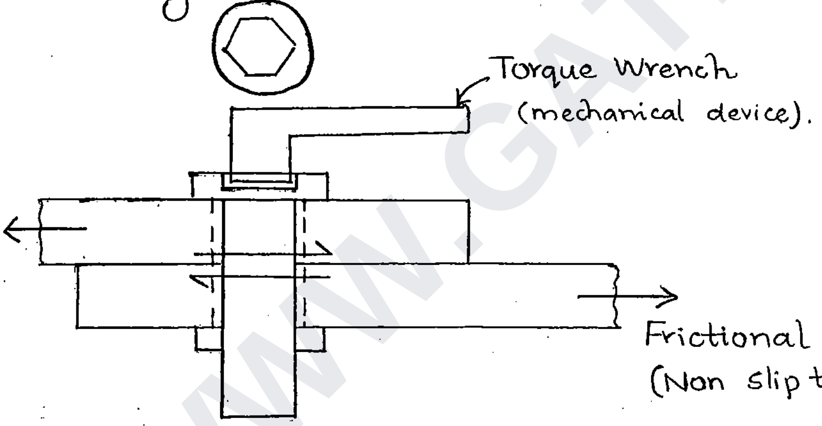
$M = P \times e.$

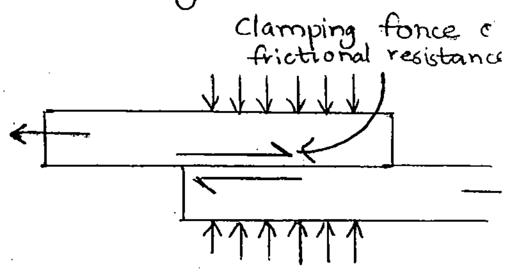
Force gets transmitted through bearing stress (compressive). Further if the grip length is more ($e \uparrow$), eccentricity will be more and bolt may fail due to tension caused by the moment ($M = P \times e$).

$$\therefore \frac{\text{Grip length}}{\text{Shank diameter}} \leq 8 \quad \text{ie} \quad \frac{l_g}{d} \leq 8$$

High Strength Friction Grip Bolts: (HSFG bolts).

- Made from medium carbon steel or high tensile strength steel.





Frictional type Connections
(Non slip type connections).

Proof load $\leq$ Yield strength of a bolt.

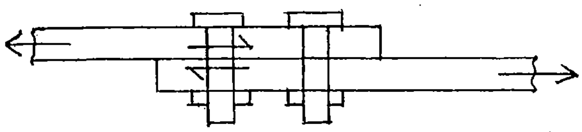
- Grade 10.9 s – 12.9 s

UTS $f_{ub} = 1040$ MPa

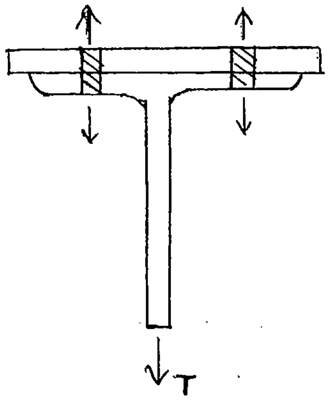
$f_{yb} = 940$ MPa

30th Aug, SATURDAY → Classification of Bolted Connection Based on Force experienced by the Bolt:

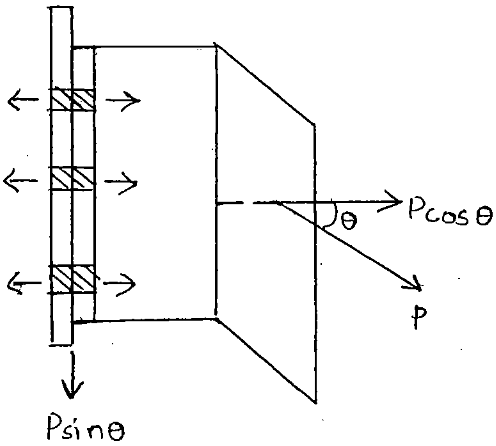
- Shear connections
- Tension connections
- Combined shear and tension connection.



Shear Connection

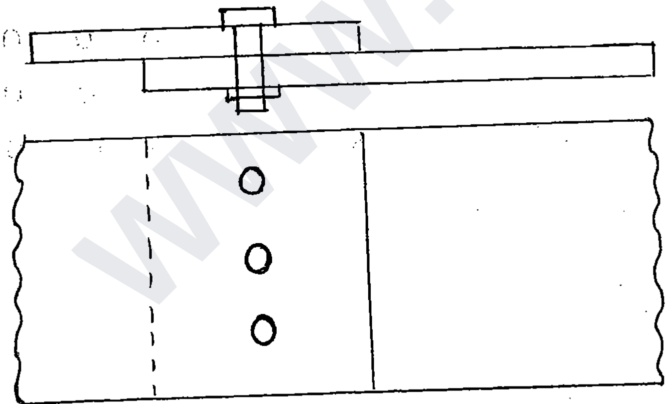


Hanger Connection

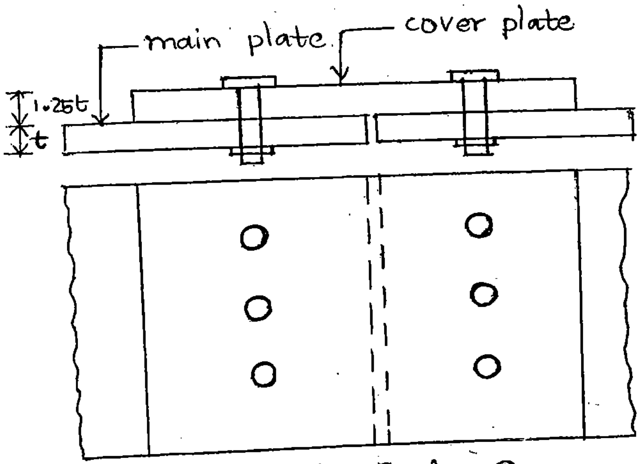


→ Types of Shear Connections:

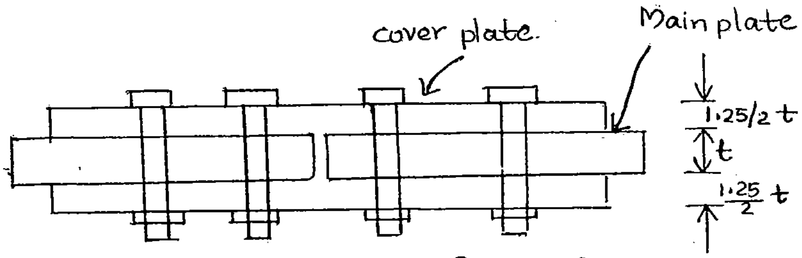
- Lap Connection
  - Single cover Butt connection
  - Double cover butt connection
- Butt connection



Single Bolted Lap Connection



Single Bolted Single Cover Butt connection.



Double Bolted Double Cover Butt connection

NOTES:

- It is desirable to use double cover butt connection for following reasons:

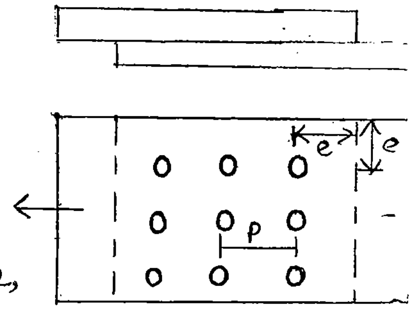
  (i) In double cover butt connection, CG of load in one connected member is lying with CG of load in another connect member. Hence connection is free from moment, whereas eccentricity of a load exist in lap connection & single cover butt connection

  (ii) Nominal shear capacity of bolt in double cover butt connection (2 no. of shear plates) is twice the nominal shear capacity of bolt in lap connection or single cover bolt connec

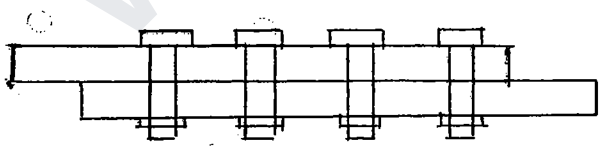
→ Specifications for Bolted Connections:

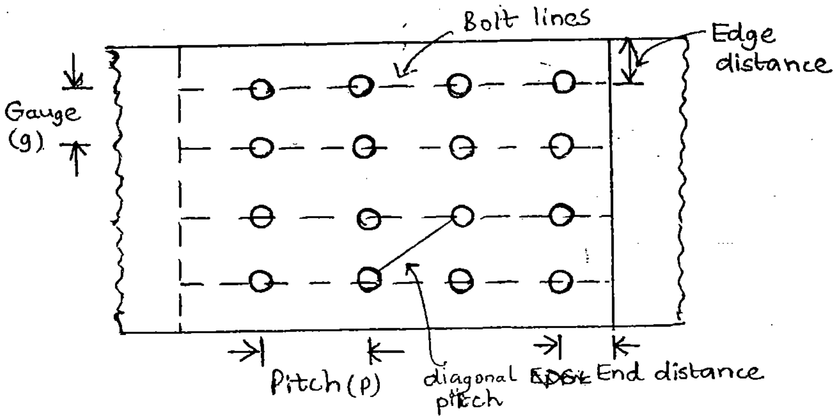
- No: of bolts, $n = \dfrac{\text{Design load}}{\text{Design strength of bolt.}}$

- Bolts are to be arranged in suitable pattern:
  a) Chain pattern
  b) Staggered pattern.
  c) Diamond pattern.



- Pitch, gauge, end distance, edge distance, diameter of bolt hole.





Pitch: It is the c/c distance b/w two adjacent bolts measured parallel to the direction of a load in a member. For wide plates, it is c/c distance b/w two adjacent bolts measured along length of connection.

* Diameter of Bolt Hole:

$$d_0 = d + 1.0 \text{ mm} \quad (12\text{mm} \leq d \leq 14 \text{ mm})$$
$$= d + 2.0 \text{ mm} \quad (16\text{mm} \leq d \leq 24\text{mm})$$
$$= d + 3.0 \text{ mm} \quad (d \geq 27\text{mm}).$$

$d \rightarrow$ shank diameter of bolt.

* Condition for Optimum Pitch:

Design strength of bolt per pitch = Design strength of plate per pitch.

* Minimum pitch, $P_{min} \not< 2.5 \times$ shank diameter of bolt.

$$P_{min} \not< 2.5\,d.$$

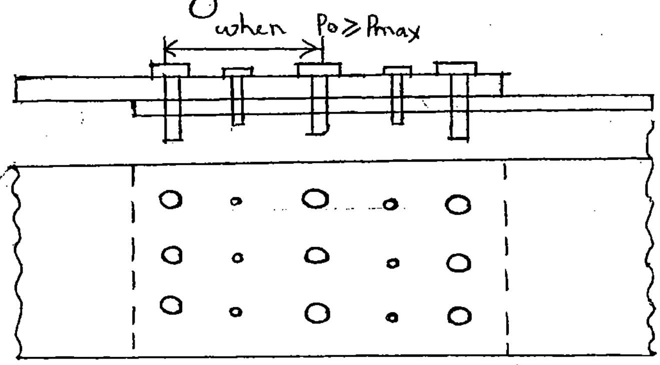
* Maximum pitch, $P_{max} = 12t$ or 200mm (whichever is le[ss]) for comp. member

$= 16t$ or 200 mm (whichever is less) for tension memb[er]

$t \rightarrow$ thickness of thinner connected member.

$$P_{min} \leq P_0 \leq P_{max}.$$

* Tacking Bolts or Stitch Bolts



When $P_0 > P_{max}$, buckling b/w members occur. To avoid the tacking bolts are used, which will prevent the unsupported member from buckling.

For plates :

Maximum pitch of tacking or stitch bolts,

$P_{max} = 32t$ or 300 mm (whichever is less, when plates are not exposed to weather

$= 16t$ or 200 mm (whichever is less when plates are ~~not~~ exposed to weather)

For angles :

- $P_{max} \ngtr 600$ mm (for compression member)
- $P_{max} \ngtr 1000$ mm (for tension member.

→ Gauge (g)

It is the c/c distance b/w two adjacent bolts measured normal to the direction of load in a member or it is distance b/w two adjacent bolt lines.

* End distance

It is distance b/w centre of bolt hole to the nearest edge of a main member or cover plate measured parallel to the direction of load in a member

* Edge Distance

It is distance b/w centre of bolt hole to the nearest edge of a main member or cover plate measured normal to the direction of load in a member.

– To ~~provide~~ prevent block shear failure of a member, min. end distance is provided.

$e_{min} \approx 1.5 *$ diameter of bolt hole $(1.5\, d_o)$

(Machine flame cut edges) $\approx 1.7\, d_o$ (for hand flame cut edges)

— Max. end / edge distance, $e_{max} = 12t\epsilon$; $\epsilon = \sqrt{\frac{250}{f_y}}$

For corrosive environment, $e_{max} = 40\text{ mm} + 4t$

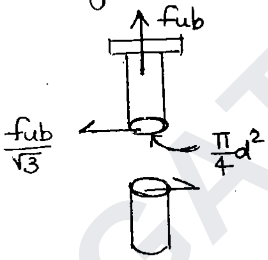
f_y = yield strength of a material.

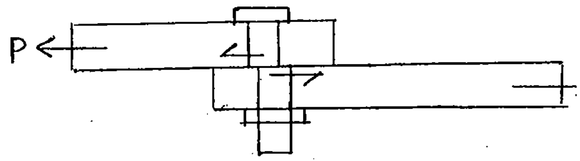
1st Sept, SUNDAY

→ Failures of Bolted Connections :

- Shear failure of bolts.
- Bearing failure of bolts.
- Tearing failure of bolts.
- Bearing failure of plate
- Tearing failure of plate.
- Block shear failure of plate.

(i) Shear Failure of bolts.

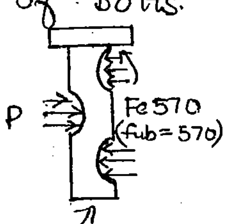


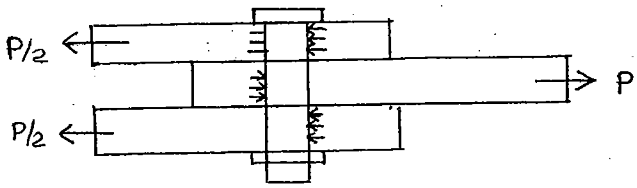


Shear stresses are generated when plates slip due to applied forces. When max. factored SF in the bolt may exceed nominal shear capacity of the bolt, shear failure of bolts may occur at plate of interface

Failure occurs when design action, $P > \frac{\pi}{4} d^2 . \frac{f_{ub}}{\sqrt{3}}$

(ii) Bearing Failure of Bolts.

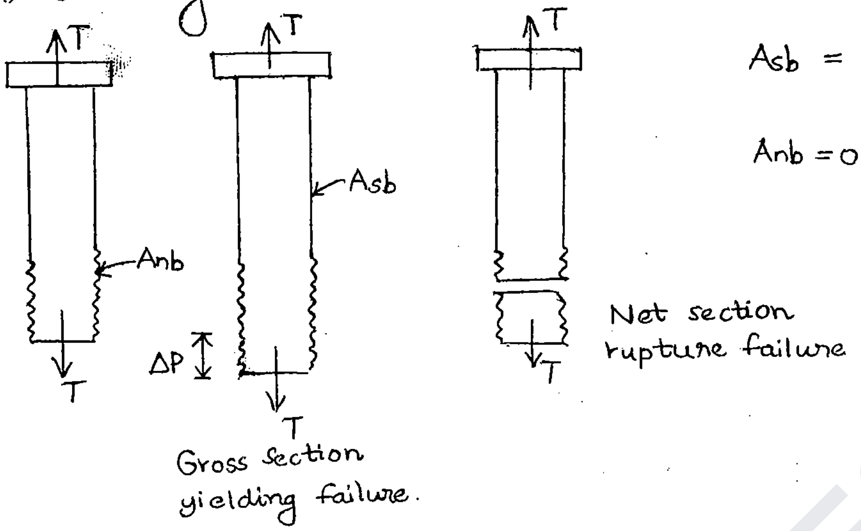




grade 4.6 (fub = 400 MPa)

The hoff shank of bolt is crushed when plate may be strong in bearing. The heavily stressed plate may ~~thus~~ press the hoffshank of the bolt. Bearing failure of bolt may not occur in practice except plates may be strong in bearing

(iii) Tearing Failure of Bolt.



$Asb = \frac{\pi}{4} d^2$; $Asb \rightarrow$ c/s area of shank

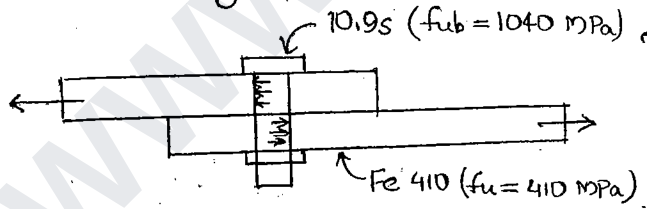
$Anb = 0.78 \frac{\pi}{4} d^2$; $Anb \rightarrow$ net area of threaded portion.

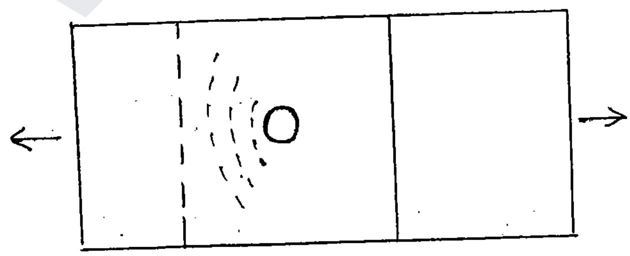
Gross section yielding failure occurs when uniform stress developed in gross area, $\frac{T}{Asb} = fyb$.

If bolt is subj. to tension, tearing failure may occur at threaded portion.

Net section rupture failure occurs when localised stress developed in net area, $\frac{T}{Anb} \approx fub$. Net section will not undergo deformation as it is subjected to localised stress alone.

(iv) Bearing Failure of Plate.

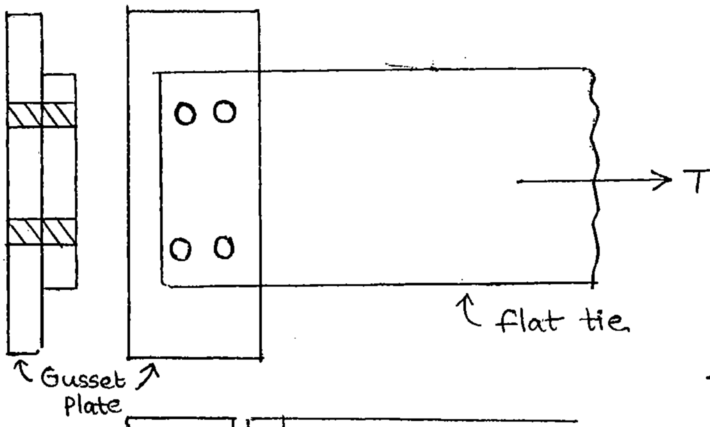




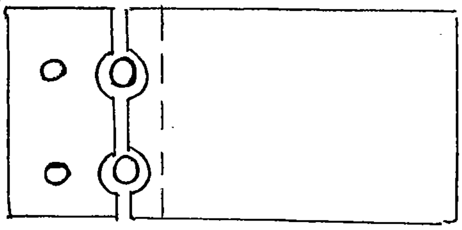
When ordinary bolt is subj. to factored SF, slip may takes place and bolt may come in contact with plate and plate material gets crushed. When plate is weak in bearing.

(v) Tearing Failure of Plate.

Tearing failure of a plate may occur when bolts are stronger than plate member.



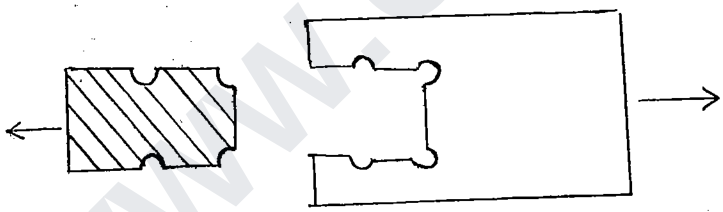
$$\frac{T}{A_n} = \frac{T}{(B-2d_0)t} = f_y.$$



At or near connection, only block shear failure & net sectio rupture take place. But block shear failure can be avoided by providing required end distance. Thus only net section rupture takes place.

(vi) Block Shear Failure of Plate.

When bolts are placed at lesser end distance than min. end distance as per IS 800 guidelines, a block of plate may seperate near end of connection. It can be eliminated by providing min. end distance.

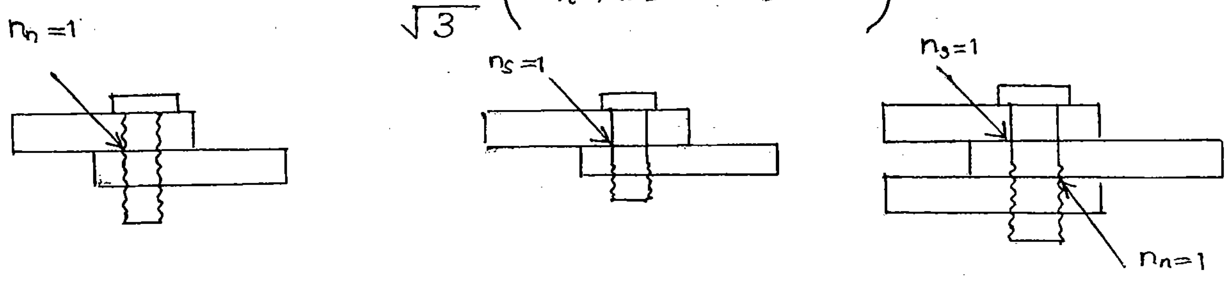


→ Design strength of Bearing Type Bolted Connection (V_{dc})

a) Design shear capacity (or) Strength of bolt (V_{dsb})

V_{nsb} = Nominal shear capacity of bolt.

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



Design shear strength of bolt, $V_{dsb} = \dfrac{V_{nsb}}{\gamma_{mb}}$

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

$f_{ub} \rightarrow$ ultimate tensile strength of bolt

$n_n \rightarrow$ no. of shear planes with thread intersecting shear plane.

$n_s \rightarrow$ no. of shear planes without thread intersecting shear plane

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

$A_{nb} \rightarrow$ net tensile area $\approx 78\%$ A_{sb} $= 0.78 \frac{\pi}{4} d^2$.

γ_{mb} = partial safety factor for bearing type bolt

$= 1.25$ for workshop bolting

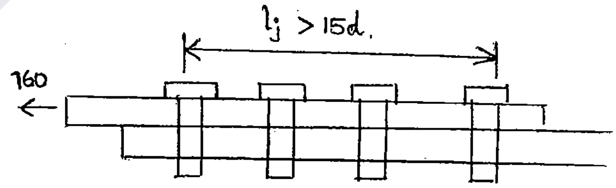
$= 1.25$ for site or field bolting

For lap connection or single cover butt connection,

$$n_n + n_s = 1$$

when $n_n = 1$, $n_s = 0$

$n_n = 0$; $n_s = 1$.



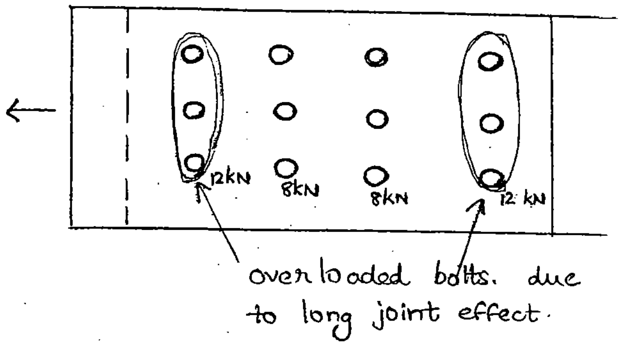
For double cover butt connection,

$$n_n + n_s = 2$$

when $n_n = 1$, $n_s = 1$

$n_n = 2$, $n_s = 0$

$n_n = 0$, $n_s = 2$.



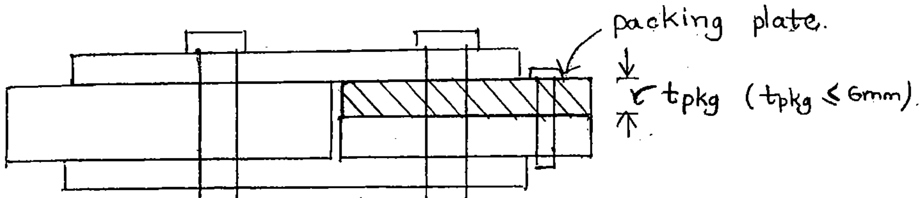
⊙ Nominal shear capacity of bolt is modified for long joint (when $l_j > 15d$), long grip bolt (when $l_g > 5d$) and thicker packing plate (when $t_{pkg} > 6$ mm)

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

$$V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}} = \frac{f_{ub}}{\sqrt{3}\,\gamma_{mb}} \left(n_n A_{nb} + n_s A_{sb}\right) \beta_{lj} \cdot \beta_{lg} \cdot \beta_{pkg}$$

* Reduction factor for Long grip bolt (β_{lg}) [when $l_g > 5d$]

$$\beta_{lg} = \frac{8d}{3d + l_g} \quad ; \quad l_g = \text{grip length of bolt } (l_g \ngtr 8d)$$



Higher the ~~excentr~~ thickness of packing plate, higher will be eccentricity and the moment. To avoid that, tacking bolts are provided when $t_{pkg} \leq 6mm$. But when $t_{pkg} > 6mm$, reduction for thickness, β_{pkg} is applied.

* Reduction factor for Thicker packing plate (when $t_{pkg} > 6mm$)

$$\beta_{pkg} = 1.0 - 0.0125\, t_{pkg} \quad ; \quad t_{pkg} \rightarrow \text{thickness in mm.}$$

b) Design Bearing strength of Bolt & Plate (V_{dpb})

Bearing area of bolt $= \pi dt$.

But hole diameter will have a tolerance of 1mm, 2mm or 3mm

$\therefore$ bearing area is $80\%\ \pi dt$.

$$80\%\ \pi dt = 2.5\, dt.$$

Nominal bearing strength of bolt and plate,

$$V_{npb} = 2.5\, dt \cdot f_u \cdot k_b.$$

$$V_{dpb} = \frac{V_{npb}}{\gamma_{mb}} = \frac{2.5\, dt \cdot f_u \cdot k_b}{\gamma_{mb}}.$$

K_b, the bearing factor is minimum of

$$\frac{e}{3d_0} \ , \ \frac{P}{3d_0} - 0.25 \ , \ f_{ub}/f_u \ , \ 1.0$$

e & p are end distance and pitch distance respectively measured parallel to bearing direction.

$d_0 \rightarrow$ diameter of bolt.

$f_u \rightarrow$ minimum value of UTS of bolt (f_{ub}) or plate.

c) Design Tensile Strength of Bolt, (T_{db})

T_{nb} = Nominal tensile strength of bolt.

$$T_{nb} = 0.9\, A_{nb}\, f_{ub} \leq A_{sb} \cdot f_{yb} \cdot \frac{\gamma_{mb}}{\gamma_{m0}}$$

0.9 as a safety factor as f_{ub} is used.

$$T_{db} = \frac{T_{nb}}{\gamma_{mb}} = \frac{0.9\, A_{nb}\, f_{ub}}{\gamma_{mb}} \leq \frac{A_{sb} \cdot f_{yb}}{\gamma_{m0}}.$$

* Design Strength of Bolt (V_{db}):

It is minimum strength of bolt based on design strength of bolt in shear (V_{dsb}), design strength of bolt in bearing (V_{dpb}) and design strength of bolt in tension (T_{db})

$$V_{db} = \text{minimum of} \begin{cases} V_{dsb} \\ V_{dpb} \\ T_{db} \text{ (if exist)} \end{cases}$$

* No. of bolts required for Concentric connection (n):

$$n = \frac{\text{Factored or design load}}{\text{Design strength of one bolt}} = \frac{P}{V_{db}}$$ (rounded off to nearest highest value)

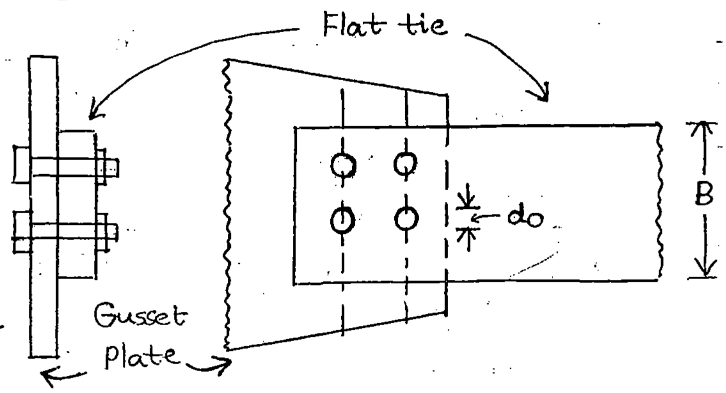
2nd Sept, TUESDAY

d) Design Tensile Strength of Plate

T_{np} = nominal tensile strength of plate.

$= 0.9\, A_n f_u$

$$T_{dp} = \frac{T_{np}}{\gamma_{m1}} = \frac{0.9 A_n f_u}{\gamma_{m1}}\,; \quad \gamma_{m1} = 1.25$$



A_n = Net effective sectional area $= (B - n d_o)t$ (for chain bolting)

* Design strength of Connection (V_{dc})

It is the minimum of V_{dsb}, V_{dpb}, T_{db} (if exist) and design tensile strength of plate

For safety connection (design requirement):

Design Action(P) ≤ Design strength of connection (V_{dc})

$$V_{dc} = \text{minimum of} \begin{cases} V_{dsb} \\ V_{dpb}. \\ T_{db} \text{ (if exist)} \\ T_{dp} \end{cases}$$

→ Efficiency of a Bolted Connection (η)

(Percentage Strength of a Bolted Connection)

$$\eta = \frac{\text{design strength of bolted connection } (V_{dc})}{\text{design strength of main plate } (T_{mp})} \times 100$$

$$\boxed{\eta = \frac{V_{dc}}{T_{mp}} \times 100}$$

* For wide plates:

$$\eta = \frac{\text{design strength of a bolted connection per pitch}}{\text{design strength of main plate per pitch}}$$

$$\boxed{\eta = \left(\frac{V_{dc}}{T_{mp}}\right)_{\text{per pitch}} \times 100}$$

* If $T_{dp} \leq V_{db}$,

$$\eta = \frac{V_{dc}}{T_{mp}} \times 100$$

$$= \frac{T_{dp}}{T_{mp}} \times 100$$

* Design strength of main plate (T_{mp})

⊙ $T_{mp} = \dfrac{A_g f_y}{\gamma_{m0}}$ (Based on gross section yielding)

$\dfrac{T}{A_g} = f_y$.

⊙ $T_{mp} = \dfrac{0.9 A_g f_y}{\gamma_{m1}}$ (Based on net section rupture)

Min. of above two will be used; but variation will be less.

$$\Rightarrow \quad \eta = \frac{0.9 A_n f_u / \gamma_{m1}}{0.9 A_g f_u / \gamma_{m}} \times 100$$

$$= \frac{A_n}{A_g} \times 100 = \frac{(B - n d_o)t}{B \times t} \times 100$$

$$\boxed{\eta = \left(\frac{B - n d_o}{B}\right) \times 100}$$

η will be maximum, when no. of bolt holes in failure is minimum. For diamond pattern of bolting, $n = 1$.

