DSS

Syllabus

Reference
- → S.K. Duggal
- → S.S. Bhavikatti
- → L.S. Ramachandra

weightage
- → AEE — 18 to 24 Marks
- → AE — 14-17 m marks
- → SSC-JE → 12 to 15 marks
- → Deputy surveyor & TPBO → 8 to 10 marks
- → GATE → 4 to 6 marks
- → Engg (prc) — 12 to 16 marks; mains — 60 to 80 marks

Syllabus:

1) Materials & specifications
2) ** connections
   - concentric connections
     - Bolted & Rivetted
     - welded
   - Eccentric connections
     - Bolted & Rivetted
     - welded
3) Tension member ✓
4) ** compression member
5) column base & column splice
6) Design of beam.
7) plate Girder.
8) Gantry Girder.
9) Roof trusses ✓

IS specifications

Codes

- ↦ IS : 800 - 1984 → working stress method (WSM)
- ↦ IS : 800 - 2007 → Limit state method (LSM)

} code of practice for structural steel

- ↦ IS : 875 → Design Loads
  - → part I → Dead Load (D.L)
  - → Part II → Imposed load (L.L)
  - → part III → wind Loads (W.L)
  - → Part IV → Snow Load (S.L)
  - → Part V → Load combinations

→ IS: 1893 – 2002 → Seismic loads

→ IS: Hand book → steel tables.

- → weight/m
- → Radius of gyration
- → moment of inertia
- → section modulum

→ steel structures

RIP — R I V = Renold's no

PIG — F I G – froud's no.

VIP – E IP – Eulers no.

CI — C I E
MIphon — M I C } — cauchy's no = (machno)^2

west indis — W. I. S — weber no

Materials & Specifications

→ steel structures are Built up with hot Rolled steel section

→ Hot Rolled steel sections are made up of structural steel.

Advantages of steel structures (structural steel)

→ Higher strength

for M_{20} concrete, $f_{ck} = 20 MPa$

for Fe-250 steel, $f_y = 250 MPa$.

$$\frac{f_y}{f_{ck}} = \frac{250}{20} = 12.5 \Rightarrow \boxed{f_y \simeq 12 f_{ck}}$$

compressive strength and tensile strength of steel are almost same. But Tensile strength of concrete is about $(1/10)^{th}$ of compressive strength of concrete

→ more economical

Higher strength to weight ratio for steel compare to concrete. Tall structures, Large stand buildings, bridges etc., are therefore constructed with structural steel

→ Rapid construction.

Fabrication & Erection of steel can be done quickley

→ Easy Repair & modification.

→ more Ductile

Ductility is expressed in terms of % of elongation.

$$= \frac{\delta l}{L} \times 100$$

If % elongation → < 5% → Brittle → Glass, concrete.

→ 5-15% → Intermediate → Aluminum (Al)

→ > 15% → Ductile → steel.

For structural steel, % of elongation > 20%.

→ 100% ~~seap~~ (scrap) value.

(re usage)

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

→ overall construction cost is Lesser

material cost, man power, maintonance, dismantling etc., are cheaper for steel.

→ Better quality and reliability

	LSM	WSM
	concrete = 1.5	concrete < 3, 4
	steel = 1.15	steel → 1.78 ≃ 1.8

- Lesser values of Factor of safety (F.O.S), Partial safety factor (P.S.F), Load Factor (LF) for steel compare to concrete because steel is factory made product which has better quality control

Disadvantages (or) Limitations.

→ Lesser fire resistance

→ Lesser corrosion resistance

Iron [98%] + carbon content [0.1–1.1%] + { sulphure, mangnese, chromium, Nickel, phosphurus } → steel

∴ High carbon content ⇒ strength ↑ Ductility ↓

Nickel and chromium content is more in stainless steel which makes Better corrosion resistance

Note:- As carbon content increases, yield strength, ultimate strength and hardness increases But ductility and toughness decreases.

Properties of steel

1) Yield strength
2) ultimate strength.
3) Toughness
4) Hardness
5) Elastic modulus
6) poissons ratio
7) Coefficient of thermal expansion

1) classification of steel (Based on carbon content)

1) Low carbon steel → 0.1 to 0.25%. (Fe 250) (High bonding Property)

2) medium carbon steel → 0.25 – 0.68%. (Fe 415) Ex: Railway tracks

3) High carbon steel → 0.68 – 1.1%. (Fe 500) (Less Bonding property) Ex:- Drilling Bit.

Low carbon steel:-

→ It is used in RCC constructions as mild steel, as Reinforce material

→ It is used in steel Genral constructions as mild steel

Medium carbon steel

→ It is used as Rails, tyres etc.,

High carbon steel

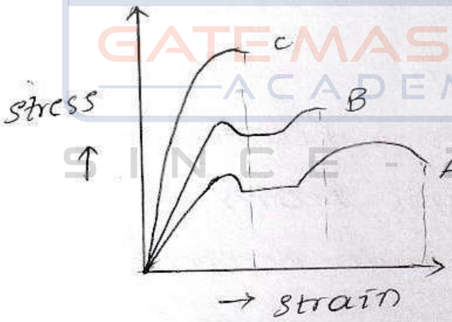
→ It is used as stone masonry tools, drills, punches etc.,

Note:-

→ Hardness must be lesser to improve weldability.

→ Ductility and toughness are very important for structural steel subjected to dynamic loading.

→ As toughness increases area under stress strain curve increases



(Because strain is increasing Ductility is more).

A → Low carbon steel

B → medium carbon steel

C → High carbon steel.

2) classification of steel based on purpose.

a) Reinforce steel → RCC

i) Fe – 250 $f_y = 250 MPa$

ii) Fe – 415

iii) Fe – 500

b) structural steel → steel structures.

i) Fe – 250 $f_y = 250\ mpa$

ii) Fe – 540

iii) Fe – 570

* For Fe -250 → yield strength (f_y) = 250MPa
ultimate strength (f_u) = 410MPa

3) Based on code

a) IS:226 → standard quality steel
b) IS: 2062 → fusion welding type.
c) IS : 961 → ordinary steel.

* standard steel sections (structural steel)

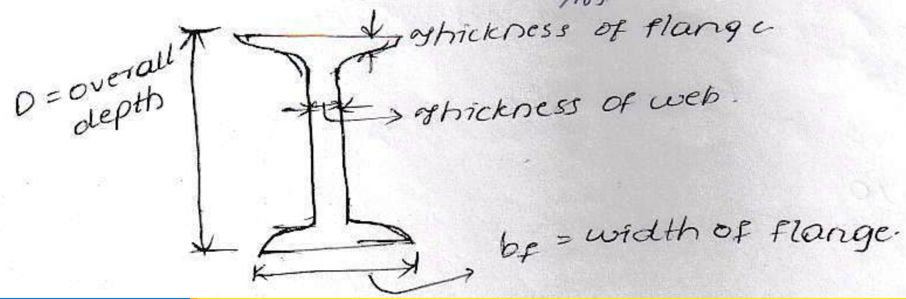
1) Indian std I-sections (Beam section)
2) channel sections
3) T-sections
4) Angle sections.
5) Indian standard tube sections. (Hollow section with less thickness)
6) Indian std steel plates
7) Indian std steel strips
8) Indian std steel flats
9) Indian std steel Rods
10) Indian std steel sheets.

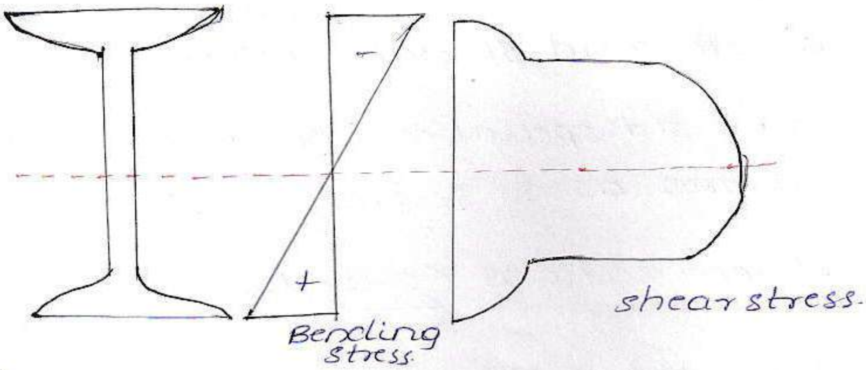
Rolled steel I-sections

1) ISJB - Indian std junior beam
2) ISLB - Indian std Right Beam
3) ISMB - Indian std medium beam
4) ISHB - Indian std Heavy beam
5) ISWB - Indian std wide flange beam

I-sections are designated as overall depth of & weight per running metre

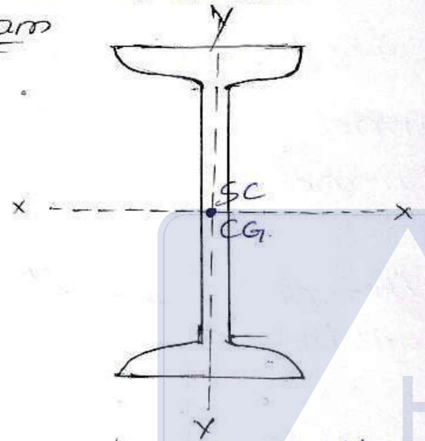
Ex:- ISMB 350 @ 720.2 N/m





→ Flanges will takecare of Bending whereas web will takecare of shear

I-Beam



S.C = shear centre

C.G = centre of gravity

Shear centre:- It is the point through which resultant Load should act to have no twisting in the section

→ If the given section is symmetric about both the axis then shear centre coincides C.G.

→ If a given section is symmetric about one axis only then shear centre lies on the axis of symmetry

I-Beam

→ when only transverse loads acts, Bending axis will be x-x axis. Therefore I_{xx} should be more compare to I_{yy}

→ To have more I_{xx}, depth of section is increased.

→ If Lateral Loads also acts on the section, there will be bending about y-y axis also. For this condition width of section is also increased.

ISWB

→ ISWB is preffered for members or sections subjected to Lateral Loads only (more width)

→ For members or sections subjected to transverse loads only ISMB is preffered.

Note :-

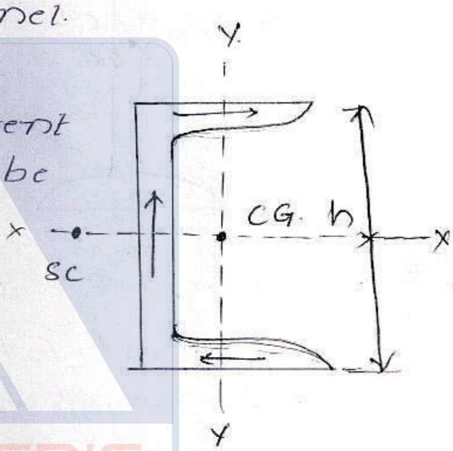
ISSC - Indian standard special circular column

→ ISSC is Indian std special column section which is used for column as I-section.

Rolled steel channel sections (C-section)

1) ISLC — Indian std light weight channel
2) ISMC — Indian std medium weight channel
3) ISMCP — Indian std medium weight will parallel plates channel.
4) ISGC — Indian std gate channel
5) ISJC — Indian std junior channel.

→ If the load acts at a point different from shear centre then there will be twisting in the section



→ shear stress, $q = \frac{V.A.\bar{y}}{I.b}$

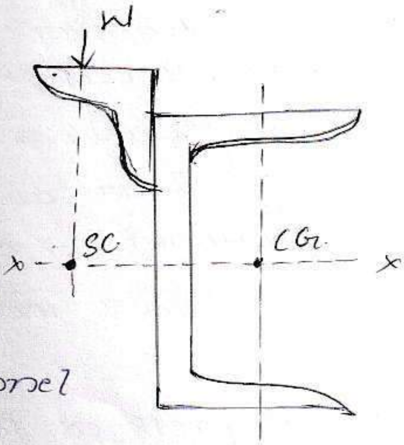
shear flow (or) force per 'm',

$Q = q.b$

$= \frac{V.A.\bar{y}}{I.b} \cdot b$

$\boxed{Q = \frac{V.A.\bar{y}}{I}}$ per 'm'

$\boxed{Q \propto \frac{1}{I}}$



→ If w acts through shearcentre.

→ The purpose of angle added to channel

i) To increase I_{xx}
ii) To increase I_{xy}
iii) To Reduce deflection (Buckling)
iv) Makes to allow load pass through shear centre.

→ channel sections are used for columns as built up sections also

→ for columns two channels placed back to back or face to face and connected by steel flats or angles

T- section

1) ISNT - Indian std Normal legged T-section

2) ISDT - Indian std Deep legged T-section

3) ISLT - Indian std Light weight T-section

4) ISMT - Indian std medium weight T-section

5) ISHT - Indian std Heavy weight T-section

Ex:- $\underbrace{\text{ISMT }325}_{\text{overall depth (mm)}}\text{@}\underbrace{472.1\text{ N/m}}_{\text{weight per m}}$

Angle - sections

1) un-Equal legged — Ex:- 70×60×6

2) Equal legged — 70×70×8

3) Bulb angle

→ used in ship buildings to prevent buckling.

Tabular sections

Hollow circular section (HCS) -

Hollow square section (HSS)

Hollow rectangle section (HRS)

→ For the members or sections subjected to torsion, Hollow sections are most economical sections.

Note

→ The best section or most economical sections for ~~co~~ solid circular over hollow circular sections because of uniform radius of gyration throughout steel plates (thickness $\geq$ 5mm)

ISPL 2000 x 1000 x 8

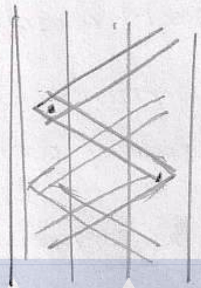
* steel sheets (t < 5mm)

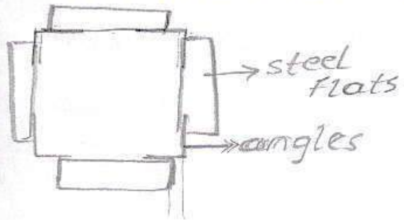
→ sheets are used as roof covering material

ISSH 2000 x 1000 x 4.

steel flats (These are used to Tie Ex: 

→ steel flats are used as lacings to connect individual elements





steel strips

→ These are used as a battens to connect individual sections.

→ steel bars or rods are used as a rivets, bolts etc.
it is designated by its diameter (Least lateral dimension)

Properties	steel	Aluminum
Density	$\rho_s = 7850\,kg/m^3$	$\rho_a = \frac{1}{3}\rho_s$
Elastic modulus	$E_s = 2\times10^5\,N/mm^2$,	$E_a \simeq \frac{1}{3}E_s$
coeff of thermal expansion	$\alpha_s = 12\times10^{-6}/^\circ C$	$\alpha_a \simeq 2\alpha_s$.

$$\left[\frac{\text{strength}}{\text{weight}}\right]_{steel} < \left[\frac{\text{strength}}{\text{weight}}\right]_{AL}$$

→ For Righter compressive loads aluminum section have more chances of buckling therefore aluminum is not used in civil constructions like buildings

→ Aluminum sections are used for aerospace constructio because weight is an important parameter

→ Aluminum sections are more corrosive resistant sections than steel sections., Hence, maintanence may lesser for allumimum sections.

Design methods

1) working stress method (WSM) → working loads are considered. (Deterministic approach)

$$F.O.S = \frac{\text{Yield stress}}{\text{working stress}}$$

2) ultimate strength method → ultimate (or) factored load. is considered. (probabilistic approach)

$$\text{load factor} = \frac{\text{ultimate load}}{\text{working load}}$$

3) Limit state method → Based on Factored (or) ultimate load & serviceability.

→ partial safety factor is used as safety norm

* Limit state method

It is the condition of the structure just before collapse is called limit state. (or)

→ It is an acceptable limit for the safety and serviceability requirements before failure occurs. is called limit state

→ It is based on probalistic approach

Note:-

In limit state method design loads are over estimated and design strength are underestimated.

Limit state of strength
1) strength
2) stability (against sliding) or over turning
3) Fracture
4) Brittle failure
5) plastic failure

L.S of serviceability
Deflection
vibration
corrosion
fire

Design Load, $(F_d) = \gamma \times F$

γ = F.O.S (or) P.S.F for Loads.

= 1.5 → Generally. → For structural steel (for loads)

For Reinforce 1.15

F = characteristic Load

characteristic load:

It is the load which has 95% probability of not been exceeded in the life time.

Design strength, $f_d = \frac{f}{\gamma_m}$

γ_m = P.S.F for material

f = characteristic strength

Characteristic strength: It is the strength below which 5% of test results are expected to fall

partial safety for material, γ_m

1) P.S.F for plate (yielding (or) buckling), $\gamma_{m_0} = 1.10$

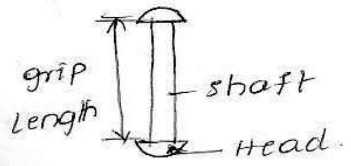
2) P.S.F for plate at (ultimate strength), $\gamma_{m_1} = 1.25$

3) For connection	workshop	Field
i) For rivets, γ_{mr}	1.25	1.25
ii) For bolts, γ_{mb}	1.25	1.25
iii) For welds, γ_{mw}	1.25	1.5

Load comination (steel & Rcc)

Load combination	L.S of strength			L.S of serviceability		
	DL	L.L	WL/EL	DL	L.L	WL/EL
DL + L.L	1.5	1.5	—	1.0	1.0	—
D.L + W.L/E.L	1.5 or 0.9	—	1.5	1.0	—	1.0
D.L+L.L+W.L/E.L	1.2	1.2	1.2	1.0	0.8	0.8

Rivetted connection



grip length $\not> 8d$

ϕ = dia of rivet

d = Gross dia of rivet

= Hole dia

$d = \phi + 1.5mm$, when $\phi \le 25mm$

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

Classification of rivets

1) Based on manufacturing process

a) cold driven rivets

b) Hot driven rivets

cold driven rivets have more strength

2) Based on driving technique

a) power driven rivets

b) Hand driven rivets

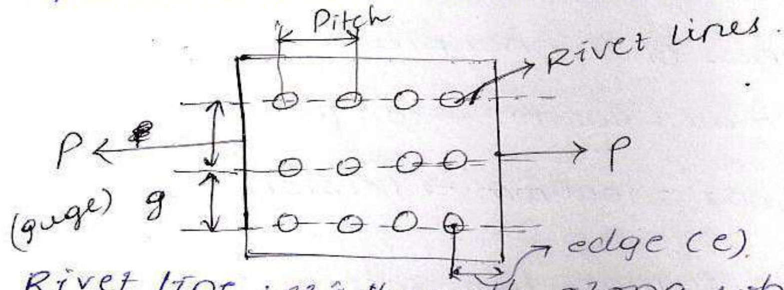
power driven rivets have more strength

3) Based on place

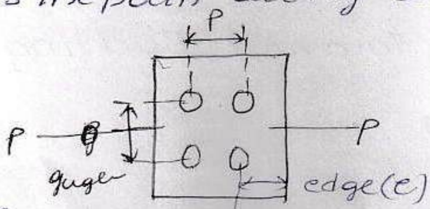
a) shop driven rivets

b) field driven rivets

shop driven rivets have more strength



Rivet line : It's the path along which rivets are placed.



