STEEL STRUCTURES

TENSION MEMBER

It is defined as a structural member subjected to tensile force in a direction parallel to its longitudinal axis. A tension member is also called *tie member* or simply a *tie*.

Types of Tension Members

The types of structure and method of end connections determine type of a tension member.

Tension members used may be broadly grouped into following four groups :

1. **Wires and Cables**

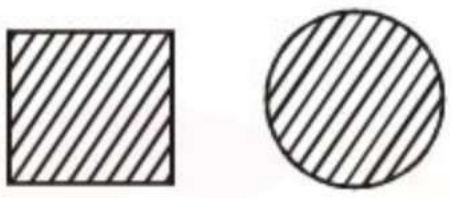
   These are used for hoists, derricks, rigging slings, guy wires and hangers for suspension bridges.

2. **Rods and Bars**

   Square and round bars as shown in figures are quite often used for small tension members.

   Round bars with threaded ends are used with pin-connections at the ends instead of threads.

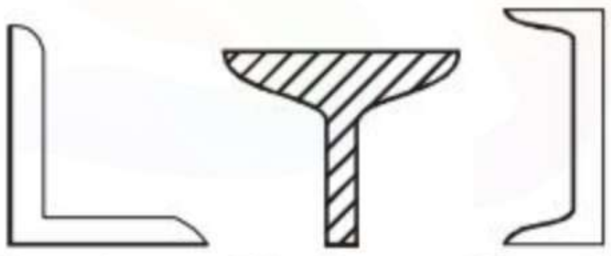
   Ends of rectangular bars or plates are enlarged by forging and bored to form eye bars. Eye bars are used with pin connections.

   Rods and bars have the disadvantage of inadequate stiffness resulting in noticeable sag under the self-weight.

3. **Single-structural shapes and Plates**

   Single structural shapes *viz*., angles sections and tee-sections as shown in figures are used as tension members. Angle sections are considerably more rigid than the wire ropes, rods and bars. When length of tension member is too long, then single angle section also becomes flexible.

   Single angle sections have disadvantage of eccentricity in both planes in a riveted connection.

   Channel section has eccentricity in one axis only. Single channel sections have high rigidity in the direction of web and low rigidity in the direction of flange.

   Occasionally, I-sections are also used as tension members. I-sections have more rigidity, and single I-sections are more economical than built-up sections.

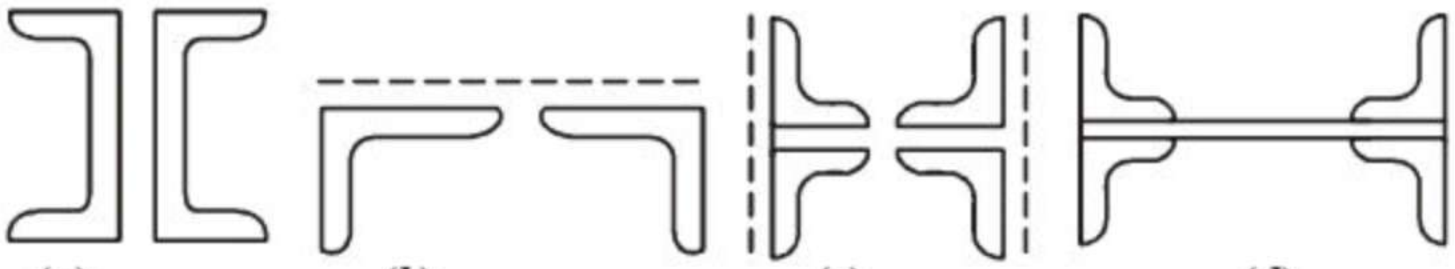
4. **Built-up Members**

   Two or more than two members are used to form built-up members. When single rolled steel sections cannot furnish the required area, then built-up sections are used.

   Double angle sections of unequal legs shown in the figure are extensively used as tension members in the roof trusses. The angle sections are placed back-to-back on two sides of a gusset plate. When both the angle sections are attached on the same sides of the gusset, then built-up section has eccentricity in one plane and is subjected to tension and bending simultaneously. The two angle sections may be arranged in the star shape (*i.e.*, angles are placed diagonally opposite to each-other with leg on outer sides). The star shape angle sections may be connected by batten plates. The batten plates are alternately placed in two perpendicular directions. The star arrangement provides a symmetrical and concentric connection.

   Two channel sections as shown in the Fig. (*a*) are used in the two-plane trusses, where two parallel gussets are used at each connection. Two-angle sections as shown in Fig. (*b*) have the advantage that distance between them could be adjusted to suit connecting members at their ends.

   Four-angle sections as shown in Fig. (*c*) are also used in the two-plane trusses. The angles are connected to two parallel gusset. For angle sections connected by plates as shown in Fig. (*d*) are used as tension members in bridge girders.



(*a*) (*b*) (*c*) (*d*)

A built-up section may be made of two channels placed back-to-back with a gusset in between them. Such sections are used for medium loads in a single-plane truss. In two-plane trusses, two channels are arranged at a distance with their flange turned inward. It simplifies the transverse connections, and also minimizes lacing. Flanges of two channels are kept outwards, as in the case of chord members of long-span girders, in order to have greater lateral rigidity.

Heavy built-up tension members in the bridge girder trusses are made of angles and plates. Such members can resist compression if reversal of stress takes place.

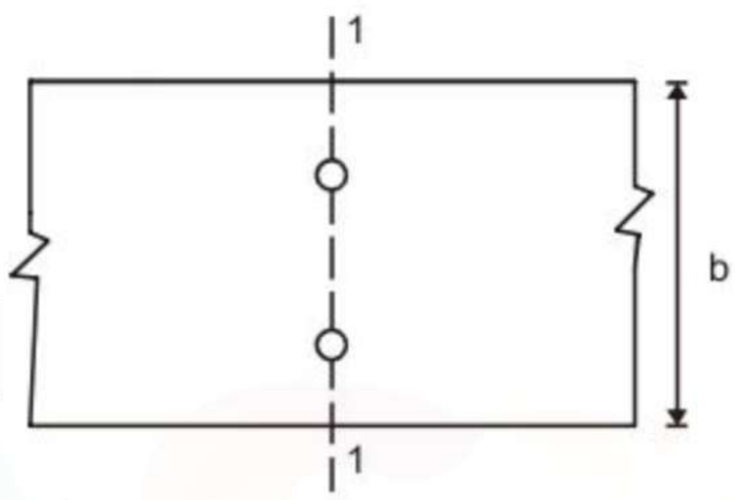
NET SECTIONAL AREA

A tension member is designed for its net sectional area at the joint. When a tension member is spliced or joined to a guesset plate by rivets or bolts, the gross sectional area is reduced by rivet holes.

The net sectional area is calculated for various cases as per IS 800-1984 as follows :

Net Sectional Area for Plate

In case of chain riveting, the net sectional area at one section is given by



$$A_{net} = t \times [b - nd]$$

where, A_{net} = net cross-sectional area along the rivet chain
b = width of plate
t = thickness of plate
d = gross diameter of rivet holes
n = no. rivets at the section

In case of a zig-zag or diagonal chain of holes, the net cross-sectional area along a chain of rivets is increased by an amount equal to,

$$\frac{S^2 \times t}{4g}$$

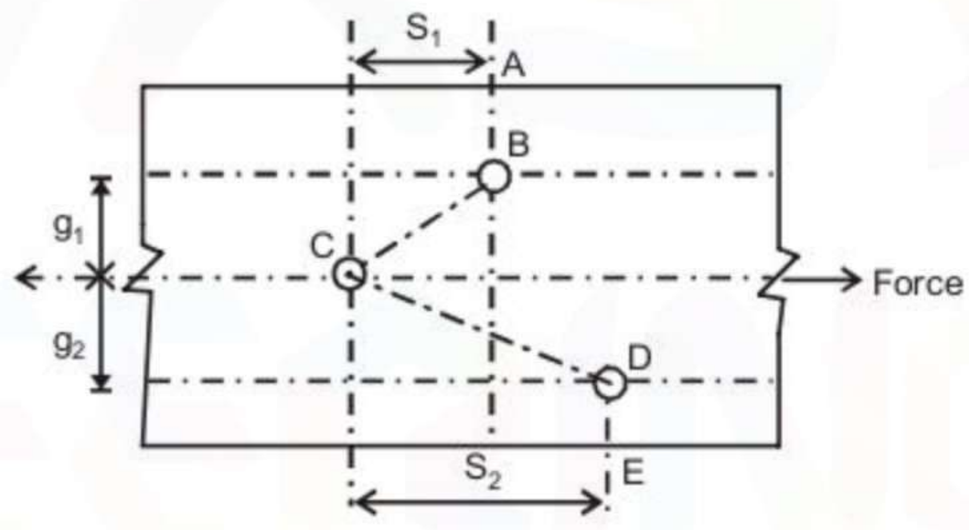


Fig.

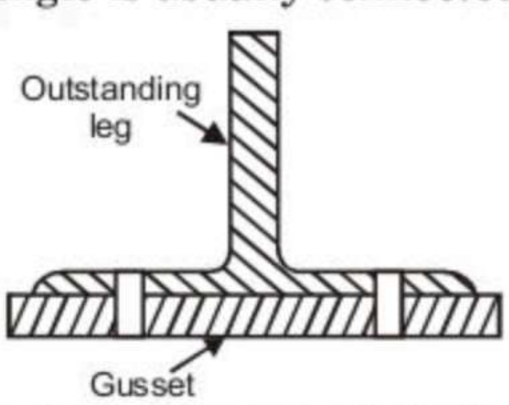
Thus, in fig net area along section ABCDE

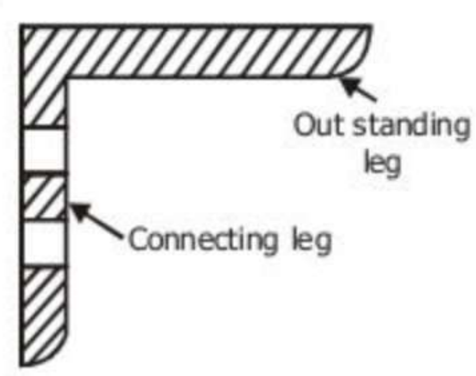
$$A_{net} = t \times [b - n \times d + \left[\frac{S_1^2}{4g_1} + \frac{S_2^2}{4g_2}\right]$$

where, n = number of rivet holes at the section
= 3 at section ABCDE
S_1, S_2 = staggered pitch
g_1, g_2 = gauge distance.

Net Effective Area for Angles and Tees in Tension

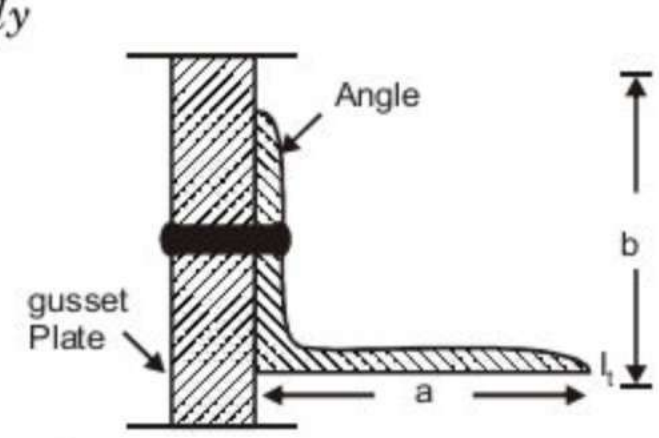
An angle is usually connected to a gusset plate by one leg and a tee is connected through its flange only.





Case 1 : *In the case of single angles in tension, connected through one leg only*

Net effective area = $A_1 + K_1 A_2$

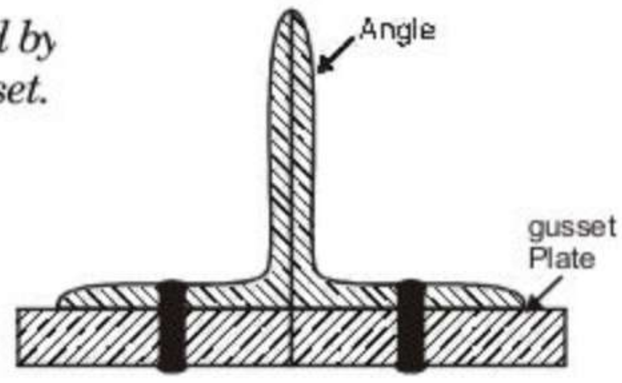


where, A_1 = area of the connected leg
A_2 = area of the outstanding leg

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

So, $A_1 = (b - d - \frac{t}{2})\,t\,;\; A_2 = (a - \frac{t}{2})\,t.$

When Lug angles are used, net sectional area of the whole angle member shall be considered.



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

$$\text{Net effective area } = A_1 + K_2A_2$$

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

A_2 = area of the outstanding leg (or web of the tee)

$$K_2 = \frac{5A_1}{5A_1 + A_2}$$

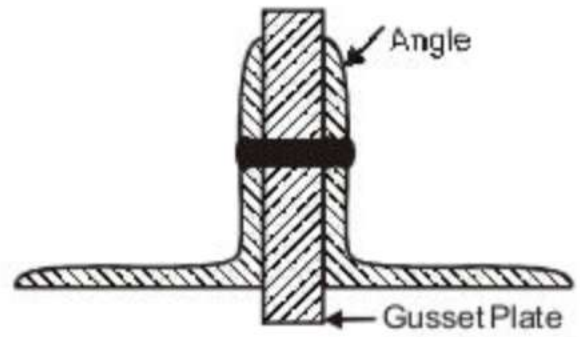


Fig. for case 3

Case 3 : *In case of double angles or tees, placed back-to-back and connected to both sides of the gusset*

Net effective area = Gross area of section – Area of holes

Note : Where the angles are back to back but are not tack riveted or welded the provision under case 2 & case 3 shall not apply and each angle shall be designed as a per provision of case 1.

Permissible Stresses in Axial Tension

It is related to the guaranteed minimum yield point of steel with an appropriate factor of safety. Most of the codes assume a factor of safety of 1.67.

The direct stress in axial tension on the effective net area should not exceed σ_{at}, *i.e.*

$$\sigma_{at} = 0.6\,\sigma_y,$$

where, σ_y = minimum yield stress of steel in N/mm^2 (MPa).

The permissible stress σ_{at} in axial tension for steel conforming to IS : 226-1975 :

(*i*) *Plates, angles, tees, I beams, channels and flats up to and*

including 20 mm thickness. .... 150 MPa

above 20 mm to 40 mm thickness .... 144 MPa

over 40 mm thickness .... 138 MPa, and

(*ii*) *Bars (round, square and hexagonal)*

up to and including 20 mm diameter .... 150 MPa

over 20 mm diameter .... 144 MPa

COMBINED AXIAL AND BENDING TENSILE STRESSES

A member subjected to both axial tension and banding should be proportioned so that the following condition is satisfied :

$$\frac{\sigma_{at,cal}}{0.6fy} + \frac{\sigma_{btx,cal}}{0.66fy} + \frac{\sigma_{bty,cal}}{0.66fy} \le 1$$

where, $\sigma_{at,cal}$ = calculated average axial tensile stress in mPa

$\sigma_{btx,cal}$ or $\sigma_{bty,cal}$ = calculated bending tensile stresses in the extrance fibre when bending is about xx-axis and yy-axis respectively.

fy = yield stress of steel in mPa.

SLENDERNESS RATIO (L)

The slenderness ratio of a tension member is the ratio of its unsupported length (l) to its least radius of gyration (r)

Table - Maximum Slenderness Ratio

S. No.	Member	Maximum Slenderness ratio
1.	A tension member in which a reversal of direct stress due to loads other than wind or seismic force occurs.	180
2.	A member normally acting as a tie in a roof truss or a brucing system but subject to possible reverse of stress resulting from the action of wind or EQ forces	350
3.	Tension member (other than pretensional members)	400

DESIGN OF AXIALLY LOADED TENSION MEMBER

The following steps are followed while designing can axially loaded tension member.

(*i*) Calculate the net sectional area required as,

$$A_{net} \text{ required} = \frac{\text{axial force}}{\text{Permissible tensile stress}}$$

(*ii*) Try a suitable section having sectional area 25% to 40% larger than the A_{net} required.

(*iii*) Calculate the A_{net} available in the trial section.

(*iv*) The trial section will be suitable if

$$A_{net} \text{ available} \geq A_{net} \text{ required}$$

(*v*) Check the slenderness ratio when the reversal of load may occur as per the table given above.

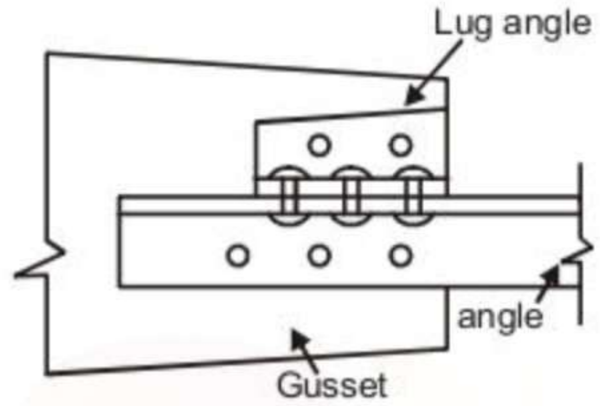
(*vi*) Design end connection.

Lug Angle

The Lug Angle is a short length of an angle section used at a joint to connect the out standing leg of a member, there by reducing the length of joint.

Lug Angles are also not very efficient in transmitting loads and a certain eccentricity is caused between b/w the load and C.G. of the rivet group. The use of lug angles is therefore avoided in general.

A Lug Angle is provided at the beginning of a joint so that it can be effective in sharing bad.



Design of lug angles

Indian Standard IS : 800 specifies followings

(1) Lug angles connecting a channel-shaped member should, as far as possible, be disposed symmetrically with respect to the section of the member.

(2) In the case of angle members, lug angles and their connections to the gusset or any other supporting member should be capable of developing a strength not less than 20% in excess of the force in the outstanding leg of the angle and attachment of the lug angle to the angle member should be capable of developing a strength 40% in excess of that force.

(3) In the case of channel sections, lug angles and their connections to the gusset or any other supporting member should be capable of developing a strength of not less than 10% in excess of the force not accounted for by the direct connection of the member, and the attachment of the lug angles to the member should be capable of developing a strength 20% in excess of that force.

(4) In no case should fewer than two bolts or rivets be used for attaching the lug angle, to the gusset or another supporting member.

(5) Effective connection of the lug angle should, as far as possible, terminate at the end of the member connected and the fastening of the lug angle to the member should preferably start in advance of the direct connection of the member to the gusset or other supporting member.

(6) Where lug angles are used to connect an angle member, whole area of the member should be taken as effective, *i.e.*

$$A_{net} = \text{gross area} - \text{deduction for holes.}$$

Tension Splice

Strength of the splice plates and the rivets connecting them with the member should atleast be equal to the design load of the tension member.

When tension members of different sizes have to be connected, filler plates may be used to bring the member in level.

Rivets or bolts carrying a calculated shear stress through a packing more than 6 mm thick should be increased above the number required by normal calculations by 2.5% for each 2 mm thickness of packing. For double shear connections packed on both sides, the number of additional rivets or bolts required is determined from the thickness of the thicker packing. Additional rivets or bolts should preferably be placed in an extension of the packing.

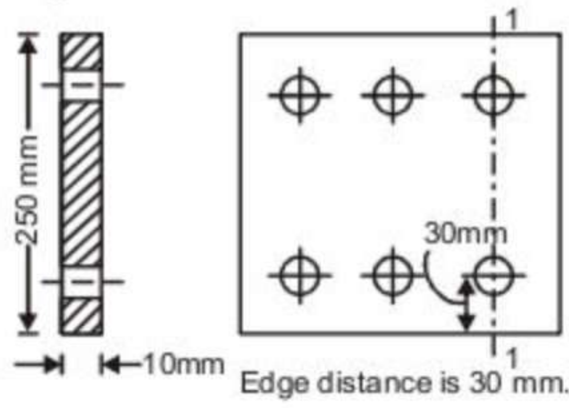
Riveted and Welded Connections

In a riveted connection, rivets are placed on the gauge lines of the angles and this results in small eccentricity of load on the joint as well as the member.

In the welded connection, lengths of the welds can be adjusted so that centre of gravity of the joint lies on the line passing through centre of gravity line of the member.

