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CE B.Tech

STEEL

Design of Steel Structure

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CONTACT US: **ZONE TECH**

H.O.: M-28, Madhuban Colony, Near Tonk Bridge, Tonk Phatak, Jaipur

B.O.: F-26/A, Behind Pinkcity Honda Showroom, Gatta Stand, Tonk Road, Jaipur

+91-9828747676

+91-9462447676

www.zonetech.in

www.onlinezonetech.in

info@zonetech.in

CHAPTER - 1
(GENERAL CONSIDERATION)

Steel as a structural material:-

1. Steel members have high strength to weight ratio therefore a steel member of small sections which has little self weight as it is able to resist heavy load.
2. This property is most important for the construction of long span bridges, tall buildings and for buildings on soil with relatively low bearing capacity.
3. Steel is a ductile material does not fail suddenly but gives proper warning before failure.
4. Steel structures can be constructed very fast.
5. Steel structures can be easily dismentaled and sold as scrap material.

Disadvantages:-

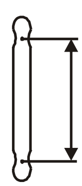
1. Steel structures when placed in exposed condition are subjected to corrosion therefore they required frequent maintanance.
2. Fire resistance technique is also required for steel structures.

Structural steel (IS 2062):-

Grade	Ultimate tensile strength	Yield strength
Fe 410	410 N/mm^2	250 N/mm^2

Minimum % elongation	Gauge length for tensile test
23%	$= 5.65\sqrt{A_0}$

A_0 = Initial cross-section area



l_g = Gauge length

Modulus of elasticity = 2×10^5 N/mm^2

Shear modulus = 0.769×10^5 N/mm^2

Poission's ratio

Elastic Limit = 0.3

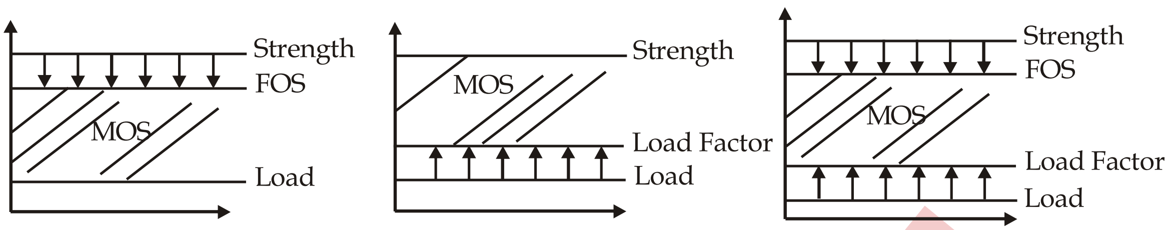
Plastic Limit = 0.5

Coefficient of thermal expansion = $12 \times 10^{-6}/°C$

Unit mass (G) = 7.85×10^3 kg/m^3

Design philosophies:-

WSM **USM** **LSM**



→ The uncertainities affecting the safety of a structure

1. The uncertainity about loading
2. Uncertainity about material strength
3. Uncertainity about structural dimension and behaviour

These uncertainity together makes it impossible to designer to guarantee that a structure will be absolutely safe. There is always a small risk of failure.

WSM (IS 800 - 1984) - or elastic method.

This method is based upon elastic theory attainment of yield stress forms the design criteria for the member.

The working stress should be less than permissible stress.

$$\text{Permissible stress} = \frac{\text{Yield stress}}{\text{FOS}}$$

Working stress (during design life) $\leq$ Permissible stress

Permissible stress in structural member:-

Type of stress	Notations	Permissible stress	FOS
(i) Axial Tension	σ_{at}	$0.6\ f_y$ $= 0.6 \times 250$ $= 150\ N/mm^2$	$0.6\ f_y = \frac{f_y}{FOS}$ $FOS = 1.67$
(ii) Axial Compression	σ_{ac}	$0.6\ f_y$	1.67
(iii) Bending Tension	σ_{bt}	$0.66\ f_y$	1.5
(iv) Bending Compression	σ_{bc}	$0.66\ f_y$	1.5
(v) Average shear	τ_{va}	$0.4\ f_y$	2.5
(vi) Maximum shear	τ_{vm}	$0.45\ f_y$	2.22
(vii) Bearing stress	σ_p	$0.75\ f_y$	1.33
(viii) Stress in slab base	σ_{bs}	$185\ N/mm^2$	—

One of the major drawback of designing steel structure by WSM approach the reserved strength beyond elastic limit is not utilized.

Limit state method:- The limit state design method was develop to take account of all conditions that can make the structure unfit for use. This theory based upon probablistic approach.

Limit state of collapse or strength :- Limit state of strength considered that structure should be safe under the action of most unfavorable loads combinations.

Limit state serviceability:- Structure should be servicible at working load conditions. Ex:- servicibility criteria deflection, vibrations, corrosion, fire resistance.

Characteristic strength (Su):- Characteristic strength of a material is defined as the value of resistance below which not more than 5% test results are expected to fail.

$$\text{Design strength} = \frac{f_u \text{ (characteristic strength)}}{\text{FOS}}$$

Characteristic load (Qu):- Characteristic load is a load which has 95% probability not being exceeded during the design life of the structure.

$$\text{Design load} = \text{Load Factor} \times Q_u$$

$$\text{Failure Probability} = 0.05 \times 0.95 + 0.05 \times 0.95 + 0.05 \times 0.05$$

$$= 0.0975 = 9.75\% \simeq 10\%$$

	Partial safety factors for materials	**FOS**
1.	Resistance governed by yielding	1.10
2.	Resistance governed by buckling	1.10
3.	Resistance governed by ultimate strength	1.25

	Partial safety Factors for connections	**Shop Fabric**	**Field Fabric**
1.	Bolted connections	1.25	1.25
2.	Welded connections	1.25	1.50

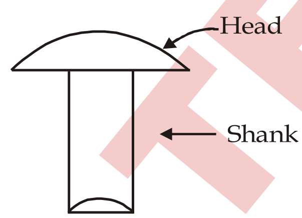
CHAPTER - 2
(CONNECTION DESIGN - RIVETED CONNECTION)

Connection Design:-

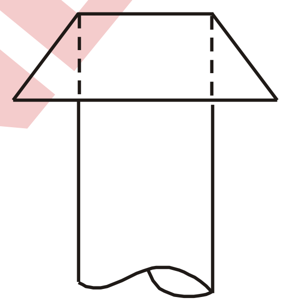
1. Riveted Connections (WSM)
2. Bolted Connections (LSM)
3. Welded Connections (WSM and LSM)

Riveted Connections:-

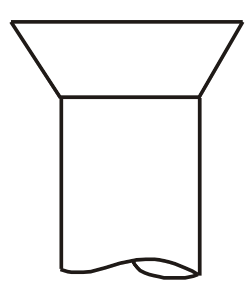
(i) **Snap Head Rivet:-**

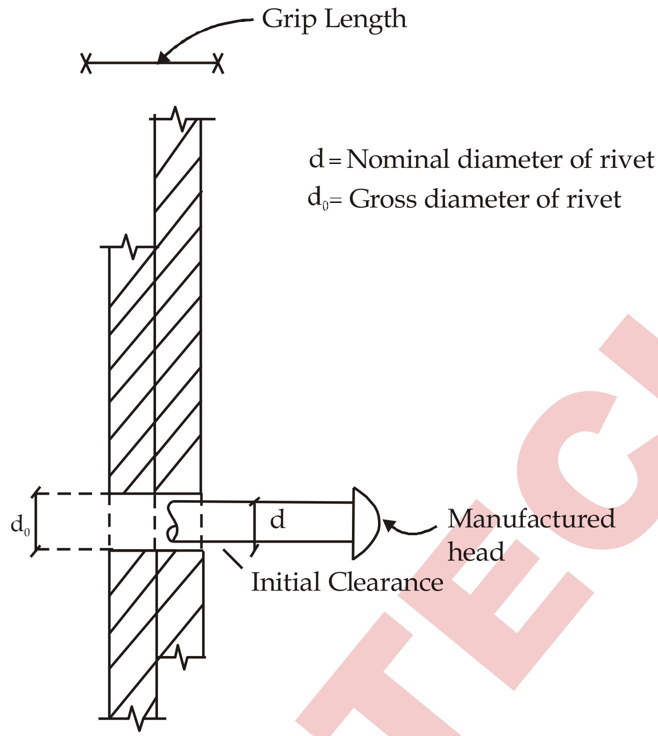


(ii) **Pan Headed Rivet:-**

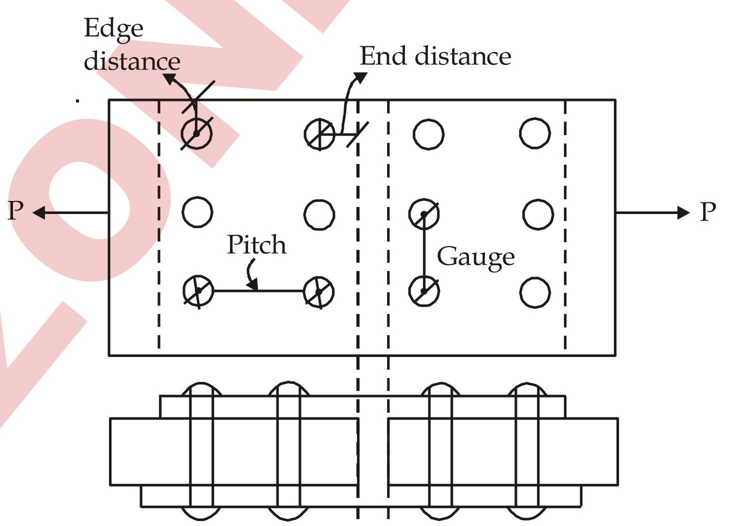


(iii) **Counter Shank Rivet:-**





Some important specifications:-



(i) **Diameter of rivet hole = d_0:-**

$$d_0 = d + 1.5 \text{ mm } (d \le 25 \text{ mm})$$

$$d_0 = d + 2 \text{ mm } (d > 25 \text{ mm})$$

2. **Minimum Pitch (p):-** Minimum pitch is ensure for the following reasons:-

(a) To prevent bearing failure of members between two rivets.

(b) To permit efficient intallation of rivets to ensure sufficient space for riveting.

$$\text{Minimum pitch} = 2.5 \times \text{nominal of rivet.}$$

$$p_{minimum} = 2.5d$$

3. Minimum end or edge distance:-

$$= 1.5d_o \text{ (For machine cut elements)}$$

$$= 1.7d_o \text{ (For hand cut elements)}$$

4. Maximum edge or end distance:-

$$= 12\, t\varepsilon$$

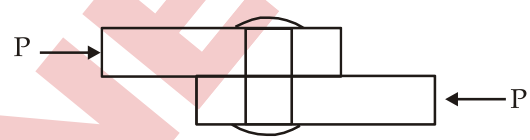
$$\varepsilon = \sqrt{\frac{250}{f_y}}$$

ε = Yield ratio

t = Thickness of thinner member of connection

f_y = yield strength = 250 N/mm²

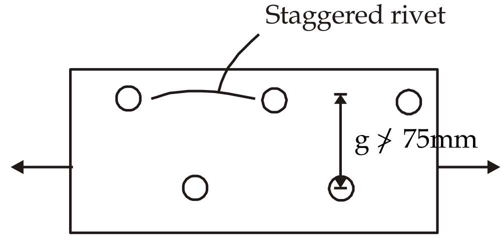
5. Maximum pitch (p):-



$$P_{maximum} = \begin{cases} 12t \text{ or } 200 \text{ mm whichever is minimum, Compression} \\ 16t \text{ or } 200 \text{ mm whichever is minimum, Tension} \end{cases}$$

t = Thickness of thinner member connected

In case of staggered rivet these values should be multiply by 1.5 times

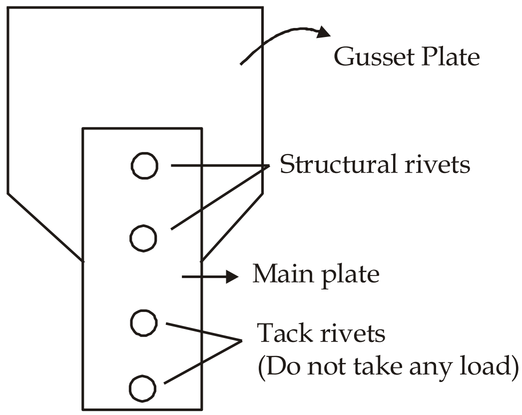


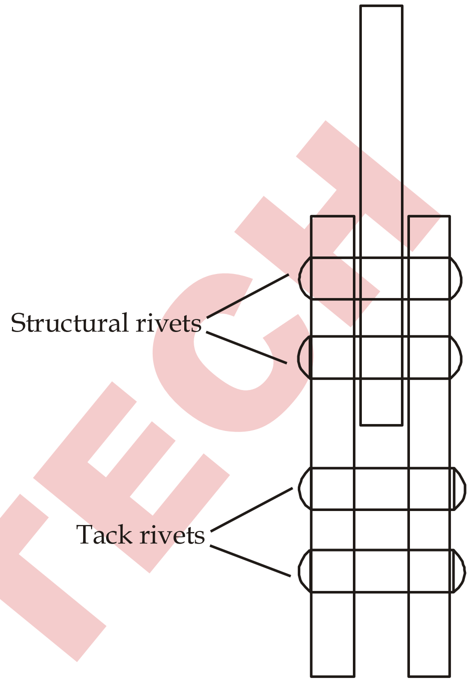
Compression (p) ≯ [18t or 300 mm] min

Tension (p) ≯ [24t or 300 mm] min

Tack riveted:-

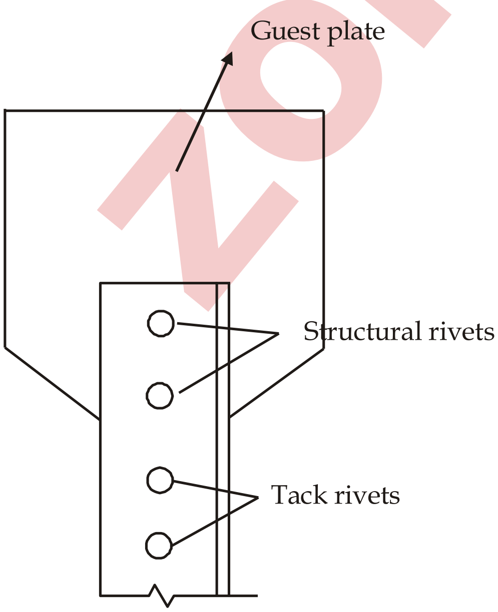
1. When plate elements are tack riveted:-

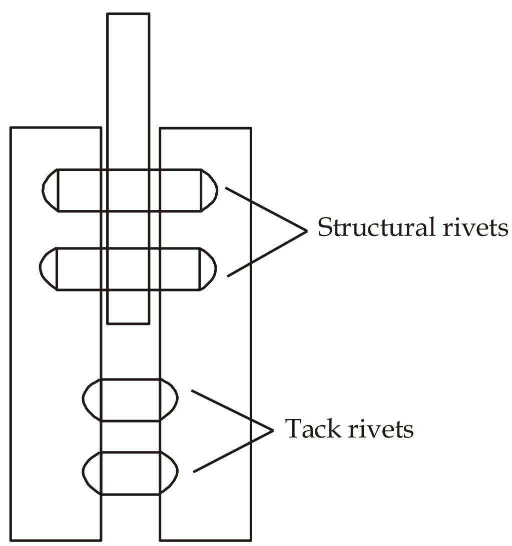




Maximum p $\ngtr$ [32t or 300 mm] min

2. When angle sections or channel sections are tack riveted:-





If Compression (P) $\not>$ 600 mm

Tension (P) $\not>$ 1000 mm

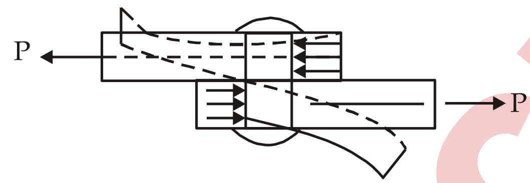
- To find nominal dia of rivet unwin's formula is used.

