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UNIT - I

MATERIAL AND SPECIFICATION (1-2M)

Construction material for Civil Engineering structures

Concrete
- → Plain cement concrete (PCC)
- → Reinforced cement concrete (RCC)
- → Prestressed cement concrete (PCC)

Advantages of structual steel sections Vs Concrete membe

1. Steel sections having a High strength than concrete.
2. More economical
3. $\left(\frac{\text{Strength}}{\text{weight}}\right)_{\text{structural steel section}} > \left(\frac{\text{Strength}}{\text{weight}}\right)_{\text{concrete membe}}$
4. Fast and Rapid construction is possible only with stee
5. Ductile material (steel) (Min. percentage of elongation of steel is

Ductility can be measured by percentage of elongation and percentage of area reduction.

$$\text{percentage of elongation} = \frac{\text{change in length}}{\text{Original length}} \times 100$$

$$= \frac{\Delta l}{l} \times 100$$

percentage of elongation $> 15\%$ – Ductile material

$> 5-15\%$ – Intermediate ductile materi

$< 5\%$ – Brittle material

6. 100% scrap value
7. Overall cost of construction is less.
8. Easy modification (or) Repairs
9. High quality and realability

Safety norms :-

Working stress method – Factor of safety
- → concrete = 3
- → steel = 1.7 – 1.

Limiting stress method – Partial safety factors (γ_m)
- → concrete = 1.5
- → steel = 1.15

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Steel :-

Iron (By % of wt. >98%)	+	small % of carbon (By % of wt. 0.1% - 1.1%)	+	Sulphur, Manganese, Silicon, phosphorous, Nickle, Chrome etc	→	Steel

Mechanical properties :-

1. yield strength (f_y)
2. Ultimate tensile strength (f_u)
3. Hardness
4. Ductility
5. Toughness

→ Mild steel Fe 250 (carbon % = 0.23) → Low carbon steel

→ High Tensile steel Fe 415, 500 (carbon = 0.27%) → Medium carbon steel.

Classification of Steel :-

Based on amount of carbon content :-

1. Low carbon steel (0.1 - 0.25% of carbon by % of weight)

   Ex :- Mild steel bars used in RCC constructions, structural steel sections used in steel buildings (or) Bridge constructions.

2. Medium carbon steel (0.25 - 0.6 % carbon)

   Ex :- High tensile steel, Rail, tyre, hammer etc.

3. High carbon steel (0.6 % - 1.1 % carbon)

   Ex :- Stone masonry tools, drills, punches etc.

** Note :-

By increasing amount of carbon content yield strength of steel (f_y), ultimate tensile strength of steel (f_u), hardness of steel increases. However it reduces ductility and toughness of steel.

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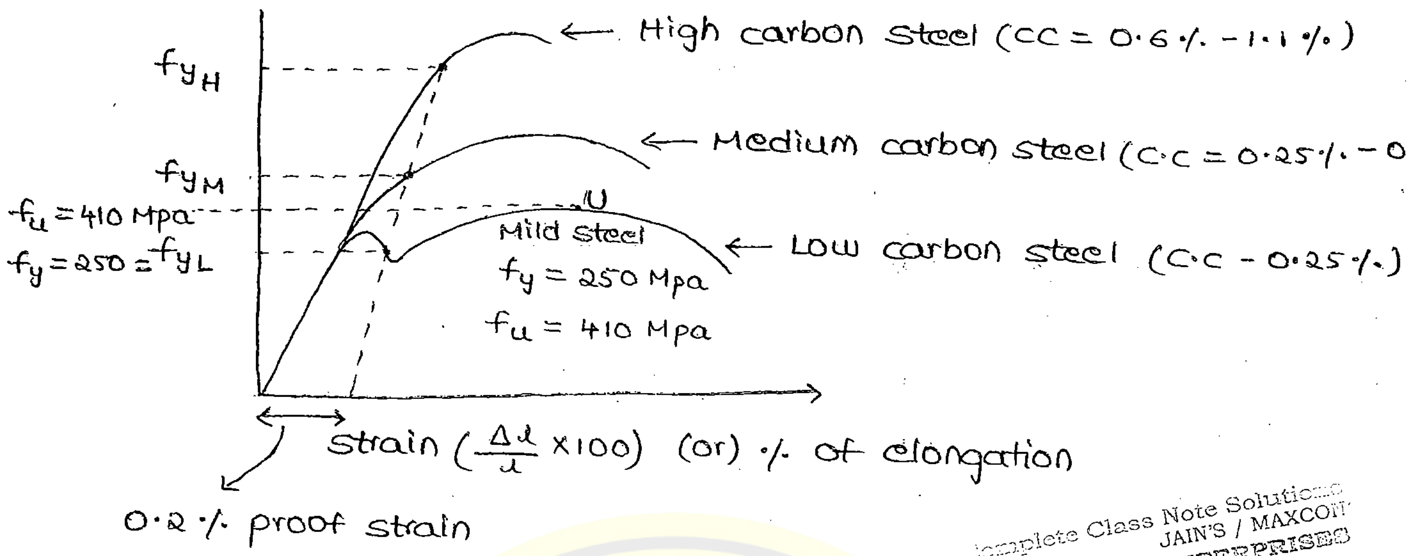
Stress strain curve :- 2



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General classification of steel :-

In general steel may be two types.

1. Reinforcing steel
2. Structural steel

Reinforcing steel :-

It is used as Reinforcing bars in R.C.C. constructi Designated by its yield strength.

EX:- Fe 250, Fe 415, Fe 500 etc.

Fe = steel
250 = yield strength of stee

Structural steel :-

It is used in steel buildings, bridges etc. Designad by its ultimate tensile strength.

EX:- Fe 410, Fe 570 etc.

Codes and standards :-

IS 800
- → 1956 - 1962 (1^{st} code book published)
- → 1984 (2^{nd}, revised, W.S.M)
- → 2007 (3^{rd}, revised, L.S.D)

} code of practise for use of structui steel in general building constructio

IS 875 : 1987 (five parts) code of practise for design loads fo building and structures.

IS Hand book no.1 (steel table)

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Rolled steel sections:-

1. Rolled steel I-sections or beam sections
2. Rolled steel channel section
3. Rolled steel Tee section
4. Rolled steel Angle section
5. Rolled steel tube section
6. Rolled steel bars
7. Rolled steel flats
8. Rolled steel plates
9. Rolled steel sheets
10. Rolled steel strips

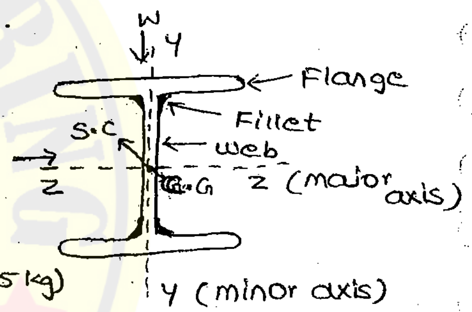
1. Rolled steel I-sections (or) Beam sections:-

An I-section may be designated by its depth and weight. C.G and S.C lies on same point.

Eg:- An ISLB 500 at 735.8 N/m

Overall depth of I-section : 500 mm (outer - outer)

Weight per running meter : 735.8 N (≈75 kg)



C.G = centre of gravity ~~(or) S.C~~

S.C = shear centre (or) centre of flexure

Note:-

For same weight (same sectional area) I-section provides large moment of inertia or sectional modulus about major axis (z-z axis). Hence I-section is best section or most efficient section for a beam or girder (major beam).

To support transfers loads which are parallel to the minor axis, more moment of inertia or section modulus required, about z-z axis.

To support lateral loads which are parallel to the major axis more moment of inertia or section modulus required about yy-axis.

I-section is arranged 90-95% of moment will be taken care by flanges and web will support 90-95% of SF.

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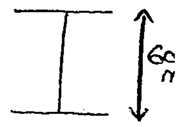
The junction between Flange and web is called Fille will minimise stress concentration due to heavy concentrated loads. More steel is provided at Fillet such that we can avoid the "web crippling failure" in I-beam. The applied load is passing through shear centre then "twisting" will not occur.

Types of I-sections:-

1. Indian Standard Junior beam (ISJB)
2. Indian standard Light beam (ISLB)
3. Indian Standard Medium weight beam (ISMB)
4. Indian standard wide flange beam (ISWB)
5. Indian standard Heavy weight beam (ISHB)



Maximum depth available is 600 mm (outer - outer)

Note:-

ISWB section has more lateral Bending strength or buckling strength as compared with its counter parts (ISLB, ISMB, ISHB)

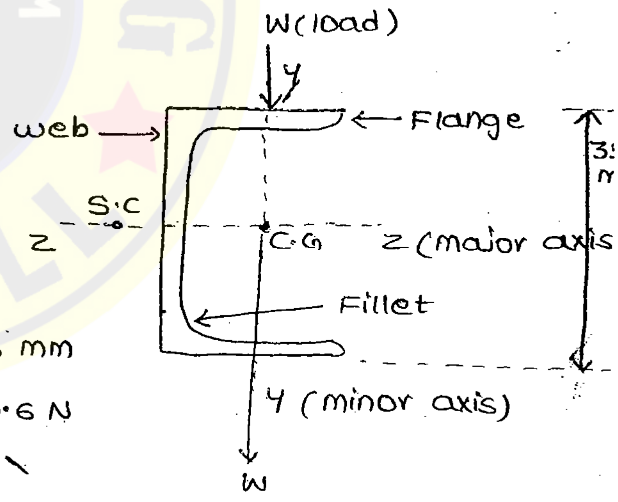
2. Rolled steel channel section:-

A channel section may be designated by its depth and weight.

Ex:- ISLC 350 at 380.63 N/m

Overall depth of channel : 350 mm

weight per running meter : 380.6 N



Types of channel sections:-

1. ISJC (Junior channel)
2. ISLC (Light weight channel)
3. ISMC (Medium weight channel)
4. ISMCP (Medium weight channel with parallel flanges)
5. ISGC (Gate channels)

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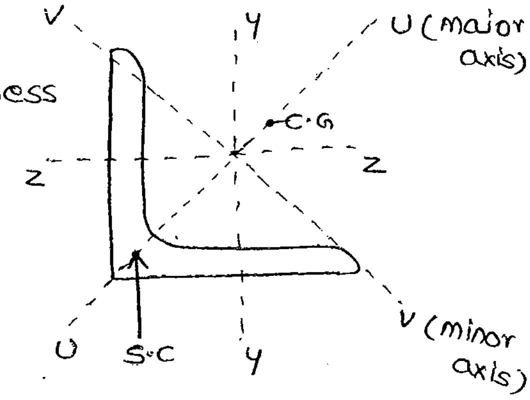
Moment of Inertia is more at z-z axis. so it is called Major axis. M.I is less at y-y axis (only one side area then it is called Minor axis. The applied load is not passing to shear centre so twisting will occur.

Rolled steel angle section (or) L-section :-

An angle section may be designated by its width and thickness

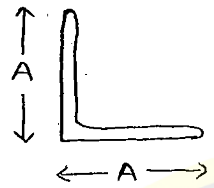
EX :- ISA 100 X 100 X 10



Types of Angle sections:-

1. Equal angles (EA)

Designated as ISEA (or) ISA

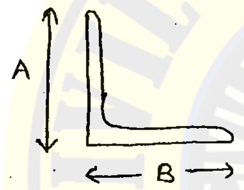


EX :- ISEA 100 X 100 X 10 (Equal angles)

It is a best section of compression member. Radius of gyration is more.

2. Unequal angles

Designated as ISA (Angles)

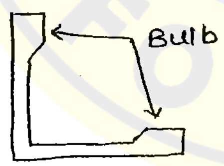


EX :- ISA 125 X 75 X 10

unequal angles are best suited for tension members.

3. Bulb angles

Designated as ISBA (Bulb angles)



Critical load (or) compressive strength of a member ($P_{c,r}$):- (Euler's theory)

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

$$P_{cr} = \frac{\pi^2 E A}{\left(\frac{L}{r}\right)^2}$$

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

$$r^2 = \frac{I}{A}$$

$$\frac{\text{Effective length}}{\text{Least radius of gyration}} = \frac{L}{r} = \text{slenderness ratio.}$$

Radius of gyration is more, slenderness ratio is less then P_{cr} value will be more.

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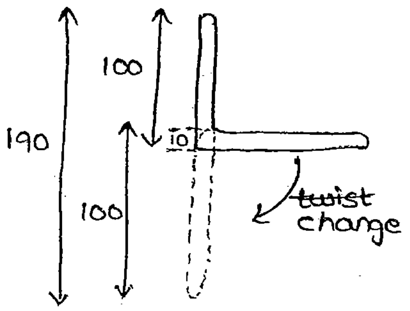
Equal angles

ISA 100 x 100 x 10



$A = 190 \times 10$

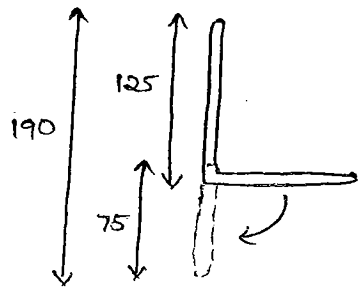
$= 1900$

$V = A \times L \Rightarrow V \times \gamma = \text{weight}$

Radius of gyration $= 19.1$ mm about minor axis

Unequal angles

ISA 125 x 75 x 10



$A = 190 \times 10$

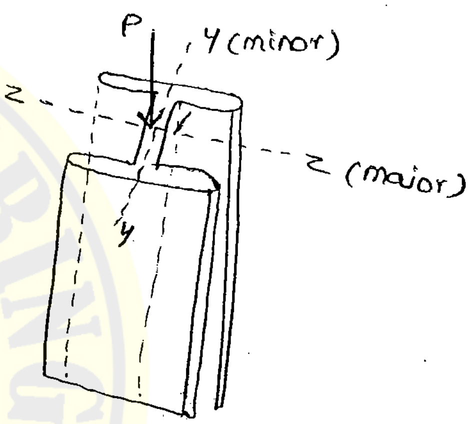
$= 1900$

Radius of gyration $= 16.4$ mm about minor axis

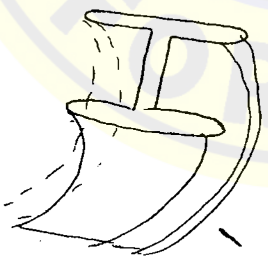
$$(P_{cr})_{zz} = \frac{\pi^2 EA}{(L/r_{zz})^2}$$

$$(P_{cr})_{yy} = \frac{\pi^2 EA}{(L/r_{yy})^2}$$

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



If $P > (P_{cr})_{yy}$ then section will be buckled



Note:-

1. Equal angle section provides higher minimum radius of gyration than unequal angle section, for same sectional area or weight. Hence equal angle section will have more axial compressive strength than unequal angle section.

2. Unequal angle section ~~longe~~ with its longer length connecte to gusset plate will provides more axial tensile strength than equal angle.

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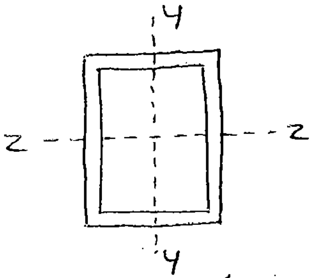
Rolled steel tube sections:-

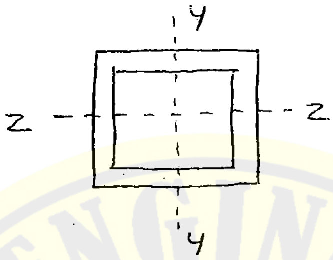
Tube sections are designated by its outside dimension and thickness

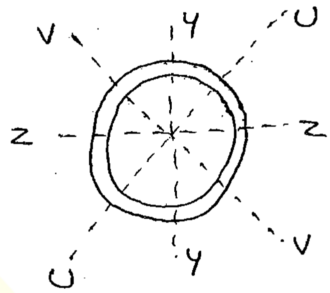
I_p = polar moment of inertia.

Types of tube sections

- Rectangular Hallow section (RHS)
- Square Hallow section (SHS)
- Circular Hallow section (CHS)





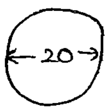


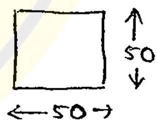
$I_{zz} = I_{yy}$

$I = I_p = I_{xx} = (I_{zz} + I_{yy})$

Rolled steel bars are two types

- Indian standard Round Bars Designated as ISRO
  Ex:- ISRO 20
- Indian standard square bars Designated as ISSQ
  Ex:- ISSQ 50





*** Note:-

1. Closed sections (Box sections) will provides higher torsional strength (or) twisting moment carrying capacity than open section with same sectional area (or) weight.

2. Solid circular sections and hallow circular sections are best sections (or) most efficient sections as a compression member such as column (or) strut.

For same weight (same cross sectional area) hallow circular section provides large value of radius of gyration than solid circular section. Hence Hallow circular sections are best sections as a compression member than solid circular section.

$\therefore (P_{cr})_{hallow} > (P_{cr})_{solid}$

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Rolled steel Flats (when thickness, $t > 5$ mm):-

Flat sections may be designated as its width and thickness.

Ex:- 100 ISF 10

$\longleftarrow B = 100 \longrightarrow$, $t = 10$ mm

Rolled steel plates (when $t \geqslant 5$ mm):-

Plates may be designated by its length, width and thickness.

Ex:- ISPL 7500 x 2500 x 10 mm

L x B x t

Rolled steel sheets (when $t < 5$ mm):-

Ex:- ISSH 1000 x 600 x 2 mm

L x B x t

Analysis and Design:-

Method of Design:-

1. Working stress method (WSM) } IS 800 : 1984 $\longrightarrow$ F.O.S
2. Ultimate Load design (ULD) } IS 800 : 1984 $\longrightarrow$ L.F
3. Limit state design (LSD) } IS 800 : 2007 $\longrightarrow$ partial safety fact

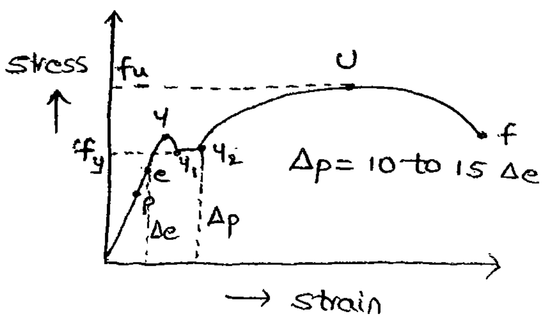
Safety norms:-

$$\text{Factor of safety (FOS)} = \frac{\text{Yield stress}}{\text{working stress}}$$

$$\text{Load factor (LF)} = \frac{\text{Ultimate load}}{\text{working (or) service load}}$$

Working stress method (WSM):- (IS 800 : 1984)

Stress strain curve of mild steel:-



P = proportionality limit
e = elastic limit
Y = upper yield point
Y_1 = Lower yield point
YY_1 = plastic yielding
U = Ultimate stress point
f = failure stress point.

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In working stress method it is assumed that the stress strain curve is linear upto yield point. In this method of design stresses are to be calculated for various critical load combinations and members are designed for working loads.

For safety of structural member working stresses due to various working load combination should be less than are equal to permissible stresses.

Permissible stresses is the fraction of yield strength of the material which is ratio between yield strength to Factor of Safety.

For safety of structural member design requirements

1. working stress due to D.L + L.L ≤ permissible stresses
2. Working stress due to D.L + W.L ≤ permissible stresses
3. working stress due to D.L + L.L + W.L ≤ 1.33 x permissible stress

Permissible stresses (allowable stress) IS 800: 1984 :-

F.O.S

1. Permissible average shear stress = $0.4 f_y$ — $\frac{f_y}{0.4 f_y} = 2.5$

2. Permissible maximum shear stress = $0.45 f_y$ — $\frac{f_y}{0.45 f_y} = 2.11$

3. permissible axial tensile stress = $0.6 f_y$ — $\frac{f_y}{0.6 f_y} = 1.67$

4. permissible bending tensile (or) compressive stress = $0.66 f_y$
5. permissible bearing stress = $0.75 f_y$
6. permissible combined bending + Bearing stress = $0.90 f_y$

Minimum thickness of main structural (serviceability criteria) member :-

1. If steel member is directly exposed to weather accesible for cleaning and painting. the min. thickness of structural member should be more than are equal to 6 mm.
2. If steel member is directly exposed to weather and not accessible for cleaning and painting. the min. thickness is 8 mm

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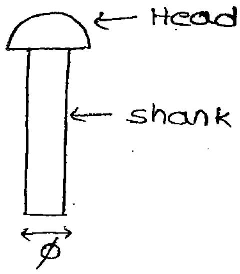
UNIT - 2

RIVERTED AND BOLTED CONNECTIONS

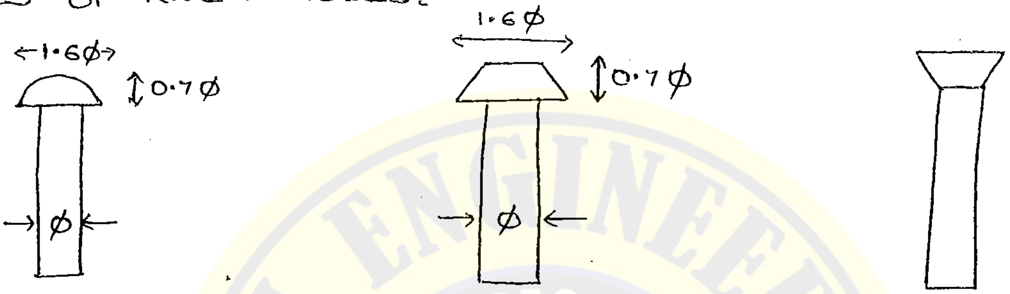
Rivert and Riverting:-



Rivert may be designated with its shank diameter (ϕ) is also called Nominal diameter (ϕ).

Rivert may consists of head and shank and are made of mild steel, or high tensile steel. The size of the rivert is the diameter of the shank.

Types of Rivert heads:-



Snap (or) Button (or) Round Head Rivert

pan head rivert

Flat counter shank rivert

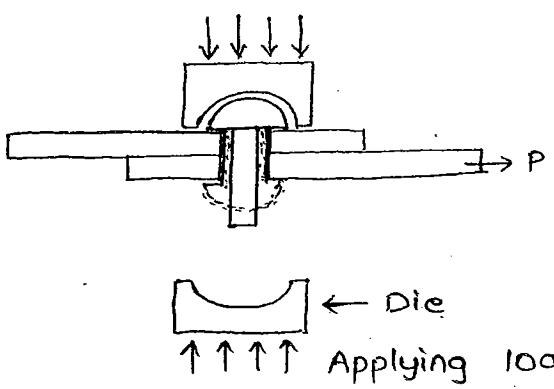
l_g = Grip length of rivert

$\phi \leq 25$ mm, $d = \phi + 1.5$ mm

$\phi > 25$ mm, $d = \phi + 2.0$ mm

Nominal diameter of rivert (ϕ):-

It is a diameter of rivert before riverting process which is also same as shank diameter of rivert.



Applying load then rivert compressed and filled the empty portion and form a Head at bottom side. This process is called Riverting process.

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Assume nominal diameter of rivert by using UNWIN'S formula

$$\phi = 6.04\sqrt{t} \quad (\phi, t \text{ are in mm})$$

$$\phi = 1.91\sqrt{t} \quad (\phi, t \text{ are in cm})$$

t = thickness of the connected member

Gross (or) effective diameter of rivert (d):-

It is the diameter of rivert after riverting process which is also same as diameter of rivert hole.

$$d = \phi + 1.5 \text{ mm} \quad (\phi \leq 25 \text{ mm})$$

$$d = \phi + 2.0 \text{ mm} \quad (\phi > 25 \text{ mm})$$

For calculating the strength of rivert used gross (or) effective diameter in equation.

Classification of Riverts:-

1. Based on method of Driving force:-

a. power driven riverts (uniform pressure can be applied)

It have a approximately 20% higher strength than hand driven riverts.

b. Hand driven riverts (load may be sometimes eccentric)

2. Based on method of placing:-

a. workshop riverts (Better quality control may be achieved for riverting)

b. Field riverts (or) site riverts (quality control may be poor).

Note:-

Field riverts have approximately 10% lesser strength than workshop riverts.

3. Based on method of heating:-

a. Hot driven riverts (riverts may be subjected to High temperature)

b. cold driven riverts (riverting may be done at normal temperature)

Note:-

cold driven rivert has relatively higher strength than Hot driven riverts.