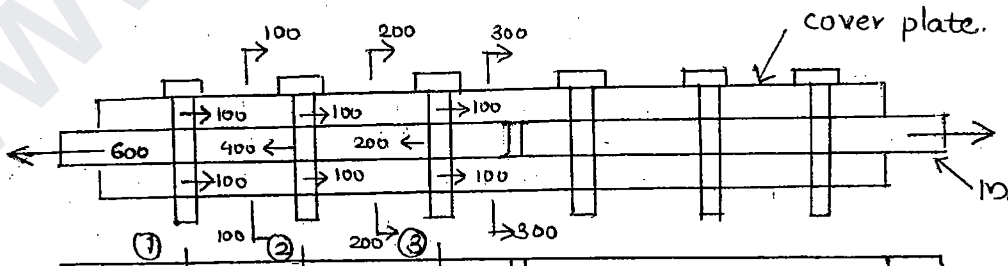
For staggered and chain pattern, $n = 3, 4, 5 \ldots$

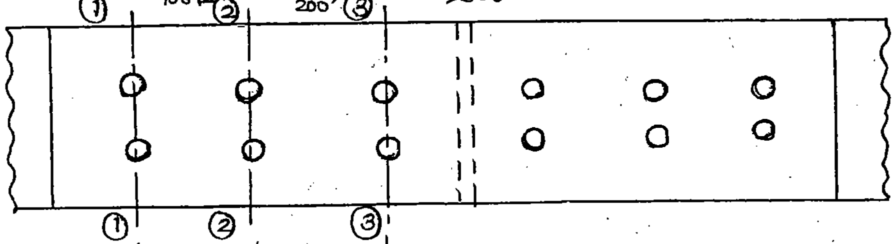
$$\eta_{\substack{\text{diamond}\\\text{pattern}}} = \left(\frac{B - d_o}{B}\right) \times 100$$

Assume,

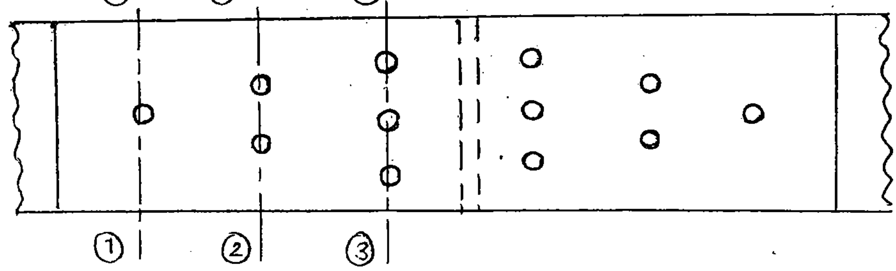
$P = 600$ kN

$V_{db} = 100$ kN.





Chain Pattern Bolting



Diamond Pattern of Bolting

* It is desirable to use diamond pattern of bolting :-

(i) Efficiency of diamond pattern of bolting is higher.

(ii) Cover plate material may be saved with diamond pattern of bolting.

(iii) Width of main plate required for diamond pattern of arrangement is less compared to chain or staggered pattern of bolting.

$$P = T_{dp} \leq V_{db}$$

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

$$\frac{P}{0.9 \frac{f_u}{\gamma_{m1}}} = A_n \Rightarrow \frac{P}{\left(0.9 \frac{f_u}{\gamma_{m1}}\right)t} + n d_o = B.$$

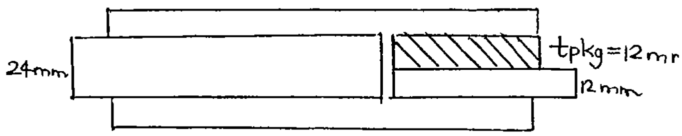
Critical section for main plate = section 1-1 (max loading)

No: of bolts in section 1-1 at chain bolting = 2.

" diamond bolting = 1.

P-16.

Q.1. $V_{dsb} = \frac{f_u}{\sqrt{3}\gamma_{mb}} (n_n A_{nb} + n_s A_{sb}) \beta_{pkg}$



Reduction factor for thickness packing plate (when $t_{pkg} \geq 6$ mm)

$$\beta_{pkg} = 1 - 0.0125\, t_{pkg}$$

$$= 1 - 0.0125 \times 12 = 0.85.$$

V_{dsb} reduced by $\underline{\underline{15\%}}$

Q.2. $P = 600$ kN

$V_{dsb} = 40$ kN ; $V_{dpb} = 60$ kN ; $T_{dp} = 50$ kN.

$$n = \frac{\text{design load}}{\text{design strength of one bolt}} = \frac{P}{V_{db}} = \frac{600}{40} = \underline{\underline{15 \text{ no.s}}}$$

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

For triple bolted double cover butt joint, $n_s = 3 \times 2 = 6$.

For one bolt in single shear, $n_s = 1$.

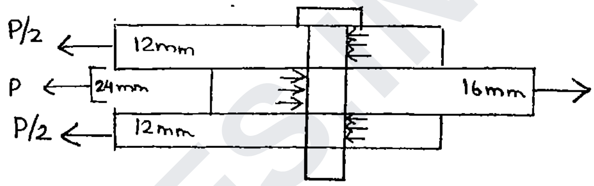
$\therefore \quad \underline{\underline{n = 6}}$

Q.5 For M16 Bolt, $d = 16$ mm

$f_{ub} = 400$ MPa, $f_y = 240$ MPa

$\gamma_{mb} = 1.25$

Bearing factor, $k_b = 0.5$.



* Since it is a double cover butt joint, only shearing and bearing failure occurs. There won't be tearing failure.

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

$$= \frac{400}{\sqrt{3} \times 1.25} \left(1 \times 2 \times \frac{\pi}{4} \times 16^2\right) = 74.2 \text{ kN}.$$

$$V_{dpb} = \frac{2.5\, d t f_u k_b}{\gamma_{mb}} = \frac{2.5 \times 16 \times 16 \times 400 \times 0.5}{1.25}$$

$$= 102.4 \text{ kN}.$$

Design strength of bolt, $V_{db} = \underline{\underline{74.2}}$ kN

Q.5. $d = 20$ mm, $f_{ub} = 400$ MPa, $f_y = 240$ MPa $\quad d_0 = d + 2 \quad (16 \leq d \leq 24)$

End distance, $e = 33$ mm, pitch $p = 50$ mm $\quad = 22$ mm

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

↳ thread Intercept shear plane.

$$= \frac{400}{\sqrt{3}} \left(0 + 1 \times \frac{\pi}{4} \times 20^2 \times 0.78\right) = 45.2 \text{ kN}.$$

$$\frac{e}{3d_0} = \frac{33}{3 \times 22} = 0.5 \qquad \Rightarrow \text{Bearing factor, } k_b = 0.5. \qquad \frac{f_{ub}}{f_u} = \frac{400}{410} = 0.97$$

$$\frac{p}{3d_0} - 0.25 = 0.507.$$

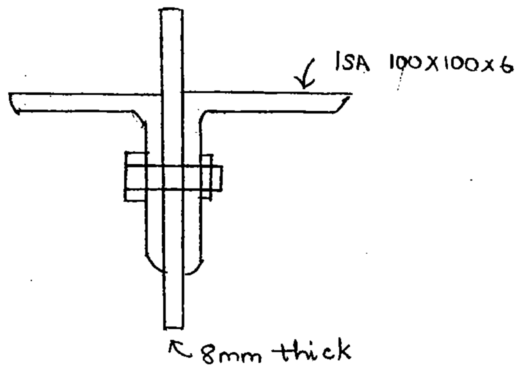
Design bearing strength of bolt, $V_{dpb} = \frac{2.5\, d\, t\, f_{ub}\, k_b}{\gamma_{mb}}$

$$= \frac{2.5 \times 20 \times 12 \times 400 \times 0.5}{1.25}$$

$$= \underline{\underline{96 \text{ kN}}}$$

7. $d = 16$ mm , Grade 4.6 $\Rightarrow f_{ub} = 400$ MPa
$f_y = 240$ MPa.

8. Ultimate load (design load/ factored load) $= P = 150$ kN.



For grade 4.6 bolt, $f_{ub} = 400$ MPa
$f_y = 240$ MPa

For grade Fe 410 plate, $f_u = 410$ MPa

$e = 30$ mm, $p = 40$ mm

M16 : $d = 16$ mm, $d_0 = 18$ mm.

$$n = \frac{P}{V_{db}}$$

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

Thread intersecting shear plane, $n_s = 0$, $n_n = 2$.

$$= \frac{400}{\sqrt{3} \times 1.25} \left(1 \times 2 \times 0.78 \times \frac{\pi}{4} 16^2 \right) = \underline{\underline{57.9 \text{ kN}}}$$

$$V_{dpb} = 2.5 \times d t \frac{f_{ub} \times K_b}{\gamma_{mb}} = 2.5 \times 16 \times 8 \times 400 \times \frac{k_b}{1.25}$$

$t_{(AM)} = 6+6 = 12$ m

$t_{(GP)} = 8$ mm

K_b is minimum of :

$$\frac{e}{3d_0} = \frac{30}{3 \times 18} = 0.55$$

$$\frac{P}{3d_0} - 0.25 = \frac{40}{3 \times 18} - 0.25 = 0.49$$

$$\frac{f_{ub}}{f_u} = \frac{400}{410} = 0.97$$

$$V_{dpb} = 2.5 \times 16 \times 8 \times 400 \times \frac{0.49}{1.25} = 50.25 \text{ kN.}$$

$$V_{db} = 50.25 \text{ kN}$$

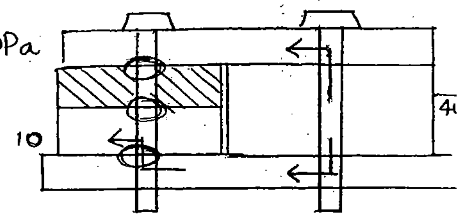
$$n = \frac{150}{50.25} = \underline{\underline{3}}$$

3rd Sept,
WEDNESDAY

10. $P = 400$ kN.

For 4.6 grade bolt, $f_{ub} = 400$ MPa, $f_u = 410$ MPa

$K_b = 0.5$, For M20 bolt , $d = 20$ mm.



No: of bolts required , $n = \frac{P}{V_{db}}$.

V_{db} = design strength of one bolt = minimum of V_{dsp} or V_{dpb}.

$$V_{dsp} = \frac{f_{ub}}{\sqrt{3}\gamma_{mb}} (n_n A_{nb} + n_s A_{sb}) \quad ; \quad n_n = 2 \quad n_n \neq 3$$

$$= \frac{400}{\sqrt{3} \times 1.25} \left(2 \times 0.78 \times \frac{\pi}{4} \times 20^2 + 0\right) \times \beta_{pkg}$$

β_{pkg} = reduction factor on thicker packing plate (when $t_{pkg} > 6$ m

$$\beta_{pkg} = 1 - 0.0125 \times t_{pkg} = 1 - 0.0125 \times 8 = \underline{\underline{0.9}}$$

$$V_{dsb} = \underline{\underline{81.25}} \text{ kN}$$

$$V_{dpb} = 2.5 d t \frac{f_{ub} K_b}{\gamma_{mb}} = 2.5 \times 20 \times 10 \times 400 \times \frac{0.5}{1.25} = \underline{\underline{80}} \text{ kN}$$

$t(MP1) = 18$ mm , $t(MP2) = 10$ mm , $t(CP) = 8+8 = 16$ mm

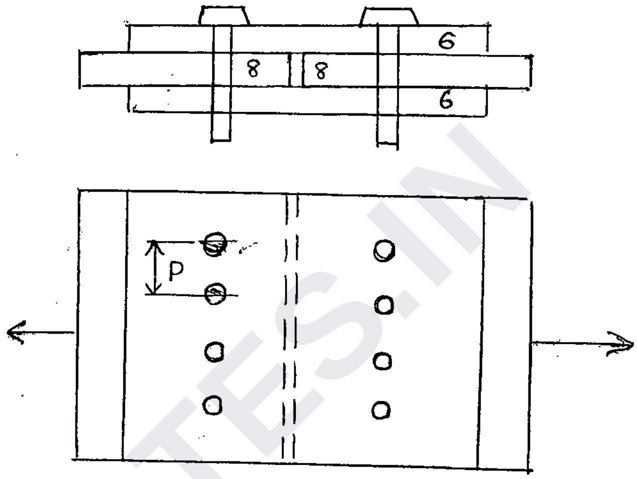
$$n = \frac{P}{V_{db}} = \frac{400}{80} = \underline{\underline{5 \text{ bolts}}}$$

11. Failures are :

SFB $\rightarrow V_{dsb}$

$f_{ub} = 400$ BFB, $f_u = 410$ BFP $\Big\}$ V_{dpb} ($f_{ub} < f_u$; bolt will fail in bearing)

TFP $\rightarrow T_{dp}$.



<u>For wide plate,</u>

Efficiency of the connection

$$\eta = \frac{\text{design strength of connection (per pitch)}}{\text{design strength of main plate (per pitch)}} \times 100 = \left(\frac{V_{dc}}{T_{mp}}\right)_{\text{per pitch}} \times 100$$

V_{dc} per pitch = min. of V_{dsb} or V_{dpb} or T_{dp} per pitch.

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

$$= \frac{400}{\sqrt{3} \times 1.25} \left(1 \times 2 \times 0.78 \times \frac{\pi}{4} \times 16^2 + 0\right) = \underline{\underline{57.95}} \text{ kN}$$

V_{dpb} = design bearing strength of bolt & plate

$$= 2.5\, d t f_{ub} \cdot \frac{K_b}{\gamma_{mb}}$$

$t(MP) = 8$ mm

$t(CP) = 6 + 6 = 12$ r

$$= 2.5 \times 16 \times 8 \times 400 \times \frac{0.55}{1.25}$$

$$= \underline{\underline{56.32}} \text{ kN}$$

Design tensile strength of plate per pitch (T_{dp}) :

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

$A_n = (B - n d_0) t$

$= (p - n d_0) t$

$= (45 - 1 \times 18) 8 = 216 \text{ mm}^2$

Per pitch length :

$B = p$

$n = 1$

$$T_{dp} = 0.9 \times 216 \times \frac{410}{1.25} = \underline{\underline{63.7}}\ kN$$

V_{dc} per pitch $= 56.32$ kN.

T_{mp} = design strength of main plate.

$$= \frac{0.9\, A_g f_u}{\gamma_{m1}} \quad \text{or} \quad \frac{A_g f_y}{\gamma_{mo}}.$$

$$= \frac{0.9 \times 45 \times 8 \times 410}{1.25} = \underline{\underline{106.27}}\ kN$$

$$\eta = \frac{56.32}{106.27} \times 100 = \underline{\underline{53\%}}$$

12. $$V_{dsb} = \frac{f_{ub}}{\sqrt{3}\,\gamma_{mb}}\left(n_n A_{nb} + n_s A_{sb}\right) = \frac{400}{\sqrt{3} \times 1.25}\Big(\underbrace{2}_{\text{2 bolts per pitch length}} \times 2 \times 0.78 \times \frac{\pi}{4} \times 16^2 + 0\Big).$$

$$= 115.88\ kN.$$

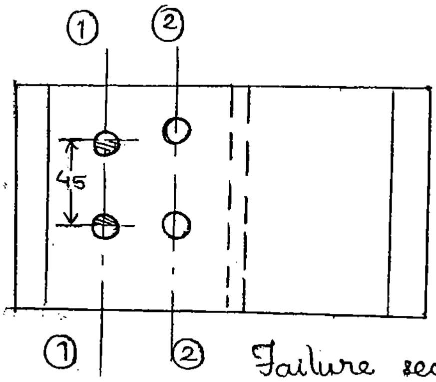
$$V_{dpb} = 2.5\, d t\, f_{ub} \cdot \frac{K_b}{\gamma_{mb}} \times 2 \rightarrow \text{no. of bolts per pitch length}$$

$$= 2.5 \times 16 \times 8 \times 400 \times \frac{0.55}{1.25} \times 2 = \underline{\underline{112.64}}\ kN$$

$T_{dp} = 0.9\,\frac{A_n f_u}{\gamma_{m1}}$; $A_n = (B - n d_o)t$; $n \rightarrow$ no. of bolts in failure section

$n = 1$

$$= 0.9\,(45 - 1 \times 18)\, 8 \times \frac{410}{1.25} = \underline{\underline{63.7}}\ kN$$



Failure section is 1–1

$$V_{dc} = \underline{\underline{63.7}}\ kN$$

$$\eta = \frac{63.7}{106.27} \times 100 = \underline{\underline{59.99}}\ \%$$

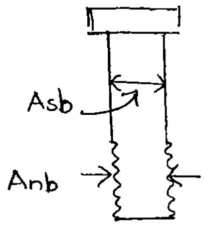
If $T_{dp} \leqslant V_{db}$, $\quad T_{dp} = 63.7$ kN, $V_{db} = 112.64$ kN.

$$\eta = \left(\frac{B - n d_o}{B}\right) \times 100$$

per pitch length: $B = p$ & $n = 1$ (for any no. of bolts).

$$\eta = \left(\frac{p - d_o}{p}\right)100 = \left(\frac{45 - 18}{45}\right) \times 100 = \underline{\underline{59.99}}\%$$

13 . For M16 bolt, $d = 16\,mm$, For grade 4.6 bolt,
$f_{ub} = 400$ MPa, $f_{yb} = 240$ MPa



$$\frac{T}{A_{sb}} = f_{yb}.$$

$$T_{nb} = f_{yb}\,A_{sb} \times \frac{\gamma_{mb}}{\gamma_{mo}}$$

$$T_{db} = \frac{f_{yb} \cdot A_{sb}}{\gamma_{mo}}$$

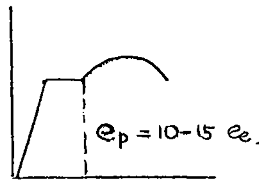
$$T_{db} = 0.9.\ A_{nb} \cdot \frac{f_{ub}}{\gamma_{mb}} \leq A_{sb} \cdot \frac{f_{yb}}{\gamma_{mo}}$$

$$= 0.9\left(0.78 \times \frac{\pi}{4} \times 16^2 \times \frac{400}{1.25}\right) \leq \frac{\pi}{4}\, 16^2 \times \frac{240}{1.1}$$

$$= 45.16 \leq 43.8 \qquad (\text{minimum of })$$

$$T_{db} = \underline{\underline{43.8}}\ kN$$

Stress distribution due to gross ($A_g = A_{sb}$) area is uniform.
∴ it causes deformations.



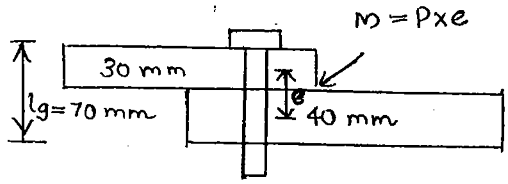
$$T_{nb} = A_{sb} \cdot f_y.$$

$$T_{db} = \frac{T_{nb}}{\gamma_{mb}} = \frac{A_{sb} \cdot f_y}{\gamma_{mb}}.$$

But γ_{mb} is used only when there are uncertainities.
But here bolt diameter is same and process of installatio at site and workshop are same and ∴ no uncertainities.
So less safety factor of γ_{mo} $(=1.1)$ is used.

But in rupture failure, there are uncertainities. like the threaded portion may be less than 78% of Asb.
So γ_{mb} $(=1.25)$ is used.

14. $\beta_{lg} \rightarrow$ reduction factor for long grip bolt



$$= \frac{8d}{3d + l_g} \qquad (l_g \not> 8d)$$

$8 \times 12 = 96\,mm$

$l_g > 5d\,(5 \times 12) = 60$ mm

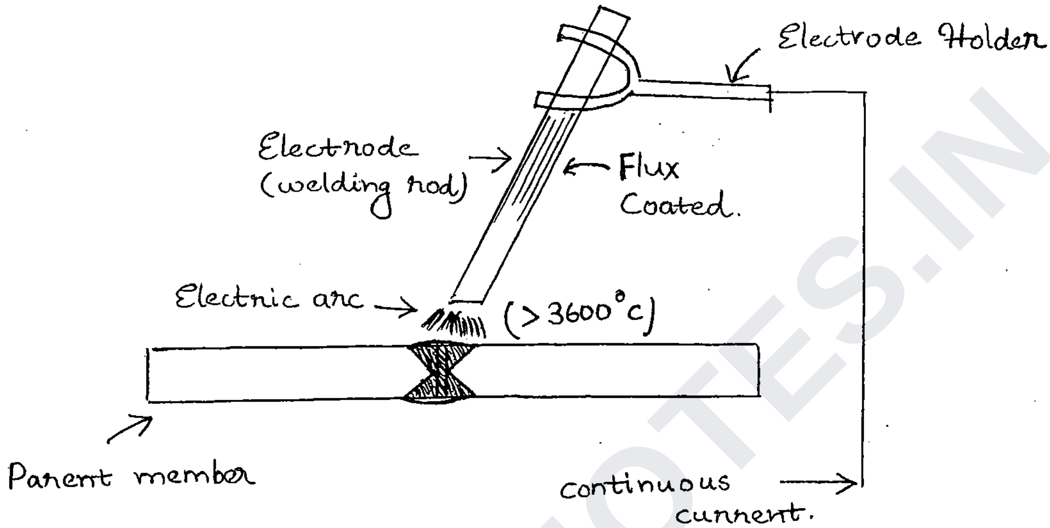
$l_g > 8d \Rightarrow$ increase d.

$$\beta_{lg} = \frac{8 \times 12}{3 \times 12 + 70} = 0.9056$$

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

V_{dsb} reduced by 9.44 % $\left(\frac{1 - 0.9056}{1} \times 100 \right)$

3. WELDED CONNECTIONS



MP of steel = 1450°

→ Welding Classification :

a) Fusion Process
- Electric arc welding or Metal arc welding.
- Gas welding.

b) Pressure Process
- Electric resistance welding.
- Forge welding.

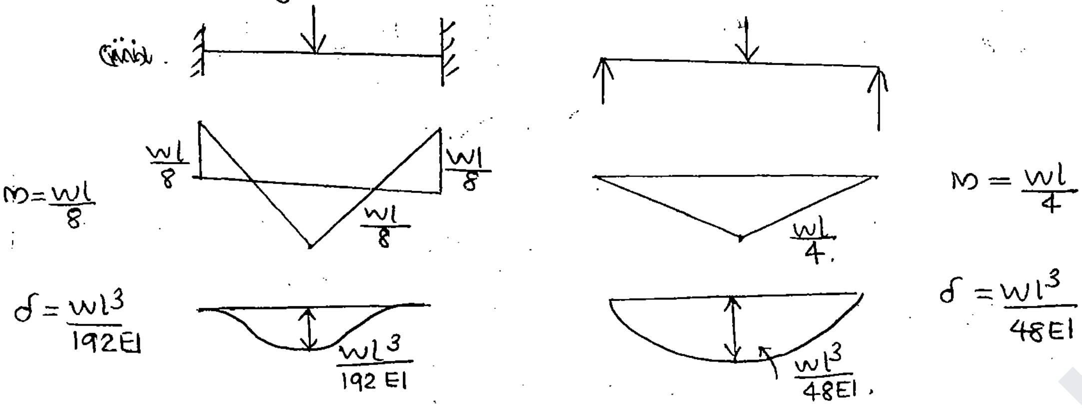
- Flux coating controls the melting of electrode. By adding some elements to the flux, mechanical properties of the joint can be enhanced.

→ Advantages :

(i) Weld joints are more efficient and stronger. Max. efficiency of bolted connection is 85%. But min. efficiency of welded connection is 95%. Bolt holes reduces η.

$$\eta = \left(\frac{B - n d_0}{B}\right) \times 100.$$

(ii) Moments and deflections are less due to rigid nature of welded joints.



Lesser the design moment, lesser depth of c/s can be used.

4th Sept, THURSDAY

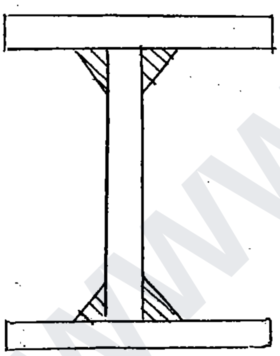
→ Types of Welds

(i) Fillet Welds or Lap welds.

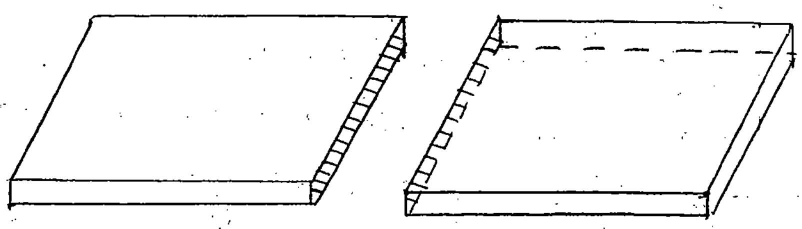
(ii) Butt or Groove welds.

(iii) Slot welds.

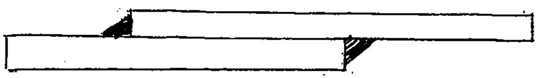
(iv) Plug welds etc.



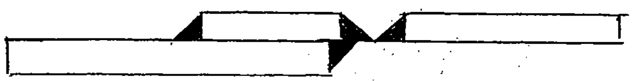
Joints two $\perp^r$ planes.



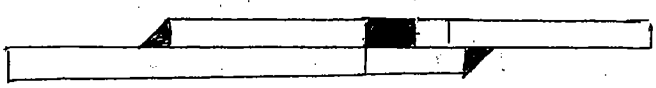
Butt weld. (Joints two // planes)



Fillet Weld.



Slot Weld

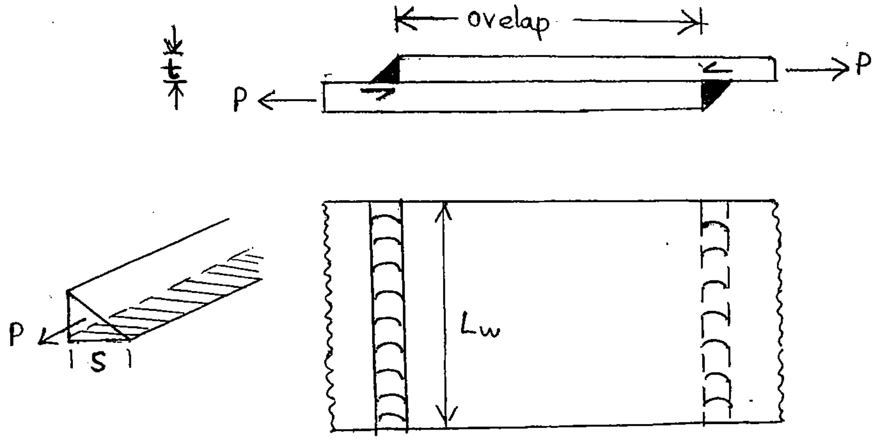


Plug Weld.

Slot weld & plug weld are the supporting welds.

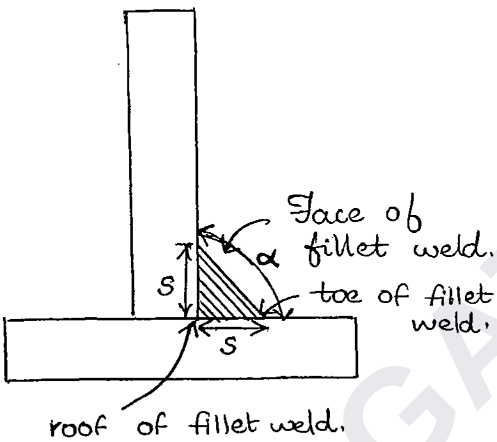
When the unsupported length b/w fillet welds ~~or butt~~ are mo. moments may be induced. To avoid that slot welds or plug welds are used.

→ Design of Fillet or Lap Weld



t → thickness of thinner connected member.

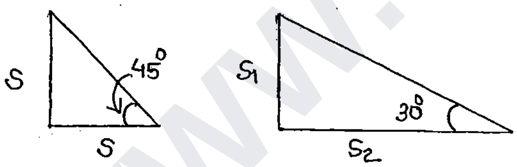
Minimum overlap required as per IS 800: 2007, ≮ 4t or 40 mm whichever is more.



Fillet weld symbol:

⊙ standard c/s fillet weld – Right angle triangle.

⊙ standard angle of fillet weld – 45°



S = minimum of S_1 & S_2

For 45° standard fillet, $S_1 = S_2 = S$

Shearing area of fillet weld $= S \times L_w$.

So minimum of S_1 & S_2 is selected to design shear stress on safer side.

* Size of Fillet Weld. (S)

– Distance b/w corner of fillet to the toe of fillet we

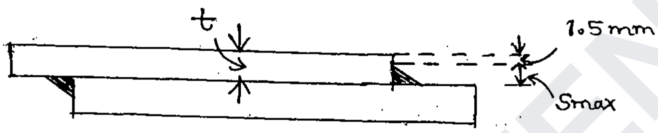
* Min. size of Fillet weld (S_{min}).

– Depends on thickness of thicker connected member.

Thickness of thicker connected member (mm)		Smin (mm)
over	upto & including	
0	10	3
10	20	5
20	32	6
32	50	8

* Maximum size of fillet of weld (S_{max})

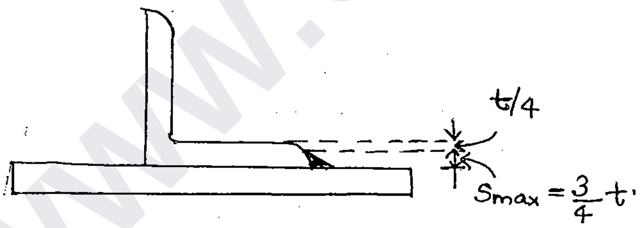
– For square edge (like plates or flat sections)



$t \rightarrow$ thickness of thinner connected member

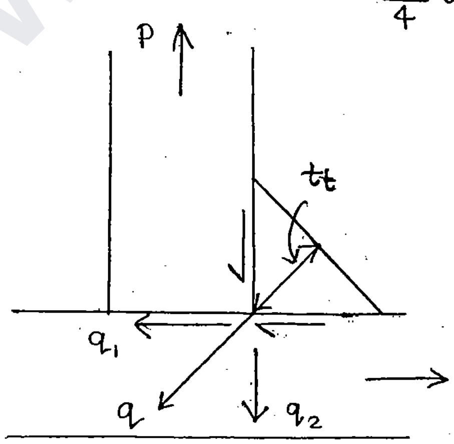
$$S_{max} = t - 1.5 \text{ mm}$$

– For round edges (like flange of an angle or flange of channel or leg of an angle etc)



$t \rightarrow$ thickness of round edge at toe.

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



$t_t \rightarrow$ thickness of throat.

Resultant shear act along throat fo critical combination of loads and $t_t < S$.

So failure will occur along th the throat of fillet weld.

$t_t \rightarrow$ min dimension in c/s of fillet weld.

$$t_{t(max)} = \frac{S}{\sqrt{2}} = 0.707 S \quad (\text{for } 45^\circ)$$

→ Effective Throat Thickness (t_t) :

– It is the distance b/w corner of fillet weld to the face of fillet weld.

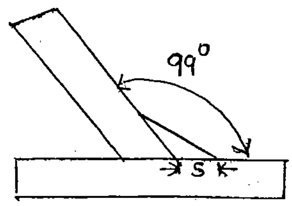
– t_t = k × size of fillet weld.

$$t_t = k \cdot s \qquad ; \qquad t_t \not< 3\text{ mm}$$

where k → constant which depends on angle b/w welds or fusion faces (α).

Angle b/w weld faces (α)	Values of K ($\alpha = 90$; $k = \frac{1}{\sqrt{2}}$)
$60^\circ - 90^\circ$	0.70
$91 - 100^\circ$	0.65
$101^\circ - 106^\circ$	0.60
$107^\circ - 113^\circ$	0.55
$114 - 120^\circ$	0.50

Q. The Effective throat thickness of fillet weld shown is: (GATE 2011)



a) 0.70 s

b) 0.65 s

c) 0.50 s

d) 0.55 s

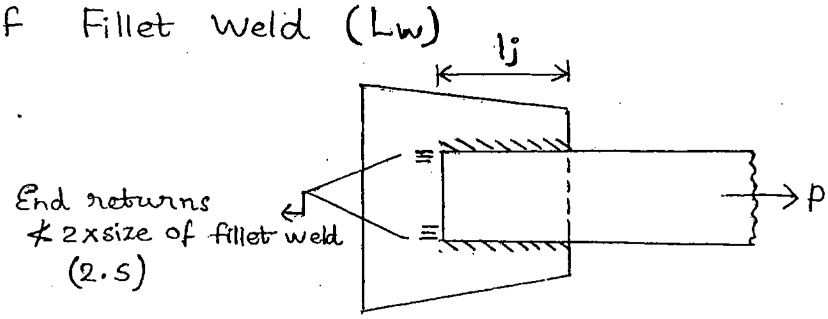
Ans : 0.65 s

- For $\alpha < 60^\circ$ & $\alpha > 120^\circ$, code does not recommend fillet weld. t_t will be very less when $\alpha > 120^\circ$ and it is very difficult to connect by fillet weld, when $\alpha < 60^\circ$.