SOLVED EXAMPLES

1. *Find net area of a plate section 250 mm × 10 mm with two rows of holes in a straight chain to accommodate 16 mm diameter rivets.*



Solution

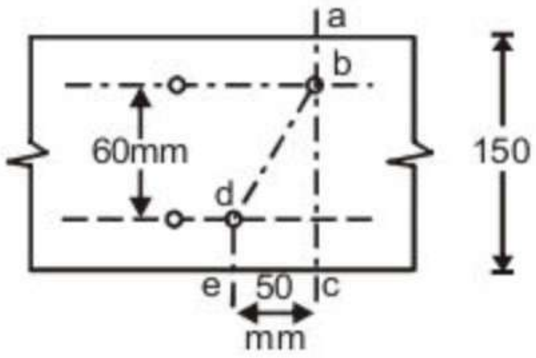
Gross area of the plate = 250 × 10 = 2500 mm^2

Gross Diameter of the rivet hole = 16 + 1.5 = 17.5 mm

Area of the two holes = 2 × 17.5 × 10 = 350 mm^2

Net area of plate = 2500 mm^2 – 350 mm^2 **= 2150 mm^2**

2. *Find minimum net section of the 15 cm × 1.0 cm plate, with rivets, 16 mm diameter, in zig-zag rows as shown in the figure*



Solution

Gross diameter of rivet hole = 16 + 1.5 = 17.5 mm

Size of plate = 150 × 10 mm

Consider the section $a - b - c$

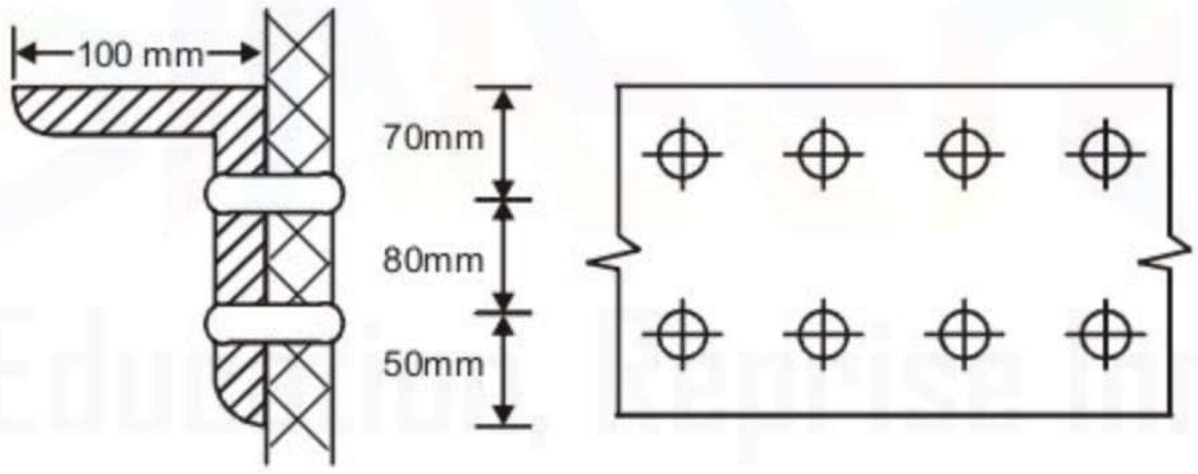
$$A_{net} = t\,[b - nd]$$
$$= 10\,[150 - 1 \times 17.5] = 1325 \text{ mm}^2$$

Consider the section $a - b - d - e$

$$A_{net} = t\,[b - nd + \frac{S^2}{4g}]$$
$$S = 50 \text{ mm}, \qquad g = 60 \text{ mm}$$
$$A_{net} = 10\,[150 - 2 \times 17.5 + \frac{50^2}{4 \times 60}]$$
$$= 10\,[150 - 35 \times 10.42]$$

A_{net} = 1254.20 mm^2 this is the minimum area of section

3. *Find net area of an angle section, ISA 200 × 100 × 10, where one leg is connected by two rows of rivets, 16 mm diameter, as shown in figure below*



Solution

For Angle, one leg connected on gusset plate gross diameter of rivet hole = 16 + 1.5 = 17.5 mm

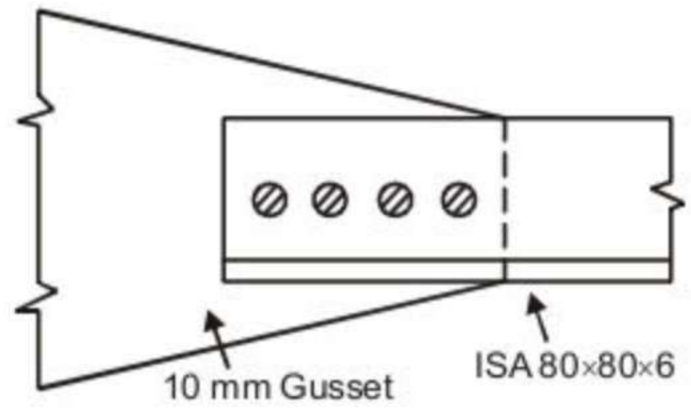
$$A_{net} = A_1 + K_1 A_1$$
$$K_1 = \frac{3A_1}{3A_1 + A_2}$$
$$A_1 = (200 - 2 \times 17.5 - 5) \times 10 = 1600 \text{ mm}^2$$
$$A_2 = (100 - 5) \times 10 = 950 \text{ mm}^2$$
$$K_1 = \frac{3 \times 1600}{3 \times 1600 + 950} = 0.835$$
$$A_{net} = 1600 + 0.835 \times 950$$
$$= 2393.04 \text{ mm}^2$$

4. *A member of a truss consists of two angles, ISA 80 × 80 × 6 mm thick placed back-to-back. It carries a direct load of 150 kN and is connected to a gusset plate of 10 mm thick placed in between the two connected legs. The number of 18 mm diameter shop rivets required for the joint. Assume that power-driven shop rivets are used, whose strengths are given below.*

For power-driven shop rivets :

$$F = 102.5 \text{ N/mm}^2 \text{ (shear)},$$
$$F_b = 236 \text{ N/mm}^2 \text{ (bearing)}.$$

Rivets are in double shear. They bear against the 10 mm thick gusset plate and the 6 mm thick connecting legs of two angles. Find number of rivets.



Solution

$$\text{Strength of rivet in double shear} = 2 \times \frac{\pi}{4} (19.5)^2 \times 102.5 = 61222.76 \text{ N}$$

$$\text{Strength of rivet in bearing on 10 mm plate} = 19.5 \times 10 \times 236 = 46020 \text{ N}$$

$$\text{Strength in bearing on two 6 mm angles} = 2 \times 19.5 \times 6 \times 236 = 55220 \text{ N}$$

Gross diameter of rivet is taken as 1.5 mm greater than the nominal diameter, therefore

$$\text{Rivet value} = 46020 \text{ N}$$

$$\text{Number of rivets required for carrying the 150 kN direct load} = \frac{150 \times 1000}{46020} = 3.26$$

Hence provide 4 rivets.

5. *A single ISA 100 × 75 × 10 is used as a tension member with the longer leg connected to a 10 mm thick gusset plate. The connection is made with the help of a lug angle. Find net area required [Use 20 diameter rivets σ_{at} = 150 MPa. Permissible shear and bearing stress in rivet is 100 MPa and 300 MPa respectively. Sections available for lug angles are :*

ISA 60 × 60 × 8 – 896 mm²; ISA 60 × 60 × 10 – 1100 mm²; ISA 70 × 70 × 8 – 1200 mm²

Solution

$$\text{Nominal diameter} = 20 + 1.5 = 21.5 \text{ mm}$$

Strength of Rivets :

$$\text{In bearing} = \frac{300}{1000} \times 21.5 \times 10 = 64.5 \text{ kN}$$

$$\text{In single shear} = \frac{100}{1000} \times \frac{\pi}{4} \times (21.5)^2 = 36.3 \text{ kN}$$

$$\therefore \quad \text{Rivet value} = 36.3 \text{ kN}$$

$$\text{Gross area of angle} = 95 \times 10 + 70 \times 10 = 1650 \text{ mm}^2$$

$$\text{Net area of angle} = 1650 - 21.5 \times 10 = 1385 \text{ mm}^2$$

$$\text{Strength of member} = \frac{150}{1000} \times 1435 = 215.25 \text{ kN}$$

Since 100 mm leg is connected to gusset plate, width of outstanding leg = 75 – 10 = 65 mm

Area of outstanding leg = 65 × 10 = 650

$$\text{Strength of outstanding leg} = \left(\frac{650}{650 + 1000}\right) \times 215.25 = 84.79 \text{ kN}$$

$$\text{Strength of connected leg} = 215.25 - 84.79 = 130.46 \text{ kN}$$

$$\text{Required strength of lug angle} = 1.2 \times 84.79 = 101.748 \text{ kN}$$

$$\text{Net area required,} \quad A_{net} = \frac{101.748 \times 1000}{150} = \mathbf{678.32 \text{ mm}^2}$$

6. *A splice is used connect a 250 mm × 20 mm plate with a 250 mm × 12 mm plate. The design load is 400 kN. Find the number of rivets required. (Taking permissible stresses in shearing and bearing in rivets are 10.05 kN/cm² & 23.2 kN/cm² respectively.)*

Solution

Assuming that one face of each plate is flush with each-other, a 250 mm × 8 mm filler plate will be required. Use two splice plates of size 250 mm × 12 mm, one on each face.

Strength of a 20 mm diameter rivet

$$\text{In double shear} = 2 \times \frac{\pi}{4}(2.15)^2 \times 10.05 = 72.97 \text{ kN}$$

$$\text{In bearing on 12 mm plate} = 2.15 \times 1.2 \times 23.2 = 59.86 \text{ kN}$$

$$\therefore \quad \text{Rivet value} = 59.86 \text{ N}$$

$$\text{Number of rivets} = \frac{400}{59.86} = \mathbf{6.68, say, 7}$$

7. *An angle ISA 150 × 75 × 10 mm thick, steel tension member is connected by its long leg to a 10 mm gussset plate. Use 6 mm fillet weld on the toe and the back. Take the allowable unit shearing stress for weld metal as 112.5 N/mm². In the welded joint find the force taken by the weld on the back.*

Solution:

$$\text{Area of connected leg, } A_1 = (150 - 5) \times 10 = 1450 \text{ mm}^2$$

$$\text{Area of outstanding leg, } A_2 = (75 - 5) \times 10 = 700 \text{ mm}^2$$

$$K_1 = \frac{3A_1}{3A_1 + A_2} = \frac{3 \times 1450}{3 \times 1450 + 700} = 0.861$$

$$\text{Effective area} = A_1 + K_1A_2$$

$$= 1450 + 0.861 \times 700 = 2053 \text{ mm}^2$$

$$\text{Allowable tensile stress} = 150 \text{ N/mm}^2$$

$$\text{Allowable load} = 150 \times 2053 = 307950 \text{ N}$$

$$\text{Strength of 6 mm fillet weld} = 6 \times 0.7 \times 112.5 = 473 \text{ N/mm.}$$

$$\text{Strength of 8 mm fillet weld} = 8 \times 0.7 \times 112.5 = 630 \text{ N/mm.}$$

Distance of centroidal axis of angle from its back is 53.2 from steel table.

$$\text{Then force taken by the weld on the back, } P_1 = \frac{150 - 53.2}{150} \times 307950 = 198730.4 \text{ N}$$

COMPRESSION MEMBERS

A structural member loaded axially in compression is generally called *compression member.* The structural members carrying compressive load in truss is called STRUTS. Vertical compression members in buildings are called *columns, posts or stanchions.* The main Compression members in roof trusses are called Raftor and in a crane is called a *boom*.

COLUMN

Short columns are subject to crushing and behave like members under pure compression.

Long columns tend to buckle out of the plane of the load axis.

Theory of Columns

Euler's formula for critical load for a pin-ended column subject to axial load is

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

where, L = length of column between the hinged ends

E = modulus of elasticity

I = moment of inertia of the column section.

The column will become unserviceable if loads are larger than P_{cr}.

In the Euler equation, it is assumed that stress is proportional to strain, therefore

$$\text{critical stress} = \frac{P_{cr}}{A} = \frac{\pi^2 EI}{AL^2} = \frac{\pi^2 E}{\left(\frac{L}{r}\right)^2} = \frac{\pi^2 E}{\lambda^2} \qquad \text{where } I = Ar^2$$

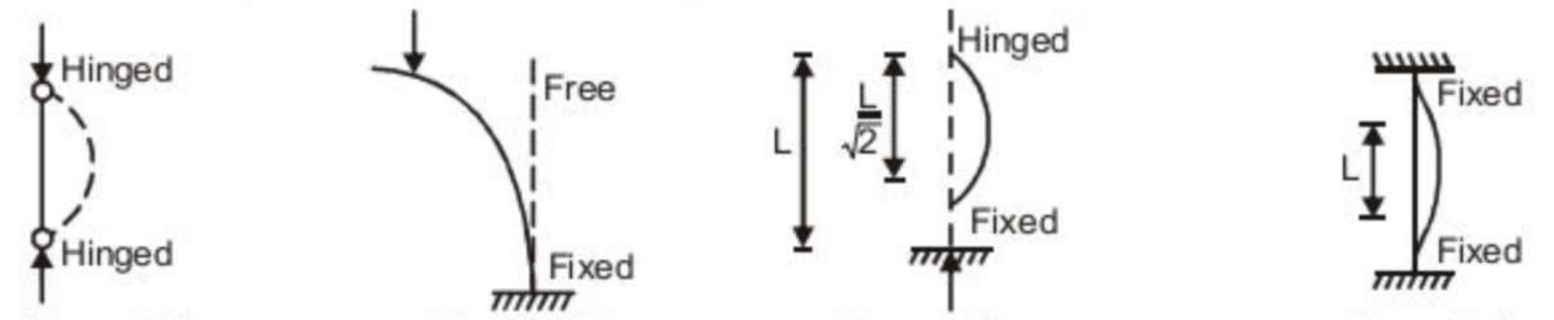
where, A = area of cross-section

r = radius of gyration about the bending axis.

λ = slenderness ratio

End Conditions

Columns with length L and effective length l are shown in the figure.



Case (1)	Case (2)	Case (3)	Case (4)
$P_{cr} = \frac{\pi^2 EI}{L^2}$	$P_{cr} = \frac{\pi^2 EI}{(2L)^2}$	$P_{cr} = \frac{2\pi^2 EI}{L^2}$	$P_{cr} = \frac{4\pi^2 EI}{L^2}$

Strength of an Axially Loaded Compression Members

Maximum axial compression load permitted on a compression member, is given by

$$P = \sigma_{ac} \times A$$

where, P = axial compressive load (N);

σ_{ac} = permissible stress in axial compression (MPa);

A = effective cross-sectional area of the member $(mm)^2$

Indian Standard IS 800 : 1984

It stipulates that direct stress on the cross-sectional area of axially loaded compression members should not exceed 0.6 f_y nor permissible stress calculated using Merchant - Rankine formula.

$$\text{Permissible stress in axial compression (MPa), } \sigma_{ac} = 0.6 \times \frac{f_{cc} \times f_y}{\left[f_{cc}^n + f_y^n\right]^{1/n}}$$

where, f_y = yield stress of steel (MPa)

f_{cc} = elastic critical stress in compression = $\frac{\pi^2 E}{\lambda^2}$

$\lambda = \frac{l}{r}$ = Slenderness ratio of the member

where l = effective length of the member

r = appropriate radius of gyration of the member

E = modulus of elasticity = 2×10^5 MPa

n = a factor assumed as 1.4

Values of σ_{ac} for some of the Indian standard structural steels are given in table 5.1 (IS-800-1984).

Effective Length

Table below gives values of the effective lengths recommended by the Indian Standard IS : 800. Actual length L of the compression member should be taken as the length from centre-to-centre of intersection of supporting members or cantilevered length in case of free standing struts.

Equivalent lengths for various End Conditions

Type	Effective length of member l
1. Effectively held in position and restrained in direction at both ends.	0.67 L
2. Effectively held in position at both ends restrained against rotation at one end.	0.80 L
3. Effectively held in position at both ends but not restrained against rotation.	L
4. Effectively held in position.	1.2 L
5. Effectively held in position and restrained in direction at one end and at the other end partially restrained in direction but not held in position.	1.5 L
6. Effectively held in position and restrained in direction at one end but not held in position nor restrained in direction at the other end.	2.0 L

Note 1 : L is the unsupported length of compression member.

Note 2 : For battened struts, effective length should be increased by 10 per cent.

Maximum Slenderness Ratio

According to Indian Standard IS : 800, slenderness ratio should not exceed the values given in the Table below.

Type of Member	Slenderess ratio, $\lambda = \frac{l}{r}$
1. Member carrying compressive loads resulting from dead and superimposed loads.	180
2. Member subject to compressive loads resulting from wind/earthquake forces provided the deformation of such members does not adversely affect the stress in any part of the structure.	250
3. Compression flunge of a beam	300

Angle Struts

Single angle discontinuous struts connected by a single rivet or bolt may be designed for axial load only provided compressive stress does not exceed 0.80 σ_{ac}. The value of σ_{ac} can be determined on the basis that effective length l of the strut is from centre-to-centre of inter-section at each end and r is minimum radius of gyration. In no case, l/r ratio for single angle struts should exceed 180. If a single discontinuous strut is connected by a weld or by two or more rivets or bolts in line along the angle at each end, it may be designed for axial load only provided compression stress does not exceed σ_{ac} arrived at on the basis that l is taken as 0.85 times the length of the strut, centre-to-centre of inter-section at each end, and r is minimum radius of gyration.

For double angle struts which are discontinuous, back-to-back connected to both sides of the gusset or section by not less than two bolts or rivets in line along the angles at each end or by the equivalent in welding, the load may be regarded as applied axially. Effective length l in the plane of end gusset could be taken between 0.7 and 0.85 times the distance between inter-sections depending on the restraint provided; in the plane perpendicular to that of the end gusset, the effective length should be taken as equal to distance between centres of inter-sections. Calculated average compressive stress should not exceed values of σ_{ac} obtained for appropriate slenderness ratios. The angles should be connected together with tack rivets or welds at intervals along their lengths.

Compression Members Composed of Back-to-Back Components.

A compression member composed of two angles, channels or tees, back-to-back, in contact or separated by a small distance should be connected together by riveting, bolting or welding so that slenderness ratio of each member between the connections is not greater than 40 nor greater than 0.6 times the most unfavourable slenderness ratio of the strut as a whole. In no case, spacing of tacking rivets in a line exceed 600 mm for such members.

For other types of build-up compression members, where cover plates are used, pitch of the tacking rivets should not exceed 32 t or 300 mm, whichever is less, where t is thickness of the thinner outside plate. Where plates are exposed to weather, the pitch should not exceed 16 t or 200 mm, whichever is less.

In these connections, rivets, welds and bolts should be sufficient to carry shear force and bending moments, if any, specified for battened struts. The diameter of the connecting rivets should not be less than minimum diameter given in the table below :

Thickness of Member	Minimum diameter of Rivets
Up to 10 mm	16 mm
Over 10 mm up to 16 mm	20 mm
Over 16 mm	22 mm

Solid packing or washers should be used for riveting, bolting or welding, where members are separated back-to-back.

Ends of struts should be connected together with not less than two rivets or bolts or their equivalent in welding and there should be not less than two additional connections spaced equidistant in the length of the strut.

A minimum of two rivets or bolts should be used in each connection; one on line of each gauge mark, where legs of the connected angles or tables of the connected tees are 125 mm wide or over, or where webs of channels are 150 mm wide or over.

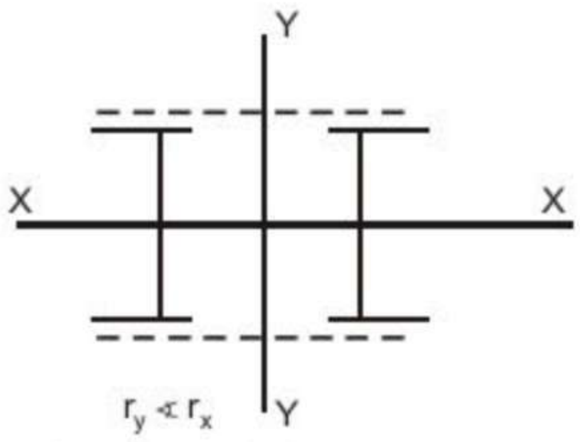
LACINGS AND BATTENS FOR BUILT-UP COMPRESSION MEMBERS

As per Indian Standard IS 800-1984, following specifications are used for design of lacing and batten plates :

In a built-up section, different components are connected together so that they act as a single column. Lacing is generally preferred in case of eccentric loads. Battening is normally used for axially loaded columns and in sections where components are not far apart. Flat bars are generally used for lacing. Angles, channels and tubular sections are also used for lacing of very heavy columns. Plates are used for battens.

Lacings

A lacing system should generally conform to the following requirements :



(1) Compression member comprising two main components laced and tied should, where practicable, have a radius of gyration about the axis perpendicular to the plane of lacing not less than the radius of gyration at right angles to that axis.

(2) Lacing system should not be varied throughout the length of the strut as far as practicable.

(3) Cross member (except tie plates) should not be provided along length of the column with lacing system, unless all forces resulting form deformation of column members are calculated and provided for in the lacing and its fastening.

(4) Single-laced systems on opposite sides of the main components should preferably be in the same direction so that one system is the shadow of the other.

(5) Laced compression members should be provided with tie plates at the ends of the lacing system and at points where lacing systems are interrupted. The tie plates should be designed by the same method as followed for battens.

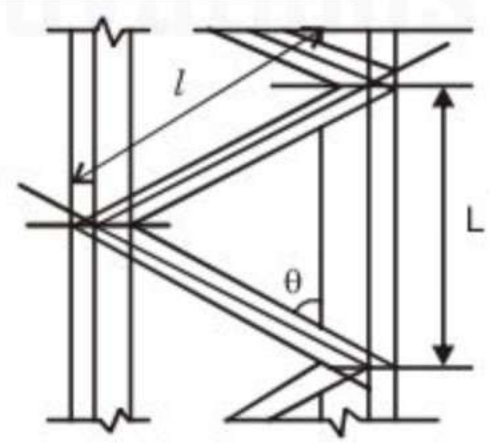
Design of Lacing System

(1) Angle of inclination of the lacing with longitudinal axis of the column should be between 40° to 70°.

(2) Slenderness ratio $\frac{l_e}{r}$ of the lacing bars should not exceed 145.

Effective length l_e of the lacing bar should be according to the Table given below :

Type of lacing	Effective length, l_e
1. Single lacing, riveted at ends.	Length between inner end rivets on lacing bar.
2. Double lacing, riveted at ends and at inter-sections.	0.7 times the length between end rivets on lacing bars.
3. Welded lacing.	0.7 times the distance between inner ends of effective lengths of welds at ends.



(3) For riveted or welded lacing system, $\left(\frac{L}{r_{\min}^c}\right) \ngtr 50$ or 0.7 times maximum slenderness ratio of the compression member as a whole, whichever is less.

Here, L = distance between centres of connections of the lattice bars and

$r_{\min}^c$ = minimum radius of gyration of the components of the compression member.

(4) Minimum width of lacing bars in riveted connections should be according to the Table given below :

Nominal rivet diameter (mm)	22	20	18	16
Width of lacing bars (mm)	65	60	55	50

(5) *Minimum thickness of lacing bars :* $t \nless \frac{l}{40}$, for single lacing

$\nless \frac{l}{60}$, for double lacing

where, l = length between inner end rivets.

(6) Lacing of compression members should be designed to resist a transverse shear, V = 2.5 per cent of the axial force in the member. The shear is divided equally among all transverse lacing systems in parallel planes. The lacing system should also be designed to resist additional shear due to bending if the compression member carries bending due to eccentric load, applied end moments, and/or lateral loading. Lacing should be cheacked for compression & tension.

(7) Riveted connections may be made in two ways, as shown in the Fig. (*a*) and Fig. (*b*).

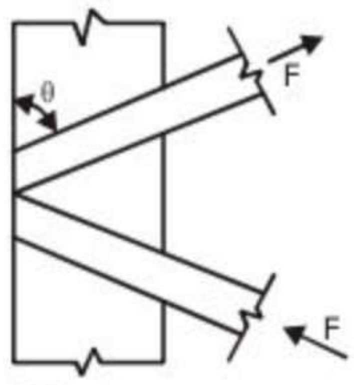


***Fig.* (*a*)**

No. of rivet required = $\dfrac{\text{F}}{\text{Rivet value}}$

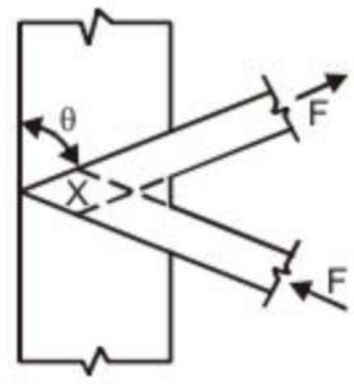


***Fig.* (*b*)**

No. of rivet required = $\dfrac{\text{F}}{\text{Rivet value}}$

Welded connections

Lap joint : Overlap should be not less than 1/4 times thickness of bar or member, whichever is less.

Butt joint : Full penetration butt weld or fillet weld on each side; lacing bar should be placed opposits to flange or stiffening component of main member.