Pitch :- centre to centre (C/c) distance two consecutive rivets measured parallel to applied loads is called pitch guage:- c/c distance b/w to consecutive rivets, measured perpendicular to the applied load

Edge distance:- It is the distance from centre of last bolt to the corresponding edge.

Specifications (codal provisions)

1) Edge distance (e)

a) minimum edge distance, (e_{min})

$e_{min} = 1.5d \rightarrow$ machine cut

$e_{min} = 1.7d \rightarrow$ Hand cut

d = hole diameter

b) max edge distance

$e_{max} = 37mm + 4t$

t = thickness of thinner plate.

2) pitch (P)

a) min pitch (P_{min}) = 2.5ϕ

ϕ = dia of rivet or nominal dia of rivet.

b) max pitch (P_{max})

i) For plates

$P_{max} = 12t$ (or) 200mm, whichever is less $\rightarrow$ compression

$P_{max} = 16t$ (or) 200mm, whichever is less $\rightarrow$ tension.

$P_{max} = 32t$ (or) 300mm, whichever is less for tacking rivets.

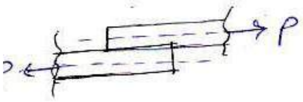
t = thickness of thinner plate.

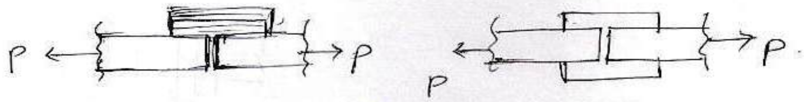
ii) For angles, $P_{max} = 600mm \rightarrow$ compression

$P_{max} = 1000mm \rightarrow$ tension

Tacking:- it is used to make two sections act together and in compression to avoid Buckling

Joints
- Lap joint
- Butt joint
  - Single cover
  - Double cover





thickness of cover plate = 1.25t

- Lap joint, single cover Butt joint has eccentricity effect, causes bending
- Thickness of cover plate in case of single cover Butt joint is $1.25t$ (or) $(5/4)t$
- Double cover Butt joint has no eccentricity effect, causes no bending
- Thickness of each cover plate in case of double cover Butt joint is $\frac{1.25t}{2}$ (or) $\frac{5}{8}t$

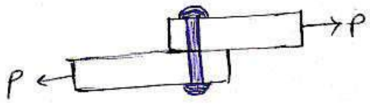
Assumptions in Rivetted connections.

- Bending stress in Rivets is neglected
- Friction b/w plates is neglected
- Rivet hole is assumed to be completely filled by the rivet
- Shear stress is assumed to be uniformly distributed over its gross area of rivet
- A load is assumed to be uniformly distributed among all the rivets.

Types of failure

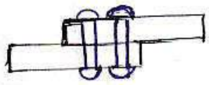
i) Shear failure of rivet
ii) Bearing /crushing failure of rivet
iii) Shear failure of plate
iv) Bearing /crushing failure of plate
v) Tearing failure of plate
vi) Splitting failure of plates

single rivetted lap joint



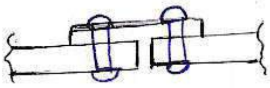
$n = 1$

Double rivetted lap joint.



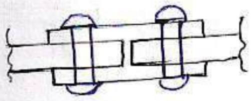
$n = 2$.

single rivetted single cover



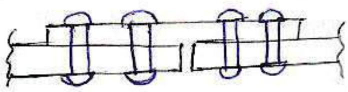
$n = 1$.

single ~~Double~~ rivetted, Double ~~single~~ cover butt joint



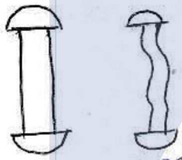
$n = 2$

Double rivetted single cover butt joint



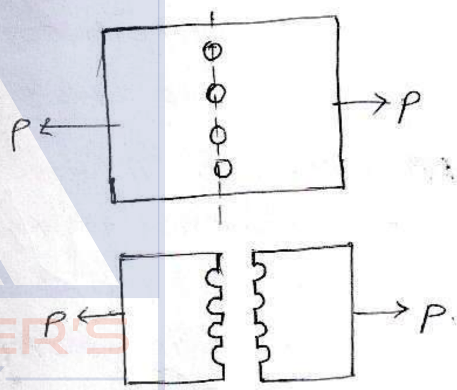
$n = 2$.

Bearing failure.

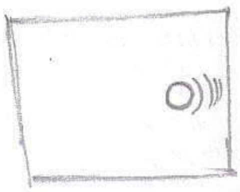


Before failure After failure

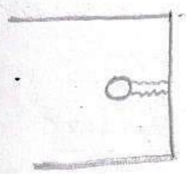
Tearing failure of plate



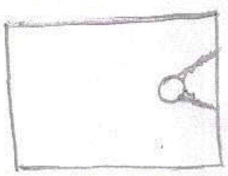
Bearing failure of plate



shear failure of plate



splitting failure of plate



→ By providing sufficient edge distance, the following failures can be eliminated 1) shear failure of plate 2) Bearing failure of plate 3) splitting failure of plate shear strength of rivet, $P_s = \frac{\pi}{4} d^2 \times \tau_{vf}$

d = dia of the whole

τ_{vf} = permissible shear stress in rivet

$= 0.4 f_y$

$= 100 MPa$ for $Fe = 250$

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

n = no. of planes

Bearing strength of rivet, $P_b = d.t.\sigma_{tf}$

σ_{tf} = Permissible bearing stress in rivet

$= 1.2 f_y$

$= 300 MPa$ for $Fe - 250$

t = thickness of thinner plate → (in Lap joint)

t = smaller of thinner main plate & sum of cover plate thickness → (Butt joint)

→ Rivet value, R_v (strength of rivet) = smaller of P_s & P_b

No. of Rivets required

$$N = \frac{\text{Axial load}}{\text{Rivet value}}$$

$$\boxed{N = \frac{P}{R_v}}$$

* Tearing / tensile strength of plate (P_t)

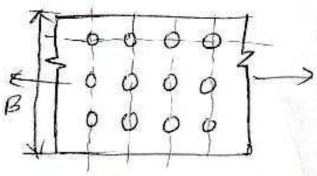
$P_t = A_{net} \times \sigma_{at}$

σ_{at} = Permissible tensile stress in plate

$= 0.6 f_y$

$= 150 MPa$ → Fe-250

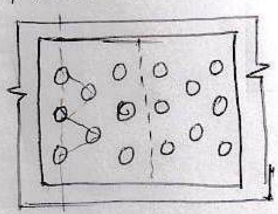
chain pattern



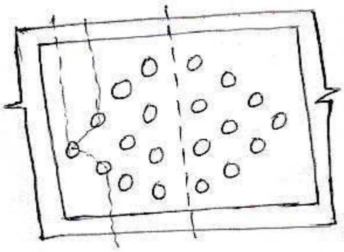
$A_{net} = Bt - n.d.t$

$= (B - nd) t$

staggerd pattern



Diamond pattern



$$A_{net} = \left[B - nd + \sum_{i=1}^{n} \frac{P_i^2}{4g_i}\right] t \longrightarrow \text{Stagged or Zig zag Pattern}$$

→ Efficiency of diamond pattern is more than stagged pattern

→ chain pattern has least efficiency

strength of joint, or connection, P_c = smaller of P_s, P_b & P_t

$$\text{Efficiency}, \eta = \frac{\text{strength of connection}}{\text{Solid strength of plate}} \times 100$$

$$\boxed{\eta = \frac{P_c}{P_{sp}} \times 100}$$

solid strength of plate, $P_{sp} = A_g \times \sigma_{at}$

A_g = gross area = $B.t$

$\sigma_{at} = 0.6 f_y$

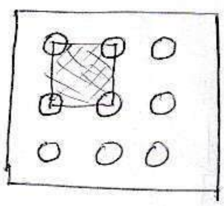
If $P_c = P_t$

$$\eta = \frac{P_t}{P_{sp}} \times 100$$

$$\eta = \frac{A_{net} * \cancel{\sigma_{at}}}{A_g \cdot \cancel{\sigma_{at}}} \times 100$$

$$\eta = \frac{[B-nd]\,t}{B.t} \Rightarrow \boxed{\eta = \frac{B-nd}{B} \times 100}$$

Per pitch



If width is not given pitch area is considered

Per pitch, $A_{net} = [P-d]\,t$

$A_g = P.t$

$$\boxed{\eta = \frac{P-d}{P}}$$

Nominal diameter of rivet based on unwin's formula

$$\phi = 6.05\sqrt{t}$$

t = thickness of thinner plate (mm)

permissible stresses in shop driven rivets

method	shear (MPa)	Bearing MPa
Power driven	100	300
Hand driven	80	250

→ For field driven rivets the above permissible values are reduced by 10%.

→ The above permissible values increased by 25% if wind or earthquake effect is considered in addition to dead Load & Live load.

Bolted connection (LSM)

Types of bolts

1) Black (or) unfinished bolt

2) High strength friction grip (HSFG) bolts. (or) Finished bolts

Black Bolts

→ These bolts are reffered to as ordinary Bolts

→ They are least expensive Bolts.

→ These Bolts are used in Light construction under static loads such as, small trusses, Bracings etc.

→ These bolts are not recommended for connections subjected to impact, vibration & fatigue loads (continuous loads)

→ For bolt of a property clause 4.6 represents, ultimate strength of bolt as 400MPa, yield strength of bolt 200MPa

4.6 → represents $\frac{f_{yb}}{f_{ub}} = 0.6$

represents $f_{ub} = 400MPa$

High strength friction grip (HSFG) Bolts:-

→ These Bolts are made from bars of medium carbon steel

→ The Bolt property clause 10.9S, 12.9S are commonly used.

→ These Bolts are available in sizes 16mm to 36mm and generally designated as M16, M18, M20 ---etc.

m stand for steel — M16 → dia of bolt in mm

it is made up of medium carbon.

→ These bolts are used for all general construction, including for dynamic loads in case of bridges d = nominal dia of bolt.

d_o (or) d_h = gross dia of bolt

= Dia of hole

d_h = d + 1mm, if dia = 12mm to 14mm

= d + 2.0mm, dia = 16mm to 24mm

= d + 3.0mm, dia > 27mm.

Note:-

For bolts in LSM and for Rivets in WSM will have same pitch, edge distance specifications except maximum edge distance.

min value of these two

max edge distance. $e_{max} = [40mm + 4t] \& 12 t \epsilon$

epsilon

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

t = thickness of thinner plate

Types of failures in bolted connection.

1) Shear failure of bolt.
2) Bearing /crushing failure of bolt
3) Tension failure of bolt
4) Shear failure of plate
5) Bearing failure of plate
6) Block shear failure

Design shear strength of bolt

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

$\gamma_{mb} = 1.25$

$$V_{nsb} = \frac{f_{ub}}{\sqrt{3}}\left[n_n A_{nb} + n_s A_{sb}\right]$$

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

A_{nb} = net area of bolt $\Rightarrow \simeq 0.78 \frac{\pi}{4} d^2$

n_n = no. of shear planes with intercepting shear planes

n_s = no. of shear planes without intercepting shear planes

for lap joint (or) single cover butt joint

$n_n + n_s = 1$

if $n_n = 0$, $n_s = 1$

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

for double cover butt joint

$n_s + n_n = 2$

if $n_n = 1$, $n_s = 1$

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

if $n_s = 0, n_n = 1$

$$V_{nsb} = \frac{f_{ub}}{\sqrt{3}}\left[n_n \cdot A_{nb} + \overset{0}{\cancel{n_s}} \cdot A_{sb}\right]$$

$$\boxed{V_{nsb} = \frac{A_{nb} f_{ub}}{\sqrt{3}}}$$

$$\boxed{V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}} = \frac{A_{nb} f_{ub}}{\sqrt{3} \cdot \gamma_{mb}}}$$

→ Nominal shear strength of the bolt will be reduced for long joints, long grip, packing plate

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

β_{lj} = reduction coeff. for long joint

β_{lj} = reduction coeff for long rip

β_{kg} = reduction coeff for packing plate.

→ In long joints if the distance from 1st bolt to last bolt exceeds 15d in the direction of load then β_{lj} is considered

$$\beta_{lj} = \left[1.075 - \frac{l_j}{200d}\right]$$ Here, l_j = long joint

* when grip length exceeds 5d then it is called long grip and β_{lg} is considered

$$\beta_{lg} = \frac{8d}{3d + l_g}$$

l_g = grip length

→ when thickness of packing plate exceeds 6mm β_{pkg} is considered

$\beta_{kg} =$

t_{pkg} = thickness of packing plate in mm

Design Bearing strength of Bolt (V_{db})

$$V_{db} = \frac{V_{nb}}{\gamma_{mb}}, \quad V_{nb} = \text{Nominal bearing strength of Bolt}$$

$$= 2.5\, k_b \cdot d \cdot t \cdot f_u'$$

k_b = smaller of i) $\frac{e}{3d_h}$ (ii) $\frac{p}{3d_h} - 0.25$ (iii) $\frac{f_{ub}}{f_u}$ & (IV) 1.0

e = edge distance, p = pitch

f_u' = smaller of f_u & f_{ub}

f_u = ultimate strength of plate

Design ~~stee~~ tensile strength of bolt

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

∵ for 4.6
$f_{ub} = 400$
$f_u = 410$
when $f_e = 250$

$A_{nb} \simeq 0.78 A_{sb}$

$\gamma_{m0} = 1.10$, f_{yb} = yield strength of Bolt.

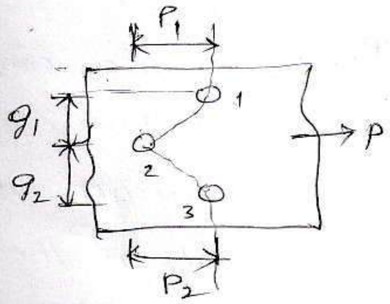
Design strength of Bolt (V_d) :-

= Smaller of V_{dsb}, V_{db} & T_{db}

No. of Bolts regard, $N = \frac{\text{Factored axial load}}{\text{Design strength of bolt}}$

$$\boxed{N = \frac{P_d}{V_d}}$$

Design strength of plate (T_{dp})



$$T_{dp} = \frac{T_{np}}{\gamma_1} = \frac{0.9 f_u \cdot A_{net}}{\gamma_{m1}}$$

A_{net} = $[B - n d_h] t$ → chain pattern

= $[p - d_h] t$ → per pitch → chain pattern

= $\left[B - n d_h + \sum_{i=1}^{n} \frac{p_i^2}{4 g_i}\right] t$ → zig zag

Design strength of joint/connection, V_{dc}.

V_{dc} = smaller of V_{dsb}, V_{db}, T_{db} & T_{dp}

efficiency, $\eta = \frac{\text{Design strength of connection}}{\text{Design strength of solid plate}} \Rightarrow \eta = \frac{V_{dc}}{T_{dsp}}$

Design strength of solid plate

$$T_{dsp} = \frac{T_{nsp}}{\gamma_{m_1}} = \frac{0.9 f_u A_g}{\gamma_{m_1}}$$

A_g = gross area = $B \cdot t$

if $V_{dc} = T_{dp} \Rightarrow \eta = \frac{T_{dp}}{T_{dsp}} \times 100$

$$\eta = \frac{\frac{0.9 f_y \cdot A_{net}}{\gamma_{m_1}}}{\frac{0.9 f_u A_g}{\gamma_{m_1}}}$$

$$\eta = \frac{A_{net}}{A_g} \times 100$$

$$\eta = \frac{A_{net}}{A_g} \times 100 \Rightarrow \eta = \left[\frac{B - n d_h}{B}\right] \times 100$$

$$\eta = \left[\frac{P - d_{(h)}^{\text{hole}}}{P}\right] \times 100$$

Permissible stress
↑
working stress method – Rivets, bolts

LSM – Bolts.
↓ yield stress
depending

Sol

1) thickness of cover plate does not consider in shear strength of rivet so, Ans, B

2) $P = 700\text{kN}$
$P_s = 60\text{kN}$ } Least = 35
$P_b = 35\text{kN}$ }
$P_t = 70\text{kN}$
$N = \frac{P}{R_v} = \frac{700}{35} = 20$

3) $P = 840\text{kN}$, $P_s = 50\text{kN}$, $P_b = 80\text{kN}$

$\frac{840}{50} = 16.8 \simeq 17$

4) $P_t = A_{net}\,\sigma_{at}$

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

of $nd \uparrow \rightarrow A_{net} \downarrow$

Ans sec ③-③

5) $R_v = ?$

Smaller of P_s & P_b

$\tau_{vf} = 90\text{MPa}$, $\sigma_{pf} = 270\text{MPa}$

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

$\phi = 16\text{m}$, $d = 16 + 1.5 = 17.5\text{m}$

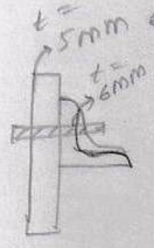
$P_s = \frac{\pi}{4}(17.5)^2 \times 90 = 21.64\text{kN}$

for double shear $= 2 \times 21.64$
$= 43.29\text{kN}$

$P_b = d \cdot t\,\sigma_{pf} = 17.5 \times 12 \times 270$
$= 56.7\text{kN}$

$R_v = 43.29\text{kN}$

6)



Note :- Gusset plate will considered individual members to transferse load.