→ Effective Length of Fillet Weld (L_w)

l_j = length of side fillet weld parallel to direction of load.

L_w = eff. length of fillet weld = $2 \cdot l_j$



- It is actual length of fillet weld shown on drawing
- End returns are provided to minimise the stress concentration due to non-uniform deformations along the length of weld.

when $l_j < 150\, t_t$, uniform shear of $\frac{P}{L_w \cdot t_t}$ occurs

But when $l_j > 150\, t_t$, stress concentration occurs near the end. So end returns are provided.

∴ in usual practise,

- Length of weld = eff. length of fillet weld + end return

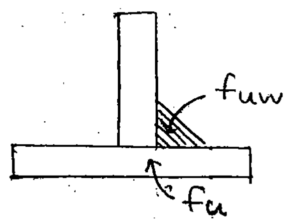
ie $L = L_w + 2s$; $s \rightarrow$ size of fillet weld.

- $L_w \nless 4s$ or 40 mm, whichever is more.

→ Design shear strength of Fillet Weld (P_{dw})

P_{dw} = effective sectional area * design shear capacity of fillet weld (f_{wd}).

$$f_{wd} = \frac{\text{Nominal shear capacity of fillet } (f_{wn})}{\text{Partial safety factor}}$$



$$f_{wd} = \frac{f_{wn}}{\gamma_{mw}}$$

$$f_{wn} = \frac{f_{u1}}{\sqrt{3}} \quad ; \quad f_{u1} = \min \text{ of } \begin{cases} f_u \\ f_{uw} \end{cases}$$

$$\Rightarrow \boxed{f_{wd} = \frac{f_{u1}}{\sqrt{3}\, \gamma_{mw}}}$$

$$P_{dw} = L_w \cdot t_t \cdot \frac{f_{u1}}{\sqrt{3}\, \gamma_{mw}}$$

$\gamma_{mw} = 1.25$ (workshop welding)

$= 1.5$ (site or field welding)

$$\boxed{P_{dw} = L_w \cdot K s \cdot \frac{f_{u1}}{\sqrt{3}\, \gamma_{mw}}}$$

– For safety criteria:

Design action (P) $\leqslant$ Design shear strength of Fillet weld (P_{dw})

→ Reduction factor for Long Joint (β_{lw})

{when $l_j > 150 t_t$}

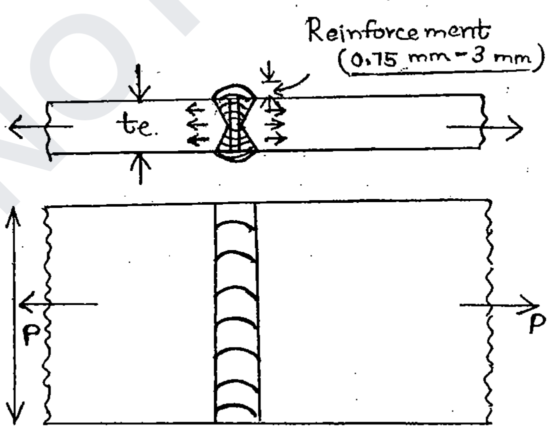
$$\beta_{lw} = \left[1.2 - \frac{0.2\, l_j}{150\, t_t}\right] \leqslant 1$$

→ Design of Butt or Groove Welds.

– Types of Butt Welds

1. Partially Penetrated or Single butt welds.

Eg: Single 'V' Butt welds.
Single 'U' Butt welds.
Single 'J' Butt welds.

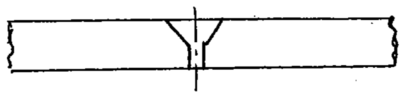
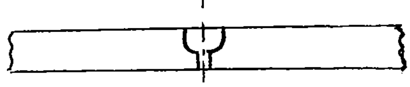
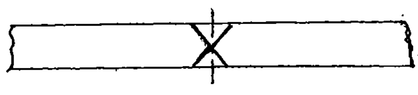
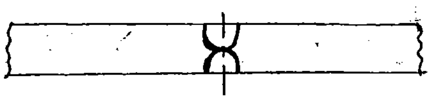


Butt Welds

2. Fully Penetrated or Double Butt Welds.

Eg: Double 'V' Butt welds.
Double 'U' Butt welds
Double 'J' Butt welds

▪ Reinforcement is provided by the welder at the site It's not part of the design

Type of Butt weld	Symbolic Representation	Symbol.	t_e
– single V butt weld.		$\widehat{V}$	$\frac{5}{8}t$
– single U butt weld.		$\widehat{U}$	$\frac{5}{8}t$
– double V butt weld.		$\widehat{X}$	t
– double U butt weld.		$\widehat{8}$	t

→ Design Axial Strength of Butt Weld (T_{dw})

T_{dw} = Effective sectional area × design axial stress

$$T_{dw} = L_w \cdot t_e \cdot \frac{f_{y1}}{\gamma_{mw}}$$

t_e → effective throat thickness of butt weld.

$t_e = \frac{5}{8}t$ (for single butt welds)

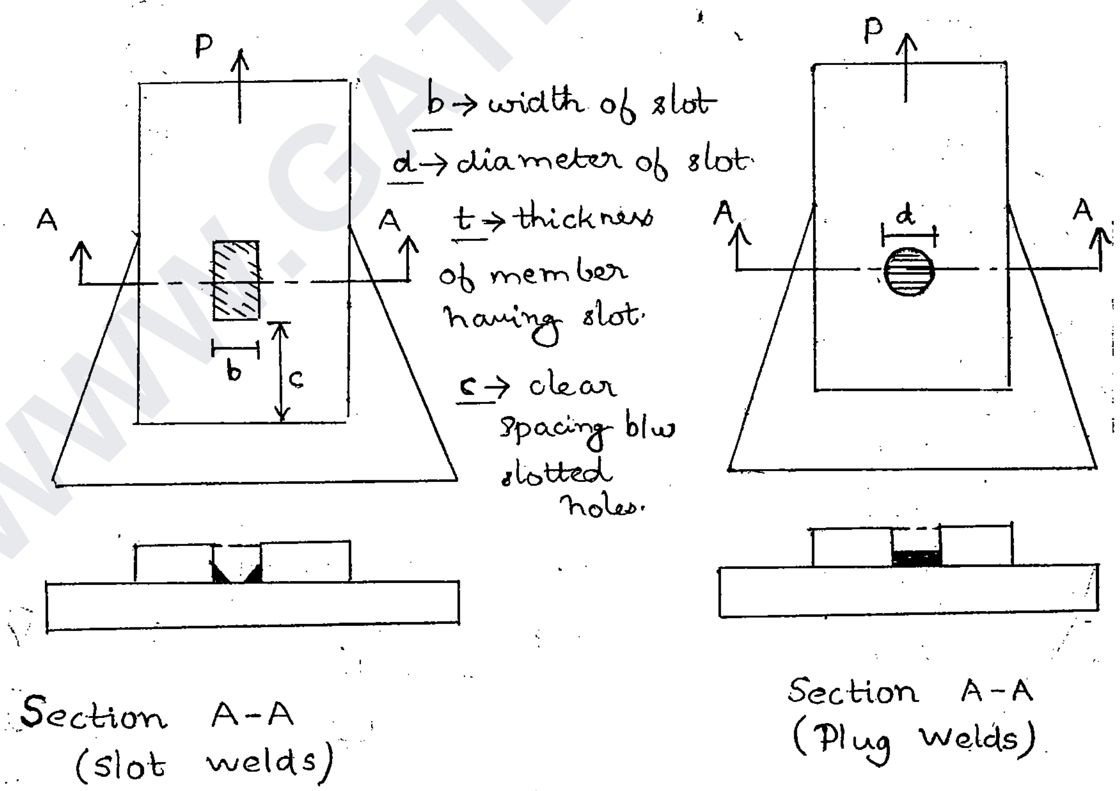
$= t$ (for double butt welds)

t : thickness of thinner connected member

L_w: effective length of butt weld.

f_{y1} : minimum of f_y & f_{yw}

th Sept, SATURDAY → Slot & Plug Welds



- Width of slot or Diameter of slot ≥ 3t & 25 mm
- Clear spacing b/w slots ≥ 2t & 25 mm
- Corner radius of slotted hole, R ≥ 1.5t & 25mm

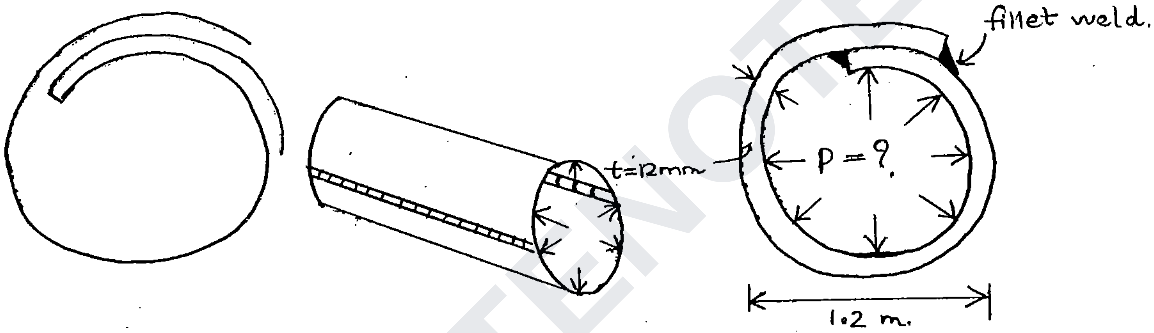
Q.1. Design shear capacity $= f_{wd} = \dfrac{f_{wn}}{\gamma_{mw}} = \dfrac{f_u}{\sqrt{3}\cdot\gamma_{mw}}$

$$= \frac{410}{\sqrt{3}\times 1.25} = \underline{\underline{189.37}}\ \text{N/mm}^2$$

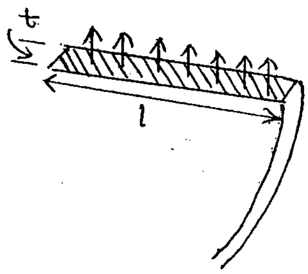
Q.2 Size of weld, $s = 8$ mm.

For grade Fe 410, $f_y = 250$ MPa, $f_u = 410$ MPa.

For workshop weld, $\gamma_{mw} = 1.25$



Design hoop stress, $f_h = \dfrac{Pd}{2t}$



Design hoop force, $F_h = f_h \times \text{area}$

$$= \frac{Pd}{2t}\times l \times t = \frac{Pdl}{2}$$

where $P \rightarrow$ design internal fluid pressure

$= \gamma_L \times$ safe internal fluid pressure

Design shear strength of fillet weld ; $P_{dw} = \left(L_w \cdot t_t \cdot \dfrac{f_u}{\sqrt{3}\,\gamma_{mw}}\right)$

$$F_h = P_{dw}$$

$$\frac{Pd}{2} = t_t \cdot \frac{f_u}{\sqrt{3}\cdot\gamma_{mw}} \times 2$$

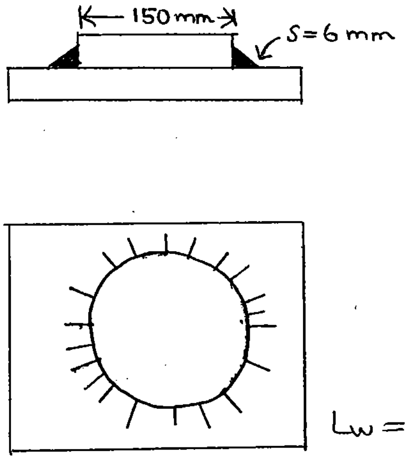
$$\frac{P\times 1200}{2} = 0.7\times 8 \times \frac{410}{\sqrt{3}\times 1.25}\times 2$$

$$P = 3.55\ \text{N/mm}^2$$

Safe internal fluid pressure (working internal fluid pressure

$$= \frac{3.55}{\gamma_L} = \frac{3.55}{1.5} = \underline{\underline{2.36}}\ N/mm^2$$

03.



$L_w = \pi d$

For Fe 410 grade steel, $f_u = 410$ MPa

For workshop welding, $\gamma_{mw} = 1.25$

Size of fillet weld, $S = 6$ mm

Eff. throat thickness, $t_t = KS$

$= 0.7 \times 6$

$= 4.2$ mm.

Design shear strength of fillet, $P_{dw} = L_w \cdot t_t \cdot \frac{f_u}{\sqrt{3}\,\gamma_{mw}}$

Ultimate twisting moment resistance, $T_u = P_{dw} \cdot \frac{d}{2}$

$$= L_w \cdot t_t \cdot \frac{f_u}{\sqrt{3}\,\gamma_{mw}} \cdot \frac{d}{2}$$

$$= \pi \times 150 \times 4.2 \times \frac{410}{\sqrt{3} \times 1.25} \times \frac{150}{2}$$

$$= \underline{\underline{28.11}}\ kNm$$

04 Single butt weld, $t_e = \frac{5}{8} t$

$$= \frac{5}{8} \times 12 = \underline{\underline{7.5}}\ mm$$

Design strength of butt weld, $T_{dw} = (L_w t_e) \cdot \frac{f_y}{\gamma_{mw}}$

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

$$= \underline{\underline{330\ kN}}$$

5 . For Fe 410 grade steel, $f_u = 410$ MPa, $f_y = 250$ MPa, $\gamma_{mw} = 1.25$

Design load or Factored load $= \gamma_L \times$ Service load

$= \gamma_L \times$ working load

$= \gamma_L \times$ characteristic load.

Size of weld = 3 mm.

Effective throat thickness, $t_t = K \cdot S = 0.7 \times 3$

$= 2.1$ mm $\not< 3$ mm

Design load (P) = Design strength of fillet weld. (P_{dw})

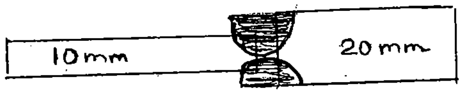
$$= L_w \cdot t_t \cdot \frac{f_u}{\sqrt{3}\,\gamma_{mw}} = \frac{3 \times 150 \times 3 \times 410}{\sqrt{3} \times 1.25}$$

$$= 255 \text{ kN}$$

Service load $= \dfrac{P}{\gamma_L} = \dfrac{255}{1.5} = \underline{\underline{170.43}}$ kN

7 $f_y = 250$ MPa,

6. $f_u = 410$ MPa, $\gamma_{mw} = 1.25$



$T \leq T_{dw}$

Design load, P = Design axial strength of butt weld (T_{dw})

$$T_{dw} = t_e \times L_w \times \frac{f_y}{\gamma_{mw}}$$

$$= T \times L_w \times \frac{f_y}{\gamma_{mw}}$$

$$= 10 \times 300 \times \frac{250}{1.25} = 600 \text{ kN}$$

Safe load allowed, $P_s = \dfrac{P}{\gamma_L} = \dfrac{600}{1.5} = \underline{\underline{400}}$ kN

8.

	S_{min}
0–10 mm	3 mm
10–20 mm	5mm

$\therefore$ Min. size of weld, $S_{min} = 5$ mm.

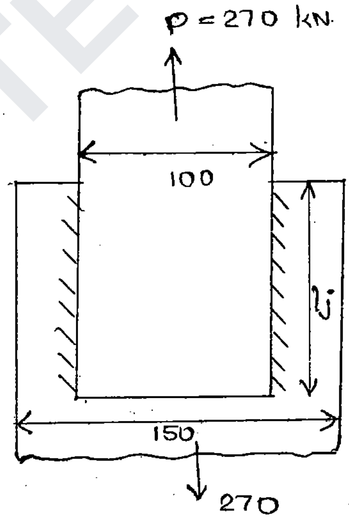
$$t_t = 0.7 \times 5 = 3.5 \text{ mm} > 3 \text{ mm}.$$

By equating $P = P_{dw}$

$$10^3 \times 300 = L_w t_t \cdot \frac{f_u}{\sqrt{3}\,\gamma_{mw}}$$

$$L_w = \frac{300 \times 10^3 \times \sqrt{3} \times 1.25}{3.5 \times 410} = \underline{\underline{452.62}} \text{ mm}$$

9. Two plates are connected by fillet weld of size 10 mm and subj. to tension as shown in the fig. Thickness of each plate is 12 mm. f_y & f_u of steel are 250 MPa & 410 MPa resptly. The weld is done in workshop ($\gamma_{mw} = 1.25$). As per limit state method of IS 800:2007, the min length (rounded off to nearest higher multiple of 5mm) of each weld to transmit a load of P = 270 kN



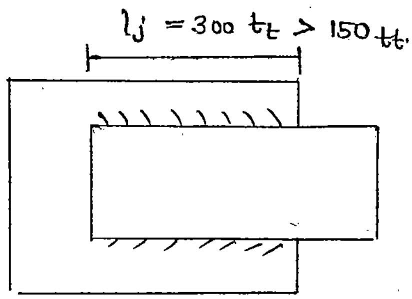
$S = 10$ mm

$$P = P_{dw}$$

$$270 \times 10^3 = L_w \cdot t_t \cdot \frac{f_u}{\sqrt{3}\,\gamma_{mw}}$$

$$= 2 \times l_j \times 0.7 \times 10 \times \frac{410}{\sqrt{3} \times 1.25}$$

$$\therefore l_j = 101.8 \text{ mm} \approx \underline{\underline{105 \text{ mm}}}$$



10. Long joint, $l_j > 150 t_t$.

So $\beta_{lw} = \left(1.2 - \frac{0.2\, l_j}{150 t_t}\right)$

$$= \left(1.2 - \frac{0.2 \times 300 t_t}{150\, t_t}\right) = \underline{\underline{0.8}}$$

Design shear capacity reduced by $\underline{\underline{20\%}}$

11 $P \le P_{dw}$

$$P_{dw} = L_w \cdot t_t \cdot \frac{f_u}{\sqrt{3}\cdot \gamma_{mw}} \times \beta_{lw}$$

$$= 1200 \times 3.5 \times \frac{410}{\sqrt{3}\cdot \gamma_{mw}} \times \beta_{lw}$$

$$= \underline{\underline{771.49}} \text{ kN}$$

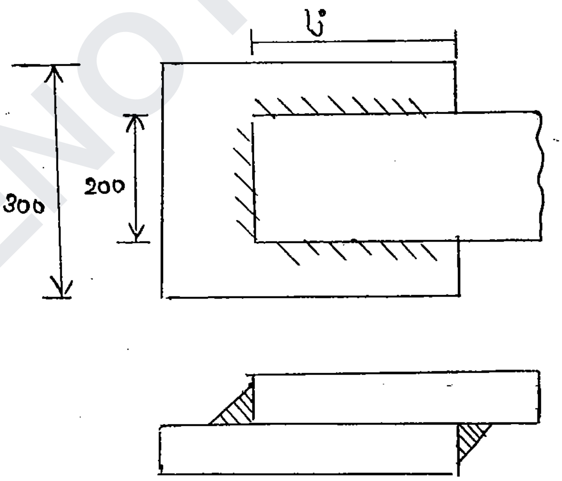
$l_j = 600$

$150 t_t = 150 \times 3.5 = 525$

$l_j > 150 t_t$

$$\beta_{lw} = 1.2 - \frac{0.2\, l_j}{150 t_t}$$

$$= 1.2 - \frac{0.2 \times 600}{525} = \underline{\underline{0.971}}$$



12. $P = P_{dw}$

$$600 \times 10^3 = L_w \times K \cdot S_{max} \frac{f_u}{\sqrt{3}\, \gamma_{mw}}$$

Max size of fillet weld,

$S_{max} = 10 - 1.5 = 8.5$ mm.

$$\Rightarrow 600 \times 10^3 = L_w \times 0.7 \times 8.5 \times \frac{410}{\sqrt{3} \times 1.25}$$

$L_w = 532.502$ mm

$L_w = 2 \times 200 + 2\, l_j$

$l_j = 66.25 \approx 70$ mm $>$ 40mm (min. overlap)

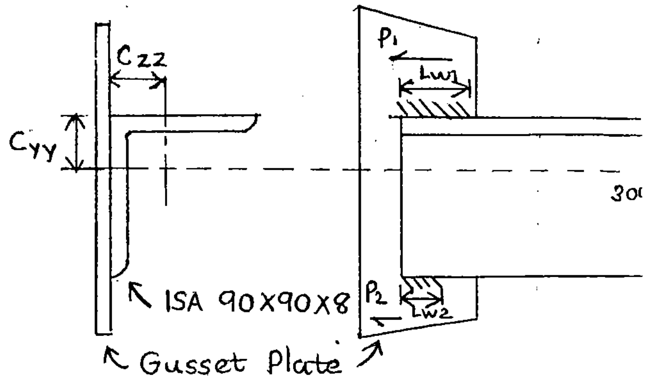
Sept, DAY

13 Fe 410 grade steel : $f_u = 410$ MPa, $f_y = 250$ MPa.

For site welding, $\gamma_{mw} = 1.5$

Max size of fillet weld for round edges, $S_{max} = \frac{3}{4}t$.

$$= \frac{3}{4} \times 8 = \underline{\underline{6 \text{ mm}}}$$



Effective throat thickness,

$$t_t = K.S.$$
$$= 0.7 \times 6 = 4.2 \text{ mm.}$$

$$P = P_{dw}$$

$$300 \times 10^3 = L_w \cdot t_t \cdot \frac{f_u}{\sqrt{3}\,\gamma_{mw}}$$

$$= L_w \cdot 4.2 \cdot \frac{410}{\sqrt{3} \times 1.50}$$

$$L_w = 452.6 \text{ mm}.$$

Let L_{w1} & L_{w2} are lengths of top weld and bottom weld respectively.

$$L_{w1} + L_{w2} = 452.6 \text{ mm} \longrightarrow (1)$$

Consider moments of strength of weld and loads on top edge of an angle.

$$P_2 \times 90 + P_1 \times 0 - 300 \times 25.1 = 0.$$

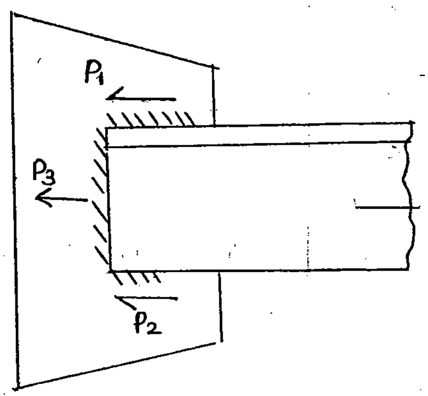
$$L_{w2}\, t_t \cdot \frac{f_u}{\sqrt{3}\,\gamma_{mw}} \times 90 + 0 - 300 \times 10^3 \times 25.1 = 0.$$

$$L_{w2} = 126.23 \text{ mm}$$

$$\therefore \quad L_{w1} = \underline{\underline{326.37}} \text{ mm.}$$

14. $L_{w1} + L_{w2} + 90 = 452.66$

$$L_{w1} + L_{w2} = 362.6 \longrightarrow (1)$$



$$P_2 \times 90 + P_3 \times \frac{90}{2} + P_1 \times 0 = 300 \times 10^3 \times 25.1$$

$$L_{w2} \cdot 4.2 \times \frac{410}{\sqrt{3} \cdot 1.5} + 90 \times 4.2 \times \frac{410}{\sqrt{3} \times 1.5} \times \frac{90}{2} + 0 = 300 \times 10^3 \times 25.1$$

$L_{w2} = 81.2 \text{ mm}$

$L_{w1} = 362.6 - 81.2 = \underline{\underline{281.4}} \text{ mm}$

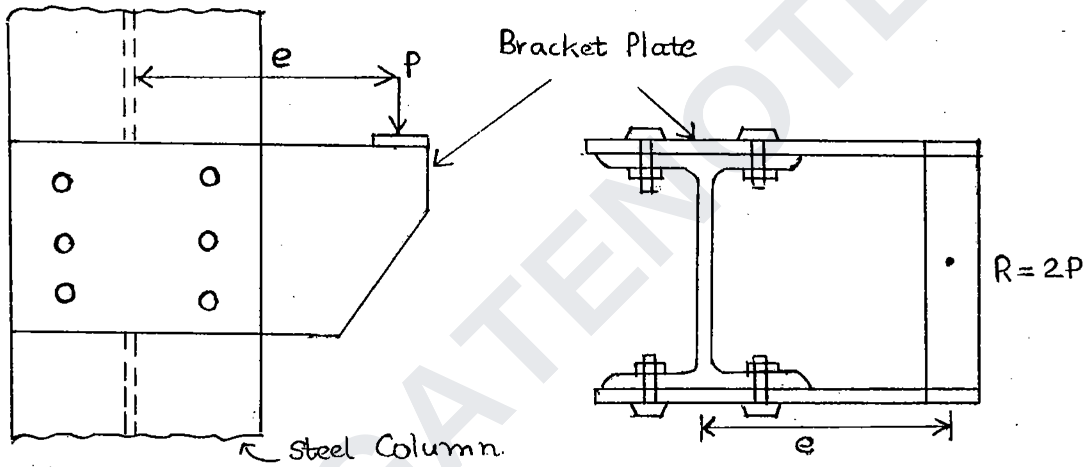
7th Sept,
SUNDAY

4. ECCENTRIC CONNECTIONS

→ ECCENTRIC BOLTED CONNECTIONS

1. Bracket Type Connection - I

- When load or moment is lying in the plane of bearing type bolt group.



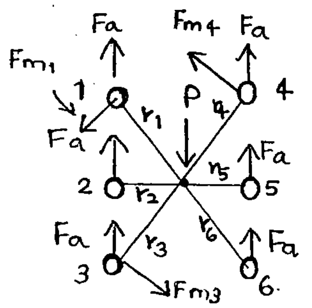
P → Factored load or Design Load.

e → eccentricity of the load. (Distance from CG of Bolt / weld group to applied load line)

- Bolt group is subjected to :
  (i) Direct concentric load (P)
  (ii) Twisting moment ($M = Pe$) [in-plane moment]

- Vertical SF in each bolt due to P is Fa

$$F_a = \frac{P}{n} \quad (n = \text{no. of bolts in bolt group}$$



$$\Rightarrow \frac{T}{J} = \frac{f_s}{r} \Rightarrow f_s = \frac{T}{I_p} \cdot r$$

$$f_s = \frac{M}{I_p} \cdot r$$

$$F_m = f_s A = \frac{M}{I_p} \cdot r \cdot A \Rightarrow F_m \propto r$$

Fm is force any bolt due to twisting moment $(M = Pe)$.

$$F_m \propto r$$

$$F_m = Kr \quad (K = \text{Elastic Constant})$$

Moment of resistance capacity of each bolt,

$$M_R = F_m \cdot r = K \cdot r \cdot r$$

$$M_R = Kr^2$$

Total moment of resistance capacity, $M_R = \Sigma Kr^2 = K\,\Sigma r^2$

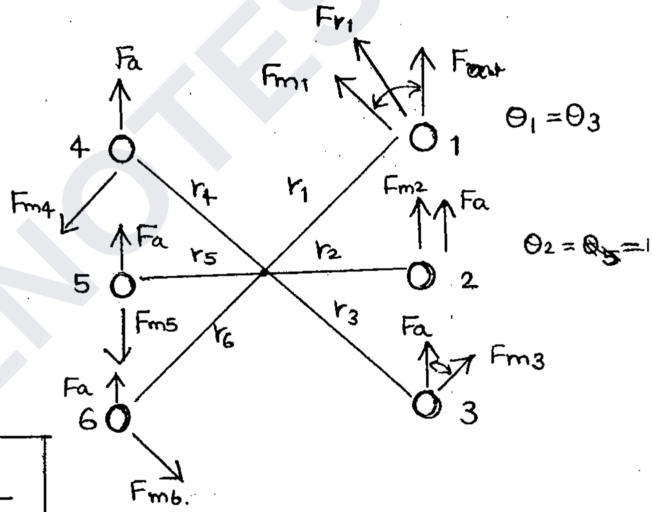
$$M_R = K \cdot \Sigma r^2$$

$$= \frac{F_m}{r} \Sigma r^2$$

$$M = M_R$$

$$Pe = \frac{F_m}{r} \Sigma r^2$$

$$\therefore \boxed{F_m = \frac{Mr}{\Sigma r^2} = \frac{Pe\,r}{\Sigma r^2}}$$



– Resultant force b/w Fa & Fm is F_R

$$F_R = \sqrt{F_a^2 + F_m^2 + 2F_a \cdot F_m \cos\theta}$$

$$F_a \downarrow \frac{P}{r} \qquad F_m \downarrow \frac{Mr}{\Sigma r^2}$$

* Conditions for Max. Resultant Force for Critical bo

$r \longrightarrow$ maximum (1, 3, 4, 6)

$\theta \longrightarrow$ minimum. (1, 3)

Bolt which is closer to resultant loading – critical bolt (1 & 3)

* For safety of Bracket Connection –

$$(F_R)_{max} \leq V_{db}$$

V_{db} = design strength of one bolt = minimum of $\begin{cases} V_{dsb} \\ V_{dpb} \\ T_{db} \text{ (if exist)} \end{cases}$

F_R is max. for critical bolt.

* Critical Bolt

Critical bolt is one at which the resultant SF is maximum, which is farthest from CG of bolt group and which may be close to the applied load line.

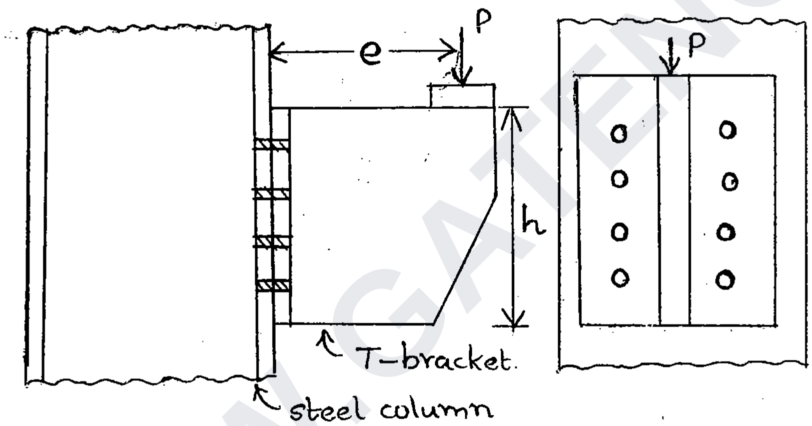
2. Bracket Type Connection - II

When load or moment is not lying in the plane of bearing type bolt group.