Battens

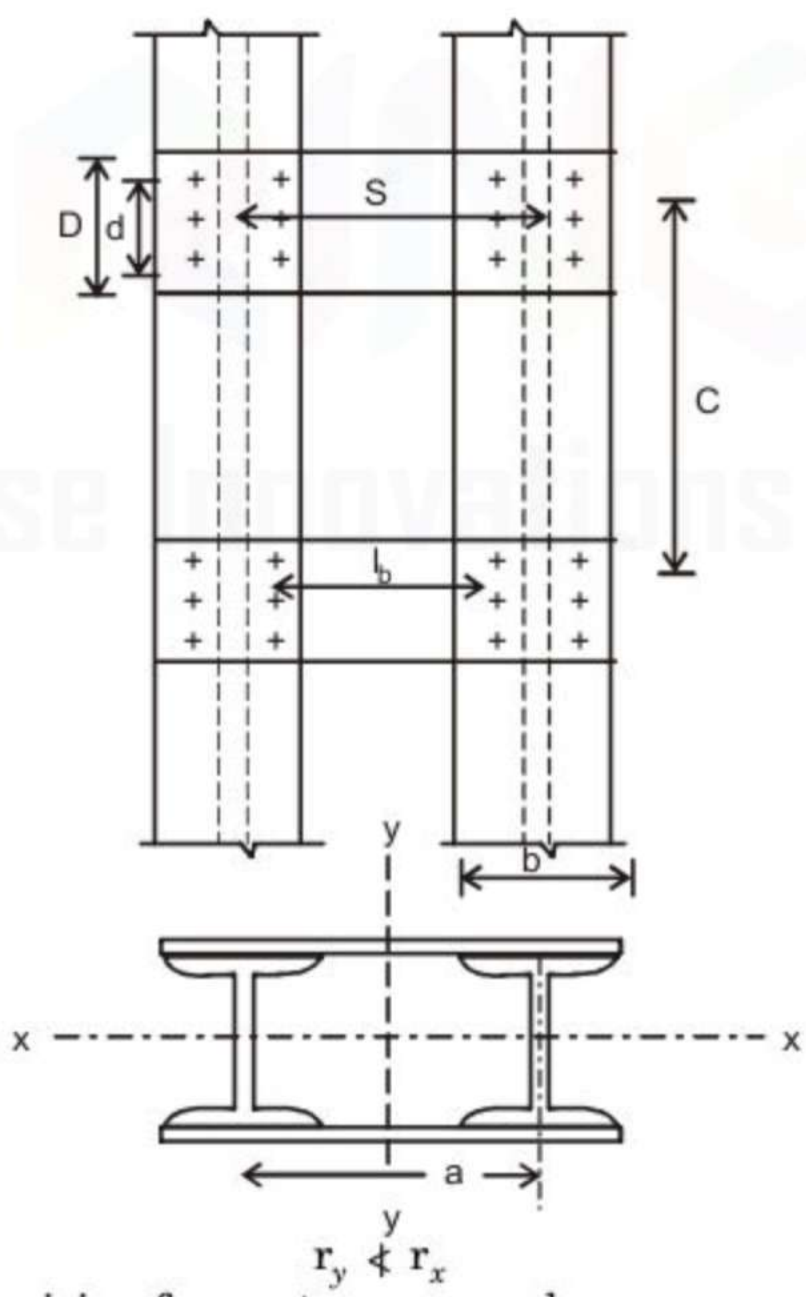
Compression members composed of two main components battened should preferably have these components of the same cross-section and symmetrically disposed about their X–X axis.

Battens should be placed opposite to each-other at each end of the member and at points where member is stayed in its length, and should as far as practicable, be spaced and proportioned uniformaly throughout.

Effective length of columns should be increased by 10 per cent.

Design of Battens

(1) Spacing of batten C, from centre-to-centre of end fastening should be such that slenderness ratio of the lesser main component, $\dfrac{C}{r^{c}_{\min}} \ngtr 50$ or 0.7 times the slenderness ratio of the compression member as a whole about the X-X axis (parallel to battens), whichever is less, where c = spacing of Batten as shown in fig.



(2) Effective depth of battens, *d* shall be taken as distance between end rivets or end welds.

For intermediate battens : $d > \dfrac{3}{4}a$

For end battens : $d > a$

For any batten : $d > 2b$

where, d = effective depth of batten

a = centroid distance of members

b = width of member in the plane of battens.

Select a suitable effective depth d according to the above requirement.

(3) Thickness of battens, $t > \dfrac{l_b}{50}$

where, l_b = distance between innermost connecting line of rivets or welds.

(4) Battens should be designed to carry bending moment and shear arising from a transverse shear,

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

where, P = total axial load in the compression member.

Transverse shear V is divided equally between parallel planes of battens. Battens and their connections to main components resist simultaneously a longitudinal shear.

$$V_1 = \frac{V \times C}{N \times S}, \text{ and moment, } M = \frac{V \times C}{2N}$$

where, C = spacing of battens

N = number of parallel planes of battens

S = minimum transverse distance between centroids of rivet group or welding.

Battens should be cheacked for longitudinal shear stress and banding.

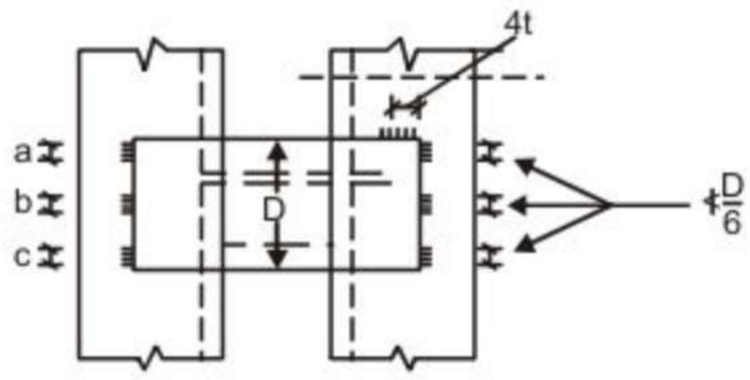
End connections should also be designed to resist the longitudinal shear force V_1 and the moment M.

For welded connection :

(1) Lap $\nless 4t$

(2) Total length of weld at edge of batten $< \frac{D}{2}$

$$a + b + c \nless \frac{D}{2}$$



Let t = thickness of batten

Length of continuous weld at each edge of batten $< \frac{1}{3}$ of total length required

Return weld along transverse axis of the column $< 4t$

where t and D are thickness and overall depth of battens respectively.

SOLVED EXAMPLES

1. *A column consisting of a steel channel section, ISMC 225 at 25.9 kgf/m, has an unsupported length of 3.5 m. It is effectively held in position and restrained against rotation at one end and restrained against rotation but not in position at the other end. Find axial load the column can carry if the steel confirms to IS : 226-1975.*

Solution: Yield stress of steel as per IS : 226 = 250 N/mm^2

Properties of ISMC 225 channel section :

$$\text{Sectional area, } a = 33.01 \times 10^2 \text{ mm}^2$$

$$\text{Radius of gyration, } r_{xx} = 90.3 \text{ mm}, r_{yy} = 23.8 \text{ mm}$$

$$\text{Effective length of column, } l = 1.2 \times 3.5 = 4.2 \text{ m} = 4200 \text{ mm}$$

$$\text{Maximum slenderness ratio, } \lambda_{maximum} = \frac{l}{r_{min}} = \frac{4200.0}{23.8} = 176.50$$

$$\text{From IS : 800, T Permissible stress, } \sigma_{ac} = 33 + (37 - 33)\frac{3.5}{10} = 34.4 \text{ MPa}$$

$$\text{Allowable axial compressive load} = \frac{34.4}{1000} \times 33.01 \times 10^2 = \mathbf{113.6\ kN}$$

2. *A single angle, ISA 60 × 60 × 6 mm of a roof truss is 1.50 m long. It is connected by one rivet at each end. Find safe load the strut can carry.*

Solution:

$$\text{Sectional area, } a = 6.84 \text{ cm}^2$$

$$\text{Least radius of gyration, } r = 1.15 \text{ cm}$$

$$\text{Effective length} = 1.50 \text{ m}$$

$$\text{Maximum slenderness ratio, } \lambda_{max.} = \frac{1.5 \times 100}{1.15} = 130.43$$

$$\text{Assume yield stress of steel} = 250 \text{ N/mm}^2$$

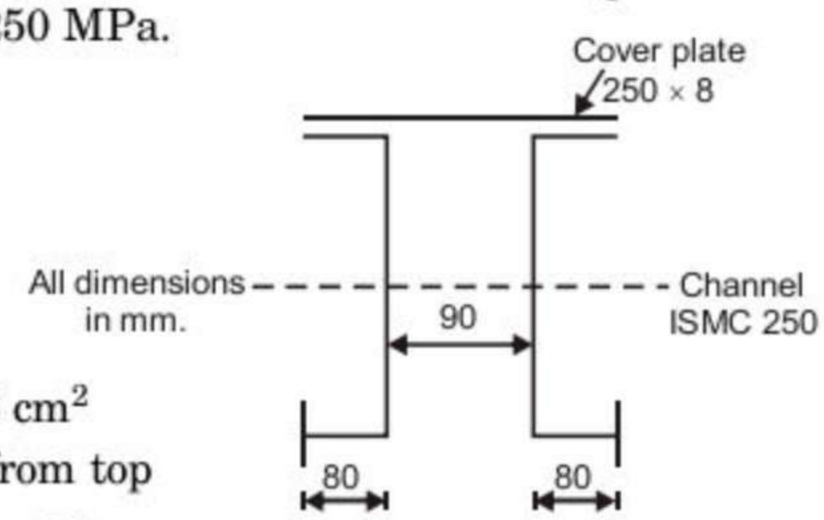
$$\sigma_{ac} = 57 - (57 - 51) \times \frac{43}{10} = 56.74 \text{ MPa (N/mm}^2)$$

Allowable axial compressive stress for discontinuous single angle strut connected by single rivets at ends

$$= 0.8 \times \sigma_{ac} = 0.8 \times 56.74 = 45.39 \text{ MPa}$$

$$\text{Permissible axial load in compression} = \frac{45.394 \times 6.84 \times 100}{1000} = \mathbf{31.05\ kN.}$$

3. Top chord of a bridge truss has an effective length of 5 m and cross-section as shown in the figure. Find compressive load the member can carry if yield stress of steel is 250 MPa.



Solution:

From steel tables for channel section, ISMC 250 at 30.6 kgf/m

Sectional area = 38.6 cm^2

Moment of inertia, $I_{xx} = 3816.8\ \text{cm}^4$

Distance of C.G. along *yy*-axis from web-face, $c_{yy} = 2.30$ cm

Cross-sectional area of chord member = $2 \times 38.6 + 25 \times 0.8 = 97.2\ \text{cm}^2$

Taking moments about top fibre, depth of *c.g.* of built-up section from top

$$\bar{y} = \frac{(2 \times 38.6) \times (12.5 + 0.8) + 25 \times 0.8 \times 0.4}{97.2} = 10.65 \text{ cm}$$

Moment of Inertia of cross-section

$$I_x = 2 \times 3816.8 + 2 \times 38.67\,(13.3 - 10.65)^2 + \frac{1}{12} \times 25 \times 0.8^3 + 25 \times 0.8\,(10.65 - 0.4)^2 = 10279.04 \text{ cm}^4$$

$$I_y = 2 \times 219.1 + 2 \times 38.67\,(4.50 + 2.30)^2 \times \frac{1}{12} \times 0.8 \times 25^3 = 5056.07 \text{ cm}^4$$

As $\qquad I_y < I_x \quad \text{and} \quad r_y < r_x$

$$\therefore \qquad r_{\min} = r_y = \sqrt{\frac{I_y}{A}} = \sqrt{\frac{5056.07}{97.20}} = 7.21 \text{ cm}$$

$$\text{Maximum slenderness ratio, } \lambda_{\max} = \frac{l}{r_{\min}} = \frac{500}{7.21} = 69.35, \text{ say, } 70$$

From IS : 800, $\sigma_{ac} = 112$ MPa

$$\therefore \qquad \text{Safe loading capacity of strut} = \frac{112}{1000}\,(97.20) \times 100 = \mathbf{1088.64\ kN}$$

Steps to design the compression member

(1) Assume some value of permissible stress σ_{ac} and calculate approximate gross sectional area, A required. For single component sections, assume σ_{ac} = 80 MPa, and for built-up sections, σ_{ac} = 110 MPa.

(2) Select a trial section having a sectional area approximately equal to the area calculated.

(3) Determine maximum $\frac{l}{r}$ ratio and the corresponding permissible σ_{ac}.

(4) Calculate safe load for the trial section. If safe load is equal or slightly more than the design load, adopt the trial section, otherwise, revise the design.

(5) For the final design, design lacing system and battens, if required by the section configuration.

DESIGN OF BEAMS AND BEAM COLUMNS

Beams transmit transverse loads normal to their axis to end supports through bending.

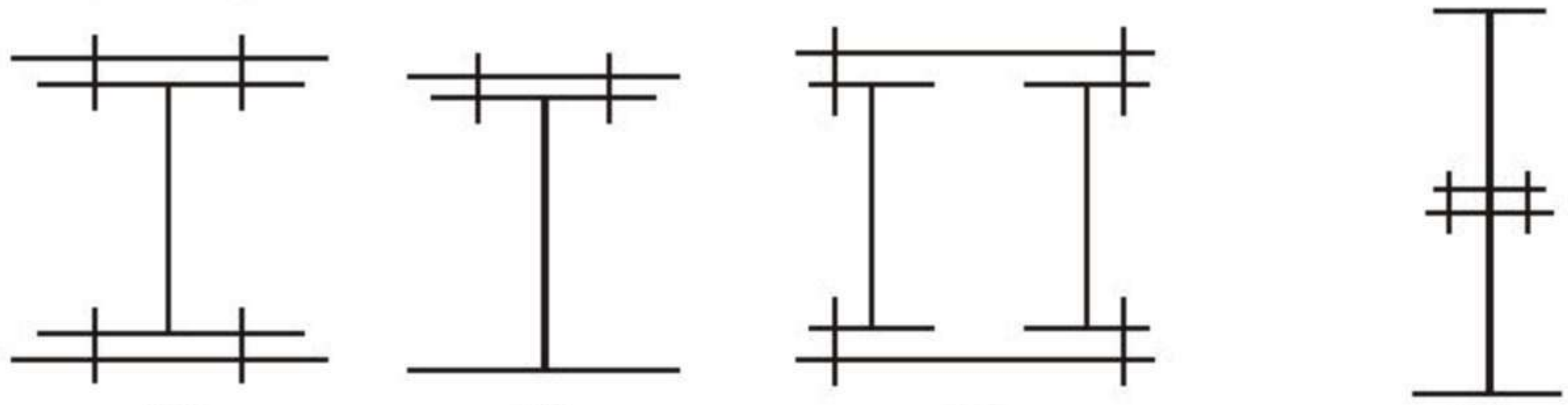
Built-up beams made up of rolled sections and cover plates are used when

(i) available rolled sections do not have adequate moment of inertia,

(ii) beams have to be provided restricted-depth due to site conditions,

(iii) spans are long, but the loads are not heavy.

Configurations generally used are shown in the figure below.



(*a*) (*b*) (*c*) (*d*)

The design of built up beams is done approximately to simplify the calculations.

Design

1. Symmetrical Built-up Beams.

Moment of inertia of a symmetrical built-up section shown in Figs. (*a*) to (*d*) is approximately given by

$$I = I_B + 2\,A_f \times \left(\frac{d}{2}\right)^2 \qquad \ldots(i)$$

where, I_B = moment of inertia of rolled I-section
A_f = area of each flange plate
d = depth of the beam

Dividing equation (*i*) by $\left(\frac{d}{2}\right)$, we get $\quad Z = Z_B + A_f\,d$

where, Z_B = modulus of section of the beam
Z = modulus of section of the built-up beam

$\because$ the area of each cover plate, $\quad A_f = \dfrac{Z - Z_B}{d} \qquad \ldots(ii)$

Required built-up section can be chosen, using an I-section with a section modulus Z_B and flange plates having a net area of A_f each. Gross cross-sectional area of the flange plate is taken 20 per cent more than the net cross-sectional area, to allow for rivet holes and approximations in calculations.

2. Unsymmetrical Built-up Beams

Area of cover plate may be calculated approximately by, $\quad A_f = \dfrac{(1.20)\,(Z - Z_B)}{d}$

After selecting size of the cover plate, neutral axis should be determined for the compound section for evaluating fibre stresses.

Rivet Connections

Rivets connecting cover plates to flanges are designed to resist the longitudinal shear stress τ_v at flange plate level.

$$\tau_v = \frac{V}{Ib}(A\bar{y})$$

or horizontal shear per unit (centimetre) length of beam is

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

where, V = shear force (vertical) at the section considered
I = moment of inertia of the compound section
$A\bar{y}$ = moment of area of plate above flange of I-section about the neutral axis.

$$= b \times t \times \left(\frac{d}{2} + \frac{t}{2}\right)$$

where, b = width of the flange plate
t = thickness of the flange plate.

If p is pitch of rivets, then

$$\text{horizontal shear force per pitch length} = \frac{V\,(A\bar{y})\,p}{I} = \frac{V \cdot b \cdot t\left(\frac{d}{2} + \frac{t}{2}\right) \cdot p}{I}$$

If rivet value is R and n rivets are in a pitch length, then

$$n \times R = \frac{V \cdot b \cdot t\left(\frac{d}{2} + \frac{t}{2}\right)}{I}$$

or $\quad$ pitch of rivets, $\quad p = \dfrac{n \times R \times I}{V \times b \times t \times \dfrac{d+t}{2}}$

Pitch of rivets provided should not exceed the value p given by above equation. Outstand of the flange plate beyond rivets in compression and tension should not exceed $16t$ and $20t$, respectively, for steel conforming to IS : 226.

Plate Thickness

Outstand of cover plates, flange plates, angle legs, etc. should be within the limits given in the Table to avoid local buckling.

Type	Width of plate section	Maximum width
1. Flanges and plates in compression, unstiffened.	Flange–half the nominal width of flange of beam (I) and tee sections.	$\frac{256t}{\sqrt{f_y}}$ subject to a maximum of $16t$
2. Flanges and plates in compression, stiffened.	Plates – distance from the tree edge to the first row of rivets or welds.	$20t$
3. Flanges and plates in tension	From edge to the innermost face of the stiffener.	$20t$

Here t is thickness of a single plate or total thickness of two or more plates effectively tacked together.

When a plate is connected to other parts of a built-up member, then width between any two adjacent lines of connections or supports should not exceed following :

(1) *For plates in compression* $\frac{1440\,t}{\sqrt{f_y}}$ subject to a maximum of $90\,t$.

However, width should not exceed $\frac{560\,t}{\sqrt{f_y}}$ subject to a maximum of $35t$ for welded plates which are not stress relieved, or $\frac{800\,t}{\sqrt{f_y}}$ subject to a maximum of $50\,t$.

For other plates, the excess width should be assumed to be located centrally and its sectional area neglected when calculating the effective geometric properties of the section.

(2) *For plates in uniform tension* : $100\,t$

However, excess width over $60\,t$ should be assumed to be located centrally and its sectional area neglected when calculating the geometrical properties of the section.

Check for Stresses

Built-up section should be checked for bending stresses in extreme fibres. Stresses are first worked out on the basis of gross moment of inertia. Then maximum flexural stresses are determined by increasing stresses arrived at on the basis of gross moment of inertia in the ratio of gross area to the effective area of the flange section. Effective area of the compression flange should be the gross area with deductions for excessive width of plates and for open holes occurring in a plane perpendicular to the direction of stress at the section considered. Effective area of tension flanges should be gross cross-sectional area of the flange with deduction for rivet holes.

The section should also be checked for maximum shear stress, considering whole area of the cross-section, having regard to the actual distribution of shear stress.

Design of Beams

A beam section is usually selected which can resist maximum bending moment occuring over its span. Shear stress and deflection for the section are then checked to determine whether they are within the permissible limits. Check for web crippling and web buckling are the secondary design requirements.

(1) Design for Bending.

Bending stress $\sigma_{bc\text{ or }bt}$ compressive or tensile, at any point on a cross-section of a beam due to bending moment, M is given by

$$\sigma_{bc\text{ or }bt} = \frac{M}{I}Y$$

where, M = bending moment

I = moment of inertia of the cross-section

$\sigma_{bc\text{ or }bt}$ = bending stress (compressive or tensile) calculated at any point at a distance y from N.A.

The ratio $\frac{I}{Y}$ is called *section modulus Z.*

Since calculated bending stress $\sigma_{bc \text{ or } bt}$ should be the permissible bending stress, therefore

$$\frac{M}{Z} \leq \sigma_{bc \text{ or } bt}, \quad \text{or} \quad Z \geq \frac{M}{\sigma_{bc \text{ or } bt}}$$

Suitable beam sections selected which has sectional modulus slightly more than Z value calculated.

Moment of Resistance = Z × $\sigma_{bc \text{ or } bt}$ (It is bending moment which a beam compresive)

(a) Permissible Bending Stress.

Laterally restrained beams : If compression flange is restrained laterally against buckling or if the beam is to bend about the axis of minimum strength (y-y axis), the premissible compressive or tensile bending stress.

$$\sigma_{bc \text{ or } bt} = 0.66 f_y$$

where, f_y = yield stress of steel

Permissible bending stress $\sigma_{bc \text{ or } bt}$ is given in the table below :

Nominal plate thickness	Yield stress, f_y (MPa)	$\sigma_{bc} = \sigma_{bt}$ (MPa)
1. Angle, tee, I channel and flat sections up to and including 20 mm	250	165
2. Over 20 mm upto and including 40 mm	240	158.4
3. Over 40 mm	230	151.8

For beams bent about the axis of maximum strength (*x-x* axis), permissible bending stress in compression,

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

where, f_{cb} = elastic critical stress in bending (MPa)

f_y = yield stress of steel (MPa)

n = *a* factor assumed as 1.4

Elastic critical stress, f_{cb}, can be determined by elastic flexural-torsional buckling analysis. In the absence of such an analysis, f_{cb} can be calculated for girders with I_y smaller than I_x, using the formula

$$f_{cb} = k_1 (X + k_2 Y) \frac{c_2}{c_1}$$

$$X = Y \sqrt{1 + \frac{1}{20} \left(\frac{T l}{r_y D} \right)^2} \text{ MPa, and } Y = \frac{26.5 \times 10^5}{\left(\frac{l}{r_y} \right)^2} \text{ MPa}$$

where, k_1 = a coefficient to allow for reduction in thickness or breadth of flanges between points of effective lateral restraint

l = effective length of compression flange

r_y = radius of gyration of the section about its axis of minimum strength (*y-y* axis)

T = mean thickness of compression flange, equal to area of the horizontal portion of flange divided by width

D = overall depth of beam

k_2 = a coefficient to allow for inequality of flanges

c_1, c_2 = lesser and greater distances from section neutral axis to the extreme fibres respectively

I_y = M.I. of the whole section about the axis lying in the plane of bending (*y-y* axis)

I_x = M.I. of the whole section about the axis normal to the plane of bending (*x-x* axis).

