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PART - A

2 Marks Questions

1.
Sol. Brittle fracture can occur when a combination of tensile stress, low temperature, thick material and rapid change of stress is present. Intermittent fillet welds laid down across tension flanges of a shape are susceptible to fracture in brittle manner.
2.
Sol. Three philosophies for designing steel structures are use :
(a) W.S.M. (b) L.S.M. (c) Plastic method
3.
Sol. There are number of advantages of bolted joints over riveted joints making the latter a history. The noise nuisance, the preheating of the rivets, the labour cost, identification and replacement cost of faulty rivets, and for the same strength requirement of lesser number of bolts, and the higher fatigue resistance of bolts give the bolted joints an edge over the riveted joints.
4.
Sol. The working stress (or maximum permissible stress) in mild steel shop rivets as per IS : 800-1984 are given in Table.

Type of Rivet	Axial Tension, σ_{tf} N/mm ² (MPa)	Shear, τ_{vf} N/mm ² (MPa)	Bearing, σ_{pf} N/mm ² (MPa)
1. Power driven rivets	100	100	300
2. Hand driven rivets	80	80	250

5.
Sol. Punched holes are preferred by the fabricators because punching is simple, time saving and cost effective but ductility and toughness is reduced that may cause brittle fracture. In structurally drilled holes these drawbacks are not present and therefore the drilled holes are better and should be recommended as far as possible.
6.
Sol. HSFG bolts are preferred. It is because the loads causing fatigue are smaller than the proof loads of HSFG bolts. Also, the fatigue strength of joints with HSFG bolts is far better than the joints with other connectors.
7.
Sol. As shown in Fig. 2, the transverse fillet welds will be subjected to more uniform stress distribution and therefore are more efficient.

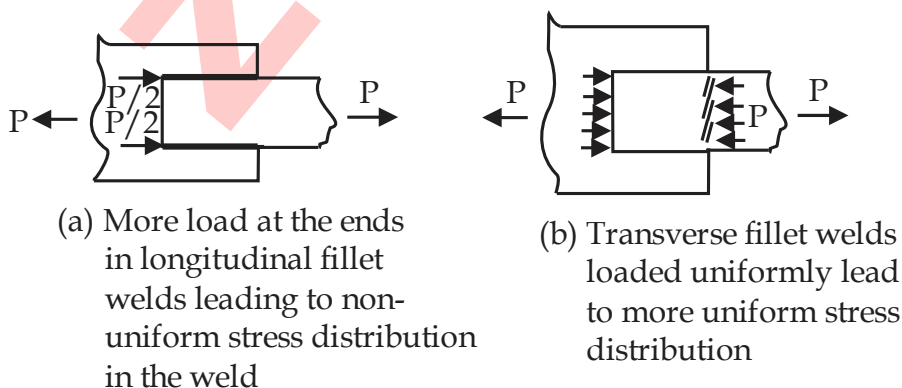


Fig.2 Efficiency of the types of fillet welds

8.
Sol. Equal legged fillet welds are preferred since they are easy to make. Unequal legged fillet welds require expertise and are used only when vertical leg length of the weld cannot be increased. The attachment of a channel shear connector is an example where only unequal leg lengths (smaller vertical leg length) are feasible. The vertical leg of the weld length at the toe will be limited by the thickness of the channel flange requiring bigger horizontal leg length to furnish the required strength.

9.
Sol. If the length of the welded joint of a splice or end connections in a compression or tension element exceeds 150 times the throat size of the weld, the reduction in weld strength as per long joint is done.

$$\left[\beta_{lw} = 1.2 - \frac{0.2 l_j}{150 t_t} \leq 1.0 \right]$$

l_j = length of joint in the direction of force.

t_t = throat size of the weld.

10.
Sol. There are two reasons.
 1. The concave fillet weld on cooling shrinks and there will be tensile stresses on its surface. This leads to development of cracks. However, when these are desired, it should be made in two passes – the first slightly convex, and the other passes are built-up to form a concave surface.
 2. The cost of forming concave weld will be high as this involves expertise, chipping and cleaning in all the operations involved.

11.
Sol. The ratio of maximum elastic stress (σ_{max}) to the mean stress (σ_{avg}) is known as stress concentration factor.

12.
Sol. The ratio of gauge length to the bolt diameter of a bolted joint in a tension member is known as geometry factor. For smaller geometry factor, the joint is more efficient and results in higher tensile strength. In fact, when steel at the net section yields, the reduction of area between holes is restrained by the metal on adjacent gross cross sections. The biaxial stresses so developed increase the tensile strength. However, when geometry factor is large, the widely spaced bolts cause non-uniform straining which initiates the local fracture of steel before stress equalisation over the net section can take place.

13.
Sol. Tension members are usually angle section, Tee-section or channel section and are connected to gusset plates by one leg only. The force transferred to the connected leg by the end connection thereafter gets transferred as tensile stress over the entire cross section by shear. Thus, the tensile stress on the section from the first bolt hole up to the last bolt hole will not be uniform. Consequently, connected leg will have higher stress at failure than the outstanding leg. Since stress in one part lags behind the other part of the section, it is referred to as shear lag. However, at some distance away from the connection the stress becomes uniform throughout the section. For larger connection lengths, therefore, the effect of shear lag will be smaller.

14.
Sol. The shear lag reduces the effectiveness of the component of the section which is not connected to the gusset plate. This results in a decrease in the strength of a tension member. Therefore, area of steel in outstanding leg should be as small as possible. This consequently results in unequal angle section with long leg connected since this will be more efficient.

15.
Sol. Block shear failure is characterised by the failure of a tension member at the end connection along a path involving tension on one plane and shear on the perpendicular plane for certain connection configurations and coped beams.
 This type of failure may occur, for instance when fewer high strength bolts are used for making a connection resulting in smaller connection length.

16.

Sol. These sections have sufficient stocky profiles and are therefore capable of developing a fully plastic stress distribution before buckling. Here, the term plastic means that all the fibers of the section are stressed to the yield stress. If sufficient ductility is required, the section invariably should be plastic.

17.

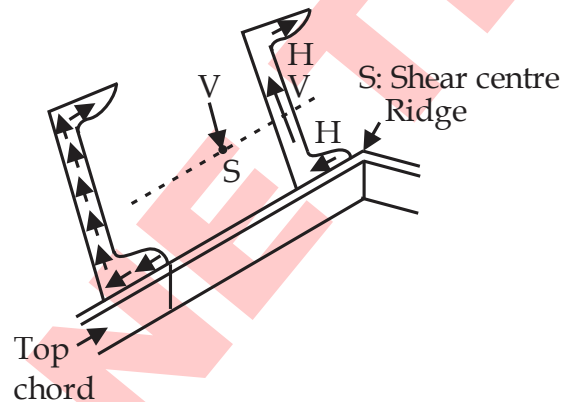
Sol. In long columns the axial buckling stress remains below the proportional limit and they buckle elastically. The failure loads for such columns are proportional to the flexural rigidity (EI) of the column and independent of the strength of steel. Whereas, for intermediate columns some of the fibers will reach the yield stress and some will not. An intermediate column fails by both yielding and buckling and its behaviour is said to be inelastic.

18.

Sol. Compression members are more critical since they have to resist buckling. The imperfections of cross section dimensions, eccentricity of the load, lateral loads, residual stresses, punched holes, erection stresses are some of the other important factors which contribute to unwanted stresses making compression members more critical.

19.

Sol. Four different column curves a, b, c, and d have been recommended by IS: 800 code of practice for different types of cross sections of columns to account for the initial imperfections of their geometry and method of manufacture.



20.

Sol. In case of laced and battened columns, slenderness is more than actual so that shear deformation effects accounted for.

21.

Sol. Web buckling (vertical buckling) occurs when the intensity of vertical compressive stress near the center of section becomes greater than the critical buckling stress for the web acting as column.

22.

Sol. In general, the columns are subjected to flexural buckling only. However, the compression flange of beam is subjected to flexural buckling as well as torsion. The longitudinal axis of the beam displaces and consequently the beam is subjected to lateral-torsional buckling.

23.

Sol. Beam is a structural member subjected to loads transverse to its longitudinal axis. Under the loading, the beam deflects in the vertical plane (plane of loading) and is characterised as bending of beam. In the process of bending one of the beam flange or its part will be in compression depending upon the support conditions for the beam. Since, the flange will be in compression it will buckle (deflect in its own plane) depending upon its slenderness ratio. Therefore, beams bend as well as buckle.

24.

Sol. The channel section purlins should be placed with flanges facing the apex of the truss. It can be explained with the shear flow inside the channel section (Fig. 1). The vertical shear is transferred at the node while the horizontal shears will form a couple that will create instability if channel flanges are not pointing towards apex of the truss.

25.

Sol. No. When a beam is loaded transverse to its minor axis (yy axis) as shown in Fig. 1, the portions of the flanges AB and DE will be in compression and the portions BC and EF in tension. The flange portions in tension and the web will not permit the flange portions in compression to buckle.

26.

Sol. The connections where bolts are required to transfer load by direct tension, an additional force is generated in the bolts due to thin jointed plates. These are called prying forces. These forces can develop only when ends of the connected plates are in contact due to external loads. The design force will be sum of the tensile force and the additional prying force on the bolt. Prying force is negligible if the plates to be jointed are thick and stiff.

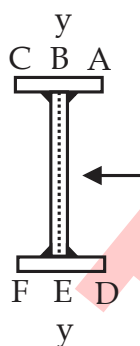


Fig.1 Beam loaded transversely

27.

Sol. When the load over the column section is too large or when the axial load is accompanied with bending moments, then requirement of base plate thickness is high. Gusset plate is used to reduce the thickness.

28.

Sol. Plastic Moment $[M_o]$:-

- In structural engineering, the plastic moment (M_p) is a property of a structural section. It is defined as the moment at which the entire cross section has reached its yield stress.

29.

Sol. (a) Z_p for rectangular section $Z_p = \frac{BD^2}{4}$

(b) Z_p for circular section $Z_p = \frac{D^3}{6}$

30.

Sol. No. of plastic hinge required for complete collapse:-

For complete collapse of a beam number of plastic hinge required is $[D_s + 1]$ because $[D_s + 1]$ hinges makes structure unstable.

Degree of static indeterminacy (D_s) = No. of unknown support equation (Rx^n) - Available Equilibrium eqⁿ

$$D_s = \text{Unknown } Rx^n - \text{Eq}^m \text{ eq}^n$$

1. $D_s > 0$: Indeterminent structure (stable)
2. $D_s = 0$: Determinent structure (stable)
3. $D_s < 0$: Unstable or collapse

31.

Sol. Plastic Hinge Length (L_p):-

- It is the length of the member in which a yielding has occur.
- It is the length of a zone in which the moment is $[M_y \leq M \leq M_p]$

32.

Sol. Shape Factor (S.F.):-

It is the ratio of plastic moment capacity to yield moment of capacity. S.F. denote the reserve strength of the beam cross section beyond the yield limit to plastic limit.

$$S.F. = \frac{M_p}{M_e} = \frac{\sigma_y Z_p}{\sigma_y Z_e} = \left[\frac{Z_p}{Z_e} \right]$$

33.

Sol. Load factor:- It is define as the ratio of plastic moment to working moment.

$$\text{Load factor} = \frac{M_p}{M} = \frac{f_y}{f} \times \frac{Z_p}{Z_e}$$

$$[L.F = F.O.S. \times S.F]$$

$$\therefore \quad F.O.S. = \frac{f_y}{f} \quad \quad S.F. = \frac{Z_p}{Z_e}$$

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

$$[M = f.Z_e]$$

34.

Sol. Tack Bolt: Tacking bolt are used to make the sections act in unison, and to prevent buckling in compression members, when two or more sections are in contact. When the distance between the centres of two consecutive bolts in such cases exceeds the maximum specified pitch of 12t or 200 mm, whichever is less in compression members; and 16t or 200 mm, whichever is less in tension members additional bolts are provided. These bolts are not subjected to calculated stresses and are called tacking or stitch bolts.

35.

Sol. Surge loads are the lateral loads produced because of sudden stopping or starting of the movement of trolley across the building. These are resisted by the compression flange alone. Drag loads also called longitudinal loads are caused by sudden starting or stopping of the crane when it moves in the longitudinal direction. These act at the rail level. The two movements of the crane, the transverse and longitudinal, cannot be had simultaneously and accordingly the two types of loads cannot act simultaneously.

36.

Sol. Roof trusses are subjected to the following types of loads :

- Dead load
- Imposed load
- Snow load
- Wind load

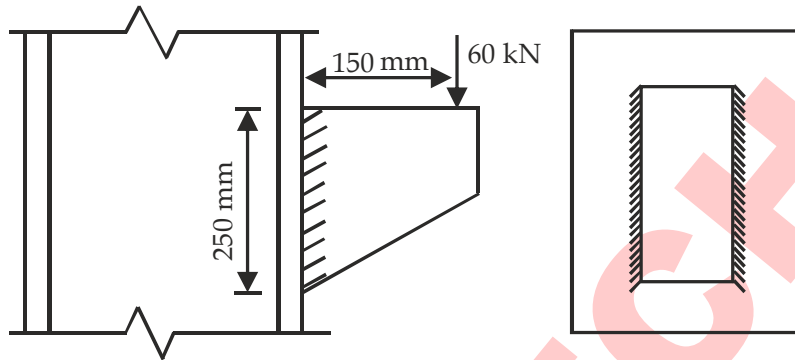
37.

Sol. Elastic cladding = Span/150
Brittle cladding = Span/180

PART - B

5 Marks Questions

38.
Sol.



Let the thickness of weld throat = t mm

Vertical shear stress

$$(f_v) = \frac{P \times 1000}{250 \times t \times 2} = \frac{60 \times 1000}{500 \times t} = \frac{120}{t} \text{ N/mm}^2$$

Horizontal shear stress due to bending at extreme fibre (f_h)

$$f_h = \frac{(P \times e) \times d / 2}{I_{xx}} = \frac{P \times e \times \frac{d}{2}}{2 \times t \times \frac{d^3}{12}} = \frac{6Pe}{2td^2}$$

$$\therefore f_h = \frac{3 \times 60 \times 1000 \times 150}{t \times 250^2} = \frac{432}{t} \text{ N/mm}^2$$

$$\therefore \text{Resultant} = \sqrt{f_v^2 + f_h^2} = \sqrt{\frac{120^2}{t^2} + \frac{432^2}{t^2}} = \frac{448.36}{t} \text{ N/mm}^2$$

For safe connection, $f_R \leq 108 \text{ N/mm}^2$

$$\frac{448.36}{t} \leq 108$$

$$\Rightarrow t \geq \frac{448.36}{108}$$

$$\Rightarrow t \geq 4.15 \text{ mm}$$

$$\Rightarrow \text{Size of weld} \geq 4.15\sqrt{2} = 5.87 \text{ mm} \cong 6 \text{ mm (say)}$$

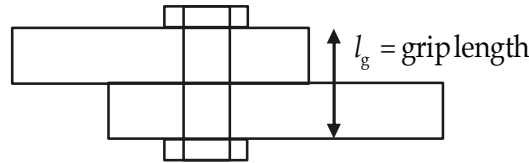
39.

Sol. • In case the connection is affected by large grip length, long joint, packing plate than shear strength of bolt is reduced, and it is determined by

$$V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}} \times \beta_{lg} \times \beta_{lj} \times \beta_{pkg}$$

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

1. $\beta_{lg} \rightarrow$ Reduction factor for large grip length.



Acc.: LSM: $L_g \not\geq 5d$

WSM: $L_g \not\geq 8d$

$$\left[\beta_{lg} = \frac{8d}{3d + L_g} \right] = \text{LSM}$$

2. $\beta_{lj} \rightarrow$ Reduction factor for long joint. When length of joint exceed $15d$ in the direction of force then shear capacity of bolt is reduced by reduction factor.

$$\left[\beta_{lj} = 1.075 - \frac{L_j}{200d} \right]$$

3. β_{pkg}

- Reduction factor due to packing plate.
- If $t_{pkg} > 6 \text{ mm}$, then strength of bolt is reduced by a Factor B_{ekg} .

$$[\beta_{pkg} = 1 - 0.0125 t_{pkg}]$$

40.

Sol. HSFG Bolt:-

- In this connection high strength bolt are pretensioned upto desired degree to generate sufficient frictional resistance between connected element through which the load is transfer.

1. **Shear strength of HSFG bolt:-**

$$\text{Design shear strength } (V_{dsf}) = \frac{\text{Nominal friction strength of HSFG}}{\gamma_{mf}}$$

$$\Rightarrow V_{dsf} = \frac{V_{nsf}}{\gamma_{mf}} \quad \gamma_{mf} = 1.25$$

$$\Rightarrow V_{nsf} = \mu_f \cdot n_e \cdot k_h \cdot F_0$$

$$[F_0 = f_0 \times A_{nb}]$$

Pre tensioned force in bolt (F_0) = (Proof stress) $\times$ (Net area of bolt)

f_0 = Proof stress

$$[f_0 = 0.7 f_{ub}]$$

$$[F_0 = 0.7 f_{ub} \times A_{nb}]$$

μ = Slip factor/frictional resistance coefficient between plates

If not given then generally

$$[\mu = 0.5]$$

n_e = No. of interface having friction

K_h = Hole factor

Perfect hole : $K_h = 1$

Large hole : $K_h = 0.85$

$$\left[V_{dsf} = \frac{\mu_f \cdot n_e \cdot k_h \cdot F_0}{\gamma_{mf}} \right]$$

41.

Sol. Advantages:

- Skilled man power is required.
- Welding is the only process that produces a one piece construction.
- No deductions are there for holes; thus the gross section is effective in carrying loads.
- No requirement of cover plate that means self-weight is reduce.
- Welding offers the best method for achieving a rigid connection [Ex:- Airtight chamber, watertight chamber]

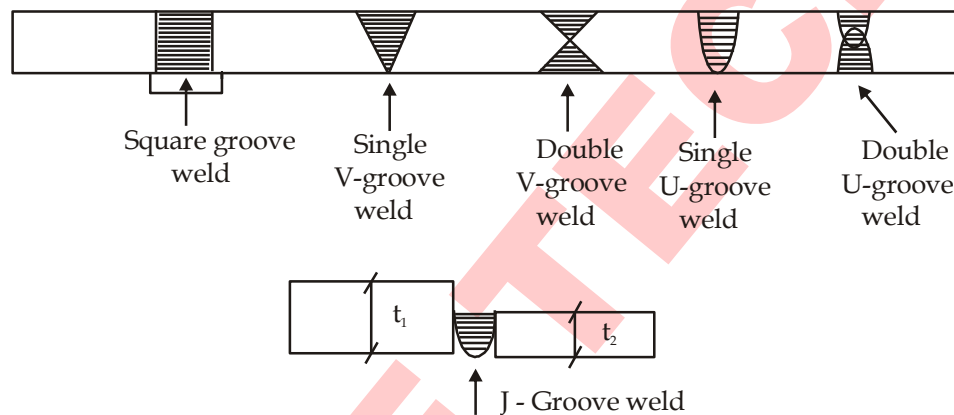
Disadvantages:

- It is affected by the surrounding condition.
- Residual stresses.
- Inspection is difficult

42.