Bolt group is subjected to:

(i) Direct Concentric Load (P) (shear)

(ii) Bending moment ($M = Pe$) (tensile)



h → depth of bracke

- Vertical SF in each bolt due to P is V_b

$$V_b = \frac{P}{n} \quad (n = \text{no. of bolts present})$$

$$\Rightarrow \frac{M}{I} = \frac{f}{y}$$

$$f = \frac{M}{I} y$$

$$T_b = f.A = \frac{M}{I} y . A.$$

$$\boxed{T_b \propto y_i}$$

T_b is tensile force in each bolt due to BM

$$T_{bi} = k y_i \quad ; (k = \text{elastic constant})$$

M_R of i^{th} bolt $= T_{bi} \times y_i$

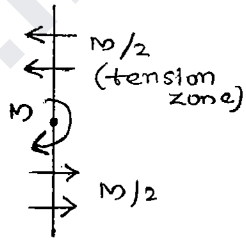
$$= K \cdot y_i \cdot y_i = K y_i^2$$

Total M_R of all bolts in tension zone, $M_R = \Sigma K y_i^2 = K \, \Sigma y_i^2$

$$M_R = \frac{T_{bi}}{y_i} \Sigma y_i^2$$

Assume axis of rotation is $\frac{h}{2}$ from top edge of T bracket

$$M' = M'' = M/2$$



$$M' = M_R$$

$$= \frac{T_{bi}}{y_i} \Sigma y_i^2$$

$$T_{bi} = \frac{M' y_i}{\Sigma y_i^2}$$

Tensile force in extreme bolt due to M is T_b,

$$\boxed{T_b = \frac{M' y_n}{\Sigma y_i^2}} \quad \text{(TENSION)}$$

Shear force in each bolt due to P is V_b,

$$\boxed{V_b = \frac{P}{n}} \quad \text{(SHEAR)}$$

- For safety of connection, interaction equation must be satisfi as per IS 800 : 2007.

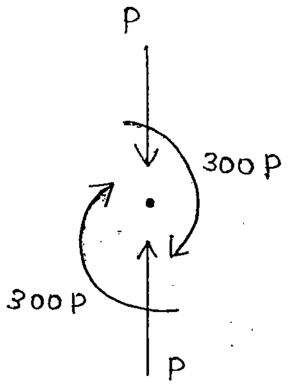
$$\boxed{\left(\frac{V_b}{V_{db}}\right)^2 + \left(\frac{T_b}{T_{db}}\right)^2 \leq 1.0}$$

$V_{db} \rightarrow$ design shear capacity of bolt

$T_{db} \rightarrow$ design tensile capacity of bolt

P-33.

1.



$$F_a = \frac{P}{n} = \frac{0}{4} = \underline{\underline{0}}$$

$$F_m = \frac{Mr}{\Sigma r^2}$$

r_{max} (1, 2, 3, 4)
θ_{min} (1, 2, 3, 4).

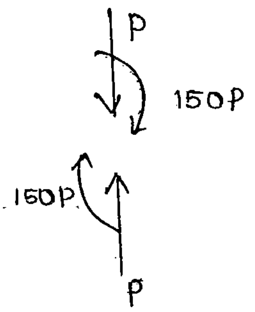
$$= \frac{600P \times 150}{4 \times 150^2}$$

$$= P.$$

$$(F_R)_{max} = \left.\begin{matrix} F_{R1} \\ F_{R2} \\ F_{R3} \\ F_{R4} \end{matrix}\right\} = \sqrt{\cancel{F_a^2}^{\,0} + F_m^2 + 2\cancel{F_a}^{\,0} F_m \cos\theta} = F_m$$

$$= \underline{\underline{P}}$$

2.



$$F_a = \frac{P}{n} = \frac{0}{4} = \underline{\underline{0}}$$

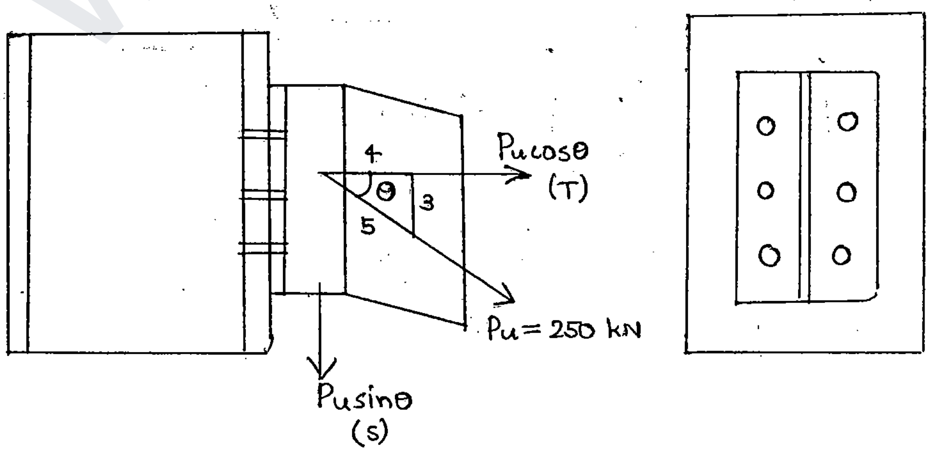
$$F_m = \frac{Mr}{\Sigma r^2}$$

$$r = \sqrt{80^2 + 60^2} = 100 \text{ mm.}$$

$$\therefore\ F_m = \frac{300P \times 100}{4 \times 100^2} = \frac{3}{4} P.$$

$$(F_R)_{max} = F_m = \underline{\underline{\frac{3}{4} P}}$$

GATE 2014: Calculate SF and Tensile force (Both in kN) at bolted connection shown in fig below.

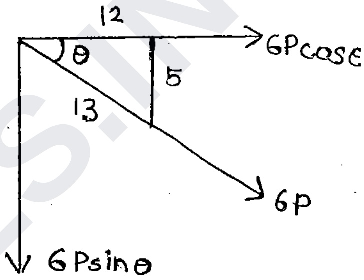


SF in each bolt due to $P_u \sin\theta = \frac{P_u \sin\theta}{n}$ $\qquad \sin\theta = \frac{3}{5}$

$$= \frac{250}{6} \times \frac{3}{5} = \underline{\underline{25 \text{ kN}}}$$

TF in each bolt due to $P_u \cos\theta = \frac{250}{6} \times \frac{4}{5} = 33.33$

03. $V_{db} = 45$ kN ; $T_{db} = 36$ kN



Tensile force in each bolt due to

$$6P\cos\theta = T_b$$

$$= \frac{6P\cos\theta}{n} = \frac{6P}{6} \times \frac{12}{13} = \underline{\underline{\frac{12P}{13}}}$$

SF in each bolt due to $6P\sin\theta$,

$$V_b = \frac{6P\sin\theta}{n} = P\sin\theta = \underline{\underline{\frac{5P}{13}}}$$

Interaction equation as per IS 800 : 2007,

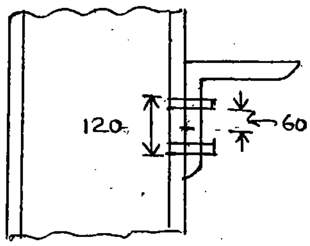
$$\left(\frac{V_b}{V_{db}}\right)^2 + \left(\frac{T_b}{T_{db}}\right)^2 \leq 1$$

$$\left(\frac{5P}{13} \cdot \frac{1}{45}\right)^2 + \left(\frac{12P}{13} \cdot \frac{1}{36}\right)^2 \leq 1$$

$$\left(\frac{P}{117}\right)^2 + \underline{\underline{\left(\frac{P}{39}\right)^2}} \leq 1.0$$

04. SF in each bolt due to P is $V_b = \frac{P}{n} = \frac{P}{4}$.

TF in any bolt due to M is $T_b = \frac{M' y_n}{\Sigma y_i^2}$



$$M = P \times e = 150P$$

$$M' = M'' = \frac{M}{2} = 75P \text{ kN mm}$$

$$\therefore T_b = \frac{75P \times 60}{2 \times 60^2} = \frac{5P}{8}$$

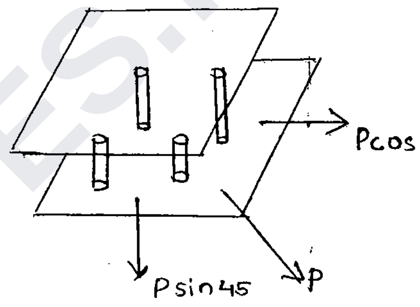
$$\left(\frac{V_b}{V_{db}}\right)^2 + \left(\frac{T_b}{T_{db}}\right)^2 \leqslant 1.0$$

$$\left(\frac{P}{4}\cdot\frac{1}{20}\right)^2 + \left(\frac{5P}{8},\frac{1}{15}\right)^2 \leqslant 1.0$$

$$\left(\frac{P}{80}\right)^2 + \left(\frac{P}{24}\right)^2 \leqslant 1.0$$

5. SF, in each bolt due $P\cos 45$,

$$V_b = \frac{P\cos 45}{n} = \frac{P}{4\sqrt{2}}$$



TF in each bolt due to $P\sin 45$,

$$T_b = \frac{P\sin 45}{n} = \frac{P}{4\sqrt{2}}.$$

For safety of connection, interaction eqn as per IS 800:2007

$$\left(\frac{V_b}{V_{db}}\right)^2 + \left(\frac{T_b}{T_{db}}\right)^2 \leqslant 1.$$

$$\left(\frac{P}{4\sqrt{2}}\times\frac{1}{30}\right)^2 + \left(\frac{P}{4\sqrt{2}}\cdot\frac{1}{40}\right)^2 = 1 \quad \text{(for max. value of}$$

$$P = \underline{\underline{135.76}}\ \text{kN}$$

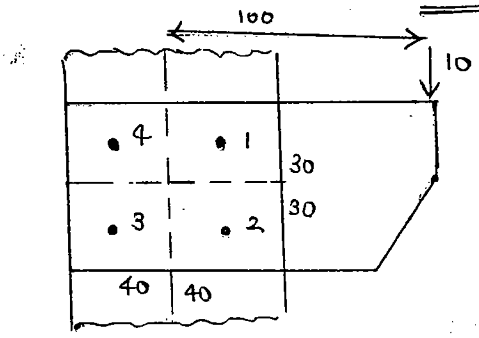
6. SF in any bolt due to M is Fm.

$$Fm_A = \frac{M r_A}{\Sigma r^2} = \frac{80\times10^3\times100}{4\times100^2} \qquad r_A = \sqrt{\left(\frac{160}{2}\right)^2 + \left(\frac{120}{2}\right)^2}$$

$$= 100 \text{ mm.}$$

$$= \underline{\underline{200\ \text{kN}}}$$

07.



$r_{max} = 1, 2, 3, 4$

$\theta_{min} = 1 \,\&\, 2$

Critical bolts : 1 & 2

$$P(e) \longrightarrow \left[\begin{array}{l} P = 10 \text{ kN} \longrightarrow F_a \\ M = Pe = 1000 \longrightarrow F_m \end{array}\right\} \rightarrow F_R$$

Vertical SF in each bolt due to $P = 10$ kN. is F_a

$$F_a = \frac{P}{n} = \frac{10}{4} = 2.5 \text{ kN}$$

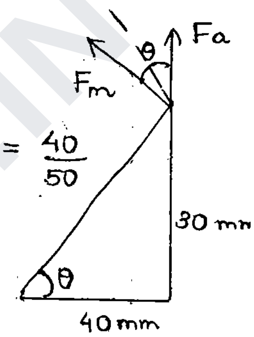
SF in any bolt due to M (1000 kNmm) is F_m.

$$F_m = \frac{M r_1}{\Sigma r^2} = \frac{1000 \times 50}{4 \times 50^2} = \underline{\underline{5 \text{ kN}}} \qquad r_1 = \sqrt{30^2 + 40^2} = 50$$

$(F_R)_{max} = F_{R1} = F_{R2}$.

$$= \sqrt{F_a^2 + F_m^2 + 2 F_a . F_m . \cos\theta} \qquad \cos\theta = \frac{40}{50}$$

$$= \sqrt{2.5^2 + 5^2 + 2 \times 2.5 \times 0.8} = \underline{\underline{7.16 \text{ kN}}}$$



8. For grade Fe 410 steel,

$f_u = 410$ MPa.

For bolt., 4.6 grade, $f_{ub} = 400$ MPa.

$d = 24$ mm

$P = 40$ kN

Critical bolt is bolt no: 4.

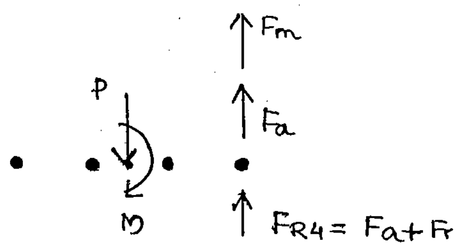
$P = 40$ kN.

$M = Pe$.

$e = 200 + \frac{220}{2} = 310$ mm.

$M = 40 \times 310 = 12400$ kNm.

$$F_a = \frac{P}{n} = \frac{40}{4} = 10 \text{ kN}$$



$$F_m = \frac{M r_4}{\Sigma r^2} = \frac{12400 \times \frac{220}{2}}{2 \times \left(\frac{220}{2}\right)^2 + 2 \times \left(\frac{140}{2}\right)^2} = 40.11 \text{ kN}$$

Max. resultant force $= (F_R)_{max} = F_{R4}$

$$= F_a + F_m = 10 + 40.11 = \underline{\underline{50.11 \text{ kN}}}$$

For safety of connection,

$$(F_R)_{max} \leq V_{db}.$$

V_{db} = lesser of V_{dsb} or V_{dpb}.

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

$$= \frac{400}{\sqrt{3} \times 1.25}\left(1 \times 1 \times 0.78 \times \frac{\pi}{4} \times 24^2 + 0\right) = \underline{\underline{65.18}} \text{ kN}$$

$$V_{dpb} = 2.5\, d t f_{ub} \cdot \frac{K_b}{\gamma_{mb}}$$

$$= 2.5 \times 24 \times 8 \times \frac{400 \times 0.26}{1.25}$$

$$= \underline{\underline{38}}$$

K_b is min of :

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

$$\frac{p}{3d_0} - 0.25 = 0.51 - 0.25 = 0.26$$

$$\frac{f_{ub}}{f_u} = \frac{400}{410} = 0.97$$

$$\underline{\underline{V_{db} = 38 \text{ kN}}}$$

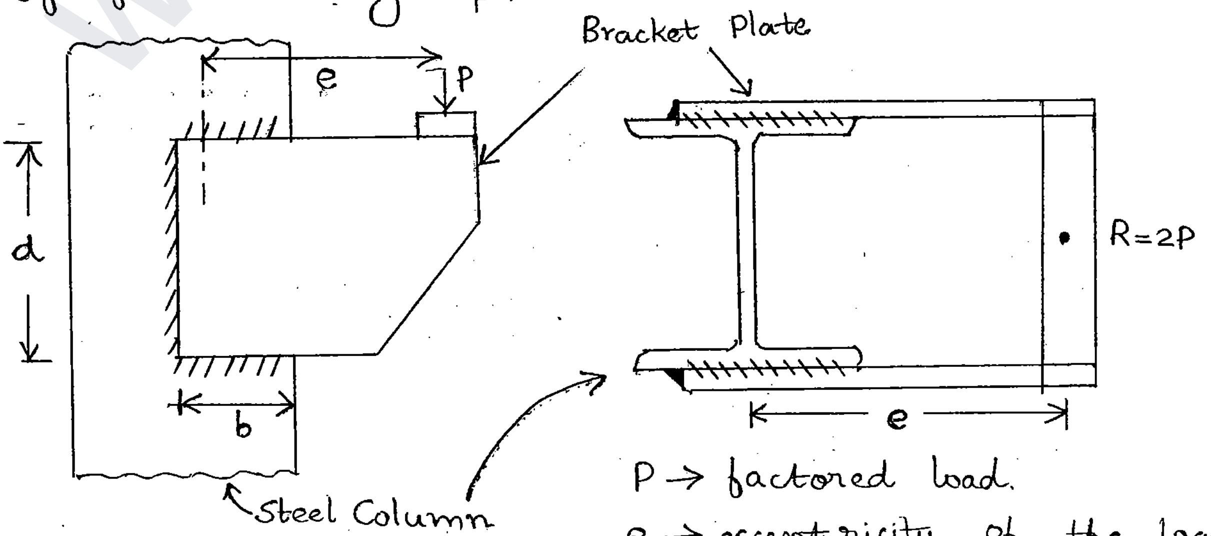
$(F_R)_{max} > V_{db}$ $\Rightarrow$ hence connection is unsafe.

→ ECCENTRIC WELDED CONNECTIONS

1. Fillet Welds

(i) Bracket Type Connection - I

When load or moment is lying in the plane of fillet weld group.



P → factored load.

e → eccentricity of the load

d → depth of bracket plate

- Vertical shear stress in weld due to P is q_1.

$$q_1 = \frac{P}{\text{Effective sectional area.}}$$

$$q_1 = \frac{P}{L_w \cdot t_t}$$

L_w = effective length of fillet weld $= (d+2b)$

t_t = Effective throat thickness $= K.S.$

S = size of fillet weld.

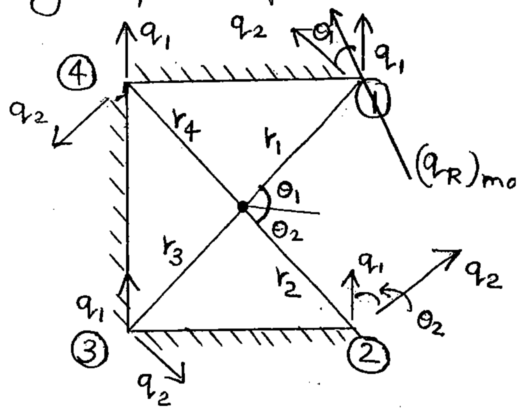
- Shear stress in weld due to twisting moment (M) is q_2

$$q_2 = \frac{M}{I_p} \times r.$$

$M = Pe$

I_p = polar MI of fillet weld $= I_{zz} + I_{yy}$ of fillet weld.

r = radial distance from CG weld group to point on weld length.



* Resultant shear stress b/w q_1 & q_2 is q_R.

$$q_R = \sqrt{q_1^2 + q_2^2 + 2q_1 q_2 \cos\theta}$$

- condition for $(q_R)_{max}$:
  - $r \to$ maximum
  - $\theta \to$ minimum.

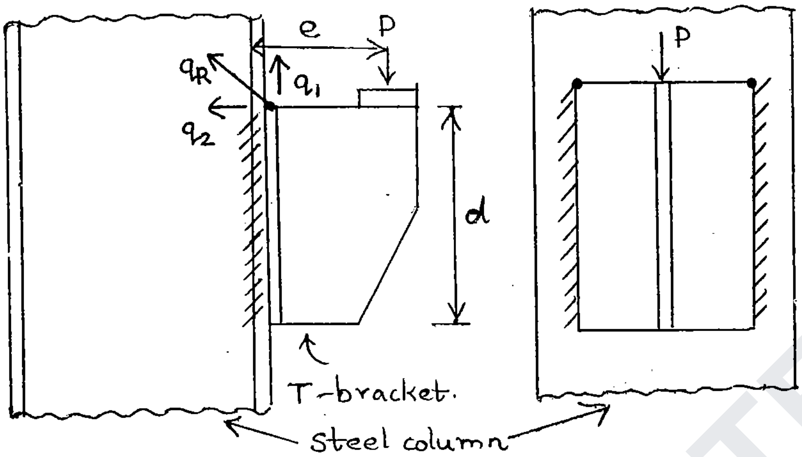
- for safety of fillet weld:

$(q_R)_{max} \leq f_{wd}$ = design shear capacity of fillet weld

$$f_{wd} = \frac{f_u}{\sqrt{3}}$$

(ii) Bracket Type Connection – II

When load or moment is <u>not</u> lying in the plane of fillet weld group.



Vertical shear stress in weld due to P is q_1,

$$q_1 = \frac{P}{\text{Eff. sectional area}} = \frac{P}{L_w \cdot t_t}$$

$L_w \rightarrow$ Effective length of fillet weld $= 2d$

$t_t \rightarrow$ Effective throat thickness $= K \cdot S$

$S \rightarrow$ size of fillet weld.

Stress in weld due to M is q_2,

$$q_2 = \frac{M}{I} y.$$

$M = Pe$

$I =$ MI of fillet weld about bending axis.

$$= \frac{t_t d^3}{12} \times 2 = \frac{t_t d^3}{6}$$

$y = \frac{d}{2}$ (for max q_2)

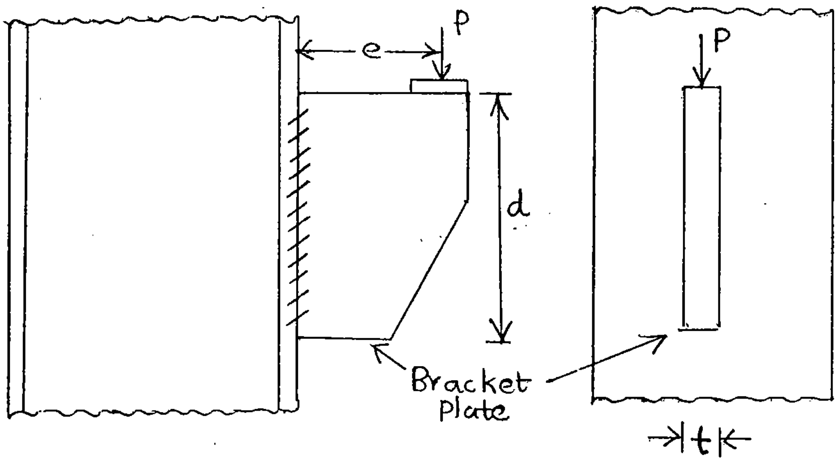
– Combined stress b/w q_1 & q_2 is q_R.

$$q_R = \sqrt{q_1^2 + q_2^2}$$

– Condition for Max. q_R : $y \rightarrow \max\left(\frac{d}{2}\right)$

– For safety of fillet weld : $(q_R)_{max} \leq f_{wd} = \frac{f_u}{\sqrt{3} \cdot \gamma_{mw}}$

2. Butt Weld is loaded eccentrically.



Butt weld is subjected:

(i) Direct concentric load (P)

(ii) Bending moment ($M = Pe$).

— Vertical shear stress in butt weld

$$q_{cal} = \frac{P}{\text{Eff. sectional area}} = \frac{P}{L_w \times t_e} = \frac{P}{d \times t}$$

L_w = eff. length of butt weld $= d$.

t_e = eff. throat thickness $= t$.

— Bending stress in weld due to M is f_{cal}.

$$f_{cal} = \frac{M}{I} \cdot y$$

$$M = P.e$$

$$y = \frac{d}{2}$$

$$I = \text{MI of butt weld} = \frac{t d^3}{12}.$$

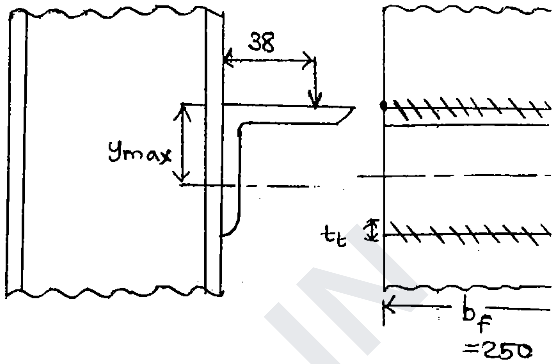
— Equivalent stress in weld, $f_e = \sqrt{3 q_{cal}^2 + f_{cal}^2}$

— For safety of butt weld:

$$(f_e)_{max} \leq \frac{f_y}{\gamma_{m0}} \quad (\text{design equivalent stress})$$

10. Let size of fillet weld is S.

For Fe 410 grade steel, $f_u = 410$ MPa



Vertical shear stress in weld due to P is q_1

$$q_1 = \frac{P}{L_w t_t} = \frac{75\times10^3}{2\times250\, t_t} = \frac{150}{t_t}\ \text{N/mm}^2.$$

Stress in weld due to M is q_2,

$$q_2 = \frac{M}{I}\cdot y$$

$$M = 75\times10^3\times38$$

$$y = \frac{d}{2} = \frac{125}{2} = 62.5\ \text{mm}$$

$$I_{zz} = \left(I_{CG} + Ay^2\right)2.$$

MI of fillet weld about bending axis,

$$I = 2\left(\frac{250\, t_t^3}{12} + 250\, t_t\times\left(\frac{125}{2}\right)^2\right)$$

Neglecting t_t^3 and t_t^2 terms,

$$I = 500\, t_t\times62.5^2\ \text{mm}^4.$$

$$q_2 = \frac{75\times10^3\times38}{500\, t_t\times62.5^2}\times62.5 = \frac{91.2}{t_t}\ \text{N/mm}^2.$$

Max. combined stress b/w q_1 & q_2 is $(q_R)_{max}$,

$$(q_R)_{max} = \sqrt{q_1^2+q_2^2}$$

$$= \sqrt{\left(\frac{150}{t_t}\right)^2 + \left(\frac{91.2}{t_t}\right)^2} = \frac{175.54}{t}\ \text{N/mm}^2$$

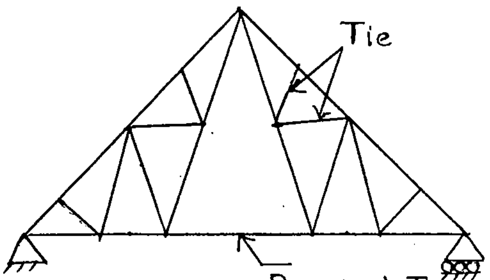
For safety of fillet weld,

$$(q_R)_{max} \leq f_{wd} = \frac{f_u}{\sqrt{3}\,\gamma_{mw}}$$

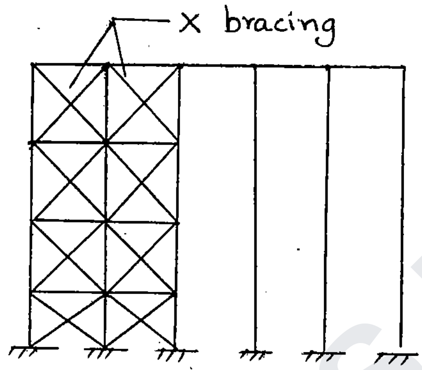
$$\frac{175.54}{t_t} = \frac{410}{\sqrt{3}\times1.50} \Rightarrow t_t = 1.18\ \text{mm}\ ;\ S = \frac{t_t}{0.7}$$

7th Sept,
TUESDAY

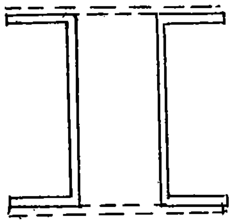
5. TENSION MEMBERS

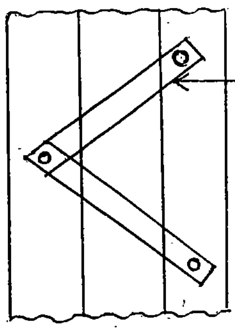


ROOF TRUSS

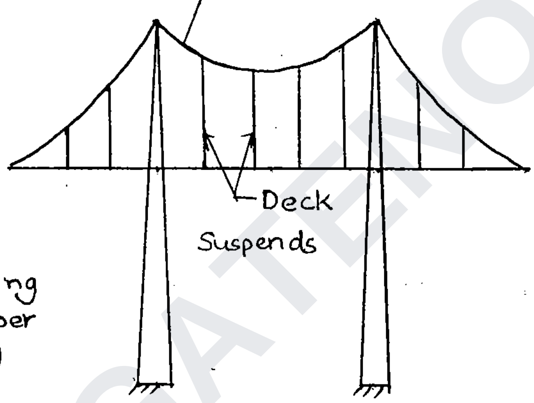


BRACED FRAME

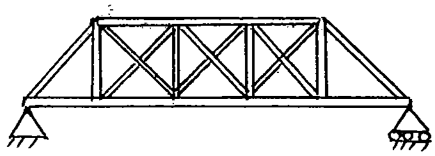




BUILT UP COLUMN WITH LACING SYSTEM



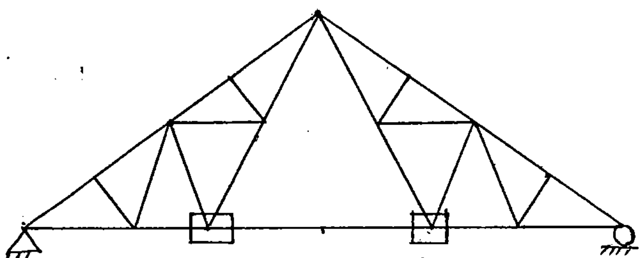
SUSPENSION BRIDGE.

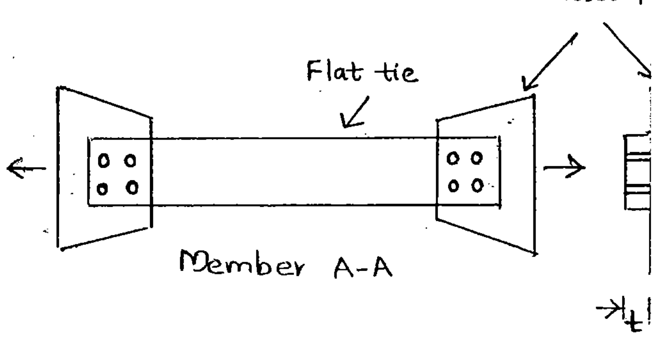


TRUSS BRIDGE

→ Types of Failures in a Tension Member:

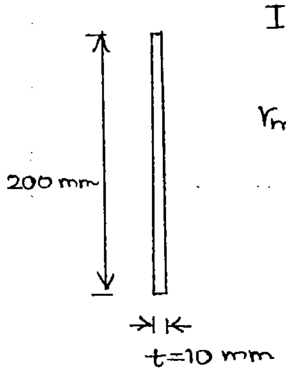
- Gross section yielding failure (T_{dg})
- Net section rupture (or) Fracture failure. (T_{dn})
- Block shear failure. (T_{db})





Member A-A

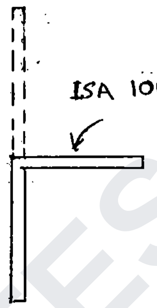
In olden constructions, flat members were used as tension members. Although flat members are strong in tension, they are weak in compression due to small value of radius of gyration. ($P_{cr} = \frac{\pi^2 EA}{(l/r)^2}$). So, angle sections now replace flat members as tension members.



$$I_{min} = \frac{bt^3}{12}$$

$$r_{min} = \sqrt{\frac{I_{min}}{A}} = \sqrt{\frac{bt^3}{12 \cdot bt}} = \frac{t}{\sqrt{2}}$$

$$= \frac{10}{\sqrt{2}} = 2.88 \text{ mm}.$$



$r_{min} = r_{vv} = 19.1$ mm

In the above eg, angle section with c/s area as that of a flat member has more radius of gyration.

In flat members, load reversals and bolt holes are th factors of failures. But in angle sections, along with these two factors, eccentricity of loading (causing moment) must also be taken into consideration.

→ Design Tensile Strength of a Tension Member (T_d)

T_d is minimum of T_{dg} or T_{dn} or T_{db}.

* Based on Gross Section Yielding Failure, (T_{dg})

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

A_g → gross sectional area o a member

f_y → yield strength of a material.

γ_{mo} → partial safety factor against yield stress (1.10)

* Based on Net Section Rupture (or) Fracture Failure (Tdn)

(i) For Plates and Flats.

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

$A_n \rightarrow$ net sectional area = gross sectional area − Area of bolt hole.

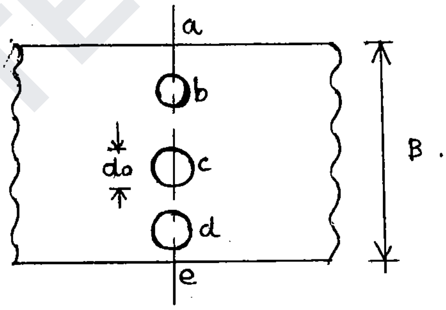
– For chain pattern of bolting:

A_n = A_g − area of bolt holes.