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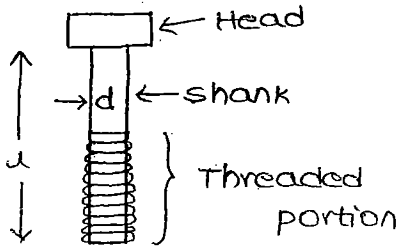
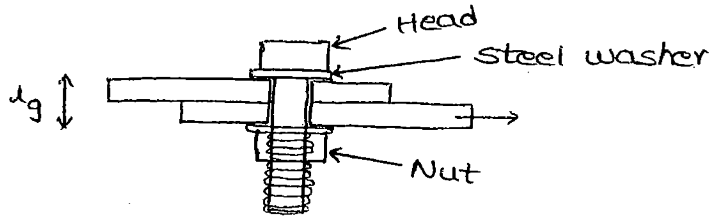
Bolts and Bolting :-





l_g = Grip length of Bolt

Bolt may be designated as M·d·L

M = Metric size

d = Shank diameter of bolt

L = Length of bolt

Ex:- A bolt may be ~~designated~~ Hex bolt designated as M16 x 70 refers [d]

(a) shank dia of bolt is 70 mm

(b) Length of bolt is 16 mm

(c) cross sectional area 16 x 70 cm^2

(d) shank dia of bolt is 16 mm

Hex = Hexagonal head

Types of Bolts:-

1. Unfinished bolts (or) black bolts
2. close tolerance bolts (or) finished bolts
3. High strength friction grip bolts (HSFGB)

$$d_o = d + 1.5 \text{ mm}$$

d_o = diameter of hole

d = shank diameter

Types of bolted (or) joints (or) riverted connections :-
(shear)

Botted connection may be two types

1. Lap connection
2. Butt connection (or) Butt joint
   a. single cover butt connection
   b. Double cover butt connection

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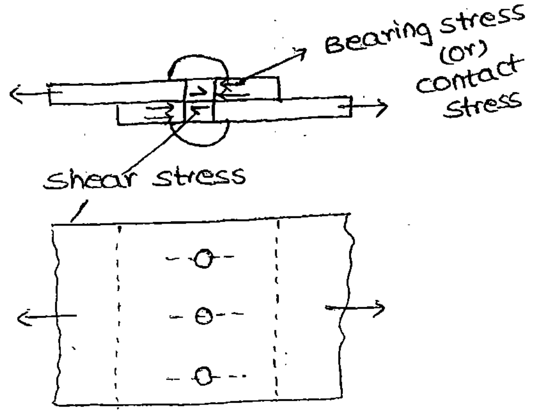
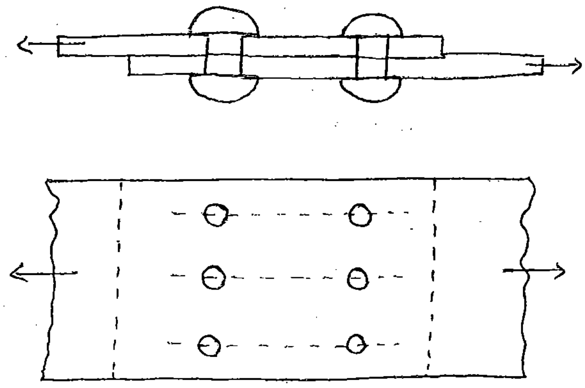
Lap connections (or) joints:-



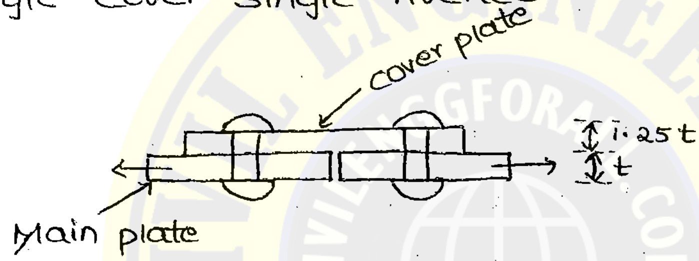
Single riverted lap connection



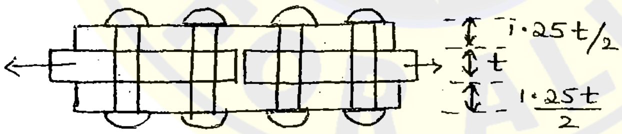
Double riverted lap connection

Butt connections (or) Butt joints:-

a. Single cover single riverted butt joint



b. Double cover double riverted butt joint



Eccentricity is zero in this connection.

**Note:-

1. It is desirable to use double cover butt connection for the following reason.
   a. In case of double cover butt connection the centre of gravity of load in one connected member is lying with centre of gravity of load in another connected member. Hence double cover butt connection is free from moment (eccentricity of load is zero).
   b. Eccentricity of load exist in lap connection or single cover butt connection.

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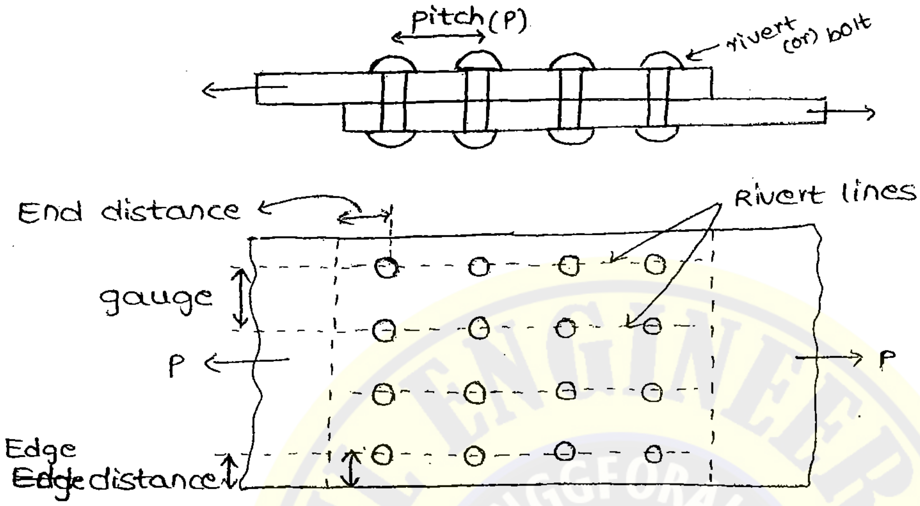
b. The shear strength of each rivert or bolt in double cover butt connection is 2 times higher than shear strength of each rivert or bolt in lap connection or single cover butt connection.

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

(1M) ** Specifications for Riverted (or) Bolted connections :-



1. pitch (P)
2. gauge (g)
3. End distance
4. Edge distance

(Lap connection with chain pattern of riverting)

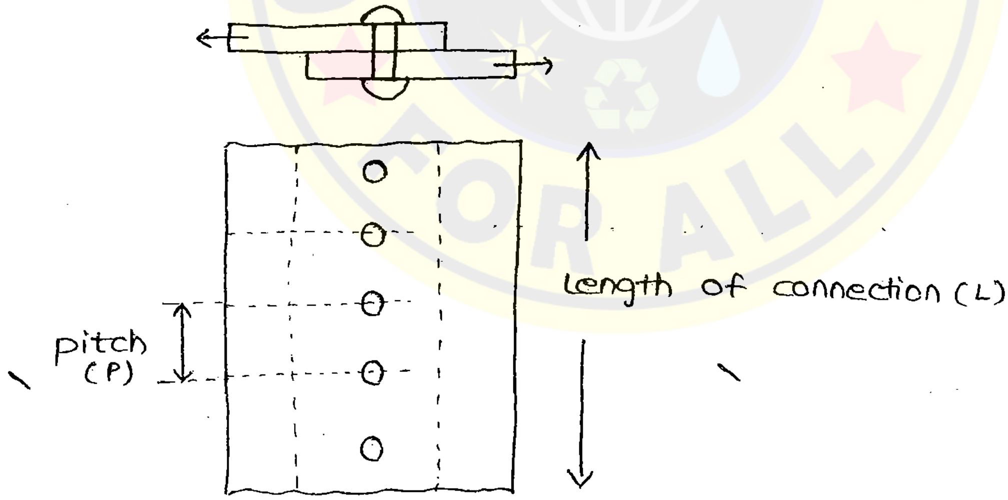
$P = 1600$ KN

~~(pitch)~~ $P_r = 100$ KN

$n = \frac{P}{P_r} = \frac{1600}{100}$

$n = 16$ no.of rivert

P_r = strength of one rivert



(Lap connection between wide plates)

pitch :-

It is a centre to centre distance between two consecutive riverts measure parallel to the direction of a load in a membe

For wide plates it is centre to centre distance between two adjacent riverts measured along length of connection.

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Gauge:-

It is centre to centre distance between two adjacent riverts measured perpendicular to the direction of load in a member (or) it is a distance between two adjacent rivert lines are bolt lines.

Minimum pitch (P_{min}):-

$P_{min} \nless 2.5 \times$ nominal dia of rivert (2.5ϕ)

(or)

$P_{min} \nless 2.5 \times$ shank diameter of Bolt

$(2.5\phi)\ P_{min} \leq P_o \leq P_{max}$

P_o = Optimum pitch

Maximum pitch (P_{max}):-

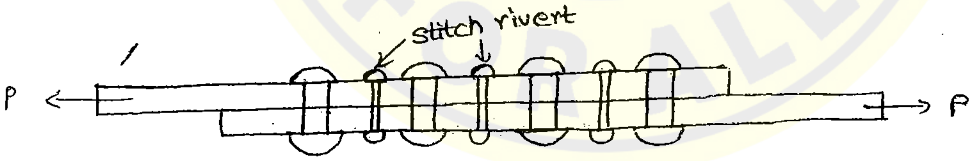
P_{max} = 12t (or) 200 mm, whichever is less for compression member

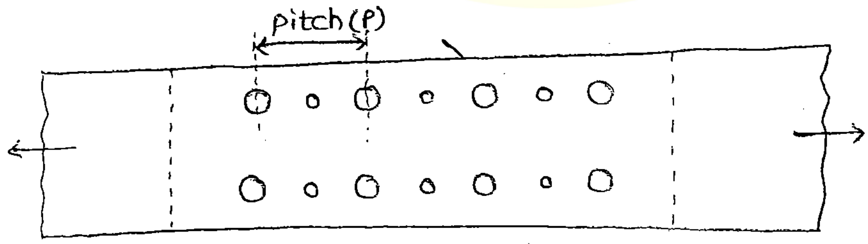
P_{max} = 16t (or) 200 mm, whichever is less for tension member

where,

t = thickness of the thinner connected member.

Tacking riverts or stitch riverts:-





P_{π} = strength of one rivert

If $P > P_{\pi}$ ~~(pitch rivert)~~ then buckling occur. we avoid such buckling, use stitch (or) Tacking riverts.

Maximum pitch for tacking riverts:-

1. For plates:-

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

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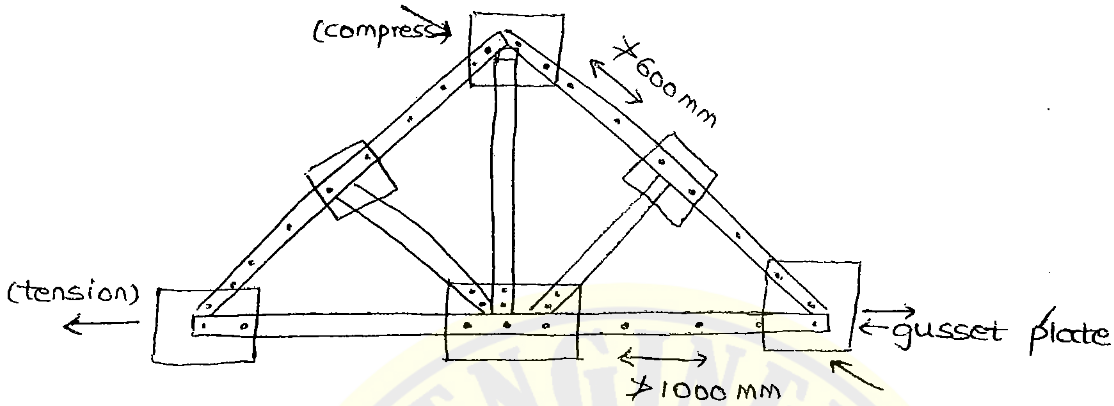
P_{max} = 16t (or) 200 mm, whichever is less, when plates exposed to weather.

2. For Angle channels:-

P_{max} = 600 mm for compression member

= 1000 mm for tension member



Note:-

Tacking rivert or stitch rivert will joint two or more sections together so that the joint sections may behaves as single rolled steel section.

End distance:-

It is the distance between centre of rivert hole to the nearest edge of a main member or cover plate measure parallel to the direction of a load in a member.

Edge distance:-

It is the distance between centre of rivert hole to the nearest edge of a main member (or) cover plate measured perpendicular to the direction of a load in a member.

Minimum end (or) edge distance (e_{min}):-

e_{min} ≈ 1.5 x gross diameter of rivert (1.5d)

(or)

1.5 x dia of bolt hole, for machine flame cut edges

e_{min} ≃ 1.7 x gross dia of rivert (or) dia of bolt hole, For hand flame cut edges

Maximum end or edge distance (e_{max}):-

e_{max} = 4t + 37 mm

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Failures of Riverted or Bolted connections:-

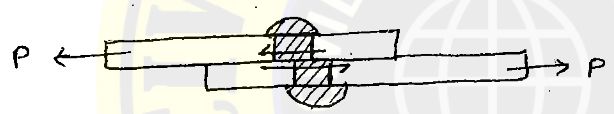
1. Shear failure of rivert (P_s)
2. Bearing failure of rivert (P_b)
3. Bearing failure of plate
4. shear failure of plate
5. splitting failure of plate (or) Edge cracking
6. Tearing failure of plate (P_t)

Note:-

By providing minimum end distance the following three failures can be eliminated.

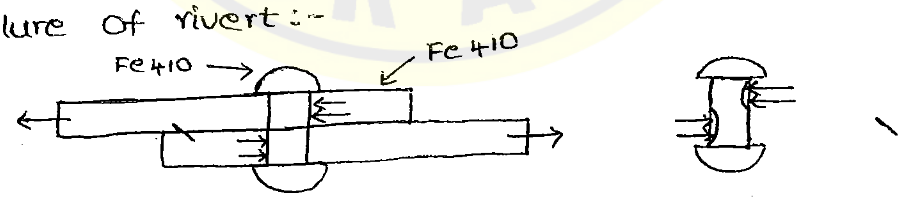
1. Bearing failure of plate
2. shear failure of plate
3. splitting failure of plate (or) edge cracking

Shear failure of rivert:-



These failure may occur when shear stresses in riverts due to applied loads may exceeds working shear stress (permissible shear stress or τ_{vf}) in riverts. shear stress are generated in riverts because plate due to applied forces.

Bearing failure of rivert:-



The half shank of the rivert may be crushed when plate may be strong in bearing and heavily bearing stress in plate may change to press the rivert shank.

This failure may not occur in practise except plates are strong in bearing

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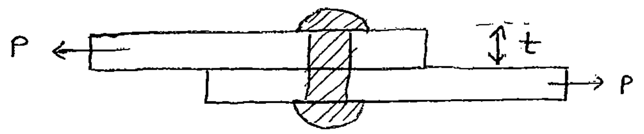
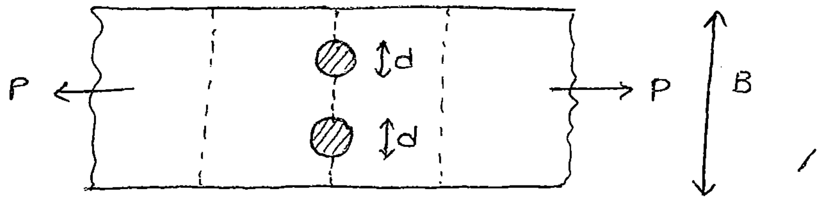
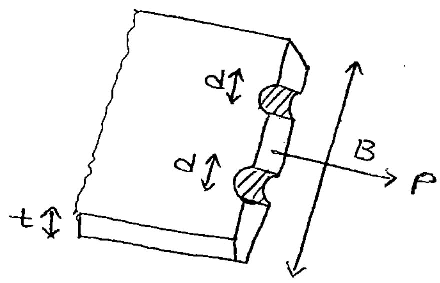
Tearing failure of plate:-







$$\text{Net Area} = (B-2d)t$$

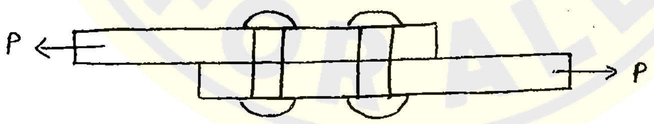
$$\text{shear stress} = \frac{P}{A_{net}} = \frac{P}{(B-2d)t}$$

The tensile stress in plate at a net section may exceeds working tensile stresses in the plate to cause these failure. This failure may also occur whe riverts or bolts may be stronger than plates.

Note:-

These failure occurs where the net area is less that portion caused failure.

Strength of riverted connection (or) joints (P_c (or) P_J):-



a. Shear strength of rivert (P_s):-

P_s = shear area x permissible shear stress in rivert(τ_{vf})

→ Shear strength of one rivert in single shear

$$P_s = \frac{\pi (d)^2}{4} \times \tau_{vf}$$

→ Shear strength of one rivert in double shear

$$P_s = 2 \times \frac{\pi d^2}{4} \times \tau_{vf}$$

→ Shear strength of riverted joint in single shear

$$P_s = n \times \frac{\pi d^2}{4} \times \tau_{vf}$$

d = Gross (or) effective dia. of rivert

→ Shear strength of riverted joint in double shear

$$P_s = 2 \times n \times \frac{\pi d^2}{4} \times \tau_{vf}$$

n = no. of riverts in joint

(b) Bearing strength of rivert (P_b):-

P_b = bearing area x permissible bearing stress in rivert (σ_{pf})

→ Bearing strength of one rivert

$$P_b = (d \times t) \times \sigma_{pf}$$

→ Bearing strength of riverted joint

$$P_b = n \times (d \times t) \times \sigma_{pf}$$

d = Gross diameter of rivert

t = thickness of thinner bearing plate

n = no. of riverts in the joint.

(c) Strength of rivert (or) rivert value (P_r (or) R_v):-

P_r (or) R_V is smaller of P_s (or) P_b

P_s = shear strength of rivert

P_b = bearing strength of rivert

(d) No. of riverts required to a support an axial load (P):-

$$n = \frac{\text{no. of axial load}}{\text{strength of one rivert}} = \frac{P}{P_r}$$

(Rounded off to nearest higher value).

(e) Tearing strength (or) Tensile strength of plate (P_t):-

P_t = Net effective sectional area (A_{net}) x permissible axial tensile stress in plate (σ_{at})

$$P_t = A_{net} \times \sigma_{at}$$

$A_{net} = (B - nd) \cdot t$ — for chain riverting

B = width of the plate

n = no. of rivert holes at a section

d = Gross diameter of rivert.

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Strength of Riverted joint (P_j) (or) connection (P_c):-

$P_c = P_j$ = minimum of P_s (or) P_b (or) P_c

= minimum of P_{π} and P_t

Efficiency of riverted joint (or) connection (η) [or] percentag strength of riverted joint:-

It is the ratio between strength of riverted joint or connection to the strength of main plate (or) solid plate

$$\eta = \frac{\text{Strength of riverted joint}}{\text{Strength of main (or) solid plate}} \times 100$$

→(without any hole)

$$\boxed{\eta = \frac{P_{j}}{P_{mp}} \times 100}$$

Strength of solid plate (or) main plate (P_{mp}) $= A_g \times \sigma_{at}$

A_g = gross sectional area of plate = $B \times t$

σ_{at} = permissible axial tensile stress in plate = $0.6f$

$$\boxed{\therefore P_{mp} = Bt \times \sigma_{at}}$$

P.g No:- 15

15. Given

$P_t = 90000$ N $\quad P_s = 60,000$ N $\quad P_b = 1,20,000$ N

$P_{mp} = 1,80,000$ N

P_c = minimum of P_s (or) P_b (or) P_t

$P_c = 60,000$ N

$$\eta = \frac{P_c}{P_{mp}} \times 100$$

$$= \frac{60,000}{180,000} \times 100$$

$\eta = 33.33\%$

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1. Given $t = 1.0$ cm

Maximum pitch of rivert for tension

$P_{max} = 16t$ (or) 200 mm (or) 20 cm whichever is less

$= 16 \times 1$

$= 16$ cm (or) 20 cm, whichever is less

$\therefore P_{max} = 16$ cm

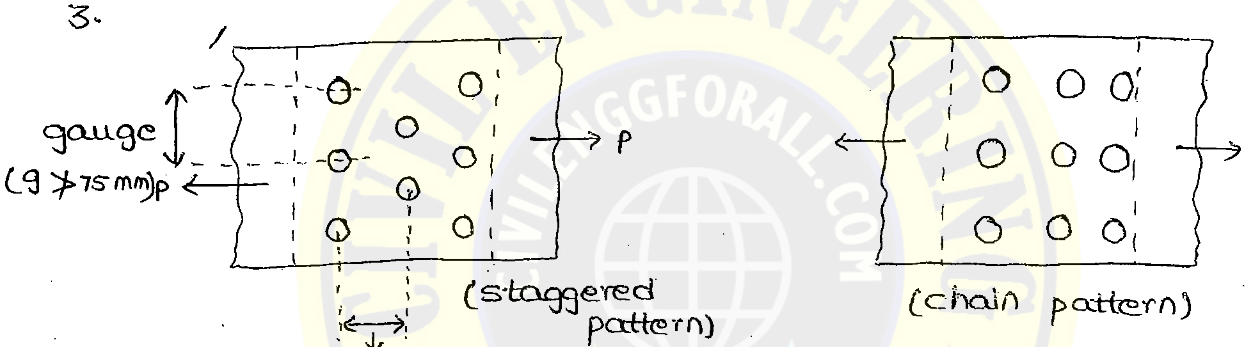
2. Minimum pitch of rivert ($\phi = 20$ mm)

$P_{min} = 2.5\phi$

$= 2.5(20)$

$= 50$ mm

3.



Staggered pitch
(or)
Reeled pitch
(or)
Alternate pitch

P_{max} (tension) $= 16t$ (or) 200 mm

P_{max} (compression) $= 12t$ (or) 200 mm

increased ← Multiply by 1.5 for both

P_{max} (tension) $= 24t$ (or) 300 mm

P_{max} (compression) $= 18t$ (or) 300 mm

It may be increased by 50%.