$P_b = d \cdot t \cdot \sigma_{pf}$
$= 16 \times 5 \times 250 = 250\text{kN}$

7) $\tau_{vf} = 100\,MPa$

$\sigma_{tf} = 300\,MPa$

R_v = smaller P_s & P_b

$P_s = 2 \times \frac{\pi}{4} d^2 \tau_{vf}$ — clearence = 1.5

$= 2 \times \frac{\pi}{4} (21.5)^2 \times 100$ — so, 20+1.5 = 21.5

$= 72.6\,kN$

$P_b = d \cdot t \cdot \sigma_{tf}$

$= 21.5 \times 14 \times 300$

$= 90.3\,kN$

{ main plate = 14mm
cover plate each one = 10mm }

$2 \times 10 = 20mm$

Least of two = 14mm

$t = 14mm$

$R_v = 72.6\,kN$

$= \underset{\text{no. of rivets}}{12} \times 72.6$

$R_v = 871.3\,kN$

8) $A_{net} = \frac{p^2}{4g}$

If pitch is more load carrying has more

$p^2 > 4gd$

Level-1

13) Permissible bearing stress,

$\sigma_{tf} = 1.2 f_y$

$\frac{f_y}{\sigma_{tf}} = \frac{1}{1.2}$

$= \frac{10^5}{12_6} = 0.83$

16) M16 x 70NL — Bolt head (nouse)

dia = 16mm.

17) $4.6 \longrightarrow f_{yb} = 0.6 \times 400$

$= 240\,MPa$

$f_{ub} = 400\,MPa$

Level-2

1) $A_{net} = [B - nd]t$

$[300 - 1 \times 19.5] \times 10$

$= 2805\,mm^2$

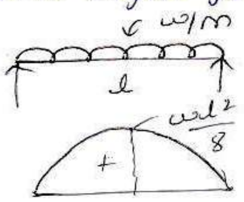
$= 28.05\,cm^2$

3) WELDED CONNECTION

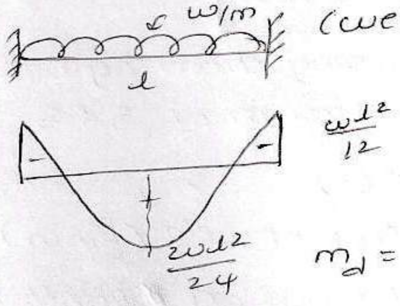
Advantages

- → Since no holes are formed, it has 100% Efficiency overall weight of joint is reduced ~~it~~
- → It has speed of fabrication
- → complete rigid joints can be achieved.

(Riveted/bolted)



$m_d = \frac{wl^2}{8}$



(welded)

$m_d = \frac{wl^2}{24}$

Disadvantages

- → welded joints are more brittle and their fatigue strength is less
- → Internal stress and warping stresses are developed
- → Defects like internal air pockets and incomplet penetration is developed

Classification welding

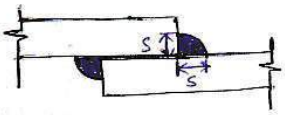
1) Fusion process.
   a) Gas welding
   b) metal arc welding (or) Electric arc welding
2) pressure process
   a) Forge welding
   b) Electrice resistance welding

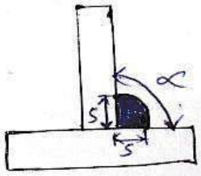
- → Generally, metal arc welding is used for structural steel
- → Flux coating controls the melting of electrode.
- → By adding some elements to the flux, mechanical properties of the joint can be improved

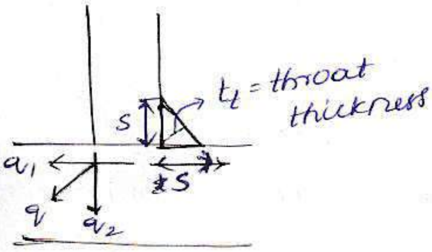
Types of welded joints

1) Lap weld (or) Fillet weld
2) Butt weld (or) Groove weld
3) Slot and plug welds

Fillet weld







s = size of weld.

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

α = angle b/w fusion faces

$$\boxed{60^\circ < \alpha < 120^\circ}$$

→ while determining shearing area of fillet weld, take size of weld as minimum of S_1 & S_2

Size of weld (s)

1) min size of weld (S_{min})

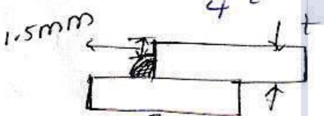
↳ It depends on ~~thinner~~ thickness of plate. thicker

S_{min}	t
3mm	≤10mm
5mm	10 – 20mm
6mm	20 – 32mm

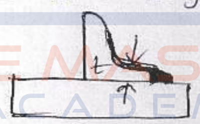
max size of weld (S_{max})

$S_{max} = t - 1.5mm$ → for square edges.

$S_{max} = \frac{3}{4}t$ → For rounded edge.



square



Rounded.

→ Fillet weld always fails along throat only because min shear area occurs along throat and throat thickness is always smaller than size of weld. ($t_t < s$)

Throat thickness

$t_t = k.s$

s = size of weld

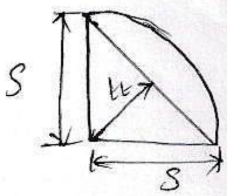
k = a const, depends on angle α

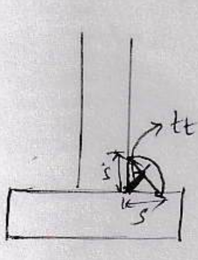
α = angle b/w fusion faces

α = 60° to 120°

k. values

α	k
60° – 90°	0.70
91° – 100°	0.65
101° – 106°	0.60
107° – 113°	0.55
114° – 120°	0.50





Effective Length

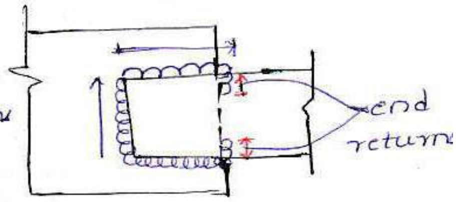
It is the actual length of the fillet weld. shown in the diagram

Total length of weld = L

Effective length of weld = L_w

L_w = Total length of weld − 2×End returns

$L_w = L - 25$



→ End returns must be provided for the welded connections subjected to dynamic loads and also in case of eccentric loads To reduce the stress concentration at the ends.

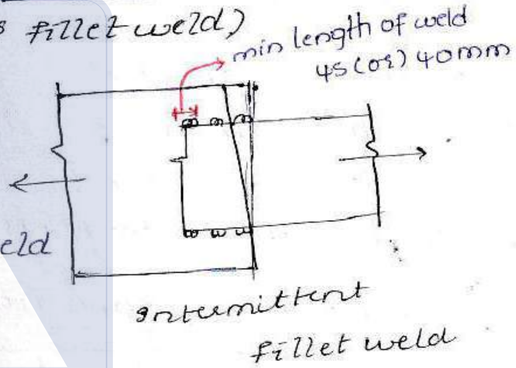
→ min effective length is 4S (or) 40mm whichever is maximum

→ The min overlap length should not be less than 5 times thickness of plate in case of WSM

→ In case of LSM min overlap length $\boxed{\not< 4t \text{ (or) } 40\text{mm}}$ whichever is max

* Intermittent fillet weld (Discontinuous fillet weld)

It is provided when required length of weld is less than available length of weld



→ clear spacing b/w intermittent fillet weld should not exceed $\not>$

i> 16 t (or) 200mm → tension

ii> 12 t (or) 200mm → compression.

Shear strength of weld (WSM)

P_w = Area of weld × permissible shear stress in weld

$P_w = A_w \times \tau_{vf}$

τ_{vf} = permissible shear stress in weld.

$= 0.45 f_y$

$$\boxed{\tau_{vf} = 108 - 110 \text{ MPa}}$$

$A_w = L_w \cdot t_t$

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

Design shear strength of fillet weld [LSM]

$$P_{dw} = \frac{P_{nw}}{\gamma_{mw}}$$

P_{nw} = Nominal shear strength of weld

γ_{mw} = Partial saftey factor weld

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

f_{wd} = design shear capacity of fillet weld

$$= \frac{f_u}{\sqrt{3}\,\gamma_{mw}}$$

~~Grooveweld~~

Groove weld (or) Butt weld

single type of weld :-

- It is also called as partial penetration weld
- single V, single U, single J, single Bewel are the example of single type of weld
- Effective throat thickness $\boxed{t_e = \frac{5}{8} t}$

$\hat{V}$

Double type of weld

- It is also called as full penetration weld double V, double U, double J, double bewel
- Effective throat thickness $\boxed{t_e = t}$

Load carrying capacity of groove weld

$$\boxed{P = L_w \cdot t_e \cdot \sigma_{tf}} \longrightarrow WSM$$

σ_{tf} = permissible bearing stress in weld.

$= 0.6 f_y$

$= 150 MPa$

Design strength of groove weld → (LSM)

$$V_{dw} = \frac{L_w \cdot t_e \cdot f_{yw}'}{\gamma_{mw}}$$

f_{yw}' = smaller of $\frac{f_{yw}}{\sqrt{3}}$ (or) $\frac{f_u}{\sqrt{3}}$.

f_{yw} = yield strength of weld

f_u = ultimate strength of parent metal

- In WSM the permissible stresses are reduced by 20% for field or site welding
- The above permissible stresses are increased by 25% if wind or earth quake effect acts in addition to gravitational effect

Slot and plug weld

These are used when available length of weld is lesser than the required length of weld.

The diameter and width of the ~~weld~~ slot ≮ 3 times the thickness of connected plate or 25mm which ever is max.

→ LSM fillet

→ LSM the design shear strength of fillet weld is reduced by β_{lw} for long joint of weld

→ If length of weld along the direction of load exceeds 150 times throat thickness then reduction co-efficient β_{lw} is considered

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

L_j = length of weld joint

Pg

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

Pg - 23

1) $s = 6mm$

$\tau_{vf} = 108 MPa$

$L_w = 100 \times 2 + 50 = 250mm$

$t_t = k \cdot s$

$= 0.7 \times 6 = 4.2mm$

strength of fillet weld, $P_w = L_w \cdot t_t \cdot \tau_{vf}$

$= 250 \times 4.2 \times 108$

$= 113.4 KN$

2) $s = 8m$, $\tau_{vf} = 110 MPa$

$t_t = k \cdot s = 0.7 \times 8 = 5.6mm$

$L_w = 80 \times 2 + 60 = 220mm$

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

$= 135 KN$

3) $s = 6mm$

$\tau_{vf} = 108 MPa$

$L_w = 60 \times 2 + 50$

$= 170mm$

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

$t_t = 0.7 \times 6 = 4.2mm$

$P_w = 77 KN$

4) $P = L_w \cdot t_e \cdot \tau_{tf}$

$\tau_{tf} = 150 MPa$

$L_w = 150 mm$

for single type

$t_e = \frac{5}{8} t$

$= \frac{5}{\cancel{8}_2} \times \cancel{12}^3 = 7.5mm$

$P = 150 \times 7.5 \times 150$

$\tau_{tf} = 168 KN$

5) Load carrying capacity

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

strength of plate is tension

$P_t = A_{g/net} \cdot \sigma_{at}$

$A_g = 100 \times 10 = 1000mm^2$

$\sigma_{at} = 0.6 f_y$

$= 150 MPa$

for design strength of weld $P_w \leq P_t$

$\rightarrow P_w = P_t$

$150 \times 10^3 = L_w \times 0.75 \times \tau_{vf}$

for square edge $s = t - 1.5$ (10-1.5)

$\frac{150 \times 10^3}{0.7 \times (10 - 1.5) \times 110} \Rightarrow L_w = 229.18 mm$

6) $P_{dw} = \dfrac{L_w \cdot t_t \cdot f_u}{\sqrt{3} \cdot \gamma_{mw}}$

To find length of weld in each side.

design load = $\dfrac{270}{2} = 135\,kN$

$P_{dw} = 135\,kN = 135 \times 10^3\,N$

$$135 \times 10^3 = L_w \times 0.7 \times 10 \times \frac{410}{\sqrt{3} \times 1.25}$$

$L_w = 101.8$ → Higher multiple value

$\simeq 105$

7) $\dfrac{M}{I} = \dfrac{f}{y} = \dfrac{E}{R}$

Eccentric connections.

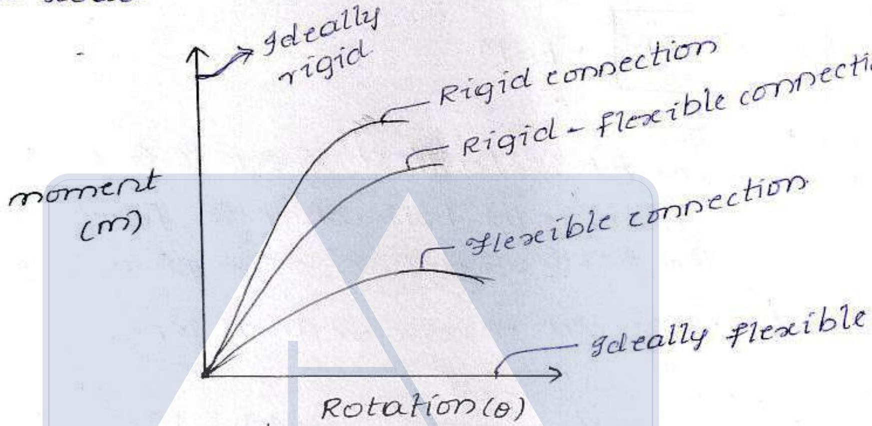
concentric Load

A Load is considered as concentric when its Line of action passes through C.G of rivet group or Bolt group or welding group

eccentric Load

A Load is considered as eccentric Load when its Line of action does not pass through C.G of Bolt group or rivet group or welding group

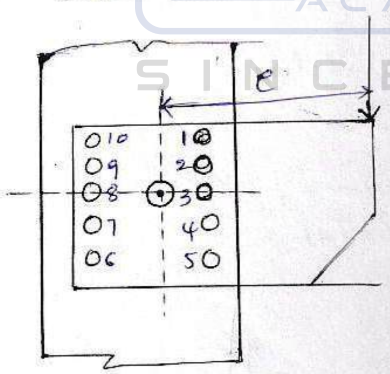
→ Because of eccentricity moment will be developed in addition to concentric Load

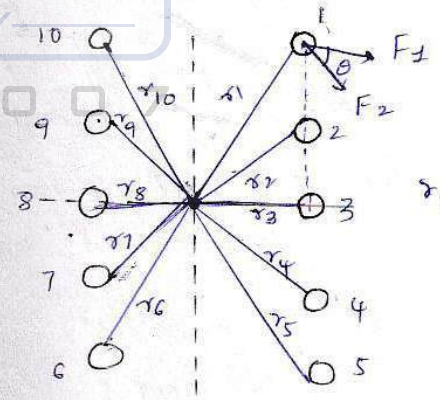


Types of eccentric connections

1) Bracket type - I
2) Bracket type - II

Bracket type - I





$r_1 = \sqrt{x_1^2 + y_1^2}$

→ These connection is subjected to direct shear force and Torsional moment

→ shear force on each bolt due to direct Load, $\boxed{F_1 = \frac{P}{n}}$

n = no. of bolts/rivets

Force on each bolt due to moment / torsion $\boxed{F_2 = \frac{M \cdot r}{\Sigma r^2}}$

For critical bolt, $\boxed{F_2 = \frac{M \cdot r_n}{\Sigma r^2}}$

$$r_n = \sqrt{x_n^2 + y_n^2}$$

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

$$r_1^2 = x_1^2 + y_1^2$$

$$r_n^2 = x_n^2 + y_n^2$$

Resultant force on critical bolt [max force]

$$F_R = F_{max} = \sqrt{F_1^2 + F_2^2 + 2F_1F_2\cos\theta}$$

For safety

$$\boxed{F_R \le R_V} \rightarrow WSM$$

$$\boxed{F_R \le V_{db}} \rightarrow LSM$$

Critical Bolt :-

- It is the Bolt which is farthest from C.G of Bolt Group
- critical most bolt is the bolt which is farthest from C.G and near to the line of action of applied load. (5th Bolt)

no. of Bolts required in one vertical line.

$$N = \sqrt{\frac{6M}{m \cdot p \cdot R_V}} \rightarrow W.S.M$$

m = no. of vertical lines.

p = pitch

R_V = smaller of P_s & P_b

$$N = \sqrt{\frac{6M}{m \cdot p \cdot V_{db}}} \rightarrow L.S.M$$

V_{db} = strength of bolt

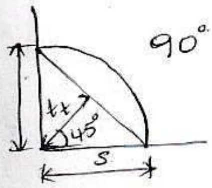
= smaller of V_{dsb}, V_{dpb} & T_{db}

e.w

9) $t_t = s\cos 45°$

$t_t = 0.707\,s$

$t_t = k.s$. $k = 0.7 \rightarrow$ for $60° - 90°$

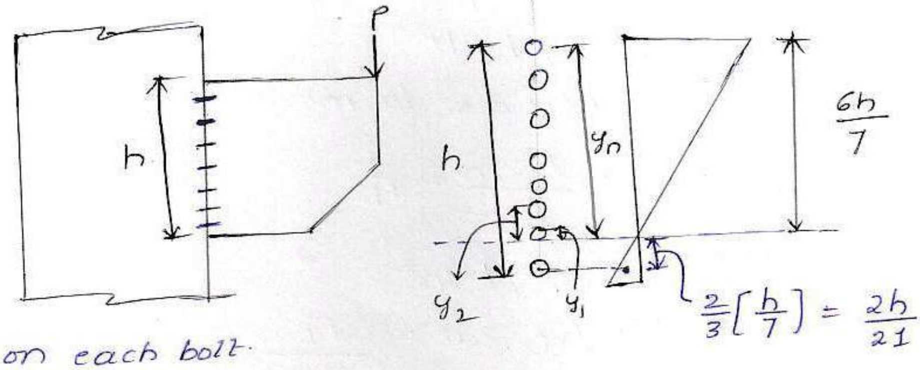


$$\frac{s}{t_t} = \frac{\sqrt{2}}{1}$$

$$\frac{t_t}{s} = \frac{1}{\sqrt{2}}$$

Bracket type - II

case - I : when neutral axis (N.A) passes through $h/7$



Tensile force on each bolt.

$$F_{a,\,calculated} = F_1 = \frac{P}{n}$$

n = no. of bolts.