Sol. Design of Butt weld:-

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

**Partially Penetration of weld material**

Single V-Groove
Single U-Groove
J-weld

Fully Penetration of weld material

Double V-Groove
Double U-Groove
Square-weld

Groove welds are usually design for direct tension or compression

Butt weld may be of Partially penetration or of Fully penetration.

- If the thickness of connected element is up to 8 mm, then edge preparation is not required, and weld provide is called square Butt Weld.
- If the thickness of element > 8 mm there is problem of incomplete penetration.
- To rectify this problem edges of plate cut to form grooves and then weld metal is placed therefore butt weld are also called groove weld.
- Size of butt weld specified by the effective throat thickness (t_e)
- As per IS code in case the full penetration of groove weld cannot be achieved an effective throat thicness of $7/8$ th of the thickness of the thinner member should be used. But for calculating the strength, the effective throat thickness is assumed to be $5/8$ th of thickness of the thinner member.
- In case of fully penetration the effective throat thickness is taken as the thickness of the thinner member jointed.

Strength of Butt weld:-

The design strength of the groove weld in tension or compression is governed by yield,

$$\left[T_{dw} = l_w \times t_e \times \frac{f_y}{\gamma_{mw}} \right]$$

f_y = smaller of yield stress of the weld (f_{yw}) and the parent metal (f_y) in MPa

l_w = effective length of the wld in mm

t_e = effective throat thickness of the weld in mm



γ_{mw} = partial safety factor
 = 1.25 for shop welding
 = 1.5 for site welding

The design strength of the butt weld in shear is also governed by yield.

$$\left[V_{dw} = l_w \times t_e \times \frac{f_y}{\sqrt{3} \times \gamma_{mw}} \right]$$

43.

Sol. Permissible shear stress,

$$\tau_{per} = 90 \text{ MPa}$$

Permissible shear stress,

$$\sigma_{br} = 270 \text{ MPa}$$

Nominal diameter of rivet,

$$d = 16 \text{ mm}$$

Gross diameter

$$d' = d + 1.5$$

$$= 17.5 \text{ mm}$$

Rivet is in double shear

Rivet value = Min {strength of rivet in shearing, strength of rivet in bearing}

$$= \text{Min} \left\{ 2 \times \frac{\pi d'^2}{4} \times \tau_{per}, (d' \times t) \sigma_{br} \right\}$$

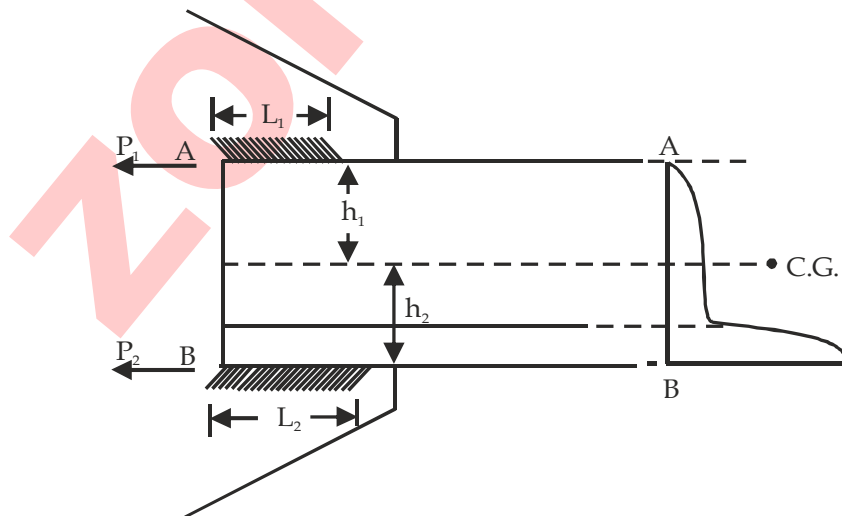
$$= \text{Min} \left\{ 2 \times \frac{\pi \times 17.5^2}{4} \times 90 \text{ N}, 17.5 \times 12 \times 270 \text{ N} \right\}$$

$$= \text{Min} \{ 43.29 \text{ kN}, 56.7 \text{ kN} \}$$

$$= 43.29 \text{ kN}$$

44.

Sol. For a moment free connection CG of the resisting forces must pass through the CG of the member.



Assuming grade of steel E - 250

Assume size of weld = 5 mm (Shop welding) $\gamma_{mw} = 1.25$

$$\text{Strength of the angle section} = \frac{A_g \times f_y}{\gamma_{m0}} = \frac{1908 \times 250}{1.1} = 433.64 \text{ kN}$$

So connection design for 433.64 kN (Strength equal to angle section)
Welding is done along l_1 and l_2

$$\text{So, } \frac{P_1}{P_2} = \frac{h_2}{h_1} \quad \dots(i)$$

(From steel table $C_{xx} = C_{yy} = 28.4 \text{ mm}$)

$$h_2 = 28.4 \text{ mm } h_1 = 100 - 28.4 = 71.6 \text{ mm}$$

$$\therefore P_1 + P_2 = 433.64 \text{ kN} \quad \dots(ii)$$

$$\text{From equation (i) and (ii) we get } \begin{aligned} P_1 &= 123.15 \text{ kN} \\ P_2 &= 310.49 \text{ kN} \end{aligned}$$

$$P_1 = l_{w1} \times t_t \times \frac{f_u}{\sqrt{3} \times \gamma_{mw}}$$

$$123.15 \times 10^3 = l_{w1} \times 0.7 \times 5 \times \frac{410}{\sqrt{3} \times 1.25} \quad \boxed{l_{w1} = 185.81 \text{ mm}}$$

$$P_2 = l_{w2} \times t_t \times \frac{f_u}{\sqrt{3} \times \gamma_{mw}}$$

$$310.49 \times 10^3 = l_{w2} \times 0.7 \times 5 \times \frac{410}{\sqrt{3} \times 1.25} \quad \boxed{l_{w2} = 468.45 \text{ mm}}$$

45.

Sol. 1. Minimum Pitch = 2.5 do

As per Is Code there has to be certain minimum distance b/w to rivet holes to avoid bearing failure of the element b/w the two holes.

2. Maximum pitch (P) maximum

Tension - (16t or 200mm)_{minimum}

Compression - (12t or 200mm)_{minimum}

IS code has limited the value of pitch distance:

1. To decrease the length of connection the size of gusset plate.
2. To have uniform stress variation in the rivet.

46.

Sol. Strength of tension member:

1. For plates and flats

$$(a) \text{ Gross sectional yielding} = T_{dg} = A_g \times \frac{f_y}{\gamma_{m0}}$$

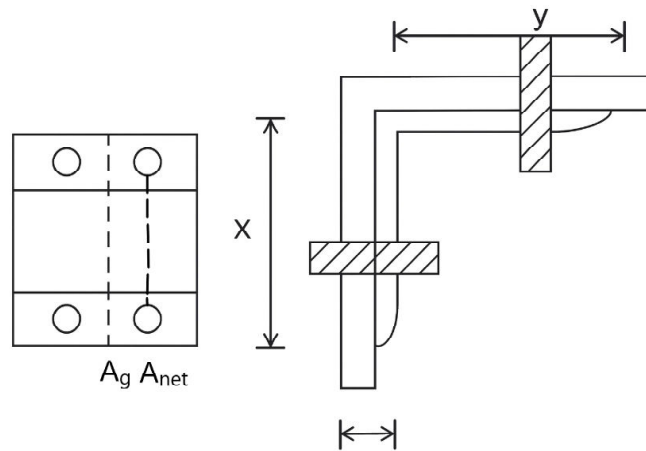
$$\gamma_{m0} = 1.10$$

$$(b) \text{ Net Section Rupture} = T_{du} = 0.9 A_{net} \times \frac{f_u}{\gamma_{m0}}$$

$$\gamma_{m0} = 1.25$$

2. Angle section

Case 1. When both legs are connected



(a) Gross-sectional yielding = $T_{dg} = A_g \times \frac{f_y}{\gamma_{m0}}$

(b) Net Section Rupture = $T_{du} = 0.9 A_{net} \times \frac{f_u}{\gamma_{m0}}$

$$A_g = (x + y - t) \times t$$

$$A_{net} = (x + y - t - 2d) \times t$$

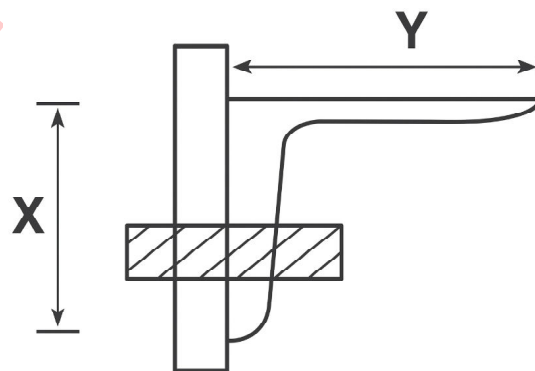
Case 2. When only one leg is connected

(a) Gross-sectional yielding = $T_{dg} = A_g \times \frac{f_y}{\gamma_{m0}}$

(b) Net Section Rupture = $T_{du} = \left(0.9 A_{nc} \frac{f_u}{\gamma_{m1}} \right) + \left(\beta \times A_g \times \frac{f_y}{\gamma_{m0}} \right)$

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

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



47.

Sol. For Fe 410 grade of steel : $f_y = 250$ MPa

Diameter of bolt, $d = 16$ mm

Diameter of bolt hole, $d_h = 16 + 2 + 2 = 20$ mm

Since the bolt holes are punched, the bolt hole diameter has been increased more by 2 mm than the hole diameter given in Table 5.3.

(a) Net area of connected leg

$$= \left(100 - 20 - \frac{10}{2} \right) \times 10 = 750 \text{ mm}^2$$

Net area of outstanding leg

$$= \left(75 - \frac{10}{2} \right) \times 10 = 700 \text{ mm}^2$$

Total net area = $750 + 700 = 1450 \text{ mm}^2$

Since only one leg of the angle is connected, the net area will be reduced depending upon the number of bolts used for making the connection.

(b) Net area of connected leg

$$= \left(100 - \frac{10}{2} \right) \times 10 = 950 \text{ mm}^2$$

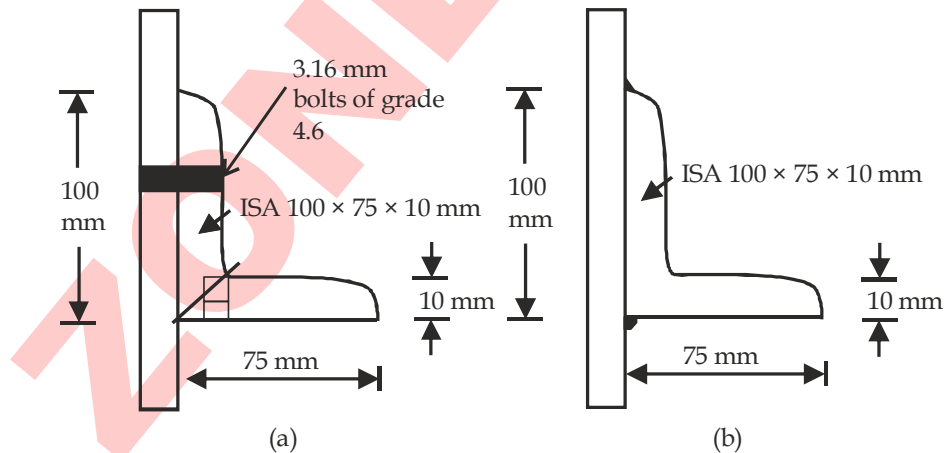
Net area of outstanding leg

$$= \left(75 - \frac{10}{2} \right) \times 10 = 700 \text{ mm}^2$$

Total net area = $950 + 700 = 1650 \text{ mm}^2$

Since only one leg of the angle is connected, the net area will be reduced. For welded connection, the reduction factor is 0.8.

Hence, effective net area, $A_n = 0.8 \times 1650 = 1320 \text{ mm}^2$



48.

Sol. For Fe 410 grade of steel: $f_u = 410$ MPa, $f_y = 250$ MPa

Partial safety factors : $\gamma_{m0} = 1.1$, $\gamma_{m1} = 1.25$

For ISA $150 \times 115 \times 8$ mm: Gross area, $A_g = 2058 \text{ mm}^2$

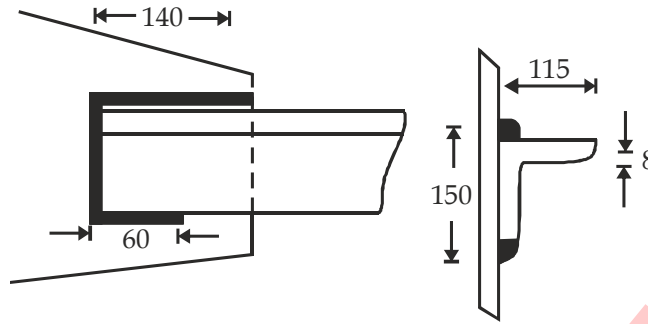
(From IS Handbook No.1)

(a) Tensile strength due to net section yielding,

$$T_{dg} = \frac{A_g f_y}{\gamma_{m0}} = \frac{2058 \times 250}{1.1} \times 10^{-3} = 467.7 \text{ kN}$$

(b) Tensile strength due to net section fracture,

The section is a single angle section ISA 150 × 115 × 8 mm with longer leg welded to the gusset plate; the short leg will be the outstanding leg as shown in Fig Ex. 7.6.



Length of outstanding leg, $w = 115$ mm

Shear leg width, $b_s = w = 115$ mm

Weld length along the direction of load, $L_c = 140$ mm

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

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

$$\geq 0.7$$

Also,
$$\frac{f_u}{f_y} \times \frac{\gamma_{m0}}{\gamma_{m1}} = \frac{410}{250} \times \frac{1.1}{1.25} = 1.44$$

Hence, $\beta = 0.85$

Net area of connected leg,

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

Gross area of outstanding leg,

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

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

$$= \left[0.9 \times \frac{1168 \times 410}{1.25} + 0.85 \times \frac{888 \times 250}{1.1} \right] \times 10^{-3} = 516.34 \text{ kN}$$

49.

Sol. An angle is usually connected to a gusset plate by one leg and tee is connected through its flange distribution on the section. The connected leg will be more highly stressed than the outstanding leg. To account for this non-uniform stress distribution, we can assume the net effective area to be smaller than the actual net area. Indian Standard Code IS : 800 - 1982 recommends the following net effective areas for angles and tees.

Case 1 : In case of single angles in tension, connected by one leg only,

$$\text{Net effective area} = A_1 + k_1 A_2 \quad \dots (i)$$

Where

A_1 = net sectional area of the connected leg

A_2 = area of the outstanding leg.

$$k_1 = \frac{3A}{3A_1 + A_2} \quad \dots(ii)$$

$$A_1 = \left(l_1 - \frac{t}{2} - d_n \right) t$$

$$A_2 = \left(l_2 - \frac{t}{2} \right) \times t$$

d_n = dia. of hole due to rivet.

Case 2: In the case of a pair of angles back to back (or single tee) in tension connected by only one leg of each angle (or by the flange of a tee) to the same side of the gusset,

$$\text{net effective area} = A_1 + k_2 A_2 \quad \dots(iii)$$

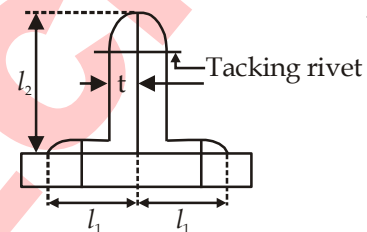
where,

A_1 = Net area of the connected legs (or flange of the tee)

$$k_2 = \frac{5A_1}{5A_1 + A_2} \quad \dots(iv)$$

$$A_1 = 2 \left(l_1 - \frac{t}{2} - d_n \right) t$$

$$A_2 = 2 \left(l_2 - \frac{t}{2} \right) \times t$$

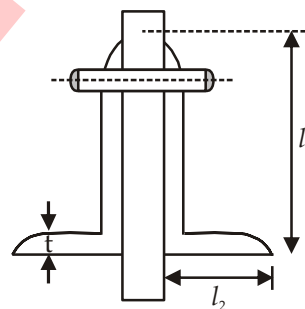


In order that the two angles act as one member, it is necessary to connect them together along their length by tacking rivets. The maximum spacing of tacking rivets shall be 1.0 metre. If tacking rivets or welds are not used, both the angles bend separately and their net effective area should be computed according to Eq. (i)

Case 3: For double angles or tees, carrying direct tension, placed back to back and connected to both sides of the gusset or to both sides of a part of a rolled section, the area to be taken in computing the mean tensile strength shall be the full gross area less the deduction for holes, provided the members are connected together along their length by tacking rivets, the spacing of which shall not exceed 1 m.

$\therefore$ Net effective area = gross area of section - area of holes

If the tacking rivet are not used, each angle or tee will behave separately. Then the effective section of an angle will be computed by Eq. (i) and the effective section of a tee by eq. (iii).



$$A_{net} = 2(l_1 + l_2 - t) \times t - 2d_n t$$

50.

Sol. For Fe 410 grade steel: $f_u = 410$ MPa, $f_y = 250$ MPa Partial safety factor for material : governed by yielding

$$\gamma_{m0} = 1.1$$

Governed by ultimate stress $\gamma_{m1} = 1.25$

Refer to Fig. The shaded area will shear out.

$$A_{vg} = (1 \times 100 + 50) \times 8 = 1200 \text{ mm}^2$$

$$A_{vn} = \left(1 \times 100 + 50 - \left(2 - \frac{1}{2} \right) \times 18 \right) \times 8 = 984 \text{ mm}^2$$

$$A_{tg} = 35 \times 8 = 280 \text{ mm}^2$$

$$A_{tn} = \left(35 - \frac{1}{2} \times 18 \right) \times 8 = 208 \text{ mm}^2$$

The block shear strength will be minimum of T_{db1} and T_{db2} as calculated below.

$$\begin{aligned} T_{db1} &= \frac{A_{vg} f_y}{\sqrt{3} \gamma_{m0}} + \frac{0.9 A_{tn} f_u}{\gamma_{m1}} \\ &= \left[\frac{1200 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 208 \times 410}{1.25} \right] \times 10^{-3} \\ &= 218.86 \text{ kN} \\ T_{db2} &= \frac{0.9 A_{vn} f_u}{\sqrt{3} \gamma_{m1}} + \frac{A_{tg} f_y}{\gamma_{m0}} \\ &= \left[\frac{0.9 \times 984 \times 410}{\sqrt{3} \times 1.25} + \frac{280 \times 250}{1.1} \right] \times 10^{-3} \\ &= 231.34 \text{ kN} \end{aligned}$$

Hence, the block shear strength of the tension member is 218.86 kN.