Along section, a−b−c−d−e:

$= Bt - n d_0 t$

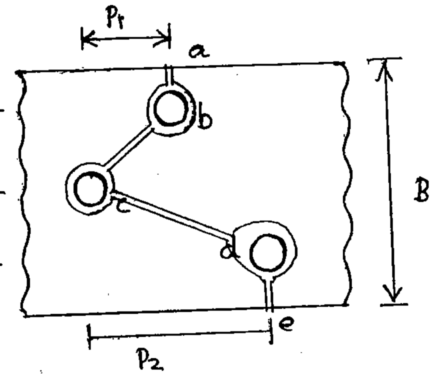
$A_n = (B - n d_0) t$



– For staggered (or) zig-zig pattern of bolting

Along section a−b−c−d−e,

$$A_n = (B - n d_0)t + \underbrace{\frac{P_1^2 t}{4g_1} + \frac{P_2^2 t}{4g_2}}_{\text{area connections}}$$



P_1 & P_2 $\rightarrow$ staggered pitches

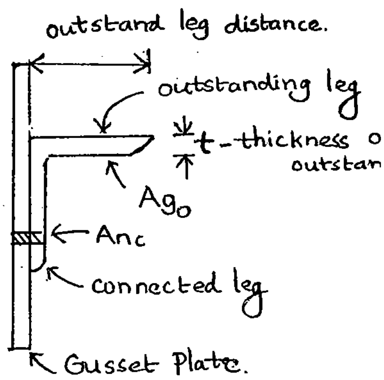
g_1 & g_2 $\rightarrow$ gauge distances.

▪ For n inclination, n number of area connections.

(ii) For Angle, Channel & Other types of Rolled Steel sections

A_{nc} = Net sectional area of connected leg

A_{go} = gross sectional area of outstanding leg.



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

unimportant for GATE

$$\beta = \left\{ 1.4 - 0.076 \left(\frac{w}{t}\right)\left(\frac{f_y}{f_u}\right)\left(\frac{b_s}{L_c}\right) \right\} \begin{array}{l} \geq 0.7 \\ \leq \left(\frac{f_u}{f_y} \cdot \frac{\gamma_{mo}}{\gamma_{m1}}\right) \end{array}$$

$b_s \rightarrow$ shear leg distance.

$L_c \rightarrow$ length of end connection

$w \rightarrow$ outstanding leg distance.

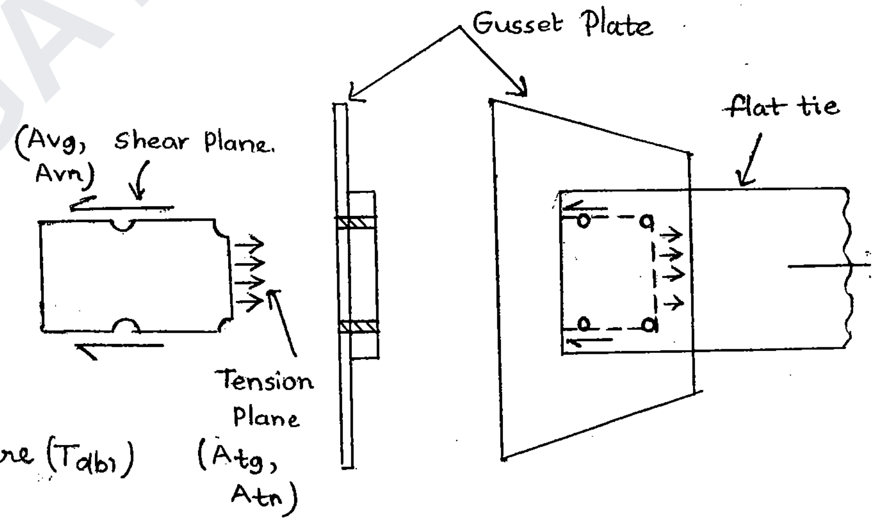
$t \rightarrow$ thickness of outstand.

$b_s = w + w_1 - t$. (bolted angle)

$b_s = w$ (welded angl

- It is better to use unequal angle section for better tensile strength. Longer leg is connected to gusset plate (higher A_{nc}) and correction factor (β) is reduced.

* Based on Block Shear Failure (T_{db})



– Shear yielding & Tension Plane rupture (T_{db1})

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

– Shear rupture & Tension Yield (T_{db2})

T_{db} : lesser of T_{db1} & T_{db2}

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

A_{vg} & $A_{vn} \rightarrow$ min gross and net area of shear plane resp'ly.

A_{tg} & $A_{tn} \rightarrow$ min gross and net area of tension plane resp'ly.

* Design requirement for Safety of Tension member

- $T \leq T_d$:- min of $\begin{cases} T_{dg} \\ T_{dn} \\ T_{db} \end{cases}$

- λ of member $\leq \lambda_{limit}$

$\lambda_{limit} \rightarrow$ limiting slenderness ratio of tension member as per IS 800 : 2007

5th WED, SEPT.

→ Design of Axially Loaded Tension Member

* Slenderness ratio of a Tension Member

$$= \frac{\text{unsupported length}}{\text{Min. radius of gyration}} = \frac{l}{r_{min.}}$$

* Limiting Slenderness Ratio (λ_{limit}).

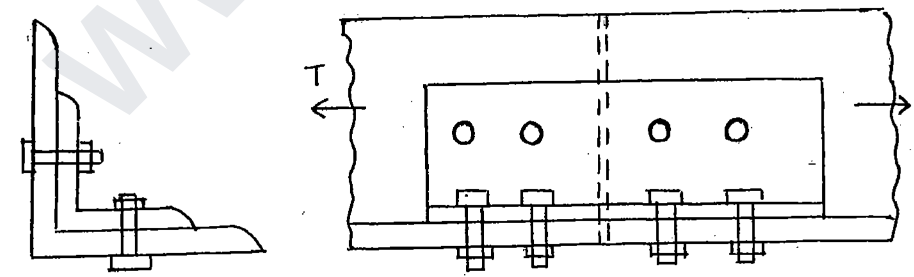
(i) A tension member is subjected to load (or) stress reversals due to loads other than wind (or) earthquake load — 180

(ii) A member used as tie in roof truss (or) in a bracing system subjected to load reversals due to loads resulting from wind (or) earthquake loads — 350

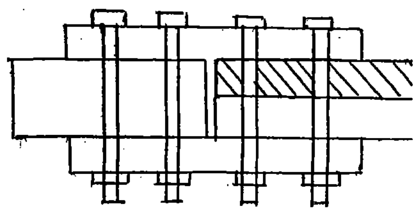
(iii) For any other tension member — 400
(other than pretensioned members)

* Tension Splice

Tension splice is a joint for tension member normally used for extending length of a tension member (where the size available from Indian Rolling Mills are limited) and also used for joining two different sizes of tension members



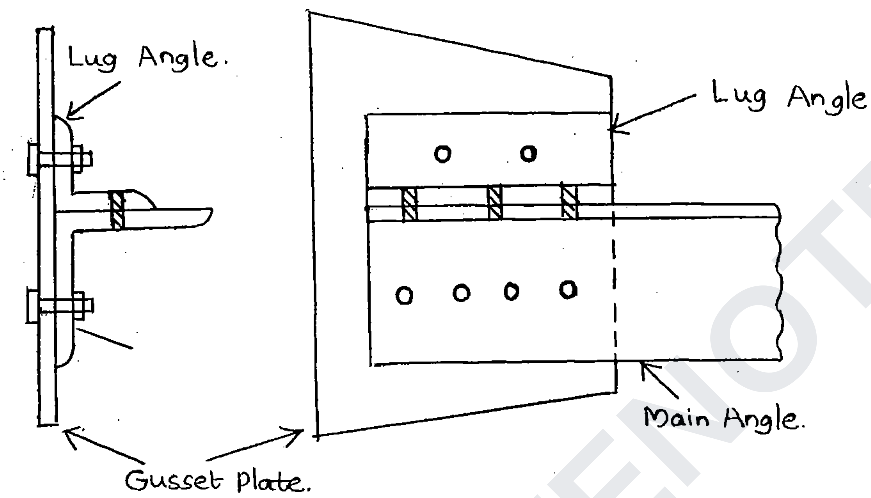
Tension Splice b/w two angle members



Tension splice b/w two diff. sizes of tension members

* Lug Angle.

Lug angle is a short length of an angle used at a joint location to join outstanding leg of an angle to the gusset plate (and also to join outstanding flange of a channel to the gusset plate) so that length of connection or joint can be reduced.



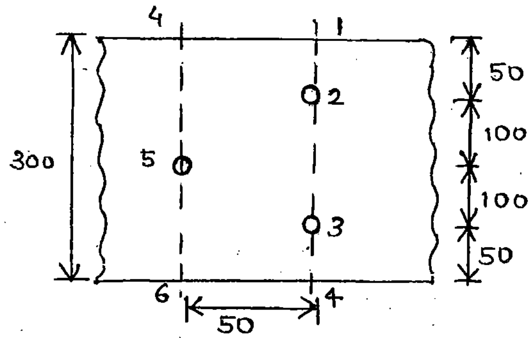
P- 41

1. $B = 300\,mm$, $t = 10\,mm$, $d = 18\,mm$.

Net sectional area of plate with one bolt hole,

$$A_n = (B - n d_0)t = (300 - 1 \times 20) \times 10 = \underline{\underline{2800}}\ mm^2$$

2. Hole diameter, $d_0 = 25\,mm$.



Failure sections are :

a) 1 – 2 – 3 – 4

b) 4 – 5 – 6

c) 1 – 2 – 5 – 6

d) 1 – 2 – 5 – 3 – 4

'An' along 1–2–3–4 : (chain pattern).

$$A_n = (300 - 2 \times 25)10$$
$$= \underline{\underline{2500}}\ mm^2$$

Along 1-2-5-6 :

$$A_n = (B - n d_o)t + \frac{p^2 t}{4g}$$

$$= (300 - 2\times25)\,10 + \frac{50^2 \times 10}{4\times100} = \underline{\underline{2562.5}}\ mm^2$$

Along 1-2-5-3-4 :

$$A_n = (300 - 3\times25)\,10 + \frac{50^2\times10}{4\times100} + \frac{50^2\times10}{4\times100}$$

$$= \underline{\underline{2375}}\ mm^2$$

Effective sectional area $= \underline{\underline{2375}}\ mm^2$

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

$$A_n = (200 - 3\times18)\times12 = 1752\ mm^2$$

$$T_{dn} = 0.9\times1752\times\frac{410}{1.25} = \underline{\underline{517.19}}\ kN$$

05. $A = 1379\ mm^2$, $f_y = 250$

$$T_{dg} = \frac{A_g f_y}{\gamma_{mo}} = \frac{1379\times250}{1.1} = \underline{\underline{313.4}}\ kN$$

06. $f_u = 410$ MPa ; $f_y = 250$ MPa ; $d_o = 18$ mm

$T_{da} \rightarrow$ min of T_{dg} (or) T_{dn} (or) T_{db}

$d_o = 18$ mm

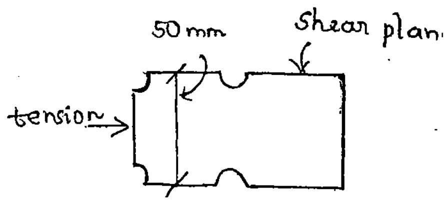
Min end distance, $e_{min} = 1.5\,d_o = 27$ mm.

$e_{provided} > e_{min}$. (no block shear failure).

$$T_{dg} = \frac{A_g f_y}{\gamma_{mo}} = \frac{120\times10\times250}{1.1} = \underline{\underline{272.7}}\ kN$$

$$T_{dn} = 0.9\,\frac{A_n f_u}{\gamma_{m1}} = 0.9\,(120 - 2\times18)\,10\times\frac{410}{1.25} = \underline{\underline{247.9}}\ kN$$

$A_{tg} = 50 \times 10 = 500 \text{ mm}^2$



$A_{tn} = \left(50 - \frac{d_o}{2} - \frac{d_o}{2}\right) = (50-18)10 = 320 \text{ mm}^2$

$A_{vg} = (50+35) \times 10 \times 2 = 1700 \text{ mm}^2$

$A_{vn} = \left((50+35) - 18 - \frac{18}{2}\right) \times 10 \times 2 = 1160 \text{ mm}^2$

Shear yield & Tension rupture (T_{db1}) :

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

$$= 1700 \times \frac{250}{\sqrt{3} \times \cancel{1.25}\, 1.1} + 0.9 \times 320 \times \frac{410}{1.25} = \underline{\underline{317.53 \text{ kN}}}$$

Shear rupture & Tension yield (T_{db2}) :

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

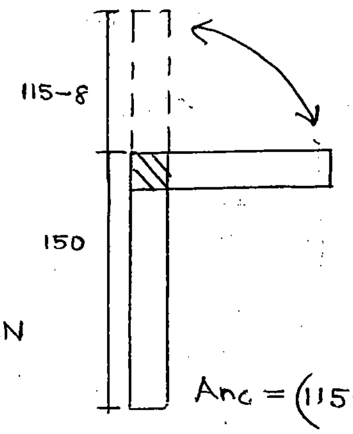
$$= 0.9 \times 1160 \times \frac{410}{\sqrt{3} \times 1.25} + \frac{500 \times 250}{1.1}.$$

$$= 311 \text{ kN}.$$

T_{db} = min of T_{db1} & T_{db2} = $\underline{\underline{311 \text{ kN}}}$

$T_d = \underline{\underline{247}}$ kN

07 $f_u = 410$ MPa, $f_y = 250$ MPa.



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

$$A_g = [(150 + 115 - 8) \times 8] \times \frac{250}{1.1} = \underline{\underline{467.7 \text{ kN}}}$$

$A_{nc} = \left(115 - \frac{8}{2}\right.$

$= 1168$

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

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

$$= 1.4 - 0.076 \times \frac{115}{8} \times \frac{250}{410} \times \frac{115}{140} \geq 0.7 = \underline{\underline{0.85}}$$

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

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

$$T_{dn} = \underline{\underline{516.3 \ kN}}$$

09. $\lambda \leq \lambda_{limit}$

$$\frac{l_{max}}{r_{min}} \leq \lambda_{limit}$$

$$\frac{l_{max}}{20} \leq 350 \quad (r_{min} = 20 \ mm).$$

$$l_{max} \leq 350 \times 20$$

$$\leq 7000 \ mm$$

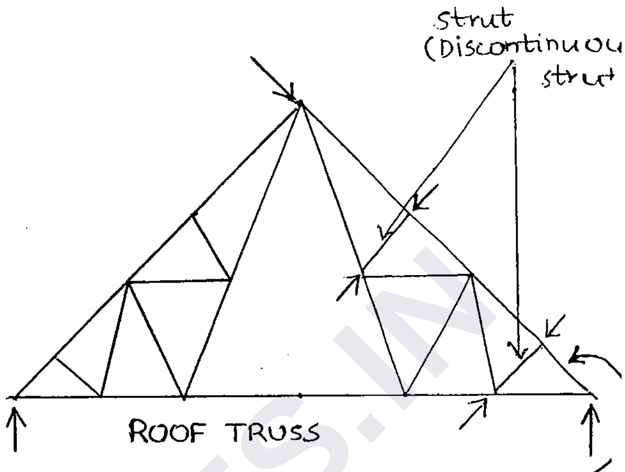
$$\leq 7.0 \ m.$$

12th Sept,
FRIDAY

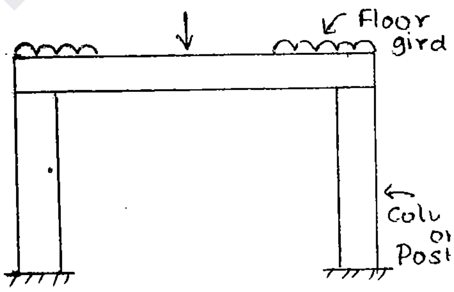
6. COMPRESSION MEMBERS

⊙ Principale Rafter –
normally used in roof truss as a main strut.

⊙ Strut –
generally used in roof truss (or) in a bracing system.



⊙ Column (or) Post (or) Stanction – (steel column)
used in industrial building to support floor (or) floor girders.



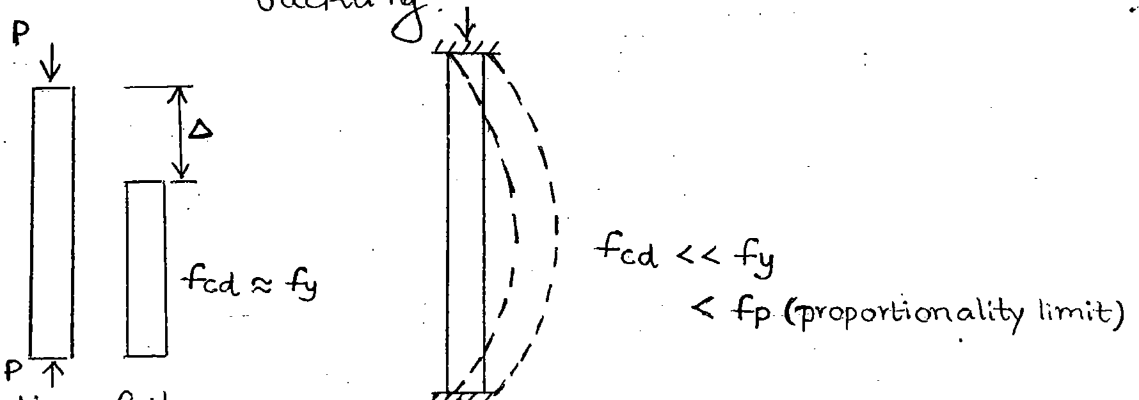
⊙ Boom –
Principle compression member in a crane system.

→ Types of Failures in a Compression Member

(i) Short Column or short strut – Normally fail by yielding or crushing of a material.

(ii) Intermediate Column or Intermediate strut – generally fail by inelastic buckling.

(iii) Long Column or Long strut – generally fail by elastic buckling.



(Yielding failure in Short Column)

(Elastic buckling failure in long column)

— Compressive strength of a column or strut is influenced by :

(i) Initial imperfection in the member (due to handling, transportation & erection).

(ii) Residual stress in the c/s (developed due to cobling from high temp while moulding to ambient temp.)

(iii) Eccentricity of loading (creating additional moments)

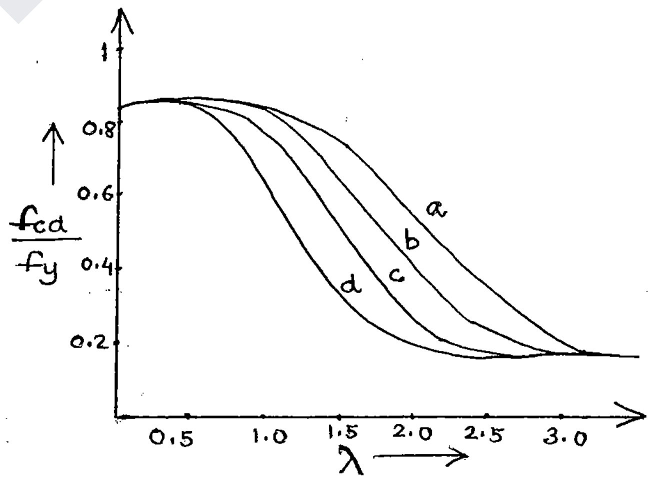
→ Design Compression Strength of a Member (P_d)

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

f_{cd} → design stress in axial compression

A_e → effective sectional area.

⊙ IS 800:2007 proposes multiple curves a, b, c & d based on PERRY ROBERTSON'S Approach.



λ = non-dimensional effective slenderness ratio.

$$\lambda = \sqrt{\frac{f_y}{f_{cc}}} = \sqrt{\frac{f_y\left(\frac{KL}{r}\right)^2}{\pi^2 E}}$$

f_{cc} = elastic buckling stress

$$= \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2}$$

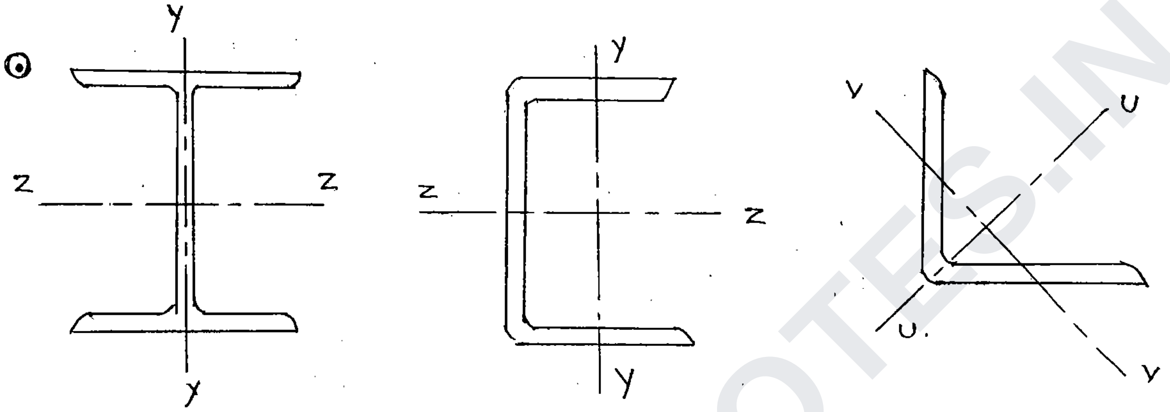
⊙ As per IS 800:1984,

$$P_d = \sigma_{ac} \cdot A_e$$

$$\sigma_{ac} = \frac{0.6 f_y f_{cc}}{\left((f_y)^n + (f_{cc})^n\right)^{1/n}} \leq 0.6 f_y$$ (MERCHENT RANKINE FORMULA)

⊙ Class a (tubular sections) members have more compressive strength compared to class c (solid circular sections). Both class a & c members are initially straight and free from eccentric loading. But residual stresses will be more in class c compared to class A as uniform cooling takes place in the interior and exterior of tubular sections.

⊙



$r_{zz} > r_{yy}$. $\qquad r_{zz} > r_{yy} \qquad r_{uu} > r_{vv}$.

$$\lambda = \sqrt{\frac{f_y \left(\frac{KL}{r}\right)^2}{\pi^2 E}}$$

∴ λ will be more for bending along y-y axis. Higher λ means lower $\frac{f_{cd}}{f_y}$ value from the curve.

$$\therefore (P_d)_{z-z} > (P_d)_{y-y}.$$

Economic sections or best sections are those in which

$$(P_d)_{zz} = (P_d)_{yy}. \quad (\text{tubular circular sections}).$$

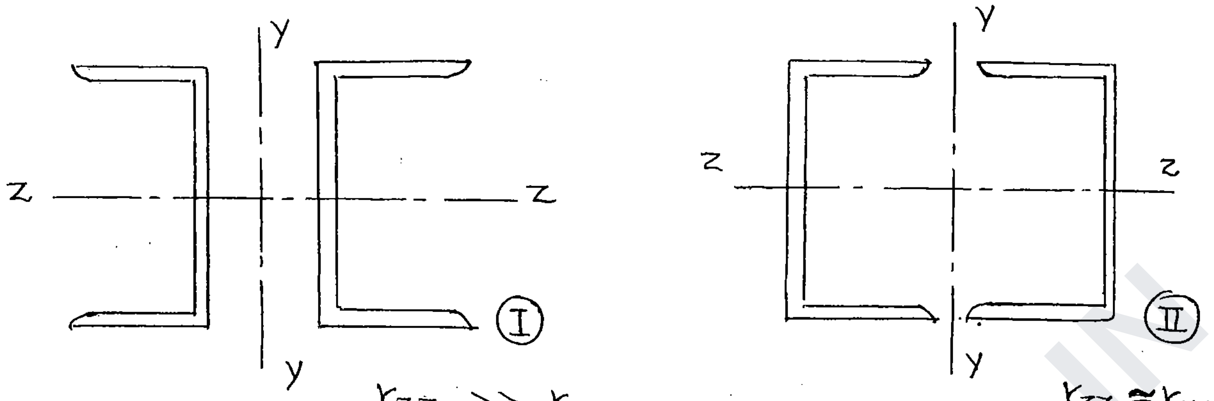
* Perry Robertson's Equation

$$f_{cd} = \frac{f_y/\gamma_{mo}}{\phi + (\phi^2 - \lambda^2)^{0.5}} \leq \frac{f_y}{\gamma_{mo}}$$

$\lambda \rightarrow$ non dimensional effective slenderness ratio $= \sqrt{\frac{f_y}{f_{cc}}}$

$$= \sqrt{\frac{f_y \left(\frac{KL}{r}\right)^2}{\pi^2 E}}$$

II^{nd} section will be subjected to eccentric loading wrt y axis. However for I^{st} section, loading will be CG of section.



$r_{zz} >> r_{yy}$. $r_{zz} \simeq r_{yy}$

For same area of section, II^{nd} section will be more economical and effecient than I^{st} section.

⊙ For struts, tacking bolts, or tacking welds or tacking rivets are used to connect different sections.

⊙ For columns, bracings or lacing systems are used.

* Connecting Systems for Built up Columns are:
  - Lacing System (preferred for eccentric loaded column)
  - Batten system (generally used for axially loaded column)

* Lacing System

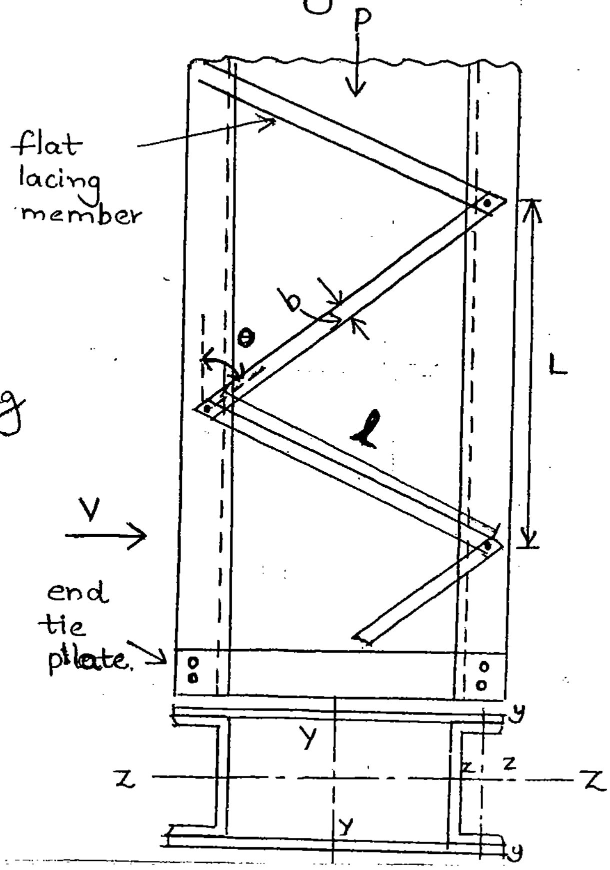
– Flats, angles, channels or tube sections are mostly used as section for lacing member.

$\theta \rightarrow$ angle of inclination of lacing with longitudinal axis.

$L \rightarrow$ spacing of lacing member

$b \rightarrow$ width of flat lacing

$l \rightarrow$ length of lacing member.



Step 3 : Select a trial section with approximate area equal to area required with higher minimum radius of gyration.

Step 4 : Based on end condition, KL and effective slenderness ratio (KL/r) and fcd of a trial section may be calculated.

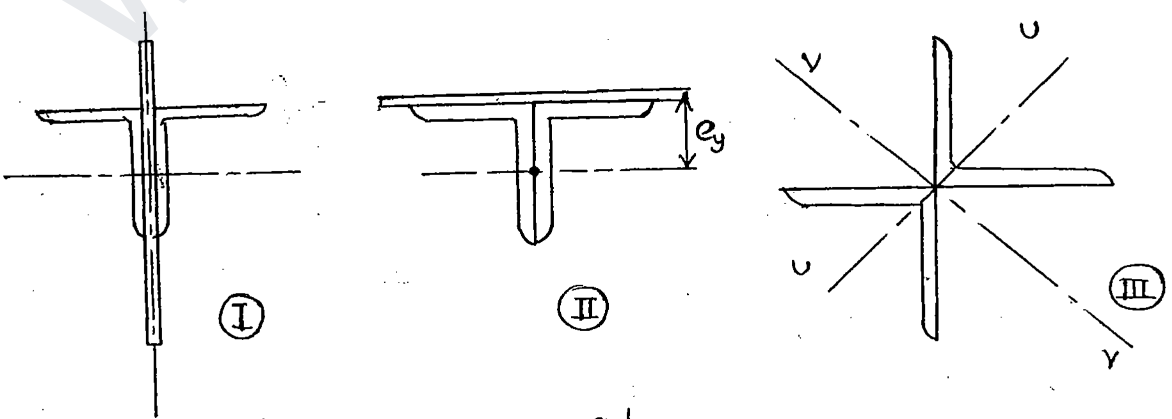
Step 5 : $(P_d)_{trial} = (f_{cd})_{trial} \times A_e$

If $P \leq (P_d)_{trial}$; section is safe. Otherwise same procedure is repeated with higher c/s area.

* Limiting Slenderness Ratio.

	Limiting λ
⊙ A compression member subjected to compressive loads resulting from DL + LL combination.	≤ 180
⊙ A member subjected to compressive loads resulting from wind load or earthquake loads, WL/EQL	≤ 250
⊙ For compression flange of beam.	≤ 300

→ Built up Sections (Built up Coloumns)



Out of three sections, III[rd] one is most effecient as more area thrown away from buckling axis and hence more r

$\frac{KL}{r}$ → effective slenderness ratio.

KL → effective length of a compression member.

K → effective length constant ; L → unsupported length.

$$\phi = 0.5\left(1 + \alpha(\lambda - 0.2) + \lambda^2\right)$$

α → imperfection factor.

Buckling Class	a	b	c	d
α	0.21	0.36	0.49	0.76

It considers —

a) Eccentricity of a load.

b) Initial straightness.

c) Residual stress present in c/s

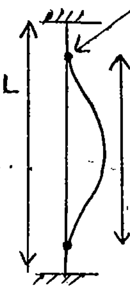
→ Effective Length of a Compression Member : (KL)

KL depends upon :—

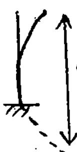
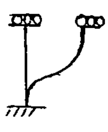
(i) type of end condition

(ii) No: of members meeting at a joint location.

(iii) No: of rivets or bolts used at a joint.



* Effective Length of a Column (KL)

Type of End Condition	Effective Length (KL)	Effective length constant (K)
Fixed — Fixed. 	0.65 L	0.65
Fixed — Pinned (or hinged) 	0.80 L	0.80
Pinned — Pinned (Hinged — Hinged) 	1.0 L	1.0
Fixed — Free  KL	2.0 L	2.0
Fixed — Moment Roller. 	1.2 L	1.2

Pinned-moment roller		2.0L	2.0

18th Sept,
THURSDAY

→ Design of Axially Loaded Compression Members.

* Design Requirement for Safety of Compression Members:

- Design compressive load (P) ≤ Design compression strength of a member.

- $\left(\frac{KL}{r}\right)_o$ of section ≤ Limiting Slenderness Ratio. (to take care of erection loads)

⊙ Design of compression member is an indirect method of design as the failure stress, fcd, depends on lot of factors.

$$f_{cd} = f\left(\frac{KL}{r}, \alpha, \phi = \sqrt{\frac{f_y}{f_{cc}}}\right)$$

⊙ For a tension member, section is dependent on A_g.
For a compression member, section is dependent on A_e & r
For a beam, section is dependent on moment of inertia.

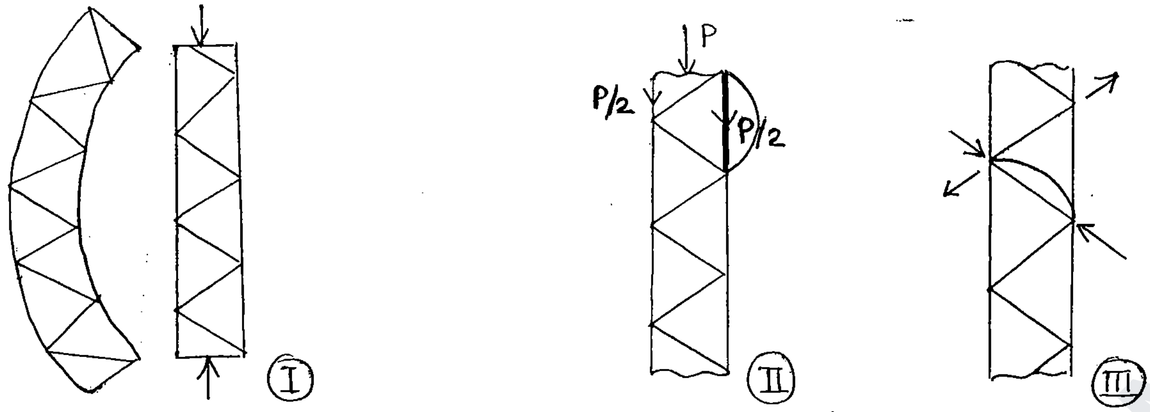
Step 1: Assume design stress in axial compression (f_{cd})

f_{cd} = 90 MPa (for angle struts)
= 135 MPa (for I-section columns)
= 200 MPa (for heavily loaded columns)

Step 2: Effective sectional area required for factored compressive load, P is :

$$(A_e)_{req} = P/(f_{cd})_{assumed}$$

– Failure of lacing system



(I) → buckling of whole component

(II) → local buckling of column component.