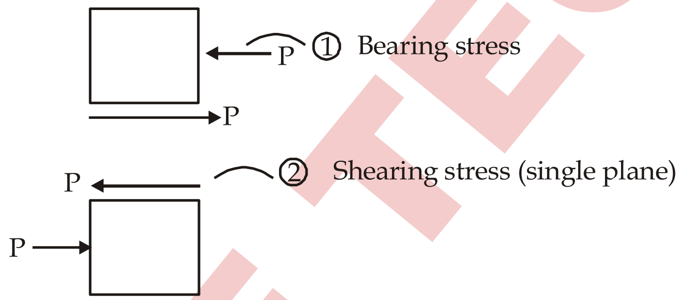
$$d = 6.05\sqrt{t} = mm$$

t = thickness of thinner member connected

Types of joints:-

1. **Lap Joint:-**

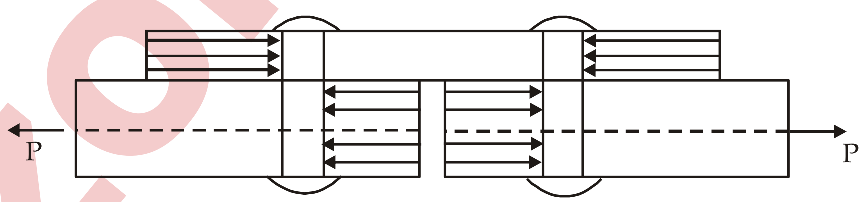


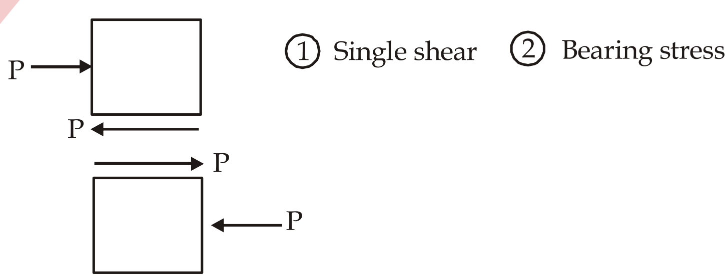


Free Body Diagram of rivet

- Less efficient joint because line of action of forces are not same due to this bending stresses are generated in rivet.

2. **Single Cover Butt Joint:-**

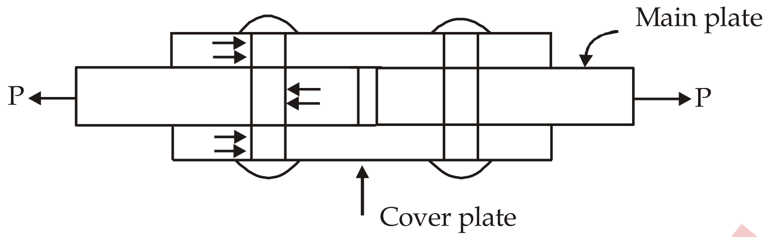


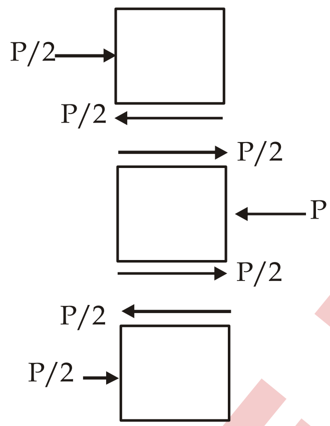


Free Body Diagram of rivet

Line of action of load is same but connection is not symmetric about load line so slightly more efficient than lap joint.

2. **Double Cover Butt Joint:-**



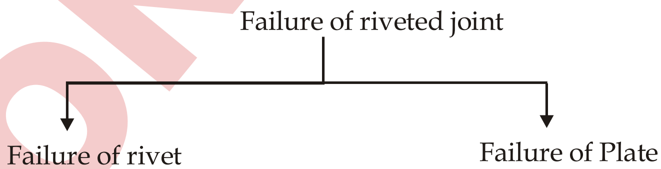


① Double shear

② Bearing stress

Free Body Diagram of rivet

Most efficient joint because load line is same and connection is also symmetric about load line.

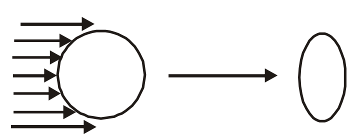


Failure of rivet:-

(1) Shear failure of rivet:-

Rivet get cut into two pieces

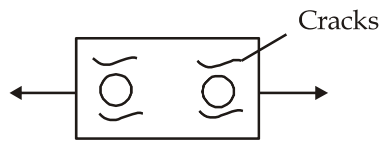
(2) Bending failure of rivet



Circular c/s changed to eliptical

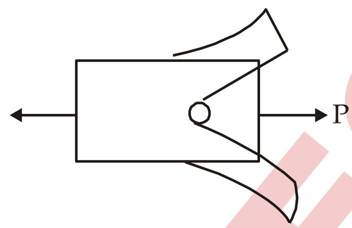
Failure of Plate:-

(1) Shear failure of plate



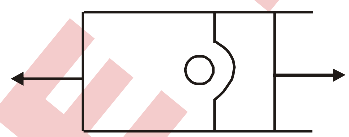
Cracks generated in the direction of force.

(2) Splitting failure of plate



Due to diagonal tension

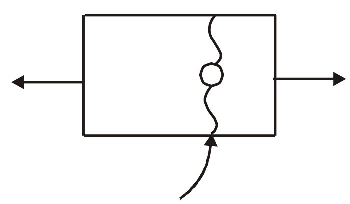
(3) Bearing failure of plate



When rivet material is strong in comparision to plate material.

Rivet will push the plate forward.

(4) Tearing failure or tension failure of plate



Cracks are generated in the perpendicular direction of load.

Note:-

1. By proper end or edge distance we can prevent shear failure, splitting failure and bearing failure of plate.

2. For a efficient design we have to ensure that shearing and bearing strength of rivet is more than tearing strength of plate (by it we can fully utilize the strength of plate (structural member)).

Strength of riveted joint:-

$$\text{Minimum of}\begin{cases}\text{Shearing strength of all rivets } (P_s)\\ \text{Bearing strength of all rivets } (P_b)\\ \text{Tearing strength of all rivets } (P_T)\end{cases}$$

Permissible stresses in rivet:-

		Permissible shear stress (τ_{vf})	Permissible bearing stress (σ_{pf})
Shop rivet	Power driven	100 N/mm²	300 N/mm²
	Hand driven	80 N/mm²	250 N/mm²

Note:-

In case of field riveting above values are reduced by 10%.

1. **Strength of lap joint:-**

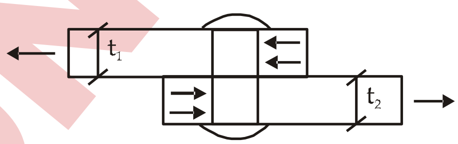
(a) Shearing strength of single rivet:-

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

d_o = dia of hole or gross dia of hole

(b) Bearing strength of single rivet:-

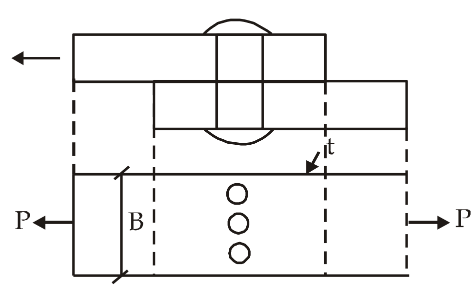
$$P_b = d_o . t \times \sigma_{pf}$$



t = Thickness of thinner plate

$$\min\begin{cases}t_1\\ t_2\end{cases}$$

(c) Tearing strength of plate:-



P_T = Tearing strength of plate

$$P_T = (B - nd_o) \times t \times \sigma_{at}$$

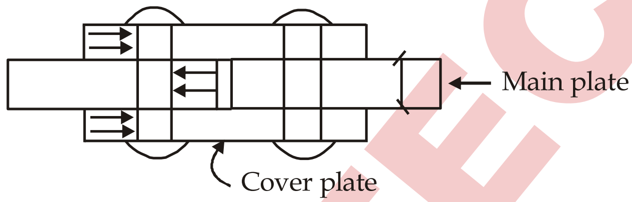
$$\text{Rivet Value } (R_V) = \min\begin{bmatrix} P_s \\ P_b \end{bmatrix}$$

$$\text{Number of rivet} = \frac{F \text{ (Load to be resisted)}}{R_V}$$

2. Double Cover Butt Joint:-

(a) Shearing strength of single rivet:-

$$P_S = 2 \times \frac{\pi}{4} . d_o^2 . \tau_{vf}$$

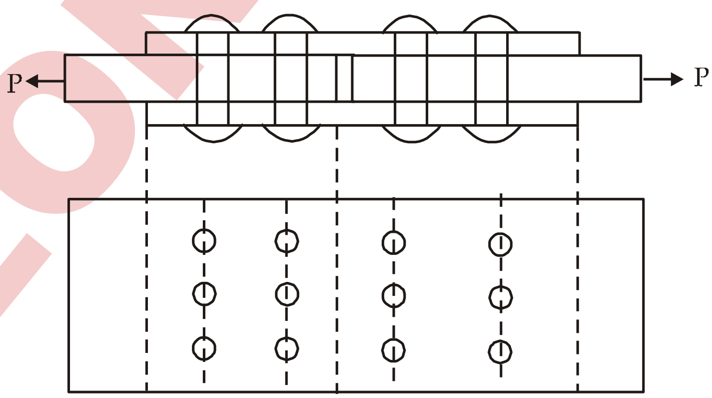


(b) Bearing strength of single rivet:-

$$P_b = d_o \times t \times \sigma_{pf}$$

$$t = \min\begin{bmatrix} \text{summation of thickness of cover plates} \\ \text{Thickness of main plate} \end{bmatrix}$$

Strength of all rivets



$$\text{Strength of all rivets} = n \times R_V$$

$$n = \text{Number of rivets} = 6 \text{ (not 12)}$$

(c) Tearing strength of plate:-

$$P_{T1-1} = (B - nd_o) . t . \sigma_{at}$$

$$P_{T2-2} = (B - 3d_o) . t . \sigma_{at} + 3R_V$$

Strength of Joint:-

$$\min\left[\begin{bmatrix} nR_V \text{(strength of all rivets)} \\ P_T \text{(tensile strength of plate)} \end{bmatrix}\right]$$

where $n =$ number of all rivets in each side of joint

Efficiency of joint:-

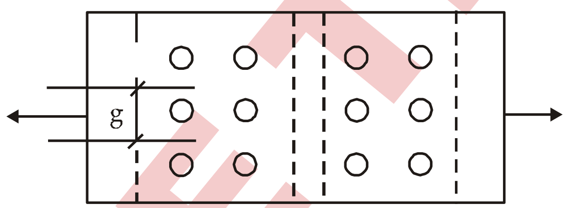
$$\eta = \frac{\text{Strength of joint}\left[\min\left(nR_V, P_T\right)\right]}{\text{Tensile strength of solid plate}\left[B t \sigma_{at}\right]}$$

For efficient joint tensile strength or tearing strength of plate should be less than or equal to strength of all rivet.

$$\eta = \frac{(B - nd_o) \times t \times \sigma_{at}}{B.t.\sigma_{at}}$$

$$\eta = \frac{(B - nd_o)}{B}$$

Note:-



1. Strength of rivet in per pitch length

$$= nR_V$$

n = Number of rivet in a pitch/gauge length

2. Tearing strength of plate per pitch length

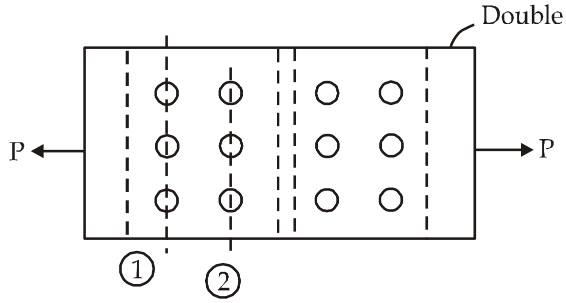
$$= (g - nd_o) \times t \times \sigma_{at}$$

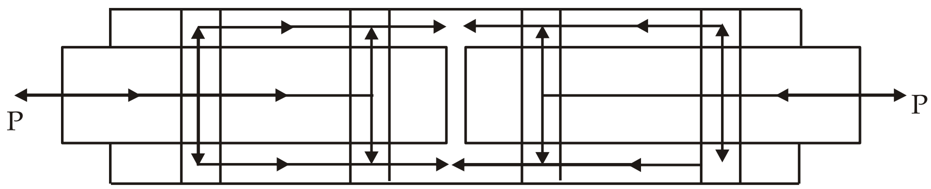
For a efficient joint tearing strength of plate should be less than equal to strength of all rivets.

$$(g - nd_o) \times t \times \sigma_{at} \le nR_V$$

Arrangement of rivets:-

1. **Chain riveting:-**





Load path diagram

Tearing strength of main plate at section 1 - 1

$$P_{T1\text{-}1} = (B - 3d_o).\sigma_{at}.t$$

$$P_{T2\text{-}2} = (B - 3d_o).t.\sigma_{at} + 3R_V$$

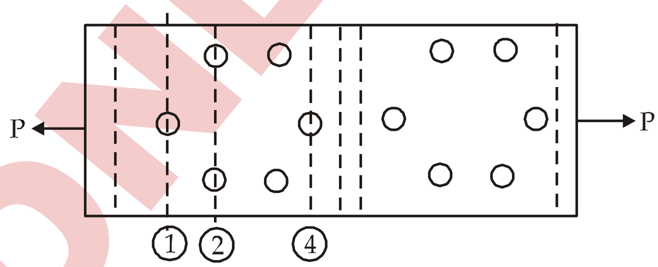
Cover plate strength at section 2 - 2

$$(B_{cover} - 3d_o) \times t.\sigma_{at})$$

t = Summation of thick of cover plates.

- First section will critical for main plate
- Last section will critical for cover plate

2. **Diamond riveting:-**



1. Tearing strength of main plate at section 1 - 1

$$P_{T11} = (B - d_o) \times t \times \sigma_{at}$$

$$P_{T22} = (B - 2d_o) \times t \times \sigma_{at} + R_V$$

Strength of cover plate at section 4 - 4

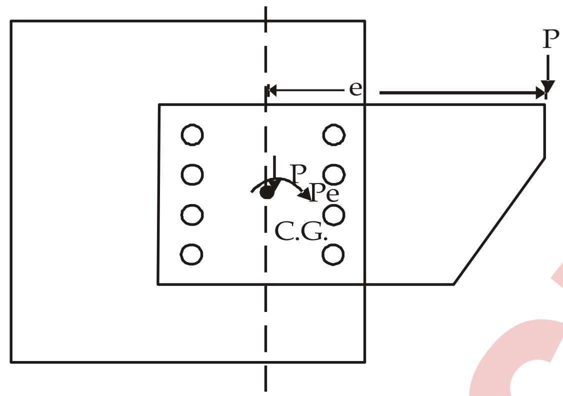
$$= (B - d_o) \times t \times \sigma_{at}$$

t = Summation of thickness of cover plate.

Note:- With the above arrangement both main plate and cover plate carry maximum tensile load.

Eccentric riveted connection:-

Case I : In plane eccentricity:-



$e = \text{eccentricity}$

Due to in plane eccentricity a direct force P and twisting moment of Pe wi ll generated at CG of rivet group.