The value of k, is dependent on another factor, Ψ, defined as ratio of both flanges at the point of least bending moment to the corresponding area at the point of the greatest bending moment between points of effective lateral restraint.

Value of k_1 is equal to 1 if there is no curtailment of flanges.

Value of k_2 is dependent on a factor, ω, defined as ratio of the moment of inertia of the compression flange alone to that of the sum of the moment of inertia of the flanges, each calculated about its own axis parallel to the *y-y* axis of the girder, at the point of maximum bending moment.

Values of f_{cb} should be increased by 20 per cent, when

$$\left(\frac{T}{t}\right) \not> 2.0; \text{ and } \left(\frac{d_1}{t}\right) \not> \frac{1344}{\sqrt{f_y}}$$

where, d_1 = appropriate depth of web depending on provision of horizontal stiffeners

t = thickness of web.

Maximum permissible bending stress in tension σ_{bt} or in compression σ_{bc} for beams bent about the axis of minimum strength (*y-y* axis) should not exceed $0.66 f_y$, where f_y is yield stress of steel.

(*b*) Permissible Bearing Stress

Bearing stress in any part of a beam when calculated on the net area of contact should not exceed the value of σ_p given by

$$\sigma_p = 0.75 f_y$$

where, σ_p = maximum permissible bearing stress

f_y = yield stress of steel.

(*c*) Permissible Shear Stress

Maximum shear stress in a member, having regard to the distribution of stresses in conformity with elastic behaviour of the member in flexure, should not exceed the value given by

$$\tau_{vm} = 0.45 f_y$$

where, τ_{vm} = maximum permissible shear stress

f_y = yield stress of steel.

(*d*) Check for Shear.

Shear stress at any point on the cross-section of a beam is given by

$$\tau = \frac{V}{I \times b} \times A \times \bar{y}$$

where, V = shear force at the section

I = moment of inertia of the section

b = width of the section at the point where shear stress is calculated

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

For rolled I section and channel section,

$$\text{Average shear stress, } \tau_{va} = \frac{\text{shear force}}{\text{depth of beam} \times \text{web thickness}} \text{ or } \tau_{va} = 0.4 f_y$$

Nominal thickness (mm)	Permissible average shear stress τ_{va} (MPa)
Upto 20	100
Over 20 upto and including 40	96
Over 40	92

(*e*) Check for Deflection.

Maximum deflection of a beam should not exceed $\frac{1}{325}$ of the span.

$$\text{Maximum deflection, } \delta = K \times \frac{W \times l^3}{E \times I_x} \text{ cm}$$

where, l = effective span

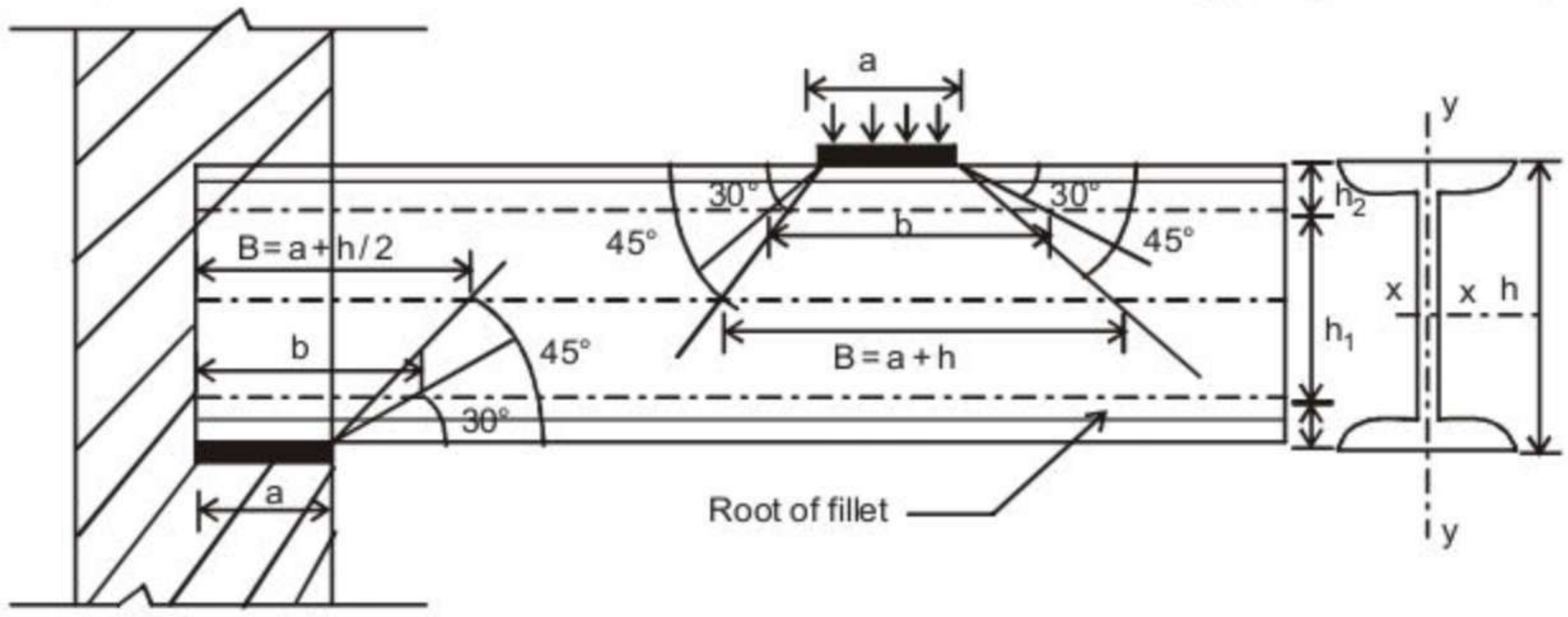
E = Young's modulus of elasticity

I_x = moment of inertia about *xx* axis

K = coefficient of maximum deflection

(f) Cheack for web crippling and web buckling

A beam may for under a concantrated load or at end reaction due to crippling of web or by buckling of web.



Web Crippling : The dispersion of load is assumed to be at 30° as shown in figure. The bearing stress in the web at the root of the fillet will be equal to

$$\frac{W}{t_w\left(a + h_2\sqrt{3}\right)} \not> \sigma_P \qquad \text{for intermediate loads}$$

$$\frac{R}{t_w\left(a + h_2\sqrt{3}\right)} \not> \sigma_P \qquad \text{for end supports}$$

Where, W = concentrated load on beam

R = end reaction at support

t_w = thickness of web (mm)

a = bearing length

h_2 = depth of the root of fillet from the top of flange (mm)

σ_P = maximum permissible bearing stress = $0.75\, f_y$

f_y = yield stress of steel.

Web buckling : The load bearing stiffeners at all points of concentrated loads (including points of supports) should be provided where

$$\text{Work} > \sigma_{a_c} \times \text{fw} \times \text{B}$$

Where, work = concentrated load or reaction at support respectively.

σ_{ac} = maximum permissible axial stress for column for slanderness ratio = $\dfrac{h_1}{t_w}\sqrt{3}$

t_w = web thickness

B = the length of the stiff portion of the bearing plus the addition length given by the dispersion of 45° to the level of neutral axis, plus thickness of seating angle if any see in figure.

h_1 = clear depth of web between roof fillets.

Example. *A beam of 7 m effective span carries a uniformly distributed load of 20 N/mm including its own weight. Assume allowable stresses and deflection according to IS 800. Compression flange is held against lateral displacement. Find required section modulus*

Solution:

$$\text{Maximum bending moment,} \quad M = \frac{20 \times 7000}{8} = 1225 \times 10^5 \text{ N/mm}$$

$$\text{Maximum share force} = \frac{20 \times 7000}{2} = 70000 \text{ N}$$

$$\text{Allowable bending stress,} \quad F_b = 165 \text{ N/mm}^2$$

$$\therefore \quad \text{Required section modulus,} \quad Z = \frac{M}{f_b} = \frac{1225 \times 10^5}{165} = \mathbf{7.24 \times 10^5\ mm^3}$$

Effective Span

Effective span of a beam should be taken as length between centres of supports, except in cases where point of application of the reaction is taken as eccentric to the support, when it is permissible to take effective span as the length between assumed points of application of reaction.

Effective Length of Compression Flange

When lateral deflection of the compression flange is prevented by providing lateral support, the beam is said to be *laterally supported*. Effective lateral restraint is the restraint which produces sufficient resistance in a plane perpendicular to the plane of bending to restrain the compression flange of a beam from lateral buckling to either side at the point of application of the restraint. In fully restrained beams, permissible compressive stress in bending is taken as equal to permissible bending tensile stress.

In laterally unsupported beams, permissible compressive stress in bending, f_{bc} is dependent on the elastic critical stress in bending f_{cb}, and for evaluating f_{cb}, effective length of compression flange, l, has to be used.

For simply supported beams and girders where no lateral restraint of the compression flanges is provided, but where each end of the beam is restrained against torsion, effective span of the compression flanges should be taken from the table below :

End Conditions	Effective span
1. Ends of compression flanges unrestrained against lateral bending *i.e.,* free to rotate in plan at the bearings.	l = span
2. Ends of compression flanges partially restrained against lateral bending *i.e.,* not fully free to rotate in plan at the bearings	$l = 0.85 \times$ span
3. Ends of compression flanges fully restrained against lateral bending *i.e.,* not free to rotate in plan at the bearings	$l = 0.70 \times$ span

Restraint against torsion : *It can be provided by following :*

(i) Web or flange cleats

(ii) Bearing stiffeners acting in conjunction with bearing of the beam

(iii)Lateral end frames or other external supports to the ends of the compression flanges

(iv) Walls into which the ends of the beams are built.

When ends of the beam are not restrained against torsion, or when load is applied to the compression flange and both load and the flange are free to move laterally, then values given in the table should be increased by 20 per cent.

DESIGN OF CONNECTIONS : COLOUMN SLABS AND GUSSETED BASES

Columns transfer their loads to the soil through column bases resting over concrete or masonry blocks. A column base distributes the load over a greater area, so that pressure on the concrete block does not exceed the permissible bearing stress.

Column Bases.

These are of following two types :

1. Slab base
2. Gusseted base

1. Slab Base

Column end is machined to rest over a steel base plate. Column with slab bases need not be provided with gussets, but fastenings should be provided, sufficient to retain the parts securely in place and to resist all moments and forces. Other than direct compression, including those arising during transit, unloading and erection.

(*i*) *When slab alone distributes the load uniformly :* Minimum thickness of a rectangular slab is

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

where, t = slab thickness (mm)

w = pressure or loading on the underside of the base (MPa)

σ_{bs} = permissible bending stress in slab bases = 185 MPa for steels

a = greater projection of the plate beyond the column (mm)

b = lesser projection of the plate beyond the column (mm) (*ii*) *When slab does not distribute the loading uniformly or where slab is not rectangular :* Special calculations are necessary to show that the stresses are within the specified limits.

(*iii*) *For solid round steel columns, where loading on the cap or under the base is uniformly distributed over the whole area including the column shaft :* Minimum thickness of the square cap or base is given by

$$t = 10\sqrt{\frac{90\ \mathrm{W}}{16\,\sigma_{bs}} \times \frac{\mathrm{B}}{\mathrm{B} - d_0}}$$

where, t = thickness of the plate (mm)

W = total axial load (kN)

B = length of the side of cap or base (mm)

σ_{bs} = permissible bending stress in slab bases = 185 MPa for steels

d_0 = diameter of the reduced end, if any, of the column (mm)

(*iv*) *When load on the cap or under the base is not uniformly distributed or where end of the column shaft is not machined with the cap or base, or where the cap or base is not square in plan :* Calculations should be made based on the allowable stress of 185 MPa.

Base or cap should not be less than 1.5 (d_0 + 75) mm in length or diameter.

$$\text{Area of slab base} = \frac{\text{Axial load in the column}}{\text{Permissible compressive stress in concrete}}$$

Example. *A rolled column section, SC 250 @ 85.6 kg/m, carries an axial load of 70 tonnes. The allowable bearing pressure on concrete is 45 kg/cm². Overall size of column section, SC 250 @ 85.6 kg/m, is 250 × 250 mm. Find required thickness of the base plate for the column.*

Solution:

Axial load of column = 70 tonnes

$$\text{Required area of slab base} = \frac{70 \times 1000}{45} = 1555.56\ \text{cm}^2$$

$$\text{Side of squre base} = \sqrt{1555.56} = 39.44\ \text{cm} \simeq 40\ \text{cm}$$

$$\text{Bearing pressure below base, } w = \frac{70 \times 1000}{40 \times 40} = 43.75\ \text{kg/cm}^2$$

$$\text{Thickness of the plate,} \quad t = \sqrt{\frac{3w}{\sigma_{bs}}\left(a^2 - \frac{b^2}{4}\right)}$$

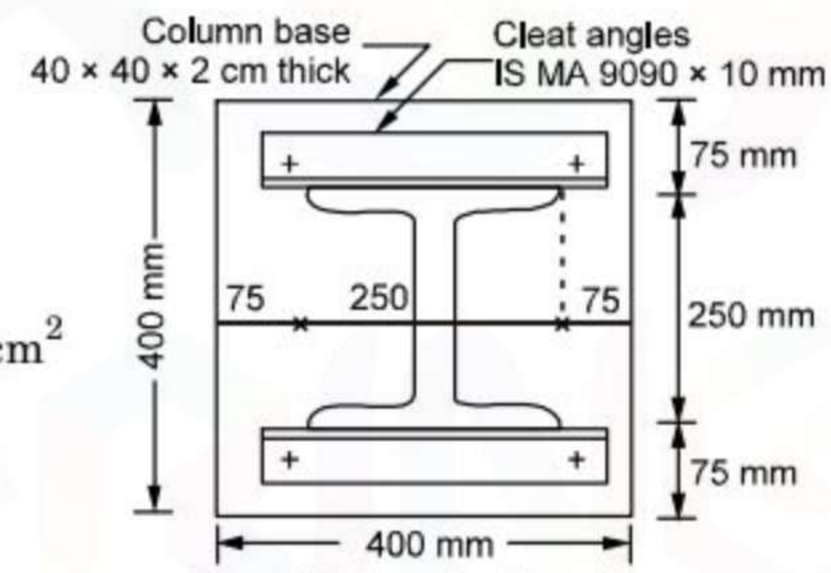


Fig. : Slab base for the column

where, w = 43.75 kg/cm²

$a = b$ = 75 mm = 7.5 cm

σ_{bs} = 185 MPa = 1885 kg/cm²

$$\therefore \qquad t = \sqrt{\frac{3 \times 43.76}{1855}\left(7.5^2 - \frac{7.5^2}{4}\right)} = 1.71\ \text{cm} \approx \mathbf{2\ cm.}$$

SOLVED EXAMPLES

1. *A column section consisting of ISHB 300 @ 63.0 kg/m with one cover plate 400 mm wide and 20 mm thick connected to each flange carries an axial load of 300 t all inclusive. In gussted base, allowable bearing pressure on concrete is 40 kg/cm². Allowable bending stress in base plate is 1800 kg/cm². Find thickness required of the gusseted plate*

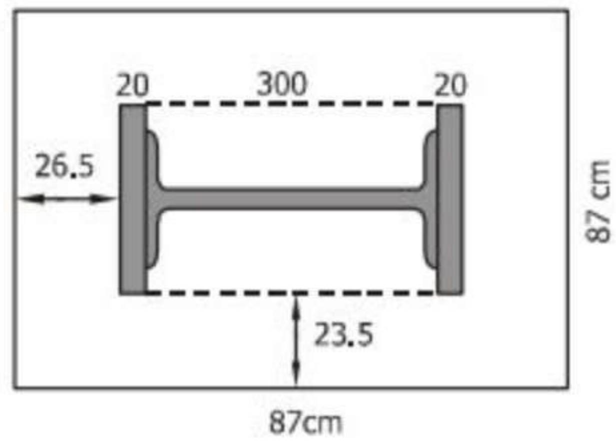
Solution:

$$\text{Area of the base plate} = \frac{\text{Axial load}}{\text{Permissible stress in compression in concrete}}$$

$$= \frac{300{,}000}{40} = 7500\ \text{cm}^2$$

Side of the square base plate = 86.6 cm ≈ 87 cm

Here adopt a slate base 87 × 87 cm.



Thickness of base plate, $t = \sqrt{\dfrac{3w}{P_{bc}}\left(A^2 - \dfrac{B^2}{4}\right)}$ ...(*i*)

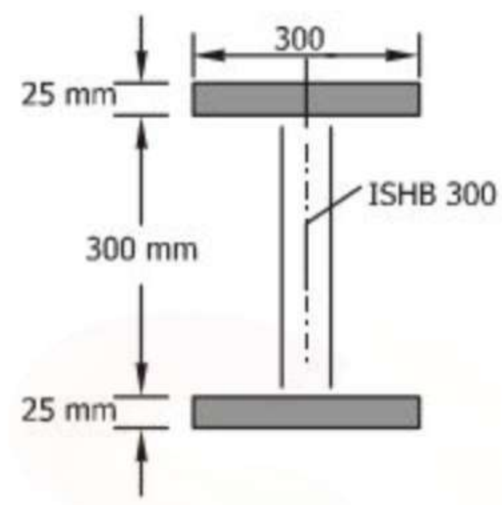
where, $w = \dfrac{300 \times 1000}{87 \times 87} = 39.6 \text{ kg/cm}^2$

$$A = \frac{870 - (300 + 20 + 20)}{2} = 265 \text{ mm} = 26.5 \text{ cm.}$$

$$B = \frac{870 - 400}{2} = 235 \text{ mm} = 23.5 \text{ cm.}$$

$$\therefore \quad t = \sqrt{\frac{3 \times 39.6}{1890}\left(26.52^2 - \frac{23.5^2}{4}\right)} = \textbf{5.955 say 6 cm.}$$

2. *In the base plate for a column of one ISHB 300 @ 63 kg and two cover plates 300 × 25 mm as shown in the figure and carring an axial load of 230 tonnes. Indicate also jointing to the base plate without going into the design details there of. The column is to be supported on a concrete pedestal. The permissible bearing pressure on concrete is 40 kg/cm². It may be assumed that the column end is machined to transfer the load to the slab base by direct bearing. The permissible bending stress for slab base can be taken as 1890 kg/cm². Find required thickness of the base plate*

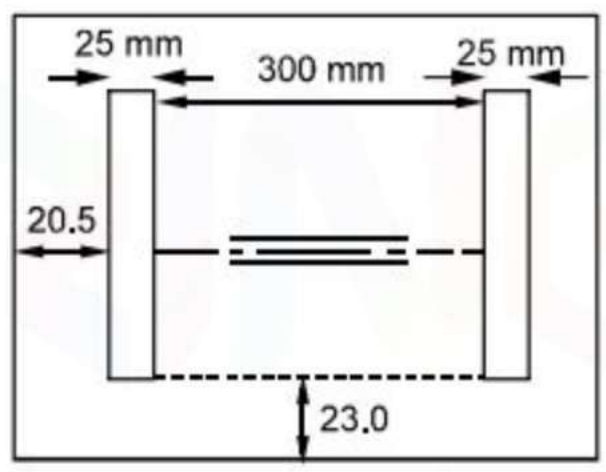


Solution:

$$\text{Area of base plate} = \frac{\text{Axial Load}}{\text{Permissible stress in compression in concrete}} = \frac{230{,}000}{40} = 5750 \text{ cm}^2$$

$\therefore$ Side of the square base plate = $\sqrt{5750}$ = 75.8, say, 76 cm.

Use a slab 76 cm × 76 cm.



Thickness of base plate, $t = \sqrt{\dfrac{3w}{P_{bc}}\left(A^2 - \dfrac{B^2}{4}\right)}$ ...(*i*)

where, $w = \dfrac{230{,}000}{76 \times 76} = 39.82$, A = 23 cm, B = 20.5 cm

$$\therefore \quad t = \sqrt{\frac{3 \times 39.82}{1890}\left(23^2 - \frac{(20.5)^2}{4}\right)} = 5.135 \text{ cm, say, } 5.2 \text{ cm}$$

3. *A column ISHB 250 carries a load 550 kN at a distance of 40 mm from center line of column and along its y-y axis. Find size of the base plate if $p_{bc} = 4 \text{ N/mm}^2$.*

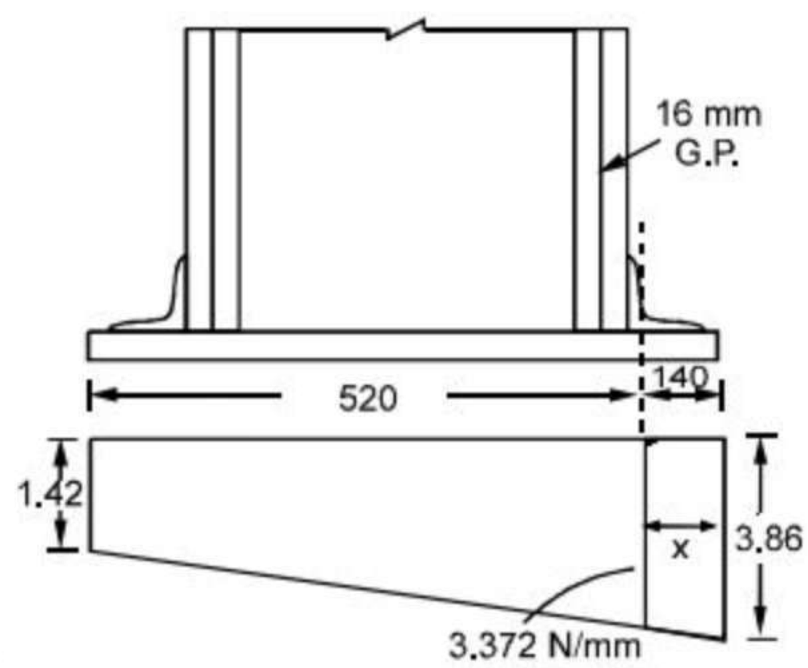
Solution:

Given : P = 550 kN; *e* = 40 mm; M = 22 × 10^3 kN-mm

Use angle ISA 150 × 115 × 15 and 16 mm thick G.P.

$$L = 250 + 2 \times 16 + 2 \times 115 = 512$$

Take L = 520



For Base Plate : $\dfrac{e}{L} = \dfrac{40}{520} < \dfrac{1}{6}$

$\therefore$ Width of base plate, $b = \dfrac{P}{L \times p_{bc}}\left(1 + 6\dfrac{e}{L}\right)$

$$\phi = \frac{550 \times 10^3}{520 \times 4}\left(1 + \frac{6 \times 40}{400}\right) = 386 \text{ mm.}$$

Adopt $b = 400$ mm

$$p_{bc} = \frac{550 \times 10^3}{520 \times 400}\left(1 \pm \frac{6 \times 40}{520}\right)$$

$\therefore$ $$p_{\max} = 3.86 \text{ N/mm}^2, \quad \text{and} \quad p_{\min} = 1.42 \text{ N/mm}^2$$

$$\bar{x} = \frac{3.37 + 2 \times 3.86}{3.37 \times 3.86} \times \frac{104}{3} = 53.17 \text{ mm}$$

$$M = \frac{3.86 + 3.37}{2} \times 104 \times 53.17 = 20 \times 10^3 \text{ N-mm}$$

Now $$\frac{1}{6} t^2 \times 165 = 20 \times 10^3$$

$\Rightarrow$ $$t = 27 \text{ mm}$$

Adopt base plate thickness = 27 – 15 = 12 mm.