Note : In the shaded area abcd there are two bolt holes. In calculating the net area along surface ab, there

will be $1\frac{1}{2}$ bolt holes and that along bc only $\frac{1}{2}$ bolt hole.

51.

Sol. For Fe 410 grade steel : $f_u = 410 \text{ MPa}$, $f_y = 250 \text{ MPa}$

Partial safety factors for material : governed by yielding $\gamma_{m0} = 1.1$

Governed by ultimate stress $\gamma_{m1} = 1.25$

Refer to Fig. The shaded area will shear out.

$$A_{vg} = (2 \times 100) \times 10 = 2000 \text{ mm}^2$$

$$A_{vn} = (2 \times 100) \times 10 = 2000 \text{ mm}^2$$

$$A_{tg} = 225 \times 10 = 2250 \text{ mm}^2$$

$$A_{tn} = 225 \times 10 = 2250 \text{ mm}^2$$

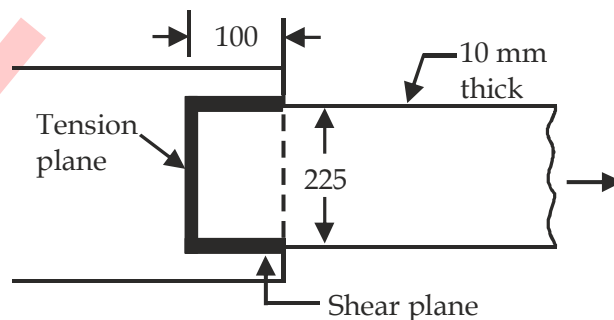


Fig.

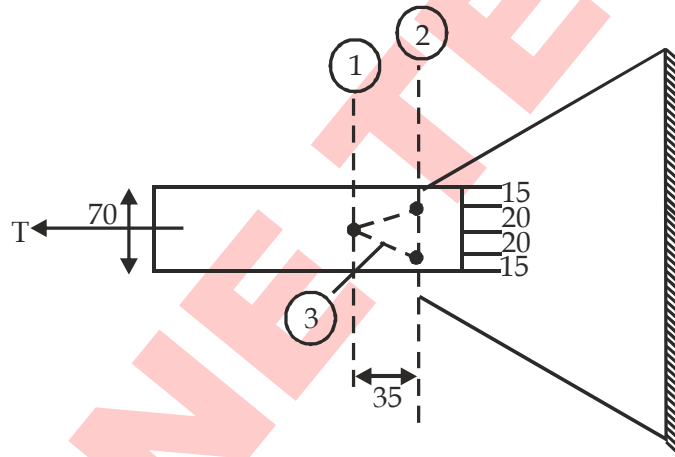
The block shear strength will be minimum of T_{db1} and T_{db2} as calculated below.

$$\begin{aligned} T_{db1} &= \frac{A_{vg} f_y}{\sqrt{3} \gamma_{m0}} + \frac{0.9 A_{tn} f_u}{\gamma_{m1}} \\ &= \left[\frac{2000 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 2250 \times 410}{1.25} \right] \times 10^{-3} \\ &= 926.631 \text{ kN} \end{aligned}$$

$$\begin{aligned} T_{db2} &= \frac{0.9A_{vn}f_u}{\sqrt{3}\gamma_{m1}} + \frac{A_{tg}f_y}{\gamma_{m0}} \\ &= \left[\frac{0.9 \times 2000 \times 410}{\sqrt{3} \times 1.25} + \frac{2250 \times 250}{1.1} \right] \times 10^{-3} \\ &= 852.23 \text{ kN} \end{aligned}$$

Hence, the block shear strength of the tension member is 852.23 kN.

52.
Sol.



$t = 6 \text{ mm}$
 $B = 70 \text{ mm}$
 $d' = 11.5 \text{ mm}$
 $s_t = 150 \text{ MPa}$
 $p = 35 \text{ mm}$
 $g = 20 \text{ mm}$
 Shear capacity of each bolt,

$$F = 15 \text{ kN} \\ = 15000 \text{ N}$$

Maximum force applied on flat = T

Case I : By failure of all the bolts in shear

$$T = 3 \times 15 = 45 \text{ kN}$$

Only option below 45 kN is option a (42.3 kN)

So answer is (a)

Diameter of hole,

$$d' = 11.5 \text{ mm}$$

Case II : By tearing of Flat at section (1)

$$T = (B - d')t$$

$$\begin{aligned}\sigma_t &= (70 - 11.5) \times 6 \times 150 \\ &= 52650 \text{ N} \\ &= 52.65 \text{ kN}\end{aligned}$$

Case III : By tearing of Flat at section (2)

$$\begin{aligned}T &= (B - 2d') \times t \sigma_t + F \\ &= [(70 - 2 \times 11.5) \times 6 \times 150 + 15000] \text{ N} \\ &= 57.3 \text{ N}\end{aligned}$$

Case IV : By tearing of Flat at section (3)

$$\begin{aligned}\text{Net area} &= (B - 3d') \times t + \left(\frac{p^2}{4g} + \frac{p^2}{4g} \right) \times t \\ &= (70 - 3 \times 11.5) \times 6 + \frac{2 \times 35^2}{4 \times 20} \times 6 \\ &= 396.75 \text{ mm}^2 \\ \text{Strength of Flat} &= \text{Net Area} \times \sigma_t \\ &= 396.75 \times 150 \\ T &= 59.512 \text{ kN}\end{aligned}$$

53.

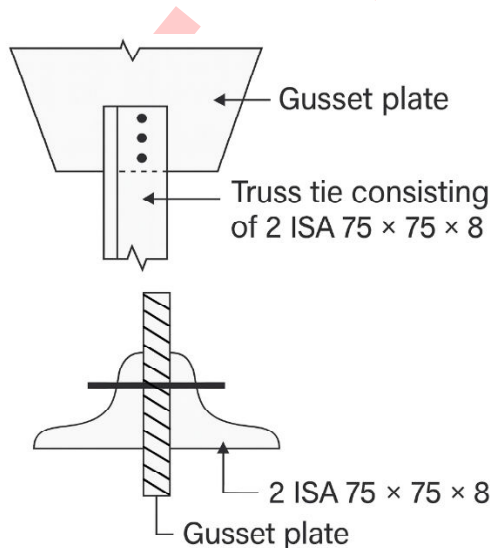
Sol. Diameter of rivet,

$$d = 18 \text{ mm}$$

Gross diameter of rivet,

$$d' = d + 1.5 = 19.5 \text{ mm}$$

In truss tie maximum tensile stress will be at the section where net effective area is minimum. It is minimum at the location of rivet.



For 2 ISA 75 × 75 × 8 mm angles, Area of connected leg of each angle,

$$A_1 = \left(75 - \frac{8}{2} - 19.5 \right) \times 8 = 412 \text{ mm}^2$$

Area of outstanding leg,

$$A_2 = \left(75 - \frac{8}{2} \right) \times 8 = 568 \text{ mm}^2$$

As the angles are not tacked along the length so they will behave individually, hence net area would be twice the area corresponding to single angle connected to gusset plate.

Net effective area of each angle,

$$A = A_1 + kA_2$$

Where,

$$k = \frac{3A_1}{3A_1 + A_2} = \frac{3 \times 412}{3 \times 412 + 568}$$

$$= 0.685$$

$$A = 412 + 0.685 \times 568$$

$$= 801.16 \text{ mm}^2$$

So, effective area of tie

$$= 2A$$

$$= 2 \times 801.16 = 1602.3$$

Maximum tensile stress

$$\frac{\text{Load}}{\text{Area}} = \frac{150 \times 10^3}{1603.39} = 93.6 \text{ N/mm}^2$$

54.

Sol. Generally if not given single, V-butt weld will be assumed to be incomplete penetration butt weld. The complete penetration is mostly ensured in double butt joint

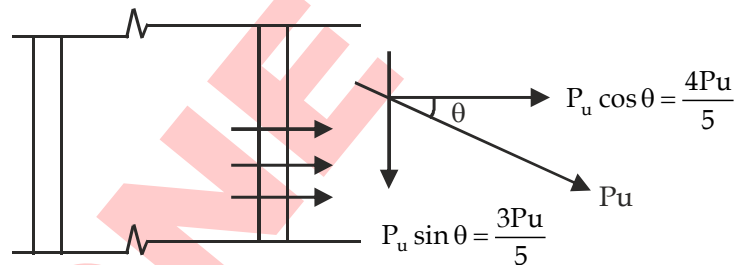
Maximum tension

$$= 150 \times \left(\frac{5}{8} \text{ of thickness of thinner plate} \right) \times \text{permissible stress}$$

$$= 150 \times \frac{5}{8} \times 12 \times 150 = 168.75 \text{ kN}$$

55.

Sol.



$$\tan \theta = \frac{3}{4}$$

$$\cos \theta = \frac{4}{5}$$

$$\sin \theta = \frac{3}{5}$$

$$\text{Tension in each bolt} = \frac{4P_u}{5 \times 6}$$

$$= \frac{4 \times 250}{6 \times 5} = 33.33 \text{ kN}$$

$$\text{Shear in each bolt} = \frac{3P_u}{5 \times 6} = 25 \text{ kN}$$

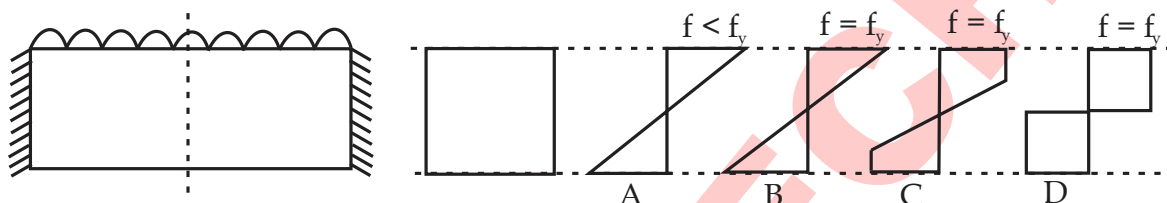
56.

Sol. Assumption:-

1. Plane section before bending remain plane after bending that means strain diagram is linear.
2. The cross section must be symmetrical with respect to plane of loading so that twisting is not developed in the section.
3. Properties of steel in compression and tension are assumed to be same.
4. Cross section should be hot rolled because hot rolled section have sufficient ductility.
5. Every layer or material is free to expand and contract laterally and longitudinally.
6. Steel is the ductile material and able to deform plastically.
7. Stress strain curve is represented by an ideal elasto-plastic material (bi-linear)

57.

Sol. • A plastic hinge can be defined as a yielded zone due to flexure in structure in which infinite rotation can take place at a constant resisting moment M_p of the section.



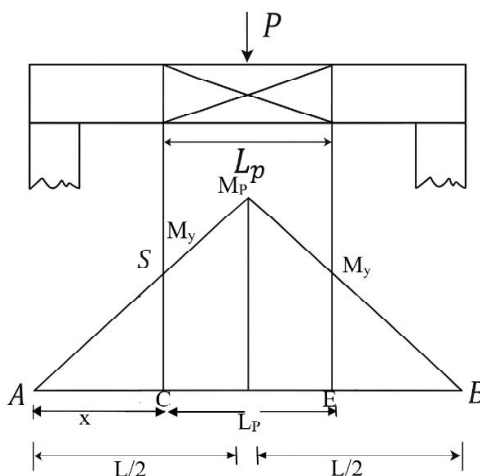
- In the fixed beam shown above when we increase loading gradually, the four conditions of bending stress at middle point of span show the various states of stress viz. yield condition and plastic condition.
- Once the section reaches to stress condition D, the section has lost its capacity to resist any further moment. Hence for moment greater than M_p the section will behave as a hinge. This hinge is called plastic hinge.
- Sometimes, plastic hinge can be thought of as a rusted hinge in which rotation doesn't occur significantly up to certain moment and beyond that large unrestrained deformation i.e., rotation occurs.

58.

Sol. Possible location for plastic hinge:-

- At location of maximum B.M.
- At the fixed support and rigid support.
- Under point load (but not at the free end as in cantilever beam)
- Point where cross section or material change (plastic hinge forms in the weakest portion of the section)

59.

Sol.

$$L = 2x + L_p$$

By similar Δ

$$[L_p = L - 2x]$$

$$\frac{M_p}{M_y} = \frac{L/2}{x}$$

$$L_p = L - 2 \times \frac{L}{2} \times \frac{1}{\text{S.F.}}$$

$$x = \frac{1}{2} \left[\frac{M_y}{M_p} \right]$$

$$\left[L_p = L \left(1 - \frac{1}{\text{S.F.}} \right) \right]$$

$$L_p = \frac{L}{2} \left(\frac{1}{\text{S.F.}} \right)$$

If cross section is Rectangular than L_p for S.S.B. is
S.F. = 1.5

$$L_p = L \left(1 - \frac{1}{\text{S.F.}} \right)$$

$$\left[L_p = \frac{L}{3} \right]$$

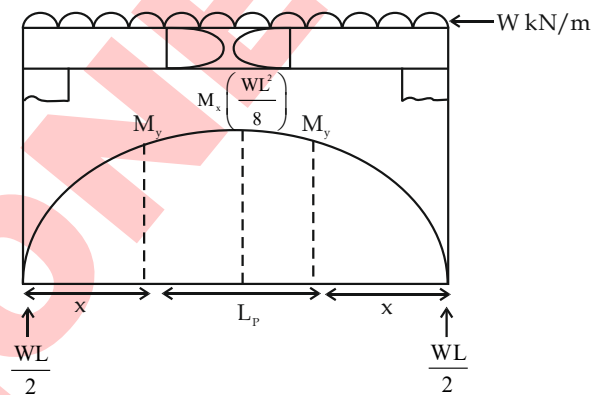
60.

Sol. S.S.B subject to UDL of W kN/m:

$$[L_p = L - 2x] \quad \dots(i)$$

$$M_x = \frac{WL}{2} \times x - W \times x \times \frac{x}{2}$$

$$M_y = M_x = \frac{W}{2} \times x \times (L - x)$$



$$\frac{M_y}{M_p} = \frac{\frac{W}{2} \times x \times (L - x)}{\frac{W \times L^2}{8}}$$

$$\frac{1}{\text{S.F.}} = \frac{4 \times x \times (L - x)}{L^2}$$

Find value of x and put in equation (i)

$$\left[L_p = L \left(\sqrt{1 - \frac{1}{\text{S.F.}}} \right) \right]$$

61.

Sol. Condition in plastic analysis:-

1. **Equilibrium condition:-** When the structure is on the verge of collapse it will satisfy equilibrium condition.
2. **Yield condition:-** At collapse, bending moment at any section should not exceed plastic moment capacity of the section.
3. **Mechanism condition:-** At collapse sufficient number of plastic hinges must be developed so that a part or entire structure must transform into mechanism leading to collapse.

62.

Sol. Theorem of plastic analysis**(a) Lower bound theorem (Static method):-**

- It satisfies equilibrium and yield condition.
- According to this collapse load found by assuming mechanism will always be less than or equal to true collapse load.
- In this method BMD are used for analysis.

(b) Upper bound theorem (Kinematic theorem)

- This method satisfies equilibrium condition and mechanism condition.
- According to this collapse load found by assuming mechanism will always be greater than or equal to true collapse load.

Note:- Uniqueness theorem:-

- If structure satisfies all the three conditions (Equilibrium, yield, mechanism) then the unique value of collapse load is determined.

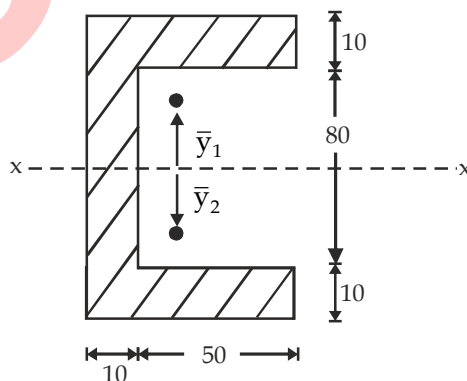
63.

Sol. Let length of weld on either side be 'l' size of weld (S) = 10 mm

$$\begin{aligned} \text{Force resisted by weld} &= \frac{0.75}{1.25} \times \left(\frac{f_u}{\sqrt{3}} \right) \times l \\ &= \frac{0.7 \times 10}{1.25} \times \frac{410}{\sqrt{3}} \times l = 270 \times 10^3 \end{aligned}$$

$$\Rightarrow l = 203.68 \text{ mm}$$

$$\begin{aligned} \therefore \text{Length required on each side} &= \frac{l}{2} = \frac{203.68}{2} \\ &= 101.81 \text{ mm} \\ &= 105 \text{ mm} \end{aligned}$$



Since x-x axis is the axis of symmetry, hence it is both equal area axis and centroidal axis. If $\bar{y}_1, \bar{y}_2$ are the distance of C.G. of area above & below equal area axis then,

$$\bar{y}_2 = \bar{y}_1 = \frac{10 \times 60 \times 45 \times 40 \times 10 \times 20}{1000} = 35 \text{ mm}$$

$$\therefore Z_P = \frac{A}{2} (\bar{y}_1 + \bar{y}_2) = 1000 \times (35 \times 2) = 7 \times 10^4 \text{ mm}^3$$

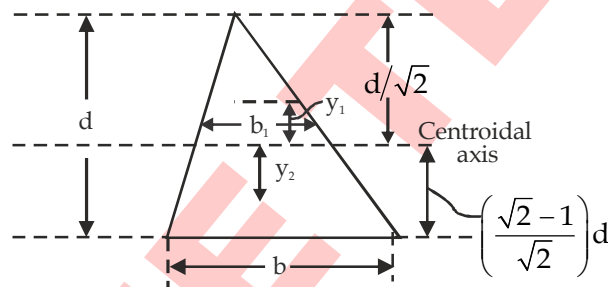
In yield condition, neutral axes is the centroides axes and due the symmetry equal area axes and the centroidal axes will be the same.

$$\begin{aligned} \text{Hence, } I &= \left(\frac{60 \times (50)^3}{3} - \frac{50 \times (40)^3}{3} \right) \times 2 \\ &= 2866666.67 \text{ mm}^4 \end{aligned}$$

$$\therefore Z = \frac{I}{y_{\max}} = \frac{2866666.67}{50} = 57333.33 \text{ mm}^3$$

$$\therefore S(\text{Shaper factor}) = \frac{Z_P}{Z} = \frac{7 \times 10^4}{57333.33} = 1.221 \text{ Ans.}$$

64.
Sol.



$$\frac{b_1}{b} = \frac{d/\sqrt{2}}{d}$$

$$\Rightarrow b_1 = \frac{b}{\sqrt{2}}$$

$$\text{Shape factor, } S = \frac{Z_P}{Z_e} = \frac{\frac{A}{2} (\bar{y}_1 + \bar{y}_2)}{\frac{I}{y}}$$

$$A = \frac{1}{2} b d$$

$$\bar{y}_1 = \frac{1}{3} \frac{d}{\sqrt{2}} = 0.2357 d$$

$$\bar{y}_2 = \frac{2b + b_1}{b + b_1} \left(\frac{\sqrt{2} - 1}{\sqrt{2}} \right) \frac{d}{3} = \frac{2b + b/\sqrt{2}}{b + b/\sqrt{2}} \frac{\sqrt{2} - 1}{3\sqrt{2}} d = 0.1548 d$$

$$Z_P = \frac{b d}{4} (0.2357 d + 0.1548 d) = 0.0976 b d^2$$

$$Z_e = \frac{I}{y} = \frac{bd^3}{36} \times \frac{3}{2d} = \frac{bd^2}{24}$$

$$S = \frac{0.0976bd^2}{bd^2/24} = 2.34$$

65.