7. $\phi = 6.04\sqrt{t}$ (ϕ, t are in mm)

$\phi = 1.91\sqrt{t}$ (ϕ, t are in cm)

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Efficiency of riverted connection:-

$$\eta = \frac{\text{strength of the riverted connection}}{\text{strength of the main plate}} \times 100$$

$$\eta = \frac{P_c}{P_{mp}} \times 100$$

P_c is minimum of $\underbrace{P_s \text{ (or) } P_b}_{P_r}$ (or) P_t (or) P_t

$$P_t = A_{net} \times \sigma_{at}$$

$$P_t = (B - nd) \times t \times \sigma_{at}$$

$$P_{mp} = B \times t \times \sigma_{at}$$

If $P_t \leq P_r$, $\quad \dfrac{P_c}{P_{mp}} \times 100 = \dfrac{P_t}{P_{mp}} \times 100$

$$= \frac{A_{net} \times \sigma_{at}}{A_g \times \sigma_{at}} \times 100$$

$$= \frac{(B-nd)t \cancel{\times \sigma}}{Bt} \times 100$$

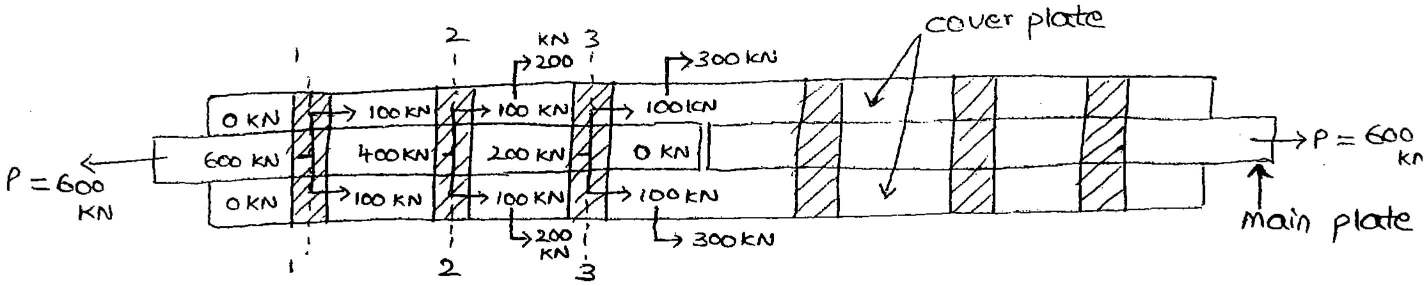
If $P_t \leq P_r$, $\eta = \left(\dfrac{B - nd}{B}\right) \times 100$

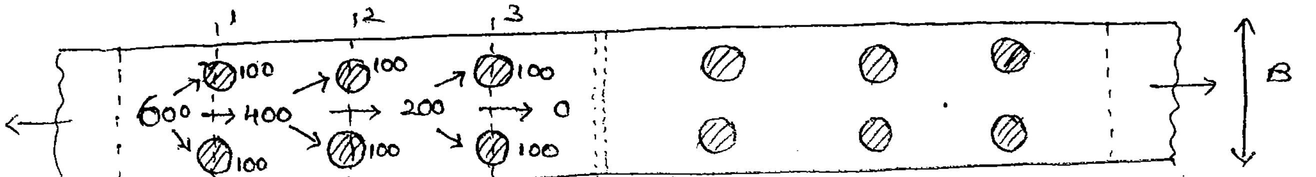
For diamond riverting, $\eta = \left(\dfrac{B-d}{B}\right) \times 100$, if $n = 1$

Ex:- Axial load $P = 600$ kN

$P_r = 100$ kN

$$\eta = \frac{P}{P_r} = \frac{600}{100} = 6 \text{ no's}$$





$(P_t)_{mp\,(1-1)} = (B-2d)t \times \sigma_{at}$

Chain pattern of riverting

$(P_t)_{...} = B t \times \sigma_{at}$

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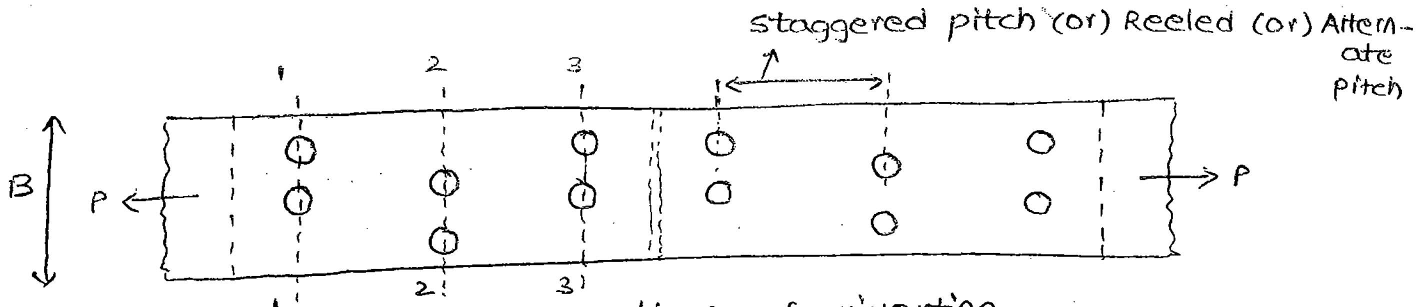
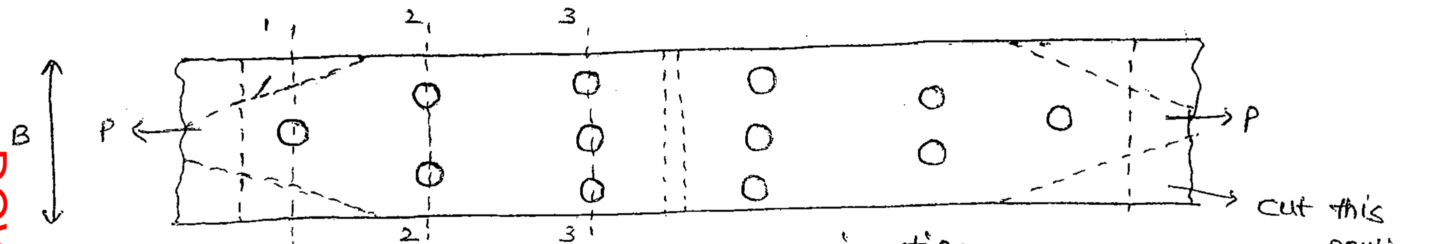


Staggered pattern of riverting



Diamond pattern of riverting

Note:-

1. It is desirable to use double cover butt connection with diamond cover riverting or bolting for the following reasons:

a. Efficiency of diamond pattern of riverting is more

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

b. Cover plate steel may be saved (weight of steel required for cover plate is less). Cover plate steel may be saved in diamond pattern of riverting.

c. Width of main plate required for diamond pattern of riverting is less.

$$B = \frac{P}{t \times \sigma_{at}} + nd$$

$n = 1$, for diamond pattern (At section ①-① only one rivert. so $n=1$)

Permissible (or) Allowable stress in workshop riverts:-

Type of rivert	Axial tension (σ_{tf}) (Mpa (or) N/mm^2)	Shear (τ_{vf}) (Mpa (or) N/mm^2)	Bearing (σ_{pf}) (Mpa)
Power driven	100	100	300
Hand driven	80	80	250
Field riverts	90	90	290

Note:-

For field riverts the above permissible stresses are decreased by 10%.

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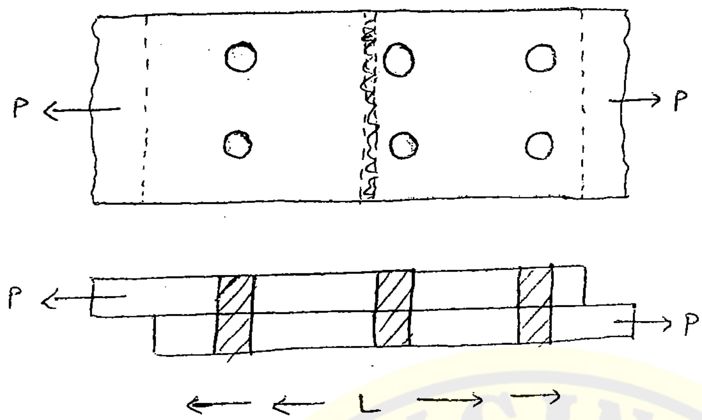
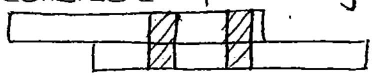
The above permissible stresses in riverts and bolts are to be increased by 25% when wind or earthquake load are considered.

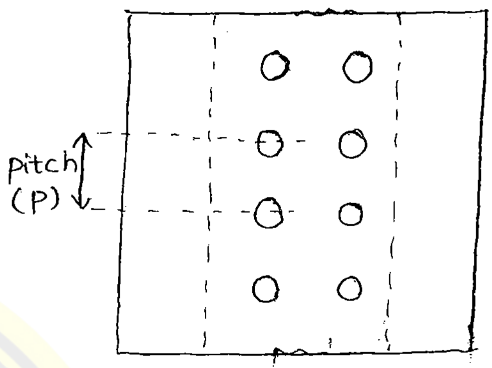
P.g No:- 11

1.



When width plate is very la
then considered pitch length





$P_s = n \times \frac{\pi}{4} d^2 \times \tau_{vf}$

$P_b = d \times t \times \sigma_{pf} \times n$

$P_t = (B - nd) t \times \sigma_{at}$

$\therefore$ p = pitch

n = no. of riverts according to pitch

$= (\frac{1}{2}$ rivert $+ \frac{1}{2}$ rivert$)$

n = 1 rivert

$P_t = (B - nd) t \times \sigma_{at}$

$= (p - 1 \times d) t \times \sigma_{at}$

$= (p - d) t \times \sigma_{at}$

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

$= \left(\frac{P - d}{P}\right) \times 100$

$= \left(\frac{2.5\phi - \phi}{2.5\phi}\right) \times 100$

$= 60\%$

10. Given

$B = 30$ cm, $t = 10$ mm $= 1$ cm $\phi = 18$ mm

$d = 18 + 1.5 = 19.5$ mm

$= 1.95$ cm

$A_{net} = (B - d) t$

$= (30 - 1.95) 1$

$= 28.05$ cm

11. $P_{max} = 32t$ (or) 300 mm

$= 16t$ (or) 200 mm

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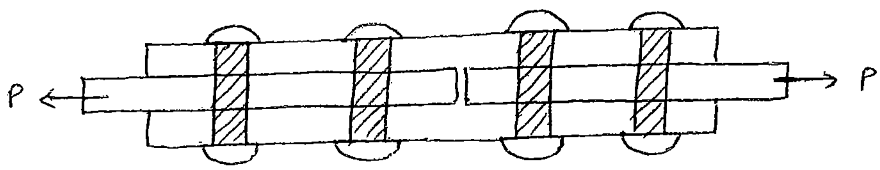
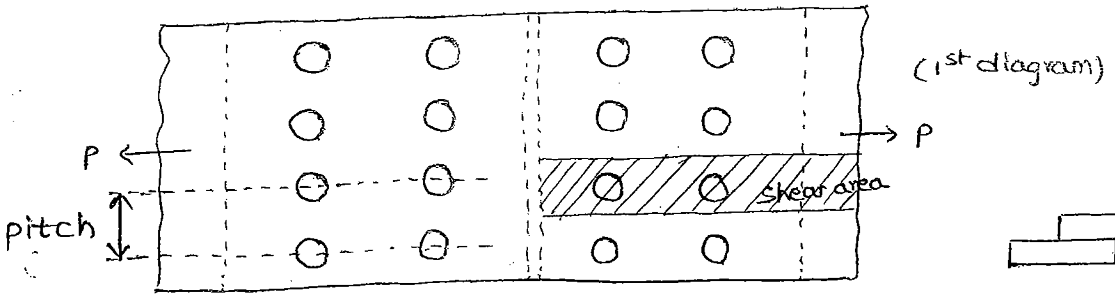
14.

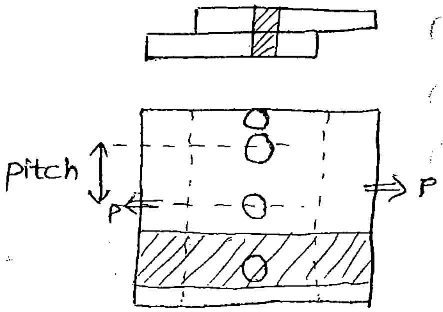




$P_s = 1 \times \frac{\pi}{4} d^2 \times \tau_{vf} \times n$ [2nd diagram]

$\therefore n = 4$

$P_s = \underbrace{2}_{(4 \times \frac{1}{2}\text{ rivets}) \text{ pitch length}} \times \underbrace{2 \times \frac{\pi}{4} d^2}_{\text{shear area (2 rivets)}} \times \tau_{vf}$ [1st diagram]



2nd diagram

15. Given

$P_s = 60$ kN

$P_b = 35$ kN

$P_t = 70$ kN

$P = 700$ kN

P_r min of P_s (or) P_b = 35 kN

$n = \frac{P}{P_r}$

$= \frac{700}{35}$

$n = 20$ no's

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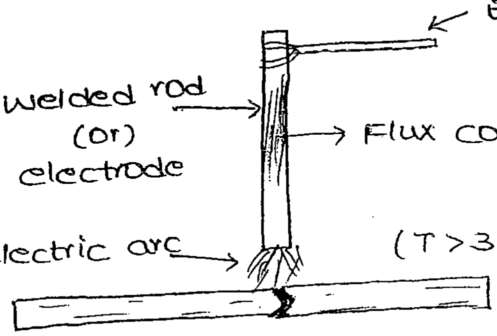
UNIT - 3

WELDED CONNECTIONS



Welded classification :-

Welded classification

- Fusion process
  1. Electric Arc welding
     (For metallic arc welding
  2. Gas welding
- pressure process
  1. Electric resistance welding
  2. Force welding

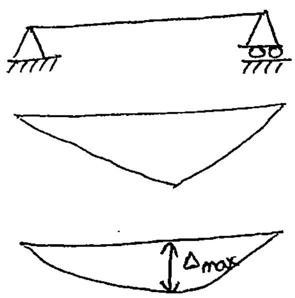
Most commonly used in steel construction is Fusion process.

Advantages:-

S.S. Beams

$$M = \frac{WL}{4}$$

$$\Delta_{max} = \frac{WL^3}{48EI}$$

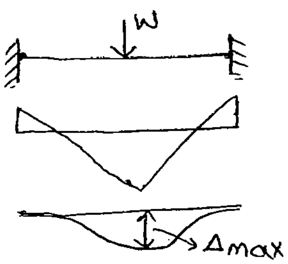


Ex:- Bolted or rivert joints

Fixed beams

$$M = \frac{WL}{8}$$

$$\Delta_{max} = \frac{WL^3}{192EI}$$



Ex:- welded joints

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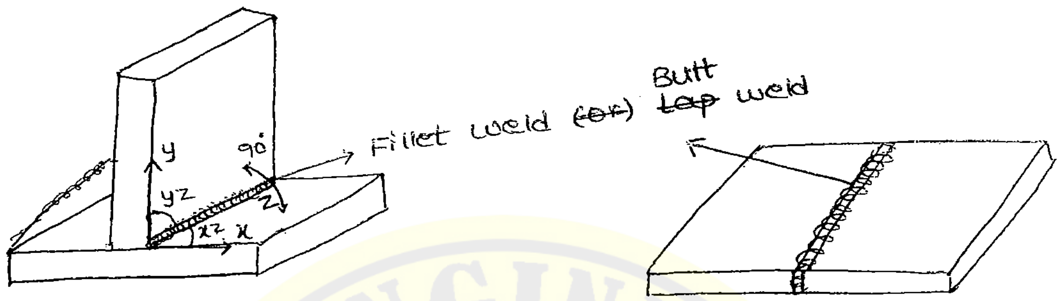
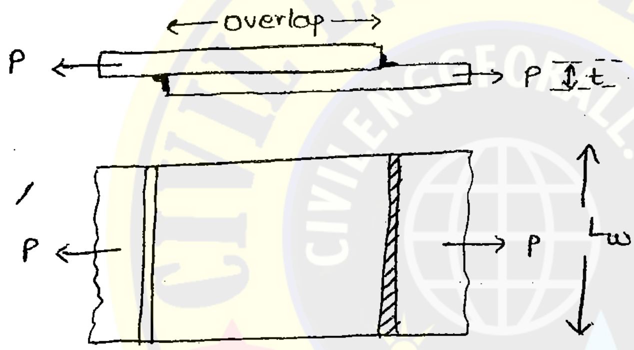
Type of welds:-

1. Fillet welds (or) Lap welds
2. Butt welds (or) Groove welds
3. Slot welds
4. plug /welds

Design of fillet welds (or) Lap welds:-





Minimum overlap:-

1. ≮ 5t (IS : 800 : 1984)
2. ≮ 4t (or) 40 mm whichever is more (IS 800 : 2007)

t = thickness of thinner connected member

Note:-

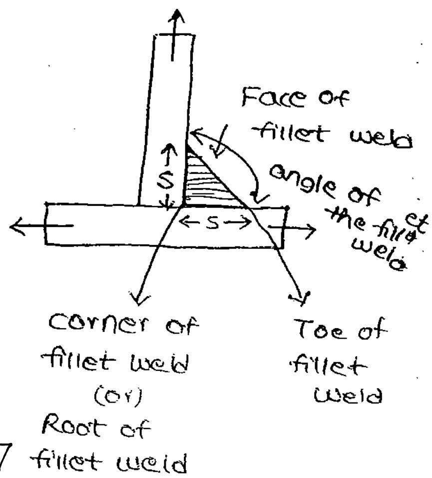
→ 3. standard cross section of fillet weld is Right angle triangle.

→ 4. Fillet weld symbol is 

→ 5. standard angle for fillet weld is 45°

→ Fillet welds are to be provided for joining two different members in two different planes (or) when two different members which an angle.

→ Fillet welds are to be designed for shear stress only. Fillet welds may chance to fail in shear.



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Size of fillet weld :- (S)

It is the distance between corner of fillet weld to the toe of fillet weld (or) it is minimum leg length of the fillet weld cross section.

$$S_{min} \leq S \leq S_{max}$$

Minimum size of fillet weld (S_{min}) :-

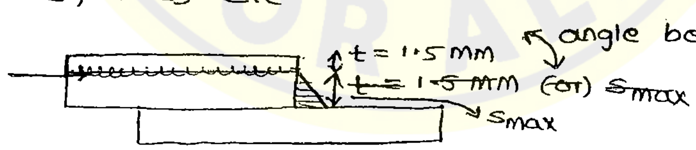
S_{min} depends on thickness of thinner connected member If thickness of thicker connected member.

Over (mm)	Upto & Including (mm)	S_{min}
0	10 mm	3 mm
10 mm	20 mm	5 mm
20 mm	32 mm	6 mm
32 mm	50 mm	8 mm

Maximum size of fillet weld (S_{max}) :-

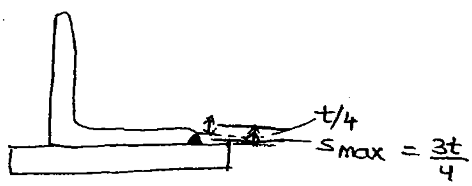
S_{max} depends on

1. Type of edge of a member
   Ex:- Round edge (or) square edge
2. Thickness of thinner connected member
3. S_{max} for square edges
   Ex:- plates, flats etc



$$S_{max} = t - 1.5 \text{ mm} \quad (\text{for square edges})$$

4. S_{max} for round edges like edge of an angle (or) Flange of an I - section



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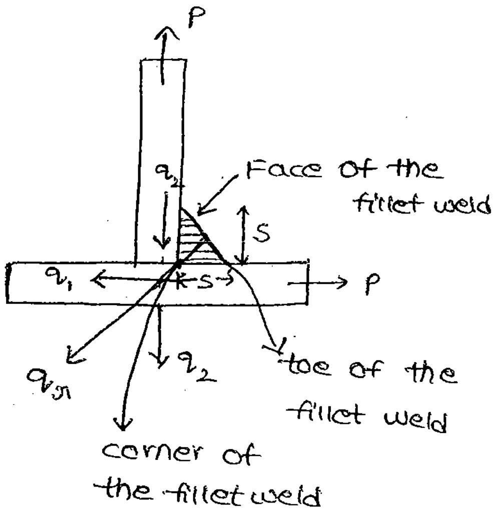
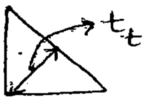
Effective throat thickness (t_t) :-

It is distance between corner of the fillet weld to the face of the fillet weld. Throat is the weakest plane in the fillet weld cross section.





t_t = K x size of fillet weld

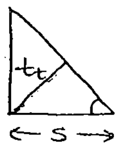
$t_t = K \cdot S$

'K' depends on angle between the weld faces (or) fusion faces (α) and size of fillet weld.

Angle between weld faces (α)	Value of K.
$60^\circ - 90^\circ$	0.70
$91^\circ - 100^\circ$	0.65
$101^\circ - 106^\circ$	0.60
$107^\circ - 113^\circ$	0.55
$114^\circ - 120^\circ$	0.50

$\alpha = 90^\circ$

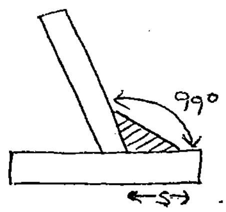
$\sin 90^\circ \; t_t = \frac{S}{\sqrt{2}}$ (or) $\sin 45 = \frac{t_t}{S}$



$t_t = 0.707S$

Ex:- The effective throat thickness of fillet weld as shown in fig is.

a) 0.7S b) 0.65S c) 0.60S d) 0.55S



$\alpha = 99^\circ$

$t_t = \frac{S}{}$

$t_t = 0.64$

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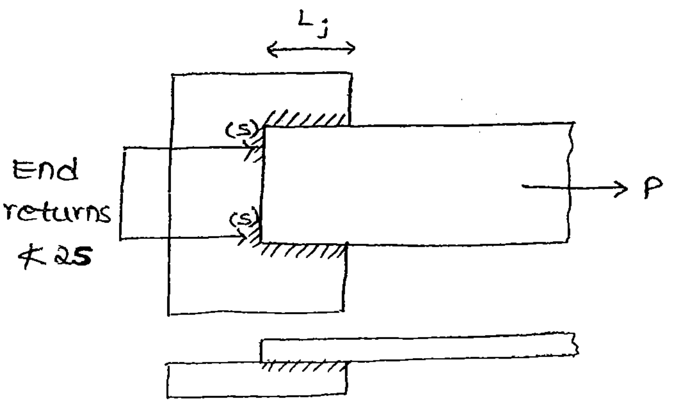
Effective length of the fillet weld (L_w):-



L_j = Length of the side fillet (Length of Lap)

L_w = Effective length of fillet weld

L = Overall length of fillet weld (or) Total length of fillet w

s = size of the fillet weld.

Note:-

End returns to be provided at terminating ends not less than 2 times size of weld will minimize stress concentration due to tensile loads or Bending moment.

$$L = L_w + 2s$$

$$L_w = L - 2s$$

$L_w \nless 4s$ (or) 40 mm whichever is more

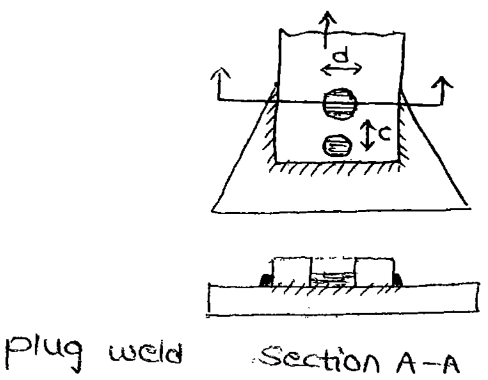
Shear strength of fillet weld (P_s):-

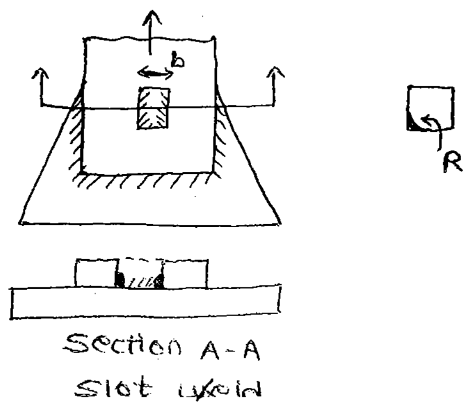
P_s = Effective shear area × permissible shear stress in fillet weld (τ_{vf})

$$\boxed{P_s = L_w \cdot t_t \times \tau_{vf}}$$

$$P_s = L_w (Ks) \times \tau_{vf}$$

Design of slot and plug welds:-





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d = diameter of slot

b = width of the slot

R = corner radius of slotted hole

c = clear distance between the slotted holes

t = thickness of the member which is having slot (or) hole

Note:-

Plug welds and slot welds may be used along with fillet weld when sufficient length of weld not available to have more shear strength for supporting given load.

plug welds and slot welds will reduce unsupported dimension between fillet weld so that local buckling failure may be checked.

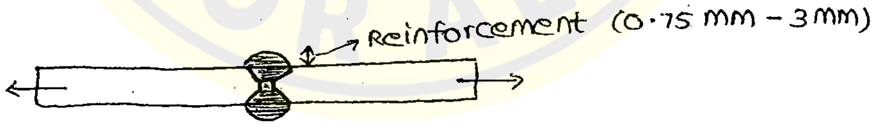
Width of the slot or diameter of the slot should be more than are equal to 25 mm and also should be more than are equal to 3t

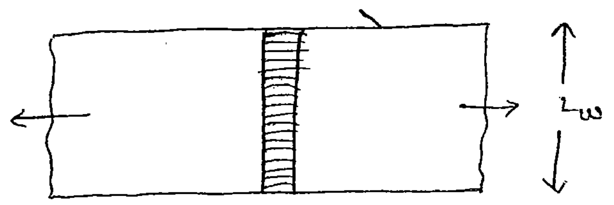
1. b (or) d $\geq$ 25 and also $\geq$ 3t whichever is more

2. R $\geq$ 1.5t and also $\geq$ 12 mm whichever is more

3. c $\geq$ 2t and also $\geq$ 25 mm whichever is more

Design of butt (or) Groove welds :-





Butt (or) Groove weld

Types of butt welds

Partially penetrated butt welds (or) single butt welds

Ex:- single 'V', single 'U', single 'J' welds etc

Fully penetrated butt welds (or) Double butt welds

Ex:- Double 'U', Double 'V', Double 'J', butt welds etc.

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Type of Butt weld	Diagramatic Representation	symbol	Throat (t_e) thickness
Single 'V'		V	$\frac{5}{8}t$
single 'U'		U	$\frac{5}{8}t$
Double 'V'		X	t
Double 'U'		X	t

Axial strength of butt weld:- (T_s):-

T_s = Effective sectional area x permissible axial stres (σ_{tf})

$$\boxed{T_s = L_w \cdot t_e \times \sigma_{tf}}$$

L_w = effective length of the butt weld

t_e = effective throat thickness

= $\frac{5t}{8}$ for single butt welds like single v, single U.