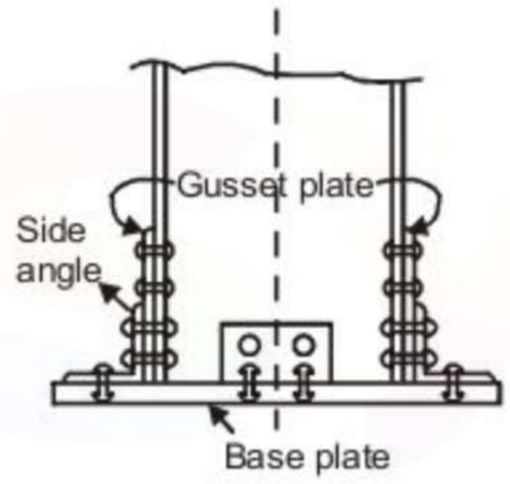
Provide 520 × 400 × 16 mm thick base plate.

2. Gusseted bases

For columns carrying heavy loads gusseted bases are provided. For such columns, fastenings including gusset plates, angle cleats, stiffeners etc. in combination with the bearing area of the shaft, all fabricated flush for bearing, shall be sufficient to take the loads, bending moments and reactions to the base plate without exceeding specified stresses.

Where ends of the column shaft and the gusset plates are not faced for complete bearing, fastenings connecting them to the base plate shall be sufficient to transmit all the forces to which the base is subjected.

Base plate shall be of adequate strength to spread the load upon the concrete, masonry or other foundations without exceeding permissible stress on such foundation under any combination of loads and bending moments.

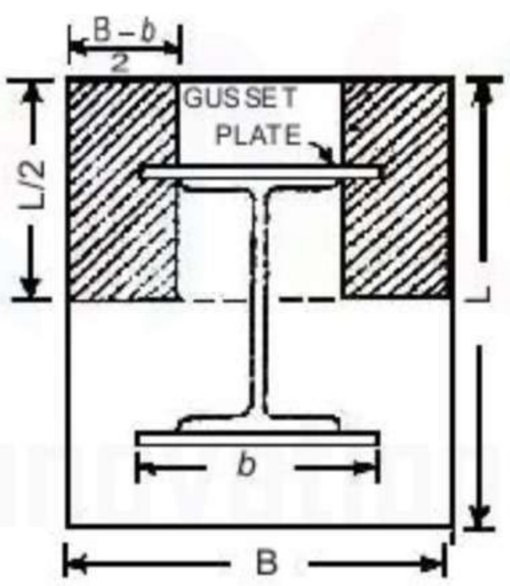


Design of gusset plate.

Force on each gusset plate will be equal to pressure on the shaded area shown in figure

$$\text{Force} = 2w \times L/2\left[\frac{B-b}{2}\right]$$

$$= \frac{wL}{2}(B-b)$$



Number of rivets connecting gusset to column will be the force in the gusset divided by the strength of rivet.

If gusset plate is connected by welding, effective length of the weld will be the force in the gusset divided by the strength of weld used per cm.

Gusset plate will be subjected to B.M.

$$= \frac{wL}{4}(B-b) \times \frac{(B-b)}{4}$$

$$= \frac{wL^2}{16}(B-b)^2$$

Section of gusset plate should be adequate to resist bending moment. Gusset plate should not be less than 1 cm. thick.

Design of side or cleat angle.

Force in gusset plate is transferred to the cleat angle by bearing. Area in bearing of the cleat angle should be sufficient so that bearing stress should not exceed 1890 kg/cm^2. Number of rivets connecting side angle to the gusset plate will be same as the number of rivets connecting gusset plate to the column.

Design of base plate.

Base plate and cleat angle are considered to act together. For this sufficient number of rivets should be provided in connecting side angle to the base plate. Number of rivets provided should be sufficient to take horizontal shear. Thickness of base plate should be at least equal to side angle.

If gusset plate is welded to the base plate, thickess of the base plate is based on the bending moment on the projection beyond the gusset plate.

DESIGN OF SEMI-RIGID AND RIGID CONNECTIONS

In case of continuous constructions, joints are used to transmit large moments in addition to shears.

Types of Connections

There are two types of connections in use for joining component members :

1. **Rigid Connection or Moment Resistant Connection**

In this joints are assumed to be completely rigid so that no change of angle is permitted.

Design

(*i*) **Moment Connection :** Moment carrying capacity of the connection is supplied by the rivets connecting the top angle Fig. (*a*) or top tee Fig. (*b*) with the column flange.

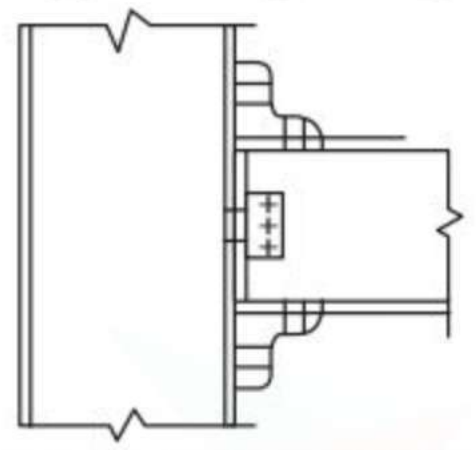


Fig. (a) Angle connection

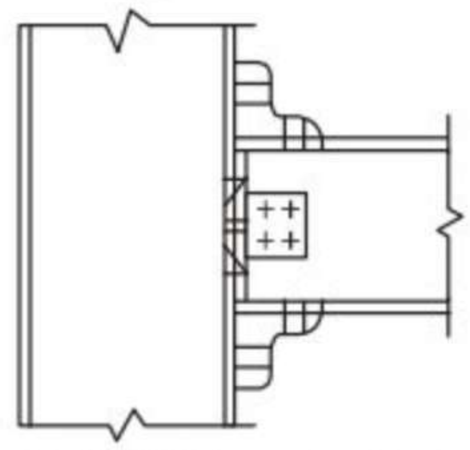


Fig. (b) Tee connection

$$\text{Force in each set of rivet} = \frac{\text{End moment of beam}}{\text{Lever arm between top and bottom rivets}}$$

To accommodate these rivets, in lieu of T-section, many times, an I section cut longitudinally at its mid-depth into two halves, called *split-beam section*, is used.

$$\text{Force in the rivets, R} = \frac{\text{Bending moment}}{\text{Lever arm}}$$

In case of angles, moment $= 0.6 \times R \times a$

In case of tee, moment $= 0.5 \times R \times a$

$$\text{Thickness of angles or tee, } t = \sqrt{\frac{M \times 6}{1890l}}$$

where, l = length of angle of tee section.

(*ii*) **Shear Connection :** Number of rivets on web of beam and number of rivets on flange of column can be calculated from the strength of rivet.

2. **Semi-rigid Connections**

Semi-rigid connections resists the end moment, but give a relative rotation between the connected members. In this deformation-moment characteristics of the joints should be known before using it. Some angle change does take place between connected parts.

There are two alternatives :

(*i*) End moment is provided for by the connection of flanges with the column and the shear force is separately catered by a framed connection on the web of the beam.

(*ii*) A deep bracket type connection is used to resist both the end shear and moment.

RIVETING AND BOLTING

Minimum Pitch

Distance between centres of rivets should not be less than 2.5 times the nominal diameter of the rivet.

Maximum Pitch

(*i*) Distance between centres of any adjacent rivets (including tacking rivets) should not exceed 300 mm or 32 t, whichever is less, where 't' is thickness of the thinner outside plate.

(*ii*) Distance between centres of two adjacent rivets in a line lying in the direction of stress should not exceed 200 mm or $16t$, whichever is less, in tension members, and 200 mm or $12t$, whichever is less in compression members.

(*iii*) Distance between centres of any two consecutive rivets in a line adjacent and parallel to an edge of an outside plate should not exceed 200 mm or (100 mm + 4t), whichever is less in compression members or tension members.

(*iv*) When rivets are staggered at equal intervals and the gauge does not exceed 75 mm, the distance specified in (*ii*) and (*iii*) between centres of rivets may be increased by 50 per cent.

Permissible Stresses in Rivets and Bolts.

1. **Calculation of Stresses.**

   In calculating shear and bending stresses, effective/gross diameter of a rivet should be taken as the hole diameter and that of a bolt as its nominal diameter.

   In calculating axial tensile stress in a rivet, gross area should be used and in calculating axial tensile stress in a bolt or screwed tension rod, net area shall be used.

2. **Calculation of Gross and Net area of Rivets and bolts.**

   Gross area of a rivet should be taken as cross-sectional area of the rivet hole.

3. **Area of Rivet and Bolt holes.**

   Gross Diameter of a rivet hole should be taken as nominal diameter of a rivet plus 1.5 mm for rivets of nominal diameter less than or equal to 25 mm, and 2.0 mm for rivets of nominal diameter exceeding 25 mm. The diameter of a bolt hole shall be taken as the nominal diameter of bolt plus 1.5 mm.

4. **Stresses in Rivets and Bolts.**

   Calculated stress in a mild steel shop rivet or in a bolt shall not exceed the allowable values.

5. Where two or more parts are connected together, a line of rivets or bolts should be provided at a distance or not more than 37 mm + 4t from the nearest edge, where 't' is thickness in mm of the thinner outside plate. In the case of work not exposed to weather, this may be increased to 12t.

WELDING

Fillet Welds

1. **Size of fillet weld.** It should not be less than the minimum allowable value given in the table.

Thickness of thicker part	Minimum size of fillet weld
Upto and including 10 mm	3 mm
Over 10 mm upto and including 20 mm	5 mm
Over 20 mm upto and including 32 mm	6 mm
Over 32 mm upto and including 50 mm	8 mm first run
10 mm minimum	

   Maximum size of the fillet weld applied to the square edge of a plate or shape should be 1.5 mm less than the nominal thickness of the edge. Size of the fillet weld used along tee of an angle or rounded endge of a flange should not exceed three-forth the nominal thickness of an angle of flange leg.

2. **Throat of fillet weld.** It is length of perpendicular from right angle corner to the hypotenuse.

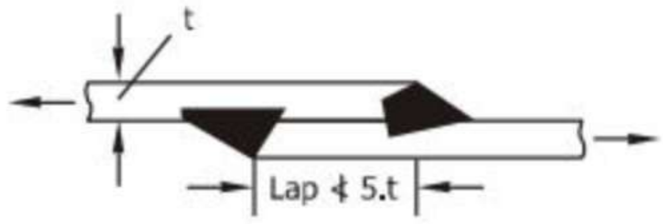
$$\text{Effective throat thickness} = k \times \text{fillet size}$$

   Value of k depends upon angle between fusion faces. Value of k decreases with increase in angle between fusion faces.

   In most cases, a right-angled fillet weld is used, for which $k = \dfrac{1}{\sqrt{2}}$ or 0.7 ·

3. **Effective length of fillet weld.** It is equal to its overall length minus twice the weld size. Effective length of a fillet weld designed to transmit loading should not be less than four times weld size. End return should be made equal to twice the size of the weld.

4. **Overlap.** Overlap in lap joint should not be less than five times the thickness of the thinner plate as shown in the figure below.

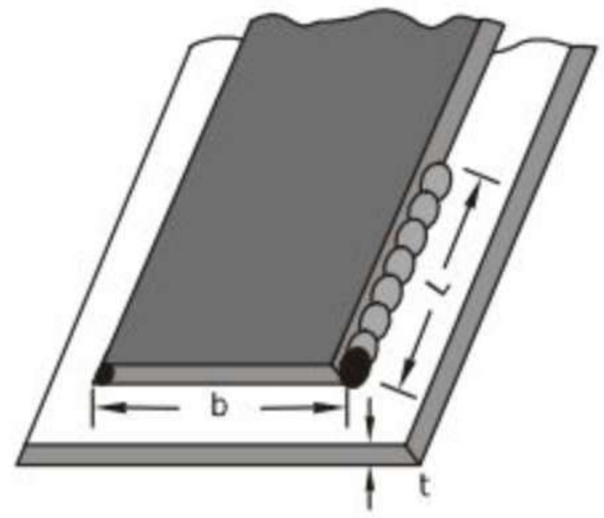


5. **Side fillet.** In a lap joint made by a side or longitudinal fillet weld, length of each fillet weld should not be less than perpendicular distances between them; the perendicular distance between side fillets should not exceed sixteen times thickness of the thinner part connected.

   In the figure, $l \nless b$ and $b \ngtr 16t$

   where t = thickness of thinner plate

   If 'b' exceeds the limit, then additional end fillet, plug weld or slot weld should be provided in addition to prevent buckling or separation of the parts.

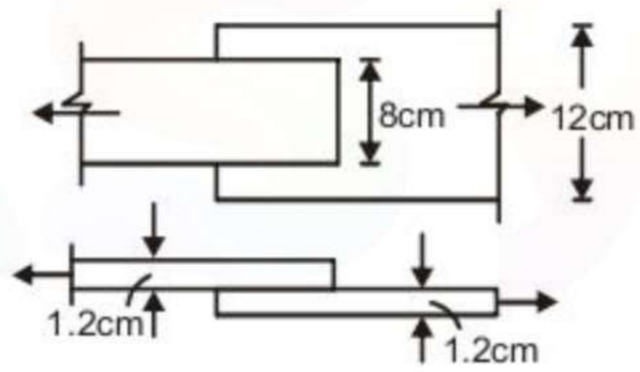


6. **Intermittent fillet weld.** Any section of an intermittent fillet weld should have an effective length of not less than four times the weld size or 40 mm, whichever is greater. Clear spacing between ends or effective lengths of inermittent fillet weld carrying stresses should not exceed $12t$ for compression and $16t$ for tension and in no case should be more than 20 cm, 't' is the thickness of thinner part joined.
7. **Permissible stress and Strength of fillet weld.**

   Permissible stress in fillet weld is 108 MPa or 1100 kgf/cm^2.

   Permissible stresses in shear and tension are reduced to 80% for the field welds made during erection. Permissible stresses are increased by 25% if wind or earthquake load are taken into account. However, size of the weld should not be less than the size required when the wind or earthquake load is considered or neglected.

Example. *Two plates are proposed to be joined by welding as shown in the figure. Determine size and length of the weld required to develop the full strength of the smaller plates which is 8 cm × 1.2 cm. Assume permissible tension in plate as 1500 kg / cm^2 and permissible shear stress in fillet weld as 1025 kg / cm^2*



Solution : Strength of 8 cm × 1.2 cm plate = 8 × 1.2 × 1500 = 14,400 kg

Maximm size of fillet weld required for thickness upto 19 mm = 5 mm

Maximum size of fillet weld is limited to thickness of plate = 12 – 1.5 = 10.5 say 10 mm

Provide a 6 mm fillet weld.

Strength per cm length of the weld = 1 × 0.7 × 0.6 × 1025 = 430.5 kg.

$$\text{Necessary length of the weld} = \frac{14400}{430.5} = 33.5 \text{ cm.}$$

Provide two side fillets of 17 cm. each.

Check : Transverse distance 8 cm is less than 16 × 1.2 = 19.2 cm. Hence safe

Design of Butt Weld

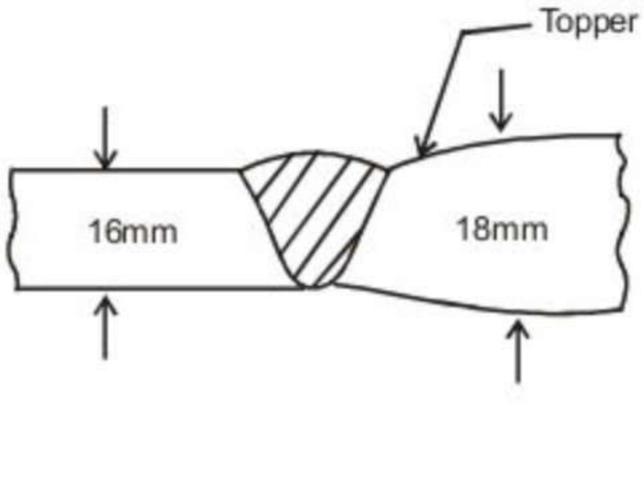
The strength of butt weld is taken equal to the strength of parts joined of full penetration of the weld metal is ensured.

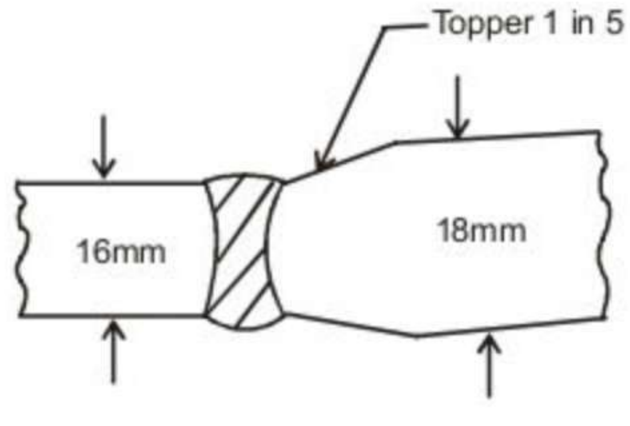
In case of single V, U, J and bevel joints, the penetration of the weld metal is generally incomplete and the effective throat thickness is taken as (5/8) × thickness of thinner part connected.

The permissible stress for butt weld is taken to be the same as that of parent metal.

Example. *The plates of dimensions 200 × 18 mm and 200 × 16 mm are joined by butt welding as shown in fig. Calculate the maximum tension the joint can transmit if (a) single V butt weld & (b) a double - V butt weld, is used. The permissible tensile stress in plate is 150 mpa.*

Solution :





$$\text{Strength of weld} = p_q \times l \times t$$

$$\text{Strength of single - V butt joint} = \frac{150}{1000} \times 200 \times \left(\frac{5}{8} \times 16\right)$$

$$= \mathbf{300\ KH}$$

Strength of double - V butt joint

$$\text{Throat thickness } t = \text{thickness of thinner plate}$$

$$\because \qquad t = 16 \text{ mm}$$

$$\because \qquad \text{Strength of double - V butt joint} = \frac{150 \times 200 \times 16}{1000} = \mathbf{480\ KH}$$

Design of Plug and Slot Weld

Plug and slot welds are used in addition to fillet welds when sufficient welding length is not available along the edges of the member.

If a slot is small and completely filted with weld metal, it is known as plug weld, but if only the periphery of the slot is fillet welded then it is known as a slot weld.

Following specifications are used for the design of Slot or Plug welds :

1. Width or diameter of slot should not be less than three times the thickness of the part in which slot is formed or 25 mm, whichever is greater.
2. Corners at the enclosed ends should be rounded to a radius not less than 1½ times the thickness of upper plate or 12 mm, whichever is greater.
3. Distance between edges of the plates and slot or between edges of adjacent slots should not be less than twice the thickness of the upper plate.
4. Permissible stress is taken as 108 MPa or 1100 kgf/cm^2.
5. Plug welds are not designed to carry loads.

PLATE GIRDERS

These are used when standard rolled sections do not provide required section modulus. Building up a beam by riveting plates and angles to increase modulus of section is one of the choices available to the designer. Plate girders are specially adopted for large spans and heavy loads.

Componets in a Riveted Plate girder.

1. Web and Flange plates
2. Flange angles
3. Stiffners (intermediate stiffners, longitudinal stiffners)
4. Splices

Web and Flange Plates Design.

Two main components of the plate girders, namely, flanges and webs are designed to resist bending moment and shear force respectively.

Proportioning of Components

(*i*) **Depth of Girders.** Approximate depth of girder is usually taken as 1/8 to 1/2 of the effective span.

$$\text{Economical depth, D} = 1.1\sqrt{\frac{M}{\sigma_{bt}.t_w}}$$

where, M = moment of resistance

the actual depth is usually taken 10% less than the economical depth.

(*ii*) **Width of flange.** Width of flange would be within the range of $\frac{L}{40}$ to $\frac{L}{45}$.

(*iii*) **Deflection.** Maximum deflection should not exceed $\frac{1}{325}$ of the span.

Allowable Stresses.

Upto 20 mm web thickness :	Bending stress = 1575 kg/cm^2
	Shear stress = 945 kg/cm^2
Over 20 mm thick web :	Bending stress = 1500 kg/cm^2
	Shear stress = 865 kg/cm^2

Minimum thickness of Plates. For girders exposed to weather but accessible for painting, it is 6 mm

For girders exposed to weather but not accessible for painting, it is 8 mm

Analysis of Plate Girders

Plate girder is a built-up beam and moment of resistance can be found out by Flexure formula.

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

where, M = moment of resistance

I = moment of inertia

f = maximum allowable stress

y = extreme fibre distance

Moment of Resistance of Plate girders can be obtained by following methods.

(i) Flange Area method : This method is approximate but is sufficiently accurate for practical purposes.

$$\text{Moment of girder section, } M = F \times D \qquad ...(i)$$

where, F = resultant compressive and tensile force

$$\text{Average bending stress, } f_b = A \qquad ...(ii)$$

where, A = flange area

D = distance between lines of action of these forces

Combining equations (*i*) and (*ii*), we get $M = f_b$ AD

$$\text{Resisting moment of the flange, } M_F = A_g \times f_b \times D$$

where, A_g = gross area of one flange

D = distance between centroids of flanges.

Assuming that fibre stress at the edges of the web plate is f_b,

$$\text{Moment of resistance of web plate, } M_w = \text{Area} \times \text{Average Stress} \times \text{Lever arm}$$

$$= t_w \times \frac{h}{2} \times \frac{f_b}{2} \times \frac{2h}{3} = \frac{1}{6} f_b h^2 t_w$$

$$\text{Total moment of resistance, } M = M_F + M_w = f_b\, DA_g + \frac{1}{6} A_w$$

For tension flange, net area is considered and rivets 22 m diameter at 10 cm centres are used commonly.

$$\text{Net effective web equivalent} = \frac{3}{4} \times \frac{1}{6} A_w = \frac{1}{8} A_w$$

Thus economical areas for compression and tension flange were $\left(A_f + \frac{A_w}{6}\right), \left(A_f + \frac{A_w}{8}\right)$ respectively.

(ii) Moment of Inertia method.

1. **Specifications for Web Plate**

Minimum thickness of web plate for unstiffend webs = $\frac{d}{85}$

Vertically stiffened webs = $\frac{1}{180}$ of the smallest clear panel dimension

= $\frac{1}{200}$ for webs with longitudinal suffeners

Specification for Flange Plates

Maximum outstand for compression flange = $16t$

Maximum outstand for tension flange = $20t$

Flange angles shall be at least $\frac{1}{3}$ of the gross flange area.