Sol. Classification of section:-

1. **Plastic section:-** It develops full plastic moment capacity and has significant rotational capacity.
2. **Compact section:-** It develops plastic hinge but has inadequate plastic hinge rotation capacity to form mechanism.
3. **Semi-compact section:-** These are those sections which can develop moment equal to yield moment capacity and then it fail by local buckling failure.
4. **Slender section:-** Those section which cannot even develop moment equal to yield moment capacity and then it fail by local buckling failure.

66.

Sol. Types of load on roof truss:-

1. **Dead load:-** A simple empirical formula for the estimation of approximate dead weight of roof truss is given by

$$\left(\frac{\text{span}}{3} + 5 \right) \times 10 \frac{\text{N}}{\text{m}^2}$$

2. **Live load:-** IS : 875 specify the following live loads to be assumed in the analysis of an industrial building.

Roof slope	Access	Live load
$\leq 10^\circ$	Provided	1.5 kN/m ² of plan area
	Not Provided	0.75 kN/m ² of plan area
$> 10^\circ$		For roof membrane sheets or purlins 0.75 kN/m ² less 0.02 kN/m ² for every degree increase in slope over 10° $\nless 0.4$ kN/m ²

3. **Wind load:-**

- The most critical load on an industrial building is the wind load. Wind load depends on various factor such as wind velocity, density of air, height of structure, topography of the surrounding ground surface and angle of wind attack.
- The most important parameter in quantify the wind load is the wind speed.
- The basic wind speed for any site can be obtain from wind map at 10 m height.

$$[V_{dz} = K_1 \cdot K_2 \cdot K_3 \cdot K_4 \cdot V_b]$$

V_{dz} → Design wind speed at (z) m height

K_1 → Risk factor/Probability factor

K_2 → Terrian roughness and height of structure

K_3 → Topography factor

K_4 → Importance factor for cyclonic region

V_b → Basic wind speed in m/sec

Design wind pressure (P_z) = $0.6 (V_{dz})^2$

67.

Sol. Economical spacing of truss:-

- The roofing system cost of industrial building depends upon- Cost of roof truss, cost of roof covering material, cost of purlins
Let the cost of roofing system = x
The cost of roof truss = t
The cost of purlin = P
The cost of roof covering material = r
 $[x = t + P + r]$

- Relation between spacing and cost of roof truss

$$t \propto \frac{1}{S}$$

$$\left[S = \frac{C_1}{t} \right]$$

- Relation between spacing and cost of purlin

$$S^2 \propto P$$

$$P = C_2 S^2$$

- Relation between spacing and cost of covering material

$$r \propto S$$

$$r = C_3 S$$

$$x = \frac{C_1}{S} + C_2 S^2 + C_3 S$$

For cost to be minimum $\frac{\partial x}{\partial S} = 0$

$$\frac{dx}{dS} = -\frac{C_1}{S^2} + 2C_2 S + C_3 = 0$$

$$2C_2 S + C_3 = \frac{C_1}{S^2}$$

$$2C_2 S^3 + C_3 S^2 - C_1 = 0$$

$$2C_2 S^2 + C_3 S - \frac{C_1}{S} = 0$$

$$2P + r - t = 0$$

$$[t = 2P + r]$$

68.

Sol. The connections are classed as flexible, rigid or semi-rigid. Flexible connections are assumed to take no moments while rigid connections are assumed to carry full moments. However, no connection can be 100 per cent flexible or rigid; all the connections fall in the semi-rigid category. To make it convenient from design point of view, it is generally agreed that the fixed supports would develop about 90 per cent or more of the fixed end-moment and that the flexible connection would not develop more than about 25 per cent of moment. Figure 1 shows the classification of building connections based on their moment-rotation characteristics.

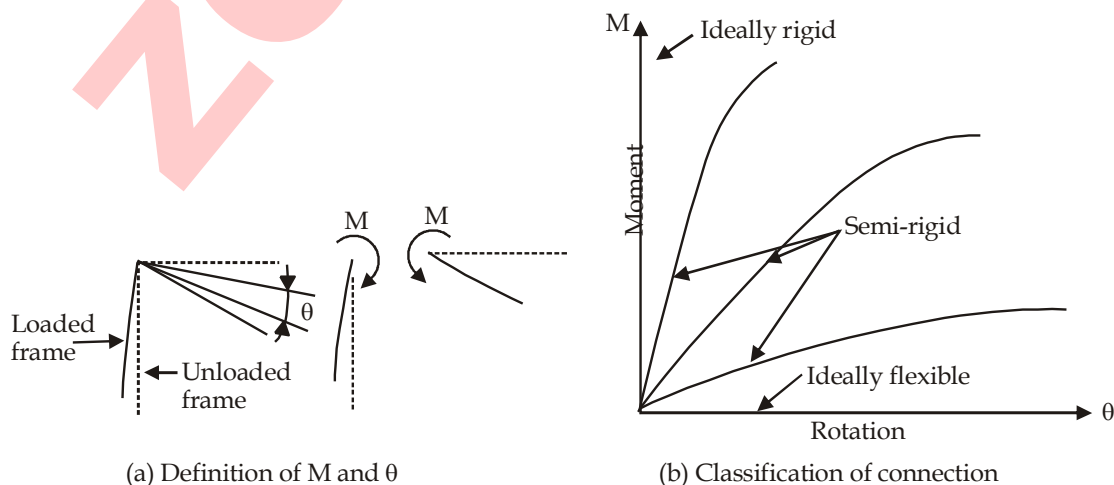
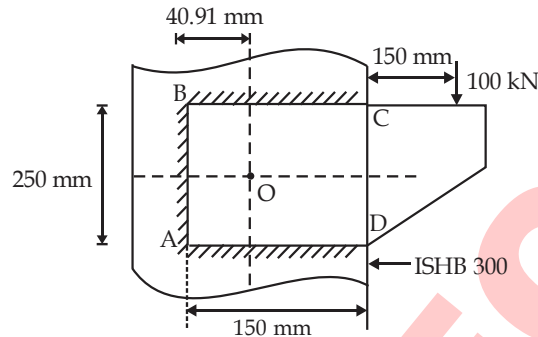


Fig.: Theoretical connection characteristics

PART - C

20 Marks Questions

69.
Sol.



Allowable stress in the weld is 110 N/mm^2

Let the size of weld = S

$\therefore$ Effective throat thickness = $0.7S = t$

$$\text{Shear stress due to direct loading } (f_d) = \frac{\text{Direct load}}{\text{Area}} = \frac{100 \times 10^3}{(2 \times 150 + 250) \times t}$$

$$\therefore f_d = \left\{ \frac{100 \times 1000}{550 \times t} \right\}$$

and

$$f_d = \frac{181.82}{t} \text{ N/mm}^2$$

Shear stress due to torsional moment (f_t),

$$f_t = \frac{(Pe).r}{J}$$

Step 1: Locate the centroid: (consider A as origin)

$$\bar{x} = \frac{250 \times t \times 0 + (150 \times t \times 75) \times 2}{250t + (150t) \times 2} = 40.91 \text{ mm}$$

From symmetry, $\bar{y}$ will be midway between AD & BC

Step 2: Determination of $e = 300 - 40.91 = 259.09 \text{ mm}$

Step 3: Determination of $r_{\max}$ i.e., OC or OD in our case,

$$OC = \sqrt{125^2 + (150 - 40.91)^2} = 165.91 \text{ mm}$$

Step 4: Calculation of J ,

$$J = I_{xx} + I_{yy}$$

$$I_{xx} = t \times \frac{250^3}{12} + 2 \times 150 \times t \times 125^2 = 5t(5989583.33) \text{ mm}^4$$

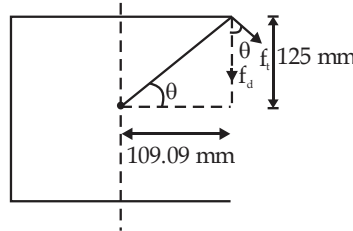
$$I_{yy} = 250 \times t \times 40.91^2 + \frac{2 \times t \times 150^3}{12} + 2 \times 150 \times t \times (75 - 40.91)^2$$

$$= t(1329545.455) \text{ mm}^4$$

$$\therefore J = t(7319128.785) \text{ mm}^4$$

$$\therefore f_t = \frac{P \times e \times r}{J} = \frac{100 \times 259.09 \times 165.91 \times 10^3}{(7319128.785)t} = \frac{587.30}{t} \text{ N/mm}^2$$

For finding the resultant force we have to find the angle between f_t and f_d .



$$\tan \theta = \frac{125}{109.09}$$

$\Rightarrow$

$$\cos \theta = 0.6575$$

$\therefore$

$$\begin{aligned} f_R &= \sqrt{f_d^2 + f_t^2 + 2f_d \cdot f_t \cos \theta} \\ &= \frac{1}{t} \sqrt{(181.82)^2 + (587.3)^2 + 2(181.82)(587.3) \times 0.6575} \\ &= \frac{720}{t} \text{ N/mm}^2 \end{aligned}$$

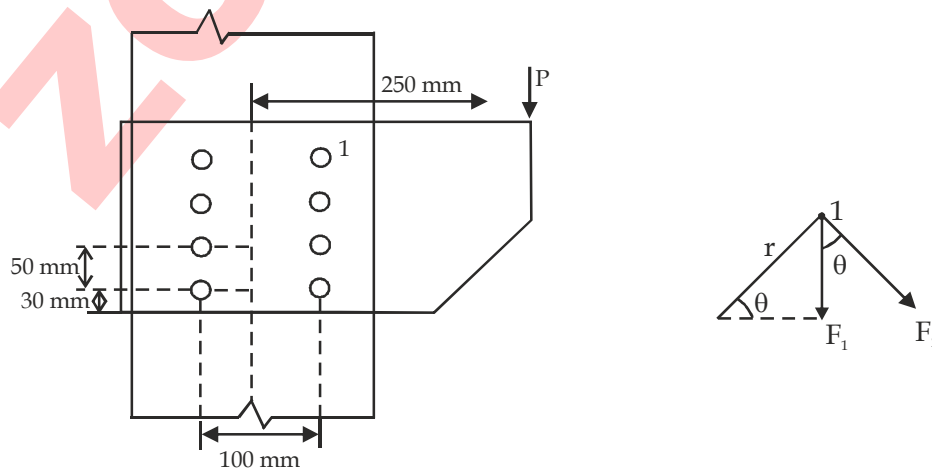
For section to be safe

$$\begin{aligned} \Rightarrow \quad \frac{720}{t} &\leq 110 \\ \Rightarrow \quad t &\geq 6.545 \text{ mm} \\ \Rightarrow \quad 0.7s &\geq 6.545 \\ s &\geq 9.35 \text{ mm} \end{aligned}$$

Let us adopt size of weld as 10 mm.

70.

Sol.



Given that ISMB 500,

M16 bolts, Grade of bolt = 4.6

Pitch, $p = 50$ mm and end distance, $e = 30$ mm

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

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

Diameter of bolt, $d = 16$ mm.

Diameter of hole, $d_o = d + 2$

$d_o = 16 + 2 = 18$ mm

$$\text{Net area of bolt} = 0.78 \times \frac{\pi}{4} \times d^2 = 0.78 \times \frac{\pi}{4} \times 16^2 = 156.83 \text{ mm}^2 \approx 157 \text{ mm}^2$$

Critical bolt should be number 1

Strength of bolt in single shear :

$$V_{dsb} = A_{nb} \times \frac{f_{ub}}{\sqrt{3} \gamma_{mb}} = 157 \times \frac{400}{\sqrt{3} \times 1.25} = 29 \times 10^3 \text{ N} = 29 \text{ kN}$$

Strength of bolt bearing :

$$V_{dpb} = 2.5 k_b d t_{\min} \frac{f_u}{\gamma_m}$$

Diameter of hole = $d + 2 = 16 + 2 = 18$ mm

$$k_b = \text{minimum of} \left[\frac{e}{3d_o}, \frac{p}{3d_o} - 0.25, \frac{f_{ub}}{f_u}, 1.0 \right]$$

$$= \text{Minimum of} \left\{ \frac{30}{3 \times 18}, \frac{50}{3 \times 18} - 0.25, \frac{400}{410}, 1.0 \right\}$$

$$= \text{minimum} [0.56, 0.67, 0.976, 1]$$

$$\boxed{k_b = 0.56}$$

$$V_{dpb} = 2.5 \times 0.56 \times 16 \times 10 \times \frac{410}{1.25} = 73.472 \times 10^3 \text{ N}$$

$$= 73.47 \text{ kN}$$

Hence strength of bolt = 29 kN

Let Q be the factored load

$$\text{Safe load (P)} = \frac{Q}{1.5}$$

The number of bolt = 8

$$\text{The direct force} = \frac{Q}{8} = F_1$$

$$\text{The force in the bolt due to torque (F}_2\text{)} = \frac{Qer}{\Sigma r^2}$$

$$\text{radius (r)} = \sqrt{(75)^2 + (50)^2} = 90.14 \text{ mm}$$

$$\Sigma r^2 = 4 \times (\sqrt{75^2 + 50^2})^2 + 4 \times (\sqrt{25^2 + 50^2})^2$$

$$= 45000 \text{ mm}^2$$

$$F_2 = \frac{Q \times 250 \times 90.14}{45000} = 0.5Q$$

$$\tan \theta = \frac{75}{50} \Rightarrow \theta = 56.3^\circ$$

The resultant force (F_R) = $\sqrt{F_1^2 + F_2^2 + 2F_1F_2 \cos \theta}$

$$= \sqrt{\left(\frac{Q}{8}\right)^2 + (0.5Q)^2 + 2 \times \frac{Q}{8} \times 0.5Q \times \cos 56.31}$$

$$= 0.579Q$$

$$F_R \leq \text{Strength of bolt}$$

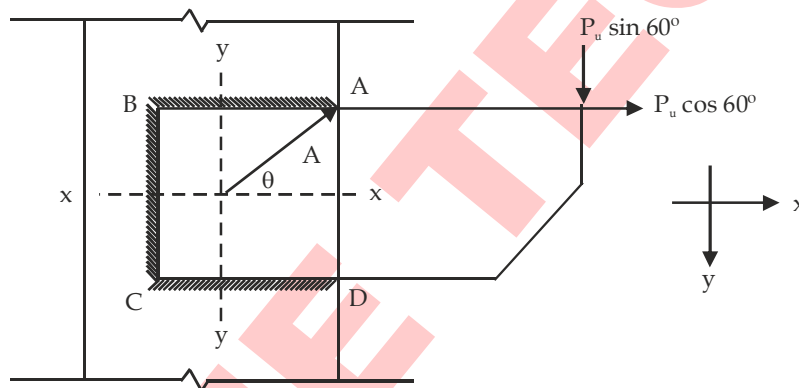
$$0.579Q \leq 29 \text{ kN}$$

$$Q \leq 50.08 \text{ kN}$$

$$\text{Safe load} = \frac{Q}{1.5} = \frac{50.08}{1.5} = 33.39 \text{ kN}$$

71.

Sol. The loading P_u can be resolved as shown below.



Most critical location for the weld will be point A where angle between torsional shear stress and direct shear stress is minimum and torsional shear stress has maximum value.

Thus, we will calculate resultant shear stress at A and by limiting it to be less than the max possible shear stress, value of size of weld can be calculated.