$t_e = t$, for double butt welds like double v, double U

t = thickness of the thinner connected member

Permissible stress in weld for mild steel members:-

1. permissible shear stress in weld τ_{vf} = 108 Mpa
2. permissible axial stress in weld, σ_{tf} = 150 Mpa
3. permissible bending compressive (or) bending tensile stress in welds $\sigma_{bc} = \sigma_{bt}$ = 165 Mpa.

Note:-

The above permissible stresses are increased by 25% if wind (or) earth quake loads are to be used in design calculations.

The above permissible stresses are decreased by 25% for site welding (or) field welding

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1. Effective sectional (shear) area $= L_w \times t_t$

$= L_w \times ks$ $\alpha = 75°$

$= L_w (0.75)$ ← standard value

$= 0.75 \times L_w$

4. $L_w = 200$ mm $\sigma_{tf} = 150$ Mpa $t = 12$ mm

Throat thickness, $t_e = \frac{5}{8} t$

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

Axial strength of Butt weld, $P_t = L_w \times t_e \times \sigma_{tf}$

$= 200 \times (7.5) \times 150$

$= 225$ KN

5. Given $s = 3$ mm $\tau_{vf} = 110$ mpa

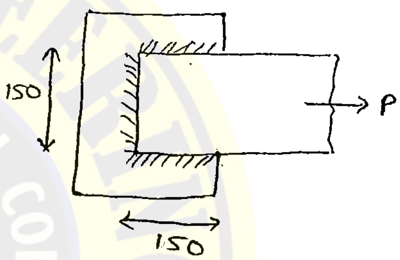
Service load allowed,

$P = P_s = L_w \times t_t \times \tau_{vf}$

$= (3 \times 150) \times (0.7 \times 3) \times 110$

$= 103.95 \times 10^3$ N

$= 103.95$ KN



P_s = shear area along failure plane

6. $t_1 = 16$ mm $t_2 = 20$ mm $t_e = t = 10$ mm

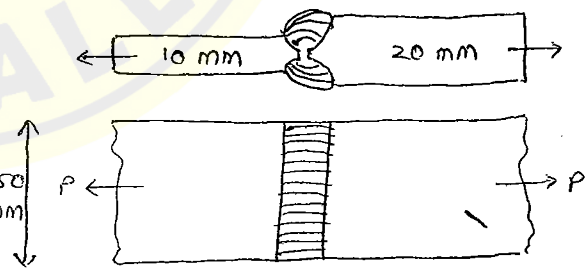
$\sigma_{tf} = 150$ mpa

Load $= P = T_s = L_w \cdot t_e \cdot \sigma_{tf}$

$= 250 \times 10 \times 150$

$= 375 \times 10^3$ N

$= 375$ kN



7. $\tau_{vf} = 108$ Mpa $t = 16$ mm

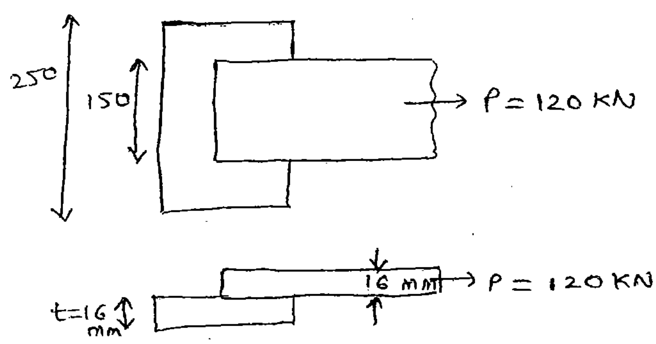
Minimum size of fillet weld

$S_{min} = s = 5$ mm

$t_t = ks$

$= 0.707 \times 5$

$= 3.5$ mm



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If $P \leq P_s$

$P = P_s$

$= L_w \cdot t_t \cdot \tau_{vf}$

$= L_w \cdot ks \cdot \tau_{vf}$

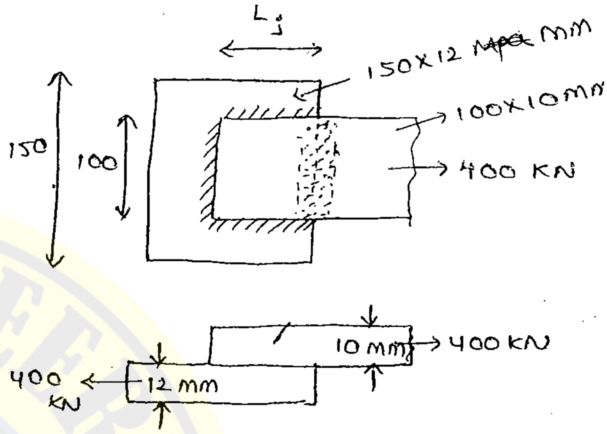
By equating $P = P_s$

$$120 \times 10^3 = L_w \times (0.707 \times 5) \times 108$$

$$L_w = 314.42 \text{ mm}$$

8. $L_w = L_j + L_j + 100 + 100$

$= 2L_j + 200$ mm



Equating load $P \leq P_s$

$P = P_s$

$P = L_w t_t \tau_{vf}$

Overlap $L_j = \dfrac{L_w - 200}{2}$

Factored load $= \gamma_L \times$ service load (or) working load

$= \gamma_L \times \dfrac{400}{\gamma_L} \Rightarrow \gamma_L \times \dfrac{400}{1.5} \Rightarrow 266.67$ kN

Maximum size of weld which is depending on thinner connected member

$S_{max} = t - 1.5$

$= 10 - 1.5$

$= 8.5$ mm

By equating $P = P_s$

$$266 \times 10^3 = L_w t_t \tau_{vf}$$

$$= L_w \, ks \, \tau_{vf}$$

$$= L_w \times (0.707 \times 8.5) \times (108)$$

$$L_w = 414.9 \text{ mm}$$

Overlap, $L_j = \dfrac{L_w - 200}{2}$

$= \dfrac{414.9 - 200}{2}$

$= 110$ mm

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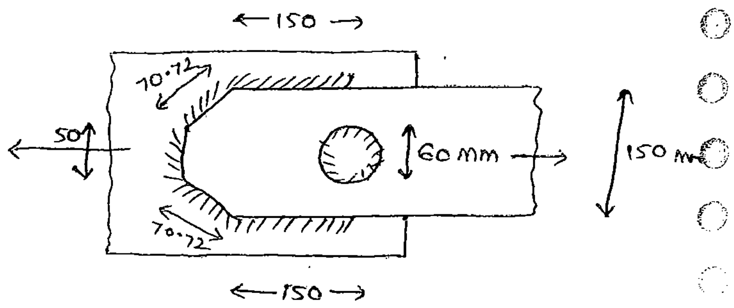
10. Allowable stress in plate,



$\sigma_{st} = 150$ Mpa

$t = 20$ mm

$B = 150$ mm

Size of weld, $s = 12$ mm

Throat thickness, $t_t = ks$

$= 0.7 \times 12$

$= 8.4$ mm

Average shear stress in weld $= \dfrac{P}{\text{Shear area}}$

$= \dfrac{P}{L_w \cdot t_t}$

Full strength of plate, $P_t = P = A_{gross} \times \sigma_{at}$

$= (150 \times 20) \times 150$

$= 450 \times 10^3$ N

Shear area of fillet weld

$A_e = L_w \times t_t$

$= [[2(150+70.72)+50] \times \pi(60)] \times (8.4)$

$A_e = 5711$ mm^2

Avg. shear stress in fillet weld $\tau = \dfrac{P}{A_e}$

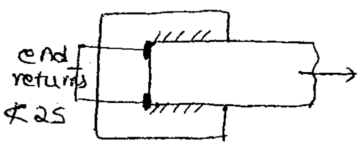
$= \dfrac{450 \times 10^3}{5711}$

$\tau = 78.79$ N/mm^2

P.g No:- 23

1. $L = L_w + 2s$

$L_w = L - 2s$

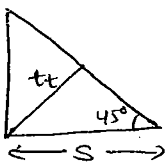


3. $\sin 45 = \dfrac{t_t}{s}$

$t_t = s \cdot \sin 45$

$= \dfrac{s}{\sqrt{2}}$

$= 0.707 s$



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4. Allowable stress in plate

$\sigma_{at} = 200$ Mpa

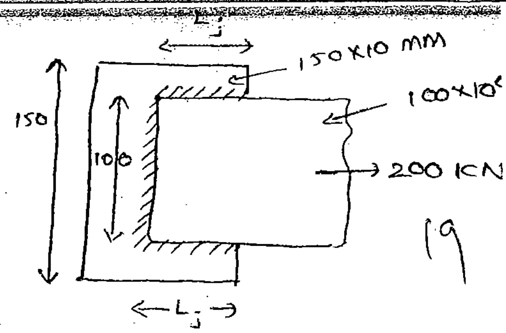
Allowable stress in weld $\tau_{vf} = 100$ Mpa

Size of weld, $s = 6$ mm

Throat thickness, $t_t = KS$

$= 0.7 \times 6$

$= 4.2$ mm



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$L_w = L_j + L_j + 100$

$L_w = 2L_j + 100$

$L_j = \frac{L_w - 100}{2}$

Strength of 150mm plate $= \underset{(Bt)}{A_g} \times \sigma_{at}$

$= 150 \times 10 \times 200$

$= 300 \times 10^3$ N

Strength of 100 mm plate $= 100 \times 100 \times 200$

$= 200$ kN

$\therefore$ safe load allowed is 200×10^3 N

By equating $P = P_s$

$= L_w \cdot t_t \cdot \tau_{vf}$

$200 \times 10^3 = \overset{L_w}{\cancel{(2L_j + 100)}} \times 4.2 \times (100)$

$L_w = 476.19$ mm

$L_j = \frac{L_w - 100}{2}$

$= 188$ mm

11. Given $s = 6$ mm

$t_t = 0.707 s$

$= 0.707 \times 6$

$= 4.24$ mm

12. $L_w = 100$ mm, $t_e = 8$ mm $\sigma_{tf} = 150$

Strength of butt weld ; $T_s = L_w t_e \sigma$

$T_s = 100 \times 8 \times 150$

$= 120 \times 10^3$ N

$= 120$ kN

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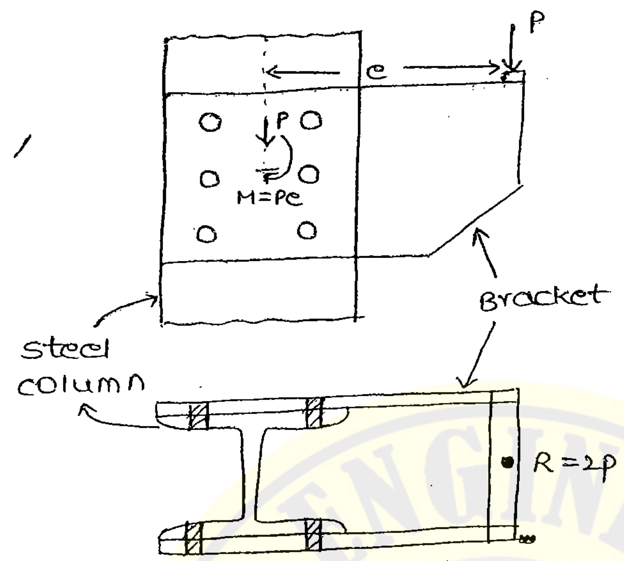
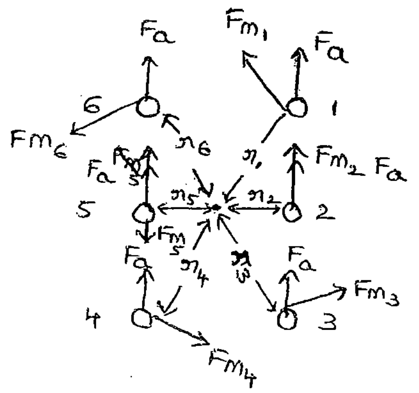
UNIT - 4

ECCENTRIC CONNECTIONS

1. Bracket type Riverted connection - I (Elastic Analysis) :-





When load (or) moment is lying in the plane of Rivert (or) Bolt group.

Rivert group is subjected to

1. Direct Eccentric load (P)
2. Twisting moment (M = Pe)

$$P(e) \rightarrow P \rightarrow F_a\,(\text{shear}); \quad M \rightarrow F_m\,(\text{shear}); \quad F_R$$

P = Eccentric load

e = distance between the C.G of Bolt (or) Rivert group to the applied load line.

→ Verticle shear force in any rivert (or) Bolt due to 'P' is F_a

$$F_a = \frac{P}{n}$$

n = no. of riverts or bolts present in the connection.

→ Shear force in each rivert (or) bolt due to twisting moment (M) is F_m.

$$F_m = \frac{M \cdot r}{\Sigma r^2} = \frac{P \cdot e \cdot r}{\Sigma r^2}$$

$$\Sigma r^2 = r_1^2 + r_2^2 + r_3^2 + \cdots + r_n^2$$

r = radial distance from C.G rivert group to the rivert.

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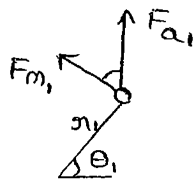
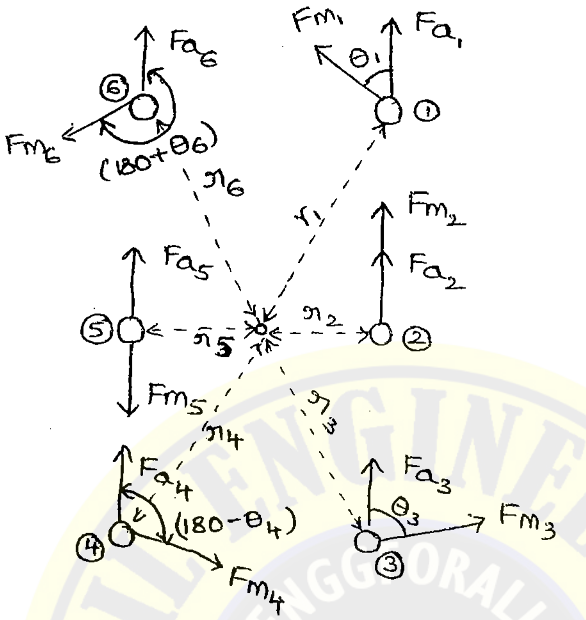
→ Resultant force between F_a and F_m is F_R



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

Condition for maximum resultant shear force ($F_{R\,max}$):-



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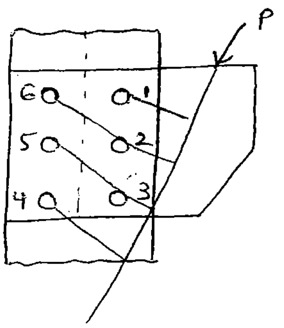
Critical bolt (or) critical rivert is one which is subjected to maximum resultant shear force is farthest from C.G of rivert group, (r is maximum) and which may be close to the applied load line.

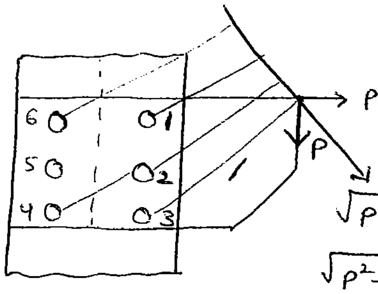
r → maximum (1, 3, 4, 6)

θ → minimum (1, 3)

Hence critical riverts (or) Bolts are Rivet/bolt no. 1 & 3

Ex:-





$\sqrt{P^2+P^2+2PP\cos}$

$\sqrt{P^2+P^2+2PP\cos}$

$\sqrt{2}\,P$

r → maximum (1, 3, 4, 6)

θ → minimum (3)

critical rivert : rivert '3'.

r → maximum (1, 3, 4, 6)

θ → minimum (1)

critical rivert : Rivert no. 1

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For safety of Rivert group:-

Design requirement:-

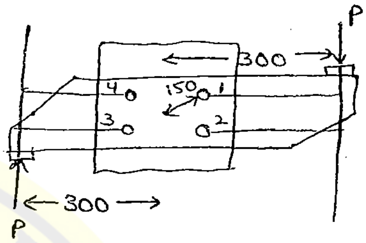
$(F_R)_{max} \leq P_r$ = Rivert value (or) strength of one rivert

P_r = minimum of P_s or P_b

P_s = shear strength of one rivert

P_b = bearing strength of one rivert

P.g No:- 29

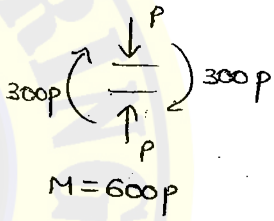


1. $r \rightarrow$ maximum (1, 2, 3, 4)

$\theta \rightarrow$ minimum (1, 2, 3, 4)

Critical riverts are Rivert no. 1, 2, 3, 4

$F_a = \dfrac{P}{n}$ $\quad P = 0 \quad F_a = 0$



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

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

$$= P$$

$$F_{R\,max} = F_{R_1} = F_{R_2} = F_{R_3} = F_{R_4}$$

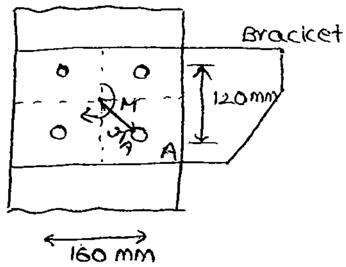
$$= \sqrt{F_a^2 + F_m^2 + 2 F_a F_m \cos\theta}$$

$$= \sqrt{F_m^2}$$

$$= F_m$$

$F_{R\,max} = F_m = P$ (since $F_a = 0$)

5. Given Moment, $M = 80 \times 10^3$ KN-mm



$$r_A = \sqrt{\left(\frac{120}{2}\right)^2 + \left(\frac{160}{2}\right)^2}$$

$$= 100 \text{ mm}$$

$P = 0, \quad F_a = 0$

$$F_{m_a} = \frac{M r_A}{\Sigma r^2}$$

$$= \frac{80 \times 10^3 \times 100}{4 \times 100^2}$$

$\therefore (F_R)_A = F_{m_a} = 200$ KN (since $F_a = 0$)

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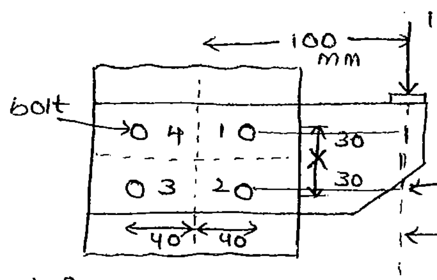
6. critical bolts

$r \rightarrow$ maximum (1, 2, 3, 4)

$\theta \rightarrow$ minimum (1, 2)



whichever is near to crack that bolt is cri bolt (1, 2)

Hence critical bolts are bolt No. 1, 2

$F_{R\,max} = F_{R_1} = F_{R_2}$

1. $P \longrightarrow F_a$

2. $M = Pe$

$= 10(100)$

$= 1000$ kN-mm $\longrightarrow F_m$

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

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

$r_1 = r_2 = r_3 = r_4 = \sqrt{(30)^2+(40)^2}$

$= 50$ mm

$= \frac{1000(50)}{4 \times (50)^2}$

$= 5$ kN

Maximum resultant shear force $(F_R)_{max} = F_{R_1} = F_{R_2}$

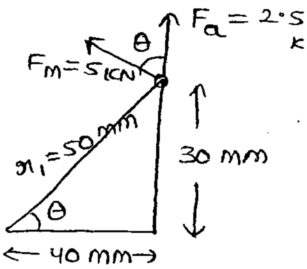
$= \sqrt{F_a^2 + F_m^2 + 2F_a F_m \cos\theta}$

$\cos\theta = \frac{40}{50}$

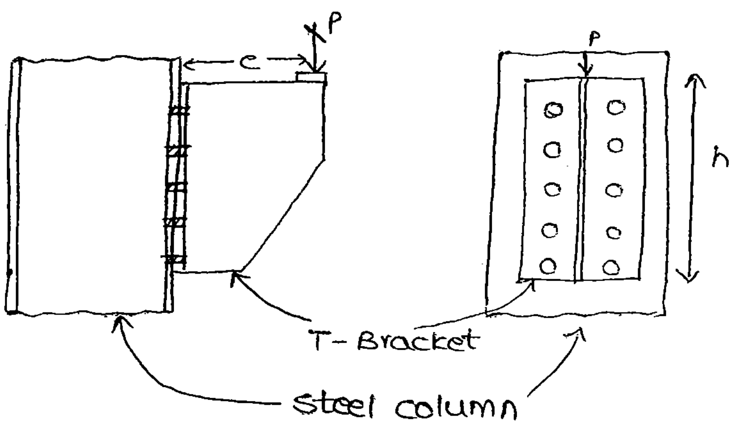
$= \sqrt{(2.5)^2 + 5^2 + 2(2.5)(5)\cos(0.8)}$

$\cos\theta = 0.8$

$= 7.18$ kN



2. Bracket type Riverted connection - II (Elastic Analysis):-



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P = eccentric load

e = eccentricity of the load

h = depth of T-bracket

When load (or) moment is not lying in the plane of rivert (cold driven rivert group - No initial tension) group (or) Bolt group. Rivert or Bolt group is subjected to in

1. Direct concentric load (P)
2. Bending Moment ($M = Pe$).

→ verticle shear force in any rivert (or) Bolt due to P is V_b.

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

n = no. of rivert (or) Bolt present in the connection.

P(e)

↓ P → $V_b = \frac{P}{n}$

↓ M → $T_b = \frac{M' y_n}{\Sigma y_i^2}$

→ Tensile force in ith bolt due to M' is the T_{bi}

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

Tensile force in extreme bolt (or) rivert due to M' is T_b

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

y_n = distance from C.G of bolt (or) rivert group to extreme bolt (or) rivert.

(IES) → If axis of bending is $\frac{h}{2}$ from bottom

$$M' = \frac{M}{2} = \frac{Pe}{2}$$

If axis of bending is $\frac{h}{7}$ from bottom edge.

$$M' = \left[\frac{M}{1 + \frac{2h}{21} \cdot \frac{\Sigma y_i}{\Sigma y_i^2}}\right]$$

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For safety (or) rivert group connection is checked when rivert is subjected to shear and Tension as per IS 800: 1984

$$\frac{\tau_{va,cal}}{\tau_{va}} + \frac{\tau_{vf,cal}}{\sigma_{tf}} \leq 1.4$$

$\tau_{va,cal}$ = calculated shear stress in rivert

$= \dfrac{V_b}{\frac{\pi (d)^2}{4}}$, $V_b = \dfrac{P}{n}$

$\sigma_{tf,cal}$ = calculated tensile stress in rivert

$= T_b / \frac{\pi}{4}(d)^2$, $T_b = \dfrac{M' y_n}{\Sigma y_i^2}$

d = Gross (or) effective diameter of rivert

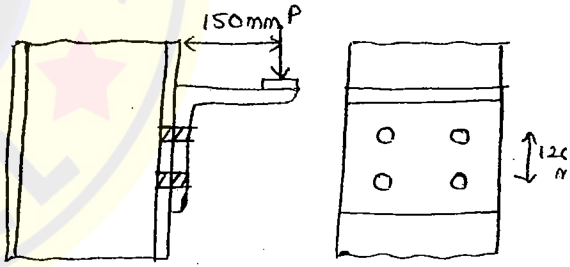
τ_{vf} = permissible shear stress in rivert

σ_{tf} = permissible tensile stress in rivert.