Analysis:-

1. $F_1 =$ shear due to direct force

P at CG

$$Fi = \frac{P}{\sum A_i} \times Ai = F_1 = \frac{P}{\sum A_i} \times A_i$$

If area of all rivets is same

$$F_1 = \frac{P}{nA} \cdot A$$

$$F_1 = \frac{P}{n}$$

$n =$ Number of rivets

Direction ↓ (vertically downward)

2. Force due to twisting moment:-

$$T = Pe$$

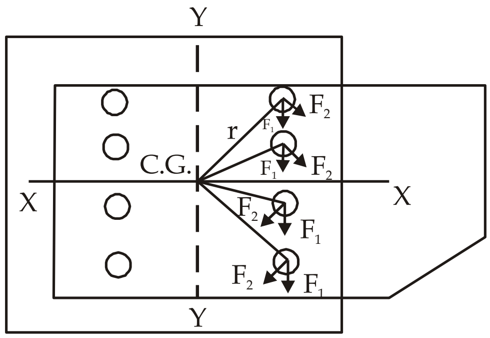
$$\frac{T}{J} = \frac{\tau}{r} = \frac{G\theta}{L}$$

$T =$ Twisting moment

$J =$ Polar moment of inertia

$\tau =$ Shear stress

$r =$ Radial distance of rivet from C.G. of rivet group



$$\frac{T}{J} = \frac{\tau}{r} = \frac{G\theta}{L}$$

$$\tau = \frac{T}{J} \times r$$

Force $= \tau \times A$

$$= \frac{T}{J} \times r \times A$$

$$F_2 = \frac{Pe}{J} \times r \times A \qquad [T = P.e]$$

$J =$ Polar moment of inertia

$$J = \Sigma A_i r_i^2 \Rightarrow A_1 r_1^2 + A_2 r_2^2 + A_3 r_3^2 ....A_n r_n^2$$

$$F_{2i} = \frac{Pe}{\Sigma A_i r_i^2} . r_i \times A_i$$

$r_i =$ radial distance of rivet from the CG of rivet group.

If area of C/S is same

$$Ar_1^2 + Ar_2^2 + ...Ar_n^2$$

$$= A \Sigma r_i^2$$

$$F_{2i} = \frac{Pe}{A \Sigma r_i^2} \times r_i \times A$$

$$F_{2i} = \frac{Pe}{\Sigma r_i^2} \times r_i$$

$$\text{Resultant} = \sqrt{F_1^2 + F_2^2 + 2F_1F_2 \cos\theta}$$

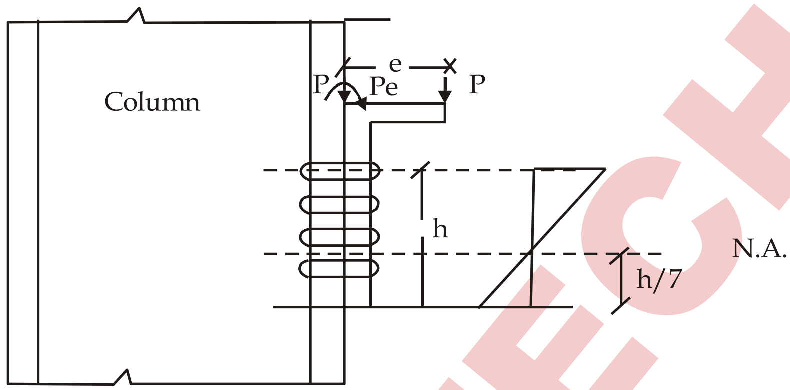
$\theta =$ Angle between F_1 and F_2 measured in anticlockwise from F_1 to F_2

- For Rivet to be safe

$$F_{resultant} \ngtr R_V$$

Note:- Generally the rivet at maximum distance from the CG of the rivet group and nearest to the load line having maximum force.

Case II : Out of plane eccentricity:-



In this arrangement load is out of plane hence B.M will generate in this connection.

Due to this BM rivets above NA will be resisted to tensile force along with direct shearing and bearing.

Rivet below NA will contribute partially in resisting compression because most of the compression effect will be resisted by plate.

Considering both the effect of tension and shear for rivet above NA these interaction equation should be check.

$$\frac{\sigma_{at,\,calculated}}{\sigma_{at}} + \frac{\tau_{va,\,calculated}}{\tau_{va}} \ngtr 1.4 \text{ (WSM)}$$

$$\left(\frac{P_{at,\,calculated}}{P_{at}}\right)^2 + \left(\frac{P_{s,\,calculated}}{P_s}\right) \le 1 \text{ (LSM)}$$

$\sigma_{at\,,\,calculated}$ = calculated axial tensile stress

σ_{at} = Permissible axial tensile stress

$\tau_{va\,,\,calculated}$ = calculated shear stress

τ_{va} = Permissible shear sterss

$P_{at,\,calculated}$ = calculated axial tensile force

P_{at} = Safe axial tensile force

$P_{s,\,calculated}$ = calculated shear force

P_s = Safe shear force

Chapter - 3
(Bolted Design (LSM))

- Unfinished bolt or ordinary bolt or Bearing type bolts.
- High strength bolt or friction type bolt

(1) **Unfinished or bearing type bolt:-** These types of bolt are also called common, rough and black bolt. It is used for light structure subjected to static load. It is not suitable for connection subjected to impact or vibrations. Load transfer by shearing and bearing.

(2) **High strength bolt (friction type):-** These bolt may be tighten until they have very high tensile stresses, two or more time ordinary bolts so that the connected parts are clamped tightly together between the bolts and nut.

In this type of bolts load transfer by friction between connected parts.

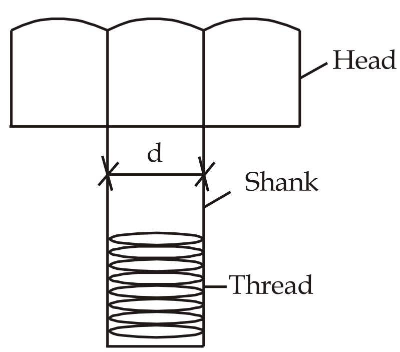
Design of ordinary bolts:-

1. Grade of bolts:-

Grade	Ultimate Stress	Yield Stress
4.6	$f_{ub} = 400\ N/mm^2$	$f_{yb} = 400 \times 0.6 = 240\ N/mm^2$
8.8	$f_{ub} = 800\ N/mm^2$	$f_{yb} = 800 \times 0.6 = 480\ N/mm^2$

2. Diameter of bolt hole:-

Diameter of bolt	Diameter of hole
12 - 14 mm	+ 1 mm
16 - 24 mm	+ 2 mm
> 24 mm	+ 3 mm



$$\text{Area of shank} = \frac{\pi}{4} d^2$$

$$\text{Area of threaded portion} = 0.78 \times \frac{\pi}{4} d^2$$

3. Shearing strength of bolt:-

For single shear:-

$$V_{dsb} = \text{Resisting area} \times \text{Permissible shear stress}$$

$$V_{dsb} = 0.78\ \frac{\pi}{4} d^2 . \frac{f_{ub}}{\sqrt{3} . \gamma_{mb}}$$

$$\gamma_{mb} = 1.25$$

For double shear:-

$$V_{dsb} = 2 \times 0.78 . \frac{\pi}{4} d^2 . \frac{f_{ub}}{\sqrt{3} \times 1.25}$$

4. Bearing strength of bolt:- (bearing failure at 6m)

$$V_{dpb} = 2.5\ K_b \times d \times t \times \frac{f'_{ub}}{1.25}$$

f'_{ub} = minimum ultimate tensile stress of plate and bolt material

d = diameter of bolt

t = thickness of thinner member connected

$$\mathbf{K_b}\text{:- minimum of} \begin{cases} \dfrac{e}{3d_0} \\ \dfrac{p}{3d_0} - 0.25 \\ \dfrac{f_{ub}}{f_u} \\ 1 \end{cases}$$

e = end distance

d_o = diameter of hole

p = Pitch

f_{ub} = Ultimate stress of bolt

f_u = Ultimate stress of plate

Bolt value (V_{db}) = strength of a bolt

$$\text{Min} \begin{cases} V_{dsb} \\ V_{dpb} \end{cases}$$

Strength of plate:-

1. Solid plate strength (Gross area strength):-

$$T_{dg} = A_g . \frac{f_y}{1.10}\ \text{[serviceability criteria]}$$

2. Net area strength (Limit state of collapse):-

$$T_{dn} = A_{net} \cdot \frac{f_u}{1.25} \times 0.9$$

Tensile strength of plate is (T_{dp})

$$\text{Min}\begin{cases} T_{dg} \\ T_{dn} \end{cases}$$

Strength of Joint:-

$$\text{Min}\ [\ T_{dn}, T_{dg}, V_{dsb}, V_{dpb}]$$

Efficiency of Joint:- Strength of Joint/Strength of solid plate

$$\frac{\text{Min}\left[T_{dn}, T_{dg}, V_{dsb}, V_{dpb}\right]}{A_g \times \frac{f_y}{1.1}}$$

3. Gross area strength of plate:-

$$T_d = A_g \times \frac{f_y}{1.10}$$

Friction bolt or High strength or HSFG - High strength friction grip bolts

- These bolts provide extremely efficient connection in dynamic load condition.
- These bolt should be tighten to their proof load.
- The HSFG connections are designed such that under service load the force does not exceed the frictional resistance.
- When the external force exceed the frictional resistance, the bolt come into contact with the plate and start bearing.

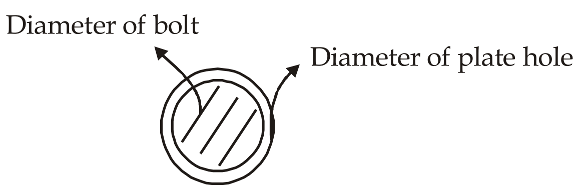
(1) Slip critical connection (slip between plates is not allowed)

- Bolts are designed on the basis of slip resistance.

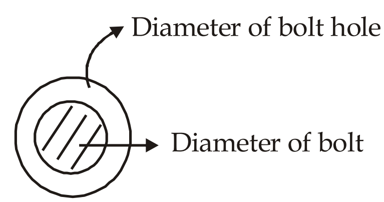
Design slip resistance of bolt $(v_{dsf}) = \frac{\mu_F \times K_h \times \eta_e \times Fo}{FOS}$

μ_F = Slip factor depends upon surface treatment (if not given then asssume $= 0.5$)

k_h = depends upon type of bolt hole

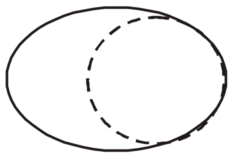


$(K_h = 1)$



$(K_h = 0.85)$

(a) Clearance hole or standard hole



$(K_h = 0.85)$

(b) Over sized hole

$(K_h = 0.7)$

(c) Short sloted hole

(d) Long sloted hole

f_o = tensile strength at proof stress

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

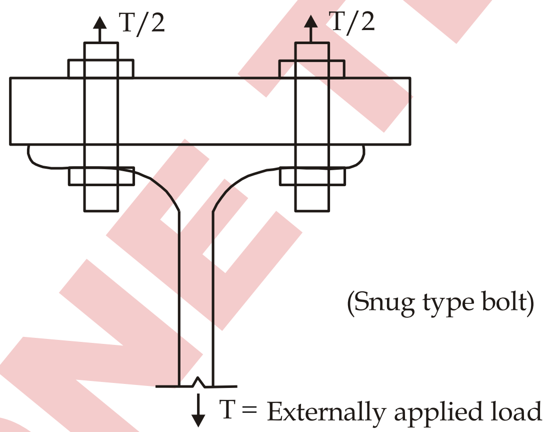
f_o = proof stress = $0.7 \times f_{ub}$, f_{ub} = ultimate tensile stress of bolt

FOS:- If slip resistance is designed for service load FOS = 1.10

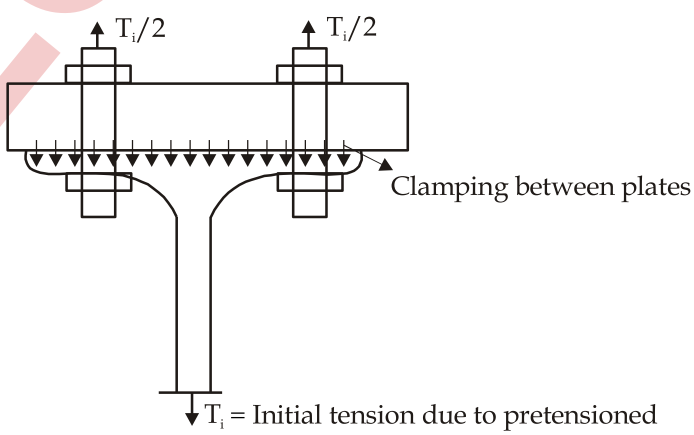
If slip resistance is designed for ultimate load FOS = 1.25

(2) Slip permitted connection:- Design same as bearing bolt.

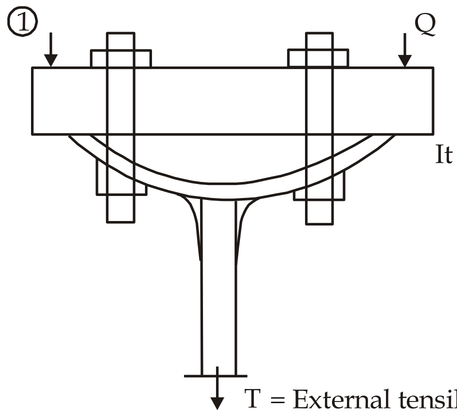
Prying Force:-



(i) Bearing type bolt



(ii) Friction type bolt



Q = Prying force

Force in bolts = Q + T / 2

1. **Bearing type connection:-** In this connection before any external tension applied, the force in the bolt is almost zero since the bolts are snug type.

As the external tension is increased it is equilibrated by the increase in bolt tension depending on the relative flexibility of the plate and the bolt. Sometimes the opening of the joint may be accomplained by prying action. This prying action increases tensile force in bolts.

HSFG bolt:- It is seen that even before any external load is applied, the force in the bolt is equal to proof load due to this there is clamping force between the plates in contact.

- When the external load is applied part of the load (approx 10%) is equilibrated by the increase in bolt force. The balance of the force is equilibrated by the reduction in contact between the plates.

  This process is continuous until the contact between the plates is maintained.

- After the external force exceeds this level the behaviour of the bolt under tension is essentially the same as that in a bearing type bolt.
- It is assumed that the load on the joint is distributed equally among on the bolt.
- If length of joint increases 15 times of diameter of bolt (15d) then the force shared by the end bolts may be so high that it may leads to progressive joint failure which is called unbuttoning effect.
- When length of joint exceed 15d then shear capacity of bolt is reduced by reduction factor.

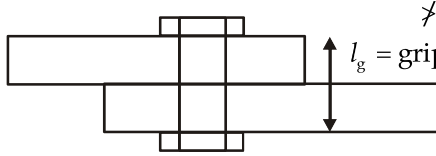
$$\beta_{lj} = 1.075 - \frac{l_j}{200d}$$

l_j = length of joint

d = diameter of bolt.

$$0.75 \le \beta_{ij} \le 1$$

2. **Grip Length:-**



- If grip length is increased then the efficiency of joint decreased because of additional bending stresses develop in the bolt.

  If grip length exceed 5d then a reduction factor is taken while calculating shear strength of bolt.

$$\beta_{lg} = \frac{8d}{3d + l_g} \qquad [l_g > 5d]$$

l_g = Grip length

[In any case $l_g \not> 8d$]

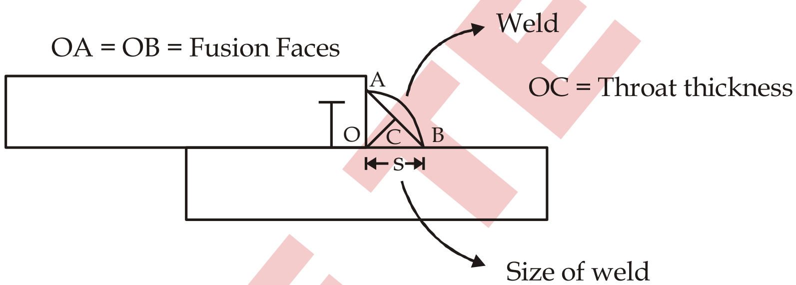
Chapter - 4
(Welded Connection)

Welding is a process of making joint between two similar metals by fusing them together at very high temperature (3600°C) or greater than melting point to produce similar section after solidification.