Force on each bolt due to moment

$$T_{m,\,calculated} = F_2 = \frac{M' y_n}{\Sigma y^2}$$

$$M' = \frac{M}{1 + \frac{2h}{21} \cdot \frac{\Sigma y}{\Sigma y^2}}$$

$$\Sigma y = y_1 + y_2 + \dots + y_n$$

$$\Sigma y^2 = y_1^2 + y_2^2 + \dots + y_n^2$$

Tensile stress, $\tau_{vf,\,calculated} = \frac{F_1}{\frac{\pi}{4} d^2}$

$$\sigma_{tf,\,cal} = \frac{F_2}{\frac{\pi}{4} d^2}$$

check for combined stresses

$$\frac{\tau_{vf,\,cal}}{\tau_{vf}} + \frac{\sigma_{tf,\,cal}}{\sigma_{tf}} \leq 1.40 \rightarrow WSM$$

τ_{vf} = permissible shear stress in rivet.

σ_{tf} = permissible tensile stress in rivet.

$$\frac{F_{a.\,calculated}}{F_a} + \frac{T_{m\,cal}}{T_m} \leq 1.40$$

F_a = shear force in rivet

T_n = Tensile force in rivet

$$\left[\frac{V_b}{V_{db}}\right]^2 + \left[\frac{T_b}{T_{db}}\right]^2 \leq 1.0 \text{ — L.S.M}$$

V_{db} = Design shear strength of bolt

T_{db} = Design tensile strength of bolt case II : when N.A passes through centre.

$$F_{a,cal} = F_1 = \frac{P}{n} = V_b.$$

n = no. of bolts

force on each bolt due to moment

$$T_{m,cal} = F_2 = \frac{M \cdot y_n}{\Sigma y^2} = T_b.$$

$$\Sigma y^2 = y_1^2 + y_2^2 + \cdots + y_n^2$$

Tensile stress, $\tau_{vf,\,cal} = \dfrac{F_1}{\frac{\pi}{4} d^2}$

$$\sigma_{tf,\,cal} = \frac{F_2}{\frac{\pi}{4} d^2}$$

check for combined stresses

$$\frac{\tau_{vf,cal}}{\tau_{vf}} + \frac{\sigma_{tf,cal}}{\sigma_{tf}} \le 1.40 \longrightarrow W.S.M$$

τ_{vf} = permissible shear stress in rivet

σ_{tf} = permissible tensile stress in rivet

$$\frac{F_{a,cal}}{F_a} + \frac{T_{m,cal}}{T_m} \le 1.40$$

F_A = shear force in rivet

T_m = tensile force in rivet

$$\left[\frac{V_b}{V_{db}}\right]^2 + \left[\frac{T_b}{T_{db}}\right]^2 \le 1.0$$

V_{db} = Design shear strength of bolt

T_{db} = Design tensile strength of bolt

c.w

Q) type-1

$$F_1 = \frac{P}{n} = \frac{0}{4} = 0$$

$$F_2 = \frac{M \cdot r_n}{\Sigma r^2}$$

$$r_n = \sqrt{x_n^2 + y_n^2} = \sqrt{50^2 + 50^2} = 50\sqrt{2}$$

$$\Sigma r^2 = r_1^2 + r_2^2 + r_3^2 + r_4^2 = 4[50^2 + 50^2] = 8 \times 50^2$$

$$F_2 = \frac{50 \times 10^6 \times 50\sqrt{2}}{8 \times 50^2}$$

$$= \frac{\cancel{100} \times 10^4 \times \sqrt{2}}{\cancel{8}}$$

$$= 12.5 \times 10^4 \times \sqrt{2}$$

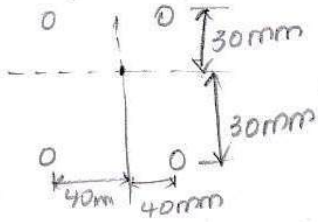
$$= 12.5 \times 1.4 \times 10^4$$

$$= 175.8 \text{ kN}$$

$$F_R = F_{max} = \sqrt{F_1^2 + F_2^2 + 2F_1F_2\cos\theta}$$

$$F_R = F_2 = 175.8 \text{ kN}$$

3)



$F_1 = \frac{P}{n} = \frac{10}{4} = 2.5\,kN$

$F_2 = \frac{M r_n}{\Sigma r^2} = \frac{P.e\, r_n}{\Sigma r^2}$

$r_n = \sqrt{40^2+30^2} = 50mm$

$\Sigma r^2 = r_1^2+r_2^2+r_3^2+r_4^2$

$= 50^2+50^2+50^2+50^2 = 4\times50^2$

$F_2 = \frac{10\times100\times50}{4\times50^2} = 5kN$

$\cos\theta = \frac{3}{r} = \frac{40}{50} = \frac{4}{5}$

$F_{max} = F_R = \sqrt{F_1^2+F_2^2+2F_1F_2\cos\theta}$

$F_{max} = \sqrt{2.5^2+5^2+2\times2.5\times5\times\frac{4}{5}}$

$= \sqrt{2.5^2+5^2+20}$

$= \sqrt{6.25+25+20}$

$= 7.15\,kN$

6) type-II

$F_{a,cal} = \frac{P}{n} = \frac{P}{4}$

$\tau_{m,cal} = \frac{M\cdot y_n}{\Sigma y^2} = \frac{P\times100\times50}{4\times50^2} = \frac{P}{2}$

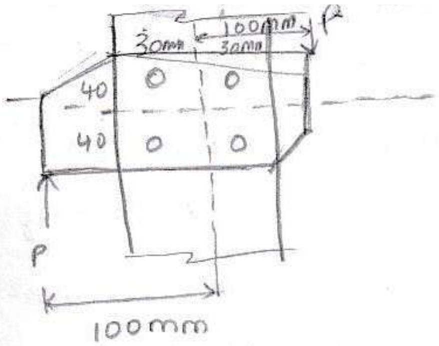
combined stresses

$\frac{F_{a,cal}}{F_a} + \frac{\tau_{m,cal}}{\tau_m} \le 1.4$

$\left[\frac{P/4}{20}\right] + \left[\frac{P/2}{15}\right] = 1.4$

$\frac{P}{80} + \frac{P}{30} = 1.4$

4)



$F_1 = \frac{P}{n} = \frac{P}{4} - \frac{P}{4} = 0$

$F_2 = \frac{P.e.r_n}{\Sigma r^2}$

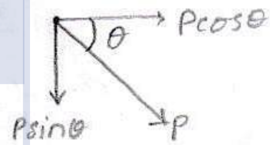
$= \frac{P\times100\times50}{4\times50^2} + \frac{P\times100\times50}{4\times50^2}$

$= \frac{P}{2} + \frac{P}{2} = P$

$F_{max} = F_R = \sqrt{F_1^2+F_2^2+2F_1F_2\cos\theta}$

$F_R = F_2 = P$

7)



$F_{1\,cal} = \frac{\text{vertical component}}{n}$

$\tau_{m\,cal} = \frac{\text{Hzl component}}{n}$

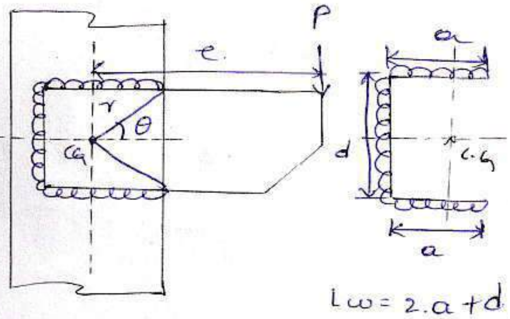
$F_{a,cal} = \frac{P\sin45^\circ}{4} = \frac{P}{4\sqrt{2}}$

$\tau_{m,cal} = \frac{P\cos45^\circ}{4} = \frac{P}{4\sqrt{2}}$

$\frac{F_{a,cal}}{F_a} + \frac{\tau_{m\,cal}}{\tau_m} \le 1.4$

$\left[\frac{\frac{P}{4\sqrt{2}}}{30}\right] + \left[\frac{\frac{P}{4\sqrt{2}}}{40}\right] = 1.4$

Bracket type - I Eccentric welding (fillet weld)



$L_w = 2.a + d$

Shear stress due to direct Load

$$q_1, \text{calculated} = q_1 = \frac{P}{L_w \cdot t_t}$$

Shear stress due to twisting moment

$$q_2, \text{cal} = q_2 = \frac{P.e\,r_n}{I_P}$$

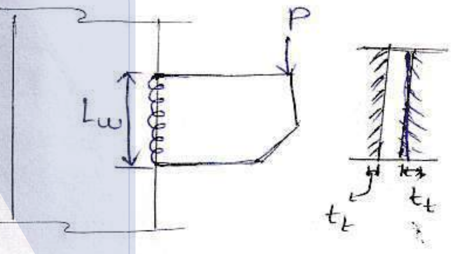
Resultant stress (or) max stress on critical point, $q_{max} = q_R = \sqrt{q_1^2 + q_2^2 + 2q_1 q_2 \cos\theta}$

for safety

$q_R \le \tau_{vf} \rightarrow$ W.S.M

$q_R \le \dfrac{f_u}{\sqrt{3}\,\gamma_{mw}} \rightarrow$ L.S.M

Bracket type - II a) fillet weld



Stress due to direct load

$$q_1, \text{cal} = q_1 = \frac{P}{2 L_w t_t}$$

stress due to moment

$$q_2, \text{cal} = q_2 = \frac{P.e}{2\left(\frac{t_t L_w^2}{6}\right)} = \frac{6Pe}{2 t_t L_w^2}$$

$\because$ from bending Eqn

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

$= \frac{M}{Z} = \frac{P.e}{Z}$

$Z = \frac{I}{y} = \frac{\frac{bd^3}{12}}{d/2} = \frac{bd^2}{6}$

$Z = \frac{t_t L_w^2}{6}$

Combined stress

$$\boxed{q_{max} = \sqrt{q_1^2 + q_2^2}}$$

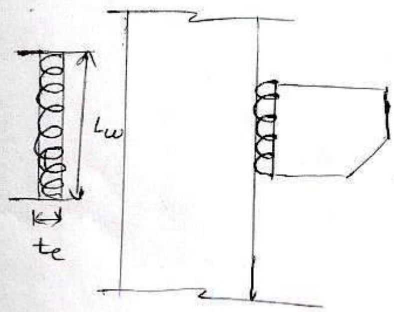
for safety

$q_{max} \le \tau_{vf} \rightarrow$ W.S.M

$q_{max} \le f_{wd} \rightarrow$ L.S.M

$$f_{wd} = \frac{f_u}{\sqrt{3}\,\gamma_{mw}}$$

b) groove weld



$$q_1, \text{cal} = q_1 = \frac{P}{L_w \cdot t_e}$$

$$q_2\,\text{Cal} = q_2 = \frac{6 \cdot P \cdot e}{t_e L_w^2}$$

Combined stress

$$\boxed{q_{max} = \sqrt{3q_1^2 + q_2^2}}$$

safety check

$q_{max} \le 0.9 f_y \rightarrow$ W.S.M

$q_{max} \le \frac{f_y}{\gamma_{m0}} \rightarrow$ L.S.M.

→ As per IS : 800 - 2007 combined shear stress & tensile stress (or) tensile stress (or) equivalent stress for fillet weld

$$\boxed{q_e = q_{max} = \sqrt{3q_1^2 + q_2^2}}$$

Pg - 32

8) Fillet weld.

$q_{max} = \sqrt{q_1^2 + q_2^2}$

$q_1 = 40 MPa$

$q_2 = 120 MPa$

$q_{max} = \sqrt{40^2 + 120^2}$

$= 10\sqrt{4^2 + 12^2}$

$= 10\sqrt{160}$

$= 10 \times 12.7$

$= 127 MPa \simeq 132 MPa$

5) Type - I

$F_1 = \frac{P}{n} = \frac{100}{5} = 20 kN$

$F_2 = \frac{P.e.r_n}{\Sigma r^2} = \frac{\cancel{100}^{25} \times \cancel{600}^{\cancel{300}^{4}} \times \cancel{75}\sqrt{2}}{\cancel{4} \times \cancel{2} \times \cancel{75^2}}$

$F_2 = 100\sqrt{2}$

$= 141.4 kN$

$\cos\theta = \frac{x}{r} = \frac{75}{75\sqrt{2}} = \frac{1}{\sqrt{2}}$

$F_R = F_{max} = \sqrt{F_1^2 + F_2^2 + 2F_1 F_2 \cos\theta}$

$= \sqrt{20^2 + 141.4^2 + 2 \times 20 \times 141.4 \times \frac{1}{\sqrt{2}}}$

$= 156.18 kN$

$r_n = \sqrt{x_n^2 + y_n^2}$

$= \sqrt{75^2 + 75^2}$

$= 75\sqrt{2}$

$\Sigma r^2 = r_1^2 + r_2^2 + r_3^2 + r_4^2 + r_5^2$

$4 \times (2 \times 75^2)$

Pg - 24

7) $s = 6mm$

$t_t = k.s$

$= 0.7 \times 6 = 4.2 mm$

$T = 8 kN\text{-}m = 8 \times 10^6 N\text{-}mm$

$\tau = q = ?$

$\frac{T}{I_p} = \frac{\tau}{R} = \frac{G\theta}{L}$

$\frac{T}{I_p} = \frac{\tau}{R}$

$\tau = \frac{T.R}{I_p}$

R = radius = 60mm

I_p = polar M.O.I

$= 2\pi R^3 t_t$

$I_p = 2\pi (60)^3 \times 4.2$

$\tau = \frac{8 \times 10^6 \times 60}{2\pi (60)^3 \times 4.2} = \frac{8 \times 100 \times 10^4}{2 \times 3 \times 60 \times 60 \times 4}$

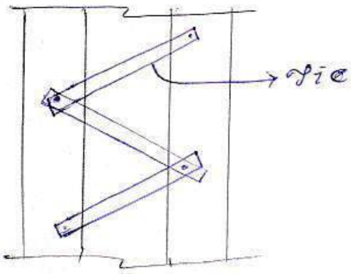
$\boxed{\tau = 84.2 MPa} \quad = \frac{10000}{3 \times 6 \times 6}$

Tension members

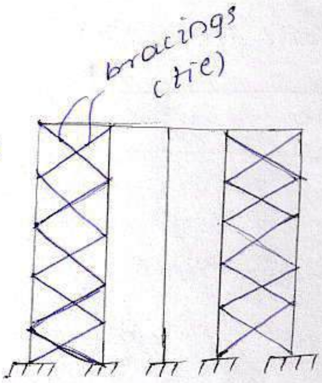
Tension member:-

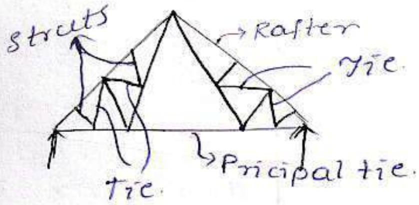
Any member subjected to axial tensile force is called tension member

Ex:- Tie

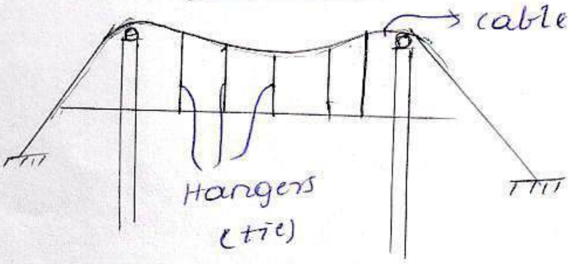


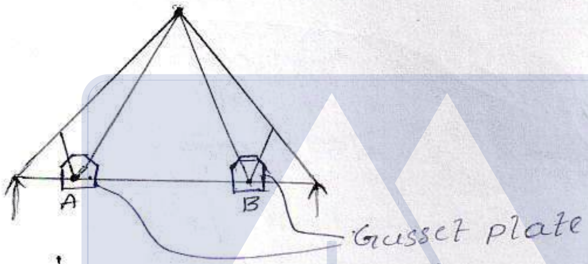
Laced column

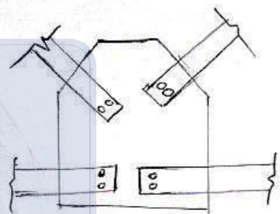




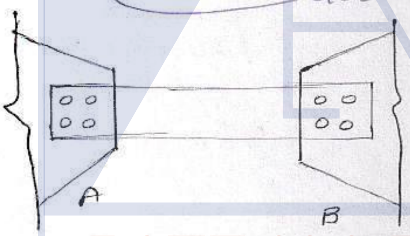
Roof truss







Gusset plate



Section A - B

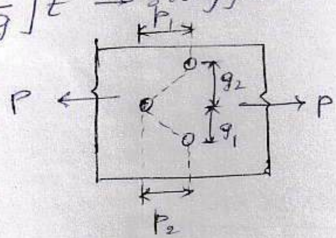
→ Tension member is designed based on net area whereas compression member is design based on Gross area (or) Effective area

To find Anet

1) For plates

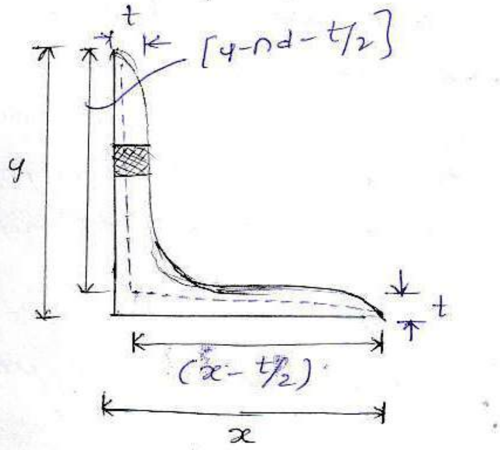
$A_{net} = [B - nd]t$ → chain pattern

$= \left[B - nd + \Sigma \frac{P^2}{4g}\right]t$ → staggared (or) Zig zag



2) For angle

i) single angle



$$A_{net} = A_1 + k.A_2$$

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

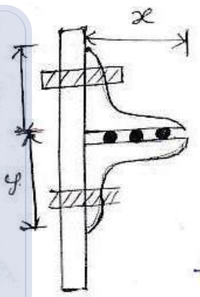
A_1 = Net section sectional Area of connected by

$= [y - nd - t/2]\,t$

A_2 = sectional area outstanding leg.