(III) → local buckling of lacing member.

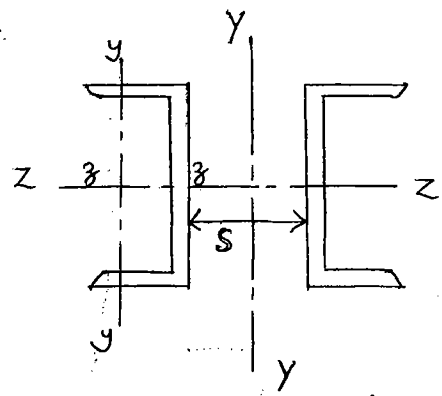
– General Specifications:

⊙ The radius of gyration normal to the plane of lacing not less than parallel to the plane of lacing

ie, $r_{yy} \nless r_{zz}$

To have economy in design, condition must be

$$r_{yy} = r_{zz} \Rightarrow I_{yy} = I_{zz}$$

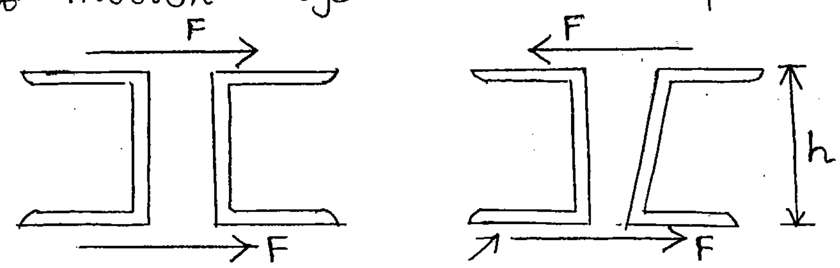


$$I_{zz} = I_{yy}$$

$$2 I_{zz} = 2\left(I_{yy} + A z^2\right)$$

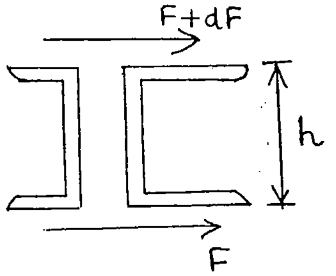
$$= 2\left(I_{yy} + A\left(\frac{S}{2} + c_{yy}\right)^2\right)$$

⊙ For single lacing system, lacing on one plane should reflect mirror image to the other plane.



$$M = F \times h$$

⊙ There should not be any variation in lacing system



$M = dF \times h$

– Design Specifications: (important)

⊙ $40^\circ \leq \theta \leq 70^\circ$

Optimum angle for lacing member, $\theta = 45^\circ$ to 50°

If $\theta < 40^\circ$, lacing member may chance to carry some of the the column load and same load may transfer from one column component to another column component like truss member.

If $\theta > 70^\circ$, the tieing force carrying capacity of lacing member is decreased.

⊙ Effective slenderness ratio of lacing member should be less than or equal to 145.

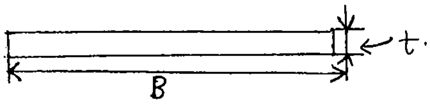
$$\left(\frac{KL}{r}\right)_{lacing} \leq 145$$

Above condition is required to avoid local buckling of lacing member b/w individual column components.

	Kl
(i) For single lacing system with one bolt or one rivet.	$1.0\,l$
(ii) For single lacing with welds.	$0.7\,l$
(iii) For double lacing system.	$0.7\,l$

where $Kl \rightarrow$ effective length of lacing member (Kl

For flat lacing (B x t)



$$I_{min} = \frac{Bt^3}{12}$$

$$r_{min} = \sqrt{\frac{I_{min}}{A}} = \sqrt{\frac{Bt^3}{12 \cdot Bt}} = \frac{t}{\sqrt{12}}$$

$$\frac{\sqrt{12}\, Kl}{t} \leq 145$$

Min width of flat lacing member, $B_{min} \simeq 3 *$ shank diameter of bolt.

Shank diameter of bolt	M_{16}	M_{18}	M_{20}	M_{22}
B (min) (mm)	50	55	60	65

- Min thickness of lacing member

$t_{min} \nless \frac{l}{40}$; for single lacing system

$\nless \frac{l}{60}$; for double lacing member.

- $\frac{L}{r^c_{min}} \leq 50$

$\leq 0.7 \times$ effective slenderness ratio of whole built up column $\left(\frac{KL}{r}\right)_0$

whichever is less.

$r^c_{min} \rightarrow$ min. radius of individual column component.

The above condition is required to eliminate local buckling of individual column component b/w lacings.

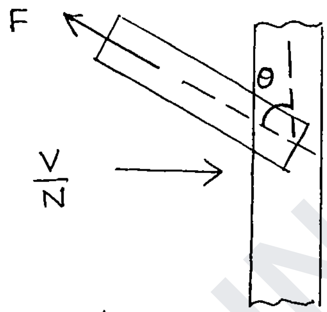
Lacing members ~~may~~ should be designed for a transverse shear of 2.5 % factored column load.

Transverse shear, V = 2.5 * factored column load

$$V = \frac{2.5\ P_u}{100}$$

$$\frac{V}{N} - F\sin\theta = 0$$

$$F = \frac{V}{N}\sin\theta$$



- Design axial force in lacing member (F)

$$F = \frac{V}{N\sin\theta}$$

$$F = \frac{V}{2\sin\theta} \quad \text{(for single lacing, } N = 2\text{)}$$

$$= \frac{V}{4\sin\theta} \quad \text{(for double lacing system } N = 4\text{)}$$

- Effective slenderness ratio of lacing column should be increased by 5% in order to take care shear deformations due to unbalanced horizontal forces in lacing members.

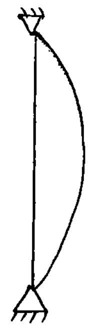
LEVEL-2

P-52

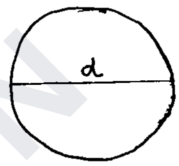
9th SEPT, FRIDAY

1. Effective slenderness ratio, $\frac{KL}{r} = 200$

$\frac{l}{d} = ?$



$KL = 1.0\,L$ (Ends are hinged)



$\frac{KL}{r} = 200$

$\therefore \frac{L}{r} = 200$

$$r = \sqrt{\frac{I}{A}} = \sqrt{\frac{\pi d^4/64}{\pi/4\, d^2}} = \frac{d}{4}$$

$$\frac{L}{d/4} = 200 \Rightarrow \underline{\underline{\frac{l}{d} = 50}}$$

4. Transverse shear, $V = \frac{2.5P}{100}$

$P \rightarrow$ Factored load. ; $P_s \rightarrow$ service load.

$P = \gamma_L \times P_s$

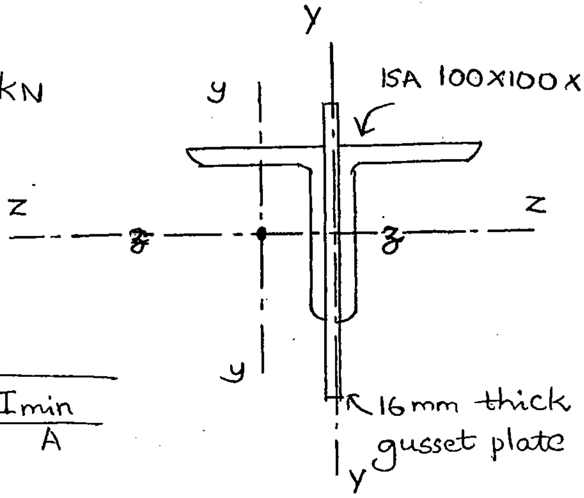
$= 1.5 \times 1000 = 1500$ kN.

$$\therefore V = \frac{2.5 \times 1500}{100} = \underline{\underline{37.5}} \text{ kN}$$

5. Axial force in lacing, $F = \frac{V}{N \sin\theta}$

$\theta = 45^0$; $N = 2$.

$$F = \frac{37.5}{2 \sin 45} = \underline{\underline{26.52}} \text{ kN}$$



6. $A = 1903 \text{ mm}^2$

$I_{zz} = I_{yy} = 177 \times 10^4 \text{ mm}^4$

Min radius of gyration, $r_{min} = \sqrt{\frac{I_{min}}{A}}$

$I_{min} = $ min of I_{yy} or I_{zz}.

$$I_{zz} = 2\left(I_{GG} + Ay^2\right)$$

$$= 2\left(I_{zz} + A0^2\right) = 2\,I_{zz}.$$

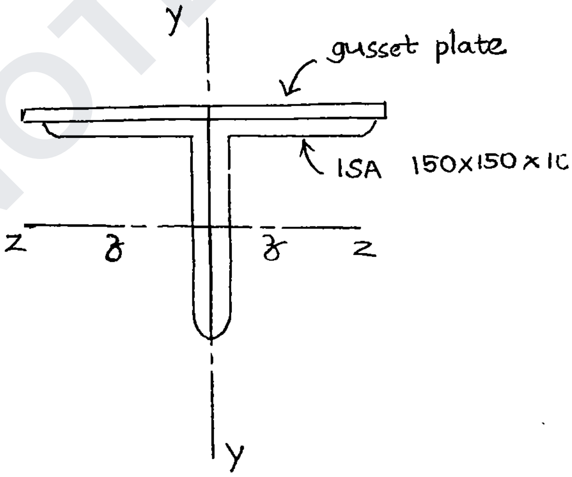
Gusset plates are provided only at points of junctions.
$\therefore$ I & A of gusset plates are not included

$$I_{yy} = 2\left(I_{yy} + A\left(t/2 + C_{zz}\right)^2\right).$$

$$\Rightarrow \quad I_{zz} < I_{yy} \quad \therefore \; I_{min} = I_{zz}.$$

$$r_{min} = \sqrt{\frac{2\,I_{zz}}{2A}} = \underline{\underline{30.6 \text{ mm}}}$$

8. $I_{min} = \min \; I_{zz}$ or I_{yy}



$$I_{zz} = 2\left(I_{zz} + \cancel{Ay^2}^{\,0}\right)$$

$$= 2\,I_{zz}$$

$$I_{yy} = 2\left(I_{yy} + Az^2\right)$$

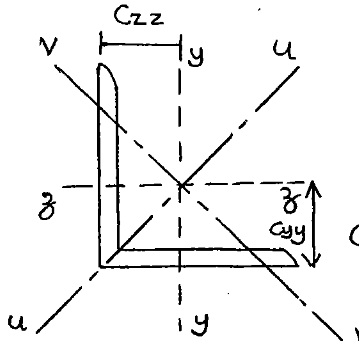
$$= 2\,I_{yy} + 2A\,(40.8)^2$$

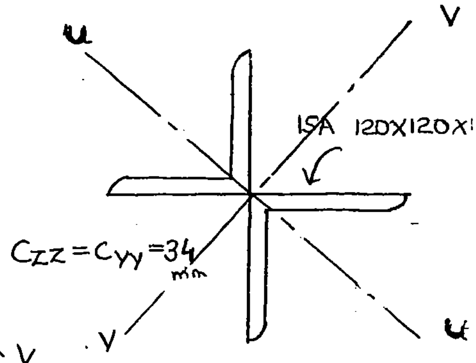
$$I_{min} = I_{zz}.$$

$$r_{min} = \sqrt{\frac{I_{min}}{A}} = \sqrt{\frac{2\,I_{zz}}{2A}} = \sqrt{\frac{6.335\times10^6}{2921}} = \underline{\underline{46.57 \text{ mm}}}$$

9. $r_{min} = \sqrt{\dfrac{I_{min}}{A}}$

$I_{min} = \min$ of I_{vv} or I_{uu}





$$I_{uu} = 2\left(I_{uu} + Av^2\right).$$

$$= 2\left(584\times10^4 + 2750\times0^2\right)$$

$$= 1168\times10^4 \text{ mm}^4$$

$$I_{vv} = 2\left(I_{vv} + Au^2\right).$$

$$= 2\left(151\times10^4 + 2750\times2312\right).$$

$$= 1573.6\times10^4$$

$A = 2750 \text{ mm}^2$

$I_{zz} = I_{yy} = 368\times10^4$

$I_{uu} = 584\times10^4$

$I_{vv} = 151\times10^4.$

$u^2 = C_{zz}^2 + C_{yy}^2 = 34^2 + 34^2 = 2312.$

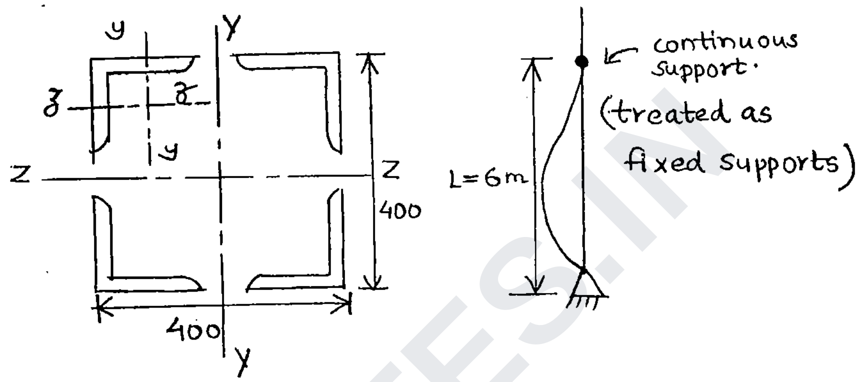
$I_{min} = I_{uu} = 1168 \times 10^4 \ mm^4.$

$$r_{min} = \sqrt{\frac{I_{uu}}{A}} = \sqrt{\frac{1168 \times 10^4}{2 \times 2750}} = \underline{\underline{46.08}} \ mm$$

10. $A = 1903$, $I_{zz} = I_{yy} = 177 \times 10^4 \ mm^4.$

$C_{yy} = C_{zz} = 28.4$ mm.

$r_{yy} = r_{zz} = 30.5$ mm.



P–47 : 6 th Point.

Buckling about any axis of built-up member → class C

Design axial compressive load, $P = f_{cd} \times A_e$.

f_{cd} = Design axial compressive stress

A_e = Effective sectional area $= 4 \times 1903 \ mm^2$

$$f_{cd} = \frac{f_y/\gamma_{mo}}{\phi + (\phi^2 - \lambda^2)^{0.5}} \leq \frac{f_y}{\gamma_{mo}}$$

Assume grade of steel, Fe 410

$\Rightarrow f_y = 250$ MPa, $\gamma_{mo} = 1.10$

Built up members — class C $\Rightarrow$ Imperfection factor, $\alpha = 0.49$

λ = non dimensional effective slenderness ratio

$$= \sqrt{\frac{f_y \left(\frac{KL}{r}\right)^2}{\pi^2 E}}$$

$KL = 0.8L$ (one end hinged & other end fixed).

$= 0.8 \times 6000 = 4800$ mm.

MI of built up section : $I_{zz} = I_{yy} = (I_{GG} + Ay^2) \, 4$

$$= 4\left(177 \times 10^4 + 1903 \times \left(\frac{400}{2} - 28.4\right)^2\right)$$

$$= 231.227 \times 10^6 \ mm^4$$

$$r_{min} = r_{yy} = r_{zz} = \sqrt{\frac{I_{zz} \text{ or } I_{yy}}{4A}}$$

$$= \sqrt{\frac{231.227 \times 10^6}{4 \times 1903}} = 174.28 \ mm.$$

Effective slenderness ratio $= 1.05 \dfrac{KL}{r}$ (5% ↑ for built up column).

$$\lambda = \sqrt{\frac{f_y \left(\frac{KL}{r}\right)^2}{\pi^2 E}} = \sqrt{\frac{250 \times \left(\frac{1.05 \times 4800}{174}\right)^2}{\pi^2 \times 2 \times 10^5}} = \underline{\underline{0.325}}$$

$$\phi = 0.5\left(1 + \alpha(\lambda - 0.2) + \lambda^2\right)$$

$$= 0.5\left(1 + 0.49(0.325 - 0.2) + 0.325^2\right)$$

$$= 0.5834$$

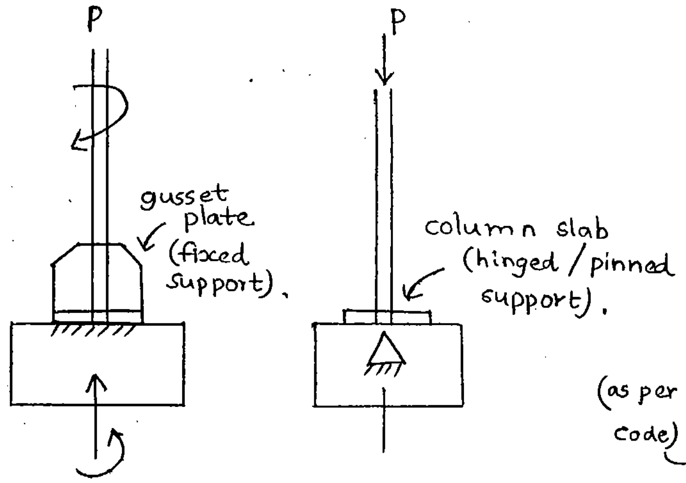
$$f_{cd} = \frac{f_y / \gamma_{m0}}{\phi + (\phi^2 - \lambda^2)^{0.5}} = \frac{250/1.10}{0.58 + (0.58^2 - 0.325^2)}$$

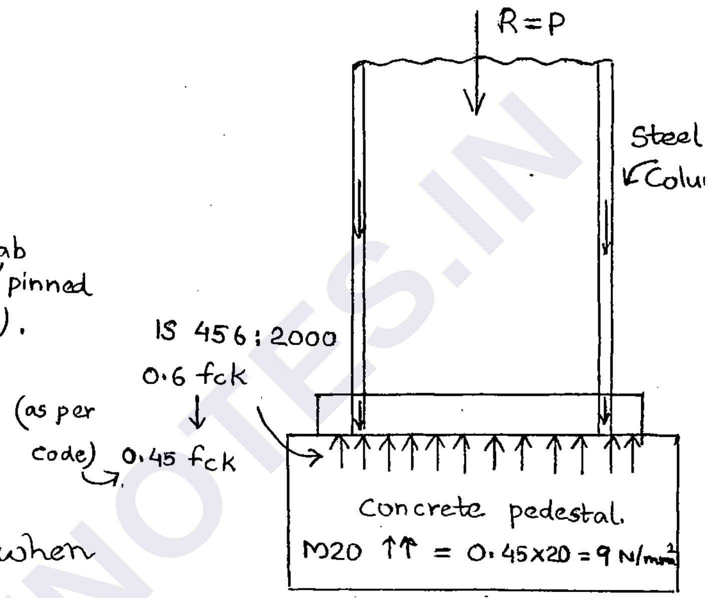
$$f_{cd} = 214.33 < \frac{250}{1.1} \ N/mm^2$$

$$P_d = f_{cd} \times A = 214\,(4 \times 1903)$$

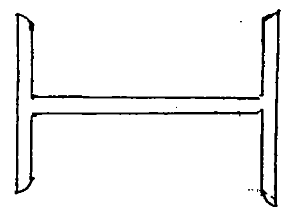
$$= 1628 \times 10^3 \ N = \underline{\underline{1628 \ kN}}$$

7. COLUMN BASES & COLUMN SPLICES





- Gusset plates are provided when moment is acting on the column.
- Column pl slabs are used when only axial loads act on the column.



→ Types of Column Bases:

– Slab bases :–

To be provided when column is subjected to direct axial loads only.

– Gusseted base :–

To be provided, when column is subjected to heavy axial loads and subjected to axial loads with moment.

⊙ Bearing strength of concrete (as per code) = 0.45 fck

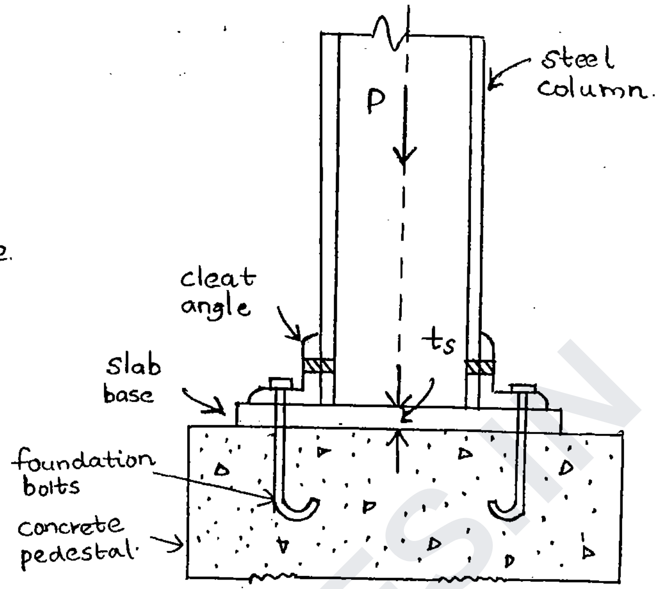
$$\frac{P}{A} \leq 0.45 \text{ fck}.$$

$$\therefore A = \frac{P}{0.45 \text{ fck}}.$$

Min. base area required to safely transmit the axial load = $\frac{P}{0.45}$

3rd Oct,
FRIDAY

→ Design of Slab base:



$L \rightarrow$ length of slab base

$B \rightarrow$ width of slab base.

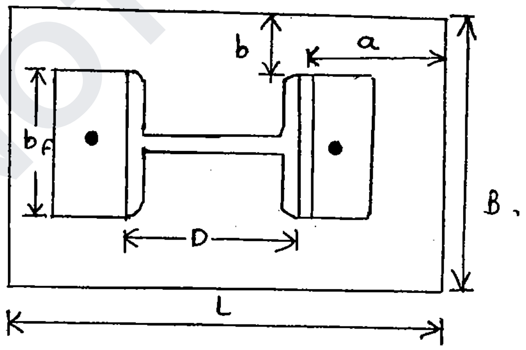
$t_s \rightarrow$ thickness of slab base.

$D \rightarrow$ depth of steel column section

$b_f \rightarrow$ width of column flange

$a \rightarrow$ bigger projection of slab base beyond the steel column.

$b \rightarrow$ smaller projection of slab base beyond the steel column.



Step 1: Assume suitable grade of concrete. Bearing strength of concrete taken as $0.45 f_{ck}$

Step 2: Area of slab base required = $\dfrac{\text{Factored column load}}{\text{Bearing strength of concrete}}$

$$\boxed{A = \frac{P}{0.45 f_{ck}}}$$

– For square slab base,

Side of slab base, $L = B = \sqrt{A}$

– For rectangular slab base,

$$A = L \times B$$

$$A = (D+2a)(b_f+2b)$$

If a & b (projections) are same, the thickness required for slab base will be optimum.

ie, condition for optimum thickness of slab base:

$$a = b$$

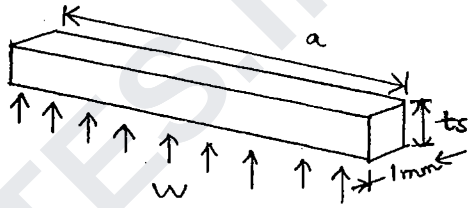
– upward pressure from concrete pedestal is w

$$w = \frac{P}{\text{Provided area of Slab base}}$$

Consider 1 mm width of slab base,

Net moment due to upward pressure, $M = \frac{wa^2}{2} - \mu \frac{wb^2}{2}$

$$= \frac{w}{2}\left(a^2 - \mu b^2\right)$$



$$\therefore \quad M = \frac{w}{2}\left(a^2 - 0.3\, b^2\right) \quad ; \quad \mu = 0.3 \text{ for steel.}$$

– Design bending strength of slab base,

$$M_d = Z_p \frac{f_y}{\gamma_{mo}}$$

$$M_d = 1.2\, Z_e \frac{f_y}{\gamma_{mo}}$$

Design condition is :-

$$\frac{w}{2}\left(a^2 - \mu b^2\right) = 1.2\, Z_e \frac{f_y}{\gamma_{mo}}$$

$$= 1.2 \frac{t_s^2}{6} \frac{f_y}{\gamma_{mo}}$$

$$Z_e = \frac{I}{y} = \frac{b t_s^3}{12} \Big/ t_s/2 = \frac{t_s^2}{6}.$$

$\Rightarrow$ Thickness of slab base, $t_s = \sqrt{\dfrac{2.5\, w\left(a^2 - 0.3\, b^2\right)\gamma_{mo}}{f_y}}$

$t_s > t_f$; $t_f \rightarrow$ thickness of column flange.

$\rightarrow$ Design of Gusseted Base.

$L \rightarrow$ length of base plate

$B \rightarrow$ width of base plate

$P \rightarrow$ Factored column load.

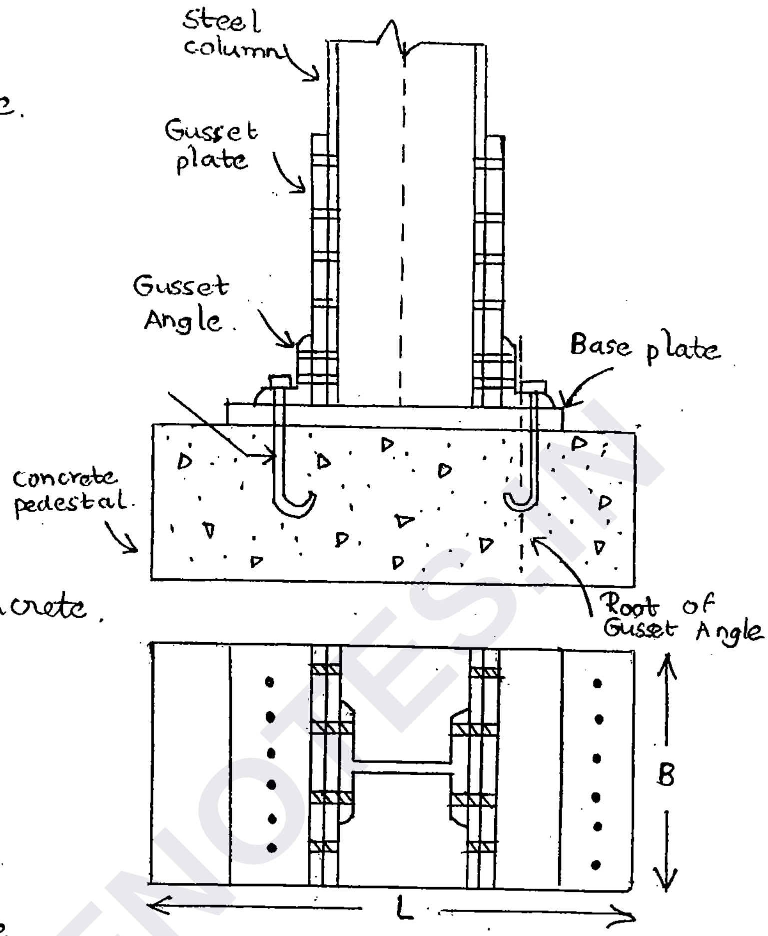
Step 1: Assume suitable grade of concrete.

Bearing strength of concrete $= 0.45\, f_{ck}$.

Area of base plate required,

$$A = \frac{\text{Factored column load}}{\text{Bearing strength of concrete}}$$

$$\boxed{A = \frac{P}{0.45\, f_{ck}}}$$



L = length of base plate.
= width of base plate parallel to web.

L = depth of steel column + 2 * thickness of GP + 2 * leg width of gusset angle + 2 * min. overhand (for bolted connection)

L = depth of steel column + 2 * thickness of GP + 2 * min overhang (for welded connection)

$$\text{Width of base plate, } B = \frac{\text{area of base plate}}{\text{length of base plate}}$$

– Upward pressure from concrete pedestal,

$$w = \frac{\text{Factored column load}}{\text{Provided area of base plate.}}$$

– Moment due to upward pressure for 1 mm width of base plate, $M = w \cdot c \cdot \frac{c}{2} = \frac{wc^2}{2}$

– Design bending strength of base plate,

$$M_d = Z_p \cdot \frac{f_y}{\gamma_{mo}} = 1.2\, Z_e \frac{f_y}{\gamma_{mo}}$$

$c \rightarrow$ cantilever projection beyond the root of gusset angle (for bolted connection)

$$M \leqslant M_d$$

$$\Rightarrow \quad \frac{wc^2}{2} \leqslant 1.2 \times \frac{t_s^2}{6} \times \frac{f_y}{\gamma_{mo}}$$

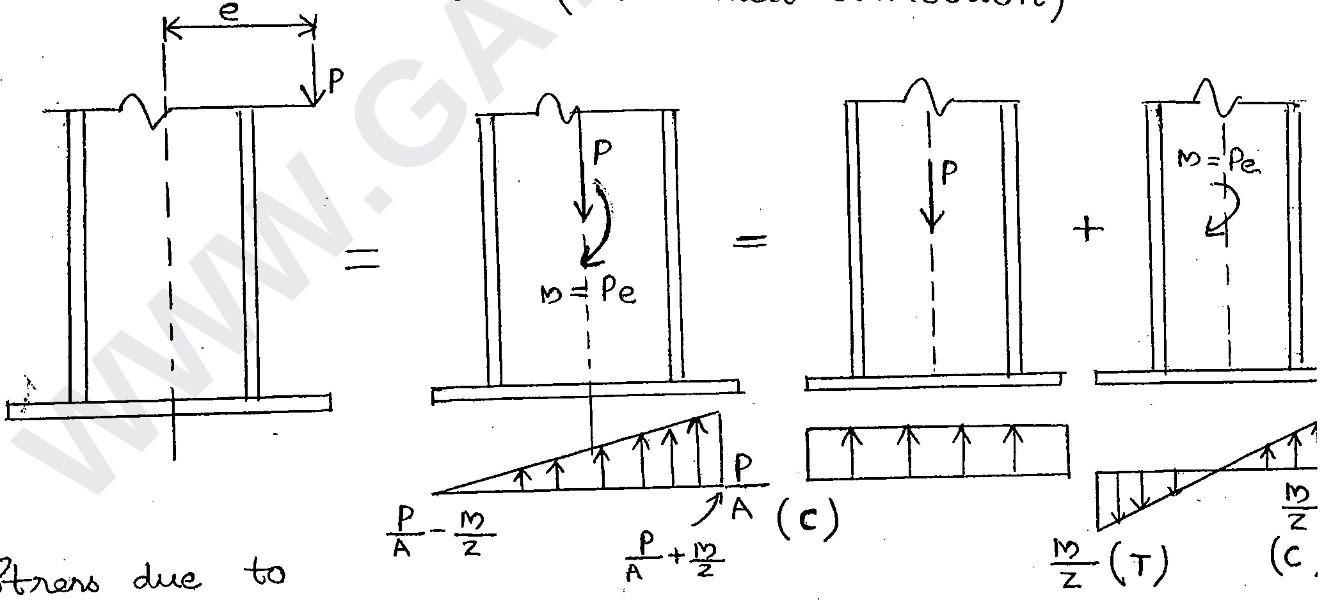
$$t_s = c\sqrt{\frac{2.75\, w}{f_y}}$$

– Thickness of base plate (t_b):

$t_b = t$ – thickness of gusset angle (for bolted connection)

$= t$ (for welded connection)

Q-57
1.



Stress due to direct axial load $= \frac{P}{A} = \frac{P}{L \times B}$ (compression).