Why welding? or Welding as a connection process:-

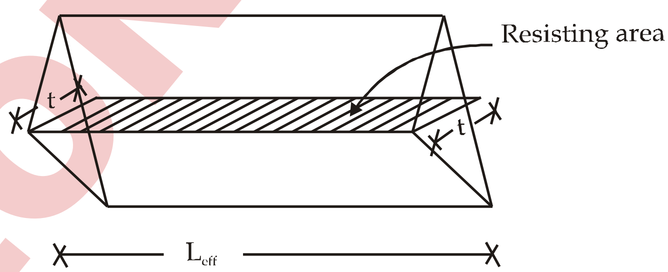
1. No deductions are there for holes thus the gross section is effective in carrying loads.
2. Welding is the only process that produces a one piece connection.
3. Welded joints are better for impact loads and vibrations.

1. Fillet Weld:-



Throat thickness, $t = S/\sqrt{2} = 0.707\,S$

Resisting area = Throat thickness × Effective length of weld

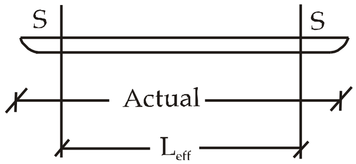


2.

Angle between Fusion faces	**Throat thickness (S= size of weld)**
60° - 90°	0.7 S
91° - 100°	0.65 S
101° - 106°	0.6 S
107° - 113°	0.55 S
114° - 120°	0.5 S

3. Effective length of weld:-

L_{eff} = actual length of weld - 2 × S where S = size of weld



- Actual length of weld should not be less than 4 times of size of weld.

4. Minimum size of weld:- Depends upon the thickness of thicker plate connected.

Plate thickness	Minimum size of weld
Upto 10 mm	3 mm
11 to 20 mm	5 mm
21 to 32 mm	6 mm
> 32 mm	8 mm

- Depends

5. Maximum size of weld:- Depends upon thickness of thinner plate.

$$t_2 > t_1$$

- Maximum size of weld = thickness of thinner plate - 1.5 mm (for square edge)
- 3/4 thickness of thinner plate (rounded edge)

6. Strength of weld:-

LSM:-

$$F_{wd} = \text{Resisting area} \times \text{Permissible stress}$$

Resisting Area = throat thickness (t) × Effective length of weld ×

$$f_{wd} = \frac{f_u}{\sqrt{3} \times 1.25}$$

$$= t \times l_{eff} \times \frac{f_u}{\sqrt{3} \times 1.25}$$

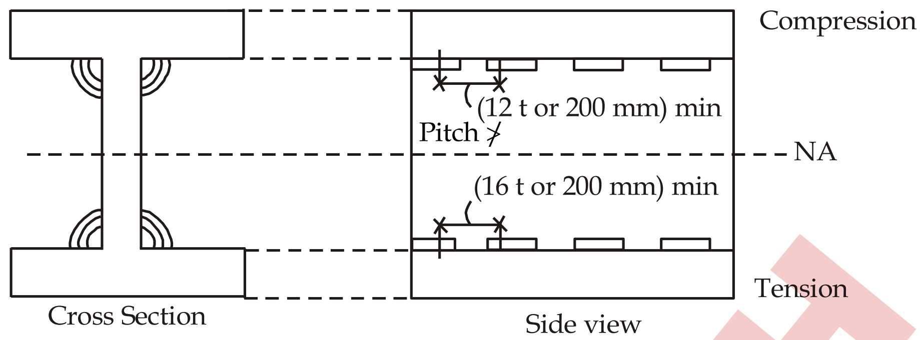
FOS:- 1.5 field welded

= 1.25 Shop weld

WSM:-

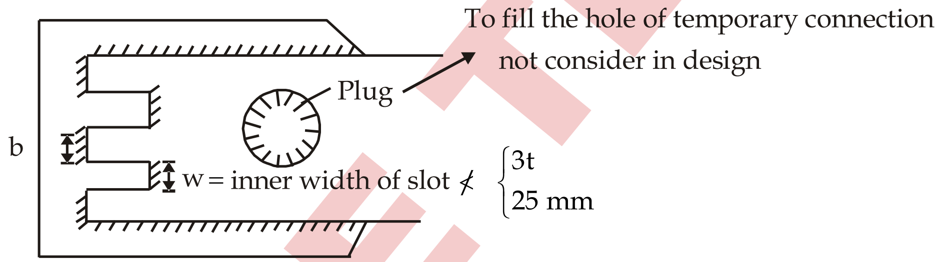
$$P_{wd} = l_{eff} \times t \times f_s$$
$$f_s = 0.4\ Fy \text{ or } 110\ N/mm^2$$
$$f_y = 250\ N/mm^2$$

Intermitent fillet weld:-



Intermitent fillet weld are provided to transfer calculated stress across the joint when the strength required is less than the developed by a continuous fillet weld.

Plug and Slot welding:-



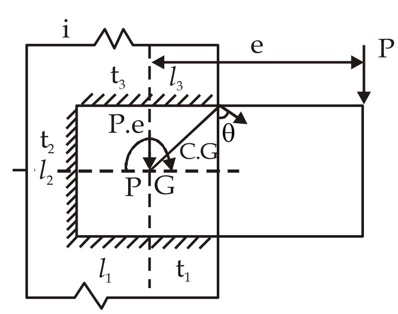
t = thickness of plate connected

b = outer width of slot ≮ 2t

→ Slot weld is used when length of overlap is restricted or limited.

Eccentric loaded connection

Type-1:- In plane eccentricity



Due to eccentric load P, C.G. of rivet group subjected a direct load P and twisting moment Pe

F_1 = stress in weld due to direct force P

$$F_1 = \frac{P}{\text{Prestressing area}}$$

$$F_1 = \frac{P}{l_{eff_1} \times t_1 + l_{eff_2} \times t_2 + l_{eff3} \times t_3}$$

F_2 = shear stress due to twisting moment Pe

$$\frac{T}{J} = T/r = F_2 = \frac{T}{J} \times r$$

J = Polar moment of inertia

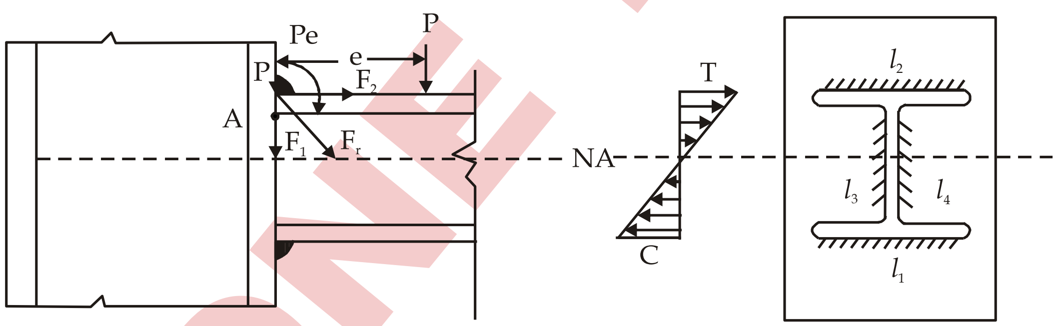
r = radial distance of point from C.G. of weld group.

$$F_r = \sqrt{F_1^2 + F_2^2 + 2F_1F_2 \cos\theta}$$

$$F_r \le \text{ or } \not> 108 \text{ N/mm}^2 \text{ (WSM)}$$

$$F_r \le \text{ or } \not> \frac{f_u}{\sqrt{3} \times \gamma_{mw}} \text{ (LSM)}$$

Out of plane eccentricity:-



$$F_1 = \frac{P}{\text{Prestressing area}}$$

$$F_1 = \frac{P}{l_1 t_1 + l_2 t_2 + l_3 t_3 + l_4 t_4}$$

F_2 = Horizontal shear stress at point A due to bending tension

$$F_r = \sqrt{F_1^2 + F_2^2} \le 108 \text{ N/mm}^2 \{WSM\}$$

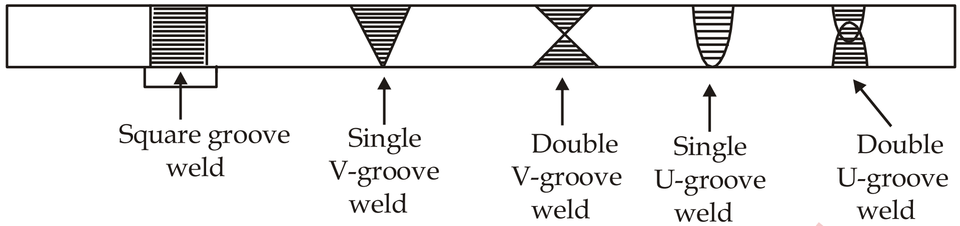
$$\frac{M}{I_{xx}} = F/y_{max}$$

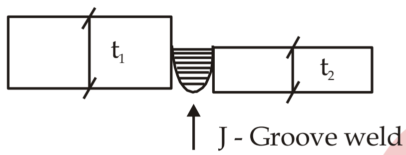
$$F_2 = \frac{M}{I_{xx}} y_{max}$$

$$F_r = \sqrt{3F_1^2 + F_2^2} \le \frac{f_u}{\sqrt{3} \times FOS} \{LSM\}$$

Designing of Butt weld or Groove weld

→ Used when the member to be jointed are lined up.





Partially Penetration of weld material
- Single V-Groove
- Single U-Groove
- J-weld

Fully Penetration of weld material
- Double V-Groove
- Double U-Groove
- Square-weld

Groove welds are usually design for direct tension or compression

Design strength of groove weld:-

It designs given by tension or compression

$$P_{Tw} = \frac{l_{eff} \times t_e \times f_{yw1}}{FOS}$$

l_{eff} = effective length of weld

$$t_e = \text{For fully penetration of weld} = \min\begin{Bmatrix} t_1 \\ t_2 \end{Bmatrix}$$

$$t_e = \text{For partially penetration of weld} = \min\begin{Bmatrix} \frac{5}{8} t_1 \\ \frac{5}{8} t_2 \end{Bmatrix}$$

$$f_{yw1} = \min\begin{cases} fy = \text{yield stress of plate material} \\ fy_w = \text{yield stress of weld material} \end{cases}$$

FOS = 1.5 [Field weld]
= 1.25 [Shop weld]

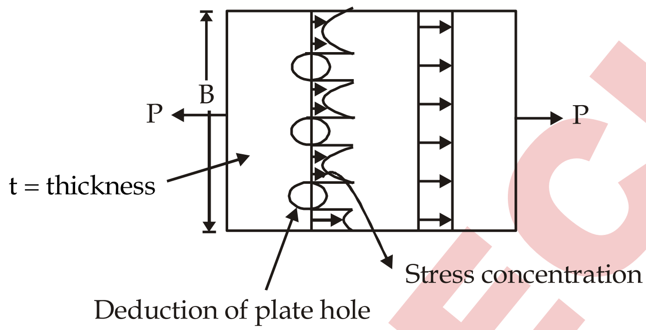
If design govern by shear

$$f_{yw2} = \min\begin{cases} \frac{f_y}{\sqrt{3}} \\ \frac{f_{yw}}{\sqrt{3}} \end{cases}$$

$$P_{Tw} = \frac{l_{eff} \times t_e \times f_{yw_2}}{FOS}$$

Chapter - 5
(Design of tension member)

Design of Pitch (P)



Net area = A_{gross} - deduction of hole

$B \times t - nd_o\, t$

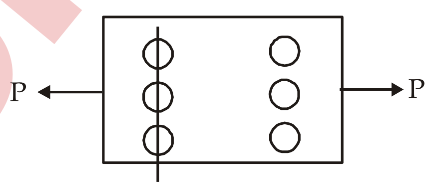
$(B - nd_o) \times t$

The Net section area of a tension member is the gross section area of the member section minus area of the maximum number of holes.

The unit stress in a tension member is increased due to the presence of a hole this is because the area of steel to which load is distributed is reduced and some concentration of stress occurs along the edges of hole.

Net section area of plate elements

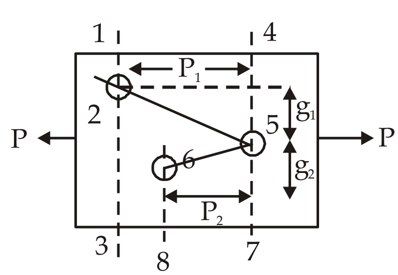
A. If rivet/Bolt holes in same line



$A_{net} = Bt - 3d_o \times t$

$(B - 3d_o) \times t$

B. ...... are staggered



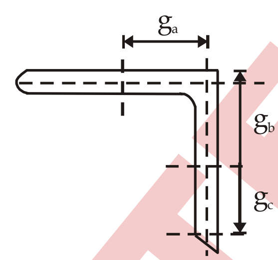
$$\text{Deduction} = nd_o \times t - \frac{\Sigma P_i^2}{4g_i} \times t$$

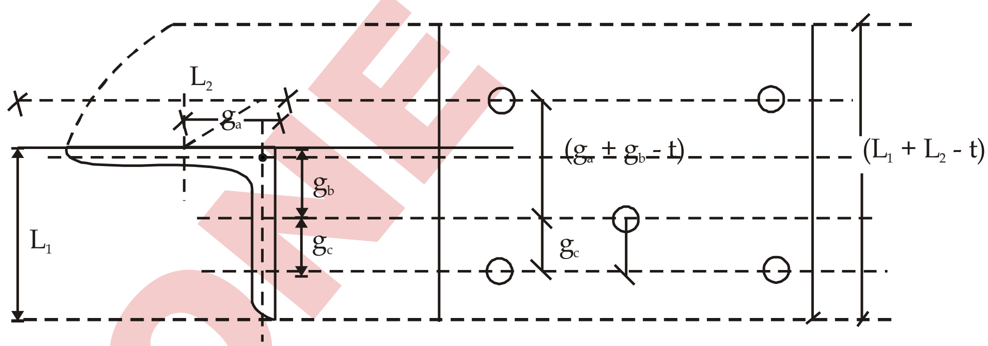
$$A_{net} = Bt - nd_o\, t + \frac{\Sigma P_i^2}{4g_i}\, t$$

$$= \left[(B - nd_o) + \frac{\Sigma P_i^2}{4g_i}\right] \times t$$

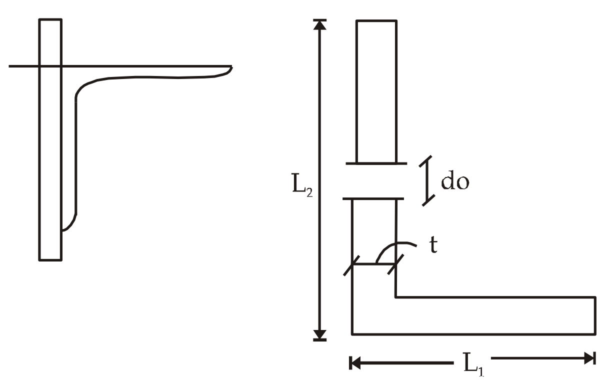
2. **Net sectional area for angle section:-**

A. When all legs are connected:-





B. If only one leg is connected:-



Net area of connected leg $= (L_2 - t/2 - d_o) \times t$

Net area of outstanding leg $= (L_1 - t/2) \times t$

$A_{net} =$ net area of angle section

$= (L_2 - t/2 - d_o)\, t + (L_1 - t/2)t$

Effective net area of angle section:- (LSM)

$A_{net\ effective} = K_1 K_2 K_3 K_4\, A_{net}$

$K_1 =$ ductility factor as per IS800 - 2007

$K_2 =$ depends upon method of fabrication

$K_2 = 1$ (For drilled hole)

$K_2 = 0.85$ (For punched hole)