P.g NO:- 29

3. Shear capacity, $V_{bc} = 20$ KN

Tension capacity, $T_{bc} = 15$ KN



1. $P \longrightarrow V_b$

2. $M = Pe \longrightarrow T_b$

$= P(150)$

$= 150P$ KN-mm

$M' = M'' = \dfrac{M}{2}$

$= \dfrac{150P}{2}$

$= 75P$

$V_b = \dfrac{P}{n} = \dfrac{P}{4}$ (shear)

$T_b = \dfrac{M' y_n}{\Sigma y_i^2}$

$y_n = \dfrac{120}{2} = 60$ mm

$= \dfrac{75P(60)}{2 \times 60^2}$

$= \dfrac{5P}{8}$ (Tension)

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$$\frac{V_b}{\sigma_{bc}} + \frac{T_b}{T_{bc}} \leq 1.4$$

$$\left(\frac{P}{4} \cdot \frac{1}{20}\right) + \left(\frac{5P}{8} \cdot \frac{1}{15}\right) = 1.4$$

$P_{max} = 25.85$ kN

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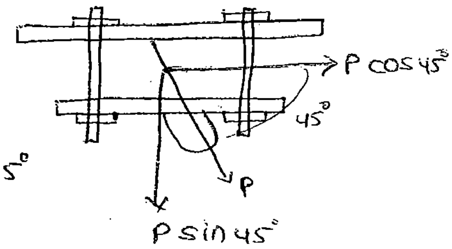


4. Shear capacity, $V_{bc} = 30$ KN

Tension capacity, $T_{bc} = 40$ KN

Shear force in rivert due to P cos 45°

$$V_b = \frac{P\cos 45^\circ}{4}$$

$$= \frac{P}{4\sqrt{2}}$$

Tension ~~due to~~ force in rivert due to P sin 45°

$$T_b = \frac{P\sin 45^\circ}{4}$$

$$= \frac{P}{4\sqrt{2}}$$

Intraction equation as per IS 800: 1984

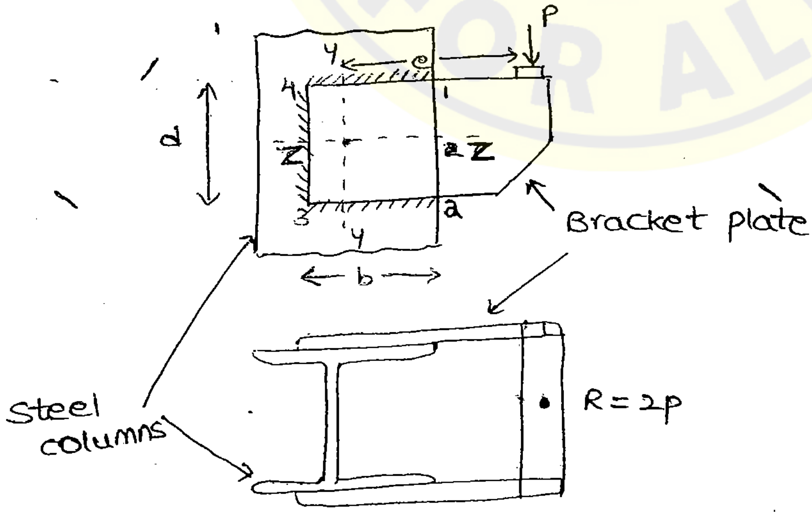
$$\frac{V_b}{V_{bc}} + \frac{T_b}{T_{bc}} \leq 1.4$$

$$\frac{P}{4\sqrt{2}} \cdot \frac{1}{30} + \frac{P}{4\sqrt{2}} \cdot \frac{1}{40} = 1.4$$

$$P_{max} = 135.76 \text{ KN}$$

Eccentric Welded connections:-

1. Fillet weld Bracket Type connection - I (Elastic Analysis):-



P = eccentric load

e = eccentricity of the load (distance from C.G of weld group to the applied load line)

d = depth of bracket plate

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

1. Direct concentric load (P)
2. Twisting moment (M = Pe)

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→ Verticle shear stress in weld due to P is q_1

$$q_1 = \frac{P}{\text{effective sectional area}}$$

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

$$= \frac{P}{(d+2b)\cdot t_t}$$

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

t_t = effective throat thickness = ks

S = size of fillet weld

→ Shear stress in weld due to M is q_2,

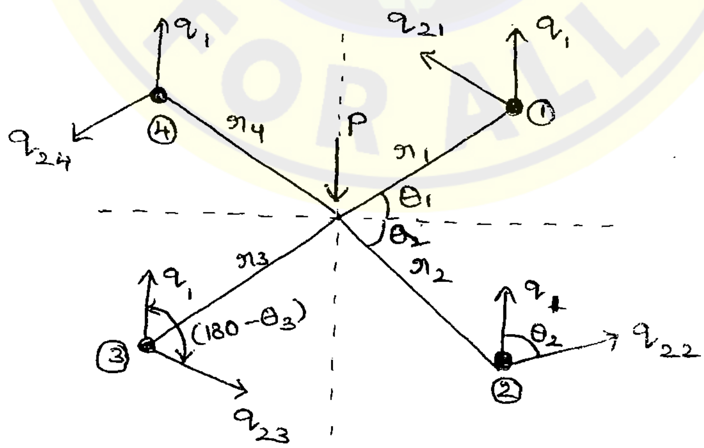
$$q_2 = \frac{M}{I_p} \times r$$

$M = Pe$

I_p = polar moment of Inertia of fillet weld

= $I_{zz} + I_{yy}$ of fillet weld

r = radial distance from C.G of weld to point on weld length.



Resultant shear stress between q_1 and q_2 is q_R

$$q_R = \sqrt{q_1^2 + q_2^2 + 2q_1q_2\cos\theta}$$

Condition for maximum Q_R ($Q_{R\,max}$):-

1. r → maximum
2. θ → minimum

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For safety of fillet weld group:-

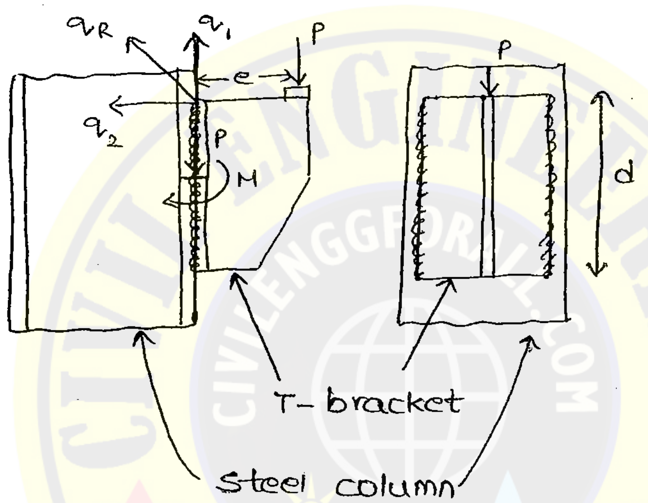
$$(q_R)_{max} \leq \tau_{vf} = \text{permissible shear stress in fillet weld}$$

$$\tau_{vf} = 108 \text{ Mpa}$$

Note:-

1. Heavily stressed point is ① at which the resultant shear stress is maximum which is farthest from C.G of weld group (R is max.) and which may be closed to the applied load line.

2. Fillet weld Bracket type connection - II (Elastic Analysis):-



P = eccentric load
e = eccentricity of the load
d = depth of T-bracket

When load (or) moment is not lying in the plane of fillet weld group. Fillet weld group is subjected to

1. Direct concentric load (P)
2. Bending Moment (M = Pe)

→ verticle shear stress in weld due to P is q_1

$$q_1 = \frac{P}{\text{effective sectional area}}$$

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

$$= \frac{P}{2d \cdot t_t}$$

L_w = effective length of fillet weld = 2d
t_t = Effective throat thickness = Ks
s = size of fillet weld.

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→ Stress is weld due to M is q_2

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

$M = Pe$

I = Moment of inertia of fillet weld about bending axis.

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

Combined stress between q_1 and q_2 is q_R

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

$$= \sqrt{q_1^2 + q_2^2 + 2q_1 q_2 \cos 90^\circ}$$

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

Condition for max. q_R :-

$y \rightarrow$ maximum.

For safety of fillet weld:-

$$(q_R)_{max} \leq \tau_{vf} = 108 \text{ Mpa}$$

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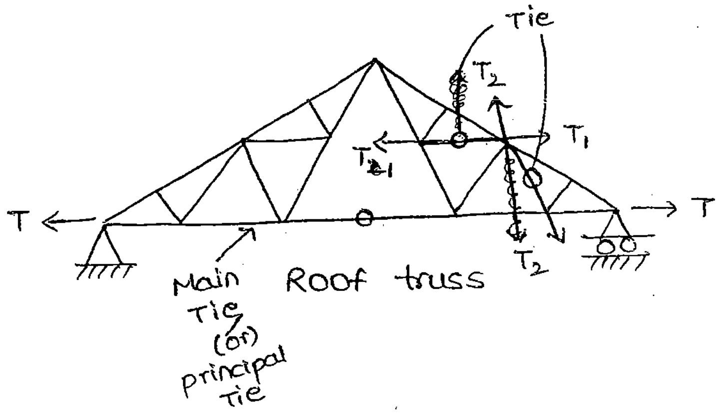
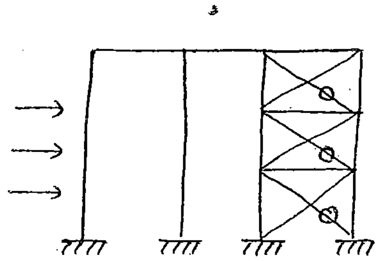
UNIT - 5

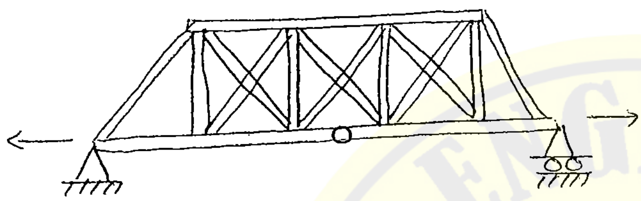
TENSION MEMBERS



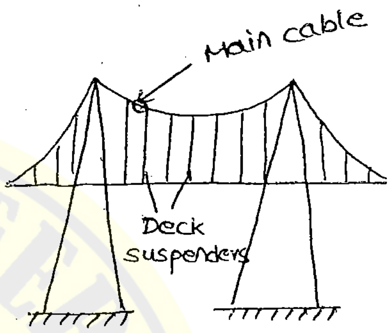
Roof truss



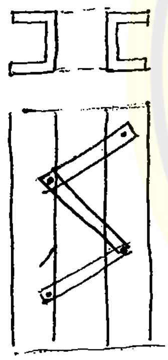
Braced Frame



Truss Bridge



Suspension bridge



Builtup column with lacing

Axial tensile strength of a member (or) a section (P_t):-

$$P_t = A_{net} \times \sigma_{at}$$

A_{net} = Net effective sectional area of a member

σ_{at} = permissible (or) Allowable axial tensile stress

= $0.6 f_y$ (IS 800 : 1984)

f_y = yield stress for steel

$$\text{working stress} = \frac{P}{A_{net}} \leq \sigma_{at} = 0.6 f_y$$

$$P \leq A_{net} \times \sigma_{at} = P_t$$

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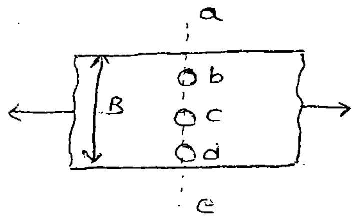
Net sectional area of a member (A_{net}):-

1. plate with chain riverting (or) bolting:

$$A_{net} = A_g - \text{Area of rivert or bolt holes}$$

A_{net} along section a-b-c-d-e



$$A_{net} = B \cdot t - n d t$$

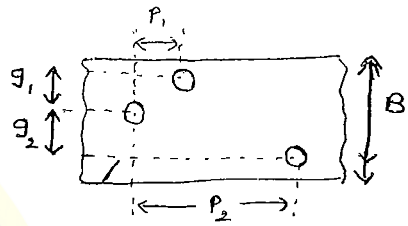
$$= (B - nd)t$$

n = no. of rivert holes along section

d = Gross dia of rivert

2. Plate with staggered (or) zig zag riverting (or) Bolting :-

A_{net} along section a-b-c-d-e



$$A_{net} = (B - nd)t + \frac{P_1^2 t}{4g_1} + \frac{P_2^2 t}{4g_2}$$

d = Gross (or) effective dia of rivert

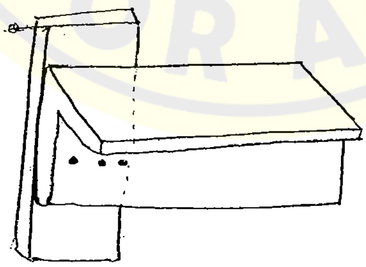
B = width of the plate

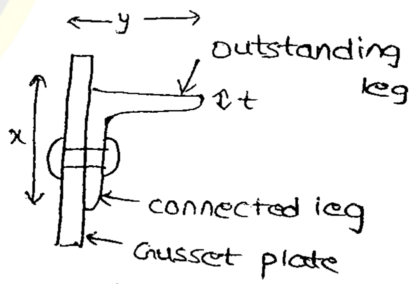
P_1 & P_2 = staggered pitches (Measured parallel to direction of a load)

g_1 & g_2 = gauge distances.

3. Single angle connected by one leg only :-

i. Riverted Angle :-





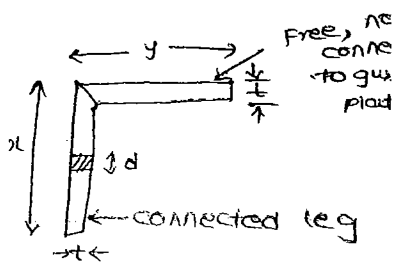
$$A_{net} = A_1 + A_2 k_1$$

k_1 = Reduction factor

$$\boxed{k_1 = \frac{3A_1}{3A_1 + A_2} = \frac{1}{1 + 0.33\, A_2/A_1}}$$

A_1 = Net sectional area of connected leg

$$= xt - dt - \tfrac{1}{2} t \cdot t$$



$$\boxed{A_1 = (x - d - \tfrac{t}{2})t}$$

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A_2 = Gross sectional area of outstanding leg

$= yt - \frac{1}{2}tt$

$$\boxed{A_2 = (y - \tfrac{t}{2})t}$$

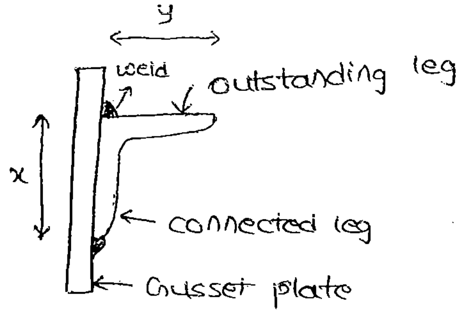
(ii) Welded Angle:-

$A_1 = (x - \frac{t}{2})t$

$A_2 = (y - \frac{t}{2})t$

$A_{net} = A_1 + A_2 K_1$

$K_1 = \dfrac{3A_1}{3A_1 + A_2}$



Note:-

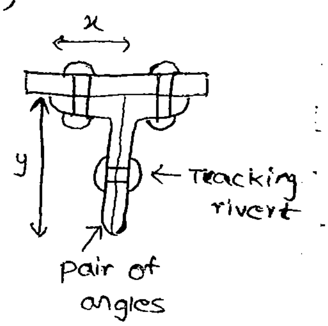
The net area of welded angle tie member is higher than net sectional area of riverted angle or bolted angle. Hence welded angle tie has more tensile strength than riverted (or) bolted angle tie.

Unequal angle tie its longer leg is connected to gusset plate provides more net area than equal angle section with same sectional area. Hence unequal angle section has more tensile strength than equal angle section.

4. Pair of an angles [(or) single Tee's] placed back to back connected by one leg of each angle [(or) flange of Tee] to the same side of gusset plate (Tracking riverts).

$A_{net} = A_1 + A_2 K_2$

$K_2 = \dfrac{5A_1}{5A_1 + A_2} = \dfrac{1}{1 + 0.2\frac{A_2}{A_1}}$

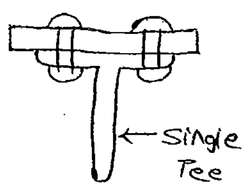


A_1 = Net sectional area of connected leg

$= 2[x - d - t/2]t$

A_2 = Gross sectional area of outstanding leg

$= 2[y - \frac{t}{2}]t$



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If not tacking riverted

$$A_{net} = A_1 + A_2 k_1$$

$$\boxed{k_1 = \frac{3A_1}{3A_1 + A_2} = \frac{1}{1 + 0.33\frac{A_2}{A_1}}}$$

$$A_1 = 2\left(x - d - \frac{t}{2}\right)t$$

$$A_2 = 2\left(y - \frac{t}{2}\right)t$$

5. Pair of angles [(or) Double Tee's] placed back to back, connected one leg of each angle [(or) Flange of Tee] to th either side of Gusset plate (Tacking Riverted).

$$A_{net} = A_g - \text{Area of rivert holes}$$

If not tacking riverted :-

$$A_{net} = A_1 + A_2 k_1$$

$$k_1 = \frac{3A_1}{3A_1 + A_2} = \frac{1}{1 + 0.33\frac{A_2}{A_1}}$$

Design of axially loaded tension members:-

Slenderness ratio of tension member (λ):-

$$\lambda = \frac{\text{unsupported length}}{\text{Min. radius of gyration}}$$

$$\lambda = \frac{L}{r_{min}}$$

** Limiting (or) Maximum slenderness ratio (λ_{limit}) (IS 800 stiffness crite

1. A tension member is subjected to load reversal (or) stress reversal, Due to loads other than wind (or) earth forces

S.R due to loads other than W.L (or) E.L $\leq$ 180

$$\frac{L}{r_{min}} \leq 180$$

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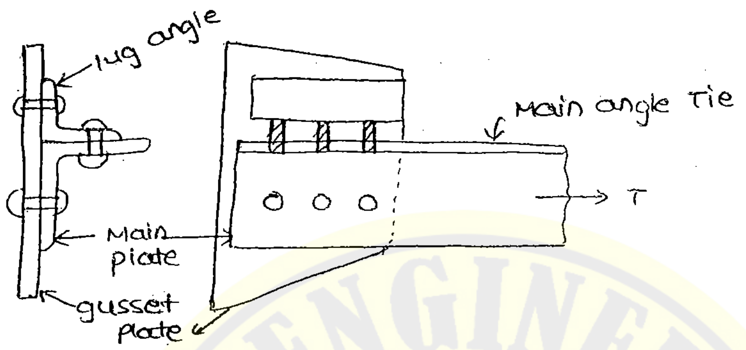
2. A member is used as a tie member in roof truss or a bracing system subjected to load or stress reversal due to wind loads (or) earthquake loads.

S.R due to W.L (or) E.L $\leq$ 350

3. For any other tension members (other than pretensioned member) $\leq$ 400

Lug angle:-



Lug angle is a short length of an angle used at a joint location to join outstanding leg of angle type to the gusset plate (Outstanding flange of a channel type to the gusset plate) so that length of connection is reduced. By using Lug angle strength and efficiency of outstanding leg of an angle type and outstanding flange of a channel type may be improved.

$$A_{net} = A_g - \text{Area of rivert holes}$$

P.g NCL-37

1. $A_{net} = A_1 + A_2 \cdot K_1$

$= a + bk$

$K_1 = \frac{3A_1}{3A_1 + A_2}$

$= \frac{3a}{3a+b}$

$= \frac{1}{1+\frac{b}{3a}}$

$= \frac{1}{1+0.33\frac{b}{a}}$

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3. Given $A_1 = 50 \text{ mm}^2$ $A_2 = 100 \text{ mm}^2$

$$A_{net} = A_1 + A_2 k_1$$

$$k_1 = \frac{3(50)}{3(50)+100}$$

$$= \frac{150}{250} = \frac{3}{5}$$

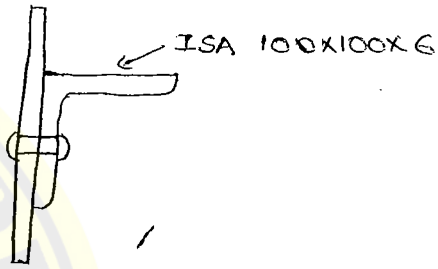
$$A_{net} = 50 + 100\left(\frac{3}{5}\right)$$

$$= 50 + 60$$

$$= 110 \text{ mm}^2$$

4. Diameter of rivert = $\underbrace{20 \text{ mm}}_{\text{Nominal dia}}$, $\underbrace{d = 21.5 \text{ mm}}_{\text{Gross dia}}$

Net effective area $A_{net} = ?$



$A_1 = \cancel{(100\times6)}$ — Net area of connected leg

$$A_1 = (x - d - t/2)t$$

$$= (100 - 21.5 - \tfrac{6}{2})6$$

$$= 453 \text{ mm}^2$$

A_2 = Gross area of Outstanding leg

$$= (y - \tfrac{t}{2})t$$

$$= (100 - \tfrac{6}{2})6$$

$$= 582 \text{ mm}^2$$

$$k_1 = \frac{3A_1}{3A_1 + A_2} = \frac{3 \times 453}{3\times453 + 582} = 0.70$$

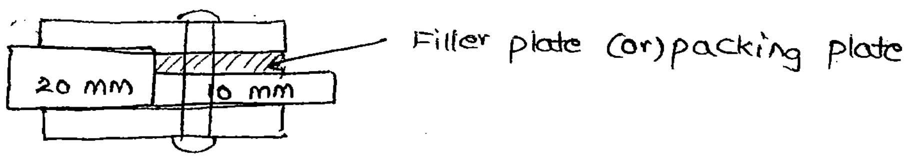
$$A_{net} = A_1 + A_2 k_1$$

$$= 453 + 582(0.7)$$

$$= 860.4 \text{ mm}^2$$

∴ If nominal dia is use then ans is wrong.

5.



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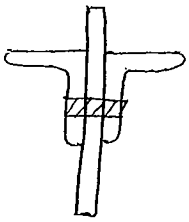
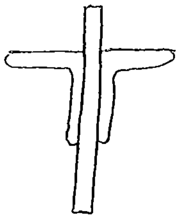
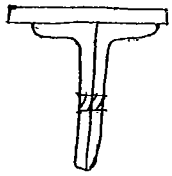
8. (a)



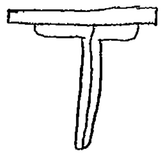
(b)



(c)



(d)



(a) $A_{net} = A_g$ - area of rivert hole

(b) $A_{net} = A_1 + A_2 K_1$, $K_1 = \frac{3A_1}{3A_1 + A_2}$

(c) $A_{net} = A_1 + A_2 K_2$, $K_2 = \frac{5A_1}{5A_1 + A_2}$

(d) $A_{net} = A_1 + A_2 K_1$, $K_1 = \frac{3A_1}{3A_1 + A_2}$

$$P_t = A_{net} \times \sigma_{at}$$

$$\boxed{\therefore P_{t(a)} > P_{t(c)} > P_{t(b)} > P_{(t)(d)}}$$

9. $f_y = 400$ Mpa

$\sigma_{at} = 0.6 f_y = 0.6 \times 400$

$= 240$ Mpa

10. Given $B = 300$ mm $t = 10$ mm $\phi = 16$ mm $d = 17.5$ mm (Gross dia)

$A_{net} = (B - d)t$

$= (300 - 17.5)10$

$= 2825 \text{ mm}^2$

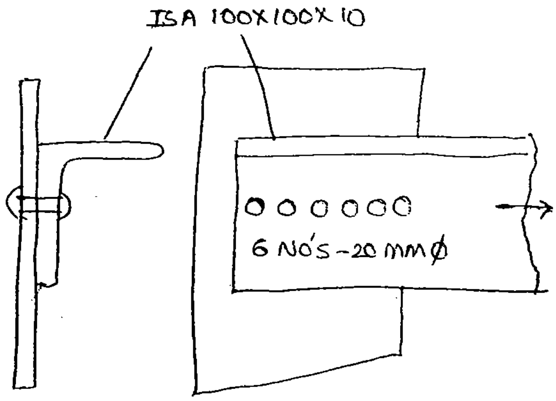
11. Given $B = 300$ mm, $t = 10$ mm, $\phi = 18$ mm, $d = 19.5$ mm

$A_{net} = (B - d)t$

$= (300 - 19.5) \times 10$

$= 2805 \text{ mm}^2$

12. $\sigma_{at} = 150 \text{ N/mm}^2$



$P_t = A_{net} \times \sigma_{at}$

$A_{net} = A_1 + A_2 K_1$

$A_1 = (x - d - \frac{t}{2})t$

$= (100 - 21.5 - \frac{10}{2})10$

$= 735 \text{ mm}^2$

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$$A_2 = \left(y - \frac{t}{2}\right)t$$

$$= \left(100 - \frac{10}{2}\right)10$$

$$= 550 \text{ mm}^2$$

$$k_1 = \frac{3A_1}{3A_1 + A_2}$$

$$= \frac{3 \times 735}{3 \times 735 + 550}$$

$$= 0.698$$

$$A_{net} = 735 + 550\,(0.698)$$

$$= 1399 \text{ mm}^2$$

$$P_t = A_{net} \times \sigma_{at}$$

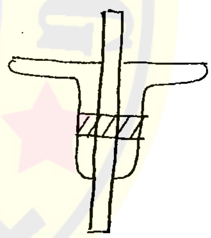
$$= 1399 \times 150$$

$$= 209.85 \times 10^3 \text{ N}$$

$$= 209.85 \text{ kN}$$

13. $A_1 = 775 \text{ mm}^2$

$A_2 = 950 \text{ mm}^2$



If not tack riverted

$$A_{net} = A_1 + A_2 k_1$$

$$k_1 = \frac{3A_1}{3A_1 + A_2}$$

$$= \frac{3(775)}{\cancel{3(775) + 950}} = \frac{3\,(2 \times 775)}{3(2 \times 775) + (2 \times 950)}$$

$$= 0.71$$

$$A_{net} = (2 \times 775) + (2 \times 950)(0.71)$$

$$= 2899 \text{ mm}^2$$

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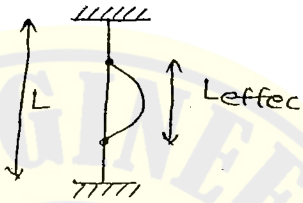
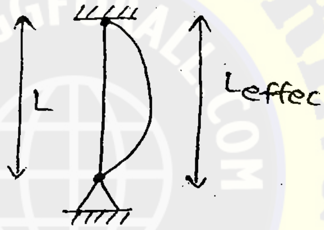
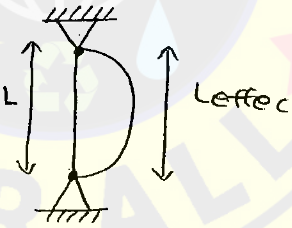
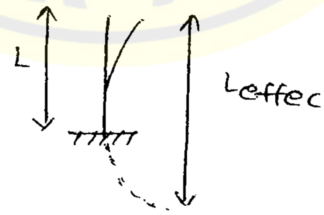
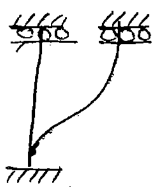
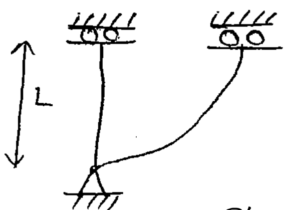
Effective Length of a compressive member(L_{effec}):-

$L_{effective}$ depends on

1. Type of end condition (such as fixed, pinned, roller, free etc)
2. No. of riverts (or) bolts (or) length of weld used at a joint
3. No. of meembers are meeting at a joint.