Stress due to BM $= \pm \frac{M}{I} y = \frac{6Pe}{BL^2}$

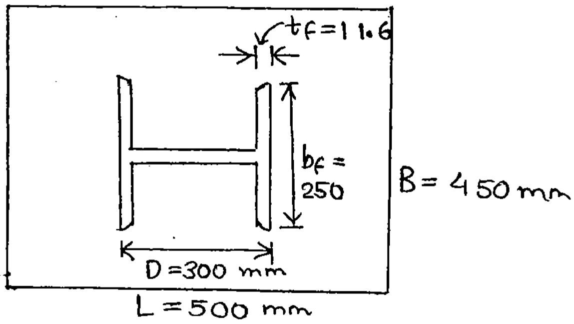
Combined stress due to P & M $= \frac{P}{A} \pm \frac{M}{Z} = \frac{P}{LB}\left(1 \pm \frac{6e}{L}\right)$

3. $a = \frac{L-D}{2} = \frac{500-300}{2}$

$= 100$ mm.

$b = \frac{B-b_f}{2} = \frac{450-250}{2}$

$= 100$ mm.



Thickness of base plate, $t_s = \sqrt{\frac{2.5w(a^2 - 0.3b^2)\gamma_{mo}}{f_y}}$

$= \sqrt{\frac{2.5 \times 9(100^2 - 0.3 \times 100^2) \times 1.10}{250}}$

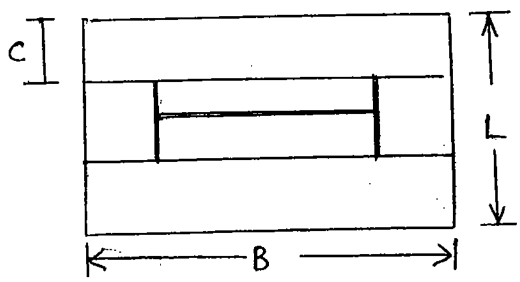
$= \underline{\underline{26.3}}$ mm ($\geq t_f = 11.6$mm).

4. $t = c\sqrt{\frac{2.75\,w}{f_y}}$

For all the 4 options given, area is same.

$\therefore$ $w = \frac{P}{A}$ is also same.

$\therefore$ $t \propto c$.



a) $c = \frac{600-140}{2} = 230$ mm

b) $c = \frac{600-400}{2} = 100$ mm.

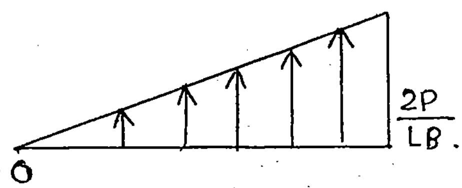
c) $c = \frac{500-140}{2} = 180$ mm.

d) $c = \frac{720-400}{2} = 160$ mm.

05. Combined stress $= \frac{P}{A} \pm \frac{M}{Z}$.

$= \frac{P}{LB}\left(1 \pm \frac{6e}{L}\right)$.

$= \frac{P}{LB}\left(1 \pm \frac{6}{L}\cdot\frac{L}{6}\right) = 0,\ \frac{2P}{LB}$



06. For Fe 410 grade steel,

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

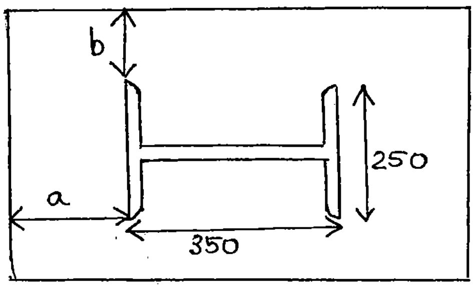
For M25 grade concrete,

$f_{ck} = 25$ MPa.

$D = 350$ mm, $b_f = 250$ mm, $t_f = 11.6$ mm.

$P = 2000$ kN

Area of base plate (slab base) $= \dfrac{P}{0.45 f_{ck}}$



$= \dfrac{2000 \times 10^3}{0.45 \times 25}$

$= 177.8 \times 10^3$ mm^2

$A = L \times B = (D+2a)(b_f + 2b)$.

$= (350+2a)(250+2b)$. $(a=b)$.

177777 $= (350+2a)(250+2a)$.

$\Rightarrow a = b = 62.29$ mm $\approx \underline{\underline{65\text{ mm}}}$

Length of base plate, $L = D + 2a$

$= 350 + 2 \times 65 = \underline{\underline{480}}$ mm.

Width of base plate, $B = b_f + 2b$.

$= 250 + 2 \times 65 = \underline{\underline{380}}$ mm

Upward pressure, $w = \dfrac{P}{\text{provided area of base plate}} = \dfrac{2000 \times 10^3}{480 \times 380}$

$= 10.96$ N/mm^2

Thickness of base plate, $t_s = \sqrt{\dfrac{2.5\, w\,(a^2 - 0.3 b^2)\, \gamma_{m0}}{f_y}}$

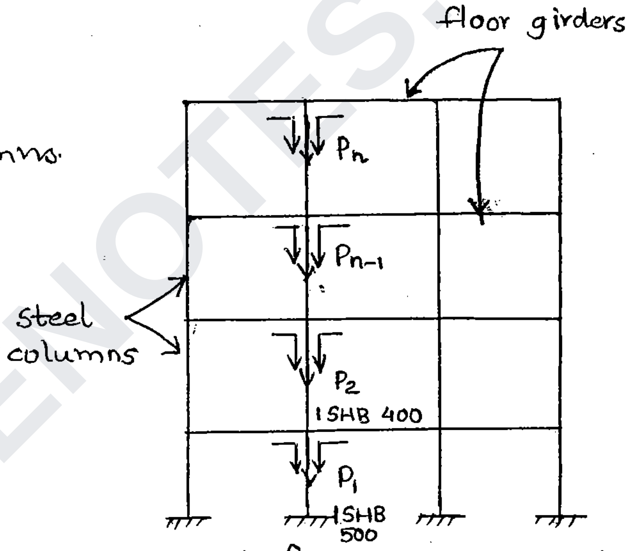
$$= \sqrt{\frac{2.5 \times 10.96\,(65^2 - 0.3 \times 65^2) \times 1.1}{250}}$$

$$= 18.88 \approx \underline{\underline{20\ \text{mm}}} \quad (> t_f = 11.6\,\text{mm})$$

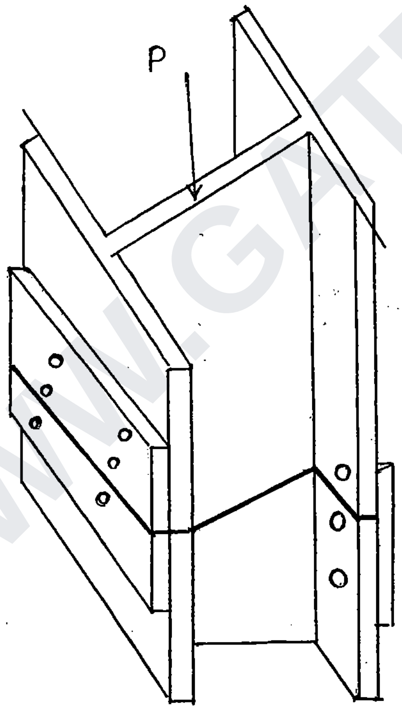
→ COLUMN SPLICE

$P_1, P_2, \cdots P_{n-1}, P_n$ factored column loads in various columns.

$$P_1 > P_2 > P_3 \cdots > P_{n-1} > P_n.$$

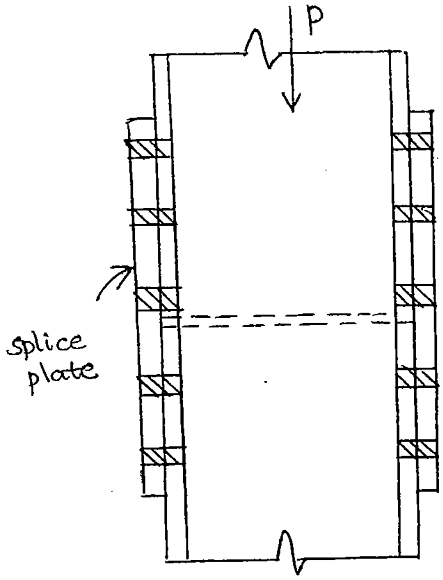


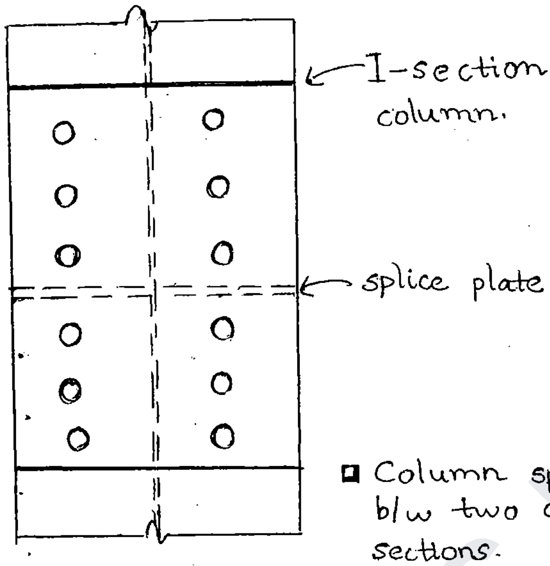
Steel frame for multi-storeyed building.



Column splice is a joint for steel column to be provided for extending length of column section. (length available from Indian Rolling mill is less) and also provided when two different sizes of columns are to be joined.

Column splice is should be designed as a short column and recommended to locate just above floor level.



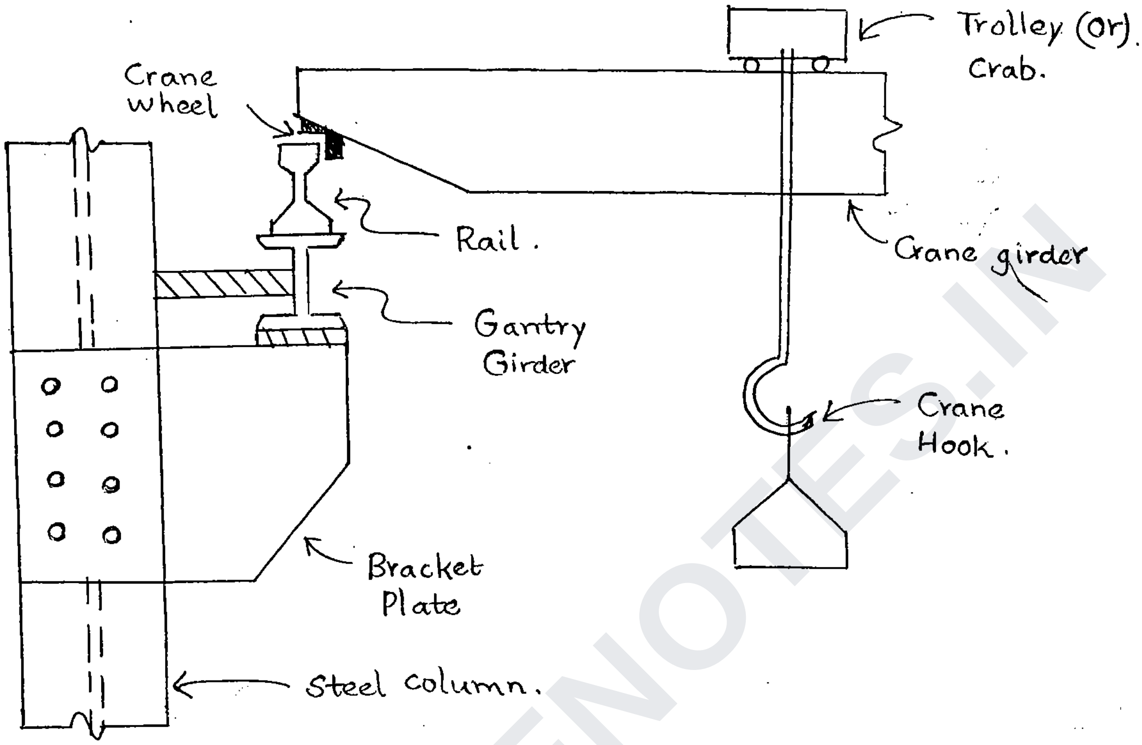


Column splice b/w two column sections.

If end of a column is machined or milled, theoretically no connecting system is required. But in practise, splice must be designed for 50 % factored column load.

3rd Oct,
FRIDAY

10. GANTRY GIRDERS



EOT Cranes :- Electrically Operated Overhead Travelling Cranes.

MOT or HOT Cranes :- Manually (or) Hand Operated Overhead Travelling Cranes.

→ Design loads on Gantry Girder:
- Vertical loads (Gravity Loads)
- Lateral loads (Surge Loads)
- Longitudinal loads (Drag Loads)
- Impact loads (Sudden Actions)

→ Additional loads on Girder due to Impact effect

(i) Vertical Loads	Additional loads
- EOT cranes	25% max. static wheel load.
- MOT cranes.	10% max. static wheel loads.

(ii) Lateral Loads

- EOT cranes
- MOT cranes

(iii) Longitudinal Loads

Additional Loads

10% (wt. of crab + wt. lifted crane)

5% (wt. of crab + wt. lifted crane)

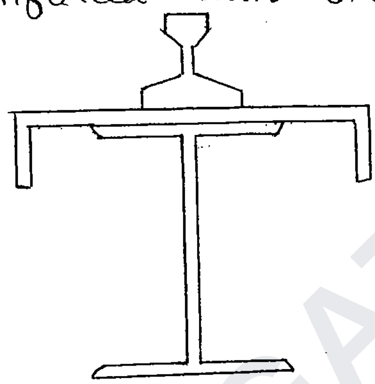
5% max. static wheel load.

→ Types of Sections

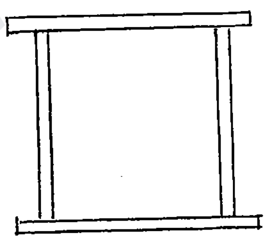
- ISWB : to resist more lateral loads.

$$(I_{yy})_{ISWB} > (I_{yy})_{ISLB/ISJB/ISMB/ISHB}.$$

- Reinforced with channel section.



- Box Girder : for more torsional resistance.



→ Limiting Deflections (IS 800: 2007)

	Δ limit
- For MOT (or) HOT cranes	L/500
- For EOT cranes with crane capacity upto 50 t or 500 kN	L/750
- EOT cranes with crane capacity more than 50 t (or) 500 kN	L/1000
- For other moving equipments like charging cars etc	L/600

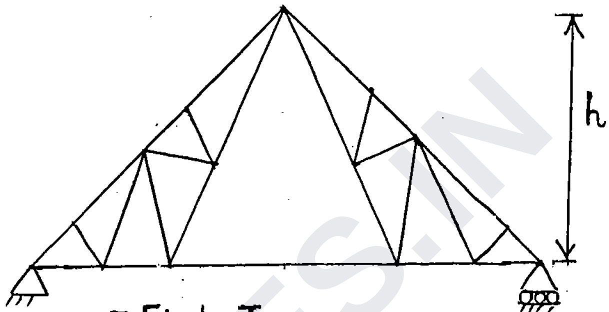
L → span of gantry girder

4th sept,
SATURDAY

11. ROOF TRUSSES

→ Selection criteria for Roof truss :

- Span of the truss.
- Pitch of the truss.



▫ Fink Truss

Pitch of the truss depends on :

(i) Type of roof covering material to be used for truss (like AC sheets, GI sheets, plastic sheets)

(ii) Lightening & ventilation requirement.

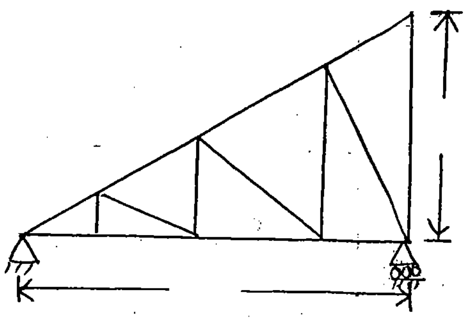
$$\text{Pitch} = \frac{\text{Rise}}{\text{Span}} = \frac{h}{L}$$

$$\text{Slope} = \tan\theta = \frac{\text{Rise}}{\text{Half span}} = \frac{h}{L/2} = \frac{2h}{L}$$

$\boxed{\text{Slope} = 2 * \text{pitch}}$ (for symmetric truss)

* North Light Roof Truss

- Spans upto 10 m
- Day light is main criteria for selection of North light roof truss.

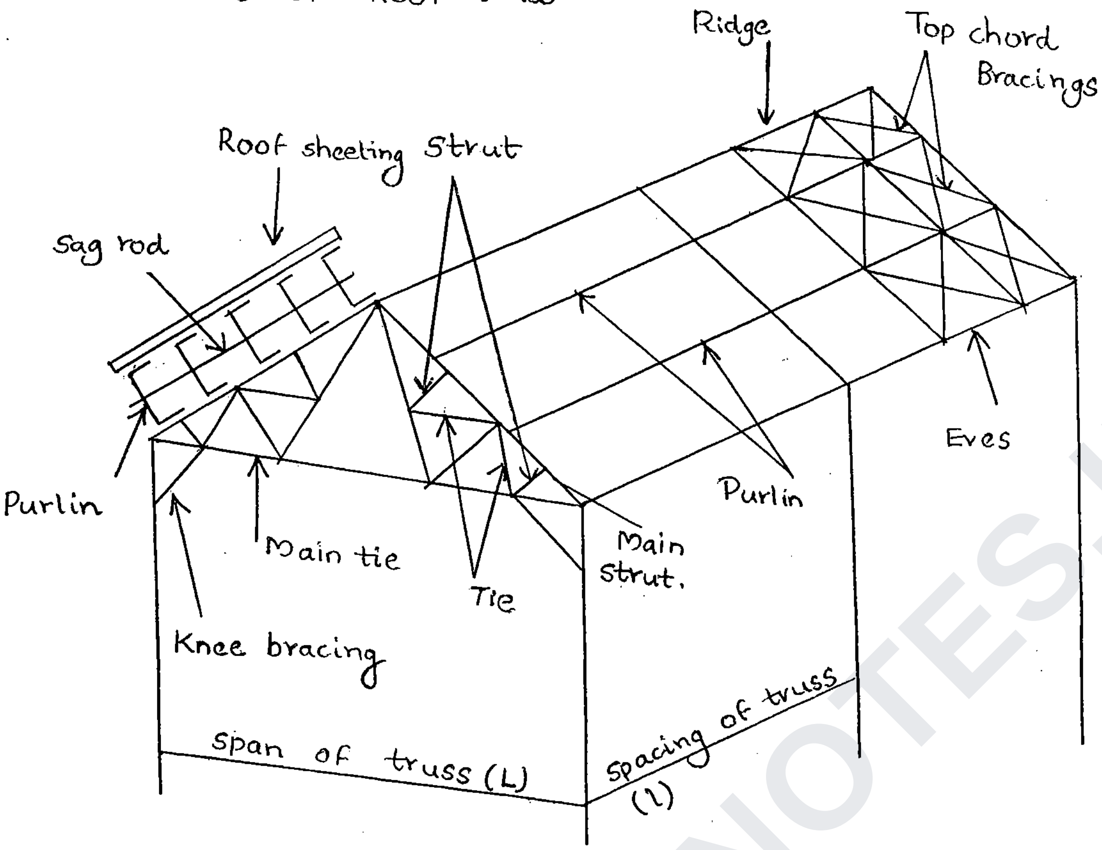


▫ North light Roof Truss

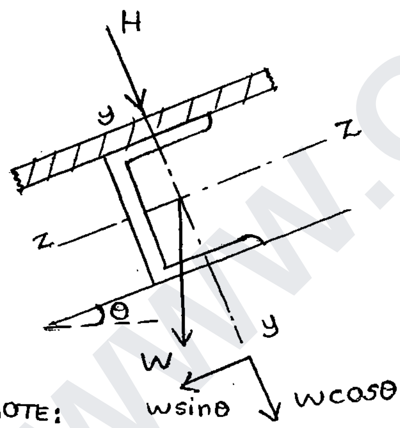
$$\text{Pitch} = \frac{h}{L}$$

$$\text{Slope} = \tan\theta = \frac{h}{L} \Rightarrow \boxed{\text{Pitch} = \text{Slope}}$$

→ Elements of Roof truss



⊙ Top Chord bracings are not required when masonry walls are provided.



$$M_{zz} = \frac{\gamma_L (H + w\cos\theta) l^2}{10}$$

$$M_{yy} = \frac{\gamma_L (w\sin\theta) l^2}{10}$$

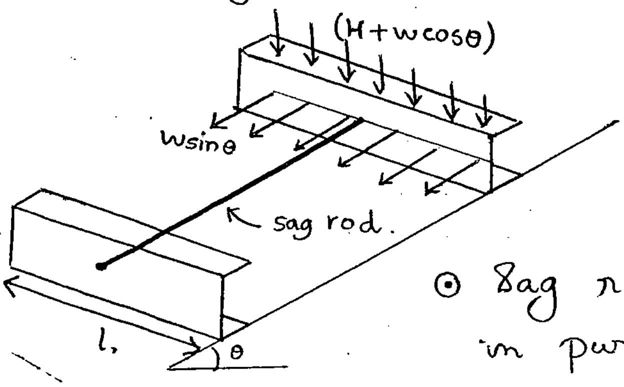
$w \rightarrow$ gravity load due to DL & LL (N/m or KN/m)

$H \rightarrow$ load due to wind pressure (N/m or kN/m)

$\gamma_L \rightarrow$ load factor.

NOTE:

⊙ Purlin member may be designed as simply supported beam or cantilever beam or continuous beam. IS 800: 2007 recommends to design as continuous beam subj to biaxial bending mome

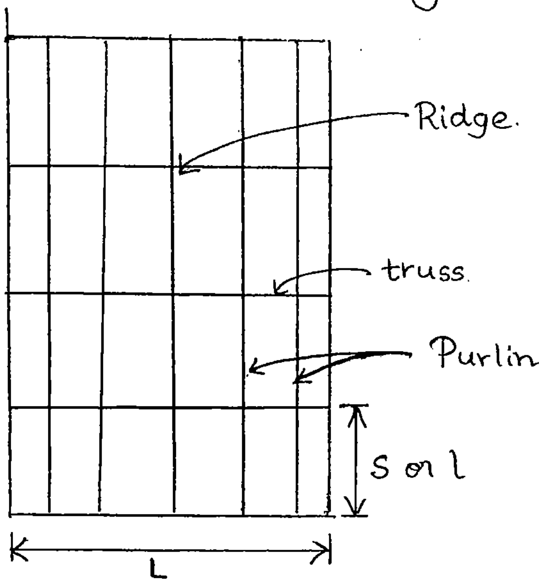


$$M_{yy} = \frac{\gamma_L w\sin\theta (l/2)^2}{10}$$

$$= \frac{1}{4} \gamma_L \frac{w\sin\theta \, l^2}{10}$$

⊙ Sag rod will minimise ~~slopes~~ deflection and BM in purlin member about <u>minor principal axis</u>

→ Economical Spacing of Truss (S or l)



$t \rightarrow$ cost of truss per unit area

$p \rightarrow$ cost of purlin per unit area

$r \rightarrow$ cost of roof sheeting per unit area.

$x \rightarrow$ total (or) overall cost of roof building per unit area.

$$x = t + p + r$$

- $t \propto \frac{1}{S} \Rightarrow t = \frac{C_1}{S} \quad \therefore C_1 = tS$

$$M = \frac{wl^2}{10} \Rightarrow p \propto S^2$$

- $p = C_2 S^2 \Rightarrow C_2 = \frac{p}{S^2}$

- As spacing increases, no: of joints b/w roofing sheets increase and as a result cost of erection increases.

$$r = C_3 S \Rightarrow C_3 = \frac{r}{S}$$

$$x = t + p + r$$

$$x = \frac{C_1}{S} + C_2 S^2 + C_3 S$$

To have minimum cost of roof building,

$$\frac{dx}{dS} = 0$$

$$\frac{d}{dS}\left(\frac{C_1}{S} + C_2 S^2 + C_3 S\right) = 0.$$

$$-\frac{C_1}{S^2} + 2C_2 S + C_3 = 0$$

$$\frac{C_1}{S^2} = 2C_2 S + C_3.$$

$$\frac{tS}{S^2} = \frac{2P}{S^2} \times S + \frac{r}{S} \Rightarrow \boxed{t = 2P + r}$$

Cost of truss per unit area = 2 * cost of purlin per unit area + cost of roof covering per unit area.

$$\boxed{l \text{ or } s = \frac{L}{3} \text{ to } \frac{L}{5}}$$

→ Design Loads on Roof truss

- design dead loads.
- design live (or) imposed loads.
- design wind loads.
- design snow loads.

* Design Dead Loads (IS 875 : 1987 Part I)

(i) Self weight of purlins.
(ii) Self weight of bracings.
(iii) Self weight of roof sheeting
(iv) Self weight of truss.

Self weight of truss $= 100\ N/m^2$ to $150\ N/m^2$ Plan area

$$= \left(\frac{L}{3} + 5\right) \times 10\ N/m^2 \text{ Plan area.}$$

(L = span of truss)

* Design Live (or) Imposed loads (IS 875:1987 Part II)

- Slope of roof, $\theta \leq 10$

LL $= 1500\ N/m^2$ (If access is provided for repair & maintenance)

$= 750\ N/m^2$ (if access is not provided for repair & maintenance)

– Slope of roof, $\theta > 10^\circ$

$$LL = \left(750 - 20(\theta - 10)\right) \text{ N/m}^2 \not< 400 \text{ N/m}^2$$

* Design Snow Load. (IS 875 : 1987 Part IV)

$SL = 25 \text{ N/m}^2$ per every 'cm' depth of snow.
$= 2.5 \text{ N/m}^2$ per every 'mm' depth of snow.

$\theta > 50^\circ$; Snow load need not to be considered.

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* Design Wind Load. (IS 875 : 1987 Part III)

V_b = Basic wind speed in m/s at a height of 10m from MSL

V_z = Design wind speed in m/s at a height z.

$$\boxed{V_z = K_1 K_2 K_3 V_b}$$

where $K_1 \rightarrow$ probability (or) risk factor.
$K_2 \rightarrow$ size, shape and structure factor.
$K_3 \rightarrow$ topography factor.

P_z = Design wind pressure at a height z of a structure.

$$P_z = K \cdot V_z^2$$

$$\boxed{P_z = 0.6\, V_z^2} \qquad (K = 0.6)$$

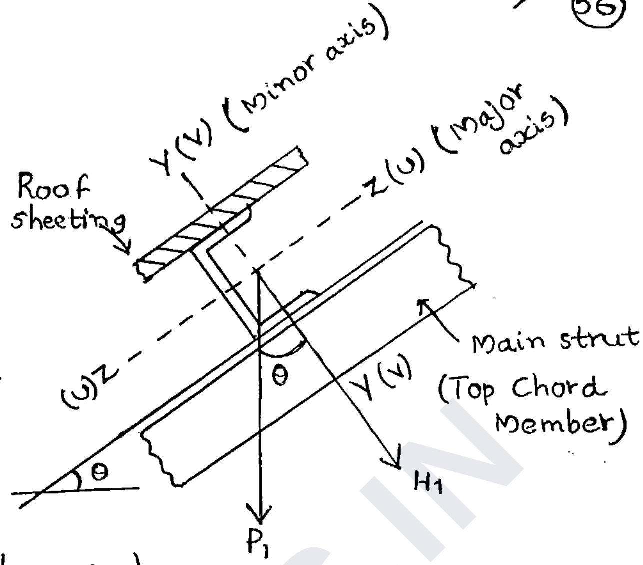
Design wind load $= (C_{pe} - C_{pi}) P_z \cdot A_e$.

where $A_e \longrightarrow$ exposed area.

$C_{pe} \rightarrow$ external wind pressure coeffecient (depends on slope of roof)

$C_{pi} \rightarrow$ internal wind pressure coeffecient (depends on degree of permeability (or) no. of openings in structure)

→ Design of Purlin



P_1 = Gravity load due to sheeting and live load. (in kN/m)

H_1 = load due to wind pressure in kN/m.

l = span of purlin (spacing b/w two adjacent trusses).

θ = slope of roof.

IS 800:2007 recommends to design a purlin as a continuous beam subjected to bi axial. (unsymmetrical) BMs.

Load along minor axis (YY axis)

$$P = \gamma_L (H_1 + P_1 \cos\theta) \longrightarrow M_{zz}$$

Load along major axis (ZZ axis)

$$H = \gamma_L (P_1 \sin\theta). \longrightarrow M_{yy}$$

Bending moment about major axis, $M_{zz} = \frac{PL^2}{10}$.

BM about minor axis, $M_{yy} = \frac{Hl^2}{10}$

* Deflection (or) Serviceability limits: (IS 800:2007)

(i) Brittle cladding (or sheet) $= \frac{L}{180}$
Eg: AC roofing sheet

(ii). Elastic cladding $= \frac{L}{150}$
Eg: GI, Plastic roofing sheet

- Design bending strength about major axis (M_{dz}).

$$M_{dz} = Z_{pz} \cdot \frac{f_y}{\gamma_{m0}}.$$

- Design bending strength about minor axis (M_{dy}).

$$M_{dy} = Z_{py} \cdot \frac{f_y}{\gamma_{m0}}$$

Z_{pz} & Z_{py} : Plastic section modulus of purlin about major & minor axis resptly.

- For safety of purlin, Interaction Equation as per IS 800 : 2007 must be satisfied:

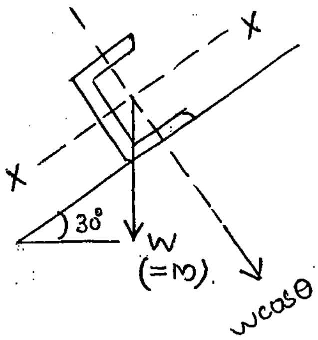
⊙ $\frac{M_{zz}}{M_{dz}} + \frac{M_{yy}}{M_{dy}} \leq 1.0$ $M_{dz} > M_{zz}$, $M_{dy} > M_{yy}$.

⊙ $\Delta_{cal} \leq \Delta_{limit}$; Δ_{limit} = limiting deflection.

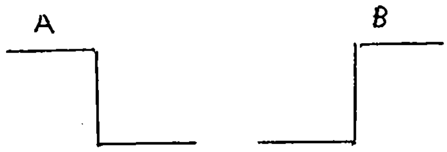
2 For w, $M_{xx} = M$

For $w\cos 30$, $M_{xx} = M\cos 30$

$$= \frac{\sqrt{3}}{2} M$$



3. $M_{dz} = Z_{pz} \frac{f_y}{\gamma_{m0}}$



$M_{dy} = Z_{py} \frac{f_y}{\gamma_{m0}}.$

Section modulus (Z_{pz} & Z_{py}). is independent of orientation of sections.

04. As per IS 800 : 1984, angle iron purlins were used

* Assumptions :

(i) Slope, $\theta \leq 30$ (exposed area for wind force gets minimised).

(ii) Bending moment about minor axis neglected.

(iii) LL = 750 N/mm^2.

08 Slope = 1

$\tan\theta = 1 \Rightarrow \underline{\underline{\theta = 45^\circ}}$

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09. PLATE GIRDERS

Beams are designed for bending moments (M) and sometimes shear force (V).

Plate girders are the major beams in a structure. For heavy loads and large spans, M will be very large.

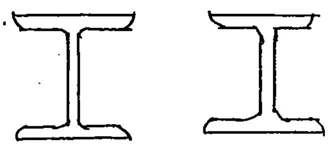
So to meet the design requirement of $M \leq M_d$, higher value of section modulus ($\propto I$), Z, is required. But the maximum depth for an I section given by Indian Rolling mills is 600 mm. So the following sections are considered:

$M_d = f(Z \to I)$.