$K_3 =$ Geometory factor $= 1$

$K_4 =$ Shear leg factor

$K_4 = 0.6 =$ If number of bolt $= 2$

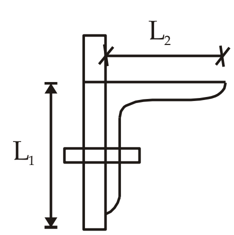
$0.7 =$ If number of bolts $= 3$

$0.8 =$ If number of bolts ≥ 4

$0.8 =$ If leg is connected by welding

WSM approach:-

1. If single angle connected to gusset plate:-



$A_{net} = A_1 + A_2$

$A_1 = (L_1 - d_o - t/2) \times t$

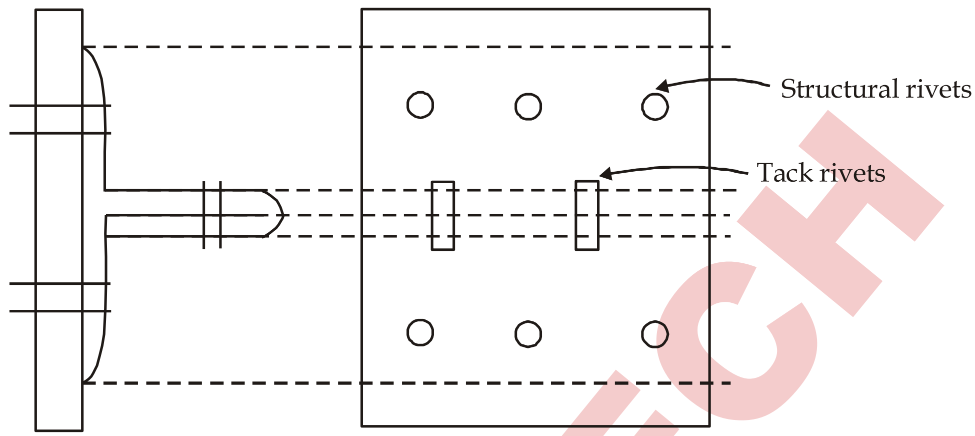
$A_2 = (L_2 - t/2) \times t$

$A_{net} = A_1 + KA_2$

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

2. If two angles are connected to gusset plate

(i) Both angles are on same side of gusset plate



(a) If both angles are tack riveted through

$$A_1 = (L_1 - d_o - t/2) \times t] \times 2$$

$$A_2 = (L_2 - t/2) \times t] \times 2$$

$$A_{net} = A_1 + KA_2$$

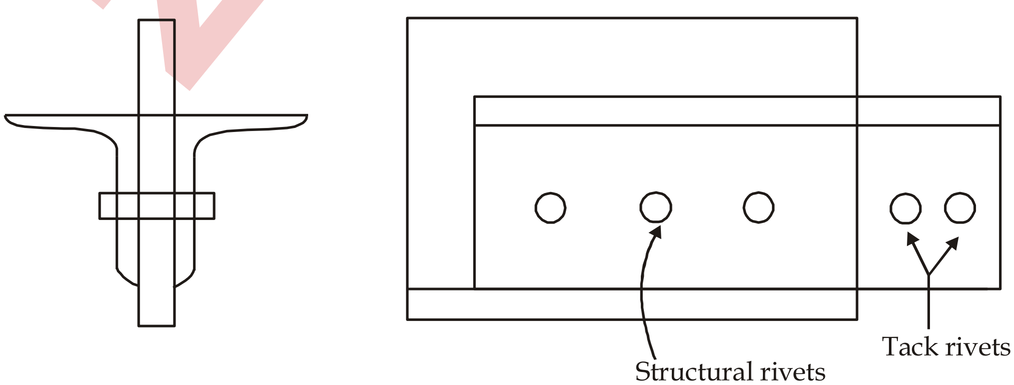
$$K = \frac{5A_1}{5A_1 + A_2}$$

(b) If both angles are not tack riveted

$$A_{net} = A_1 + KA_2$$

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

(ii) Both angles are on either side of gusset plate:-



(a) If both angles are tack riveted:-

$$A_{net} = A_1 + KA_2$$

$$K = 1$$

(b) If both angles are not tack riveted:-

$$A_{net} = A_1 + KA_2$$

$$K = \frac{3A_1}{3A_1 + A_2} \text{ (same as single angle)}$$

Design strength of tension member:-

(i) Plate or Flats:-

(a) Gross Area strength:-

$$P_{Tg} = \frac{A_{gross} \times f_y}{1.1}$$

(b) Net section Area strength:-

$$P_{Tn} = \frac{0.9 A_{net} \times f_u}{1.25}$$

$$\text{Strength} = \text{Min } [P_{Tg}, P_{Tn}]$$

(ii) If one leg of angle section connected:-

(a) Gross Area strength:-

$$P_{Tg} = A_{gross} \times \frac{f_y}{1.10}$$

(b) Net area strength:-

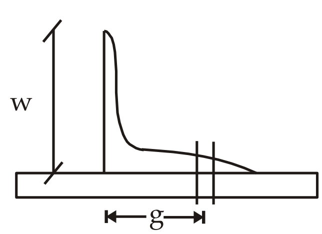
$$P_{Tn} = \text{Tearing strength of angle section}$$

$$= 0.9\, A_{net\ connected} \times \frac{f_u}{1.25} + \beta \times A_{gross\ outstanding} \times \frac{f_y}{1.10}$$

$$\beta = 1.4 - 0.076 \times \frac{w}{t} \times \frac{f_y}{f_u} \times \frac{b_s}{L_c} \geq 0.7 \leq \frac{f_u}{f_y} \times \frac{1.10}{1.25}$$

A_{nc} = Net area of connected leg

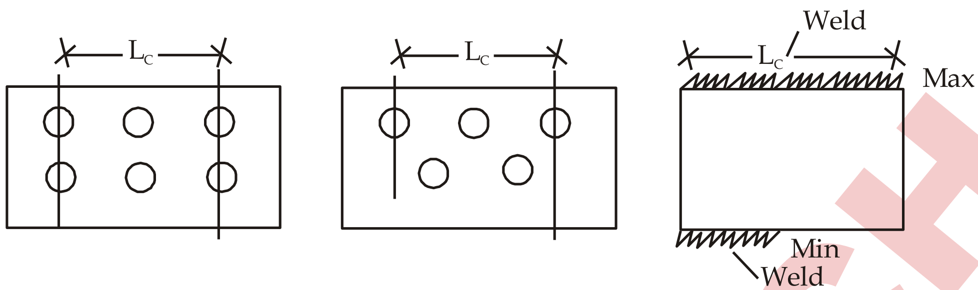
A_{go} = Gross area of outstanding leg



w = length of outstanding leg

$b_s = w + g - t$

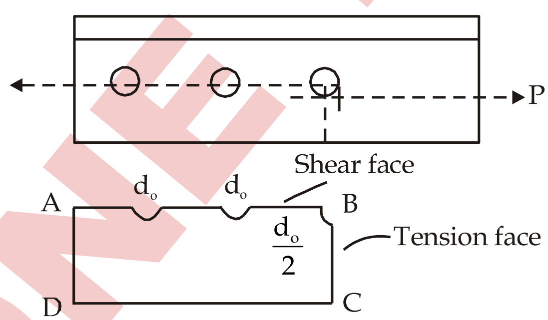
L_c = distance between end bolts



L_c = distance between end bolts or maximum length of weld = L_c

Alternate Approach:-

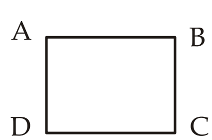
$$P_{Tn} = \frac{K_1K_2K_3K_4 \times A_{net} \times 0.9 \times f_u}{1.25}$$



2. Tension yielding and shear fracture

$$P_{Tb_1} = \frac{A_{Tg} \times f_y}{1.10} + \frac{A_{vn} \times 0.9 \times f_u}{\sqrt{3} \times 1.25}$$

A_{Tg} = Gross area of tensile force = BC × t



A_{vn} = Net area of shear force

$= (AB - d_o - d_o - d_o/2) \times t$

(ii) Shear yielding and tension fracture:-

$$P_{Tb_2} = \frac{A_{vg} \times f_y}{\sqrt{3} \times 1.10} + \frac{0.9 \times A_{Tn} \times f_u}{1.25}$$

A_{vg} = Gross area of shear force

= AB × t

A_{Tn} = Net area of tensile force

$$= (BC - \frac{d_o}{2}).\ t$$

Block shear strength

$$= \text{Min} \begin{cases} P_{Tb_1} \\ P_{Tb_2} \end{cases}$$

Tensile strength of member:-

$$= \text{Minimum of } [\ P_{Tg}, P_{Tn}, P_{Tb_1}, P_{Tb_2}\ ]\ (\text{LSM})$$

Tensile strength (WSM):-

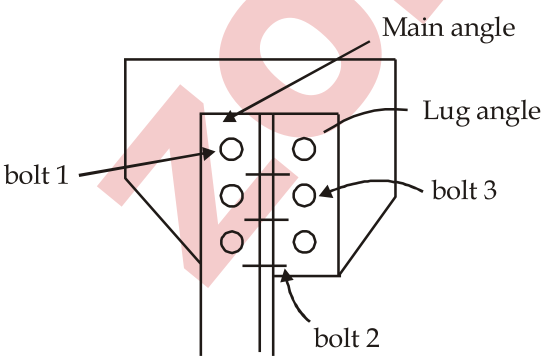
$$P_T = A_{net} \times 0.6\ Fy$$

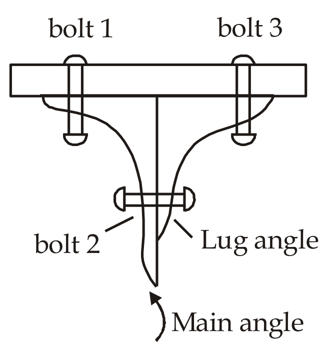
or

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

σ_{at} = Permissible stress in axial tension

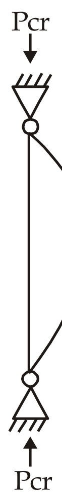
Lug angle:-

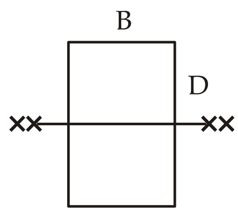




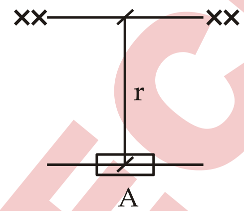
- Lug angle is a small piece of angle used to connect outstand leg of the member to gusset plate.
- If lug angle is used the outstand leg of the main angle behaves like a connected leg and it reduces the shear leg effect.

CHAPTER - 6
(DESIGN OF COMPRESSION MEMBER)





$I_{xx} = \frac{BD^3}{12}$



$I_{xx} = Ar^2$

r = Radius of gyration

Euler Bucking Formula:-

$$P_{cr} = \frac{\pi^2 EI}{L^2 eff}$$

$$I = Ar^2$$

where r = radius of gyration

A = cross-x area

$r = I/A$

$$Pcr = \frac{\pi^2 EAr^2}{Leff^2}$$

$$P_{cr} = \frac{\pi^2 EA}{(L_{eff}/r)^2} \quad \lambda = l_{eff}/r$$ where r = appropriate radius of gyration

$$\frac{P_{cr}}{A} = \frac{\pi^2 E}{(L_{eff}/r)^2}$$ λ = Slenderness ratio, l_{eff} = effective length

$$f_{cr} = \frac{\pi^2 E}{\lambda^2}$$

f_{cr} = Critical buckling stress

For Fcr minimum

λ should be maximum

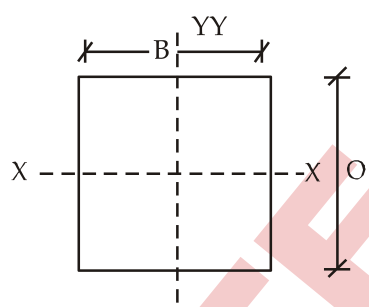
$\lambda \uparrow$ then r should be minimum

Section should be buckle about minimum radius of gyration

(For r min)

Case I:-

If one of the centroidal axis is axis of symmetry.



$I_{XX} = BD^3/12$

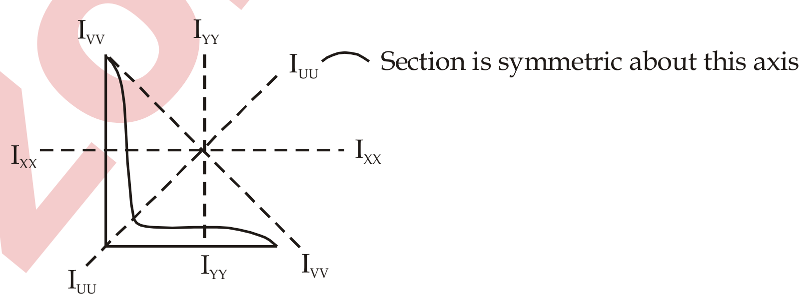
$I_{YY} = DB^3/12$

I_{YY} is minimum so buckling is about I_{YY}.

The section symmetrical about both the centroidal axis or even with one axis of symmetry will buckle about one of the centroidal axis.

Case II:-

If none of the centroidal axis is not axis of symmetry.

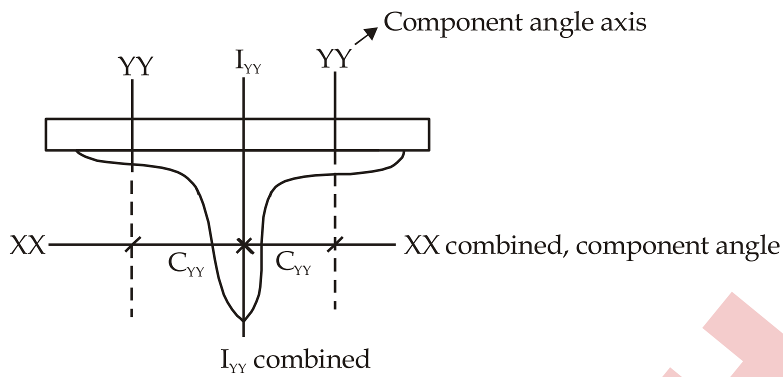


I_{UU} and I_{VV} are principle axis.

The section is asymmetrical about the centroidal axis will buckle about the principle axis for which radius of gyration will be minimum.

Case III:-

If two angles are placed same side of gusset plate and tack riveted throughout the length.



I_{XX} combined:-

$= I_{XX}$ component × 2

I_{YY} combined:-

$= 2\,[I_{YY}$ component + A. $C_{YY}^2]$

Minimum [I_{XX} combined or I_{YY} combined]

Section will be buckle about minimum of I_{XX} combined or I_{YY} combined.

I_{XX} is minimum moment of inertia-

$$r_{XX}\text{ component} = \sqrt{\frac{I_{XX}\text{ combined}}{A\text{ combined}}}$$

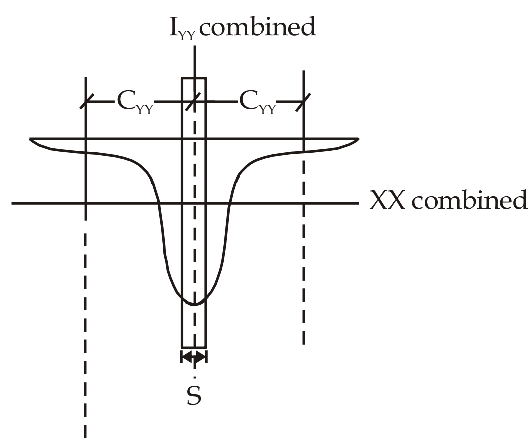
$$= \sqrt{\frac{2 \times I_{XX}\text{ component}}{2 \times \text{Area component}}}$$

$$= \sqrt{\frac{I_{XX}\text{ component}}{\text{Area component}}}$$

$$r_{XX}\text{ combined} = r_{XX}\text{ component}$$

Case IV:-

If two angle section are placed either side of gusset plate.