L = unsupported Length of column.

***

End condition	Diagramatic Representation	Actual Length	Effective length
1. Fixed - Fixed		(0.5L)	0.65L
2. Fixed - Hinged		(0.707L)	0.80L
3. pinned - pinned			1.0L
4. Fixed - Free			2.0L
5. Fixed - moment roller			1.2L
6. pinned - Moment roller			2.0L

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Design of axially loaded compression member:-

1. Allowable axial compressive stress in compression member is assumed.

   For angle strut $\sigma_{ac} = 65$ Mpa

   For beam section columns, $\sigma_{ac} = 80$ Mpa

   For Heavy axial columns load, $\sigma_{ac} = 110$ Mpa

2. Calculate approximate sectional area required to support axial compressive load, P

$$A_e \text{ required} = \frac{P}{\sigma_{ac} \text{ assumed}}$$

3. Select a trail section from steel table to match with sectional area required and calculate minimum radius of gyration of total section.

4. Effective length of trial section is to be calculated base on end conditions and slenderness ratio of trial section to be calculated which should be less than maximum slenderness ratio as per IS 800.

** Maximum slenderness ratio (Stiffness Requirement):-

1. A member is subjected to compressive loads due to ~~loads~~ dead and imposed loads

$$\lambda_{max} \leq 180$$

2. A member is subjected to compressive load due to wind (or) Earthquake loads

$$\lambda_{max} \leq 250$$

3. For compression flange of beam, $\lambda_{max} \leq 300$

Built up sections (or) columns:-

Connecting systems used for builtup columns are:

1. Lacing system (Generally preffered for eccentrically loaded columns)

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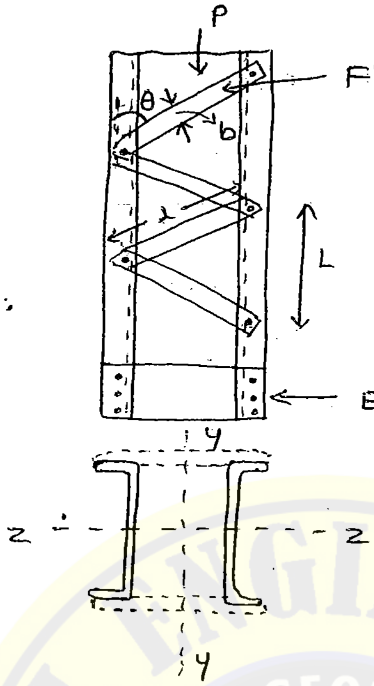
2. Battens (Battens are normally used for axially loaded columns)

Lacing system:-



Flats, angles, channels and tube sections are used for section of Lacing members.

θ = angle of inclination with longitudinal axis

L = spacing of Lacing

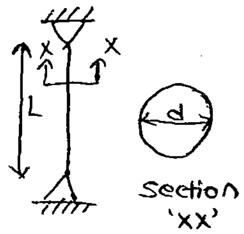
l = Length of lacing member

b = width of flat lacing member

t = thickness of flat lacing

P = Axial column load.

P.g No:- 46



1. Slenderness ratio, $\lambda = \dfrac{\text{effective length}}{\text{min. radius of gyration}}$

$$L_{effec} = L$$

$$r_{min} = \sqrt{\frac{I_{min}}{A}} = \sqrt{\frac{I}{A}}$$

$$= \sqrt{\frac{\pi d^4}{64} \Big/ \frac{\pi d^2}{4}}$$

$$= \frac{d}{4}$$

$$\lambda = \frac{l_{eff}}{r_{min}} = \frac{L}{\frac{d}{4}}$$

$$200 = \frac{4l}{d} \Rightarrow \frac{l}{d} = 50$$

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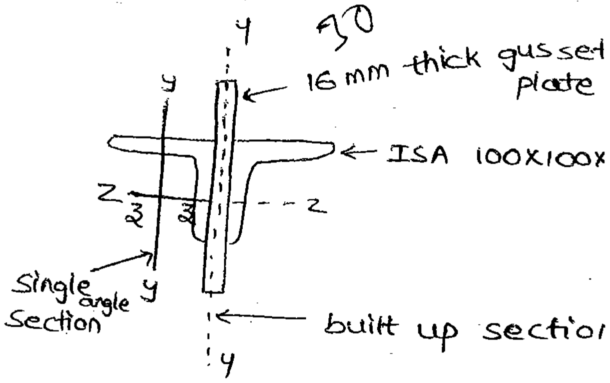
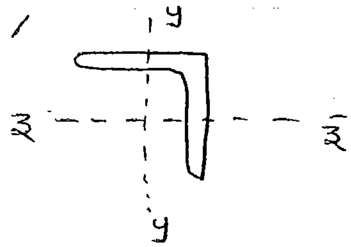
6. Given ISA $100 \times 100 \times 10$



$$A = 1903 \text{ mm}^2$$

$$I_{33} = I_{yy} = 177 \times 10^4$$

$$r_{zz} = \sqrt{\frac{I_{zz}}{A_e}} \qquad r_{yy} = \sqrt{\frac{I_{yy}}{A_e}}$$



$$I_{zz} = 2\left[I_{C.G} + A y^2\right]$$

$$= 2\left[I_{33} + Ay^2\right]$$

$$= 2\left[I_{33} + A(0)^2\right]$$

$$= 2 \cdot I_{33} \qquad \therefore I_{min} = I_{zz} = 2 \cdot I_{33}$$

$$I_{yy} = 2\left[I_{yy} + A3^2\right]$$

$$= 2 I_{yy} + 2 \cdot A 3^2$$

A_e = combined angle area $= A + A$ $= 2A$

$$r_{min} = \sqrt{\frac{I_{min}}{A_e}}$$

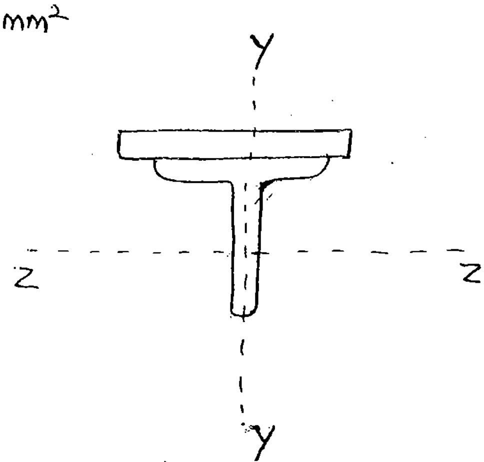
$$= \sqrt{\frac{2 I_{33}}{2A}}$$

$$= \sqrt{\frac{177 \times 10^4}{1903}}$$

$$= 30.6 \text{ mm}$$

8. Given ISA $150 \times 150 \times 10$ mm $A = 2921 \text{ mm}^2$

$$I_{yy} = I_{33} = 6335000 \text{ mm}^4$$



I_{min} = minimum of I_{zz} (or) I_{yy}

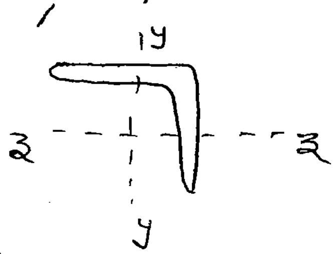
$$I_{zz} = 2\left[I_{33} + Ay^2\right]$$

$$= 2 I_{33} + 2A(0)^2$$

$$I_{zz} = 2I_{33}$$

$$I_{yy} = 2\left[I_{yy} + A3^2\right]$$

$$= 2I_{yy} + 2A(40.8)^2$$



$$\therefore I_{min} = 2 I_{33}$$

$$r_{min} = \sqrt{\frac{I_{min}}{A_e}} = \sqrt{\frac{2I_{33}}{2A}} = \sqrt{\frac{6335000}{2921}} = 40.6 \text{ mm}$$

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1M ** Lacing system :-

General specification :-

1. The radius of gyration normal to the plane of lacing not less than parallel to the plane of lacing.

$$r_{yy} \not< r_{zz}$$

To have a optimum condition (or) economical design

$$r_{yy} = r_{zz}$$

$$I_{yy} = I_{zz}$$

$$2[I_{yy} + A z^2] = 2[I_{zz} + Ay^2]$$

$$2 I_{yy} + 2A z^2 = 2 I_{zz} \qquad \therefore y = 0$$

Note :-

For single lacing system the lacing on one plane should reflect mirror image to the lacing on other plane.

2. There should not be any variation in the lacing system (The spacing of lacing, angle of inclination of lacing should be maintained throughout the height of the column).

Design specification :-

1. $40^\circ \leq \theta \leq 70^\circ$

Optimum angle used in practise

$\theta = 45^\circ$ to 50°

** 2. slenderness ratio of lacing member $(\lambda)_{lacing} \leq 145^\circ$

$$(\lambda)_{lacing} = \left(\frac{L_{effec}}{r_{min}}\right)_{of\ lacing} \leq 145^\circ$$

The above condition is required to eliminate local buckling failure of lacing member.

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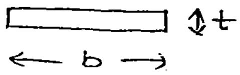
** Effective length of lacing member (l_e) :-

	l_{eff}
1. For single lacing with one rivert (or) one bolt	l
2. For Double lacing	$0.7l$
3. For welded lacing	$0.7l$

For Flat lacing (bxt)



Min. radius of gyration

$$r_{min} = \sqrt{\frac{I_{min}}{A}}$$

$$= \sqrt{\frac{bt^3/12}{bt}}$$

$$= \frac{t}{\sqrt{12}}$$

Minimum width of flat lacing

$(b)_{min}$ = 3 x Nominal diameter of rivert
(or)
shank diameter of ~~rivert~~ Bolt

Nominal dia of rivert (mm)	16	18	20	22
Min. width of flat lacing b_{min} (mm)	50	55	60	65

Minimum thickness of flat lacing

$t_{min} \not< \frac{l}{40}$ for single lacing

$\not< \frac{l}{60}$ for Double lacing

$$\lambda_{lacing} = \frac{l_{effec}}{r_{min}}$$

$$= \frac{l_{effec}}{t/\sqrt{2}} \leq 145$$

$$= \frac{\sqrt{2} \cdot l_{effec}}{t} \leq 145 \text{ for flat lacing}$$

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4. $\frac{L}{r_c^{min}} \leq 50$

$\leq 0.7 \times$ slenderness ratio of whole builtup column whichever is less.

5. r_c^{min} = minimum radius of gyration of individual column component.

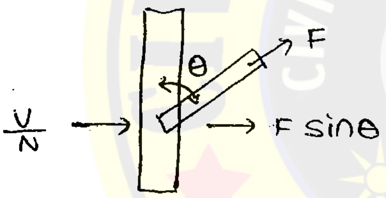
The above condition is required to eliminate local buckling failure of individual column component between lacings.

The lacing member should be design for transfer shear of 2.5% axial column load.

5. Transfer shear force (V) = 2.5% Axial column load (P)

$$\boxed{V = \frac{2.5\,P}{100}}$$

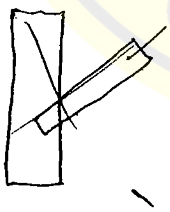
6. Axial force in lacing member is $F = \frac{V}{N\sin\theta}$



$\Sigma F_x = 0$

$\frac{V}{N} + F\sin\theta = 0$

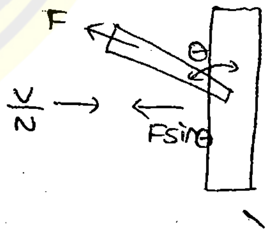
$F = \frac{-V}{N\sin\theta}$



$\Sigma F_x = 0$

$\frac{V}{N} - F\sin\theta = 0$

$F = \frac{V}{N\sin\theta}$



7. Axial force in lacing member (F)

$F = \frac{V}{2\sin\theta}$ (N = 2, for single lacing)

$F = \frac{V}{4\sin\theta}$ (N = 4, for double lacing)

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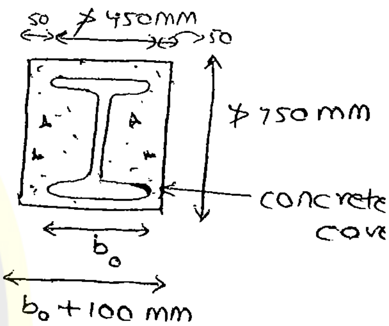
Batten system:-

General specification:-

1. The effective length of batten & column should be increased by 10% as per IS specification.
2. The batten should divide the entire column into minimum of three components. (A min. four no. of battens may required for batten column out of this 4 battens two are intermediate battens and other two are end battens).

Encased column (or) cased column:-



1. Minimum width of encased column

$$= b_o + 100 \text{ mm}$$

b_o = width of steel column

2. Maximum dimensions of encased column ≯ 450 mm × 750 mm
3. Minimum concrete cover = 50 mm
4. Minimum radius of gyration of encased column

$$\boxed{r_{min} = 0.2(b_o + 100)}$$

P.g No:- 46

4. Given $N = 2$, $\theta = 45^\circ$ $P = 1000$ kN

Transverse shear, $V = \dfrac{2.5P}{100}$

$$= \frac{2.5(1000)}{100}$$

$$= 25 \text{ kN}$$

5. Axial force in ~~leading~~ lacing

$$F = \frac{V}{N\sin\theta}$$

$$= \frac{V}{2\sin 45^\circ}$$

$$= \frac{25}{2\sin 45^\circ}$$

$$= 17.68 \text{ kN}$$

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P.g NO:- 47

1. Given $r_{min} = 25\ mm$

Maximum length of compression member L_{max}

$$\frac{L_{effec}}{r_{min}} \leq \lambda_{limit}$$

$$\frac{l_{effec}}{25} \leq 180$$

$$l_{effec} \leq 180 \times 25$$

$$\leq 4500\ mm$$

$$\leq 4.5\ m$$

2. $t_{min} \geq \frac{L}{40}$

$$\geq \frac{L}{60} = \frac{500}{60} = 8.33\ mm$$

$$= 10\ mm$$

13. $P = 2.5 \times 10^6\ N$

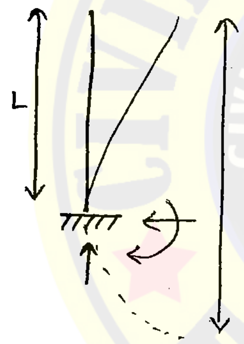
$$= 2.5 \times 10^3\ kN$$

$$V = \frac{2.5\ P}{100}$$

$$= \frac{2.5\ (2.5 \times 10^3)}{100}$$

$$V = 62.5\ kN$$

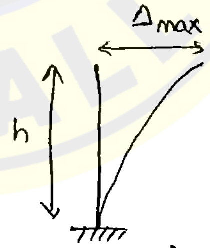
6.



$L_{effec} = 2L$

8. $\Delta_{max} \leq 2 \times \frac{h}{325}$

$$\frac{\Delta_{max}}{h} \leq \frac{2}{325}$$



9. $L_{effec} = 0.8\ L$

$$= 0.8 \times 5$$

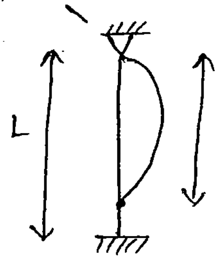
$$= 4\ m$$

$r_{min} = 100$

$$\lambda = \frac{l_{eff}}{r_{min}}$$

$$= \frac{4000}{100}$$

$$= 40$$



$L_{effec} = 0.8L$

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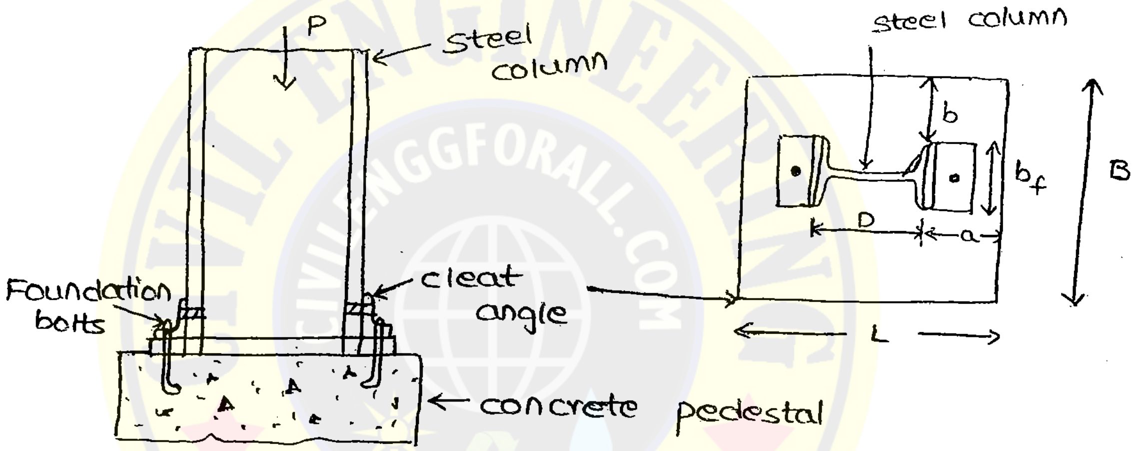
UNIT - 7

DESIGN OF COLUMN BASES & COLUMN SPLICES

Types of column base:-

1. Slab base (Generally provided under axially loaded columns
2. Gusseted base (Normally preffered under heavy axially loaded columns and column is subjected to axial load with moment).
3. Grillage foundation (when heavy loaded columns is supported by black cotton soils).

Design of slab base:-



P = axial load in column

L = length of slab base

B = width of slab base

t_s = thickness of slab base

a = Bigger projection of slab base beyond the steel column

b = lesser projection of slab base beyond the steel column

→ Area of slab base, $A = \dfrac{\text{Axial of column load}}{\text{Permissible bearing stress in concrete}}$

$$A = \frac{P}{\sigma_c}$$

→ For square slab base, $L = B = \sqrt{A}$

side of slab base, $L = B = \sqrt{A}$

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→ For rectangular slab base :-

$A = L \times B$ $\therefore a = b$

$$\boxed{A = (D+2a)(b_f+2b)}$$

→ To have a optimum thickness of slab base, the condition must be $a=b$

$$\boxed{A = (D+2a)(b_f+2a)}$$

→ Upward pressure from concrete pedestal (w)

$$w = \frac{\text{Axial column load}}{\text{provided area of Slab base}}$$

$$\boxed{w = \frac{P}{A_1}}$$

→ Thickness of slab base (t_s)

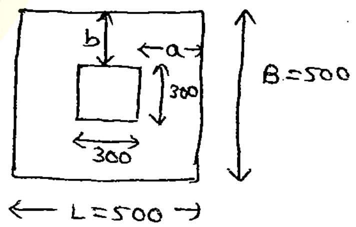
$$\boxed{t_s = \sqrt{\frac{3w}{\sigma_{bs}}\left(a^2 - \frac{b^2}{4}\right)}}$$

σ_{bs} = permissible bending stress in slab base

σ_{bs} = 185 Mpa (for all steels)

P.g No:- 52

1. Axial column load, $P = 2\,MN$
$= 2 \times 10^6\,N$



$\sigma_{bs} = 200$ Mpa

$$t_s = \sqrt{\frac{3w}{\sigma_{bs}}\left(a^2 - \frac{b^2}{4}\right)}$$

$$a = b = \frac{500-300}{2} = 100$$

$$w = \frac{P}{A} = \frac{2\times10^6}{500\times500} = 8\ N/mm^2$$

$$t_s = \sqrt{\frac{3\times8}{200}\left(100^2 - \frac{100^2}{4}\right)}$$

$= 30$ mm

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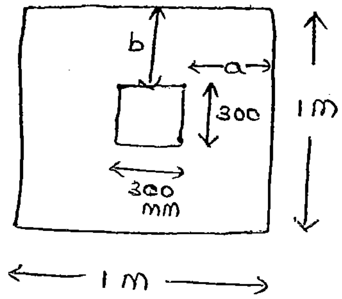
P.g NO:- 51

3. Given $P = 2000\ kN$

$\sigma_{bc} = 185\ Mpa$



$$w = \frac{P}{A} = \frac{2000\times10^3}{1000\times1000}$$

$$= 2\ N/mm^2$$

$$a = b = \frac{1000-300}{2} = 350\ mm$$

$$t_s = \sqrt{\frac{3w}{\sigma_{bs}}\left(a^2 - \frac{b^2}{4}\right)}$$

$$= \sqrt{\frac{3\times2}{185}\left(350^2 - \frac{350^2}{4}\right)}$$

$$= 55\ mm$$

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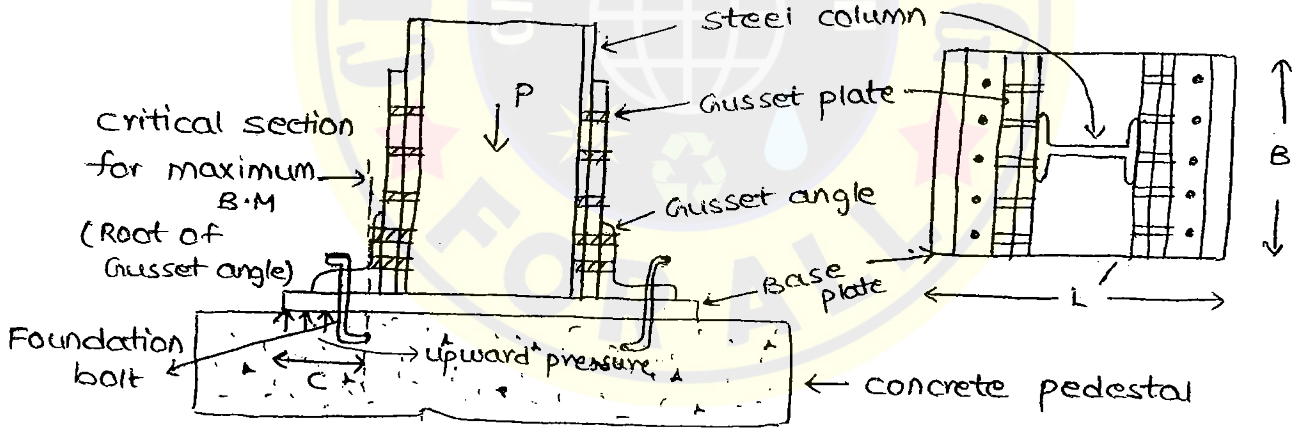
Design of Gusseted Base:-



L = Length of Base plate

B = width of Base plate

t = thickness of base plate

c = cantilever projection beyond the root of Gusset angle

Area of Base plate, $A = \dfrac{\text{Axial column load}}{\text{permissible bearing stress in concrete}}$

$$\boxed{A = \frac{P}{\sigma_c}}$$

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→ Length of Base plate, L = Depth of steel column + 2 x thickness of gusset plate + 2 x leg width of Gusset angle + 2 x min. overhand (for riverted / bolted connection)

/ L = Depth of steel column + 2 x thickness of Gusset plate + 2 x Min. overhand (for welded connection)

→ Width of the base plate, $B = \dfrac{\text{Area of Base plate}}{\text{length of Base plate}}$