$= [x - t/2]\,t$

ii) Two angles placed back to back on same side of gusset plate with tack rivetting



$$A_{net} = A_1 + kA_2$$

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

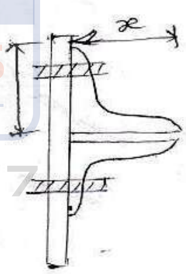
A_1 = area of connected leg

$= [y - nd - t/2]\,t \times 2$

$A_2 = [x - t/2]\,t \times 2.$

t = thickness of angle

iii) without tack rivetting



$$A_{net} = A_1 + kA_2$$

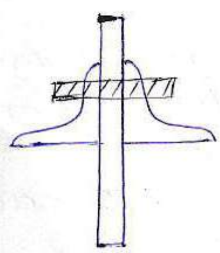
$$k = \frac{3A_1}{3A_1 + A_2}$$

$A_1 = [y - nd - t/2]\,t \times 2$

$A_2 = [x - t/2]\,t \times 2.$

iv) Two angles are placed back to back on opposite side of gusset plate [Like double cover butt joint]

A_{net} = gross area - hole area

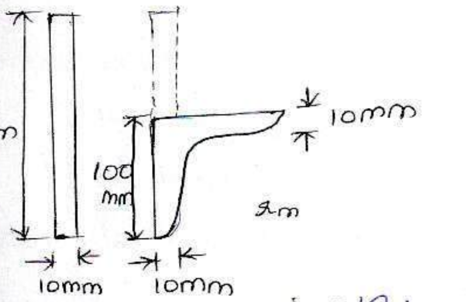


Comparision b/w plate & angle

→ In olden constructions flat members (or) plates were used as tension members although flat members strong in tension, they are weak in compression and weak in bending due to small value of radius of gyration

r = radius of gyration = $\sqrt{\frac{I}{A}}$

λ = slenderness ratio = $\frac{\text{unsupported/ effective length}}{\text{radius of gyration}}$



$r = \sqrt{\frac{I_{xx}}{A}} = \sqrt{\frac{\frac{td^3}{12}}{t \cdot d}} = \frac{d}{\sqrt{12}}$

$r_{min} = 19.1\,mm$

$r_{min} = \sqrt{\frac{I_{min}}{A}}$; I_{min} = smaller of I_{xx}, I_{yy}

$= I_{yy} = \frac{dt^3}{12}$

$r_{min} = \sqrt{\frac{I_{min}}{A}} = \sqrt{\frac{\frac{dt^3}{12}}{d \cdot t}} = \frac{t}{\sqrt{12}} = r_{min} = \frac{10}{\sqrt{12}} = 3.33$

→ In the above example, angle sections with same c/s area as flat plate has more radius of gyration.

→ In flat members load reversals and bold holes are the factors of failures But in angle sections along with these two factors eccentricity of loading must also be considered.

Design steps

→ Determine required net area of section from the relation

A_{net} required $= \frac{P \text{ (or) } P_t}{\sigma_{at}}$

→ Try a suitable section which is having sectional area 20 – 40% more than net sectional area (i.e. $A_{g\,req}$ $= (1.2 \text{ to } 1.4) A_{net,req}$)

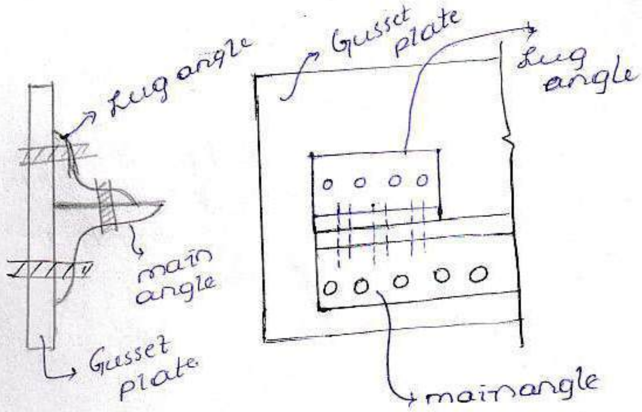
→ calculate the net area available for choosen section

→ Determine

→ A_{net} for choosen section is more than A_{net} required then the section will be ok (safe) otherwise redesign the section

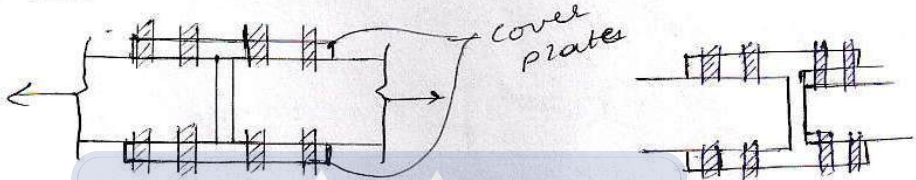
→ check for slenderness ratio

Lug angle (additional angle)



- → It is a short length of an angles used at a joint to connect the outstanding Lug on main angle
- → It is used to reduce the length of the joint
- → It is provided at the begining of joint so, that it can be effective in sharing the Load.
- → min no. of rivets or bolts used to connect Lug angle is 2 no's

Tension splice



- → It is a joint in a tension member
- → It is used when
  - i) Available length of member is less than that of required. Length of member
  - ii) To change c/s of members
  - iii) It is provided as double cover butt joint
- → min no. of rivets used are 2 numbers.

L.S.M (Limit state method)

Types of failure

1) Gross section yielding failure (T_{dg})
2) Net section rupture failure (T_{dn})
3) Block shear failure (T_{db})

$$T_d = T = \text{Smaller of } T_{dg},\ T_{dn}\ \&\ T_{db}.$$

Gross section Yielding

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

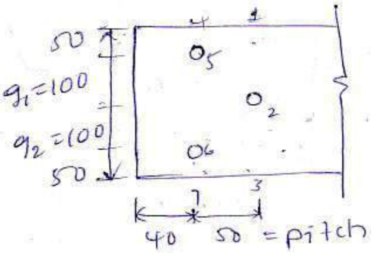
Net section rupture (T_{dn})

$$T_{dn} = 0.9\, A_{net} \cdot \frac{f_u}{\gamma_{m1}} \quad \rightarrow \text{for plate}$$

3) $A_{net} = [B - nd]t$

Net width $= [B - nd] \rightarrow$ chain

$= \left[B - nd + \Sigma \frac{p^2}{4g}\right] \rightarrow$ staggared.



Along 1-2-3, $B_{net_1} = (B - nd)$

$n = 1.$

$B_{net_1} = (300 - 1 \times 25)$

$= 275mm$

Along 4-5-6-7 $\quad B_{net} = [B - nd]$

$= (300 - 2 \times 25)$

$= 250mm$

Along 4-5-2-3

$B_{net3} = \left[B - nd + \Sigma \frac{p^2}{4g}\right]$

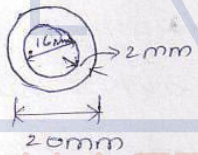
$B_{net3} = \left[300 - 2 \times 25 + \frac{50^2}{4 \times 100}\right] = 256.25mm$

Along 4-5-2-6-7

$B_{net4} = \left[B - nd + \frac{\Sigma p^2}{4g}\right] = \left[300 - 3 \times 25 + 2 \times \frac{50^2}{4 \times 100}\right] = 237.5mm$

B_{net} = smaller B_{net1}, B_{net3} & B_{net4} $= 237.5mm.$

4) $P_t = A_{net}\, \sigma_{at}$



$\sigma_{at} = 0.6 f_y$

$= 0.6 \times 300$

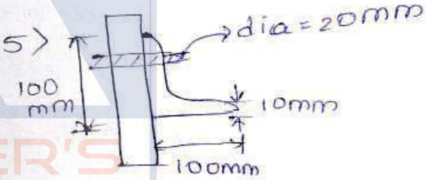
$= 180 MPa$

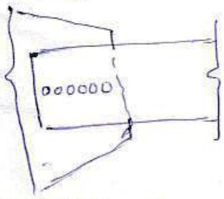
$A_{net} = A_g = \frac{\pi}{4}(20^2 - 16^2)$

$P_t = \frac{\pi}{4}(20^2 - 16^2) \times 180$

$P_t = 20.35 kN$

5)





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

$\sigma_{at} = 0.6 f_y = 150 MPa$ for Fe-250

$A_1 = [y - nd - t/2]t$

$= (100 - 1 \times 21.5 - \frac{10}{2}) \times 10$

$= 735mm^2$

$A_2 = [x - t/2]t = (100 - \frac{10}{2})10$

$= 950mm^2$

~~A_{net}~~ $k = \frac{3A_1}{3A_1 + A_2} = \frac{3 \times 735}{3 \times 735 + 950}$

$= 0.69$

$A_{net} = A_1 + k.A_2 =$

$735 + 0.69 \times 950 = 1390mm^2$

$P_t = 1390 \times 150 = 208kN \simeq 210kN$

Compression members

Any member carrying axial compressive load is called compression member

Ex:- (strut (In roof truss)
Stanchion (In buildings)
Boom (in cranes)
Rafter (in truss)

Load carrying capacity of comp member

$$P_c = A\,\sigma_{ac}$$

A = effective (or) gross area

σ_{ac} = permissible (or) Allowable comp stress

Merchant-Rankine's Relation to find σ_{ac}.

$$\boxed{\sigma_{ac} = \frac{0.6\,f_y f_{cc}}{\left[f_y^n + f_{cc}^n\right]^{1/n}}}$$

n = a constant = 1.4

fy = yield stress

fcc = Elastic critical comp stress (Buckling stress)

(Buckling Load) $P_{cr} = \dfrac{\pi^2 EI}{L_{eff}^2}$ $r = \sqrt{\dfrac{I}{A}} \Rightarrow r^2 = \dfrac{I}{A} \Rightarrow r^2.A = I$

$P_{cr} = \dfrac{\pi^2 E r^2 A}{Le^2}$ $\because \lambda = \dfrac{Le}{r}$

$\dfrac{P_{cr}}{A} = \dfrac{\pi^2 E r^2}{Le^2} \Rightarrow \dfrac{P_{cr}}{A} = f_{cc} = \dfrac{\pi^2 E}{\lambda^2}$

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

* Types of failures

1) Short column → fails by crushing.
2) Intermediate columns → fails by inelastic buckling
3) Long column → fails by elastic buckling by Euler

* Slenderness ratio (λ) :- $\dfrac{\text{Effective Length}}{\text{min radius of gyration}} = \dfrac{L_{eff}}{r_{min}} = \dfrac{K.L}{r_{min}}$

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

* Effective Length (K.L):-

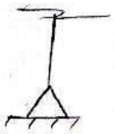
K = Effective Length constant

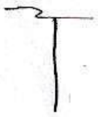
K depend on

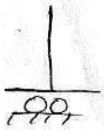
i) End conditions

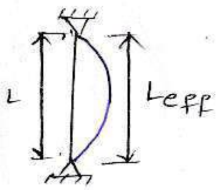
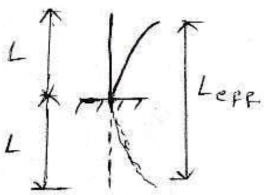
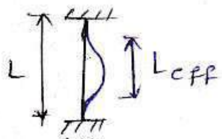
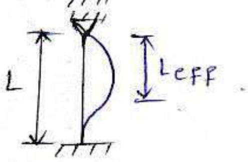
ii) no. of members connected.

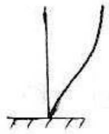

→ Restrained against translation (Held in position) & restrained against rotation


→ Restrained against translation (Held in position) but free against rotation


→ Not restrained against translation (Not held in position) & not Restrained against rotation


→ Not held in position but restrained about rotation

Type of support	Theoretical values [S.O.M]	Recommended values [RCC/steel]
	L	L
	2L	2L
	$\frac{L}{2} = 0.5L$	0.65L
	$\frac{L}{\sqrt{2}} = 0.707L$	0.8L

	2 L	2 L
	—	1·20L
 [Top support is partially restrained against translation & rotation]	—	1.5L

For angles used as strut	Effective length	Allowable stress (σ_{ac})
1) Discontinuous angle		
i) single angle	L	$0.8\,\sigma_{ac}$
ii) Double angle	0.85L	σ_{ac}
2) continuous angle For single, double	0.7 to 0.85L	σ_{ac}

→ Max (or) Allowable slenderness ratio (λ_{max})

1) A member carrying compressive loads resulting from dead load and live load — $\lambda_{max} \leq 180$

2) A tension member carrying dead load an live load — $\lambda_{max} \leq 180$

3) A tension member in which reversal of stresses due to wind or earthquake forces — $\lambda_{max} \leq 350$

4) A compression member subjected to reversal of stress due to wind or earthquake — $\lambda_{max} \leq 250$

5) A simple tension member — $\lambda_{max} \leq 400$

6) compression flange of an I-beam — $\lambda_{max} \leq 300$

Design steps

1) → Assume σ_{ac} values and determine approximate area required.

$$A_{approx.\,reqd} = \frac{P}{\sigma_{ac}}$$

(i) For single angles, L-sections, I-sections $\sigma_{ac} = 65 + 80\ N/mm^2$

(ii) For Build up section $\sigma_{ac} = 100$ to $110\ N/mm^2$

Setp-2 choose a trial section having area equal to area required.

3) Determine slenderness ratio for the choosen section and compare with the allowable slenderness ratio.

→ If choosen section $\lambda \leq$ allowable λ then it is ok otherwise (redesign) revise the section

4) Determine compressive load carrying capacity for the choosen section

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8) $A = 1908mm^2$

$f_y = 260N/mm^2$

$P_t = ?$

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

$A_{net} = A_1 + K A_2 = 1908mm^2$

$\sigma_{at} = 0.6f_y = 0.6 \times 200$

$= 156N/mm^2$

$P_t = 1908 \times 156 = 300kN$

9) $\lambda_{max} \leq 350$

$\frac{l_{eff}}{r_{min}} \leq 350$

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

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

$r = \frac{16}{4} = 4mm$

$\frac{l_{eff}}{r} = 350$

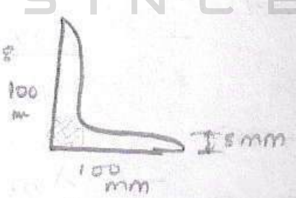
$l_{eff} = 350 \times 4$

$l_{eff} = 1400m$

11) $100 \times 100 \times 8mm$

$A = 100 \times 8 + 100 \times 8 - 8 \times 8$

$A = 1536mm^2$



$\sigma_{ac} = 0.8\,\sigma_{ac}$

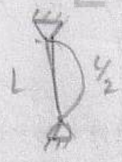
$= 0.8 \times 44$

$= 35.2\ MPa$

$P_c = A\,\sigma_{ac}$

$= 1536 \times 35.2 = 56.32kN$

Pg: 48



Q3) $L_{eff} = L$

$P_{cr} = \frac{\pi^2 EI}{l_e^2} = \frac{\pi^2 EI}{L^2} = 200kN$

$L/2 = L_e$

$L/2 = L_e$

$P_{cr} = \frac{\pi^2 EI}{(L/2)^2} = \frac{4\pi^2 EI}{L^2}$

$P_{cr} = 4 \times 200 = 800kN$

compression - wind

4) $\lambda_{max} \le 250$

$$\lambda_{max} = \frac{L_{eff}}{r_{min}}$$

$$= \frac{1500}{22}$$

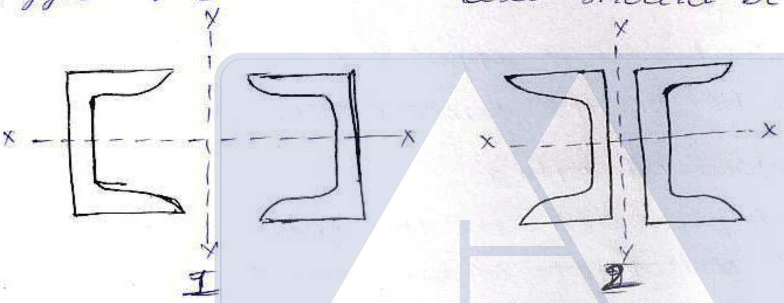
$$= 68$$

Built up section

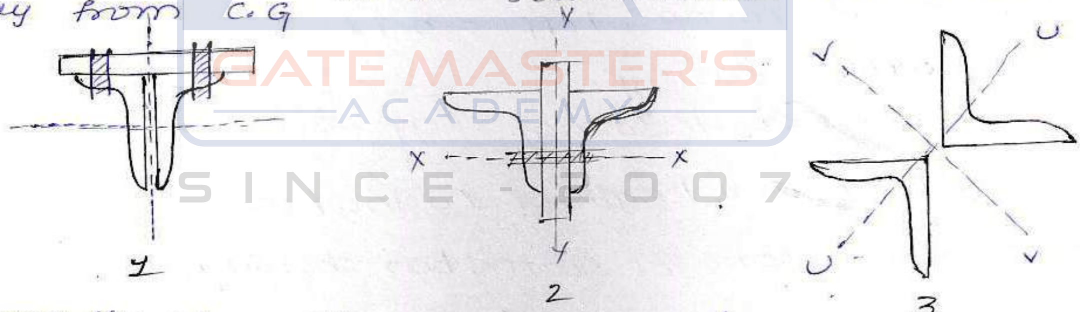
→ These sections are used when available rolled sections are insufficient to carry the loads

→ Different rolled sections are placed with spacing b/w them to carry the loads uniformly

→ The different-Rolled sections are placed such that radius of gyration about both axis should be equal (i.e., $r_{xx} = r_{yy}$)



→ For same area of c/s I^{st} section is more Efficient and economical compare to 2^{nd} section because more area thrown away from C.G



→ Among the above three sections 3^{rd} section is more Efficient & Economical

→ Different rolled section used as built up sections are connected by Lacings, battens to act as a single section

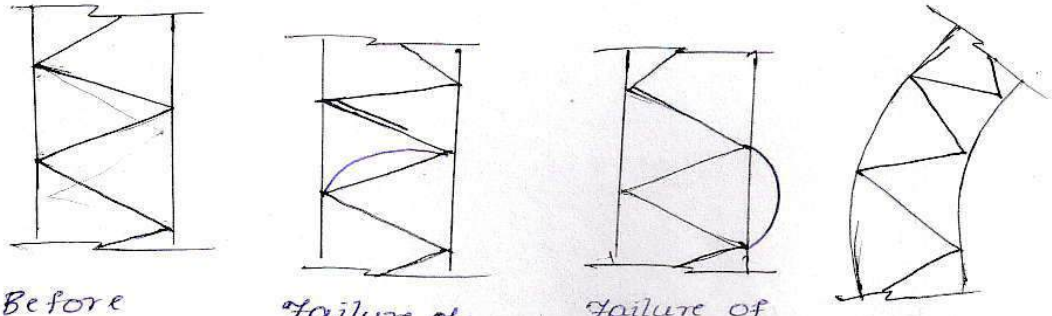
→ Lacing system is preffered for eccentric loads

→ Batten system is preffered for Axial loads

Lacings

Failure of lacing system

1) Failure of column component
2) Failure of lacing component
3) Failure of lacing system as a whole (Global failure)



Specifications

- Falt plates, angles, channels, tubular sections are used as Lacings.
- Lacing system should not be varried (uniform) throughout the section
- The single Laced systems one opposite sides of the main components should be in the same direction so that one will be the shadow of other
- Tie plates should be provided at the ends of Lacing system to avoid distortion of column components
- Effective Length of Lacing bar

  For Riveted / Bolted
  - i) For single Lacing, $L_{eff} = L$
  - ii) For double Lacing, $L_{eff} = 0.7l$
  - iii) For welded Lacing.