I_{XX} combined $= 2 \times I_{XX}$ component

I_{YY} combined $= 2 \times [I_{YY}$ component $+ A \times (C_{YY} + S/2)^2]$

→ Minimum slenderness ratio for structural member:-

1. Tension member subjected to live load $\lambda = 400$
2. Tension member in which reversal of load due to load other than wind or earthquake forces $\lambda = 180$.
3. Compression member subjected to live load $\lambda = 180$.
4. Tension member reversal of stresses due to wind or earthquake forces $\lambda = 350$.
5. Member subjected to compressive forces only due to earthquake and wind $\lambda = 250$.
6. Compression flange of beam restrained against lateral tensional buckling $\lambda = 300$

Design column formulas:-

IS 800-1984 - (WSM):- This code recommended the use of MERCHANT RENKINE'S formula.

$$\sigma_{ac} = \frac{Fcc \times Fy \times 0.6}{\left[(Fcc)^n + (Fy)^n\right]^{1/n}}$$

f_y = yield stress

f_{cc} = critical buckling strength

η = imperfection facrtor

$\eta = 1.4$

IS 800-2007:- Recommended the use of suitable column curve based upon PERRY ROBERISON approach.

1. Design strength of compression member by WSM approach:-

Step I:- Find out $\lambda = \frac{Leff}{r_{appropriate}}$

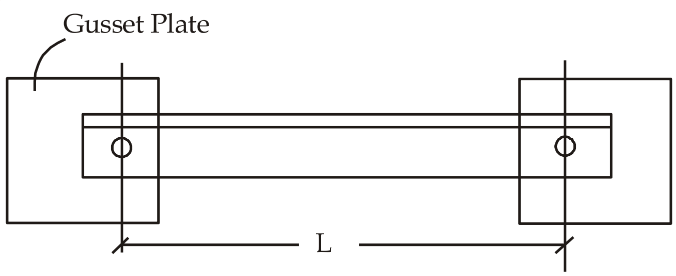
Step II:- By the use of λ value of σ_{ac} will be find out by use of IS-800-1984.

Step III:- Design strength of compression member

$$P_{cd} = \sigma_{ac} \times A_{gross}$$

Effective Length of Strut:-

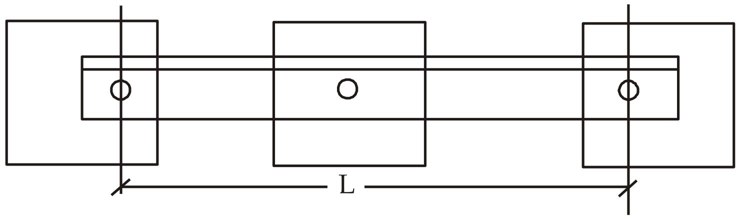
1. Discontinuous strut:-



$Leff = L$

Single angle connected by one bolt or rivet.

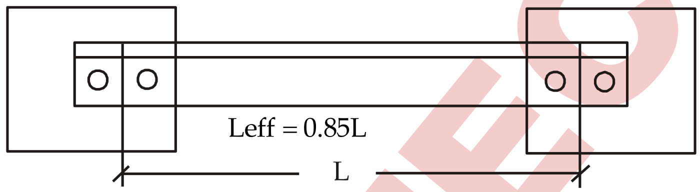
2. Continuous strut:-



$L_{eff} = 0.7L$ to L

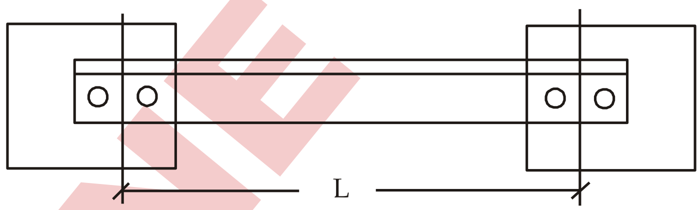
Single or double angle connected with one or more bolt and weld.

3. Discontinuous strut:-



Single angle with two or more bolt or weld.

4. Discontinuous strut:-



$L_{eff} = 0.7L$ to $0.85L$

Double angle with one or more than one bolt or weld.

Specifications for tack rivets:- If tack rivets are used to connect components of the column the slenderness ratio for each component between tack rivet.

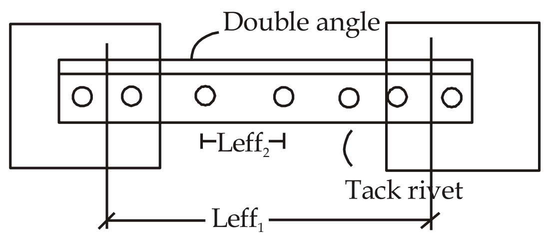
λ component between two tack rivets

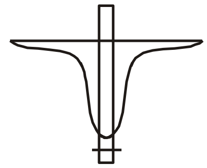
$\ngtr 40$

$\ngtr 0.6\ \lambda_{whole}$

λ_{whole} or $\lambda_{combined}$:-

$$= \frac{L_{eff_1}}{\gamma_{XX}\ \text{component}}$$





λ component between two tack rivets:-

$$= \text{Leff}_2 / \text{Min}\,(r_{VV} \text{ or } r_{UU})$$

Strength of column according to LSM:-

1. Design f_{cd} can be calculated as per clause no - 7.1.2.1 of IS 800 - 2007.

   It can also be calculated by table no. 8/ table no. 9 of IS 800-2007.

2. The value of Fcd depends upon slenderness ratio and buckling curve of given section.

   Alternate approach to calculate f_{cd} as per IS 800 - 2007 clause no. 7.1.2.1

f_{cd} = design compressive strength

$$Fcd = \frac{Fy}{1.10} \times \chi \qquad \eta = \text{Reduction factor}$$

$$\eta = \frac{1}{\phi + \left[\phi^2 - \lambda^2\right]^{0.5}}$$

$$Fcd = \frac{Fy/1.10}{\phi + \left[\phi^2 - \lambda^2\right]^{0.5}}$$

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

λ = Non dimensional slenderness ratio

$= \sqrt{f_y / f_{cc}}$ - critical buckling strength

α = Imperfection factor depends upon buckling curves.

Buckling curve	a	b	c	d
α	0.21	0.34	0.49	0.76

Design of compression member:- When load is given and size of column is required.

Step I:- Assumptions of σ_{ac} or f_{cd}

(a) For hot rolled sections = 60 MPa - 80 MPa

(b) For built up section = 110 MPa - 125 MPa

Step II:- Area required for column section:-

P = Load which is to be resisted

σ_{ac} assumed.

Step III:- According to this area required choose a appropriate area from steel table SP-6

Step IV:- After selecting area

r_{XX} and r_{YY} will be find out

and for r_{min}

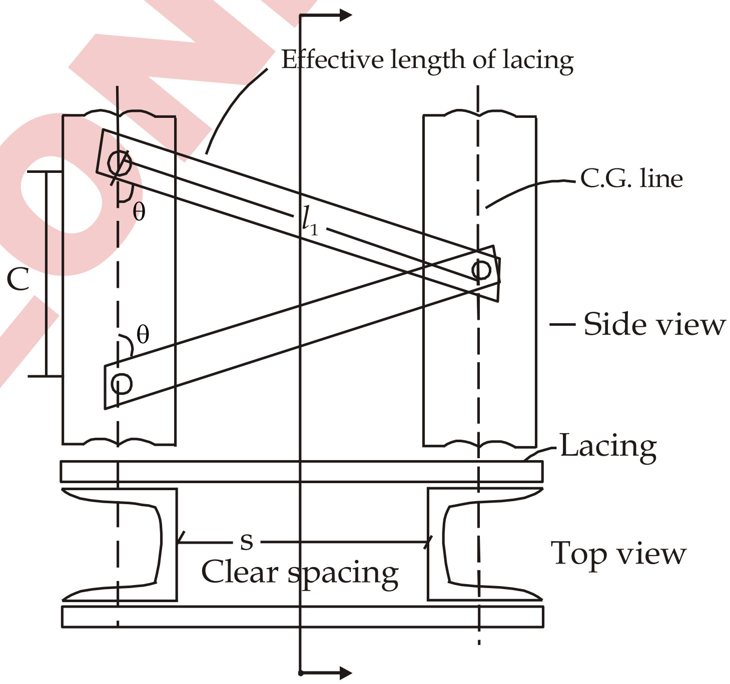
λ = Leff/ r_{min} will be find out

Step V:- According to this λ

σ_{ac} and Fcd will be find out and load carrying capacity of section is determined.

$P_{cd} = \sigma_{ac} \times A_{gross} > P$ = Load for which compression member is to be designed.

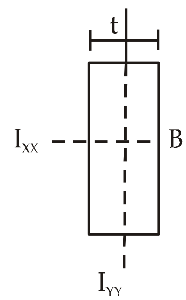
Design of Lacing system:-



1. Maximum slenderness ratio of lacing flat - 145

2. Angle from vertical should not less than 40° and should not be greater than 70°

3. $\lambda_{Lacing} = Leff/\gamma_{min}$



$I_{YY} = I_{min}$

$= \frac{Bt^3}{12}$

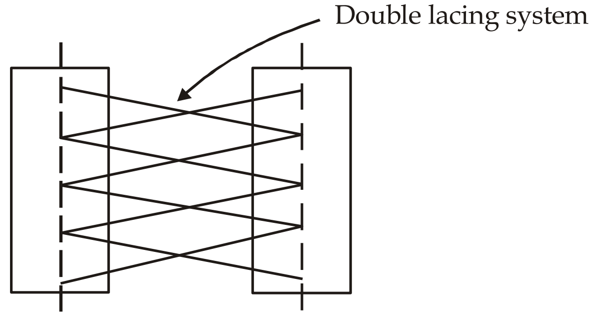
$r_{min} = \sqrt{\frac{Bt^3/12}{Bt}}$

$\lambda_{Lacing} = t/\sqrt{12}$

4. Minimum width of lacing bar

Nominal diameter of rivet	Width
16	50
18	55
20	60
22	65 mm

Double lacing system:-



The thickness of lacing bar:-

$\frac{l_1}{40}$ = For single lacing

$\frac{l_1}{60}$ = For double lacing

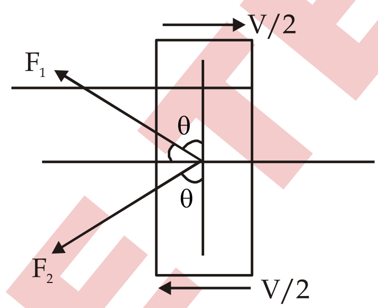
To prvent local buckling of column component between any two lacing.

$$\lambda_{component} = \frac{C}{r_{component}} \ngtr 50 \quad \ngtr 0.7\lambda \text{ whole}$$

Forces in lacing bar:-

Lacing system is designed for transverse shear of magnitude 2.5% of column axial load.

To take care of accidental load and initial eccentricity.



$\Sigma fx = 0$

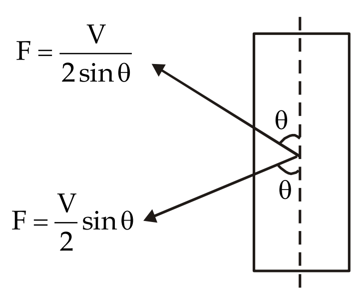
$F_1 \sin\theta = V/2$

$F_1 = V/2\sin\theta$ (Tension)

$F_2 \sin\theta + V/2 = 0$

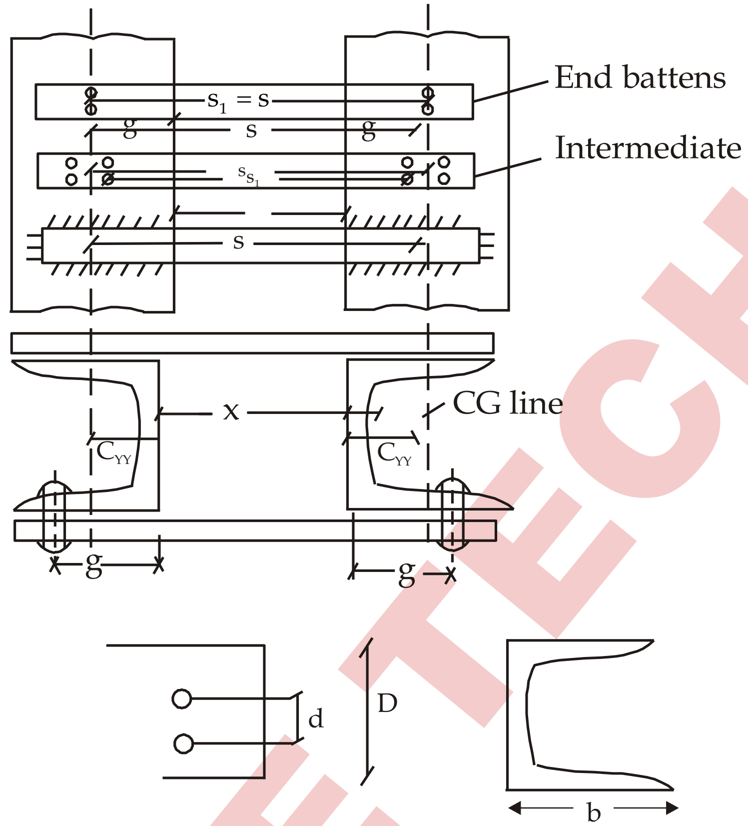
$F_2 = -V/2\sin\theta$ (Compression)

Forces in rivets:-



Forces in rivet $= 2F\cos\theta$

Design of battens:-



C = Length of column component between any two battens.
X = Clear spacing between two battens
a = x + 2Cyy = distance between the CG of two column components.
S_1 = distance between two inner rivets or bolts.
S = CG of distance between C.G. of rivet group or bolt group or weld.
S = X + 2g

1. In case of battens the effective length of column is increased by 10%
2. Minimum number of battens should be 4.
3. To avoid local buckling between any two battens slenderness ratio should not be greater than

$$\lambda = \frac{C}{\gamma_{min\ component}} \begin{array}{l} \not> 50 \\ \not> 0.7\lambda \text{ whole} \end{array}$$

1. Effective depth of intermediate batten:-

$d \not< 3/4\ a$

$d \not< 2b$

a = distance between CG of two column component
b = width of flange of column component.

2. Effective depth of end battens:-

$$d \nless a$$

$$d \nless 2b$$

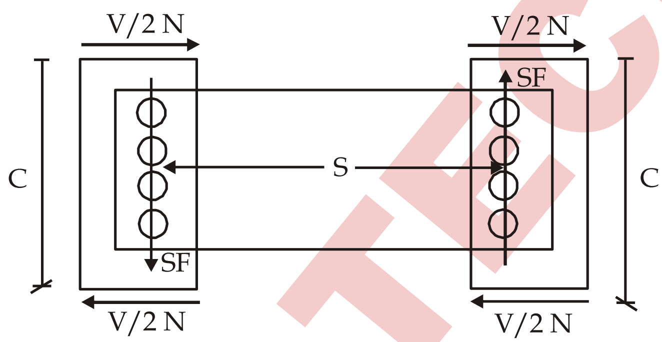
3. Thickness of batten:-

$$t = \frac{S_1}{50}$$

S_1 = distance between two inner rivets or bolts.