→ Upward pressure from concrete pedestal, $\boxed{w = \dfrac{P}{A_1}}$

$$M = w \cdot c \cdot \frac{c}{2}$$

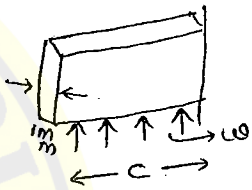
$$M \leq M_d$$

$$\leq \sigma_{bs} \cdot z$$

$$\frac{wc^2}{2} = \sigma_{bs} \cdot \frac{t^2}{6}$$

$$z = \frac{I}{y} = \frac{\frac{1 \cdot t^3}{12}}{t/2}$$

$$z = t^2/6$$



$$t = c\sqrt{\frac{3w}{\sigma_{bs}}}$$

→ ∴ Thickness required, $\boxed{t = c\sqrt{\dfrac{3w}{\sigma_{bs}}}}$

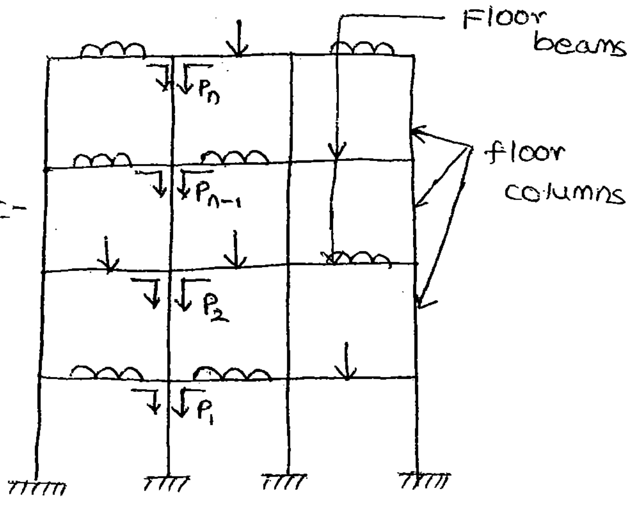
→ Thickness of base plate, $t_b = t -$ thickness of Gusset angle (for riverted / bolted connection)

$t_b = t$ (for welded connection)

Column splice :-

Let $P_1, P_2, \ldots, P_{n-1}, P_n$ axial column loads in various floor columns from bottom floor respectively.

$$P_1 > P_2 > \ldots > P_{n-1} > P_n.$$



Steel frame

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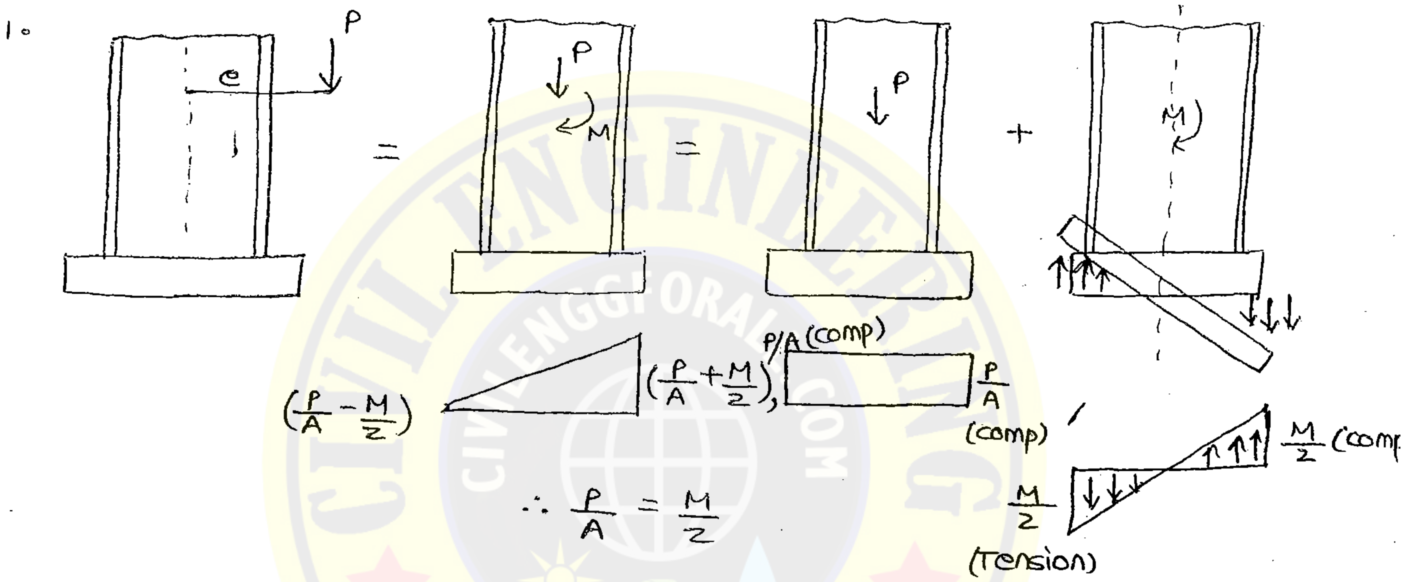
Note:-

If ends of columns are machined or milled (or) smooth end theoretically no connecting system is required. But in practice connection to be designed for 50% column load and full moment if any.

If ends of columns are not machined complete bearing connecting system to be designed for full column load and full moment if any.

P.g NO:-51

1.



$\therefore \frac{P}{A} = \frac{M}{Z}$

P.g NO:-52

4. Given $\sigma_{bs} = 185$ Mpa

$$w = \frac{P}{A}$$

In all four diagrams Area is same than w is const

$$t = c\sqrt{\frac{3w}{\sigma_{bs}}}$$

$$t \propto c$$

If 'c' is less then base plate thickness is minimum

ⓐ → $c = \frac{600-140}{2} = 230$

ⓑ → $c = \frac{600-400}{2} = 100$

ⓒ → $c = \frac{500-140}{2} = 80$.

ⓓ → $c = \frac{720-400}{2} = 160$

$\therefore \downarrow t \propto \downarrow c$

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1-2-20

UNIT - 8

DESIGN OF BEAMS

Different terms used to designate a beam

1. Joist
2. Floor Beam
3. Girder (or) Floor beam
4. Lintel
5. purlin
6. Spandrel
7. Stringer
8. Grit
9. Header

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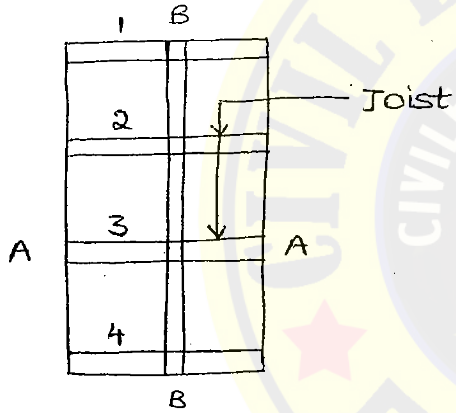
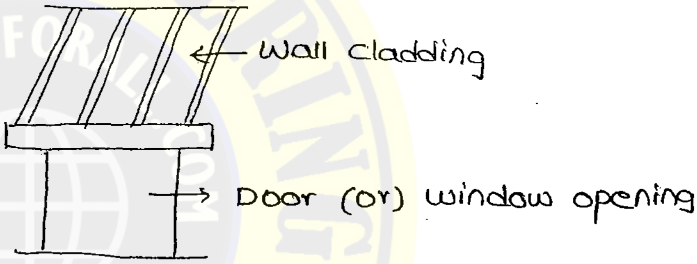
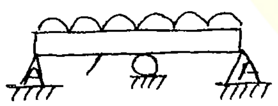
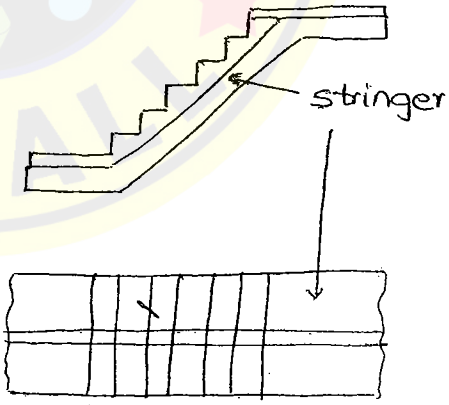
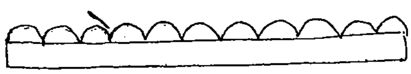




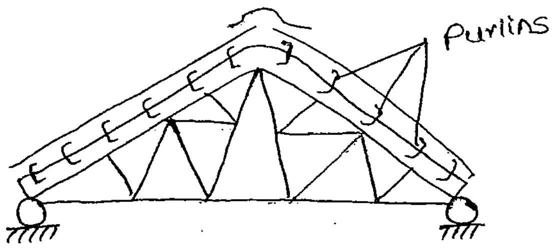


Beam A-A

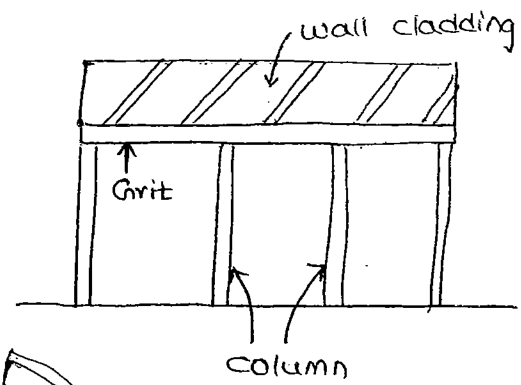


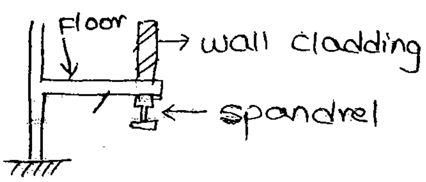


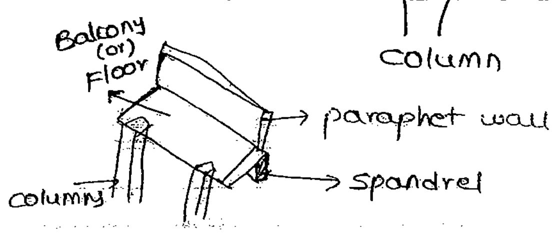
Floor beam (B-B)



Roof Building







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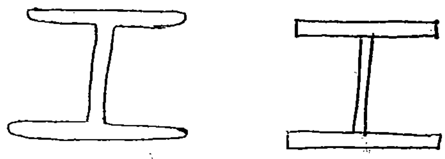
Types of section :-

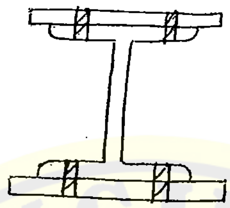
1. Rolled I - section (or) I - beam (or) I - section is built with plates.



2. Rolled channel section (for light transverse loads)

   Ex:- purlin, lintel, Grit etc.

3. Built up beams (or) plated beams (or) compounded beams



Design criteria of Beam:-

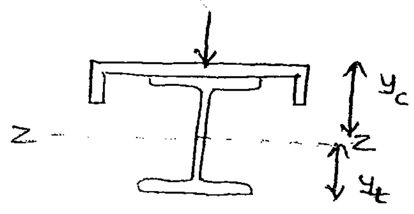
1. Design for Bending moment (M)
2. Design for shear force (V)
3. Check for deflections ($\Delta_{cal} \leq \Delta_{max}$)
4. Check for secondary affects like under concentrated loads and Reactions.
   i. Web buckling
   ii. Web crippling

Lateral stability of beams:-

Based on lateral stability beams are two types.

1. ~~Supported~~ Laterally supported beams (or) Laterally Restraine beams (where compression flange of a beam is not affected by lateral buckling (or) lateral torsional buckling
2. Laterally unrestrained beams (compression flange of a bea is affected to either torsional (or) lateral (or) lateral torsional buckling

Design of Bending Moment (M):-



The actual (or) calculated Bending compressive stress (σ_{bc} cal) (or) calculated Bending tensile stress (σ_{bt} cal)

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$$\sigma_{bc\,cal} = \frac{M}{I} \cdot y_c$$

$$\sigma_{bt\,cal} = \frac{M}{I} \cdot y_t$$

M = Maximum B.M

I = Moment of Inertia about Bending axis

y_c = Distance from axis of bending extreme compressive fibre

y_t = Distance from axis of bending extreme tension fibre.

Permissible bending stresses:-

1. For laterally restrained beams

   a. permissible bending tensile stress

   $$\sigma_{bt} = 0.66 f_y$$

   b. permissible bending compressive stress

   $$\sigma_{bc} = 0.66 f_y$$

2. For laterally unrestrained beams

   $$\sigma_{bt} = 0.66 f_y$$

   σ_{bc} may be calculated by using Rankine merchant formula based on (L/r_{yy})

$$\sigma_{bc} = \frac{0.66 f_y \, f_{cb}}{[(f_y)^n + (f_{cb})^n]^{1/n}} \leq 0.66 f_y$$

f_{cb} = Elastic critical stress in bending $= \dfrac{\pi^2 E}{(L/r_{yy})^2}$.

Note:-

The above permissible bending stresses are increased by $33\frac{1}{3}\%$ when wind loads are used for design calculations.

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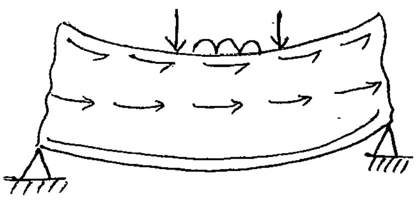
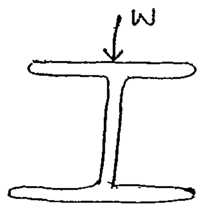
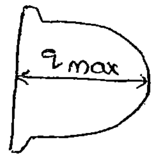
Design for shear force:-

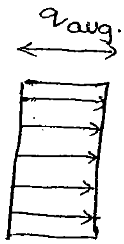


Longitudinal (or) Horizontal shear distribution.

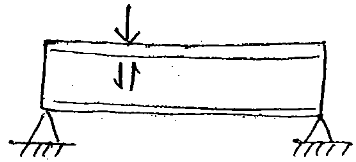




Actual shear stress distribution



Avg. shear stress / (assumed)



Longitudinal shear stress at any level, $q = \dfrac{VA\bar{y}}{Ib}$

V = maximum SF

$A\bar{y}$ = First moment of area

b = width of section, where Intensity of shear stress is calculated.

I = moment of inertia of section about bending axis

Calculated avg. shear stress $\tau_{va\,cal} = \dfrac{V}{Dt_w}$

D = Overall depth of beam

t_w = thickness of a web.

Permissible shear stress:-

Maximum permissible shear stress, $\tau_{va\,max} = 0.45\,f_y$

Permissible avg. shear stress, $= 0.4\,f_y$

Note:-

The above permissible stresses are increased by $33\frac{1}{3}\%$ if wind loads are considered in design calculations

Check for deflections:-

$$\Delta_{cal} \leq \Delta_{max} \text{ (or) } \Delta_{limit}$$

Δ_{cal} = calculated deflection due to external transverse loads.

Δ_{max} (or) Δ_{limit} = Maximum (or) limiting deflections as per IS 800 : 1984.

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Maximum deflection (Δ_{max}) (IS 800 : 1984):-

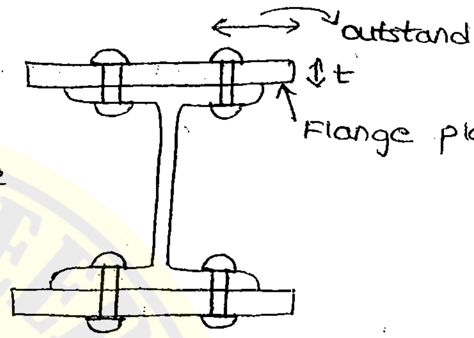
1. For simple span $= \frac{span}{325}$ (or) $\frac{l}{325}$

2. For cantilever beam $= \frac{2 \times span}{325}$ (or) $\frac{2l}{325}$ (or) $2 \times (\Delta_{cal})_{S.S\ Beam}$

3. For purlin $= \frac{span}{200}$ (or) $\frac{l}{200}$

Builtup beams (or) plated (or) compounded Beams :-

Builtup beams are used when beam member is subjected to heavy transverse load with large spans and also used where depth of beam section is restricted to have more clear height.



Builtup beam

M = Maximum Bending moment

Z_r = required section modulus

$$Z_r = \frac{M}{\sigma_{bt}}$$

σ_{bt} = permissible bending stress

Z_a = available section modulus of rolled I-section.

If $Z_r > Z_a$ (plated or builtup beam is to be provided).

Area of each flange plate, $A_p = \frac{Z_r - Z_a}{d}$

d = Overall depth of a beam.

$$I_{B.S} = I_{R.B} + I_{P.L}$$

$$\frac{I_R}{d/2} = \frac{I_a}{d/2} + \frac{A_p \cdot \left(\frac{d}{2}\right)^2 \times 2}{\frac{d}{2}}$$

$$Z_r = Z_a + 2 A_p \cdot d/2$$

$$\boxed{A_p = \frac{Z_r - Z_a}{d}} \qquad \boxed{t = \frac{A_p}{200}}$$

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Outstand :-

1. Outstand $\not>$ 16T (or) $\frac{256T}{\sqrt{f_y}}$ for compression flange (whichever is less)

2. Outstand $\not>$ 20T for Tension flange.

Note :-

The distance from centre of rivert hole (or) bolt hole to the extreme edge of flange plate (outstand) to be limited to avoid local buckling failure of flange plate

T = thickness of outstand.

P.g NO:- 58

1. Given $f_y = 300$ Mpa

$\tau_{va\ max} = 0.45 f_y$
$= 0.45 (300)$
$= 135$ Mpa

2. permissible bearing stress $= 0.75 f_y$
$= 0.75 \times 250$
$= 187.5$ Mpa

6. Given $f_y = 250$ Mpa

$\tau_{va\ max} = 0.45 f_y$
$= 0.45 (250)$
$= 112.5$ Mpa

7. For mild steel, $f_y = 250$ Mpa

$\sigma_{bt} = 0.66 f_y$
$= 0.66 (250)$
$= 165$ Mpa

8. Mild steel, $f_y = 250$ Mpa

$\tau_{va} = 0.4 f_y$
$= 0.4 (250)$
$= 100$ Mpa

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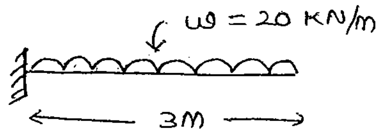
P.9 NO:- 57

1. For ISLB 300

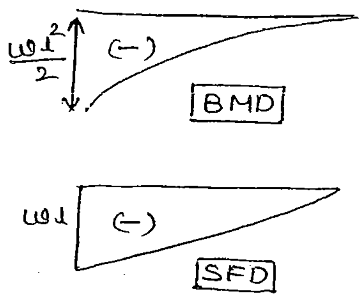


$d = 300$ mm $\quad t_w = 5.4$ mm

$I_{xx} = 1696.6\ cm^4$

$= 1696.6 \times 10^4\ mm^4$

Flange 100 mm × 7.3 mm



$$\text{Max. BM} = \frac{wL^2}{2} = \frac{20(3)^2}{2}$$

$= 90$ KN-m

$= 90 \times 10^6$ N-mm

$$\sigma_{bc\ cal} = \sigma_{bt\ cal} = \frac{M}{I_{xx}} \cdot y$$

$$= \frac{90 \times 10^6}{1696.6 \times 10^4} \times \left(\frac{200}{2}\right)$$

$= 530.45\ N/mm^2$

Max. SF $= V = wl$

$= 20(3)$

$= 60 \times 10^3$ N

$$\tau_{va\ cal} = \frac{V}{d\,t_w}$$

$$= \frac{60 \times 10^3}{200 \times 5.4}$$

$= 55.55\ N/mm^2$

2. Given $V = 56 \times 10^3$ N

ISWB 350, $d = 350$ mm

$t_w = 8$ mm

$$\tau_{va\ cal} = \frac{V}{d\,t_w}$$

$$= \frac{56 \times 10^3}{350 \times 8}$$

$= 20\ N/mm^2$

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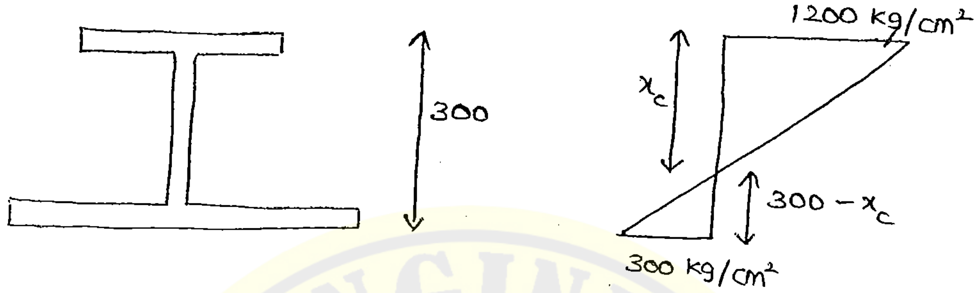
3. 1. $M \le M_d = \sigma_{bt} \cdot z$

2. $\frac{\text{span}}{\text{depth}}$ ratio S.S. Beam – 20, continuous beam – 26, cantilever beam – 7

$\Delta_{cal} < \Delta_{max} = \frac{\text{span}}{325}$

$= 2 \times \frac{\text{span}}{325}$

7.



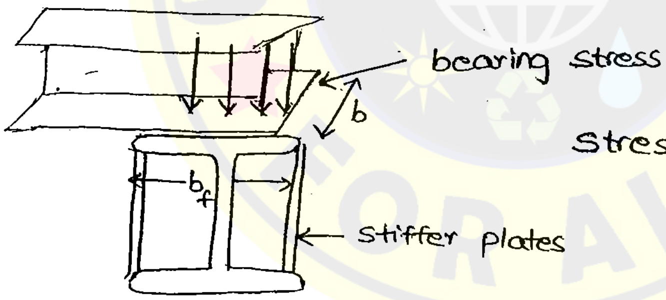
$$\frac{x_c}{1200} = \frac{300 - x_c}{300}$$

$$300\, x_c = 1200 \times 300 - 1200\, x_c$$

$$1500\, x_c = 1200 \times 300$$

$$x_c = 240 \text{ mm}$$

8.



$$\text{Stress} = \frac{W}{b_f \times b} \le \text{permissible stress} \le 0.75 f_y$$

9. $\tau_{va\,cal} = \frac{V}{d \times t_w}$

If increase thickness (t_w), $\tau_{va\,cal}$ will decrease → 1st priority

If increase height (d), $\tau_{va\,cal}$ will decrease → 2nd priority

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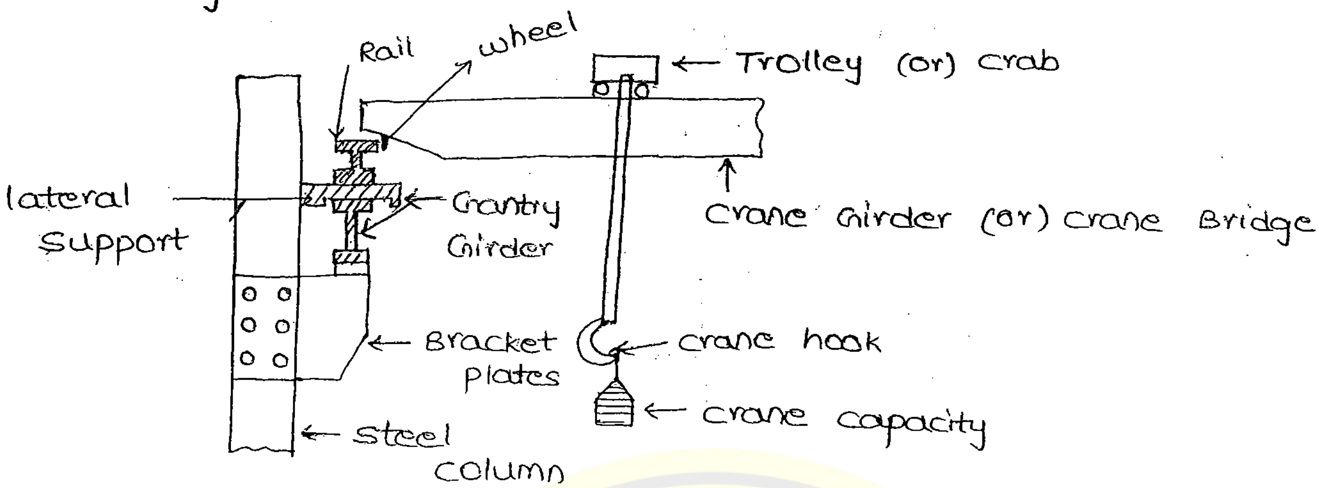
UNIT-10

GANTRY GIRDERS

Gantry Girder:-



EOT cranes:-

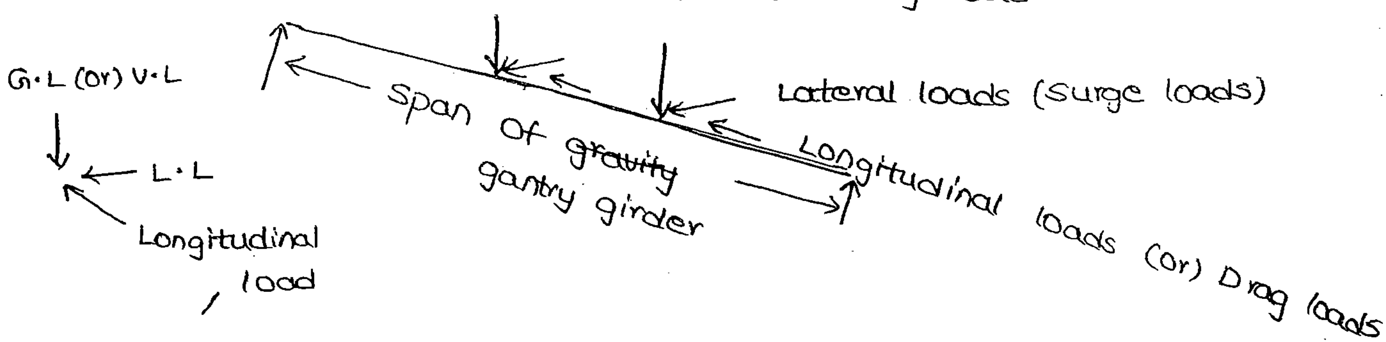
Electrically Operated overhead travelling cranes

MOT cranes (or) HOT cranes:-

Manually (or) Hand Operated Overhead travelling cranes

Design loads on Gantry Girders:-

1. Gravity loads (or) verticle loads
2. Horizontal loads (or) Normal ~~loads~~ rails (or) lateral loads (↔) (or) Surge loads (Due to crab movement)
3. Longitudinal loads (or) Drag loads (Due to crane movement)
4. Impact loads.