    [single & Double Lacing] , $L_{eff} = 0.7L$

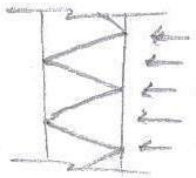
  where, l = actual length of Lacing bar
- The slenderness ratio of Lacing bar should not exceed 45
- ~~Min thickness~~ Effective Length of Laced column is increased by 5%.
- Lacing bars are provid with inclination angle b/w 40°–70° (Preferable angle - 45°) with verticle
- min thickness of Lacing bar.
  - i) For single Lacing, $t_{min} \nless \frac{l}{40}$
  - ii) For double Lacing, $t_{min} \nless \frac{l}{60}$

    l = Length of Lacing bar.
- Slenderness ratio of Lacing system should not exceed 50 or 0.7λ

where λ = slenderness ratio of compression member as a whole

→ Lacing bar is designed for transverse shear force, V = 2.5% of axial load on column

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

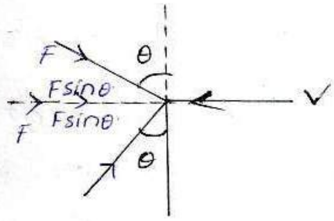


→ Lacing bar is designed for longitudinal shear

$\Sigma H = 0.$

$F\sin\theta + F\sin\theta = V$

$F = \dfrac{V}{2\sin\theta}$ → single lacing



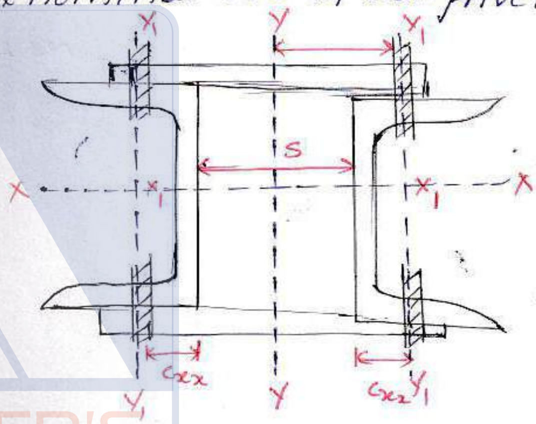
$F = \dfrac{V}{4\sin\theta}$ → double

→ min width of lacing bar ≃ 3 x nominal dia of bolt/rivet

$I_{xx} = I_{yy}$

$2 I_{x_1-x_1} = 2\left[I_{y_1-y_1} + A\left(C_{xx} + \frac{S}{2}\right)^2\right]$

$$\boxed{I_{x_1-x_1} = I_{y_1-y_1} + A\left[C_{xx} + \frac{S}{2}\right]^2}$$



$r_{xx} = r_{yy}$

Battens

→ Effective length of battened column should be increased by 10%.

→ The no. of battens should be such that the member is divided into not less the parts i.e., min 4 no. of battens are provided

→ min thickness of batten should not be less than $\frac{L}{50}$

→ Spacing b/w battens is determined with help of slenderness ratio relation

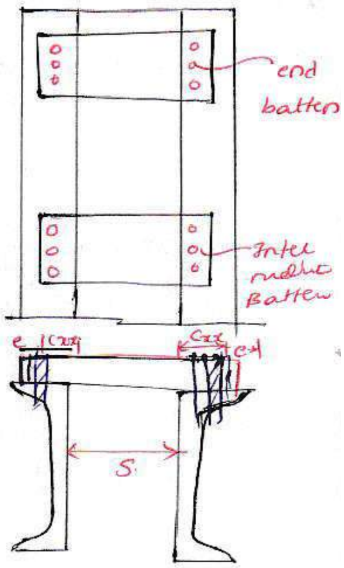
$$\frac{C}{r'_{min}} \ngtr 50 \text{ (or) } 0.7\lambda$$

λ = slenderness ratio of compression member of whole

r' = min radius of gyration of components of compression member

* Length of batten, $l = d_1 = S + 2C_{xx} + 2\,\text{edge dist}$

$$d' = S + 2C_{xx}$$



* Depth of end batten, $d_1' = d'$

Depth of Intermediate batten, $d_1' = \frac{3}{4} d'$

→ Battens are designed for a transeverse shear force $V = 2.5\%$ of axial load on column

→ Battens also design for longitudinal (shear & movement longitudinal shear on batten $= \frac{V.C}{N.S}$

moment on batten $= \frac{V.C}{2N}$

$N = 2$

Pg: 48

7) $P = 160$ tonnes

$V = 2.5\%\, P$

$= \frac{2.5}{100} \times 160$

$= 4$ tonnes

8) $P = 1200\,kN$

$V = 2.5\%\, P$

$= \frac{2.5}{100} \times 1200$

$V = 30\,kN$

9) $P = 500\,kN$

Hinged support

$F = \frac{V}{2\sin\theta}$

$V = 2.5\%\, P$

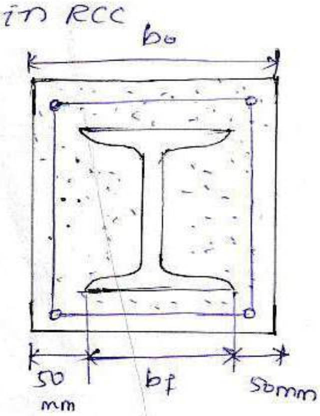
$V = \frac{2.5}{100} \times 500$

$V = 12.5$

$F = \frac{12.5}{2\sin(55)}$

* Encased column

Steel column is encased in cement concrete in RCC



purpose

→ To have better architectural appearence
→ To Increase fire resistance
→ To check corrosion

$\boxed{b_o = b_f + 100mm}$

$\boxed{r_{yy} = 0.2\,[b_f + 100mm]}$

Specifications :-

→ column should be unpainted and Encased with min grade of concrete (as per W.S.M min grade of concrete M_{15} as per LSM, M_{20}

→ In designing a cased column the entire Load is assume to be taken by steel only

→ The concrete increases stiffness of the column

→ The Load carrying capacity of encased column 2 times of uncased column

* column base & column splice

column base :-

It is used to distribute Loads over a greater area

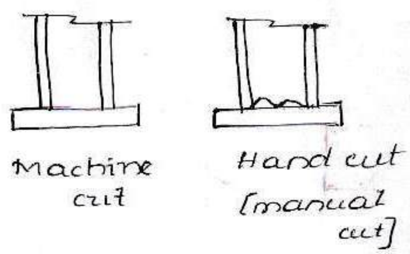
Types of column base:-

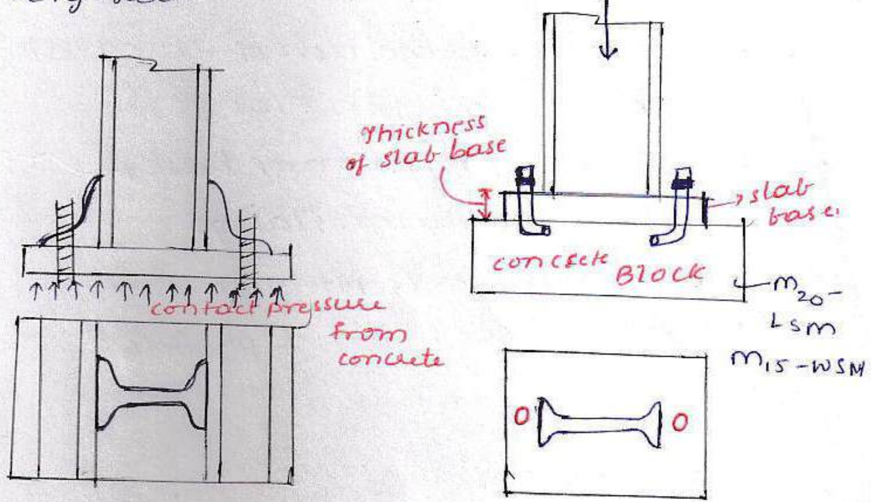
1) **Slab base** :- (provided) used when column is subjected to axial loads only

2) **Gusseted base** :- when column is subjected to eccentric loads

3) **Grillage foundation** :- when column is subjected to heavy Loads & bearing capacity of soil is very Low

slab base:-





→ column end is machined to transfer loads to the slab base by direct bearing (through plate only)

→ Fastening are provided to retain column securly in place and to resist moments and forces except direct bearing.

→ Foundation bolts or anchor bolts are used to resist uplift pressure from concrete.

Concrete pressure from concrete = 4 MPa (WSM) → M_{15}
(Bearing stress of concrete)

→ Design contact pressure of concrete = $0.45 f_{ck}$ → LSM (M_{20})
(bearing stress of concrete)

→ Area of base plate required, $A_{req} = \dfrac{\text{Total Load on base plate}}{\text{bearing stress of concrete}}$

$$A_{req} = \frac{P}{\sigma_{cc}}$$

For square plate, $B \times B = A_{req}$.
rectangle plate $B \times L = A_{req}$.

→ Bearing stress in concrete

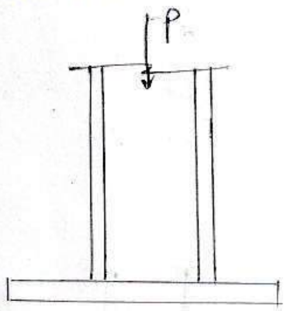
$$w = \frac{\text{Total axial Load}}{\text{Area of base plate provided}}$$

$$w = \frac{P}{A_{provided}}$$

→ Thickness of base plate

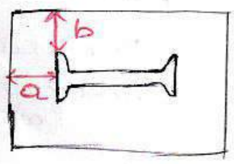
$$t_s = t_b = \sqrt{\frac{3w}{\sigma_{bs}}\left[a^2 - \frac{b^2}{4}\right]} > t_f$$

σ_{bs} = permissible bearing stress in slab base
$= 0.75 f_y = 185\ \text{N/mm}^2$



a = greater projection of base plate beyond column flange

b = shorter projection of base plate beyond column flange



→ For optimum or economical thickness of base plate, a & b are kept same

L.S.M

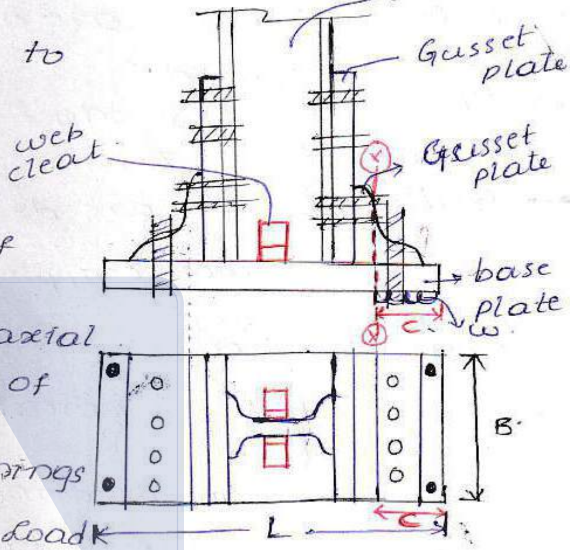
thickness of base plate $\boxed{t_s = t_b = \sqrt{\dfrac{2.5\,w\,[a^2 - 0.3b^2]\cdot \gamma_{mo}}{f_y}}}$

w = design bearing stress of concrete

$$\boxed{w = \frac{P}{A_{provide}}}$$

Gussetted base

- It consist of a base plate connected to the column through gusset plate, gusset angles and web cleats
- Thickness of base plate in this case is smaller than it required in case of slab base
- If the end is machined, 50% of axial load is considered in the design of fastenings
- If end is not machined then fastenings will be designed for 100% column load



- critical section is considered @ root of fillet as shown in fig

* moment due to contact pressure

w/m

l

⇒ SF = Area of loading

BM = Area of loading x centroidal distance

Moment due to applied load

$$\boxed{M = \frac{wc^2}{2}} \rightarrow \text{per mm width}$$

$\because \frac{m}{I} = \frac{f}{y}$

Resisting moment of plate per mm width

$$\boxed{M_r = f \cdot \frac{t_b^2}{6}}$$

$m_{resisting\ moment} = f \cdot \frac{I}{y}$

$m_r = f \cdot Z = f_y \frac{t_b^2}{6}$

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

$Z = \frac{1\,t_b^2}{6} = \frac{t_b^2}{6}$

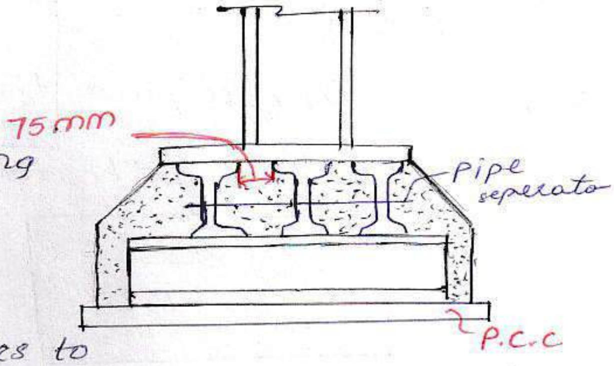
for safety, $M \le m_r$

$$\frac{wc^2}{\cancel{2}} = f \cdot \frac{t_b^2}{\cancel{6}_{3}}$$

$$\boxed{t_b = c\sqrt{\frac{3w}{f}}}$$

c = projection of base plate from critical section

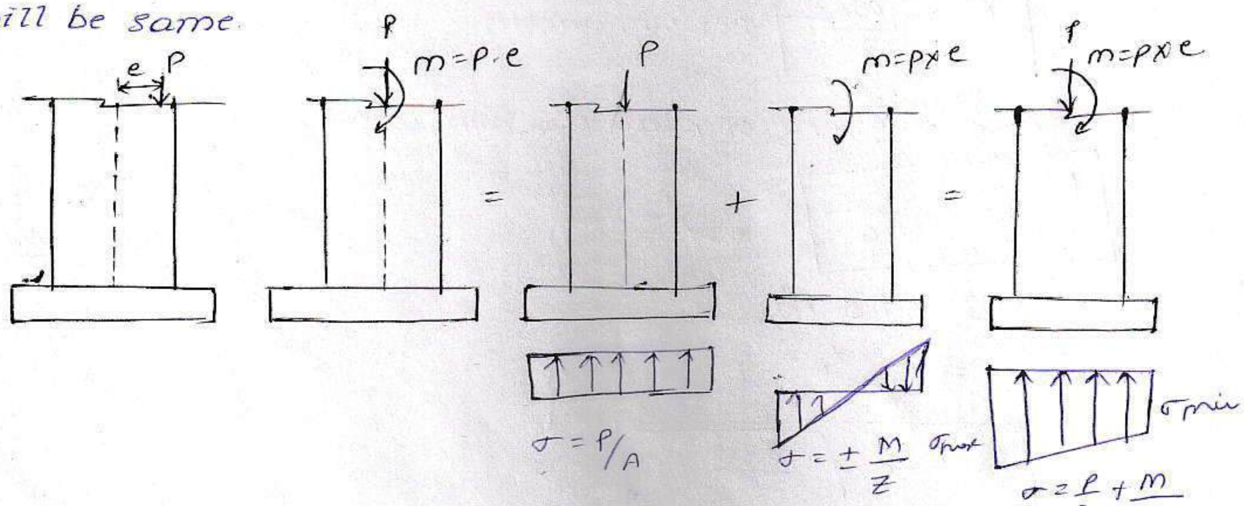
Grillage foundation



- It is used when column carries extremly heavy loads and bearing capacity of soil is very low
- It consists two or more tiers of steel beams are placed one above the other at right angles to each other and embedded in concrete
- pipe-separaters are used to keep Grillage beams properly spaced
- The distance b/w edges of adjacent flanges shall not be less than 75mm
- Grillage beams are designed for B.M and checked for shear and web crippling

Column splice

- It is a joint in compression member
- It is provided
  1. To change the dimension of columns
  2. when available length of column is ~~more~~ less than require length of column
  3. The ends of compression member should be machined to ensure perfect contact surface in bearing
- where column is machined 50% of the load is transfer directly and remaing 50% will be transferred through splice
- The slenderness ratio of splice plates is assumed to be '0' so, the permissible stresses in tension as well as compression will be same



$$\boxed{\sigma = \frac{P}{A} \pm \frac{P.e}{Z}}$$

Pg-61

Q 2) $\sigma_{min} = 0$

$$\sigma_{min} = \frac{P}{A} - \frac{P.e}{Z}$$

$$Z = \frac{bd^2}{6} = \frac{1.L^2}{6}$$

$$0 = \frac{P}{1\times L} - \frac{P.e}{\frac{1\times L^2}{6}}$$

$$\frac{P}{L} = \frac{P.e}{\frac{L^2}{6}}$$

$$e = \frac{L}{6}$$

Q-3) $t_b = c\sqrt{\frac{2.75\,w}{f_y}}$

$t_b \propto c$

Note:- Column splice is not provided at location where Bm is more than 50% compare to resisting moment.

→ splicing is done at $l/3$ to $l/4$ distance from supports (or) ends

Design of beam

Beam

Any member carrying transverse load is called beam

→ beam is designed for B.M and can be checked for S.F & deflections.