2. **Curtailment of Flange Plates**

Since maximum bending moment occurs usually at centre, designed cross-section is only required at centre. Cutting out top plates at some points on the girder, as bending moment reduces from centre towards support, is called *curtailment of flange plates*.

This can be done by two following methods :

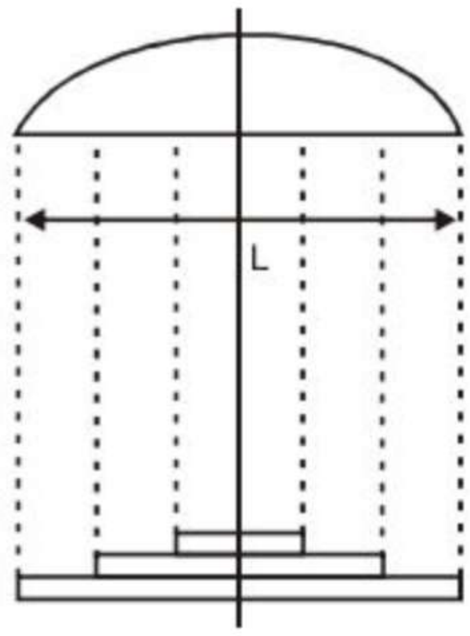
(*i*) *Graphical method* (applicable to any system of applied loads).

(*ii*) *Analytical method* (used, if plate girder supports only uniformly distributed loads)

3. Stiffeners

Intermediate stiffeners are required to prevent web plate from buckling under a complex and variable stress distribution resulting from combined shear and bending moment.

(*i*) Web Stiffeners (as per IS: 800).



(*a*) When $\frac{d_1}{t_w} \leq 85$; no stiffener is required.

(*b*) When $\frac{d_2}{t_w} \leq 200$; vertical stiffener are provided, no horizontal stiffener were required.

(*c*) When $\frac{d_2}{t_w} \leq 250$; vertical stiffeners and one horizontal stiffener are provided.

Spacing of stiffeners, $C \nless \frac{d}{3}$ and $\ngtr 1.5\,d$

Moment of inertia of the stiffener, $I_s \nless \frac{1.5 d^3 \cdot t^3}{C^2}$

where, d = distance between flange angles.

Width of the stiffener should not exceed 16 times the thickness of the stiffener.

(*ii*) Bearing Stiffeners.

Function of bearing stiffeners is to transmit concentrated load so as to avoid local bending failure of the flanges and local crippling or buckling of the web.

Bearing stiffeners together with the web plate shall be designed as a column with an equivalent reduced slenderness ratio.

$$\text{Effective length} = 0.7 \times \text{actual length}$$

(*iii*) Longitudinal Stiffeners.

In addition to the vertical intermediate stiffeners, longitudinal stiffeners should be provided if

$$t_w < \frac{d}{200}.$$

One horizontal stiffener should be placed on the web at a distance from the compression flange

= 2/5 of the distance from compression flange to the neutral axis.

Connections.

(*i*) Connection of Flange plate to Flange angles :

$$\text{Pith of rivets} = \frac{n \times R \times I}{F \cdot A\bar{y}}$$

where, R = rivet value

n = number of rivets in one pitch length

I = moment of inertia

$A\bar{y}$ = moment of the net area of flange plate about neutral axis

F = shear force at the section

(*ii*) Connection of Intermediate stiffeners to Web :

Intermediate vertical and horizontal stiffeners not subjected to external loads shall be connected to the web by rivets or welds, so as to withstand a shearing force, between each component of the stiffener and the web of not less than $\frac{125 t_w^2}{h}$ kN/m

where, t_w = web thickness in mm

h = outstand of stiffener in mm

(*iii*) Connection of Bearing stiffener to Web :

Connection should be designed to transmit the entire concentrated load to the web.

4. Splices

Splices are required for following reasons :

(*i*) Length of the plate girder may be large whereas plate lengths available are limited.

(*ii*) Large size plates are difficult to handle, erect and place as they may get twisted.

Type of Splices.

These are of following three types

(*i*) Web splice

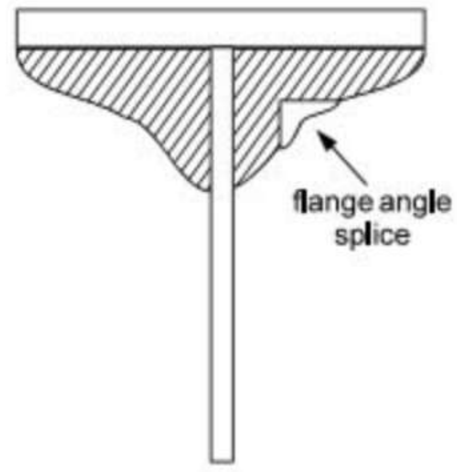
(*ii*) Flange plate splice

(*iii*) Flange angle splice.

Web Splice.

Since there is a limit to the length and weight of plates rolled, the webs of long span girders have preferred to be in two or more lengths. For riveted girders, splice consists of two plates, one on each side of the web.

Web splice should be designed for the full strength of web in both shear and bending.



Types of Plates used :

(*i*) Plates 'A' called *moment plates* to resist the web's share of the bending moment Size of moment plates A is decided on the basis of number of rivets to be accomodated.

(*ii*) Plates 'B' called *shear plates* to resist the shear are used in this type. Size of the shear plates B can be determined on the basis of shear area required.

Design of Plate 'A' :

Moment resisted by web = $\frac{1}{8} A_w f \times h$

Moment of resistance of plates = $F \times h_1$

Equating, we get $F = \frac{1}{8} \cdot \frac{A_w \cdot f h}{h_1}$

where, A_w = area of the web

f = allowable stress

h = distance between centroids of the flanges

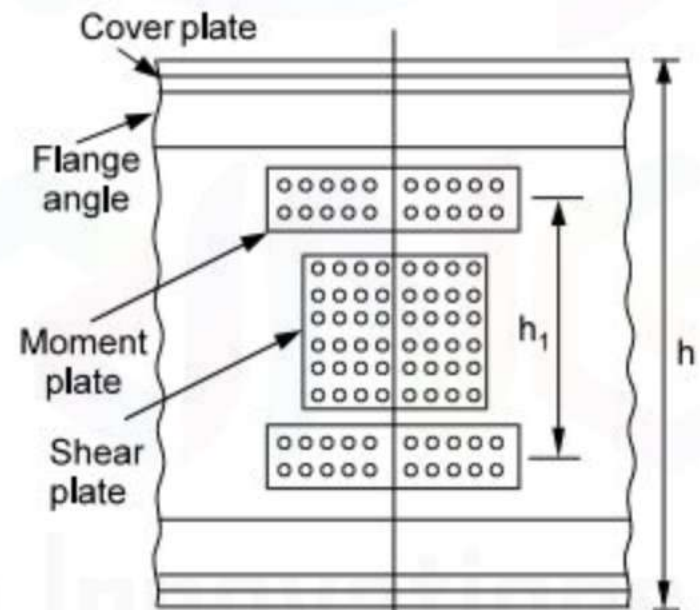
h_1 = distance between the centroids of outer plates

Area of the plates required, $A = \frac{F}{f_1}$

where, f_1 = allowable stress in bending for the plates

But area of one pair of plates= $2 \times d_1 \times t_1$

Assuming suitable thickness t_1, $d_1 = \frac{f_1}{2t_1}$



Total number of rivets required on each side of the splice to connect each pair of plates A = $\frac{f}{r_1}$

where, $r_1 = \frac{h_1}{h} R$

R = least rivet value

ROOF TRUSSES

Truss is a framework in which members are connected at their ends. Pitch of the roof truss is ratio of height of the truss to the span should be 1/4 to 1/6 for proper drainage.

Common spacing of trusses ranges from 3 to 5 m. Economical spacing varies from $\frac{1}{3}$ to $\frac{1}{5}$ of span. Roof truss usually require very light members.

Member of roof trusses are designed as axially loaded tension or compression member if they are slender and their resistance to bending is neglected.

Loads

(1) **Dead Loads.**

(*i*) **Sheeting :** GI sheeting–15 kg/m^2
AC sheeting–18 kg/m^2

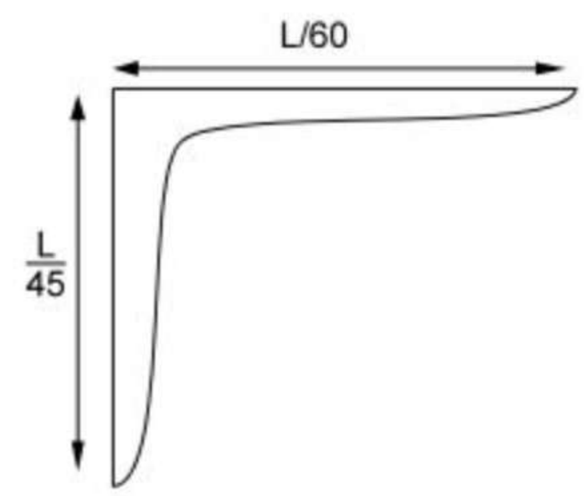
(*ii*) **Purlins :** Self-weight of purlins with corrugated sheets varies from 6 to 9 kg/m^2 area covered by purlin.

(*iii*) **Trusses :** Weight of truss $\left(\frac{L}{3}+5\right)$ kg/m^2,

where, L = span of truss in metres.

(2) **Live Loads.**

For sloping roofs of greater than 10º slope, live load is 75 kg/m^2 less 1 kg/m^2 for every degree increase in slope up to 20º.



(3) **Snow Loads.**

No snow load is considered if slope is greater than 50º.

For other slopes, 2.5 kg/m^2 per cm depth of snow may be taken.

(4) **Wind Loads.**

External wind pressure plus internal air pressure.

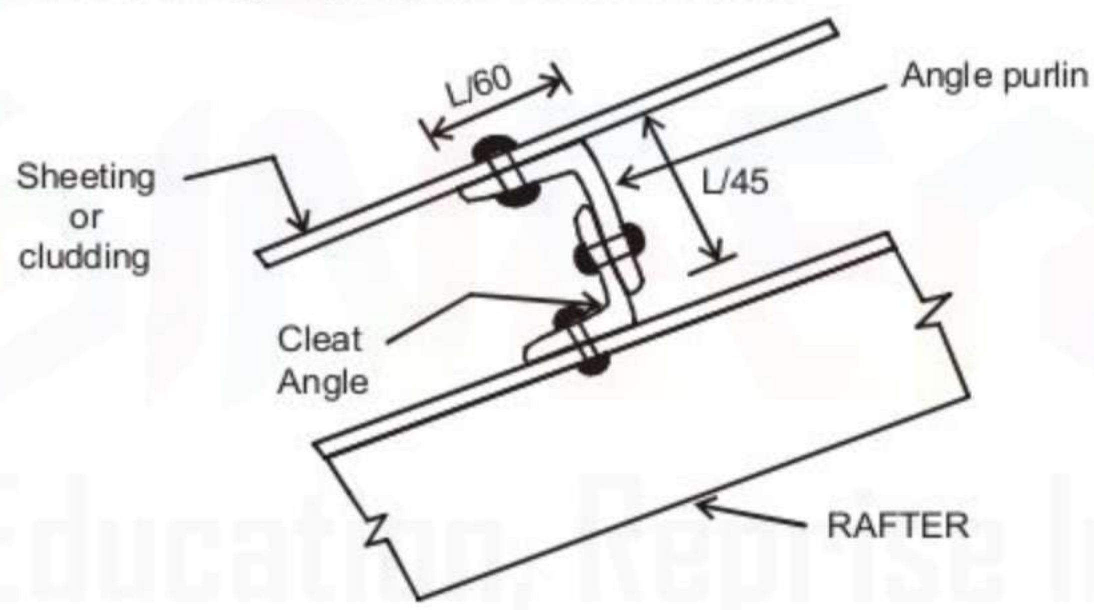
Design of Purlins.

Angle, channel I & Z section are used for purlins to support cladding. Its spacing varies from 1.0 to 2.25 m depending upon the type of cladding material, wind/snow/live load and the spacing of joints of the roof truss. Purlins are designed as beam subjected to bending moment about both *x* & *y* axes.

The deflection of purlins should not be more than the permissible deflection for the type of roof covering used.

Design of Purlins as Per IS 800-1984

It provides the design procedure for angle purlins when the roof slopes not exceeding 30° based on a minimum live load of 750 N/m^2 if the following requirements are fulfilled.



(*i*) width or depth of angle leg in the plane perpendicular to the roof covering $\geq \frac{L}{45}$

(*ii*) width or depth of angle leg in the plane parallel to roof covering $\geq \frac{L}{60}$

(*iii*) Maximum bending moment in the purlin

$$M = \frac{w \times L^2}{10} = \frac{W.L}{10}$$

$$W = wL$$

Where, w = uniformly distributed load per unit length on purlin including wind load.

L = Span of purlin.

The bending moment about minor axis may be neglected and the angle purlin may be designed for the above moment

$$Z_{x\,required} = \frac{M}{\sigma_{bc}}$$

σ_{bc} = permissible bending stress in compression = 0.66 f_y

PLASTIC ANALYSIS OF BEAMS AND PORTALS

Through ductility, structural steel is able to absorb large deformations beyond elastic limit without danger of the fracture. It is this characteristic feature of steel that makes possible application of plastic analysis to structural design.

Plastic Theory

Stress-strain diagram for mild steel in tension is shown in Fig. (*a*). Here '*ab*' represent elastic range; '*bc*' represent the range where strain increases without load (Plastic flow) '*d*' represents ultimate tensile strength; and '*e*' represent breaking load.

Assumptions

(***i***) Plastic range is entered on reaching the yield point;

(***ii***) Strain hardening is ignored;

(***iii***) Stress-strain relation for tension is the same as that for compression;

(***iv***) Plane sections remain plane; and

(***v***) The upper and lower points *b* and *b'* merge into one.

The stress-strain diagram shown in Fig. (*b*) is used in plastic theory upto the point of failure; the line will represent the plastic strain.

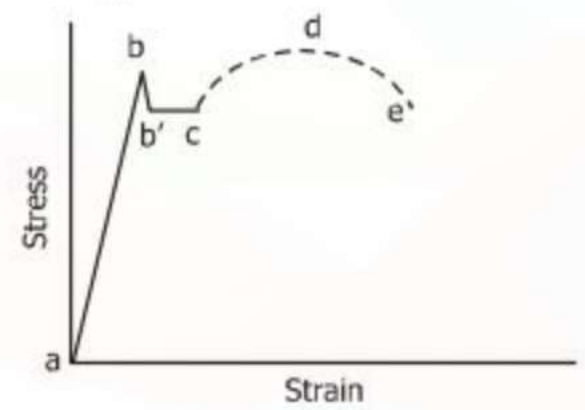


Fig (a) stress-strain diagram (for mild steel)

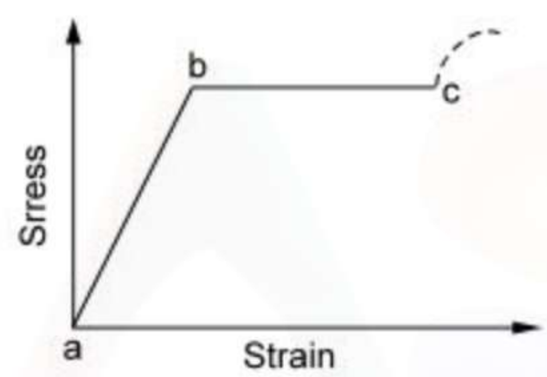


Fig. (b) Modified plastic strain diagram

Consider a simple beam under bending Fig.(*a*) subjected to a load W at mid-point. Fig. (*b*) represents stress distribution due to both self-weight (dead load) and live load W. As the load W increases, stress in the extreme fibers reaches the yield point and extreme fibres offer no further resistance, but inner fibres have not yet been stressed to the yield limit. In Fig. (*c*), some fibres are stressed to yield point and other still under-stressed, whereas in Fig. (*d*), all fibres are stressed to the yield level. It can be assumed, at this point, that the section has become '*fully plastic*'. Any further increase of load is assumed to increase the deflection substantially and '*fully plastic*' section may be treated as a *plastic hinge*. Deflection under the increased load will lead to collapse of the beam.

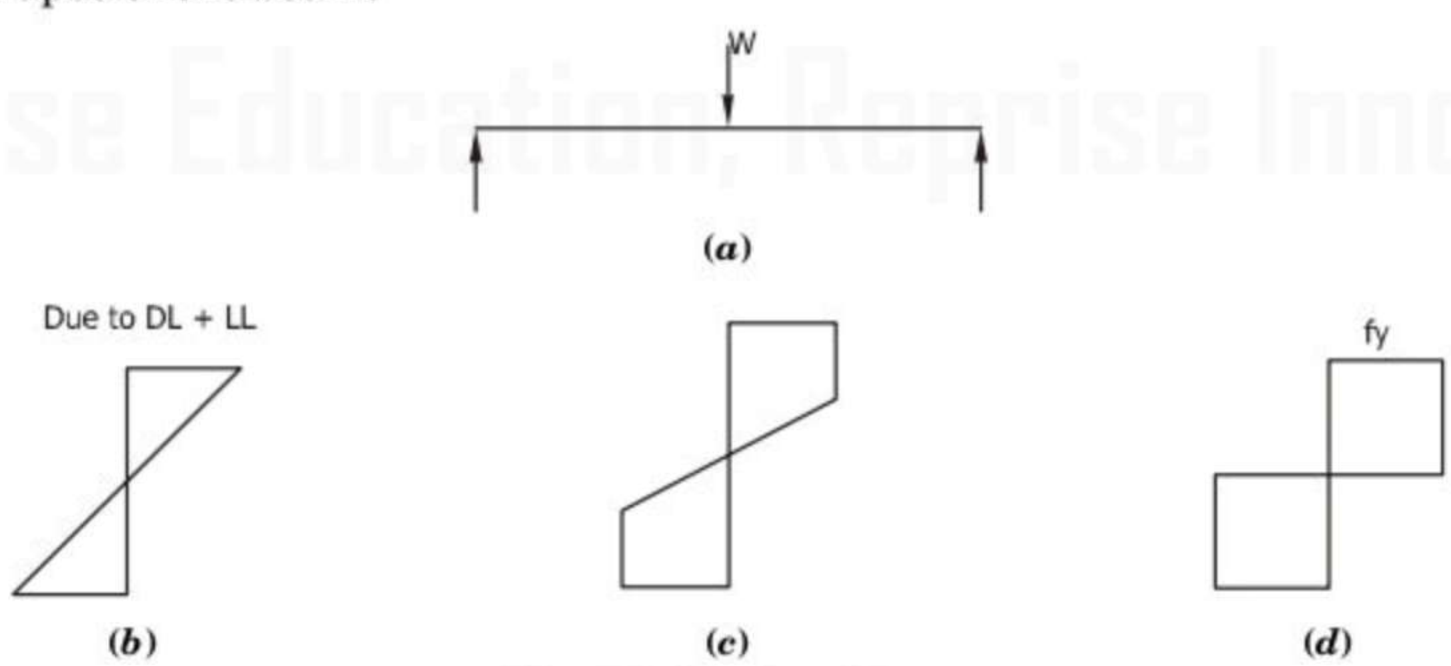


Fig. Distribution of stress

Load Factor (λ)

Ratio of the load producing collapse to the working load is called *load factor*. As working stress is dependent on the shape of the section *i.e.,* I and Z values, so also collapse load is dependent on the shape of the section.

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

Rectangular Beam

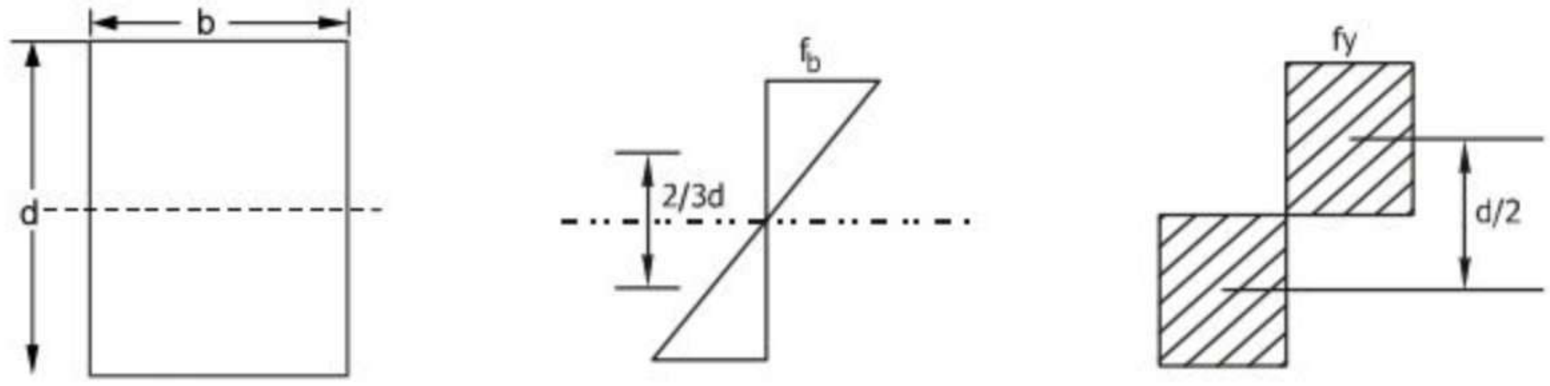


Fig. Load Factor

Moment of resistance under working load = $\dfrac{bd}{2} \times \dfrac{f_b}{2} \times \dfrac{2d}{3} = \dfrac{bd^2}{6} f_b$

For collapse load, Moment of Resistance = $\dfrac{bd}{2} \times f_y \times \dfrac{d}{2} = \dfrac{bd^2}{4} f_y$

$$\text{Load Factor} = \frac{f_y}{f_b} \times \frac{bd^2}{4} \Big/ \frac{bd^2}{6} = 1.5\frac{f_y}{f_b}$$

Assuming factor of safety as 1.5 in bending,

$$\text{Load Factor} = 1.5 \times 1.5 = 2.25$$

Shape factor or Form factor.

Fully plastic stage in the section is said to have occurred when tensile as well as compressive zones both have f_y at all points.

$$\text{Plastic moment, } M_p = \text{Force} \times \text{Lever arm}$$

$$= f_y \frac{bd^2}{4} = \frac{3}{2} f_y \cdot \frac{bd^2}{6} = 1.5\, M_y \qquad \left[Z_y = \frac{bd^2}{6}\right]$$

Thus, bending strength of rectangular member is given by M_p, which is 1.5 times its yield strength M_y. This ratio is called *shape factor* (f).

$\therefore$ Shape factor, $f = \dfrac{M_p}{M_y}$

Also, $f = \dfrac{M_p}{M_y} = \dfrac{f_y Z_p}{f_y Z_e} = \dfrac{Z_p}{Z_e}$

Shape factor is a property dependent upon geometry of the section only.