→ Lateral loads are weight of crab + wt. lifted by crane

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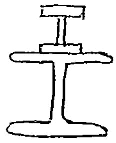
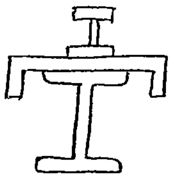
Types of sections:-

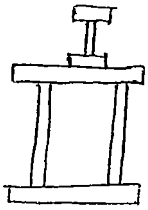
1. ISWB section



2. A channel section is reinforced to compression flange of ISWB section.



3. Box girder



Design criteria of gantry girder:-

1. Design for Bending Moment (M) $(M \leq M_d)$
2. Design for shear force $(V \leq V_d)$
3. Design for deflections $(\Delta_{cal} \leq \Delta_{limit}$ (or) $\Delta_{max})$

** Limiting deflections (or) Maximum deflections (Δ_{max}) (IS 800):-

	Δ_{max}
1. For HOT (or) MOT crane	$L/500$
2. For EOT crane (or) crane capacity upto 50 tons (or) 500 kN	$L/750$
3. For EOT cranes with crane capacity more than 50 tons (or) 500 kN	$L/1000$
4. For other moving equipments	$L/600$

$\therefore$ L = span of Gantry Girder.

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1-2-2015

UNIT - II

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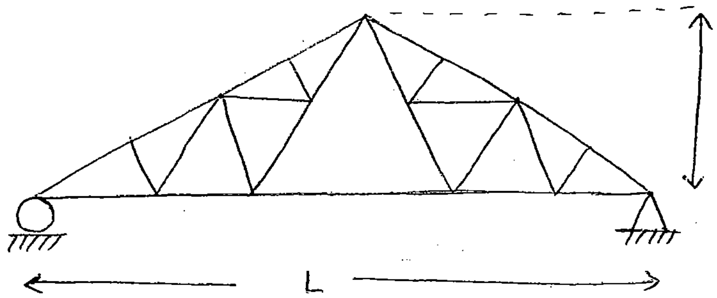
ROOF TRUSSES



Compound Fink Truss

L = span of the truss

h = Rise of the truss

Selection criteria for roof truss:-

1. span of the truss (L)
2. pitch of the truss.

pitch of the truss depends on

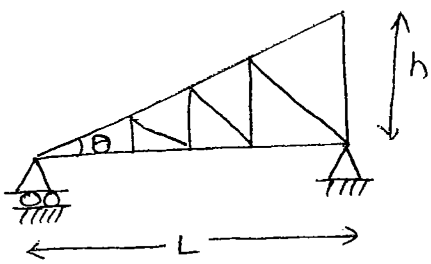
1. Type of roof covering material to be used (like AC sheets, GI sheets, plastic sheets etc).
2. Light of ventilation requirement.

3. $\text{pitch} = \frac{\text{Rise}}{\text{span}} = \frac{h}{L}$

4. $\text{Slope} = \tan\theta = \frac{\text{Rise}}{\text{half span}} = \frac{h}{L/2} = \frac{2h}{L}$

$\boxed{\text{Slope} = 2 \times \text{pitch}}$ for symmetric truss

North light truss:-



$\text{slope} = \tan\theta = \frac{h}{L}$

$\text{pitch} = \frac{h}{L}$

$\text{slope} = \text{pitch} = \frac{h}{L}$ for north light roof truss

Day light is main criteria for selection of North light roof truss.

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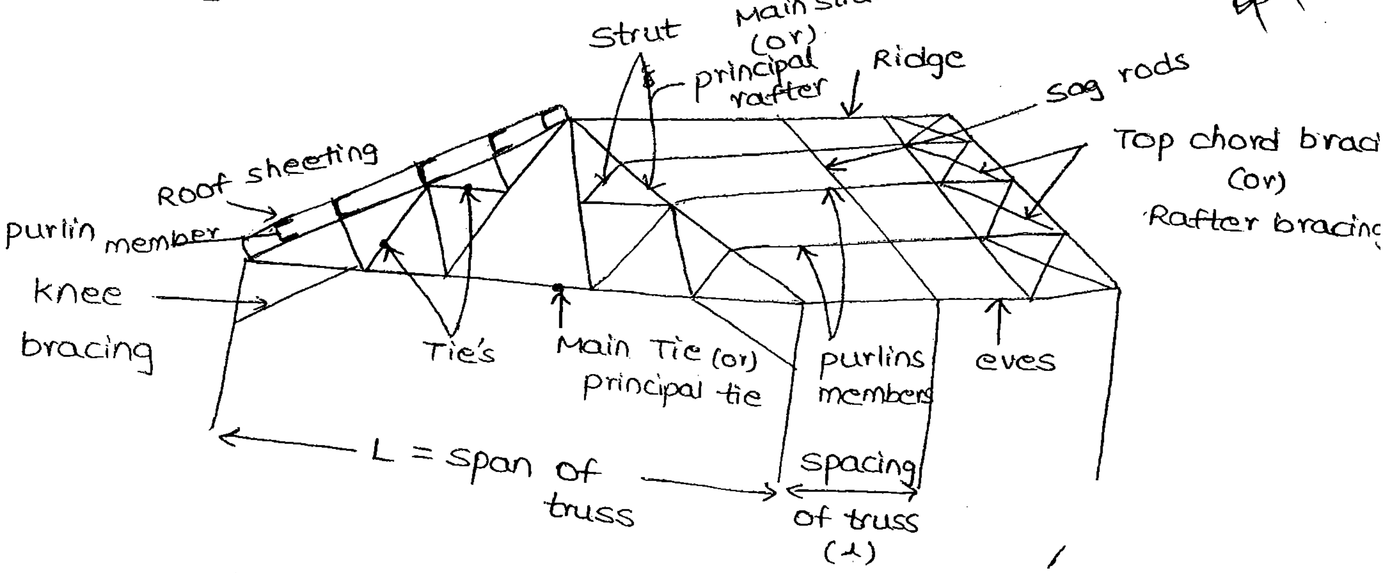
Elements of roof truss :-



purlin member always design as continuous beam.

Economical spacing of truss (l) :-

Spacing at which the total cost of the roof building is minimum.

Economical spacing of truss $(l) = \frac{L}{3}$ to $\frac{L}{5}$

L = span of the truss

Cost of the truss per unit area (t) $=$ 2x cost of purlin per unit area + cost of roof sheeting per unit area (r)

$$\boxed{t = 2P + r}$$

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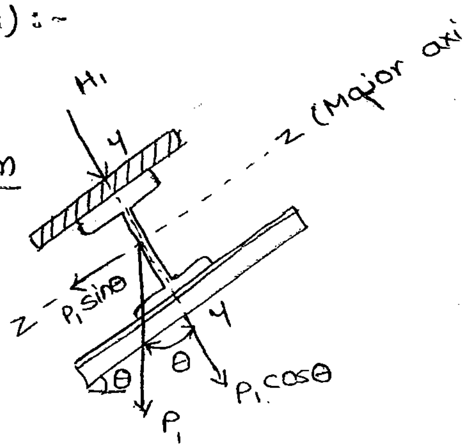
Design of purlin (working stress method) :-

IS 800 is recommended to design as purlin as a continuous beam and subjected to axial bending moment



P_1 = Gravity (or) verticle load due to self weight of purlin, self weight of sheeting, live load.

H_1 = Load due to wind pressure

P = Load along minor axis of purlin = $(H_1 + P_1 \cos\theta)$

H = Load along major axis of purlin = $P_1 \sin\theta$

θ = slope of roof.

l = Span of purlin (i.e., spacing between truss)

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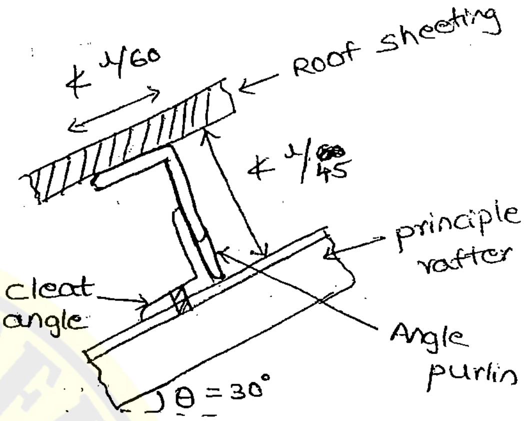
Design criteria:-

$$\text{Actual bending stress} = \left[\frac{M_{zz}}{I_{zz}} \cdot y + \frac{M_{yy}}{I_{yy}} \cdot z\right]$$

$$\sigma_{bc\,cal} \text{ (or) } \sigma_{bt\,cal} = \left[\frac{M_{zz}}{Z_{zz}} + \frac{M_{yy}}{Z_{yy}}\right] \leq \text{permissible bending Stresses}$$

$$\Delta_{cal} \leq \Delta_{max} = \frac{\text{Span}}{200} \text{ (maximum or limiting deflection)}$$

Design of angle purlin:-



IS 800 : 1984 recommends to use an angle member as purlin

1. When slope of roof $\theta \leq 30°$
   $\not> 30°$
2. Minimum live load to be used $\not< 750\ N/m^2$
3. Bending moment about minor axis is neglected.
4. Maximum bending moment, $M = \frac{wl^2}{10}$

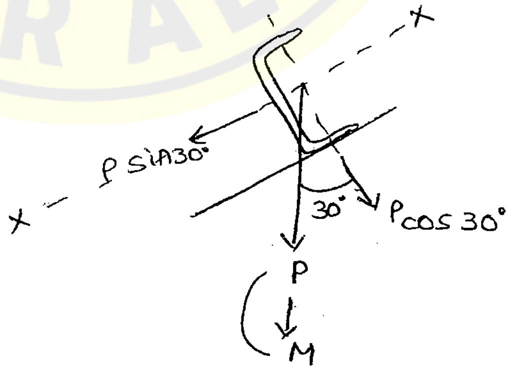
   l = span of purlin

   w = uniformly distributed load inclusive of wind load.

P.g No:- 74

2.

$$M_{xx} = P\cos 30°$$
$$= \frac{\sqrt{3}}{2} P$$
$$= \frac{\sqrt{3}}{2} M$$



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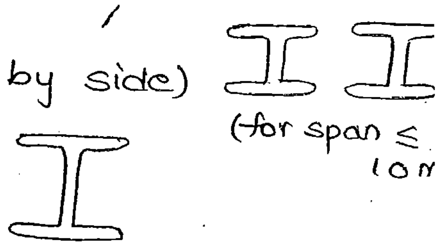
1-2-2015

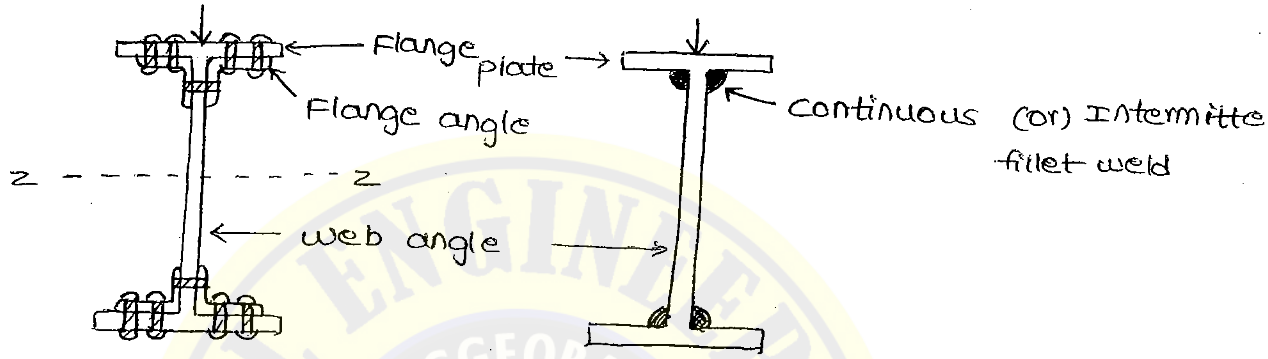
UNIT - 9

PLATE GIRDERS

Alternatives for large span beam with heavy transver loads.

1. Two rolled I - section (placed side by side)
2. Plate girders (for span 20m - 100 m)
3. Truss girders (for span > 100 m)





Riverted or Bolted plate girders

welded plate girder

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Note:-

In terms of simplicity and efficiency welded plate girders are far superior than bolted or riverted plate girder.

To support the same transverse load welded plate are demanding 5 - 15% of lesser area of steel than Bolted or riverted plate girder.

Self weight of Riverted (or) Bolted plate girder (w)

$$w = \frac{W}{300} \text{ kN/m}$$

self weight of welded plate girder

$$w = \frac{W}{400} \text{ kN/m}$$

W = Total superimposed load in kN

→ The riverts or bolts used between flange angle and flange plate to be designed for horizontal shear due to transverse load. Here riverts or bolts subjected to single shear only.

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→ The riverted connection between flange angle and web plate to be designed for horizontal shear due to transverse load and verticle live load. Here riverts are subjected to double shear.

Web equivalent:-

It is a portion of web between flange angles is behaves as part of flange area is called web equivalent.

Area of web plate, $A_w = d_w \cdot t_w$

d_w = Depth of web plate

t_w = Thickness of web plate.

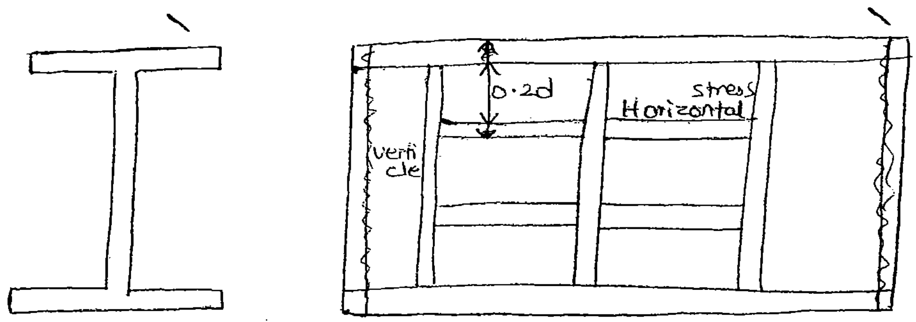
Area of web equivalent (A_{we})

Gross web equivalent, $A_{we} = \frac{A_w}{6}$ (used in compressive side)

Net web equivalent, $A_{we} = \frac{A_w}{8}$

Local buckling failures of web plate in a plate girder:-

1. Shear buckling failure (or) Diagonal compression ~~(or)~~ buckling failure of web.
2. Horizontal (or) Longitudinal (or) bending buckling failure of web
3. Verticle (or) Bearing buckling failure of web.



Elements of plate girder:- Photo Copy By Jain's 09700291147

1. Web plate
2. Flange plates only for welded plate girder. Flange angles and Flange plates for Bolted (or) Riverted plate girder.

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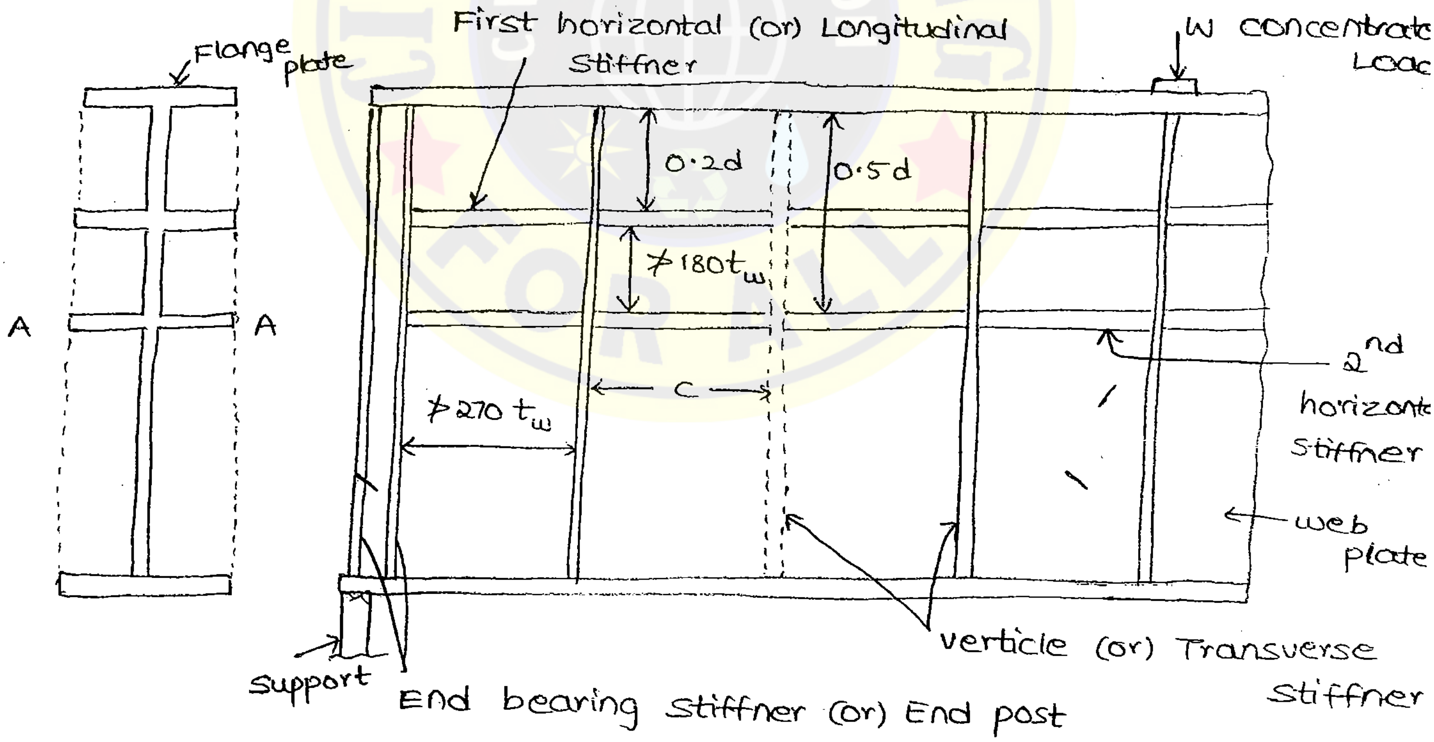
Stiffners :-

- Stiffners
  - Intermediate stiffner
    - verticle (or) transuerse (or) stability stiffner
    - Longitudinal (or) Horizontal stiffner
  - Bearing stiffner
    - Load bearing stiffner
    - End bearing stiffner

Splices :-

- Splices
  - Web splices
  - Flange splices



d_w = Depth of web plate

t_w = thickness of web plate

c = spacing between verticle (or) Transverse stiffner

d = depth of plate girder

d_2 = 2 x distance from compression flange plate (or) Flange angle to neutral axis.

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Web plate :-

Minimum thickness of web plate (t_w):-

1. $t_w = 6$ mm (when web plate is exposed to weather but accessible for painting)

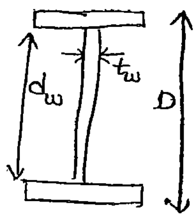
2. $t_w = 8$ mm (when web plate exposed to weather but inaccessible for painting)

Economical depth of plate girder (D):-

It is concept based on minimum area of steel (or) minimum weight of steel required for plate girder.

For riverted plate girder, $D = 1.1\sqrt{\frac{M}{\sigma_{bt} \cdot t_w}}$

For welded plate girder, $D = 5\sqrt[3]{\frac{M}{\sigma_{bc}}}$



M = Maximum bending moment

σ_{bt} = permissible bending stresses

d_w =

Web stiffners (IS 800 : 1984):-

1. When $\frac{d_1}{t_w} \leq$ lesser of $\frac{816}{\sqrt{\tau_{va,cal}}}$ and 85 — No stiffner is required

2. When $85 < \frac{d_2}{t_w} \leq 200$ — verticle, transverse stiffner are to be provided.

3. when $200 < \frac{d_2}{t_w} \leq 250$ — Transverse stiffner + 1st horizontal stiffner at 0.2d from compression flange.

4. When $250 < \frac{d_2}{t_w} \leq 400$ — Transverse stiffner + 1st Horizontal stiffner + 2nd horizontal stiffner at Neutral Axis.

Verticle transverse stiffner:- Photo Copy By Jain's 09700291147

Verticle stiffners (or) transverse stiffners are to be provided to eliminate shear buckling failure of web plate.

Flat stiffner to be selected for welded plate girder or an angle member to be used as a stiffner as riverted (or) bolted connection.

Minimum moment of Inertia for transverse stiffner

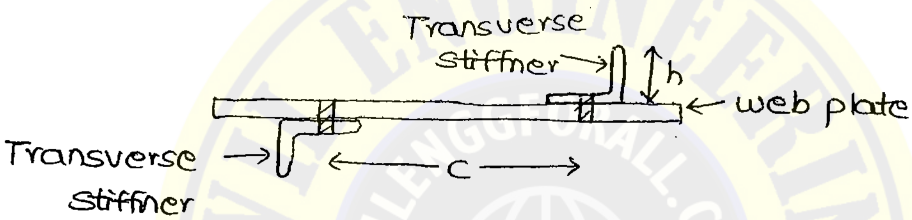
$$I_s \geq \frac{1.5 d_1^3 t_w^3}{c^2}$$

t_w = minimum required thickness of web

c = maximum permitted distance between verticle stiffner. ~~stress~~

Maximum spacing between transverse stiffner = $1.5 d_1$

Minimum spacing between transverse stiffner = $0.33 d_1$



h = Outstand of stiffner.

Horizontal stiffner (or) Longitudinal stiffner:-

1. First horizontal stiffner at $0.2d$ from compression flang

Minimum M.I $I_s \geq 4 c t_w^3$

2. Second horizontal stiffner at Neutral axis

Minimum M.I $I_s \geq d_2 t_w^3$

d_2 = 2x distance from compression flange plate (or) flange angle to N.A.

Note:-

Horizontal (or) Longitudinal stiffners are to be provide to eliminate longitudinal (or) bending (or) Horizontal buckling failure of web plate.

The connection between verticle stiffner to the web plate (or) Horizontal stiffner to the web plate to be design for a S.F not less than $\frac{125\ t_w^2}{h}$ (kN/m)

$\therefore$ h = Outstand of stiffner

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P.g No:- 65

2. Given $d_1 = 1800$ mm $\tau_{va,cal} = 81$ Mpa.

$$\frac{d_1}{t_w} \leq \frac{816}{\sqrt{\tau_{va\,cal}}}$$

$$\frac{1800}{t_w} \leq \frac{816}{\sqrt{81}}$$

$$t_w \geq \frac{1800 \times 9}{816}$$

$$t_w \geq 20 \text{ mm}$$

3. Given $d = 1000$ mm $t_w = 10$ mm

$$\frac{d_2}{t_w} = \frac{1000}{10} = 100$$

$\frac{d_1}{t_w} \leq 85$ — No stiffner required

$85 < \frac{d_2}{t_w} \leq 200$ — Intermediate verticle stiffner required

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