Let us calculate the C.G. of weld ($\bar{x}$)

$$\bar{x} = \frac{2(200 \times t) \times 100 + 0}{700t} = \frac{400}{7} = 57.143 \text{ mm} \quad [t = \text{Throat thickness of weld}]$$

$$r_A = \sqrt{(200 - 57.143)^2 + (150)^2} = 207.143 \text{ mm}$$

$$\theta = \tan^{-1} \left(\frac{50}{200 - 57.143} \right) = 46.397^\circ$$

f_{dx} = Direct shear stress in x direction

$$= \frac{P_u \cos 60^\circ}{700t} \quad [t = \text{Throat thickness in mm}]$$

$$= \frac{100 \cos 60^\circ \times 10^3 \text{ N}}{700t \text{ mm}^2} = \frac{500}{7t} \text{ N/mm}^2 = \frac{71.429}{t} \text{ N/mm}^2$$

f_{dy} = Direct shear stress in y direction

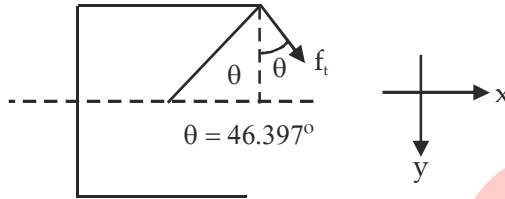
$$f_t = \text{Torsional shear stress} = \frac{T r_A}{J}$$

$$T = P_u \sin 60^\circ \times (100 + 200 - 57.1432) + P_u \cos 60^\circ \times 150$$

$$= 28532.033 \text{ kNmm} = 28.532 \text{ kN/m}$$

$$J = I_{xx} + I_{yy}$$

$$= \frac{t(300)^3}{12} + 2(200 \times t)(150)^2 + 300 \times t(57.143)^2$$



$$+ 2 \times \frac{t(200)^3}{12} + 2 \times 200 \times t(100 - 57.143)^2$$

$$= 14297619.05t \text{ mm}^4$$

$$f_t = \frac{28.532 \times 10^6 \text{ Nmm} \times 207.143 \text{ mm}}{14297619.05t} = \frac{413.909}{t}$$

$$f_{tx} = f_t \sin \theta = \frac{413.909}{t} \times \sin 46.397 = \frac{299.726}{t} \text{ N/mm}^2$$

$$f_{ty} = \frac{413.909}{t} \times \cos \theta = \frac{413.909}{t} \times \cos 46.397^\circ = \frac{285.455}{t}$$

$$f_x = f_{dx} + f_{tx} = \frac{71.429 + 299.726}{t} = \frac{371.155}{t} \text{ N/mm}^2$$

$$f_y = f_{dy} + f_{ty} = \frac{123.718 + 285.455}{t} = \frac{409.173}{t} \text{ N/mm}^2$$

$$\text{Resultant shear stress} = \sqrt{f_x^2 + f_y^2} = \frac{552.43}{t} \text{ N/mm}^2$$

$$\text{or safety of connection resultant shear stress} < \frac{f_u}{\sqrt{3} \times \gamma_{mw}}$$

$$\Rightarrow \frac{552.43}{t} < \frac{410}{\sqrt{3} \times 1.25} \quad [\text{for shop weld } \gamma_{mw} = 1.25]$$

$$t > \frac{552.43 \times \sqrt{3} \times 1.25}{410}$$

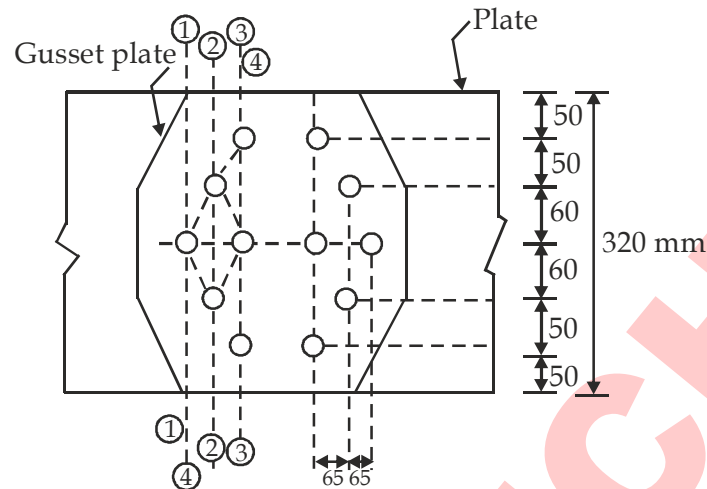
$$t > 2.917 \text{ mm}$$

$$\Rightarrow 0.7 S > 2.917 \text{ mm}$$

$$S > 4.167 \text{ mm}$$

Adopt size of weld as 5 mm.

72.
Sol.



$d = 25 \text{ mm}$; $d_0 = 28 \text{ mm}$; $t = 16 \text{ mm}$; $f_u = 440 \text{ MPa}$; $f_y = 290 \text{ MPa}$;
 $P = 65 \text{ mm}$; $e = 50 \text{ mm}$; $g_1 = 50 \text{ mm}$; $g_2 = 60 \text{ mm}$

Tensile strength of section will be minimum of :

- (i) $6 \times \text{bolt value}$
- (ii) Strength of plate in gross section yielding
- (iii) Strength of plate in net section rupture.

(i) Bolt value = min of $\begin{cases} \text{Shear strength of bolt, } V_{ds} \\ \text{bearing strength of bolt, } V_{dpb} \end{cases}$

$$V_{dsb} = \frac{f_u}{\sqrt{3} \gamma_{m1}} \times (A_{nb} \times n_b \times A_{sb} \times n_s)$$

$$\gamma_{m1} = 1.25$$

$$A_{nb} = \text{Area of bolt at thread} = 0.78 \times \frac{\pi}{4} \times d^2$$

$$A_{sb} = \text{Area of bolt at shaft} = \frac{\pi}{4} \times d^2$$

n_n, n_s = no. of shear plane through thread & shaft, respectively

Note : Taking the joint as double cover bolt joint.

Assuming $n_s = 1$ and $n_n = 1$

$$\therefore V_{dsb} = \frac{440}{\sqrt{3} \times 1.25} \left(1 \times 0.78 \times \frac{\pi}{4} \times 25^2 + 1 \times \frac{\pi}{4} \times 25^2 \right) \text{ N} = 177.57 \text{ kN}$$

$$V_{bpb} = 2.5 \frac{f_u}{\gamma_{m1}} k_b d t$$

$$K_b = \min \left[\frac{e}{3d_0}, \frac{P}{3d_0} - 0.25, \frac{f_{ub}}{f_u}, 1.0 \right]$$

$$= \min \left[\frac{50}{3 \times 28}, \frac{65}{3 \times 28} - 0.25, \frac{440}{440}, 1.0 \right]$$

$$= \min \text{ of } [0.60, 0.52, 1.0, 1.0] = 0.52$$

$$V_{dpb} = 2.5 \times \frac{440}{1.25} 0.52 \times 25 \times (16) \text{ N} = 183.04 \text{ kN}$$

$$\therefore \text{ Bolt value} = 177.57 \text{ kN} = V_{db}$$

$$6 \times \text{bolt value} = 6 \times 177.57 = 1065.42 \text{ kN}$$

(ii) Strength of plate in gross section yielding

$$f_{yg} = \frac{f_y}{\gamma_{m0}} \times A_g = \frac{290}{1.1} \times 320 \times 16 \text{ N}$$

$$f_{yg} = 1349.82 \text{ kN}$$

(iii) Strength of plate in net section rupture, f_{un}

(a) Strength of main plate at section (1)-(1)

$$= 0.9 \frac{f_u}{\gamma_{m1}} A_{net}$$

$$= 0.9 \times \frac{440}{1.25} \times [(320 - 28) \times 16] \text{ N} = 1480.09 \text{ kN}$$

(b) Strength of main plate at section (2)-(2)

$$= 0.9 \times \frac{440}{1.25} \times [(320 - 2 \times 28) \times 16] + V_{db}$$

$$= 1338.16 + 177.57 = 1515.73 \text{ kN}$$

$$> 1480.09 \text{ kN}$$

Since, strength of main plate at (2)-(2) > at (1)-(1)

hence, strength of main plate at (3)-(3) will be > at (2)-(2)

(c) Strength of gusset plate at section (3)-(3)

$$= 0.9 \times \frac{440}{1.25} \times [(320 - 3 \times 28) \times 16 \times 10^{-3}] + 3 \times 177.57$$

$$= 1728.95 \text{ kN}$$

(d) Strength of main plate at section (4)-(4)

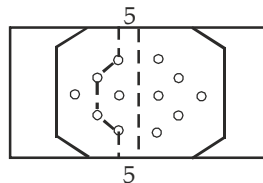
$$= 0.9 \times \frac{440}{1.25} \times \left[\left(320 - 3 \times 28 + \frac{65^2}{4 \times 50} + \frac{65^2}{4 \times 60} \right) \times 16 \right] \text{ N}$$

$$= 1392.547 \text{ kN}$$

(e) Strength of main plate at section (5)-(5)

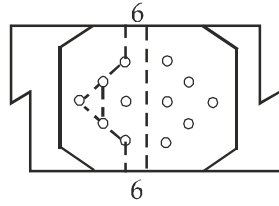
$$= 0.9 \times \frac{440}{1.25} \left[320 - 4 \times 28 + \left(\frac{65^2}{4 \times 50} \right) \times 2 \right] \times 16 \times 10^{-3} + 1 \times 177.57$$

$$= 1446 \text{ kN}$$



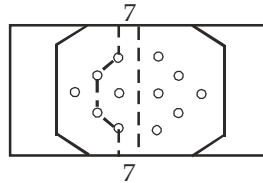
(f) Strength of main plate at section (6)-(6)

$$= \frac{0.9 \times 440}{1.25} \left[320 - 5 \times 28 + 2 \left(\frac{65^2}{4 \times 50} + \frac{65^2}{4 \times 60} \right) \right] \times 16 \times 10^{-3} = 1305 \text{ kN}$$



(g) Strength of main plate for section (7)-(7)

$$= \frac{0.9 \times 440}{1.25} \left[320 - 5 \times 28 + 2 \left(\frac{65^2}{4 \times 50} + \frac{65^2}{4 \times 60} \right) \right] \times 16 \times 10^{-3} + 177.57 = 1482.57 \text{ kN}$$

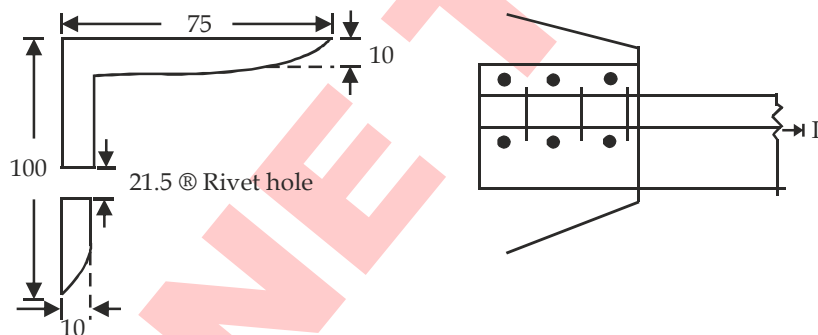


Strength of joint = 1065.42 kN (Governed by strength of bolts)

73.

Sol. In this problem force for which design is to be done is not given hence, we will design it for maximum force that main member can resist.

The main member will be reduced by minimum of one rivet hole, as shown in figure below:



$$A_1 = \left(100 - \frac{10}{2} - 21.5 \right) \times 10 = 735 \text{ mm}^2$$

$$A_2 = \left(75 - \frac{10}{2} \right) \times 10 = 700 \text{ mm}^2$$

$$F_{\text{design}} = \frac{150 \times (735 + 700)}{1000} = 215.25 \text{ kN}$$

$$\therefore \text{Force in outstanding leg of main angle} = \frac{F_{\text{design}} \times A_2 (\text{without hole})}{A_1 (\text{without hole}) + A_2 (\text{without hole})}$$

$$= \frac{215.25 \times (75 - 5) \times 10}{(100 + 75 - 10) \times 10} = 91.318 \text{ kN}$$

[This is an approximate method to calculate the force in outstanding leg]

Lug angle and its connection with the gusset plate is designed for force 1.2 times the force in outstanding leg.

Hence design force for lug angle = $1.2 \times 91.318 = 109.58 \text{ kN}$

Rivets connecting to lug angle with the outstanding leg of main member will be designed for force
 $= 1.4 \times 91.318 = 127.84 \text{ kN}$

Section of lug angle required

$$A_{\text{net required}} = \frac{109.58 \times 1000}{150} = 730.53 \text{ mm}^2$$

Net area provided by ISA $60 \times 60 \times 8$ is taken as gross area minus deduction due to one hole. This is because at one section lug angle will be reduced by one rivet hole.

Net area Provided $= 896 - 21.5 \times 8 = 724 \text{ mm}^2 < 730.53 \text{ mm}^2$ (ie $A_{\text{net req}}$)

Hence this angle is not safe for design.

Gross area provided by ISA $60 \times 60 \times 10 = 1100 \text{ mm}^2$

Assuming one hole reduction, net area provided $= 1100 - 21.5 \times 10 = 885 \text{ mm}^2 > 730.53 \text{ mm}^2$

Hence we adopt Lug angle as ISA $60 \times 60 \times 10$

Designing of rivet

Rivet value calculation

$$\text{Shearing strength in single shear} = \frac{\pi}{4} \times 21.5^2 \times 100 = 36.305 \text{ kN}$$

$$\text{Bearing strength} = d' \times t \times \sigma_{br} = 21.5 \times 10 \times 300 = 64.5 \text{ kN}$$

⇒ Rivet value (Smaller of shearing & bearing strength of Rivet) $= 36.305 \text{ kN}$

This rivet value can be used for all connections because everywhere there is single shear and governing thickness for bearing strength calculation is also 10 mm everywhere.

(1) Number of rivets used for connecting the lug angle to the main plate

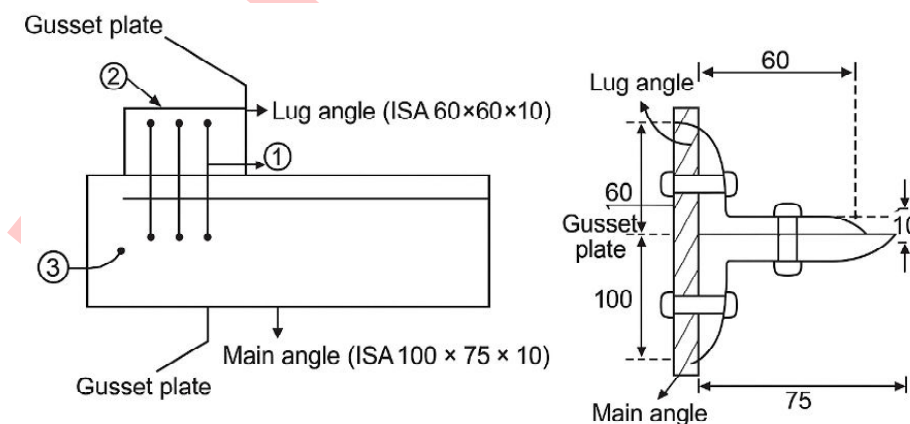
$$= \frac{127.84}{36.305} = 3.52 \cong \text{Adopt } 4$$

(2) Number of rivets used for connecting the lug angle to the gusset plate

$$= \frac{109.50}{36.305} = 3.018 \cong \text{Adopt } (4)$$

(3) Number of rivets used for connecting the main plate to the gusset plate

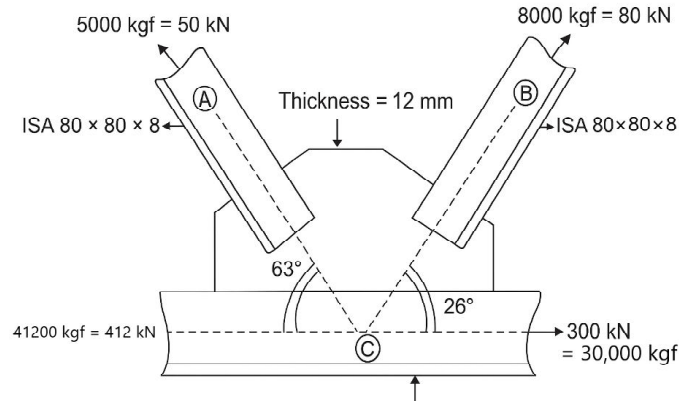
$$= \frac{215.25 - 91.318}{36.305} = 3.41 \cong \text{adopt } (4)$$



It is important to note while drawing the diagram that the rivet connecting outstanding leg of main member with the lug angle (i.e. rivet 1) should start in advance of all other rivets. This will ensure that forces are transferred to the lug angle effectively.

74.

Sol.

2 ISA $\times$ 115 $\times$ 8 (Back to back)

$$\sigma_s^{\text{rivet}} = 102.5 \text{ N/mm}^2$$

$$\sigma_s^{\text{rivet}} = 236 \text{ N/mm}^2$$

Nominal diameter of rivet = 20 mm

Gross diameter of rivet = 21.5 mm

$$\therefore \text{Shearing strength of one rivet in single shear} = \frac{\pi}{4} \times 21.5^2 \times \frac{102.5}{1000} = 37.21 \text{ kN}$$

$$\text{and bearing strength of one rivet} = \frac{236 \times 8 \times 21.5}{1000} = 40.59 \text{ kN}$$

Hence rivet value of rivets used for section A and B is = 37.21 kN

$$\text{Hence member of rivets required for section A} = \frac{50}{37.21} = 1.34 \approx 2 \text{ rivets}$$

$$\text{and number of rivets required for section B} = \frac{80}{37.21} = 2.149 \approx 3 \text{ rivets}$$

For section C, rivet value will be changed because the rivets are in double shear and the thickness of thinner member will be 12 mm,

$$\text{Shear strength of one rivet in double shear} = \frac{\pi}{4} \times 21.5^2 \times 2 \times \frac{102.5}{1000} = 74.42 \text{ kN}$$

$$\text{Bearing strength of one rivet} = \frac{236 \times 21.5 \times 12}{1000} = 60.89 \text{ kN}$$

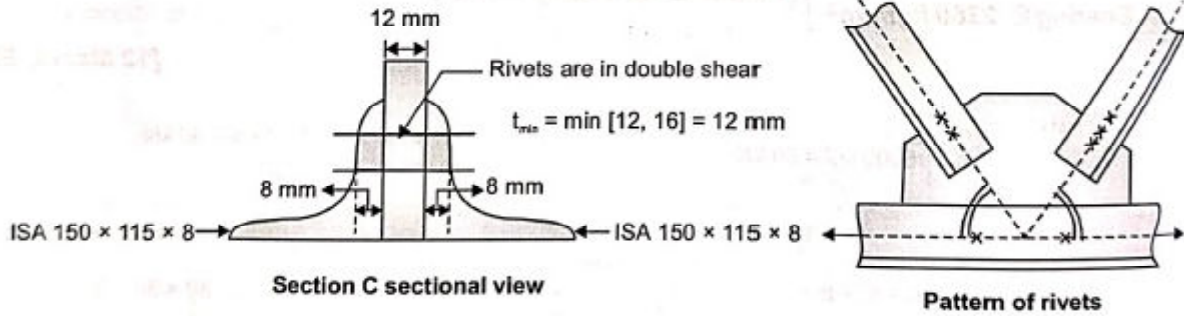
Hence rivet value of rivet attached in section (C) = 60.89 kN

$$\therefore \text{Number of rivet required} = \frac{412 - 300}{60.89} = 1.83 @ 2$$

$$\text{Pitch of rivet} = 2.5 \times d_m = 2.5 \times 20 = 50 \text{ mm}$$

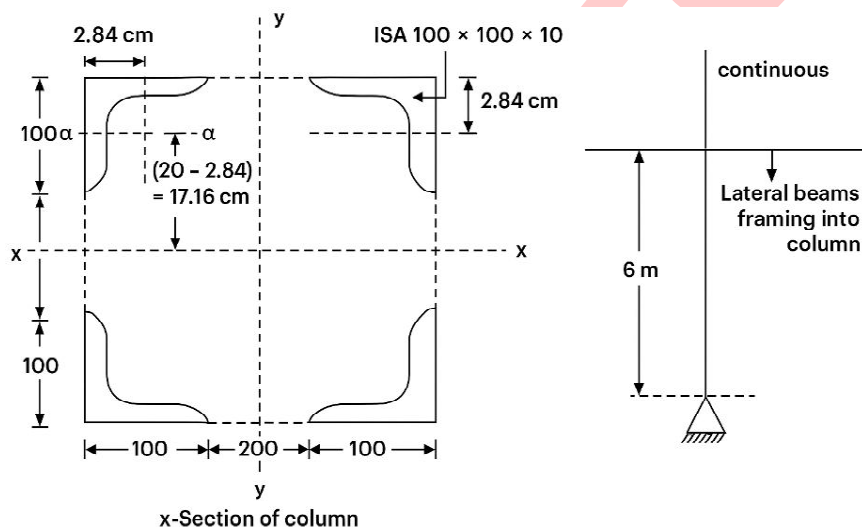
$$\text{Edge distance} = 2 \times d_n = 2 \times 21.5 = 43 \text{ mm}$$





As the size of angle is 150×115 mm hence if longer leg is connected leg, two rows of rivets can be attached on the longer leg (length being larger than 125 mm).