4. Forces in battens:-

Design for - 2.5% of column axial load

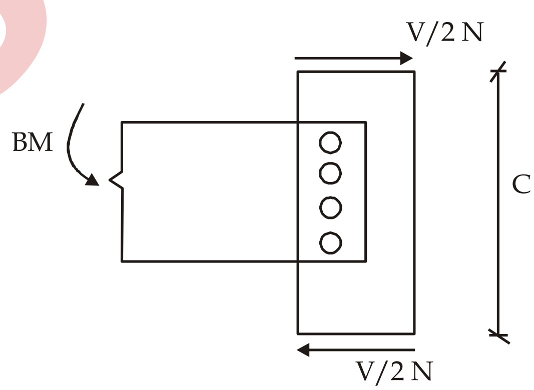


If N = Number of batten plane

$$SF \times S = \frac{V}{2N} \times C + \frac{V}{2N} \times C$$

$$SF = \frac{VC}{NS}$$

5. Bending moment



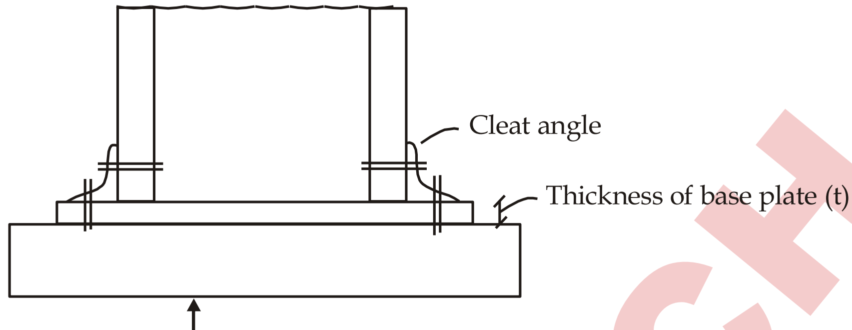
$$\Sigma Mz = 0$$

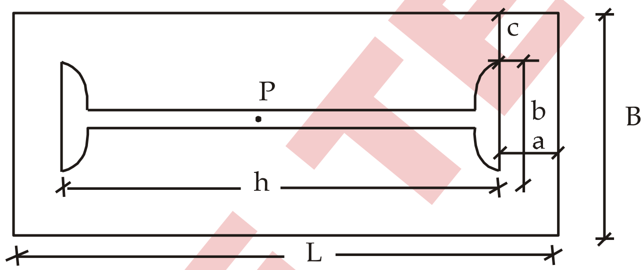
$$BM = \frac{VC}{2N}$$

Design of base plates:-

Types of base plates:-

(a) Slab base:-





Top view

a ≥ c

If ends of column is properly maintained then it is assumed that the bearing will develop between the column and bearing plate.

- It is assumed that 50% of load is passing through cleat angle. (machine end)
- If ends are not properly machined then cleat angles will be design for total load which is to be transfer to the foundation.

WSM:-

$$t = \sqrt{\frac{3w}{b_s}\left[a^2 - \frac{c^2}{4}\right]}$$

$w =$ Pressure on the base plate

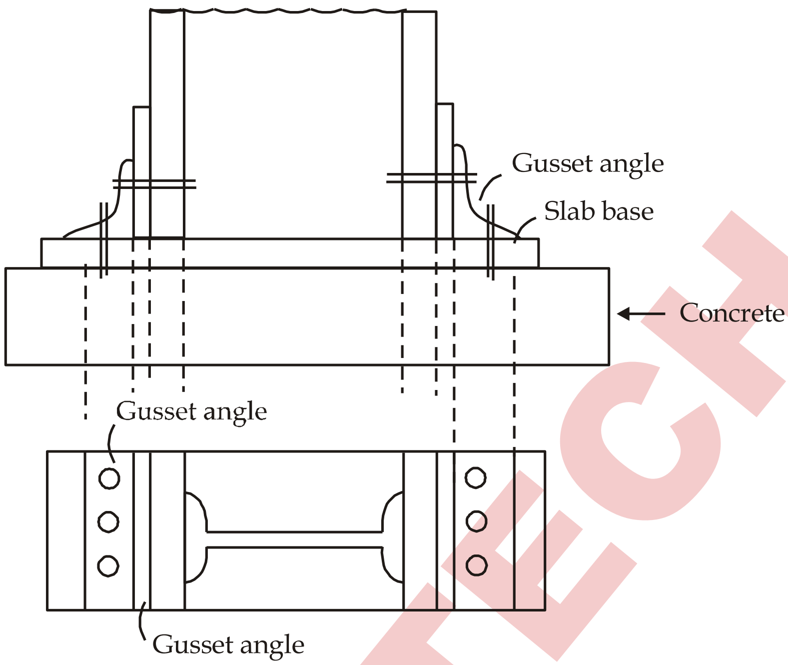
$b_s = 185\ N/mm^2$

LSM:-

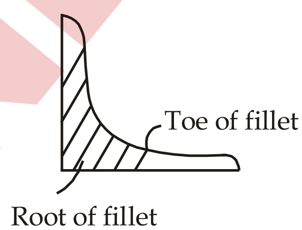
$$t = \sqrt{\frac{2.5W_u}{f_y / 1.10} \times \left(a^2 - 0.3c^2\right)}$$

Wu = Factored pressure on base plate (w × 1.5)

(b) Gusseted slab base:-



- In a gusset slab base, base plate, gusset plate, gusset angle are used.
- Gusset plate and gusset angle are used to increase the stiffness of base plate against bending.
- The critical section for B.M. is taken at toe of fillet.



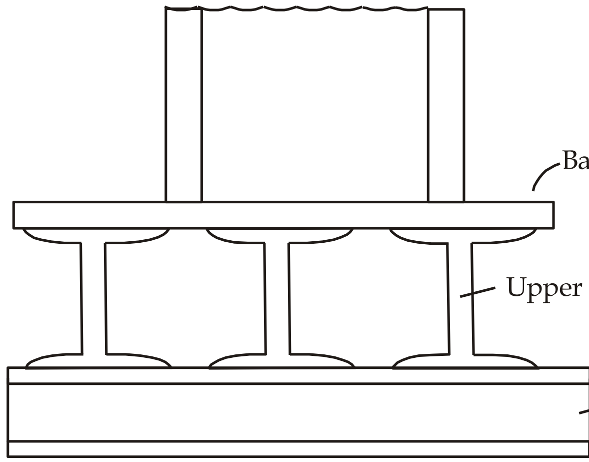
3. Grillage Foundation:-

Requirements of Grillage foundation:-

1. The load on the column are extremely heavy base plate requirement in such case will be of very large dimension.
2. Bearing capacity of the soil on which concrete foundation is to be placed may be poor which results in large dimension of concrete foundation and base plate.

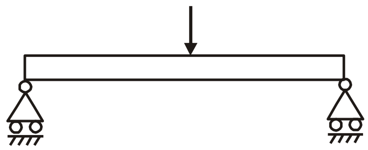
Assumptions for grillage foundation:-

1. Load from the column is uniformly distributed over the base plate.
2. Each tier of beam distribute the load uniformly to lower tier.
3. Intensity of bearing pressure is uniform.



- Minimum spacing between flange of two consecutive beams is not less than 75 mm.
- A concrete cover of minimum 100 mm is provided all around the beam arrangement.

CHAPTER - 7
(DESIGN OF FLEXURAL MEMBER)

1. 

Flexural member:- member subjected to transverse load.

Flexural Formula:-

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

$$f_{max}=\frac{M}{I}.y_{max}$$

$$\frac{I}{y_{max}}=Z=\text{sectional modulus}$$ where $Z=$ bending strength of section

$$f_{max}=\frac{M}{I/y_{max}}$$

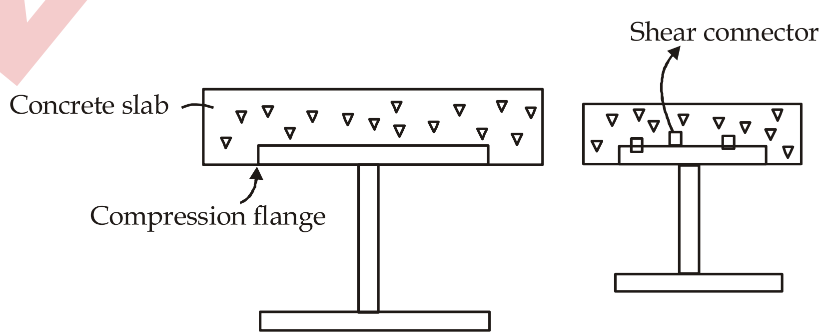
$$f_{max}=\frac{M}{Z}$$

WSM:-

$f_{max}=0.66\,f_y$ for compression and tension.

(a) **Laterally restrained beam or laterally supported beam:-**

If compression flange of beam is completely restrained against lateral movement then it is called laterally supported beam.



Permissible bending stress in compression flange $=0.66Fy$

(b) **Laterally unsupported beam:-** If compression flange of beam is free to move laterally then this type of beam is called laterally unsupported beam.

$$\sigma_{ac} = \min\begin{cases} \dfrac{f_{cc} \times 0.66 f_y}{\left[(f_{cc})^n + (f_y)^n\right]^{1/n}} \\ 0.66 f_y \end{cases}$$

Note:- In the beam design it is assumed that bending moment is resisted by flange and shear force is resisted by web.

Design Criteria for Beams

Primary Design Criteria
(i) Design for max BM
(ii) Check for shear
(iii) Check for permissible deflection

Secondary Design Criteria
(i) Local buckling of web and flange
(ii) Web buckling
(iii) Web cripling

1. **Primary design criteria:-**

Design steps for beam:-
(Laterally supported beam)

Step I:- After determination of maximum B.M.

$$Z_{req} = \frac{\text{Max BM}}{\sigma_{bt} \text{ or } \sigma_{bc}}$$

Step II:- Provide suitable section according to Z_{erq} by the use of steel table.

Step III:- Check for shear

$$\tau_{v\ calculated} = \frac{\text{Max SF}}{d_w \times t_w} \not> 0.4 f_y$$

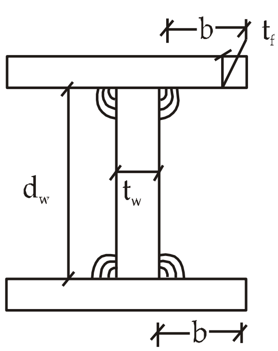
Step IV:- Check for maximum deflection

$$\delta_{Max} \not> \text{span}/325$$

$$\frac{5}{384} \cdot \frac{wl^4}{EI} \not> \text{span}/325$$

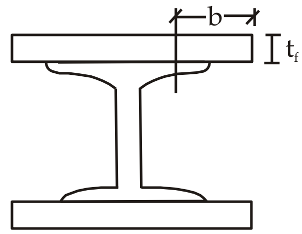
2. **Secondary design criteria:-**

(i) **Local buckling of web and flange:-**



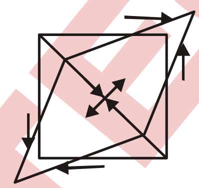
$b/tf - \ngtr 16$

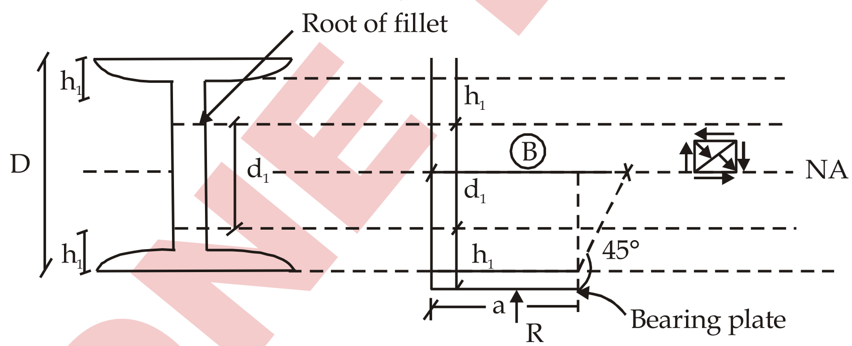
$dw/tw \ngtr 50$



$b/t_f \ngtr 50$

2. **Web buckling:-** Web buckling will occur due to diagonal compression which is produced due to high shear stresses.





$$B = (a + h_1 + d/2)$$

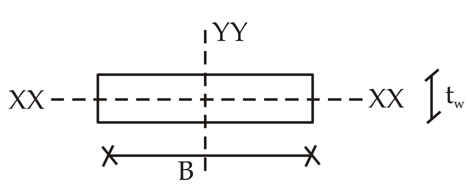
$$B = (a + D/2)$$

Length of column $= d_1$

Effective length of column $= d_{1/2}$ (both end fixed)

Slenderness ratio of d_1 length column

$$\lambda = \frac{leff}{\gamma_{min}}$$



$$I_{min} = I_{XX} = \frac{B.tw^3}{12}$$

$$Area = B.tw$$

$$\gamma_{min} = \sqrt{\frac{B.tw^3}{12 / Btw}} = \frac{tw}{12} = \frac{tw}{2\sqrt{3}}$$

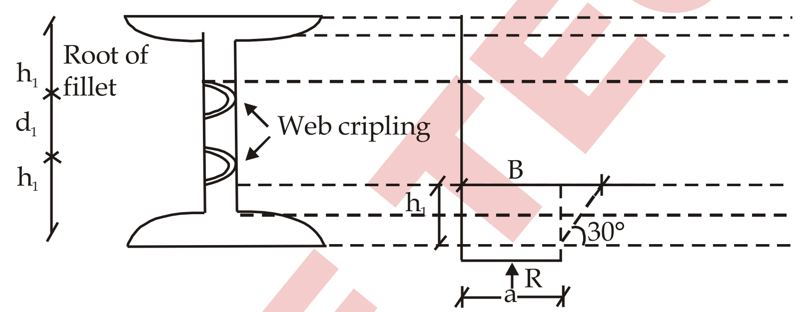
$$\left[\lambda = \frac{Leff}{\gamma_{min}} = \frac{d_{1/2}}{tw / 2\sqrt{3}} = \frac{\sqrt{3}d_1}{tw}\right]$$

By the use of λ, σ_{ac} will find out

Buckling strength of column

$= B.tw \times \sigma_{ac} \geq R$ prevent for web buckling

Web cripling or web cripling:-



$$B = (a + \sqrt{3}\, h_1)$$

Bearing area

$= B \times t_w$

Bearing stress at root of fillet:-

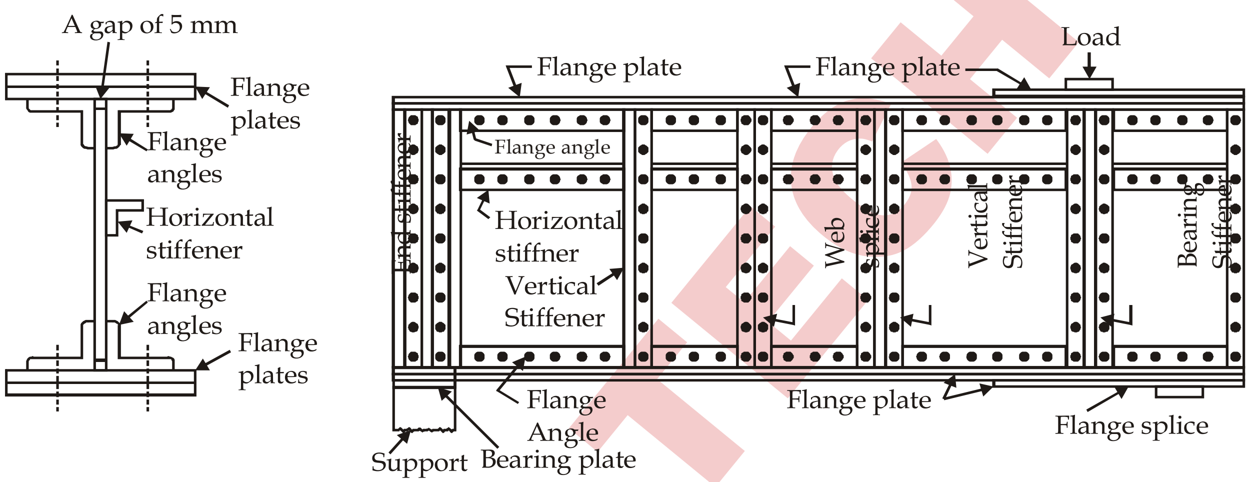
$$= \frac{R}{B.tw} \leq 0.75\, f_y$$

↑

To prevent web cripling

$0.75\, f_y$ permissible bearing stress

CHAPTER - 8
(PLATE GIRDER)



☞ **Some Important point:**

- 5mm is provided to isolate flanges from web so that direct bearing action on web avoided and web should exist shear only.
- Effective flange area in compression zone:-

$$A_f + \frac{A_w}{6}$$

A_f = area of compression flange

A_w = area fo web

- Effective area in tension zone will be

$$A_f + \frac{A_w}{8}$$

- Area of flange angles will be equal

$$\frac{A_f}{3}$$

- If unequal angle sections are used then longer leg will be connected to flange (To reduce bearing)

- Self wt. of plate girder $\frac{W}{300}$

  where

  W $=$ Total live load or imposed load

- Economical depth of plate girder

$$d_e = 1.1\sqrt{\frac{M}{\sigma_{bc} \times t_w}}$$

  where

  σ_{bc} $=$ Permissible stress in bending compression

$$= .66\, f_y \text{ or } \frac{0.66\, f_y . f_r}{\left[(f_{er})^n + (f_y)^n\right]^{1/n}}$$

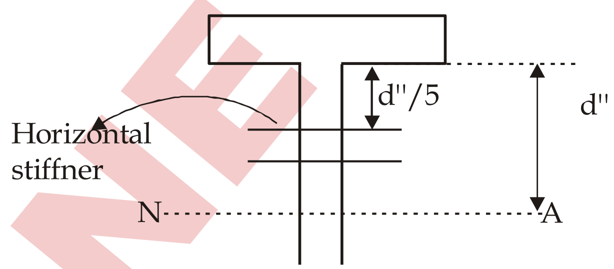
☞ **Vertical stiffners:** These stiffners are provide to resist chances of web buckling due to shear.