Types of Beams

→ Floor Beam :- The a major beam supporting floors

→ Lintel :- A beam supportin wall directly over door or window or any other openings

→ stringer :- A Beam supporting stair case

purlin :- A beam supported by roof truss and supporting roof sheet

Header :- A beam provided at stair openings

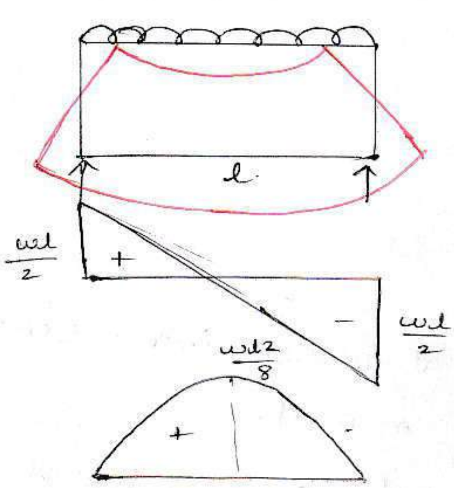
Girder :- Any major Beam in a structure which is carrying heavy loads is called girder.

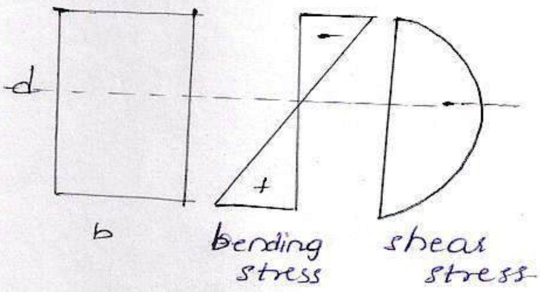
Spandrel :- A Beam on the outside supporting floors as well as walls above it

Joist :- A Beam supporting floors but no other beams

Girt :- It is an external beam in case of Industrial buildings to support ~~sh~~ wall sheeting common

Rafter :- A Beam supporting purlins in roof truss

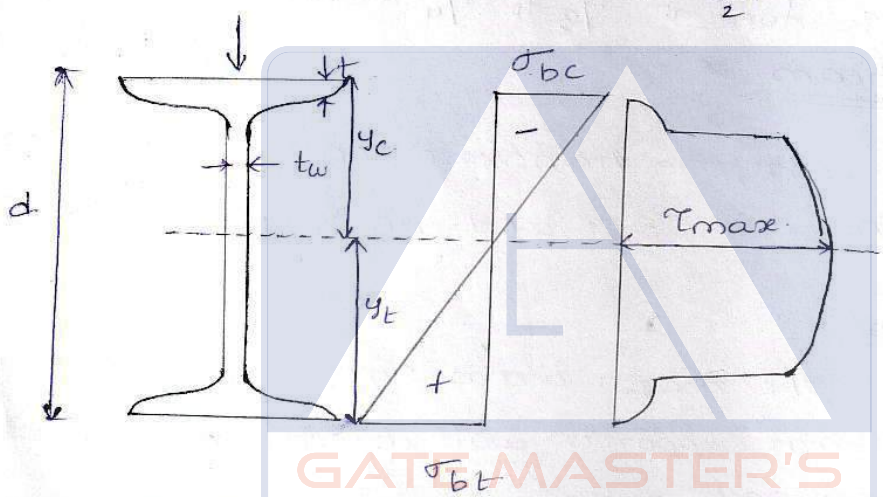




→ Desing

$BM = \frac{wl^2}{8}$

$SF = \frac{wl}{2}$



Bending relation:-

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

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

$$\boxed{\sigma_{bc\,calculated} = \frac{M}{I} y_c}$$

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

σ_{bc} = permissible bending stress in compression

σ_{bt} = permissible bending stress in tension

Laterally restrained beams

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

Laterally unrestrained beam

$\sigma_{bt} = 0.66 f_y$, $\sigma_{bc} < 0.66 f_y$

* σ_{bc} is determined by Rankine's - Merchant formula

$$\boxed{\sigma_{bc} = \sigma_{ac} = \frac{0.6 f_y f_{cc}}{[f_y^n + f_{cc}^n]^{1/n}}}$$

For safety

$bc, cal \le \sigma_{bc}$

$\sigma_{bt, cal} \le \sigma_{bt}$

→ when a beam is prevented from undergoing lateral buckling or bending then the beam is called as Laterally restrained beam

→ Since the lateral buckling occurs due to compression the compression flange is only to be Embedded in concrete (or) added with channel

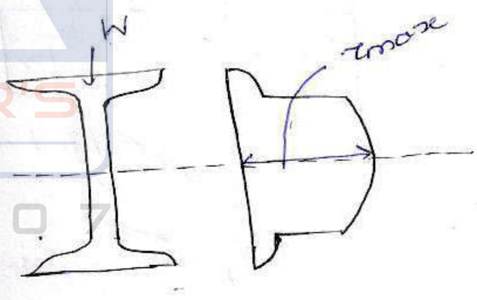
$M \rightarrow$ moment due to applied load

$M_R \rightarrow$ Resisting moment

for safety, $M \le M_R$

$M_R = \frac{f \cdot y}{I} \Rightarrow M_R = \frac{\sigma_b}{Z}$

Shear force



$\tau_{vmax, cal} = \frac{V.A\bar{y}}{I.b}$

$\tau_{vavg, cal} = \frac{V}{bd}$

For safety:-

$\tau_{vmax\,cal} \le \tau_{max}$

$\tau_{vavg, cal} \le \tau_{vavg}$

τ_{vmax} = permissible max shear stress $= 0.45 f_y$

τ_{vavg} = Permissible avg shear stress $= 0.4 f_y$

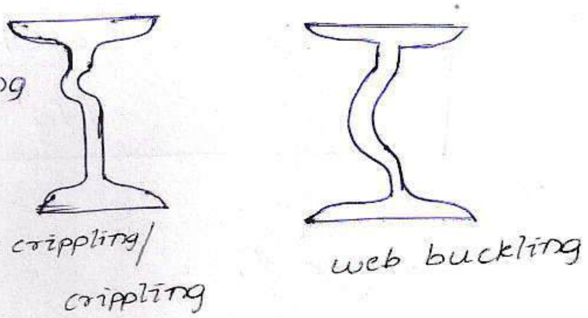
Deflection check

Allowable (or) permitted deflection, $\delta = \frac{span}{S} \rightarrow$ s.s. beam

$\delta = 2 \times \frac{span}{325} \rightarrow$ cantilever

web buckling & web crippling :-

→ Beam may also fail under concentrated load due to crippling of web or buckling of web



web crippling :-

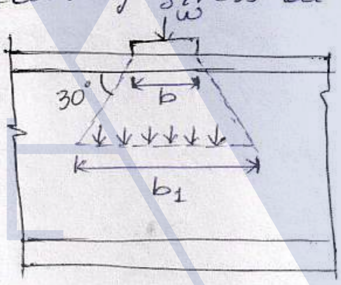
It occurs at a section where there is more bearing stress at the root of fillet occurs

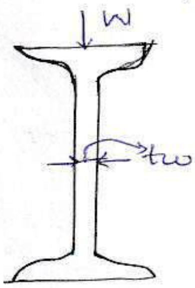
Web buckling

It occurs due to columnar action of web under concentrated loads and due to diagonal compression due to shear

→ Load dispersion under concentrated load is assumed to be 30° with H.S. for the determination of bearing stress at the root of fillet

bearing stress,





$$\boxed{\sigma_{br,\,cal} = \frac{W}{b_1\, b_w}}$$

for safety,

$$\boxed{\sigma_{br,\,cal} \le \sigma_{br}}$$

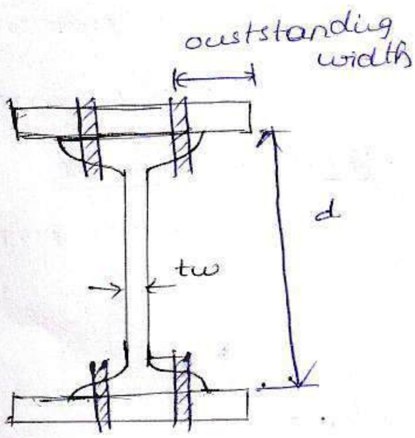
σ_{br} = permissible bearing stress $= 0.75 f_y$

Note :-

→ The above permissible stresses (σ_{bc}, σ_{bt}, $\tau_{v\,max}$, $\tau_{v\,avg}$, σ_{br}) are increased by $33\frac{1}{3}$rd % when wind or earth quake effect is considered.

Built-up-sections

→ It is used when available Rolled sections are insufficient to transfer the load and depth is restricted.



Section modulus required for plates

$$\boxed{Z_{pl} = Z_{req} - Z_{available}}$$

$A_p \times d = Z_{req} - Z_a$ $\quad \because Z = \frac{I}{y} = \frac{A \cdot y^2}{y} = A \cdot y$

$\Rightarrow \quad A_p = \frac{Z_{req} - Z_a}{d}$

→ outstanding width should not exceed $16t$ in compression
t = thickness of plate

→ outstanding width should not exceed $20t$ in tension
t = thickness of plate.

<u>Bending stresses for rivetted / bolted</u>

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

$\sigma_{bc,cal} = \frac{M \cdot y_c}{I_{gf}} \Rightarrow M \cdot y_c = \sigma_{bc,cal} \, I_{gf}$ —①

$\sigma_{bt,cal} = \frac{M \cdot y_t}{I_{nf}} \Rightarrow M \cdot y_t = \sigma_{bt,cal} \, I_{nf}$ —②

For symmetrical sections, $y = y_c = y_t$

from ① & ②

$\sigma_{bt,cal} \cdot I_{nf} = \sigma_{bc,cal} \, I_{gf}$

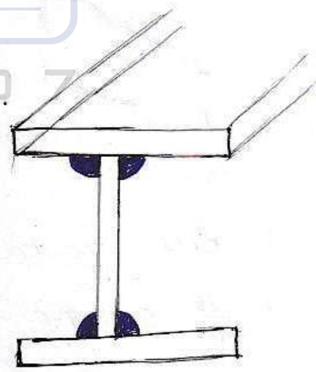
$\sigma_{bt\,cal} \cdot A_{nf} = \sigma_{bc,cal} \, A_{gf}$

$\sigma_{bc,cal} = \sigma_{bt,cal} \times \frac{A_{nf}}{A_{gf}}$

A_{nf} = Net area of flange

A_{gf} = gross area of flange

<u>For welded</u>



$\tau_v = \frac{V A \cdot \bar{y}}{I \cdot b}$

shear force per "mm" length

$F = \tau_v \times Area$

$F = \frac{V \cdot A \cdot \bar{y}}{I \cdot b} \times (b \times 1)$

$F = \frac{V \cdot A \cdot \bar{y}}{I}$

Strength of fillet weld, $P_s = 2 l_w \, t_t \, \tau_{vf}$

Per 'mm' length $\boxed{P_s = 2 t_t \cdot \tau_{vf}}$

For safety $\boxed{F \le P_s}$

Design steps

- Determine shear force and B.M
- stress in Bending and shear should be within the limits
- Deflection should be within limits
- Secondary failures like local buckling of compression flange or web web crippling should be checked.

LSM

Beam sections are classified into 4 classes Based on yield moment, plastic moment, $\frac{d}{t_w}$ ratio

class-1 :- plastic section

- C/s which can develop plastic hinges and have rotation capacity ~~to~~ required for failure. by formation of plastic mechanism

class-2 : compact section

- C/s which can develop plastic moment of resistance but have inadequate, plastic hinge rotation capacity for formation of plastic mechanism before buckling.

class 3:- semicompact

- C/s in which extreme fibre in compression can reach yield stress. but cannot develop the plastic moment of resistance due to local buckling.

class 4 :- slender section

- C/s in which elements buckle locally even before attaining yield stress

1) Laterally restrained beams

A) → If $\frac{d}{t_w} \le 67\varepsilon$ → No shear failure

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

B) → If $\frac{d}{t_w} > 67\varepsilon$ → shear failure may occur.

case I: Low shear ($V \le 0.6 V_d$)

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

for S.S.b, $M_d \leq \frac{1.2 Z_e f_y}{\gamma_{m0}}$

cantilever beam, $M_d \leq \frac{1.5 Z_e f_y}{\gamma_{m0}}$

$\beta_b = 1.0 \rightarrow$ [plastic & compact section]

$\beta_b = \frac{Z_e}{Z_p} \rightarrow$ Semi-compact

Z_e = Elastic modulus

Z_p = plastic modulus

case II :- High shear [$V > 0.6 V_d$]

V_d = Design shear force

V = factored S.F

Design shear force, $V_d = \frac{A_v \cdot f_{yw}}{\sqrt{3}\, \gamma_{m0}}$ $\quad$ f_{yw} = yield strength of web

A_v = shear area = $d \cdot t_w$.

$$\boxed{V_d = \frac{d \cdot t_w\, f_{yw}}{\sqrt{3} \cdot \gamma_{m0}}}$$

* Deflection Limits

→ for S.S.Beam

i) elastic cladding, deflection should not exceed, $\delta \not> \frac{\text{Span}}{240}$.

ii) Brittle cladding, deflection, $\delta \not> \frac{\text{Span}}{300}$

→ for cantilever Beam

i) Elastic cladding, deflection ~~shall~~ (δ) $\not> \frac{\text{Span}}{120}$

ii) Brittle cladding, deflection (δ) $\not> \frac{\text{Span}}{150}$

Pg:- 70

Q2) t = 10mm

f_y = 250MPa

$\tau_{avg} = \frac{V}{d \cdot t_w}$

$V = \tau_{avg} \times d \cdot t_w$

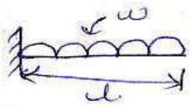
$= 0.4 f_y\, d \cdot t_w$

$= 0.4 \times 250 \times 300 \times 10$

V = 300kN

Q3) l = 3m $\quad$ cantilever Beam

P = 20kN/m.



S.F = $w \times l$

= 20×3 = 60kN

B.M = $\frac{wl^2}{2} = \frac{20 \times 3^2}{2} = \frac{20 \times 9}{2} = \frac{\overset{90}{\cancel{180}}}{2}$

B.M = 90kN-m

$\frac{M}{I} = \frac{F}{y} \Rightarrow F = \frac{M}{I} \times y = \frac{90 \times 10^6 \times 100}{1696.6}$

$\frac{90 \times 100 \times 10^2}{1696.6}$

F = 530.47N/mm²

$$\tau_{avg} = \frac{V}{d t_w} = \frac{60\times10^3}{200\times5.4} = 55.55\,MPa$$

6) $\not> 16T$

$\not> 20T$

$$\frac{256T}{\sqrt{250}} = 16T$$

Ans - $\underline{C}$

7) - a) $(V > 0.6V_d)$

8) semi-compact (elastic)

$Z_c = 500cm^3 = 500\times10^3 mm^3$

$Z_p = 650\,cm^3 = 650\times10^3 mm^3$

$\sigma_b = 200\,MPa$

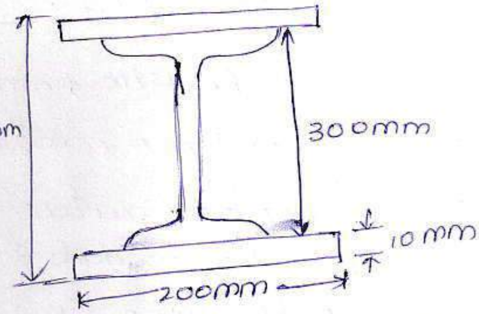
$M_e = kN\text{-}m - ?$

$M_e = \sigma_b \cdot Z_e$

$= 200\times500\times10^3$

$M_e = 100\,kN\text{-}m$

5)



ISMB 300 $\rightarrow Z_1 = 600\times10^3 mm^3$

Z_2 - plates

$Z_1 = \frac{I_1}{y_1} \Rightarrow I_1 = Z_1 \cdot y_1$

$= 600\times10^3\times150$

$I_1 = 9\times10^7 mm^3$

$I_2 = I_{x_2x_2} + A_2\bar{y}^2$

$= \frac{200\times10^3}{12} + (200\times10)(155)^2\times2$

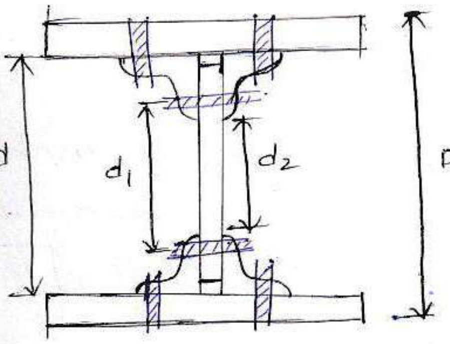
$=$

$Z = \frac{I}{y} = \frac{\quad}{160} =$

* plate Girder

→ It is provided for long spans and heavy loads in case of bridges

→ 5mm gap is kept b/w plates to avoid direct bearing or crushing of plates



Depth of plate girder

i) For rivetted / bolted, depth $= \frac{span}{10}$ to $\frac{span}{12}$

ii) For welded, depth $= \frac{span}{8}$ to $\frac{span}{12}$.