75.
Sol.



To calculate the max load that the column can support, we assume the continuous end to the fixed due to heavy beam and column framing into each other at the continuous end.

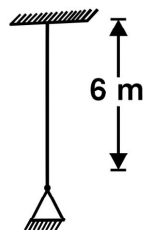
Thus effective length of column is corresponding to the column support condition shown below.

$l_{eff} = 0.8l$ (as per is code recommendation)

Axial load carrying capacity $= \sigma_{ac} \cdot A$

Area $= 4 \times 19.03 \text{ cm}^2 = 76.12 \text{ cm}^2$

σ_{ac} is a function of slenderness ratio (λ)



$$\text{Slenderness ratio} = \frac{l_{eff}}{r_{min}}$$

Where, $r_{\min}$ = Minimum radius of gyration

Minimum radius of gyration is about xx or yy because xx and yy axis are the axis of symmetry and hence are the principal axes. As the x-x and y-y axis are the axis of symmetry, $r_{xx} = r_{yy}$, $r_{\min} = r_{\max}$.

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

$$I_{xx} = 4 \times [I_{xx} + A_{\text{one}} (17.16)^2]$$

$$= 4 [177 + 19.03 (17.16)^2] \text{ cm}^4$$

$$I_{xx} = 23122.72 \text{ cm}^4$$

$$r_{xx} = \sqrt{\frac{23122.72}{4 \times 19.03}} \text{ cm} = 17.429 \text{ cm}$$

$$\Rightarrow \lambda = \frac{l_{\text{eff}}}{r_{\min}} = \frac{l_{\text{eff}}}{r_{xx}} = \frac{0.8 \times 600}{17.429} = 27.54$$

$$\Rightarrow \sigma_{ac} = 148 - \frac{3 \times 7.54}{10} = 145.738 \text{ N/mm}^2$$

$$\Rightarrow \text{Max load carrying capacity} = \sigma_{ac} \times A = 145.738 \times 4 \times 19.03 \times 10^2 \text{ N}$$

$$= 1109.357 \text{ kN}$$

Design of slab base

As the loading is axial, slab base will also be symmetrical

$$\text{Area of base plate required} = \frac{\text{Load}}{\text{Bearing capacity of concrete}}$$

Let us increase the load by 10% to account for the D.L. of the structure

$$\Rightarrow \text{Area required} = \frac{1.1 \times 1109.357 \text{ kN}}{3 \times 10^6 \text{ N/m}^2} = \frac{1.1 \times 1109.357 \times 10^3 \text{ N}}{3 \times 10^6 \text{ N/m}^2}$$

$$\Rightarrow B \times B = (0.638)^2 \text{ m}$$

$$\Rightarrow B = 0.638 \text{ m}$$

Adopt $B = 0.65 \text{ m}$

To calculate the thickness of base plate we use the formula

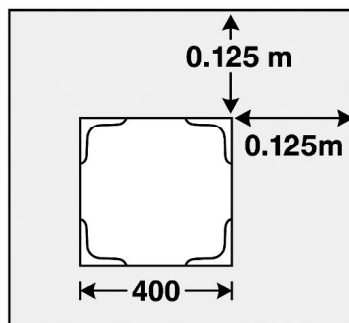
$$t = \sqrt{\frac{3W}{\sigma_{bs}} \left(a^2 - \frac{b^2}{4} \right)}$$

$$\text{Where, } w = \text{base pressure} = \frac{1.1 \times 1109.357 \times 10^3 \text{ N}}{(0.65 \times 10^3) \text{ mm}^2} = 2.89 \text{ N/mm}^2$$

$$\sigma_{bs} = 185 \text{ N/mm}^2$$

$$a = \text{Larger overhang} = 0.125 \text{ m}$$

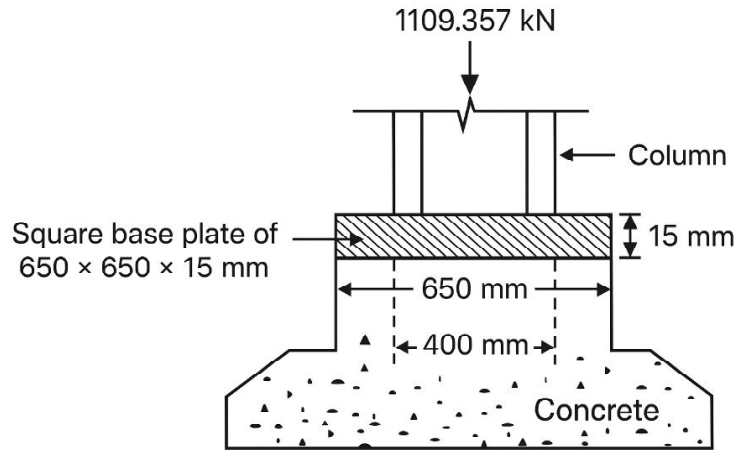
$$b = \text{Smaller overhang} = 0.125 \text{ m}$$



$$\Rightarrow t = \sqrt{\frac{2.89}{185} (0.125)^2 \times \frac{3}{4}}$$

$$t = 0.01353 \text{ m} = 13.53 \text{ mm}$$

let us adopt $t = 15 \text{ mm}$

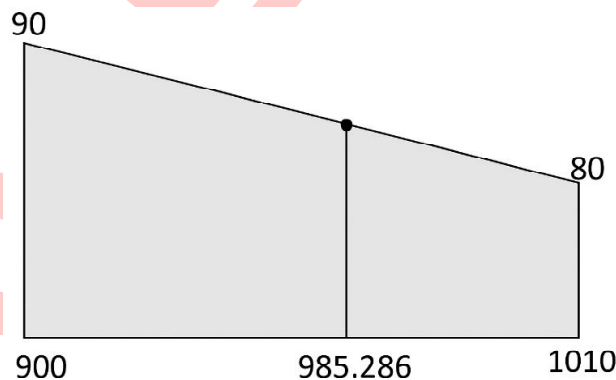


76.

Sol. Given loading for which we have to design the compression in member is $(P) = 75000 \text{ kg}$
 Area of section provided $= 4 \times 19.03 \times 100 = 7612 \text{ mm}^2 = 76.12 \text{ cm}^2$

$$\sigma_{ac} = \frac{P}{\text{Area}} = \frac{75000}{76.12} \text{ kg} = 985.286 \text{ kg / cm}^2$$

Hence slenderness ratio (λ) can be determined by inter polation as follows



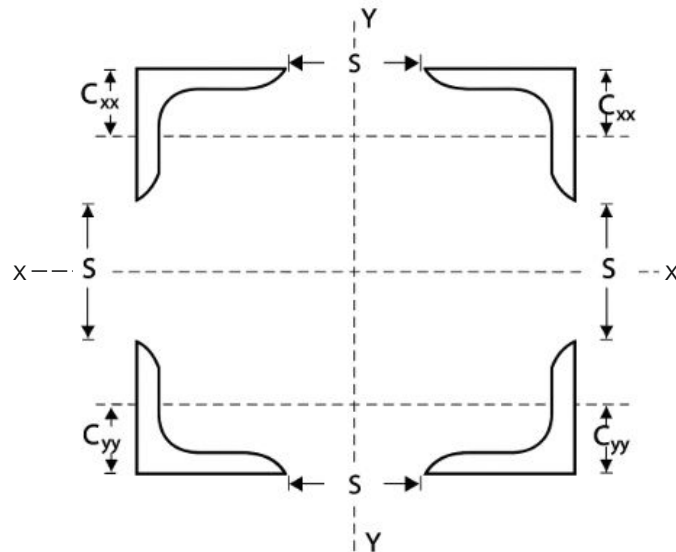
$$\lambda = 80 + \frac{90 - 80}{1010 - 900} \times (1010 - 985.286) = 82.247$$

Effective length of column $= 9.4 \text{ m}$

$$\therefore r_{\min} = \frac{l_{\text{eff}}}{\lambda} = \frac{9.4 \times 1000}{82.247} = 114.29 \text{ mm}$$

$$\therefore I_{\min} = A r_{\min}^2 = 4 \times 1903 \times 114.29^2 = 99.4295 \times 10^6 \text{ mm}^4$$

Let the four angles are arranged in such a way that the spacing (s) between them is equal on all the four faces of the column as shown below



For most efficient column section,

$$I_{xx} = I_{yy} = I_{\min}$$

$$\Rightarrow I_{xx} = 4 \left[177 \times 10^4 + 1903 \times \left(100 + \frac{S}{2} - C_{xx} \right)^2 \right]$$

$$\Rightarrow 99.4295 \times 10^6 = 4 \left[177 \times 10^4 + 1903 \left(100 - 28.4 + \frac{S}{2} \right)^2 \right]$$

$$\Rightarrow 99.4295 \times 10^6 = 7080000 + 7612 (71.6 + S/2)^2$$

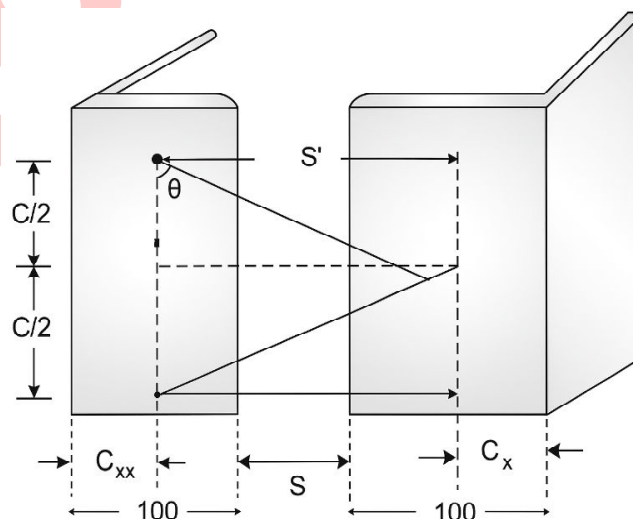
$$\therefore S = 77.09 \text{ mm}$$

$$\therefore \text{Adopting spacing} = 80 \text{ mm}$$

$$\therefore S = 80 \text{ mm}$$

Design of lacing

Assuming the rivets are connected at the C.G. of angles



Assuming

$$\theta = 45^\circ$$

And as,

$$S' = 2(100 + S/2 - C_{xx}) = 2(100 + 40 - 28.4) = 232.2 \text{ m}$$

$$\therefore \tan \theta = \frac{S'}{C/2} \Rightarrow C = S' \times 2 = 2 \times 223.2 = 446.4 \text{ mm}$$

$$\Rightarrow \frac{C}{r_{\min} \text{ of single section}} = \frac{446.4}{r_w} = \frac{446.44}{19.4} = 23.01 \nless 50 \text{ (OK)}$$

and

$$0.7 \lambda_{\text{whole}} = 0.7 \times 82.247 = 57.573$$

$$\Rightarrow 23.01 > 57.513 \text{ (OK)}$$

Hence single lacing is sufficient at an angle with vertical equal to 45° and at a spacing of 445 mm. Although the problem is complete at this stage, for illustration purpose, Let us design the lacing.

Assuming diameter of rivet to be used = 20 mm

Hence, Width of lacing (b) = 60 mm

$$\text{and Effective length (L)} = \frac{S'}{\sin \theta} = \frac{S'}{\frac{1}{\sqrt{2}}} = \sqrt{2} S' = \sqrt{2} \times 223.2 = 315.60 \text{ mm}$$

$$\text{Hence Thickness of lacing bar (t)} = \frac{L}{40} = 7.89 \text{ mm}$$

Thus adopt thickness of bar = 8 mm

Check for slenderness ratio of lacing

$$\text{Slenderness ratio of lacing } (\lambda) = \frac{l_{\text{eff}}}{\left\{ \frac{t}{\sqrt{12}} \right\}} = \frac{315.60}{\left(\frac{8}{\sqrt{12}} \right)} = 136.33 \nless 145 \text{ (Hence OK)}$$

calculation of Force for which lacing is to be designed

$$F = \frac{V}{2 \sin \theta} = \frac{2.5P}{200 \sin \theta} = \frac{2.5 \times P \times \sqrt{2}}{200} = \frac{2.5 \times 75000 \times \sqrt{2}}{200} = 1325.8 \text{ kg}$$

Check that lacing can sustained the compressive force F or not:

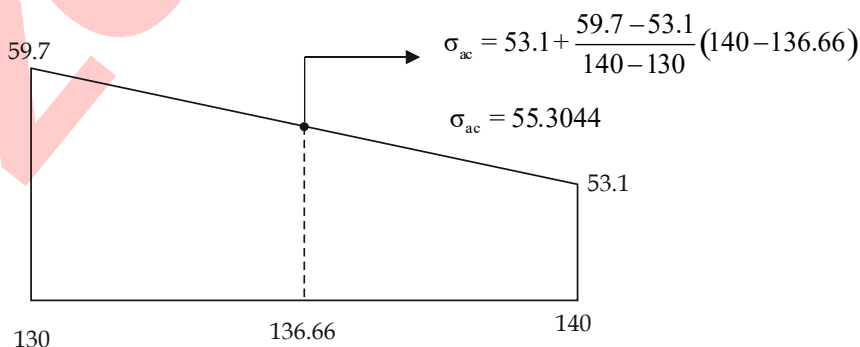
$[(\lambda_{\text{ac}})_{\text{of lacing}} \times (A_{\text{gross}})_{\text{lacing}}]$ should be greater than F

$$\lambda_{\text{lacing}} = 136.66$$

Hence by interpolation assuming that for $\lambda = 130$, $\sigma_{\text{ac}} = 59.7 \text{ N/mm}^2$ and for

$$\lambda = 140,$$

$$\sigma_{\text{ac}} = 53.1 \text{ N/mm}^2$$



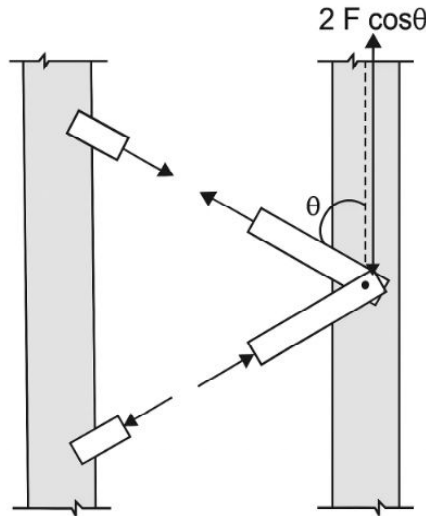
$$\frac{(55.3044) \times 60 \times 8}{1000} = 26.546 \text{ kN} > 13.26 \text{ kN (Hence OK)}$$

Check that lacing is safe for tension or not

$$(B - d) \times t \times \sigma_{\text{at}} > F$$

Connection design

$$\Rightarrow \frac{(60 - 21.5) \times 8 \times 0.6 \times 250}{1000} = 46.2 > 13.26 \text{ kN (Hence OK)}$$



$$\text{number of rivets} = \frac{2F \cos \theta}{R_v}$$

Determination of Rivet value (R_v)

$$\text{shearing strength} = \frac{\pi}{4} \times 21.5^2 \times 100 = 36.30 \text{ kN}$$

$$\therefore \text{Bearing strength} = \sigma_{br} \times t \times d_h = 300 \times 8 \times 21.5 = 51.6 \text{ kN}$$

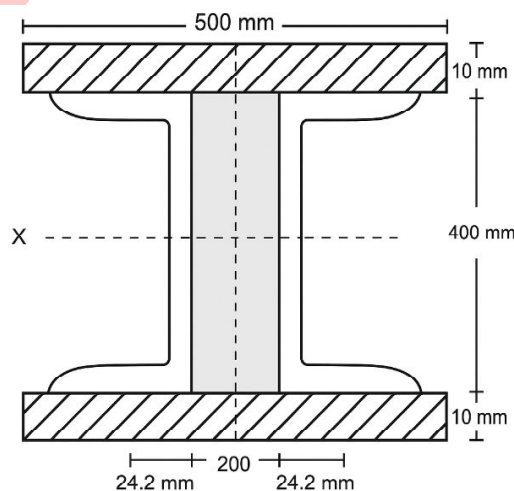
$$R_v = 36.30 \text{ kV.}$$

$$\therefore \text{Number of rivet} = \frac{2 \times 13.26 \times 1}{36.30 \times \sqrt{2}} = 0.516$$

Hence one rivet is sufficient.

77.

Sol. The cross-section of the column can be shown in figure below,



- The safe axial load carrying capacity can be determined as,

$$P = \sigma_{ac} \times A_{gross}$$
 where P = Load carrying capacity of column
 σ_{ac} = Permissible axial compressive stress in column section
 A_{gross} = Gross Area of column section.
- Area of column section

$$= 2 \times [\text{Area of channel} + \text{Area of plate}]$$

$$= 2 \times [6293 + 500 \times 10] = 22586 \text{ mm}^2$$
- σ_{ac} depends upon slenderness ratio which in turn depends r_{min} . Hence the problem reduces to findings out r_{min} of the section, when the section has axis of symmetry then it is well known that the minimum M.O.I will be either about the axis of symmetry or, perpendicular to the axis of symmetry. Hence in this case we will have to calculate I_{xx} and I_{yy} for obtaining the minimum M.O.I.

$$I_{xx} = (2 \times 150828 \times 10^3) + \left[\frac{500 \times 10^3}{12} + 500 \times 10(200 + 5^2) \right] \times 2$$

$$I_{xx} = 721989333.3 \text{ mm}^4 = 721.99 \times 10^6 \text{ mm}^4$$

$$I_{yy} = [2 \times 5048 \times 10^3 + 2 \times 6293 \times 124.2^2] + \frac{10 \times 500^2}{12} \times 2$$

$$I_{yy} = 412576438.4 \text{ mm}^4 = 412.576 \times 10^6 \text{ mm}^4$$

$$\therefore I_{yy} < I_{xx}$$

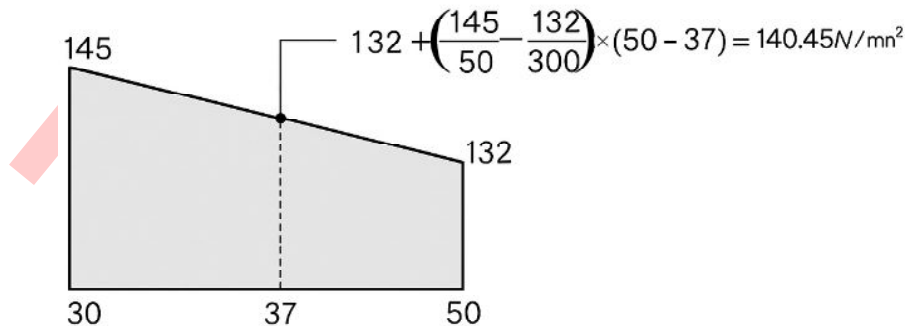
$$r_{min} < \sqrt{\frac{I_{min}}{A}} = \sqrt{\frac{I_{yy}}{A}} = \sqrt{\frac{412.576 \times 10^6}{(2 \times 6293 + 2 \times 500 \times 10)}} = 135.155 \text{ mm}$$

$$\therefore I_{eff} = 5\text{m} = 5000 \text{ mm (given)}$$

$$\therefore \lambda \text{ (slenderness ratio)} = \frac{l_{eff}}{r_{min}} = \frac{5000}{135.155} = 36.99 \approx 37 < 180$$

From the given relationship between λ and σ_{sc} we can interpolate the value of σ_{ac} when $\lambda = 37$

λ	$\sigma_{ac} \text{ (N/mm}^2\text{)}$
30	145
50	132
37	?