PLASTIC HINGE AND MECHANISM

Consider a simply supported beam of span L carrying a concentrated load 'W' at mid-point. The beam will fail when centre section becomes fully plastic. With simple supports at the ends and a plastic hinge at centre, the beam will transform into a mechanism consisting of two links.

Length of the plastic zone depends upon the ratio M_p to M_y. Greater the ratio, larger will be the length of the plastic zone. The section in this length will be at different stages of $M - \phi$ curve about the yield value.

When beam is carrying uniformly distributed load,

$$\text{length of plastic hinge} = L\sqrt{1 - \frac{1}{\alpha}}$$

where, α = shape factor

When beam is carrying concentrated load,

$$\text{Length of platic hinge} = L\left(1 - \frac{1}{\alpha}\right)$$

where, α = shape factor

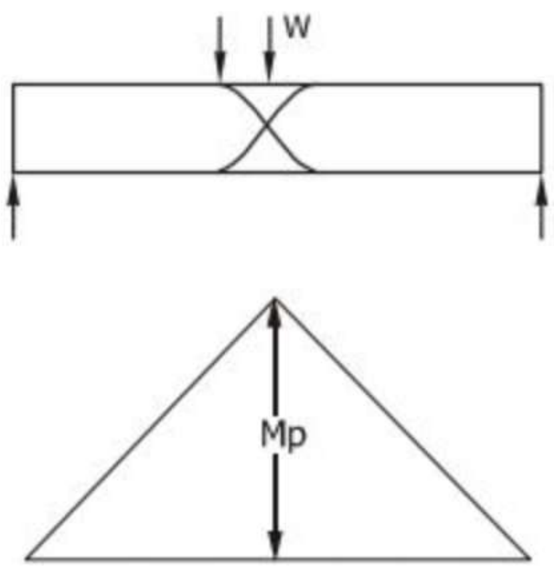


Fig (a). Plastic Hinge

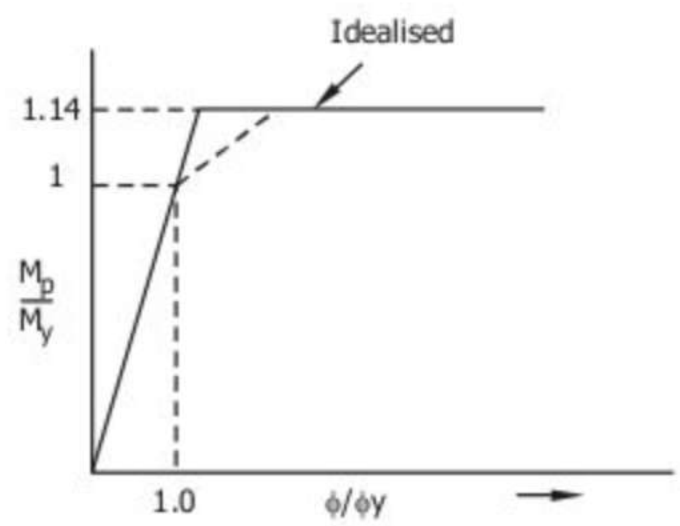
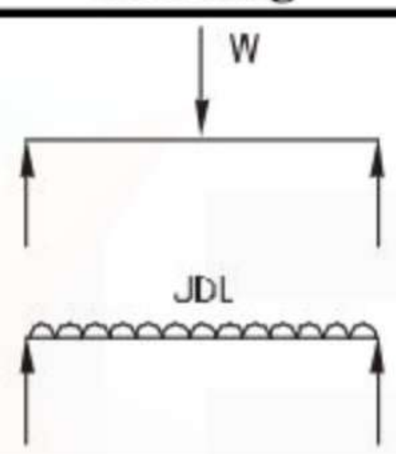
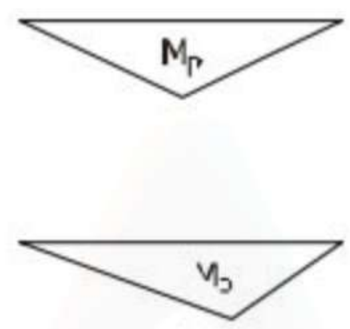
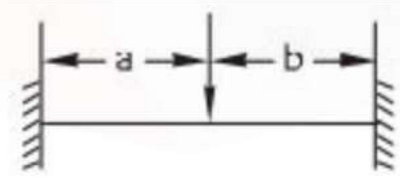
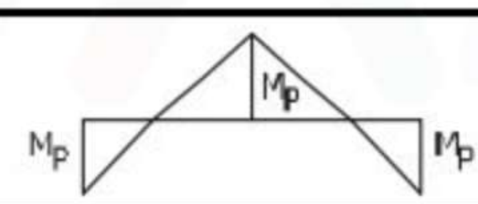
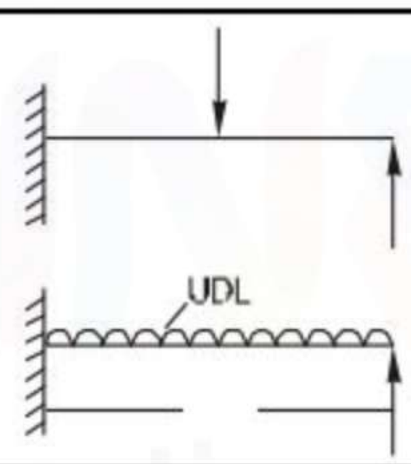
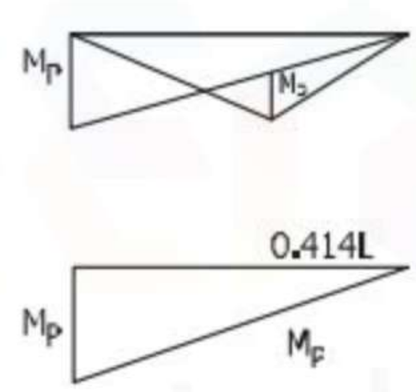
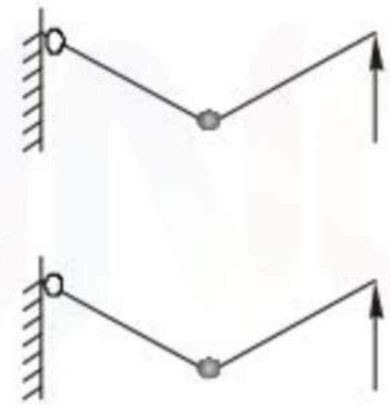


Fig (b). Idealised M-ϕ curve for I-section

To simplify plastic analysis, the M – ϕ curve may be idealised by two straight lines as shown in Fig. (*b*).

Types of Mechanisms.

Types	Loading	Bending moment	Plastic hinges
1. Simple beams			
2. Fixed beams			
3. Propped cantilever			

Collapse Mechanism

The insertion of a real hinge, or pin joint , into a statically indeterminate frame reduces number of indeterminate moments by one, so that, if number of indeterminates is m, then addition of n hinges produces a simple statically-determinate structure. The addition of one more hinge will allow the structure to move with one degree of freedom, *i.e.* a mechanism is formed; then number of hinges to form a mechanism is $(n + 1)$. This criterion must be applied to each element of a structure as well as the structure as a whole, because collapse of one part represents practical failure.

Number of independent mechanism, $n = N - r$

where, N = number of possible plastic hinges

r = number of redundancies

Typical collapse Mechanisms.

EXERCISE – I

MCQ TYPE QUESTIONS

1. The plastic section modulus for a rectangular section of width b and depth d is

(*a*) $\frac{bd^3}{3}$ (*b*) $\frac{bd^2}{6}$

(*c*) $\frac{bd^2}{4}$ (*d*) $\frac{bd^2}{12}$

2. Ratio of plastic section modulus to elastic section modulus for circular section is

(*a*) $\frac{14}{3\pi}$ (*b*) $\frac{16}{5\pi}$

(*c*) $\frac{16}{3\pi}$ (*d*) $\frac{9}{5\pi}$

3. Plastic modulus for a circular section of diameter *d* is

(*a*) $\frac{d^3}{6}$ (*b*) $\frac{d^3}{3}$

(*c*) $\frac{d^3}{4}$ (*d*) $\frac{d^3}{5}$

4. Shape factor is the property which depends

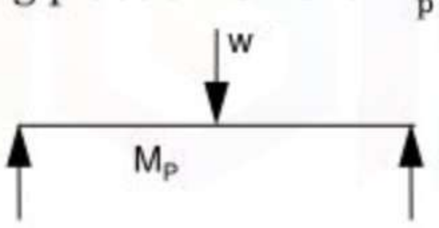
(*a*) ultimate stress of the material

(*b*) field stress of the material

(*c*) geometry of the section

(*d*) yield stress and ultimate stress of material

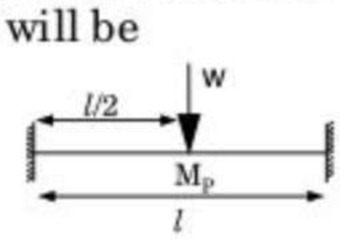
5. The value of collapse load of simply supported beam having plastic moment M_p is



(*a*) $\frac{6M_p}{L}$ (*b*) $\frac{4M_p}{L}$

(*c*) $\frac{8M_p}{L}$ (*d*) $\frac{5M_p}{L}$

6. The value of collapse load of the beam as shown in the figure will be



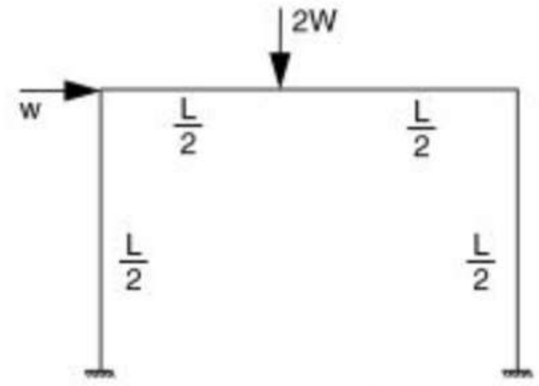
(*a*) $\frac{4M_p}{L}$

(*b*) $\frac{5M_p}{L}$

(*c*) $\frac{6M_p}{L}$

(*d*) $\frac{8M_p}{L}$

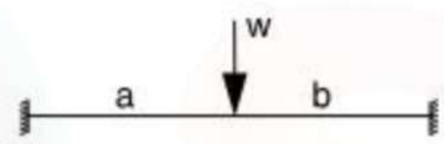
7. If M_p is plastic moment capacity of the cross section of the portal frame shown in the figure, value of load W at collapse in that frame will be



(*a*) $\frac{6M_p}{L}$ (*b*) $\frac{M_p}{L}$

(*c*) $\frac{4M_p}{L}$ (*d*) $\frac{2M_p}{L}$

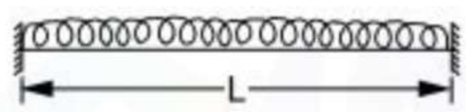
8. If plastic moment capacity of the beam is M_p as shown in the figure, then value of collapse load W will be



(*a*) $\frac{2M_pL}{a-b}$ (*b*) $\frac{2M_pL}{ab}$

(*c*) $\frac{3M_pL}{ab}$ (*d*) $\frac{4M_pL}{ab}$

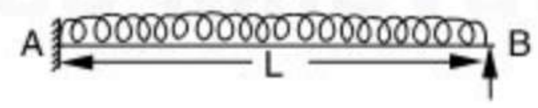
9. Collapse load of beam as shown in the figure will be



(*a*) $\frac{4M_p}{L}$ (*b*) $\frac{6M_p}{L}$

(*c*) $\frac{8M_p}{L}$ (*d*) $\frac{16M_p}{L}$

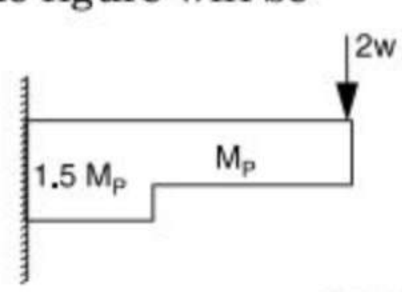
10. In proped cantilever loaded as shown is the figure, plastic hinge will form



(*a*) at B (*b*) at $\frac{L}{2}$ from B

(*c*) at 0.414 L from B (*d*) at 0.414 L from A

11. The load at collapse for the cantilever beam as shown in the figure will be



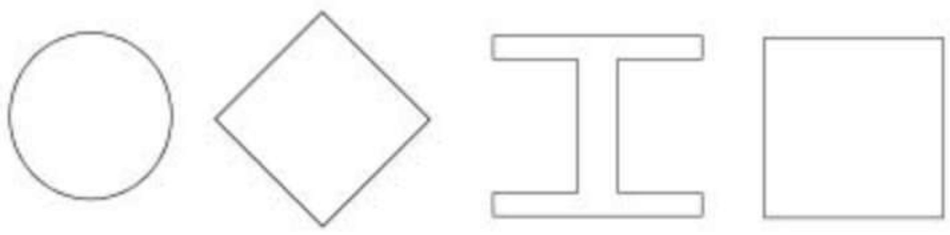
(*a*) $\frac{0.5\ M_p}{L}$ (*b*) $\frac{0.75\ M_p}{L}$

(*c*) $\frac{0.6\ M_p}{L}$ (*d*) $\frac{0.45\ M_p}{L}$

12. At fully plastic section, infinite rotation can occur at

(*a*) zero moment
(*b*) constant elastic moment
(*c*) constant plastic moment
(*d*) all of these

13. The four cross-sections shown below are required to be ordered in the increasing order of their respective shape factors.

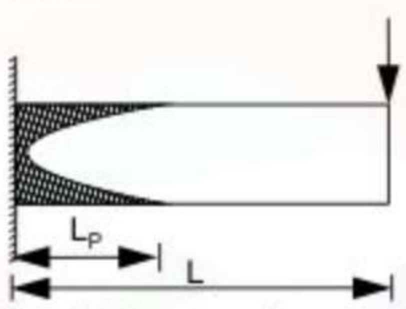


I II III IV

Which of the following order is correct ?

(*a*) III, I, IV, II
(*b*) I, II, III, IV
(*c*) III, IV, I, II
(*d*) III, IV, II, I

14. A cantilever beam of length L and a cross-section with shape factor 'f' supports a concentrated load P as shown below.



The length L_p of the plastic zone, when maximum bending moment equals the plastic moment M_p, given by

(*a*) $\dfrac{L_p}{L} = \dfrac{1}{f}$ (*b*) $\dfrac{L_p}{L} = L(1-f)$

(*c*) $\dfrac{L_p}{L} = 1 - \dfrac{1}{\sqrt{f}}$ (*d*) $\dfrac{L_p}{L} = 1 - \dfrac{1}{f}$

15. Effect of axial force and shear force on the plastic moment capacity of a section are

(*a*) to decrease and to increase the plastic moment respectively
(*b*) to increase and to decrease the plastic moment respectively
(*c*) to increase plastic moment capacity in both cases
(*d*) to decrease plastic moment capacity in both cases

16. The statical method of plastic analysis satisfies

(*a*) equilibrium and mechanism conditions
(*b*) equilibrium and plastic moment conditions
(*c*) mechanism and plastic moment conditions
(*d*) equilibrium condition only

17. The mechanism method of plastic analysis satisfies

(*a*) equilibrium and mechanism conditions
(*b*) equilibrium and plastic moment conditions
(*c*) mechanism and plastic moment conditions
(*d*) equilibrium condition only

18. Which of the following conditions is to be satisfied both in elastic and plastic analysis ?

(*a*) Equilibrium condition
(*b*) Yield condition
(*c*) Plastic moment condition
(*d*) Mechanism condition

19. A simply supported beam of rectangular section and span L is subjected to a uniformly distributed load at the centre. The length of elastoplastic zone of the plastic hinge will be

(*a*) L/3 (*b*) $L/\sqrt{3}$ (*c*) L/2 (*d*) L/8

20. The plastic design method is an advantageous replacement over elastic design method for the structures stressed primarily in bending in case of

(*a*) statically loaded structures
(*b*) indeterminate structures
(*c*) both (*a*) and (*b*)
(*d*) none of these

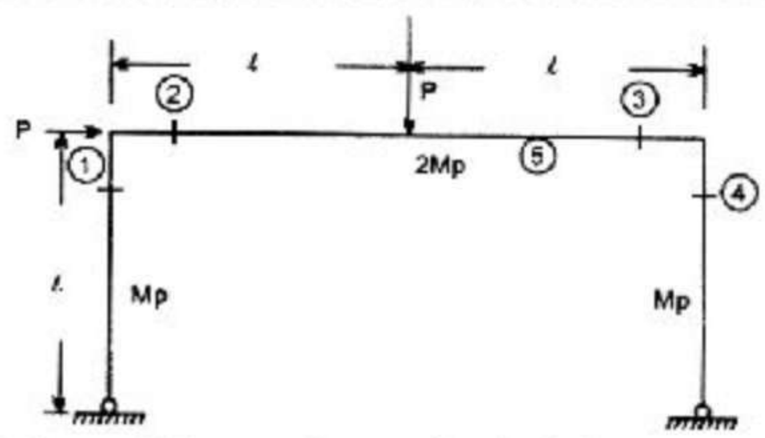
21. As per IS:800, in the plastic design, which of the following pairs are correctly matched ?

Working Loads	Load factor
(*a*) Dead load	1.7
(*b*) Dead load + load due to wind or seismic forces	1.3
(*c*) Dead load + imposed load + load due to wind or seismic forces	1.7
(*d*) none of these	

22. In case of plastic design, calculated maximum shear capacity of a beam as per IS:800 shall be

(*a*) $0.55\,A_w f_y$ (*b*) $0.65\,A_w f_y$
(*c*) $0.75\,A_w f_y$ (*d*) $0.85\,A_w f_y$

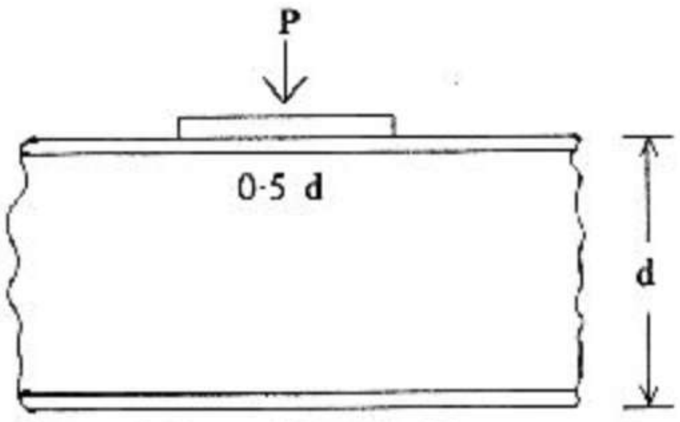
23. A portal frame subjected to central concentrated load and horizontal load is shown in the figure



Likely positions where plastic hinges can form in combined mechanism would include

(*a*) 2, 3 and 5 (*b*) 1, 4 and 5
(*c*) 4 and 5 (*d*) 3 and 5

24.



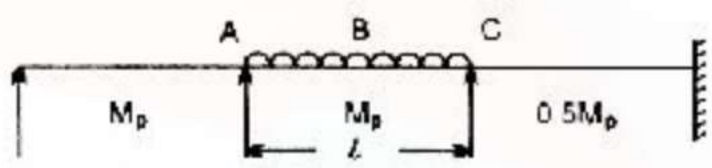
A plate girder of depth d bears a concentrated load P through a distribution plate of width 0.5d as shown in the figure. If the maximum allowable critical buckling stress is calculated as f, what is the value of P ? Web plate thickness = t

(*a*) 0.5 dt f (*b*) 1.0 dt f
(*c*) 1.5 dt f (*d*) 2.0 dt f

25. A continuous beam of constant M_p has three equal spans and carries total uniformly distributed load 'w' on each span. The value of collapse load for the beam will be

(*a*) $12M_p/L$ (*b*) $11.656\, M_p/L$
(*c*) $8.65\, M_p/L$ (*d*) $4\, M_p/L$

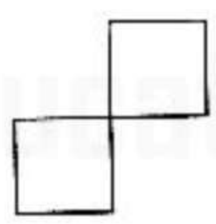
26. A continuous beam with plastic moment capacities is shown in the figure

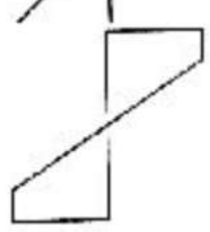
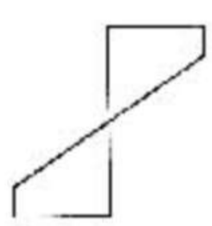


The correct sequence in which the plastic hinges will form in the beam is

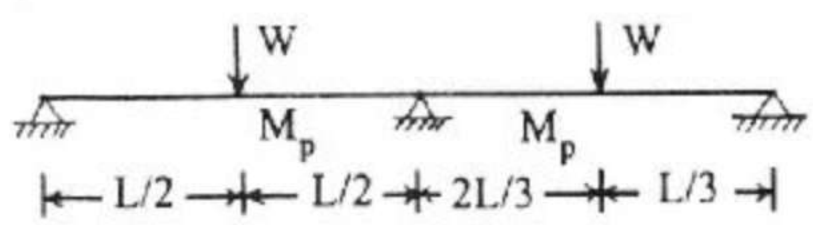
(*a*) C, A, B
(*b*) A and C simultaneously, followed by B
(*c*) B, C, A
(*d*) B first, then A and C simultaneously

27. In a plastic hinge, the actual distribution of strain across the section will be as in

(*a*)  (*b*) 

(*c*)  (*d*) 

28. A continuous beam with constant EI is shown in the given figure. Collapse load for this beam will be equal to



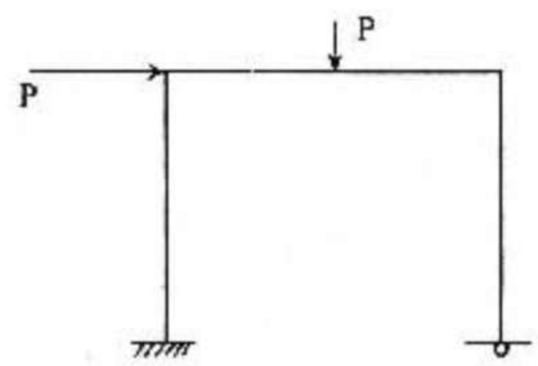
(*a*) $\frac{16M_P}{L}$ (*b*) $\frac{12M_P}{L}$

(*c*) $\frac{8M_P}{L}$ (*d*) $\frac{6M_P}{L}$

29. Which of the following statements is/are correct?

(*a*) Plastic hinges are reached first at sections subjected to greatest curvature.
(*b*) Formation of plastic hinges allows a subsequent redistribution of moment until fully plastic moment is reached at each critical section.
(*c*) The maximum load is attained when a mechanism forms
(*d*) both (*b*) and (*c*)