Economical depth of plate girder

→ It can be obtained if weight of plate girder is minimum

i) Rivetted / bolted, depth $= 1.1\sqrt{\frac{M}{\sigma_b \cdot t_w}}$

ii) Welded, depth $= 5\sqrt{\frac{M}{\sigma_b \cdot t_w}}$

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

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

$M = \frac{\sigma \cdot t_w d^2}{6}$

$d^2 = \frac{6M}{\sigma t_w}$

$d = 1.1\sqrt{\frac{M}{\sigma t_w}}$

self weight of plate girder (Approximate)

1) Rivetted / bolted $= \frac{W}{300}$ → kN

2) welded $= \frac{W}{400}$

* Thickness of plate girder

→ minimum thickness of plate Girder is based on serviceability criteria

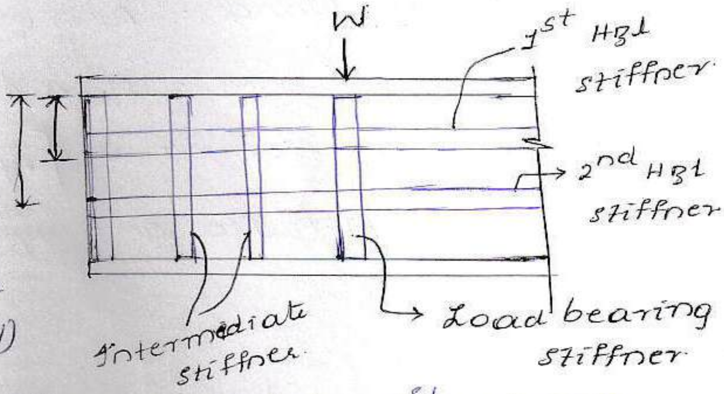
→ $t_w(min) = 6mm$ when exposed to weather and accessible for painting

→ $t_w(min) = 8mm$ when not exposed to weather and inaccessible for painting

As per IS : 800 - 1984

1) If $\frac{d}{t_w} \le 85$ → No stiffner is provided

2) If $85 < \frac{d_2}{t_w} \le 200$ → verticle (transverse stiffness is provided)



3) If $200 < \frac{d_2}{t_w} \le 250$ → verticle stiffner is provided + 1st HBL stiffner is provided

4) If $250 < \frac{d_2}{t_w} \le 400 \rightarrow$ stiffner + 1st HSL + 2nd HSL stiffner is provided

5) If $\frac{d_2}{t_w} > 400 \rightarrow$ Re design

For Flanges

Moment of Resistance, $M_R = \sigma_b \cdot Z$

M = moment due to applied

for safety, $\boxed{M \le M_R}$

$$\sigma_{bc,cal} = \frac{M \cdot y_c}{I_{gf}} \; \& \; \sigma_{bt,cal} = \frac{M \cdot y_t}{I_{nf}}$$

(or)

$$\sigma_{bc,cal} = \sigma_{bt,cal} \times \frac{A_{nf}}{A_{gf}}$$

for safety, $\sigma_{bc,cal} \le \sigma_{bc}$

$\sigma_{bt,cal} \le \sigma_{bt}$

For web plates

$$\tau_{vavg_{cal}} = \frac{V}{d \cdot t_w}$$

For safety, $\boxed{\tau_{vavg,cal} \le \tau_{avg}}$

Web Equivalent (A_{we})

some portion of web behaves as the flange is called web equivalent.

For comp side, $A_{we} = \frac{A_w}{6}$

For tension side, $A_{we} = \frac{A_w}{8}$

Area of comp flange $= (A_f + A_{we}) = \left[A_f + \frac{A_w}{6}\right]$

Area of Tension flange $= A_f + A_{we} = \left[A_f + \frac{A_w}{8}\right]$

$M = \sigma_b \cdot Z \Rightarrow M = \sigma_b \cdot A \cdot d$

moment comp flange $= \sigma_b \left[A_f + \frac{A_w}{6}\right] d$ $\qquad \because A \cdot d = Z$

Tension flange $= \sigma_b \left[A_f + \frac{A_w}{8}\right] d$

For the first HZL stiffner $I_s \geq 4 c t_s^3$

For 2nd HZL stiffner at N.A $I_s \geq d_2 t_s^3$

Force on stiffner (vertical or HZL stiffner)

The connection b/w web and stiffners is designed to withstand a S.F b/w the web and the each component of stiffner

$$F_s \geq \frac{125 t_w^2}{h}$$

t_w = thickness of web

h = out stand width of stiffness.

Load bearing stiffner

It is provided under concentrated loads and at the ends to resist reactions

→ Bearing stiffner should not be joggled

→ It is provided to avoid local bending failure of the flange and local crippling and buckling of web

→ Effective length of bearing stiffner = 0.7 l

$$\sigma_{br,cal} = \frac{\text{load on stiffner}}{\text{bearing area of stiffner}}$$

bearing area = bearing length of stiffness × thickness of web

σ_{br} ·

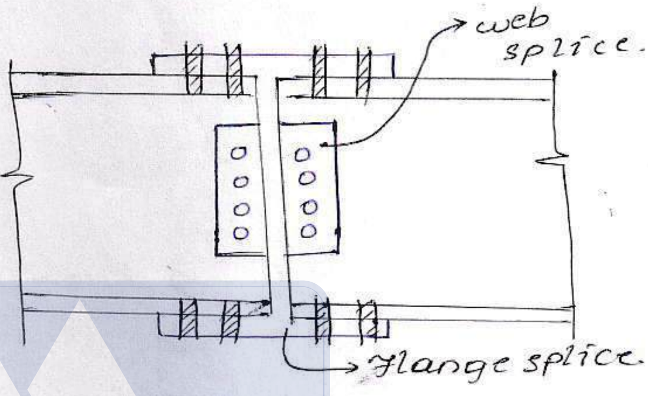
for safety,

σ_{br} = permissible bearing stress

$\sigma_{br} = 0.75 f_y$

→ the Bearing stiffner with web plate shall be designed as a column. the area of section which resists compression is area of stiffner + area of web. for a length of $20 t_w$ on both sides of the centre line of the stiffner

web splice and flange splice



→ It is a joint to increase the length of Girder

→ splices should not be located at the section where BM is max

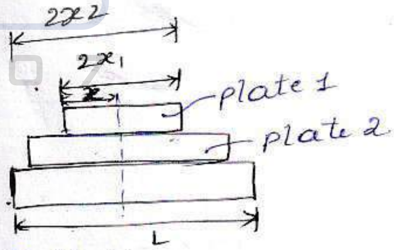
→ web splices are designed to resist S.F and moment

→ web splices are the plates provided on Both sides of web as a double cover butt joint

→ connection b/w flange plates and flange angles is designed for HBL S.F

→ connection b/w web plate and flange angles is designed for HBL and verticle S.F. In this case the connection is like a double coverbutt joint

Curtailment of plates



No. of curtailment should no be more than '3'

curtailment means minimizing the length of plate

$$x_1 = \frac{L}{2}\sqrt{\frac{A_1}{A_f + A_{we}}}$$

A_1 = Area of 1^{st} plate

A_f = Area of flange plate

A_{we} = Area of web equivalent

$$x_2 = \frac{L}{2}\sqrt{\frac{A_1 + A_2}{A_f + A_{we}}}$$

$$x_n = \frac{L}{2}\sqrt{\frac{A_1 + A_2 + \cdots + A_n}{A_f + A_{we}}}$$

LSM

Economical depth of plate girder

$$\boxed{d = \left[\frac{Mk}{f_y}\right]^{1/3}}$$

$$\boxed{k = \frac{d}{t_w}}$$

c.w

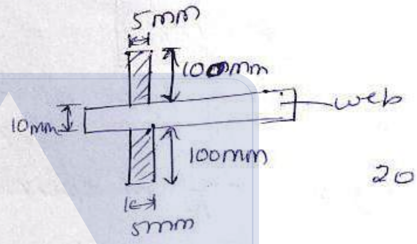
1a) $\frac{d}{t_w} \le 85 \rightarrow$ No stiffner is provided [unstiffened plate girder]

$t_w \ge \frac{d}{85}$

2) $\frac{d}{t_w} = \frac{1000}{6} = >166.67 \rightarrow$ Range b/w 85 to 200.

Hence, provide vertical stiffner.

3) Area of bearing stiffner



$= 2\{20 t_w \times t_w ~~+~~ h \times t\}$

$20 t_w \times t_w$

$= 2(100 \times 5 + 20 \times 10 \times 10 \times 10)$

$= 5000 mm^2$

4) $A_f + \frac{A_w}{8} \rightarrow$ tension

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

5) $Z_e = Z = [A_f + \frac{A_w}{8}] D \rightarrow$ tension

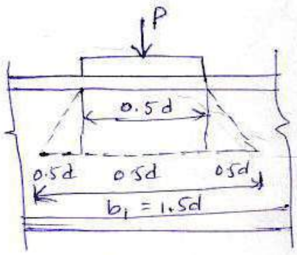
$Z_e = Z = [A_f + \frac{A_w}{6}] D \rightarrow$ comp

$Z_p = (A_f + \frac{A_w}{4}) D$

Z_p should be more than Z_e.

Z_e = elastic modulus.

6)



f, P = ?

$\sigma_{br, cal} = f = \frac{\text{Load}}{\text{bearing area}}$

$f = \frac{P}{b_1 \times t}$

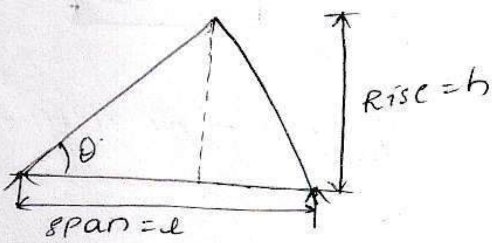
$f = \frac{P}{1.5 d t}$

$P = 1.5 d t f.$

Roof Trusses

Assumptions:-

- All the joints are to be assumed as pinned or hinged
- All the Load applied at Joints only
- All the members of roof truss are subjected axial force only



$$\text{pitch}(p) = \frac{\text{Rise}}{\text{span}} = \frac{h}{L}$$

$$\text{slope}(\theta) = \frac{\text{Rise}}{\left(\frac{\text{span}}{2}\right)} = \frac{h}{(L/2)}$$

$$= 2 \times \frac{h}{L}$$

$$\boxed{\text{slope}(\theta) = 2 \times \text{pitch}(p)}$$

- pitch depends on a type of roofing material, Light requirement
- For small pitch $< \frac{1}{12}$ of span Length
- medium pitch → $\frac{1}{5}$ to $\frac{1}{12}$ of span
- Large pitch → $> \frac{1}{5}$ of span

  G.I → Galvanized iron sheets

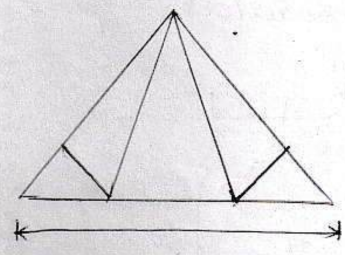
  A.C → Asbestas cement sheets
- common pitches for different roof sheets

  pitch → $\frac{1}{6}$ for G.I sheet

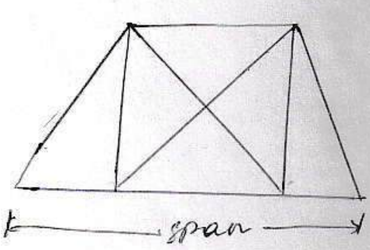
Type of Trusses

- It is based on span and pitch

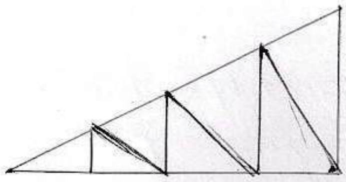
1) King post



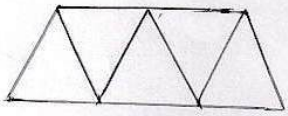
Queen post (6m to 9m)



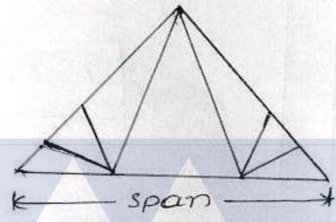
North Light roof truss (≤ 9m)



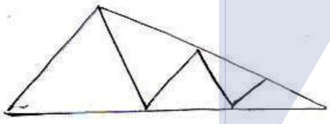
warren truss (< 6m)



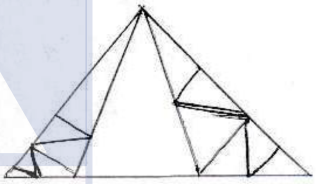
single fan type truss



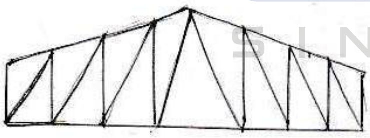
saw tooth type truss.



compount truss (or) flink type truss (10m – 20m)

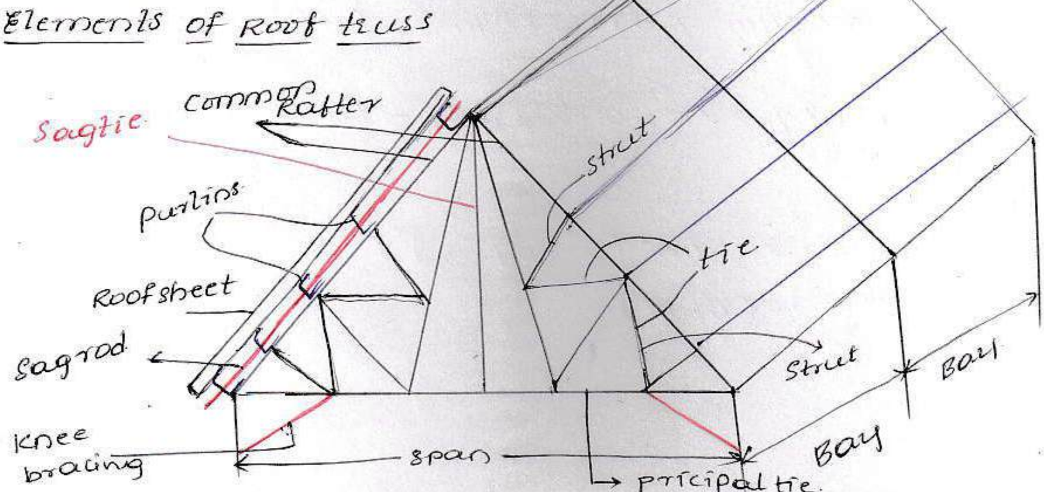


Pratt type (8m to 12m)



- For small spans warren type of truss is used
- For medium spans pratt type truss is used
- For large spans compound type of truss is used

Elements of Roof truss

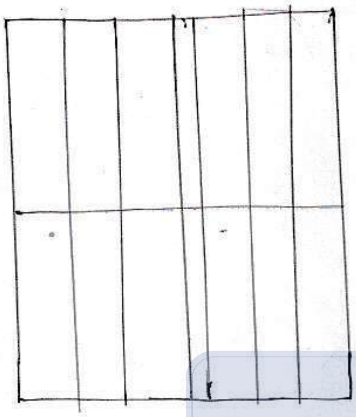


• purlin :- It is used to support Roof sheeting

→ It is subjected to B.M Because it carries transeverse loads from sheet

→ purlin's are Hzl Beams placed b/w two trusses and generally spacing varies b/w 2 to 3 meters

Economical spacing of truss



T = cost of truss

P = cost of purlin

R = cost of roof sheet

$$\boxed{T = 2P + R}$$

Types of Loads :-

1) Dead Load (D.L)
2) Imposed Load (L.L) (Live Load)
3) wind Load (W.L)
4) Snow Load (S.L)

Dead Load

i) weight of sheeting material
ii) weight of purlin
iii) self wt of truss :- Generally taken as $100 N/m^2$ to $150 N/m^2$

(or)

$$= \left[\frac{span}{3} + 5\right] 10 \; N/m^2$$

Imposed Load (L.L)

i) If $\theta \leq 10°$

L.L $= 1500 N/m^2$ (or) $1.5 kN/m^2$ (when accessibility is provided)

$= 750 N/m^2$ (or) $0.75 kN/m^2$ → (when no accessibility is provided).

ii) If $\theta > 10°$

L.L $= 1500 - 20 N/m^2 [\theta - 10°]$

$= 750 - 20 [\theta - 10°]$

Snow Load

→ S.L = $25 N/m^2$ per 'cm' depth of snow when $\theta \le 50^\circ$

→ If $\theta > 50^\circ$, no effect of snow load on truss

Wind Load

Design wind speed = $\boxed{V_Z = k_1 \cdot k_2 \cdot k_3 V_b}$

V_b = basic wind spee → m/sec

k_1 = Risk (or) probability factor.

k_2 = terrain, size & shape factor

k_3 = topography factor

Design wind pressure (P_Z) = $\boxed{0.6 V_Z^2}$

Design wind load, $F = [C_{Pe} - C_{Pi}] A_e \cdot P_Z$

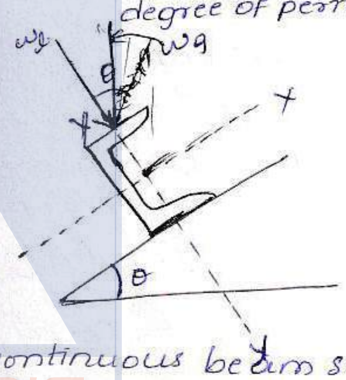
A_e = Effective frontal area

C_{Pe} = External wind pressure coefficient → depends on slope

C_{Pi} = Internal wind pressure coefficient → depends on degree of permeability

External wind pressure (material)

Purlin design



w_g = Gravity load

w_l = wind load

→ purlin is considered as fixed or continuous beam subjected to transeverse loads

→ channels, angles are generally used as purlin sections

→ Round tubular sections are also used as purlin sections

→ purlin is designed un-symmetrical beam subjected to biaxial moment

→ purlin is analyzed bi using moment distribution method

vertical load, $V = (w_l + w_g \cos\theta) \rightarrow kN/m$

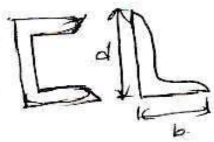
HZL load, $H = w_g \sin\theta \rightarrow kN/m$

$$M_{x-x} = \frac{VL^2}{20}$$

$$M_{y-y} = \frac{HL^2}{10}$$

$$\sigma_{b,cal} = \frac{M}{Z}$$

for safety, $\sigma_{bcal} \le \sigma_b$.



depth of angle / channel section $= \frac{L}{45}$

width of angle / channel section $= \frac{L}{60}$

L.S.M

deflection check

for elastic cladding, $\delta \ngtr \frac{span}{150}$

for Brittle cladding, $\delta \ngtr \frac{span}{180}$

CW

2) $\theta = 15^\circ > 10^\circ$

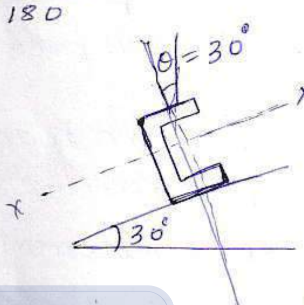
$L.L = 750 - 20(\theta - 10^\circ)$

$= 750 - 20(15 - 10^\circ)$

$= 650\, N/m^2$

$= 0.65\, kN/m^2$

3)



$m_{xx} = M\cos 30^\circ = \frac{\sqrt{3}}{2} M$

4) brace / lacing is designed for 2.5% of axial load

Force, $V = 2.5\% \cdot (120 + 120) = \frac{2.5}{100} \times 240 = 6\, kN$

Gantry Girder

Design Forces

1) vertical force

2) Lateral force

3) Longitudinal force

* Impact