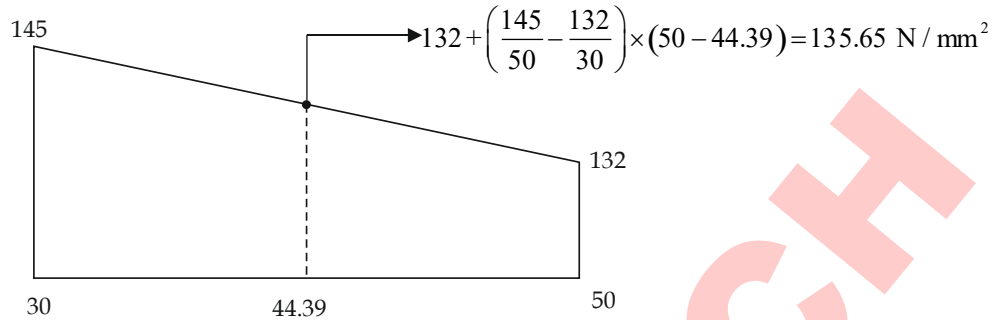
$$P = \sigma_{ac} \times A = \frac{140.45 \times 22586}{1000} = 3172.2 \text{ kN}$$

Thus safe load carrying capacity = 3172.2 kN
 when the effective length (I_{eff}) = 6 m

$$\lambda \text{ (slenderness ratio)} = \frac{6000}{135.15} = 44.39$$

By interpolation,

λ	$\sigma_{ac} \text{ (N/mm}^2\text{)}$
30	145
50	132
44.33	-



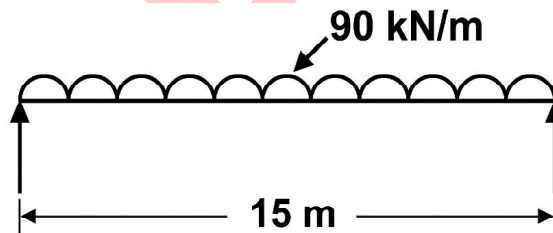
$$P = \sigma_{ac} \times A = \frac{135.65 \times 22586}{1000} = 3063.71 \text{ kN Sol.}$$

78.

Sol. Uniformly distributed load inclusive of self wt. = 60 kN/m
Factored udl = $1.5 \times 60 = 90 \text{ kN/m}$

$$\text{Max. Factored BM} = M = \frac{90(15)^2}{8} = 2531.25 \text{ kN/m}$$

$$\text{Max. Factored V} = \frac{90 \times 15}{2} = 675 \text{ kN}$$



$$Z_p \text{ (Plastic modulus) required for trial section} = \frac{M}{f_y / \gamma_{mo}} = \frac{2531.25 \times 10^6 \text{ Nmm}}{\frac{250}{1.1} \text{ N/mm}^2} = 11137.5 \text{ cm}^3$$

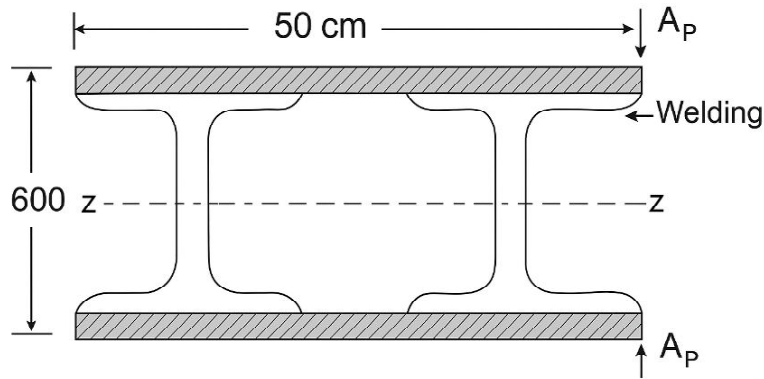
Assume laterally restrained beam condition.

As the Z_p value for the steel section is not given, let us assume the shape factor to be 1.1

$$Z_e \text{ for ISWB 600} = \frac{106198.5}{30} = 3539.95 \text{ cm}^3$$

$$\text{Approx } Z_p = 3539.95 \times 1.1 = 3893.945 \text{ cm}^3$$

Let us use two ISWB 600 section along with plate.



$$\begin{aligned} \text{Area of plate required } A_p &= \frac{Z_{\text{preq}} - Z_{\text{p rolled}}}{h} \\ &= \frac{11137.5 - 2 \times 3893.945}{60} = 55.826 \text{ cm}^2 \end{aligned}$$

Let us adopt 20 mm thickness plate

$$\Rightarrow \text{Width of plate required} = \frac{55.826}{2.0} = 27.913 \text{ cm}$$

Adopt width of plate as 50 cm

Hence plate dimension is 50 cm × 20 mm

As the width (b_f), (t_w), (t_f) etc are not given, classification of the section is not possible.

Let us assume the beam section to be plastic section.

Check for shear

$$V_d = \left[\frac{f_y}{\sqrt{3} \times \gamma_{mo}} \times ht_w \right] \times 2 \quad [\text{Shear area} = \text{web area}]$$

$$= \frac{250}{\sqrt{3} \times 1.1} \times 600 \times 11 \times 2 \quad [t_w \text{ has not been given, we have assumed it to be 11 mm}]$$

$$= 1732.05 \text{ kN}$$

$$V < V_d \quad \Rightarrow \text{Beam is safe in shear.}$$

$$0.6 V_d = 1732.05 \times 0.6 = 1039.23 \text{ kN}$$

$$V = 675 \text{ kN}$$

$$\Rightarrow V < 0.6 V_d \quad \Rightarrow \text{It is a low shear case.}$$

Note :

$$\frac{d}{t_w} = \frac{600 - (t_f + R)}{t_w}$$

Neglecting $t_f + R$ [t_f = thickness of flange R = radius of fillet]

$$\frac{d}{t_w} = \frac{600}{11} = 54.54 < 67\epsilon \quad \left[\epsilon = \sqrt{\frac{250}{f_y}} = 1 \right]$$

$$\Rightarrow \text{Check for web crippling not required}$$

Check for Bending

$$M_d = \beta_b \cdot Z_p \cdot \frac{f_y}{\gamma_{mo}}$$

$$< 1.2 \frac{Z_e f_y}{\gamma_{mo}}$$

$\beta_b = 1$ (because plastic section has been assumed)

$$Z_p \text{ of the built up section} = Z_{p_{\text{rolled}}} + A_p \cdot h = 2 \times 3893.945 + 50 \times 2.0 \times 62 \\ = 13987.89 \text{ cm}^3$$

$$Z_e \text{ for the built up section} = \frac{I_{zz}}{32 \text{ mm}}$$

$$= \frac{2 \times 106198.5 + 2 \times \left[\frac{50(2)^3}{12} + 50 \times 2(31)^2 \right]}{32} = \frac{404663.67}{32} \\ = 12645.74 \text{ cm}^3$$

$$\beta_b \cdot Z_p \cdot \frac{f_y}{\gamma_{mo}} = \left[1 \times 13987.89 \times 10^3 \text{ mm}^3 \times \frac{250}{1.1} \text{ N/mm}^2 \right] \times 10^{-6} \text{ kNm} \\ = 3179.065 \text{ kNm}$$

$$1.2 \times Z_e \cdot \frac{f_y}{\gamma_{mo}} = 1.2 \times 12645.74 \times 10^3 \times \frac{250}{1.1} \times 10^{-6} \text{ kNm} = 3448.84 \text{ kN}$$

$$\Rightarrow M_d = 3179.06 \text{ kNm} \\ M = 2531.25 \text{ kNm}$$

Since $M < M_d$

$\Rightarrow$ Section is safe in bending.

Check for deflection

Moment of inertia about bending axis $I = 404663.67 \text{ cm}^4$

$$\text{Max. deflection} = \frac{5}{384} \cdot \frac{wl^4}{EI}$$

$$= \frac{5}{384} \times \frac{60 \frac{\text{N}}{\text{mm}} \times (15000)^4 \text{ mm}^4}{2 \times 10^5 \times \frac{\text{N}}{\text{mm}^2} \times 404663.67 \times 10^4 \text{ mm}^4} \\ = 48.87 \text{ mm}$$

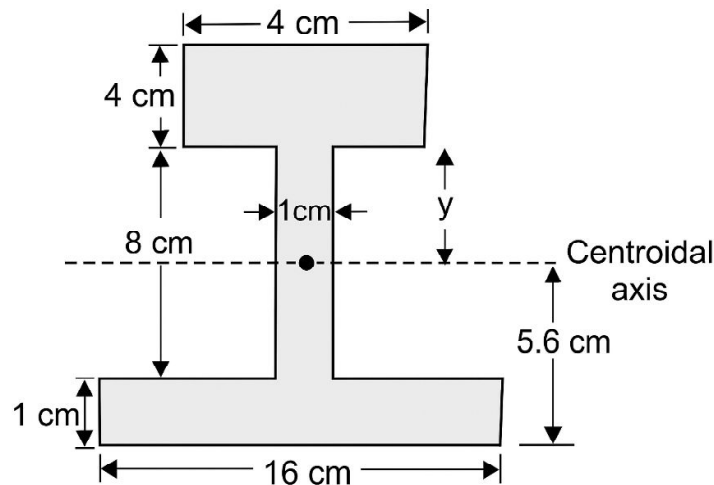
$$\text{Permissible deflection} = \frac{\text{span}}{300} = \frac{15000}{300} = 50 \text{ mm}$$

$\Rightarrow$ Max. deflection $<$ Permissible deflection.

$\Rightarrow$ Section is safe in deflection.

79.

Sol.



- Shape factor is defined as the ratio of plastic moment and the yield moment of the section or, as the ratio of plastic modulus to elastic section modulus.
- It depends upon shape of section.
- It signifies the reserve strength available in the section from yielding to fully plastic state.

$$S \text{ (shape factor)} = \frac{M_p}{M_y} = \frac{f_y Z_p}{f_y Z} = \frac{Z_p}{Z}$$

Neutral axes determination

For plastic state, Equal area axis will be the neutral axis

Hence, $(4 \times 4) + (1 \times y) = 16 \times 1 + 1 \times (8 - y)$

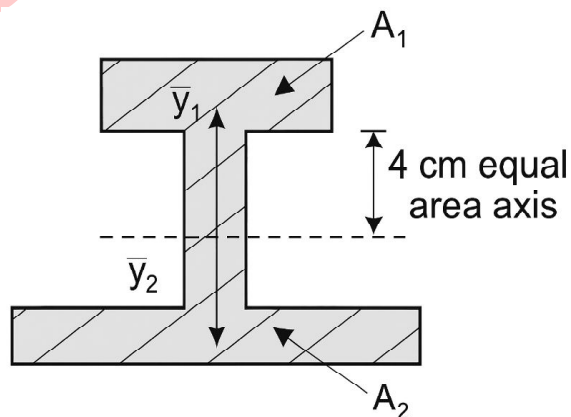
$$\Rightarrow 16 + y = 16 + 8 - y$$

$$\Rightarrow 2y = 8 \quad \therefore y = 4 \text{ cm}$$

$$\therefore Z_p = A_1 \bar{y}_1 + A_2 \bar{y}_2$$

$$A_1 \bar{y}_1 = 4 \times 4 \times (4 + 2) + 4 \times 1 \times 2 = 104 \text{ cm}^3$$

$$A_2 \bar{y}_2 = 16 \times 1 \times 4.5 + 4 \times 1 \times 2 = 80 \text{ cm}^3$$



$$\Rightarrow Z_p = 184 \text{ cm}^3$$



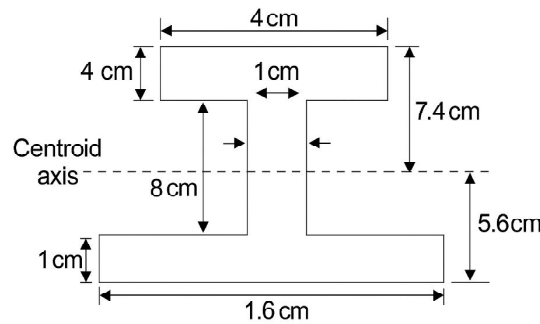
For yielding, centroidal axis will be the neutral axis,

$$I_{c.g.} = \frac{4(7.4)^3}{3} - \frac{3 \times (7.4 - 4)^3}{3} + \frac{16(5.6)^3}{3} - \frac{(16 - 1)(5.6 - 1)^3}{3}$$

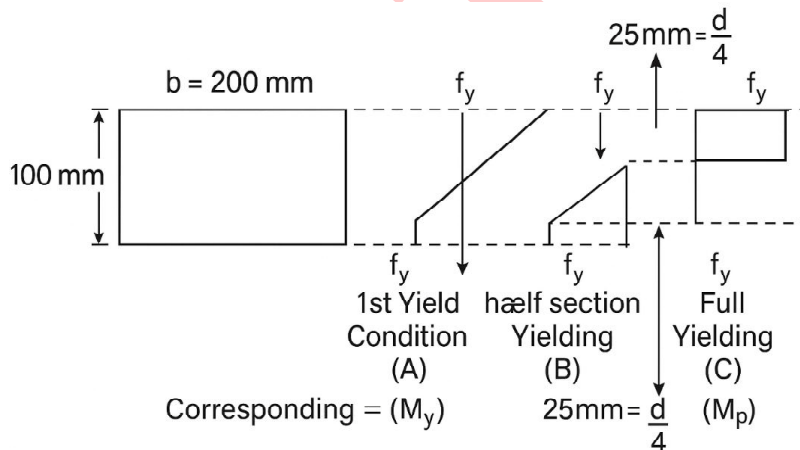
$$= 950.933 \text{ cm}^4$$

$$Z = \frac{950.933}{7.4} \text{ cm}^3 = 128.5045 \text{ cm}^3$$

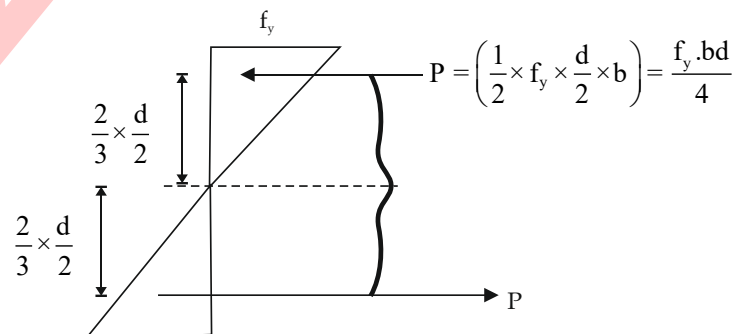
Hence, shape factor (S) = $\frac{Z_p}{Z} = \frac{184}{128.50} = 1.432$



80.
Sol.



Bending moment corresponding to 1st yield (M_y)



$$\therefore \text{Lever arm} = \frac{2d}{3}$$

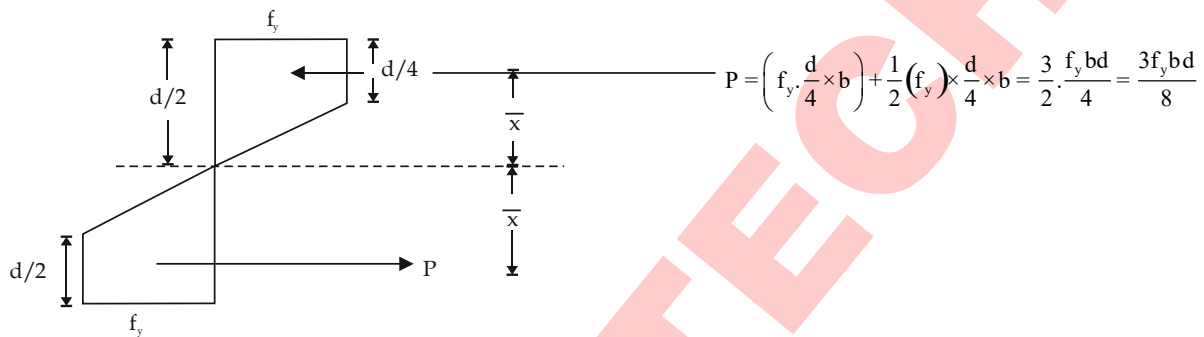
$$\Rightarrow M_y = P \times \frac{2d}{3} = f_y \cdot \frac{bd}{4} \times \frac{2d}{3}$$

$$\Rightarrow M_y = \frac{250 \times 200 \times (100)^2 \times 2 \times 100}{6}$$

$$= 83.33 \times 10^6 \text{ Nm-m}$$

$$M_y = 83.33 \text{ kNm}$$

BM Corresponding to the case when 1/2 of the section has Yielded (M_z)



$$\text{Lever arm} = 2\bar{x}$$

$$\bar{x} = \frac{\frac{f_y bd}{8} \times \frac{2}{3} \times \frac{d}{4} + \frac{f_y bd}{4} \times \left(\frac{d}{4} + \frac{d}{8}\right)}{\frac{3f_y bd}{8}}$$

$$\bar{x} = \frac{\frac{2d}{12} + 2 \times \left(\frac{3d}{8}\right)}{3} = \frac{d}{6} + \frac{3d}{4}$$

$$\bar{x} = \frac{22d}{72}$$

$$\Rightarrow \text{Lever arm} = 2\bar{x} = \frac{22d}{36} = \frac{11d}{18}$$

$$\Rightarrow M_z = \frac{3f_y bd}{8} \times \frac{11d}{18} = \frac{11f_y bd^2}{48}$$

$$\Rightarrow M_z = \frac{11f_y bd^2}{48} = \frac{11 \times 250 \times 200 \times (100)^2}{48} = 114.583 \text{ kNm.}$$