☞ **Horizontal stiffners:** Thuse are provided in compression zone to reduces chances of lateral buckling of web due to bending compression. These are also called as longitudinal stiffner.

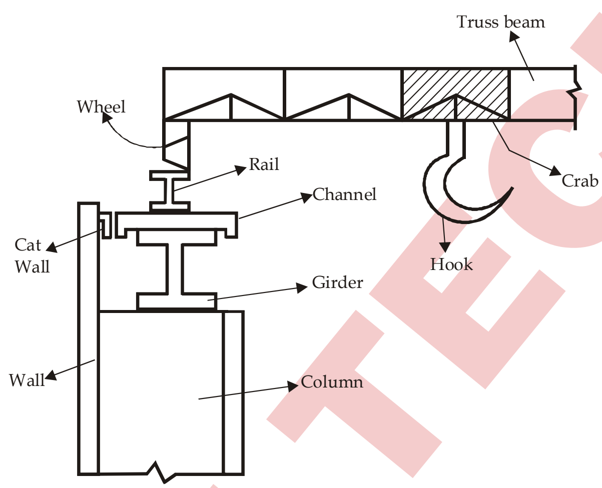
☞ **Load Bearing stiffners:** Load bearing stiffners are used to avoid crippling by transferring load from are flange to another flange.

Criteria for providing stiffners

	WSM	LSM
1.	$\frac{d_1}{t_w} < 85$ No stiffner requred	$\frac{d_1}{t_w} < 90$ No stiffner required
2.	$85 < \frac{d_1}{t_w} < 200$ Only vertical stiffners required	$90 < \frac{d_1}{t_w} \le 200$ Only vertical stiffner required
3.	$200 < \frac{d_1}{t_w} \le 250$ Along with vertical stiffners 1 horizontal stiffner is provided 1/5 of distance of compression flange from N. A	$200 < \frac{d_1}{t_w} \le 250$ Along with vertical stiffners 1 horizontal stiffner is required 1/5 of distance of compression flange from N. A
4.	$250 < \frac{d_1}{t_w} \le 400$ → Vertical stiffners +2 horizontal stiffners → Second horizontal stiffner will be provided at N.A.	$250 < \frac{d_1}{t_w} \le 400$ → Vertical stiffners +2 horizontal stiffners → Second horizontal stiffner will be provided at N.A.



CHAPTER - 9
(GANTRY GIRDER)



☞ **Gautry girder is subjected to there perpendicular forces:**

- **Vertical Force:** Due to self wt, wt. of material. wt of crab, truss girder.
- **Lateral load:** Due to different positions of crab.
- **Longidudinal load:** Impact due to movement of wheel in longitudinal direction.
- At a time where wheel will move crab will be at stop, and when crab will move wheel will be at stop means it is assumed that lateral and longitudinal forces will not act together.

☞ **Load Combinations:**

(i) Vertical + lateral

(ii) Vertical + longitudinal

- **Rail Channel:** Rail channel will increase the lateral strength of gantry girder.
- **Cat wall:** Cat wall is used to resist lateral movement of compression flange so that the chance lateral buckling can reduce significanly.

- **Channel secction:** Channel section over gantry girder is not used to provide torsional restrained becuse it is wt connected with web.
- **Web stiffners:** Are used to increase torsional resistance of web.
- The following impact factors are used for moving loads:
- **25%** of standing load .(For electrically operated)
- **10%** of standing load (For hand operated)

  {These two factors will multiplied in vertical load}
- **Longitudinal load:** 5% of static load.
- Vertical deflection of gantery girder should not exceed valus as:

1. When the crans are manually operated. deflection should not exceed = $\boxed{\frac{L}{500}}$

where

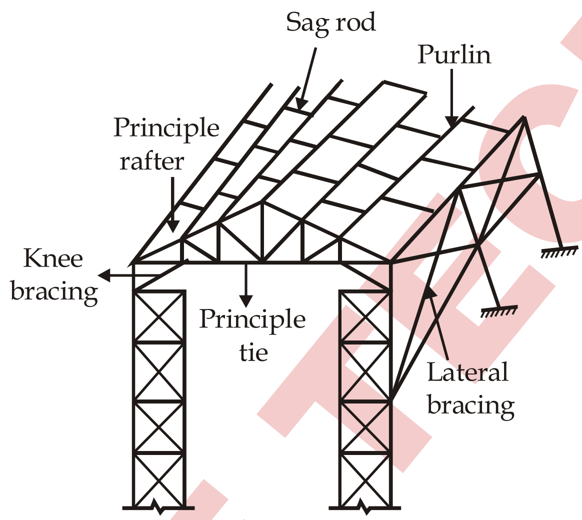
L = span of gantry girder.

2. Electric overhead travelling crans upto 500 kN capacity :

$$\boxed{\text{Deflection} = \frac{L}{750}}$$

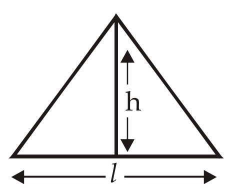
3. Electric overhed travelling crans for greater than 500 kN capacity, $\boxed{\Delta = \frac{L}{1000}}$

CHAPTER - 10
(ROOF TRUSSES)



☞ **Pitch of Roof:**

$P = \frac{h}{L}$ $\tan\theta = \frac{2h}{L}$



1. Always consider wind load perpendicular to the roof.

   Where

   L = span of truss

2. Economical spacing of truss varries from $\frac{L}{3}$ to $\frac{L}{5}$

3. Reversal of stress takes place due to wind or earthquake.

4. Bracings are designed to resist lateral loads.

5. Sag rods are designed for tensil load.

6. Purlins are designed as continuous member over trusses. (Continuous beam)

7. Design of purlins:

- Roofing material is supported by purlins

- The depth of purlin $\not< \frac{L}{45}$

  L = Length of purlin

- Width of purlin $\not< \frac{L}{60}$

- Maximum B.M. in purlin $\not> \frac{WL}{10}$

  W = Total load from roof.

- After calculating maximum B.M. check for bending stress generate in purlins.

  (σ_{bc}) calculate ≤ 0.66 fy

  (σ_{bt})calculate ≤ 0.66 fy

- **Check for deflection:** Deflection $\not> \frac{L}{325}$

☞ **Calculation of load:**

- **Dead Load :** For a given spacing of trusses as 4 m and pitch equal to 1/4, the self wt. of truss is taken as:

$$W = (l \times B)\left\{\frac{l}{3} + b\right\} \text{kg}$$

l = length of span

B = spacing of truss

- **Live load calculation:**

  (i)When $\theta < 10°$: L.L. $= 0.75\ kN/m^2$

  (ii)For $\theta > 10°$ L.L. $= 0.75 - (\theta - 10) \times 0.02$

  $\nless 0.4\ kN/m^2$

- Maximum roof angle $\ngtr 30°$

- **Snow load :–** For every 1 mm thickness of snow.

  Snow load $= 2.5\ N/m^2$

- To minimize the total cost of roof truss the ratio of cost of truss to cost of purlins shouls be 2.

- **Wind load:** When wind blows is generate wind drag (pressure) on roof surface.

- The wind pressure depends on slope of roof.

- Design wind pressure $P_z = 0.6 V_z^2 = K.V_z^2$

  where

  $P_z =$ design wind pressure.

- $V_z =$ design wind velocity

  $V_z = V_b.K_1.K_2.K_.$

  $V_b =$ Basic wind speed at height 10 m

  $K_1 =$ Probability factor (Risk factor)

  $K_2 =$ Terrain, height of structure and size factor $K_3 =$ Topography factor

CHAPTER - 11
(MISCELLANEOUS TOPICS)

Steel Bridges

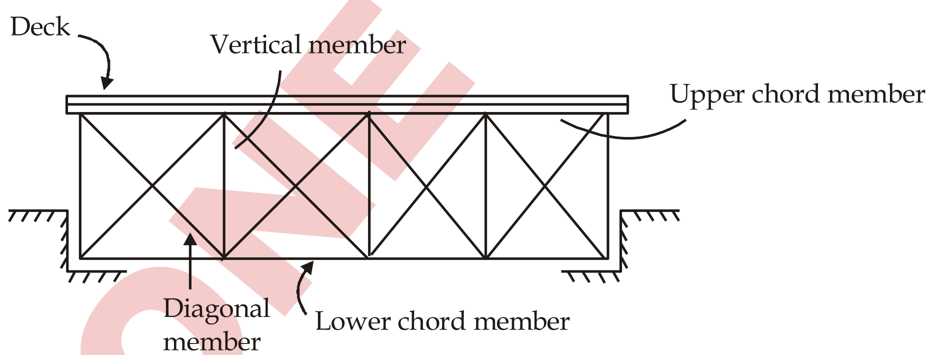
Types of steel bridges:-

1. T-section bridge
2. Plate girder
3. Truss girder bridge
4. Suspension bridge
5. Cable stayed bridge

Truss girder bridge:-

1. Deck type bridge (movement of vehicle at upper chord)
2. Through type bridge (movement of vehicle at lower chord)

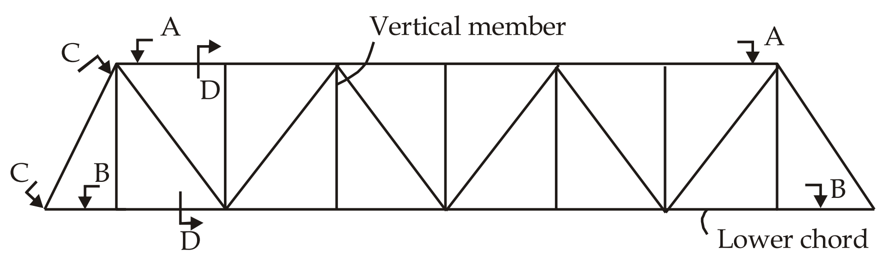
1. **Deck type bridge:-**



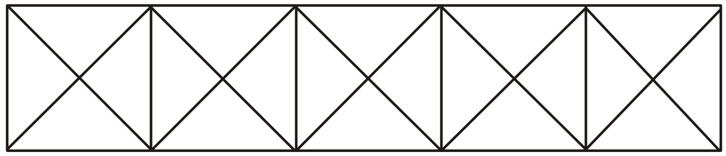
Front view

- In this type of bridge floor system is supported on top chord members.
- In this type of bridge sway bracings are provided to prevent rectangular shape of bridge.

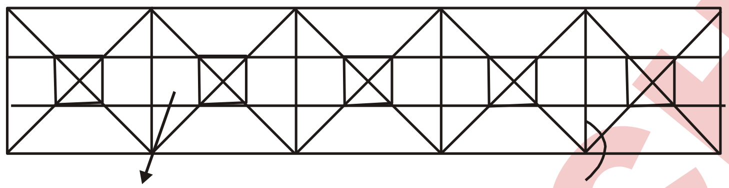
2. **Through type bridge:-**



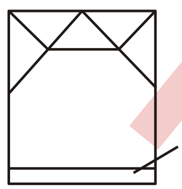
(a) Front view or Elevation of truss girder



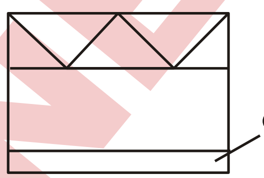
(b) Top lateral bracing view (A - A)



(c) Bottom lateral bracing view (B - B)



Portal bracing or end bracing view at C - C



Sway bracing view at D - D

This bridge consist of 1 main vertical trusses.

1. Floor system with bottom lateral bracings
2. Top lateral bracing
3. Portal bracings or end bracings
4. Sway bracings

- Top chord bracings and bottom chord bracings are provided to resist lateral load due to wind earthquake and raking forces.
- Top chord bracings are designed for a load addition to wind load =2.5% of sum of compression forces in top chord.
- Bottom chord bracings are subjected to wind load raking forces and longitudinal forces due to movement of vehicles.

- Portal bracings and sway bracings are provided to prevent rectangular shape of bridge.
- Portal bracings are designed for wind load and 1.25% of load on both top and bottom chord members.
- Sway bracing are provided to transfer 50% of top panel wind load to bottom panel.

Plate girder, Gantry girder, Roof truss

STEEL TANKS

- These are built to contain water, petrolium, other chemicals.
- **Types of steel tanks:-**

1. Rectangular type steel tank
2. Circular type steel tank
3. Pressed type steel tank

Pressed type steel tank:-

- Made of plates which are pressed in hot condition to get connected.
- These type of tanks have very less chances of seepage.
- The minimum capacity of pressed tank as per IS : 805 is 1950 litres.
- In case of steel tanks when rivets are provided there spacing in maximum case should not be greater than 10t. and minimum spacing should not be less than 3d.
- The minimum thickness of plates used for steel tanks should not be less than 6 mm in normal conditions. If storage of salt is taking place, minimum thickness should not be less than 5 mm.
- These two criterias are not applicable for roof of tank.

Permissible stresses in steel tanks:-

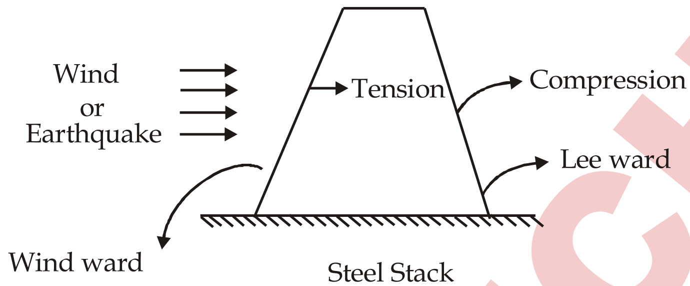
- The permissible stresses are reduced to take care of corrosion.
- **For steel plates (Permissible stresses):-**

1. Direct Tension/Compression = $0.8 \times 0.6\ f_y$
2. Bending Tension/Bending compression = $0.8 \times 0.66\ f_y$
3. In all other conditions = 0.8 × working permissible stresses.

- Efficiency of joing (connection) is considered as 70 to 75% in case of tension.

STEEL STACKS (CHIMNEY)

- Stacks are designed as per IS : 6533.
- In stacks minimum thickness should not be less than 6 mm except upper portions where thickness should not be less than 8 mm (To resist more corrosion at top)
- Width of steel plate is taken as 0.9 to 2.5 m.



- Temperature in steel stacks ≯ 400°C
- Permissible stresses are considered as 0.7 to 0.8 times of normal/working permissible stresses.
- In case of wind and earthquake, wind ward side is designed for tension and lee ward side is designed for compression and permissible stresses are increased by 15% in this case.