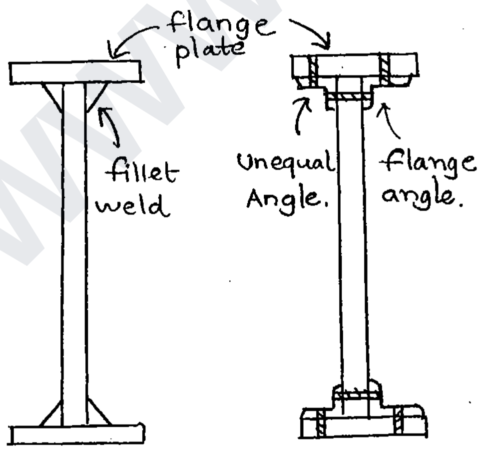
(i) Two I - sections placed side-by-side.

(ii) Plate Girder (spans 20 m–100 m)

(iii) Truss Girder (spans > 100 m).



→ Plate Girders



■ Welded PG

■ Bolted PG

Unequal angles are provided to:

(i) increase moment of inertia.

⊙ Weight of bolted/riveted plate girder $= \frac{W}{300}$ kN/m

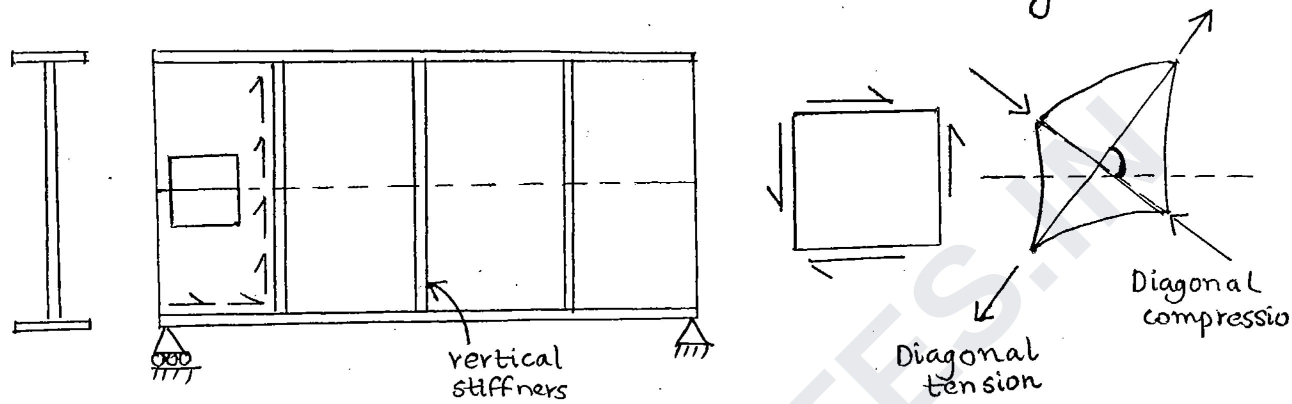
⊙ Weight of welded plate girder $= \frac{W}{400}$ kN/m

W → superimposed load in kN

– For plate girders, $\frac{d}{t_w}$ ratio will be very high, leading to local buckling failures before yielding failures

(i) Shear buckling Failure.

Also called Diagonal compression buckling failure.



⊙ No shear buckling failure in web when

$$\frac{d}{t_w} \leq 67\,\epsilon \;; \quad \epsilon = \sqrt{\frac{250}{f_y}}$$

⊙ Shear buckling can also be minimised by providing vertical stiffners so that one component of diagonal compression taken care of by vertical stiffners and another component by flange plates.

(ii) Horizontal (or) Longitudinal buckling failure.

Due to compressive bending stresses, web plate may have a chance to buckle. ($I_{zz} >>> I_y$). about minor axis. So to avoid this horizontal stiffners are provided.

(iii) Vertical (or) bearing buckling failure of web plate.

Stiffners are to be provided at concentrated loads or supports. Such stiffners are called Load bearing stiffners

19. For unstiffened web plate, no stiffener is required.

For Fe 450 grade steel, $f_y = 250$

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

$$\frac{d}{t_w} \leq 200\,\epsilon.$$

$$\therefore\ t_w \geq \frac{d}{200\,\epsilon}$$

$$\Rightarrow\ t_w \geq \frac{2000}{200 \times 1} = \underline{\underline{10\text{ mm}}}$$

20. $I_s \geq d_2\, t_w^3$

$$= 2000 \times 10^3$$

$$= \underline{\underline{2 \times 10^6}}\ \text{mm}^4$$

→ Elements of a Plate Girder

- Web plate
- Flange plate with flange angles for bolted/riverted plate girder.

Flange plates only for welded plate girder.

* Stiffeners :

(i) Intermediate Stiffeners

a) Vertical or Stability or Transverse stiffeners.

b) Horizontal or longitudinal stiffeners.

(ii) Bearing Stiffeners.

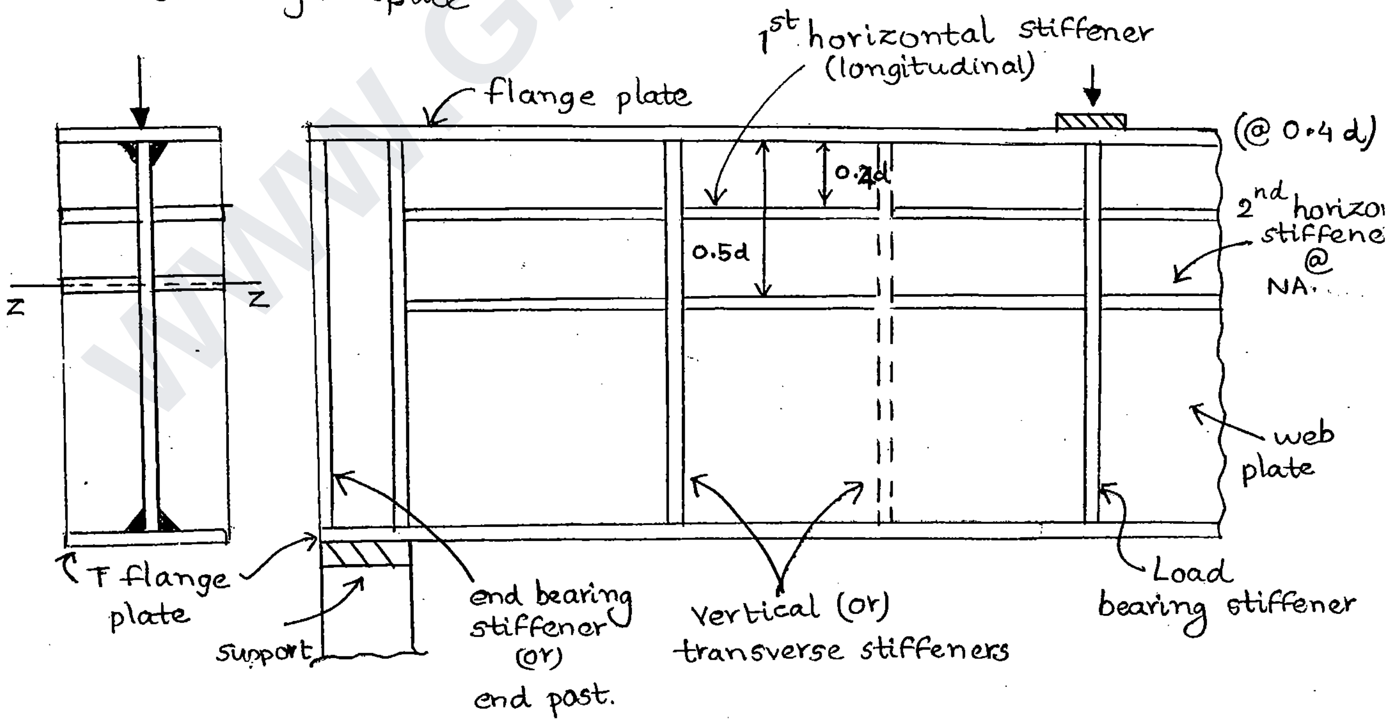
a) Load bearing Stiffeners (under conc. loads).

b) End bearing stiffeners. (at supports).

* Splices:

(i) Web Splice.

(ii) Flange Splice



Length of plate available from Indian Rolling Mills is only 7.5 m. But plate girders are built for a length of 20 m. So web splice is used to join web plates and flange splice is used to connect flange plates.

Web plates are provided to support SF. So web splices are not provided at points of max. SF like supports, under conc loads etc. Similarly, flange plates are designed to support the moments. So flange splices are not provided at max. BM locations like under conc. loads. Splices at these critical locations increase the cost and no: of bolts and rivets.

→ Web Plate.

Economical depth of web plate (IS concept based on minimum area of steel (min. wt) to be provided for girder)

$$\boxed{d = \left(\frac{M_z K}{f_y}\right)^{1/3}} \quad ; \quad K = \frac{d}{t_w}$$

M_z = design bending moment
f_y = yield strength of material.

* Min. thickness of Web Plate (should meet serviceability criteria & compression flange buckling requirement)

- min thickness of web plate (based on serviceability criteria)

a) No vertical (Transverse) stiffeners required.

- $\frac{d}{t_w} \leq 200\,\epsilon$ (web connected to flange along both longitudinal edges)

- $\frac{d}{t_w} \leq 90\epsilon$ (web connected flange along one longitudinal edge only)

b) When vertical stiffeners are to be provided.

- $\frac{d}{t_w} \leq 200\,\epsilon_w$ (for $3d \geq c \geq d$)

- $\frac{c}{t_w} \leq 200\,\epsilon_w$ ($0.74\,d \leq c < d$)

- $\frac{d}{t_w} \leq 270\,\epsilon_w$ ($c < 0.74\,d$) ; $\epsilon_w = \sqrt{\frac{250}{f_{yw}}}$

When $c > 3d$, plate girder to be treated as unstiffened.

c = spacing b/w vertical stiffeners.

d) When vertical stiffeners + 1st horizontal stiffener + 2nd horizontal stiffener at NA.

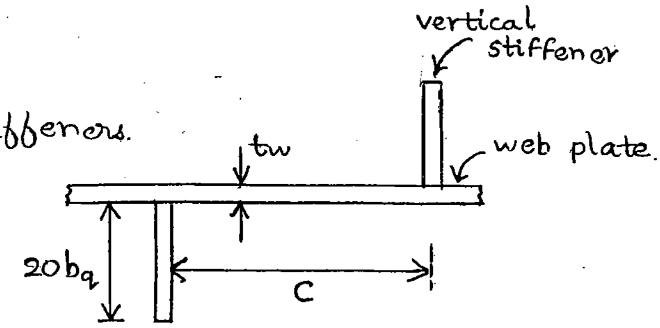
- $\frac{d}{t_w} \leq 400\,\epsilon_w$

→ Stiffeners

- Vertical (or) Transverse (or) Stability Stiffeners.

b_q → outstand of stiffener

c → spacing b/w vertical stiffeners.



NOTE:

Vertical stiffeners are to be provided to eliminate shear buckling failure in web plate. The shape of the stiffener must be angle section for bolted plate girder and flat section.

for welded plate girder.

- minimum MI required (I_s)

$$I_s = 0.75\, d\, t_w^3 \quad \left(\text{when } \frac{c}{d} \geq \sqrt{2}\right)$$

$$I_s = \frac{1.5\, d^3 t_w^3}{c^2} \quad \left(\text{when } \frac{c}{d} < \sqrt{2}\right)$$

- 1st horizontal (longitudinal) stiffener @ $\frac{2}{5}d$ from compression flange to NA.

  Minimum MI required : $I_s \geq c\, t_w^3$

- 2nd horizontal stiffener at NA.

$$I_s > d_2 t_w^3$$

where d_2 = 2 * distance from compression flange to NA (=d)

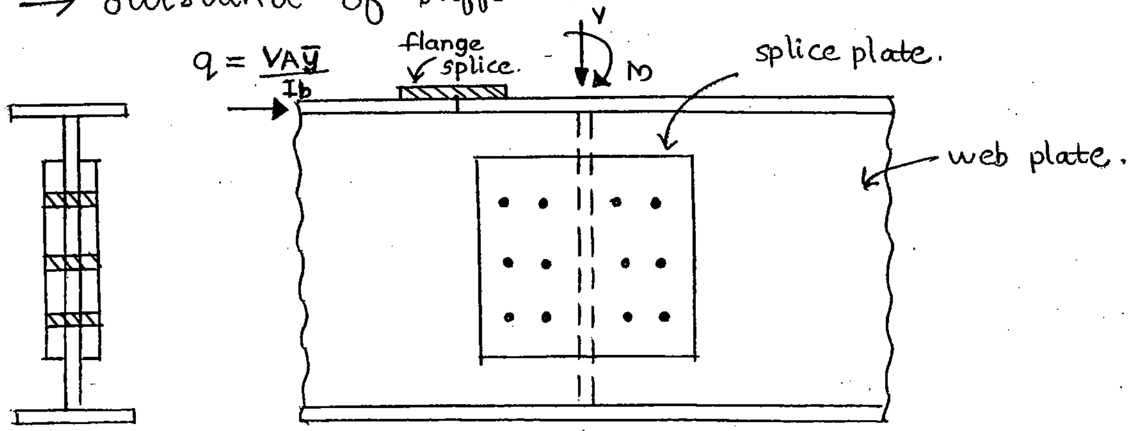
NOTE:

The connection b/w vertical stiffener to the web plate, horizontal stiffener to the web plate should be designed min. shear not less than $\frac{t_w^2}{5 b_s}$

$$\text{Minimum shear} = \frac{t_w^2}{5 b_s} \quad (\text{kN/mm})$$

$t_w \rightarrow$ thickness of web plate

$b_s \rightarrow$ outstand of stiffener.



web splice

→ Web Splice.

- it is recommended to locate web splice at a point away from max. shear.

- web splice must be designed for shear force & BM at spliced locations.

- web splice is a joint for web plate to be used for extending length of web plate.

→ Flange Splice.

- It is joined for flange plate for extending length of flange plate

- flange splice should not be located at a point of max. BM.

- flange splice should be designed for horizontal shear (axial load to the flange plates) due to transverse loads

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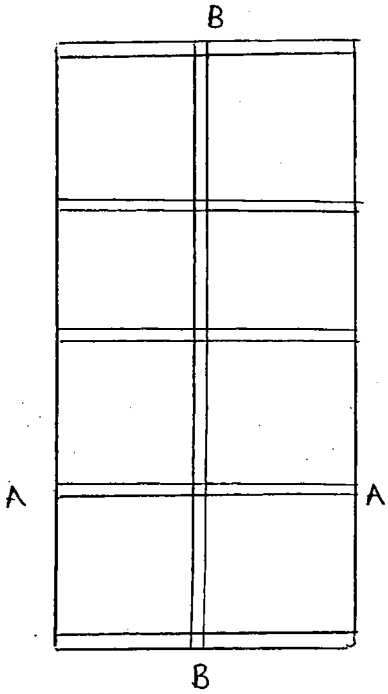
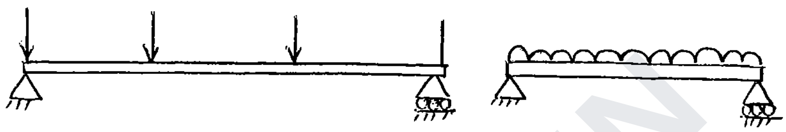
12th Oct,
SUNDAY

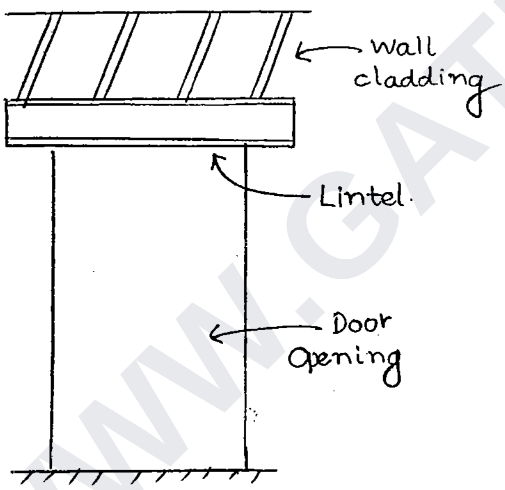
BEAMS



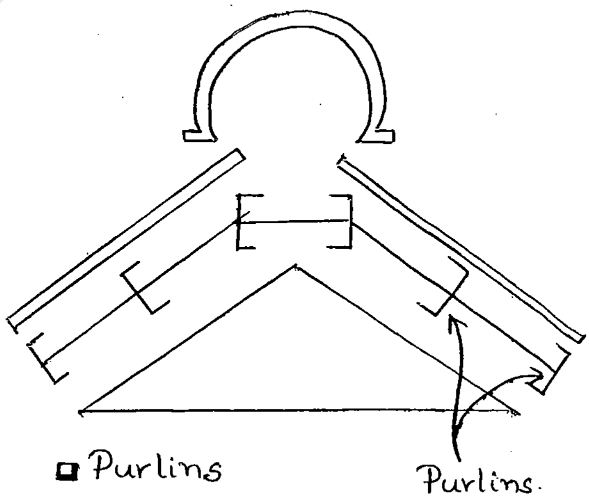


Beam B–B
(FLOOR BEAM).

Beam A–A
(JOIST)



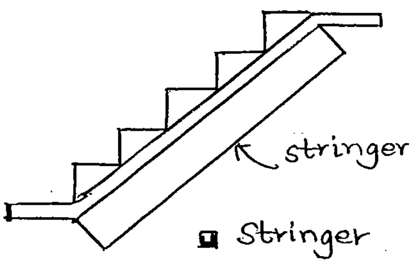
■ Lintel.



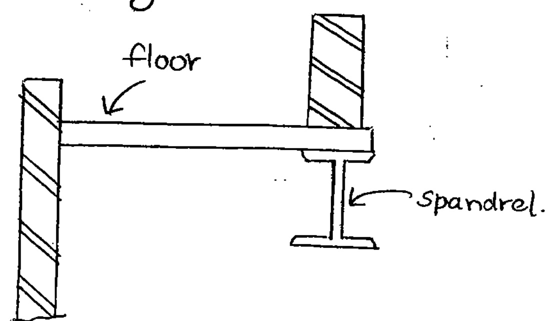
■ Purlins

* Girder – major beam.

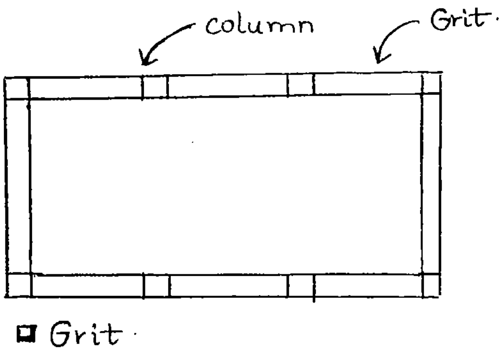
Eg: Floor beam in an industrial building.



■ Stringer



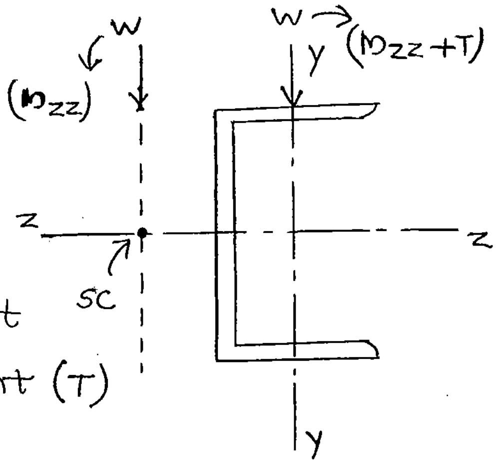
■ Spandrel.



* Header — transverse loaded structure provided in well openings of stairs.

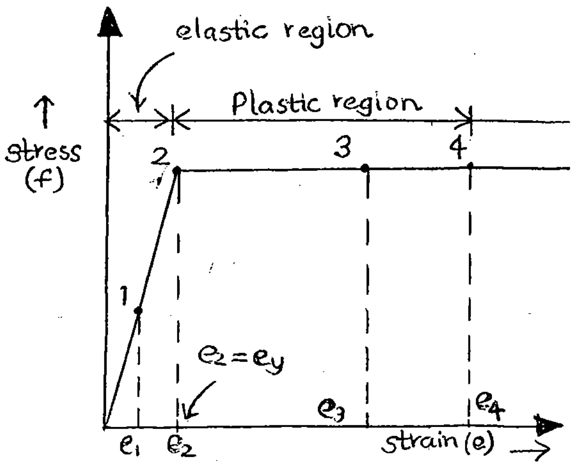
Bending moment is a function of loading and span.
$M = f(w, l)$ or $M = f(w, l^2)$. But tranverse loaded structures like grit, purlin, lintel, etc are secondary beams with shorter spans. So the design BM will be less for them.
∴ they are designed with channel sections although I-section are the best beam sections; as I-sections becomes uneconomical here.

For channel sections, load must pass through the SC to produce simple bending along ZZ axis. If they act at a different point, it will cause twisting moment (T) and BM about ZZ axis (M_{zz}).



→ Behaviour of Beam in Flexure.

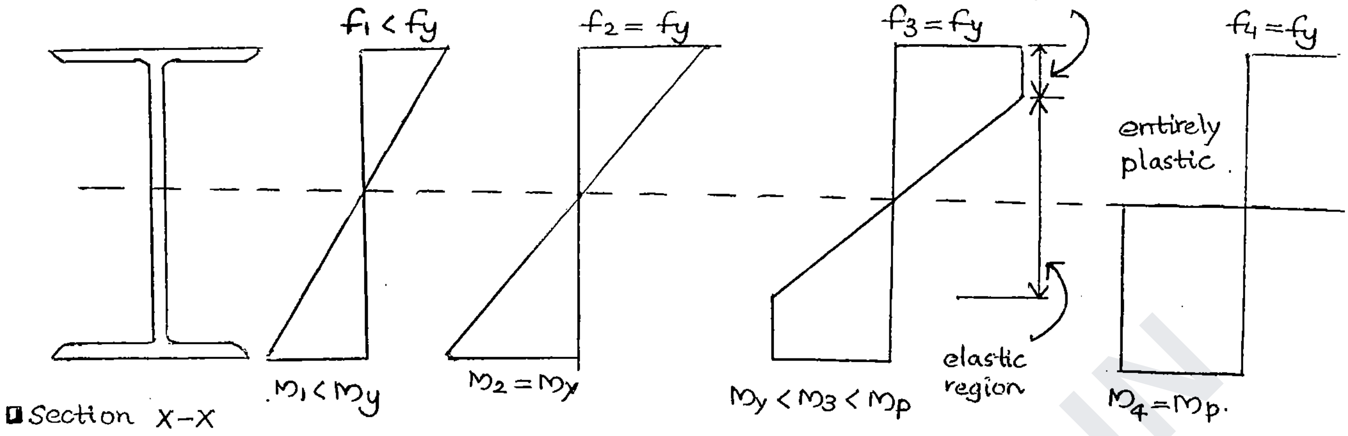
Idealised stress strain curve of mild steel is also known as elasto-plastic strain curve.

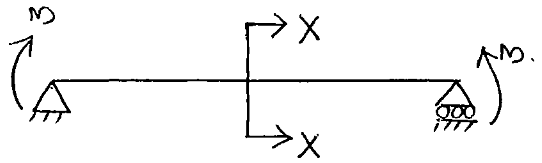


$$\frac{M}{I} = \frac{f}{y}$$

$$M_e = f\frac{I}{y} = f Z_e$$

Bending Stress Distributions corresponding to Various points on stress strain curve:-





$$M_p = f_y Z_p.$$

$$\boxed{M_p = (1.10 \text{ to } 1.20) M_y}$$

→ Classifications of Sections :

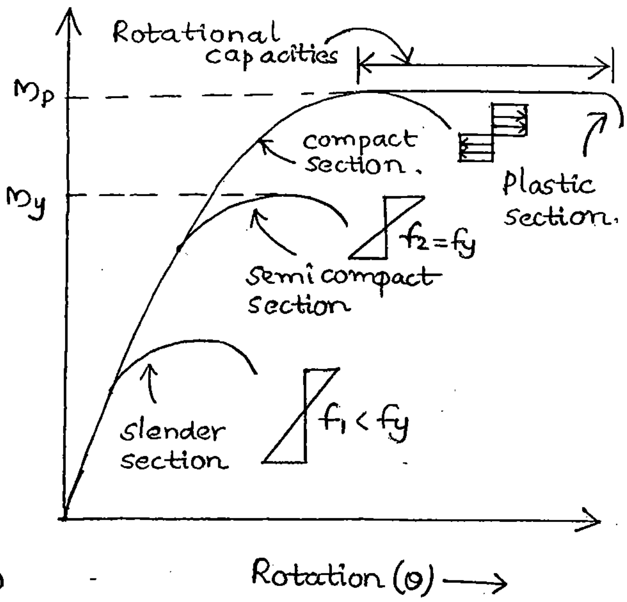
Based on yield moment (M_y), plastic moment (M_p), $\frac{d}{t_w}$ ratio of flange & web, the four classes of sections are:

(i) Plastic section.

(ii) Compact Section.

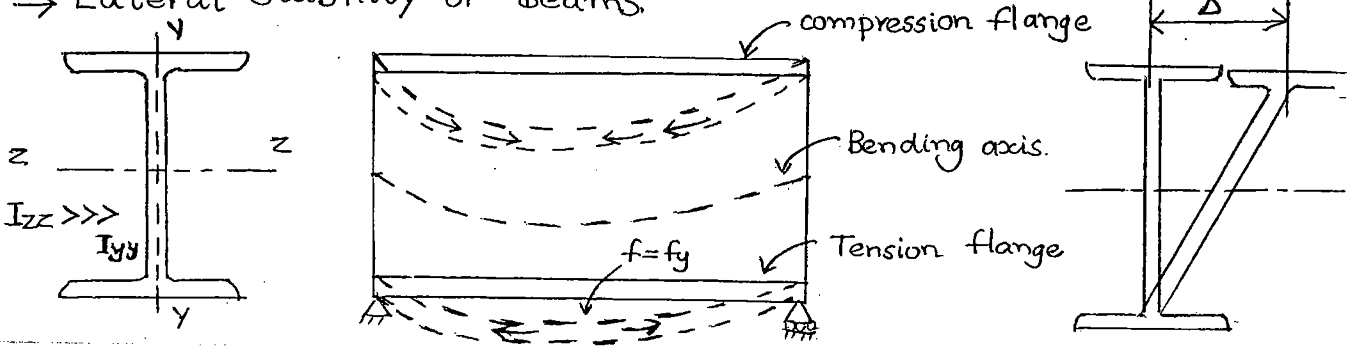
(iii) Semi compact Section (Non compact).

(iv) Slender Section.



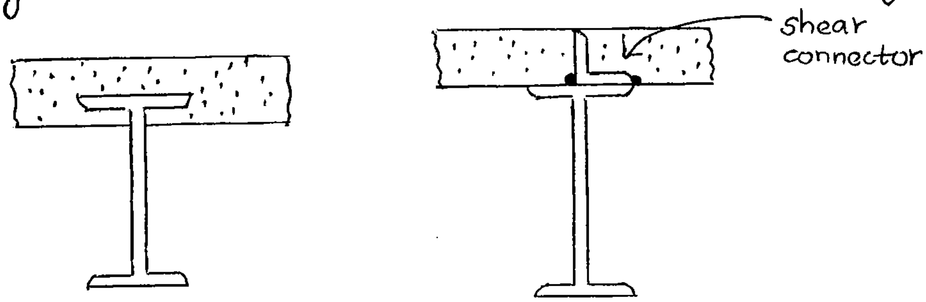
I-sections and channel sections have plastic moment of resistance whereas plate girders do not develop plastic moment of resistance.

→ Lateral Stability of Beams.



(i) Laterally Restrained Beams (or) Laterally Supported Beams

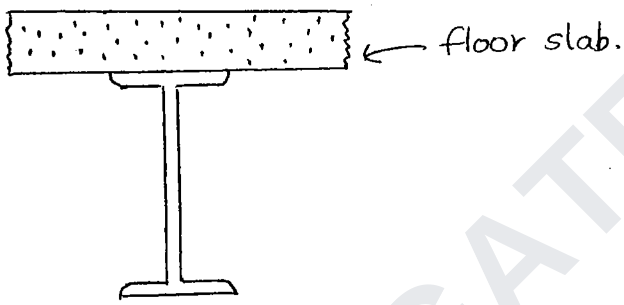
Here compression flange of beam is not affected by lateral or lateral torsional buckling.



~~(ii) Lateral Stability of Beams.~~

(ii). Laterally unrestrained (or) laterally unsupported beams.

Compression flange of beam section is affected by lateral (or) lateral torsional buckling.



→ Design Criteria of Beam:

- Flexural Strength Criteria (Design for BM (M)).
- Shear Strength Criteria. (Design for SF (V)).
- Deflection Criteria. (Serviceability Criteria)
- Check for secondary effects like web buckling & web crippling failures.

* Design. flexural (or) Bending Strength. (M_d).

a) Laterally Restrained Beam. (No lateral or lateral torsional buckling).

b) When $\frac{d}{t_w} \leq 67\,\epsilon$ (No shear buckling of web).

c). Low shear case (when $V \leq 0.6\,V_d$)

V → design shear force

Vd → design shear strength of a beam.

$$\frac{f_y}{\sqrt{3}} = 0.577 f_y = 0.6 f_y$$

$V > 0.6\, V_d$ (High Shear Case)

$$\boxed{M_d = \beta_b Z_p \frac{f_y}{\gamma_{mo}} \leq 1.2 Z_e \frac{f_y}{\gamma_{mo}}}$$

(for simply supported beams)

$\beta_b = 1.0$ (for plastic & compact sections)

$= \frac{Z_e}{Z_p}$ (for semi-compact sections). $\boxed{M \leq M_d}$

$M_d = Z_e f'$ (f' = reduced bending stress)
(for slender section).

For semi compact sections,

$$M_d = \frac{M_y}{\gamma_{mo}} = \frac{f_y}{\gamma_{mo}} \cdot Z_e . \qquad \left(\Rightarrow \beta_b = \frac{Z_e}{Z_p}\right).$$

For laterally unrestrained beams,

$$M_d = \beta_b \cdot Z_p \cdot f_{cd} .$$

f_{cd} → compressive strength of flange of a beam which is calculated by Perry Robertson equation.

f_{cd} depends on slenderness ratio.

Slenderness ratio for compression flange of a beam is limited to 300.

* Design Shear Strength of the beam (Vd)

a) Lateral restrained (or). Supported beams.

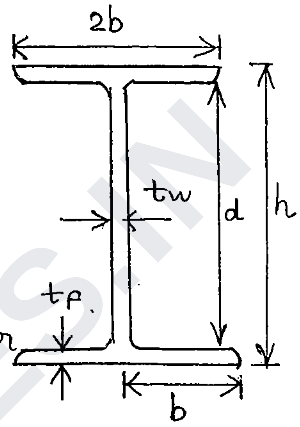
b) when $\frac{d}{t_w} \leq 67\epsilon$ (No shear buckling of web).

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

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

V_p = Plastic shear strength of the section

$$= \text{Shear area} * \frac{f_y}{\sqrt{3}}$$



$$\therefore \quad V_d = \text{Shear area} * \frac{f_y}{\sqrt{3} \cdot \gamma_{mo}}$$

For rolled I-section :

Shear area = $h \cdot t_w$ (about major axis)

For welded I-section :

Shear area = $d \cdot t_w$ (about major axis).

* Limiting Deflections : (As per IS 800 : 2007).

– For simply supported beam

$$\Delta_{limit} = \frac{\text{Span}}{240} \text{ (Elastic cladding)}$$

$$= \frac{\text{Span}}{300} \text{ (Brittle cladding)}$$

– For cantilever beam = 2* simply supported beam

$$\Delta_{limit} = \frac{\text{Span}}{120} \text{ (Elastic cladding)}$$

$$= \frac{\text{Span}}{150} \text{ (Brittle cladding)}$$

$$\boxed{\Delta_{cal} \leq \Delta_{limit}}$$

Q64.

30. $V_d = \text{Shear area} \times \frac{f_y}{\sqrt{3} \cdot \gamma_{mo}} = 500 \times 10.2 \times \frac{250}{\sqrt{3} \times 1.1} = \underline{\underline{669.20 \text{ kN}}}$