BM Corresponding to fully plastic state (M_p)

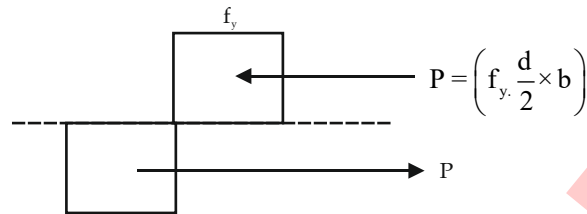
$$M_p = \frac{f_y bd^2}{4} = \frac{250 \times 200 \times 100}{4}$$

$$= 125 \text{ kNm}$$

Curvature at initial yield (k_y)

$$\frac{M_y}{I} = \frac{f_y}{d/2} = (E \cdot K_y)$$

$$\Rightarrow K_y = \frac{f_y}{d/2 \times E} = \frac{250 \times 2}{100 \times 2 \times 10^5} = 2.5 \times 10^{-5} \text{ Per mm}$$



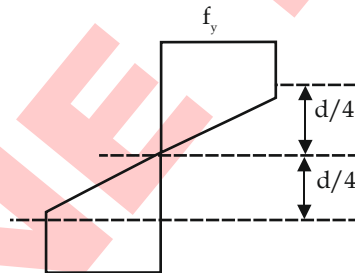
Curvature when 1/2 of the section has yielded (k_z)

We know that

$$\text{Strain} = \epsilon = \frac{y}{R} = k_z y$$

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

$$\epsilon = \frac{f_y}{E}$$

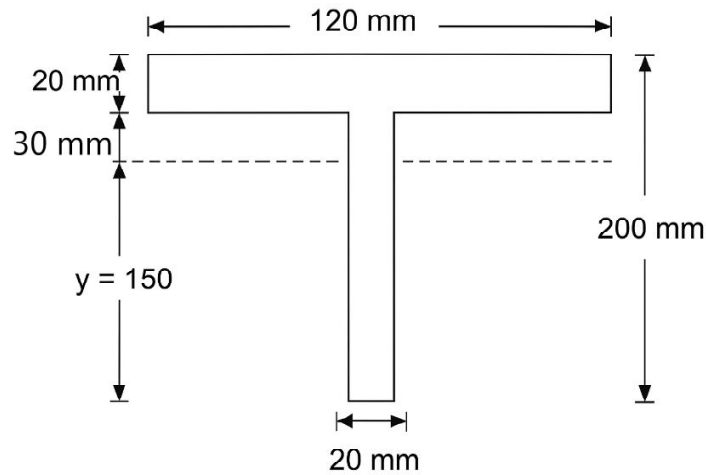


$$\Rightarrow \frac{f_y}{E} = K_z \times \frac{d}{4}$$

$$\Rightarrow K_z = \frac{f_y}{E \frac{d}{4}} = \frac{4f_y}{Ed} = \frac{4 \times 250}{2 \times 10^5 \times 100} = 5 \times 10^{-5}$$

81.

Sol. For fully plastic condition:



Total area of cross section = $120 \times 20 + 180 \times 20 = 300 \times 20 = 6000 \text{ mm}^2$

For fully plastic condition neutral axis is the equal area axis.

$$\Rightarrow 20 y = \frac{6000}{2}$$

$$y = 150 \text{ mm}$$

For upper part, distance of CG of area above equal area axis = $\bar{y}_1$

$$\bar{y}_1 = \frac{120 \times 20 \times 40 + 30 \times 20 \times 15}{3000} = 35 \text{ mm}$$

For lower part, distance of CG of area below equal area axis = $\bar{y}_2$

$$\bar{y}_2 = 75 \text{ mm}$$

$$\therefore M_p = \frac{A}{2} (\bar{y}_1 + \bar{y}_2) \times f_y = \left[\frac{6000}{2} (35 + 75) \times 250 \times 10^{-6} \right] \text{ kN-m}$$

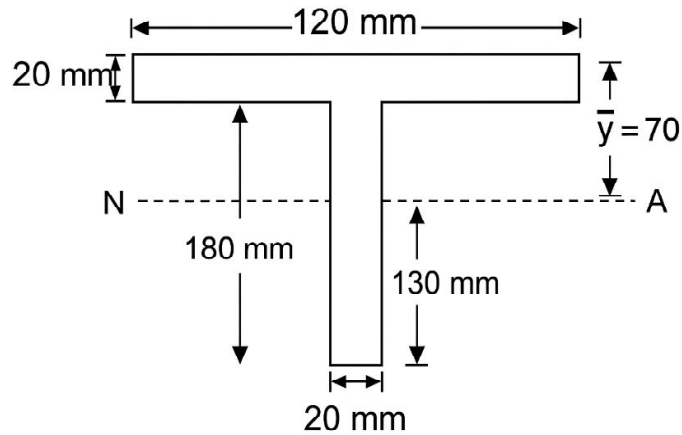
$$M_p = 82.5 \text{ kN-m}$$

For initial yield

Neutral axis is the centroidal axis

Let us assume the depth of neutral axes from the top the flange is $\bar{y}$.

$$\bar{y} = \frac{120 \times 20 \times 10 + 20 \times 180 \times 110}{6000} = 70 \text{ mm}$$



Moment of inertia of the section about Neutral axis

$$I_{NA} = \frac{120 \times 20^3}{12} + 120 \times 20 \times 60^2 + \frac{20 \times 180^3}{12} + 20 \times 180 \times 40^2 = 2.42 \times 10^7$$

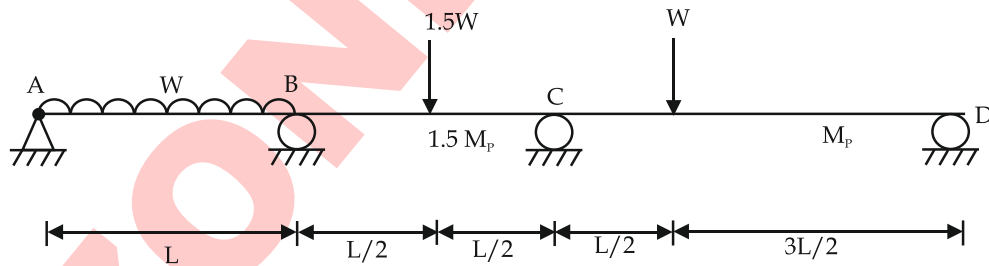
$$\therefore Z = \frac{1.8427 \times 10^7}{110} = 1.8615 \times 10^5$$

$$\therefore M_y = f_y \times Z = 250 \times 1.8615 \times 10^5 = 4.653 \times 10^7$$

$$\therefore M_y = 46.53 \text{ kN-m}$$

$$\text{Hence Shape factor (S)} = \frac{M_p}{M_y} = \frac{82.5}{46.53} = 1.77$$

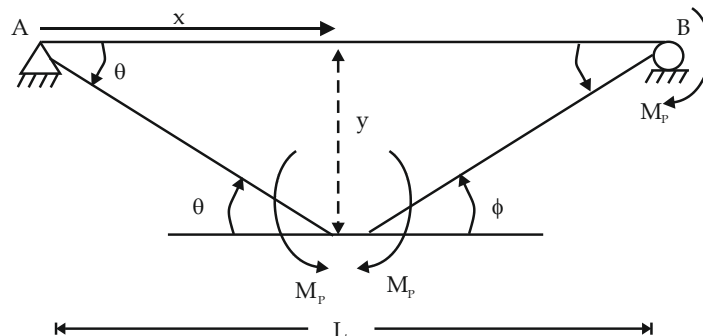
82.
Sol.



Static indeterminacy = 2

No. of hinges for a mechanism formation = 3

Span AB



Let hinge form at distance x from A

$$y = x\theta = (L-x)\phi \Rightarrow \theta = \left(\frac{L-x}{x}\right)\phi$$

$$\text{External work done} = \frac{1}{2} \times L \times y \times \frac{W}{L} (\text{udL} \times \text{Area})$$

$$\text{Internal work done} = M_p\theta \times M_p\phi + M_p\phi$$

$$\therefore \text{External work done} = \text{internal work done}$$

$$\Rightarrow \frac{1}{2} \times L \times (L-x)\phi \times \frac{W}{L} = M_p \left(\frac{L-x}{x}\right)\phi + 2M_p\phi$$

$$\text{or } W = \frac{M_p \left(\frac{L-x}{x} + 2\right) \times 2}{L-x}$$

$$\text{or } W = \frac{M_p(L+x)}{(L-x)x}$$

For w to be minimum

$$\frac{dW}{dx} = 0 \Rightarrow 2M_p \left[\frac{(L-x)x - (L+x)\{L-2x\}}{(L-x)^2 x^2} \right] = 0$$

$$\text{i.e., } Lx - x^2 - L^2 + 2Lx - Lx + 2x^2 = 0$$

$$\Rightarrow x^2 + 2Lx - L^2 = 0$$

$$\Rightarrow x = \frac{-2L \pm \sqrt{4L^2 + 4L^2}}{2} = \frac{-2L \pm 2\sqrt{2}L}{2} = -L \pm \sqrt{2}L = -L\sqrt{2}L = 0.414L$$

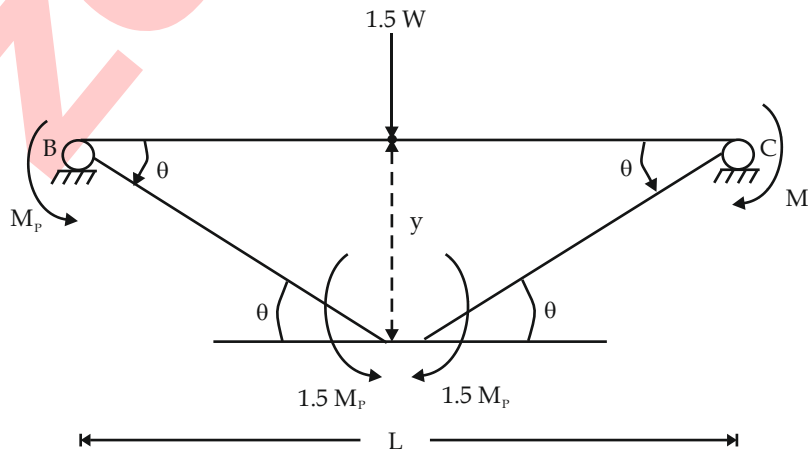
Substituting $x = 0.414L$ in equation (1)

$$W = \frac{2M_p(L + 0.414L)}{(L - 0.414L) \times 0.414L} = \frac{11.656M_p}{L}$$

Note : For propped cantilever beam loaded under udL , plastic hinge forms at $0.414L$ from propped

$$\text{and } M_p = \frac{11.656M_p}{L}$$

Span BC



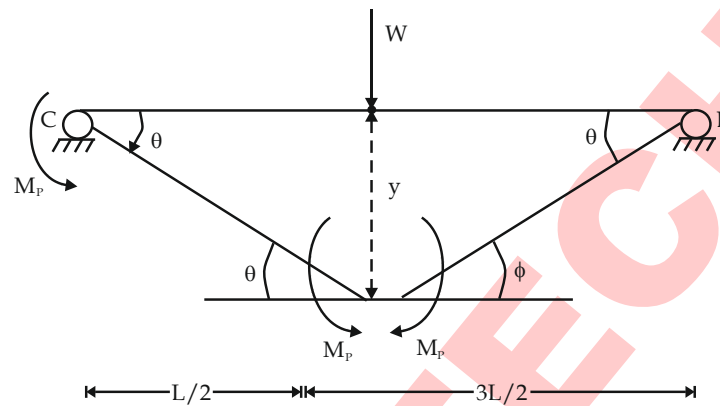
Internal virtual work = External virtual work

$$\Rightarrow M_p \theta + 1.5M_p \theta + 1.5M_p \theta + M_p \theta = 1.5W \times \frac{L}{2} \theta$$

$$\Rightarrow 5M_p \theta = \frac{1.5WL}{2} \theta$$

$$\text{or } w = \frac{10}{1.5} \frac{M_p}{L} = 6.67 \frac{M_p}{L}$$

Span CD



$$y = \frac{L}{2} \theta = \frac{3L}{2} \phi$$

$$\Rightarrow \theta = 3\phi$$

Internal virtual work = external virtual work

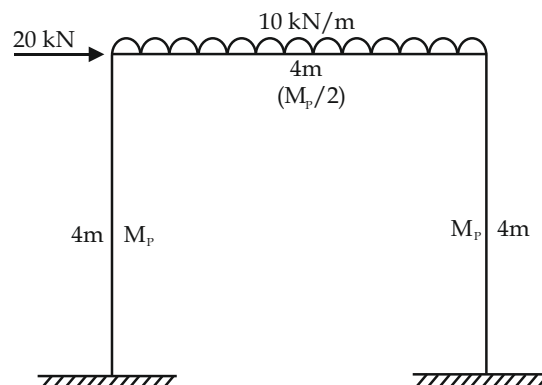
$$\Rightarrow M_p \theta + M_p \theta + M_p \phi = W \times \frac{3L}{2} \phi$$

$$\text{or } 2M_p \times 3\phi + M_p \phi = \frac{3WL}{2} \phi$$

$$\text{or } W = \frac{14}{3} \times \frac{M_p}{L}$$

$$\therefore \text{Collapse load} = \min \left(\frac{11.656 M_p}{L}, \frac{6.67 M_p}{L}, \frac{14 M_p}{3 L} \right) = \frac{14 M_p}{3 L}$$

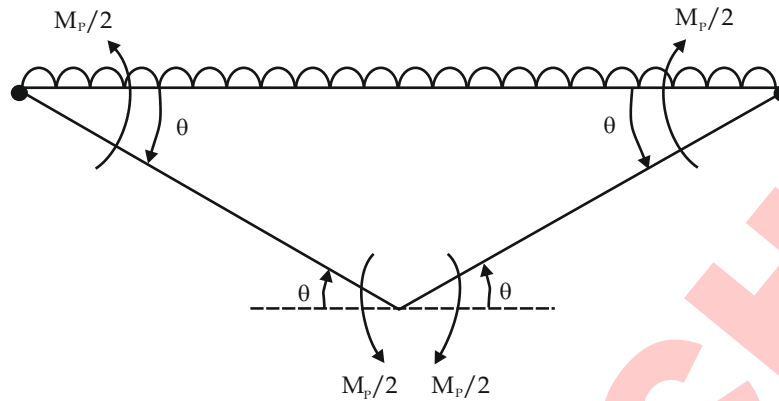
83.
Sol.



Degree of indeterminacy = $6 - 3 = 3$

⇒ No. of plastic hinge req. for complete collapse = 4

For beam mechanism



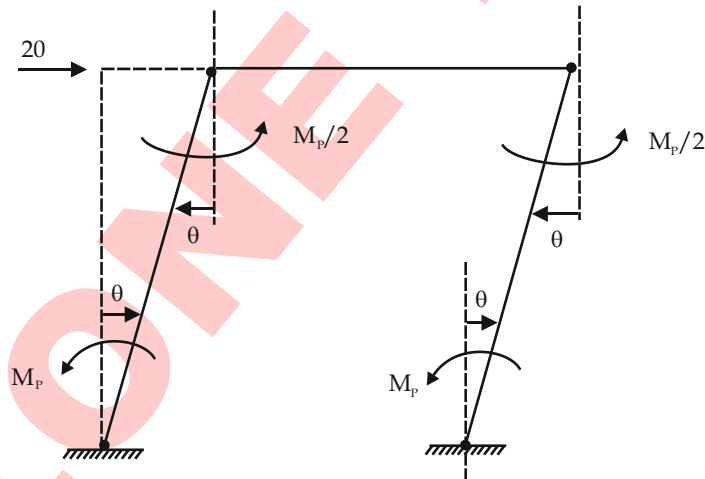
By applying principle of virtual work,

$$-4 \left(\frac{M_p}{2} \right) \theta + w \left(\frac{1}{2} \times 4 \times 2 \theta \right) = 0$$

$$\Rightarrow M_p = 2w = 2 \times 10 \text{ kNm}$$

$$\Rightarrow M_p = 20 \text{ kN-m}$$

For sway mechanism



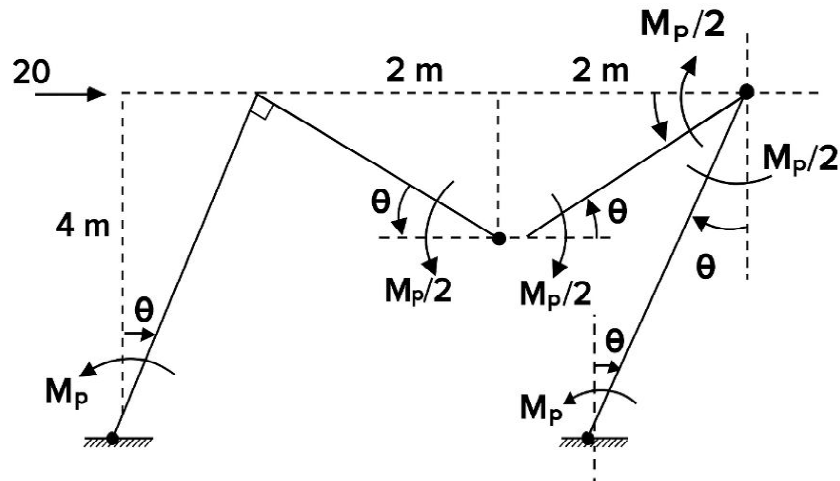
By applying principle of virtual work,

$$-M_p \theta - M_p \theta - 2 \times \frac{M_p}{2} \times \theta + 20 \times 4 \theta = 0$$

$$\Rightarrow -3M_p \theta + 80 \theta = 0$$

$$\therefore M_p = \frac{80}{3} = 26.67 \text{ kN-m}$$

Combined Mechanism



By applying principle of virtual work

$$-M_p\theta - \frac{M_p\theta}{2} - \frac{M_p\theta}{2} - \frac{M_p\theta}{2} - M_p\theta + 20 \times 4\theta + 10 \times \frac{1}{2} \times 4 \times 2\theta = 0$$

$$\Rightarrow -4M_p\theta + 80\theta + 40\theta = 0$$

$$\Rightarrow 4M_p = 120$$

$$\Rightarrow M_p = 30 \text{ kN-m}$$

Thus the maximum value of $M_p = 30 \text{ kN-m}$ Sol.

Note the max. value of M_p is chosen so that the given loading leads to just collapse. If we chosen smaller value of M_p a load less than the given load may also lead to collapse.