30. The number of possible independent mechanisms for a portal frame shown in the given figure is



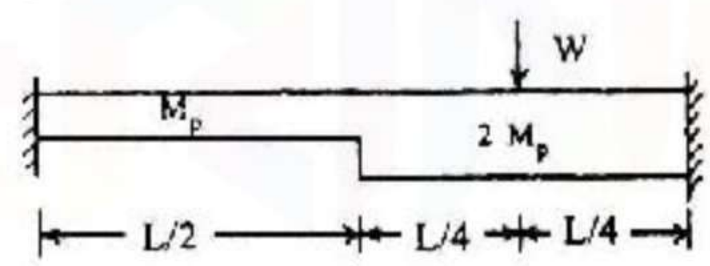
(*a*) 2 (*b*) 4 (*c*) 1 (*d*) 30

31. A propped cantilever bean of span 'L' and constant plastic moment capacity M_p carries a concentrated load at mid-span, then load at collapse will be

(*a*) $\frac{8M_P}{L}$ (*b*) $\frac{6M_P}{L}$

(*c*) $\frac{4M_P}{L}$ (*d*) $\frac{2M_P}{L}$

32. Figure given below shows a fixed beam of steel

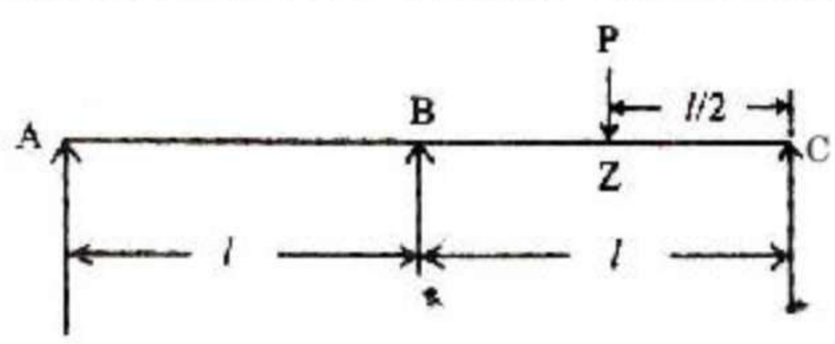


At the point of collapse, the value of load W will be

(*a*) $\frac{10M_P}{L}$ (*b*) $\frac{15M_P}{L}$

(*c*) $\frac{20M_P}{L}$ (*d*) $\frac{25M_P}{L}$

33. A continuous beam ABC of two equal spans AB and BC carries a load P at Z, the centre of BC. Then the magnitude of collapse load P is equal to



(*a*) $\frac{2M_P}{l}$ (*b*) $\frac{4M_P}{l}$

(*c*) $\frac{6M_P}{l}$ (*d*) $\frac{8M_P}{l}$

34. The shape factor for a solid circular section of diameter D is equal to

(*a*) $\frac{D}{2\pi}$ (*b*) $\frac{15}{2\pi}$

(*c*) $\frac{16}{2\pi}$ (*d*) $\frac{\pi D}{8}$

35. At the location of the plastic hinge of a deformed structure

(*a*) curvature is infinite

(*b*) radius of curvature is infinite

(*c*) moment is infinite

(*d*) flexural stress is infinite

36. Which of the following conditions are to be satisfied by an ideal plastic material ?

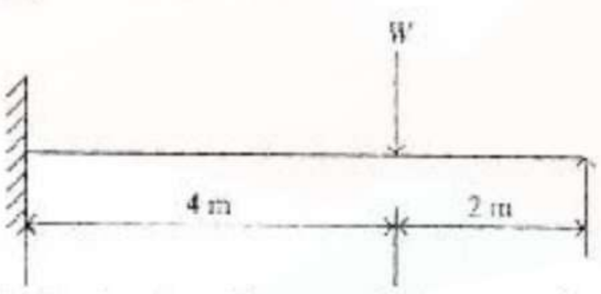
(*a*) Strain up to the strain hardening in tension and compression are to be the same.

(*b*) The material property should be different in tension and compression.

(*c*) The values of yield stress in tension and compression should be different.

(*d*) All of these

37.

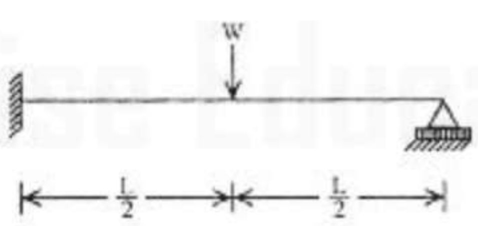


What is the collapse load for propped cantilever beam shown in the above diagram with a plastic moment capacity of M_p ?

(*a*) 1.25 M_p (*b*) 1.5 M_p

(*c*) M_p (*d*) 2 M_p

38. For the propped cantilever beam shown in the figure given above, the plastic moment capacity is M_p. What is the value of its collapse load ?



(*a*) $4M_p/L$ (*b*) $6M_p/L$

(*c*) $8M_p/L$ (*d*) $11.7M_p/L$

39. In a plastic analysis of structures, the segment between any two successive plastic hinges is assumed to deform as a/an

(*a*) plastic material (*b*) rigid material

(*c*) elastic material (*d*) inelastic material

40. Which one of the following is not correct ?

(*a*) The shape factor for any section is equal to Mp/My

(*b*) For a thin-web wide-flange beam, shape factor is close to unity

(*c*) For a circular section, shape factor is nearly 1.7

(*d*) Shape factor for an I-section sometimes may be more than that for a rectangular section

41. Fillet weld is not recommended if the angle between fusion faces is

(*i*) less than 45°

(*ii*) greater than 120°

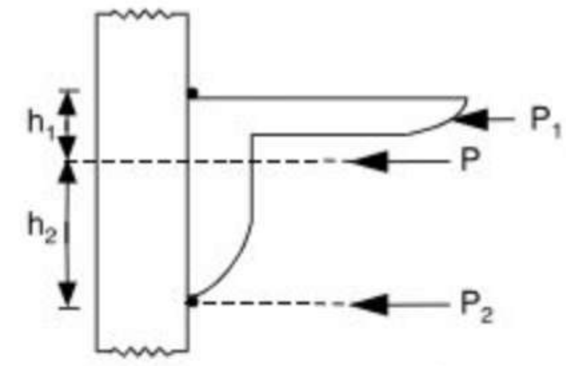
(*iii*) less than 60°

(*iv*) greater than 145°

(*a*) (*i*) and (*ii*) (*b*) (*i*) and (*iv*)

(*c*) (*ii*) and (*iii*) (*d*) (*iii*) and (*iv*)

42. If P is the force acting on centroid of section, the ratio of two forces P_1 and P_2 acting on two sides of fillet weld as shown in the figure, will be



(*a*) $\frac{h_2}{h_1}$ (*b*) $\frac{2h_2}{h_1}$

(*c*) $\frac{h_1}{h_2}$ (*d*) $\frac{2h_1}{h_2}$

43. For design of compression member,

(*a*) Slenderness ratio should be minimum

(*b*) Radius of gyration should be large

(*c*) both (*a*) and (*b*)

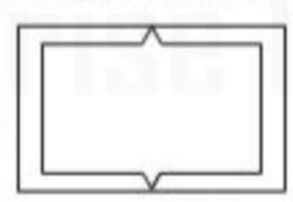
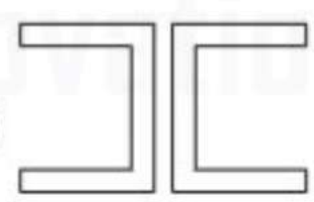
(*d*) none of these

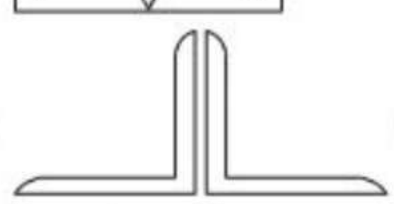
44. A column has effective length l, when both ends are fixed. What will be the new effective length if one end become hinged ?

(*a*) l (*b*) $0.5\,l$

(*c*) $1.41\,l$ (*d*) $2\,l$

45. Which of the following section is most efficient column section ?

(*a*)  (*b*) 

(*c*)  (*d*) 

46. Angle of inclination of the lacing bar with the longitudinal axis of the column should preferably be between

(*a*) 10° to 30° (*b*) 30° to 80°

(*c*) 40° to 70° (*d*) 20° to 70°

47. In design of compression member

(*a*) Lacing are not used with batten.

(*b*) Batten are acted by longitudinal shear and moment both.

(*c*) Lacing is than batten.

(*d*) All of these

48. If column has machined end, then connection between splice and column is designed to carry

(*a*) 25% axial load and any tension

(*b*) 50% axial load and any tension

(*c*) only tension

(*d*) 25% axial load and 75% tension

49. A column section ISHB 350 @ 724 N/m of 250 mm width carries an axial load 3500 kN. If square base plate having 0.90 m sides is used, then what will be the minimum thickness of base plate ?

(*a*) 85 mm (*b*) 70.5 mm

(*c*) 94.37 mm (*d*) 78 mm

50. A column base is subjected to moment. If intensity of bearing pressure due to axial load is equal to stress due to the moment, then bearing pressure between base and the concrete is

(*a*) uniform compression throughout

(*b*) zero at one end and compression at other end

(*c*) tension at one end and compression at other end

(*d*) uniform tension throughout

51. A grillage base is checked for

(*a*) bending (*b*) shear

(*c*) web crippling (*d*) all of these

52. A column carrying a load P and rests on two tier grillage foundation. Then maximum shear force is

(*a*) $\frac{P}{2}\left(\frac{L-a}{L}\right)$ (*b*) $\frac{P}{L-a}\left(\frac{L}{2}\right)$

(*c*) $2P \times \left(\frac{L}{L-a}\right)$ (*d*) $\frac{2}{P}\left(\frac{L-a}{L}\right)$

where, a and L are length of base plate and upper tier beam respectively.

53. The most critical consideration in the design of rolled steel columns carrying axial load is the

(*a*) percent elongation at yield and the net cross-sectional area

(*b*) critical bending strength and axial yield strength of the material

(*c*) buckling strength based on the net area of the section and percent elongation at ultimate load

(*d*) compressive strength based on slenderness ratio and gross cross-sectional area of the member

54. Which one of the following four shapes for a column of same effective height formed with two equal angles, has the highest axial compression load carrying capacity?

(*a*)  (*b*) 

(*c*)  (*d*) 

55. The design of eccentrically loaded column needs revision when

(*a*) $\frac{f_c'}{f_c} + \frac{f_b'}{f_b} < 1$ (*b*) $\frac{f_c'}{f_c} - \frac{f_b'}{f_b} < 1$

(*c*) $\frac{f_c'}{f_c} - \frac{f_b'}{f_b} > 1$ (*d*) $\frac{f_c'}{f_c} + \frac{f_b'}{f_b} > 1$

where,

f_c = calculated average axial compressive stress

f_c = allowable axial compressive stress

f_b' = calculated bonding stress in the extreme fibre

f_b' = allowable bending compressive stress on the extreme fibre

56. Buckling load in a steel column is

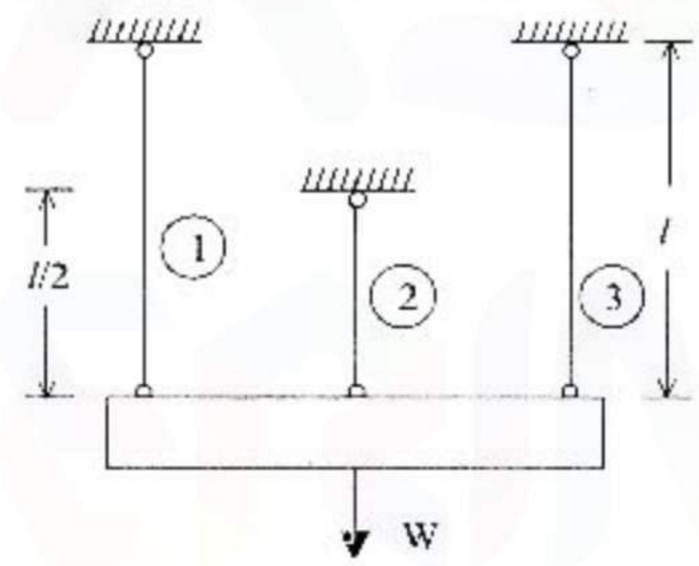
(*a*) related to length

(*b*) directly proportional to the slenderness ratio

(*c*) inversely proportional to the slenderness ratio

(*d*) non linearity of the slenderness ratio

57.



Three wires of steel 1, 2 and 3, each having area 'A' support a load W. What is the ratio between collapse load and the load corresponding to yielding of one of the wires ?

(*a*) 3 : 1 (*b*) 3 : 2

(*c*) 3 : 2.5 (*d*) 3 : 3

58. Lug angles

(*a*) are used to reduce the length of connection

(*b*) are unequal angles

(*c*) increases shear legs

(*d*) all of these

59. Lug angles are used to

(*a*) increase lengths of the end connections angle section

(*b*) increase lengths of the end connections of channel section

(*c*) decrease lengths of the end connections of channel section

(*d*) all of these

60. Lug angles
(a) are necessarily unequal angles
(b) increase the shear resistance of joint
(c) reduce the length of joint
(d) both (b) and (c)

61. A single angle tie of a welded steel truss in an industrial shed is required to designed for an axial tension of 50 kN. If permissible tensile stress is 150 MPa, then most suitable section satisfying IS 800 codal requirements will be
(a) ISA 75 60 6 (b) ISA 60 40 5
(c) ISA 50 30 4 (d) ISA 45 30 5

62. Lug angles are designed
(a) to develop a strength not less than 20% in excess of the force in the outstanding leg of an angle
(b) the attachment of the lug angle to the angle member should be capable of developing 40% in excess of that force
(c) both (a) and (b)
(d) none of these

63. If a tension member subjected to axial load and bending moment, then
(a) $\frac{\sigma_{ad,cal}}{0.66 f_y} + \frac{\sigma_{bt,cal}}{0.6 f_y} \le 1$
(b) $\frac{\sigma_{at,cal}}{0.6 f_y} + \frac{\sigma_{bd,cal}}{0.66 f_y} \le 1$
(c) $\frac{\sigma_{at,cal}}{0.6 f_y} + \frac{\sigma_{bd,cal}}{0.66 f_y} \le 1.4$
(d) $\frac{\sigma_{at,cal}}{0.66 f_y} + \frac{\sigma_{bd,cal}}{0.6 f_y} \le 1.4$

64. Splice covers and its connection in a tension member should be designed
(a) to develop net tensile strength of main member
(b) to carry 50% load of main member
(c) in tension member splices are not recommended
(d) to carry $33\frac{1}{3}\%$ load of main member

65. Most efficient and economical section used as a beam is
(a) I section (b) circular section
(c) angles (d) H section

66. When a beam is subjected to lateral load, then designer choice for section should be
(a) angle
(b) H
(c) I section with channel section at tap flange
(d) I section

67. Bearing stiffeners are provided at the
(a) support
(b) point of application of concentrated load
(c) both (a) and (b)
(d) none of these

68. In case of buckling, the dispersion of load from bearing plate to neutral axis takes place at
(a) 30° (b) 60°
(c) 45° (d) 10°

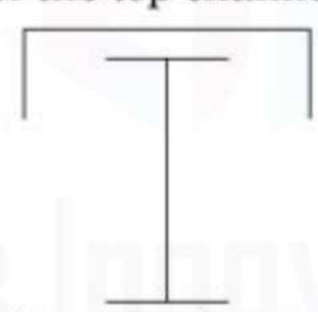
69. In case of web crippling, the dispersion of load from bearing plate takes place at
(a) 30° (b) 60°
(c) 45° (d) 10°

70. Shear determines the design of beams, which
(a) depth of beam section is small and is loaded uniformly
(b) large concentrated loads are placed near beam supports
(c) both (a) and (b)
(d) none of these

71. Maximum deflection in steel beam is limited to
(a) $\frac{L}{360}$ (b) $\frac{L}{325}$
(c) $\frac{L}{250}$ (d) $\frac{L}{100}$

72. Maximum bending moment in a purlin of length 'L' when subjected to a distributed load 'w' is assumed
(a) $\frac{wL^2}{6}$ (b) $\frac{wL^2}{8}$
(c) $\frac{wL^2}{12}$ (d) $\frac{wL^2}{10}$

73. Given figure shows a typical section of a crane girder. consider the following statements in this regard. The function of the top channel is to increase



(a) moment of inertia about vertical axis
(b) torsional stiffness
(c) lateral buckling strength
(d) all of these

74. An angle section can be used as purlin when slope of the roof truss is
(a) between 40° and 70°
(b) less than 30°
(c) greater than 30°
(d) less than 45°

75. Apart from gravity loads which of the following loads are also considered in the design of a gantry girder located within an industrial building ?
(a) Longitudinal load (b) Lateral load
(c) both (a) and (b) (d) none of these

76. A simply supported beam of rectangular section and span L is subjected to a concentrated load at the mid span. The length of elastoplastic zone of the plastic hinge will be

(*a*) $\frac{L}{5}$ (*b*) $\frac{L}{7}$

(*c*) $\frac{L}{4}$ (*d*) $\frac{L}{3}$

77. A rectangular steel section of width '*b*' and depth '*h*' has been stressed upto yield point (σ_y) up to a depth of $\frac{h}{4}$ from both the top and bottom faces under the action of moment 'M'. The magnitude of the moment M is

(*a*) $\frac{10}{24}bh^2\,\sigma_y$ (*b*) $\frac{1}{4}bh^2\,\sigma_y$

(*c*) $\frac{11}{48}bh^2\,\sigma_y$ (*d*) $\frac{13}{36}bh^2\,\sigma_y$

78. The advantage of pin connections is/are

(*a*) moment at pin connection is zero

(*b*) Only one pin is used in a connection

(*c*) Secondary stresses do not occur

(*d*) all of these

79. A cantilever steel beam of 3 m span carried a unformly distributed load of 20 kn/m inclusive of self weight. The beam comprises ISLB 200 @ 198 N/m, flange 100 mm × 7.3 mm, web thickness 5.4 mm, I_{xx} = 1696.6 mm^4, I_{yy} = 115.4 cm^4. Bending and shear stresses in the beam are respectively

(*a*) 530.47 N/mm^2 and 55.55 N/mm^2

(*b*) 3899.48 N/mm^2 and 82.19 N/mm^2

(*c*) 132.62 N/mm^2 and 27.78 N/mm^2

(*d*) 1949.74 N/mm^2 and 41.10 N/mm^2

80. Critical stress in an axially compressed plate of width b and thickness 't' is

(*a*) directly proportional to b/t

(*b*) inversely proportional to b/t

(*c*) directly proportional to $(b/t)^2$

(*d*) inversely proportional to $(b/t)^2$

81. For unstiffened flange of a beam in flexural compression, maximum allowable outstand is equal to _______ *t*

(*a*) 20 *t* (*b*) 16 *t*

(*c*) 32 *t* (*d*) 14 *t*

82. A welded steel plate girder consisting of two flange plates of 350 mm × 16 mm and a web plate of 1000 mm × 6 mm requires

(*a*) no stiffners

(*b*) vertical stiffners

(*c*) intermediate vertical stiffners

(*d*) vertical and Horizontal stiffners

83. The rolled steel section used in a cased beam has width 'B' mm and diameter 'D' mm. The minimum width of the finished cased beam is given by

(*a*) (B + 50) mm (*b*) (B + 100) mm

(*c*) (B/D + 100) mm (*d*) 2 (B + D) mm

84. As per IS-800, in case of a plate girder with vertical and horizontal stiffners, the greater and lesser unsupported clear dimension of a web pannel in terms of web thickness 't_w' should not exceed respectively

(*a*) 180 t_w and 85 t_w (*b*) 270 t_w and 200 t_w

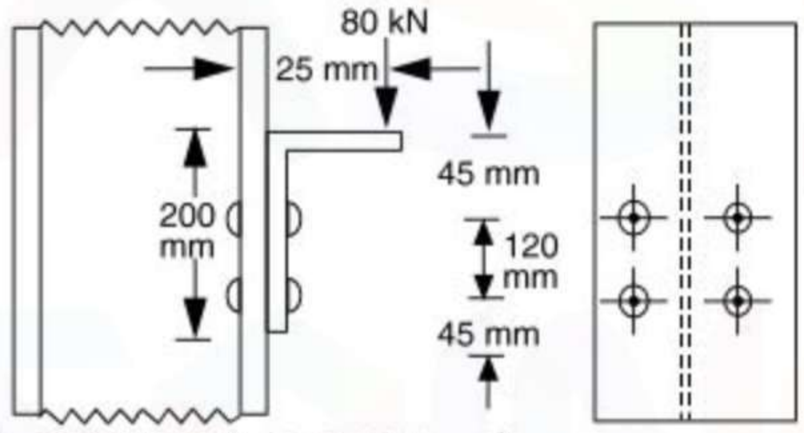
(*c*) 270 t_w and 180 t_w (*d*) 400 t_w and 250 t_w

85. Effective flange area in tension of a plate girder is equal to

(*a*) A_f (*b*) $A_f + \frac{A_w}{2}$

(*c*) $A_f + \frac{A_w}{8}$ (*d*) $A_f + \frac{A_w}{6}$

86. The induced shear stress and induced tensile stress on the the extreme rivet of given eccentric riveted connection to support a load of 80 kN at an eccentricity of 25 mm from the column face will be respectively diameter at rivet 18 mm.



(*a*) 67 N/mm^2, 27.9 N/mm^2

(*b*) 27.9 N/mm^2, 67 N/mm^2

(*c*) 27.9 N/mm^2, 60 N/mm^2

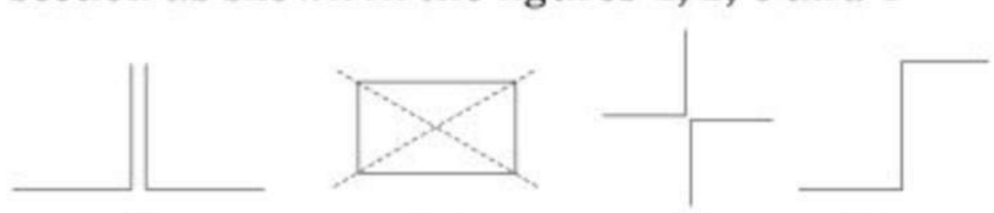
(*d*) 60 N/mm^2, 27 N/mm^2

87. The tension member of a roof truss consists of two unequal angles 70 × 45 × 8 with the longer legs connected by 16 mm diameter rivets. If angles are one on either side of the gusset plate, then safe tension for the member will be (assuming, permissible stress in tension = 150 MPa & sectional area of angle 858 mm^2)

(*a*) 205 kN (*b*) 215.4 kN

(*c*) 310.4 kN (*d*) 320 kN

88. Two equal angles form a compound column cross-section as shown in the figures 1, 2, 3 and 4



(1) (2) (3) (4)

Among these, those which have the some axial compression, load carrying capacity would include

(*a*) 1 and 2 (*b*) 1 and 3

(*c*) 2 and 3 (*d*) 3 and 4

89. The problem of lateral buckling can arise only in those steel beams which has

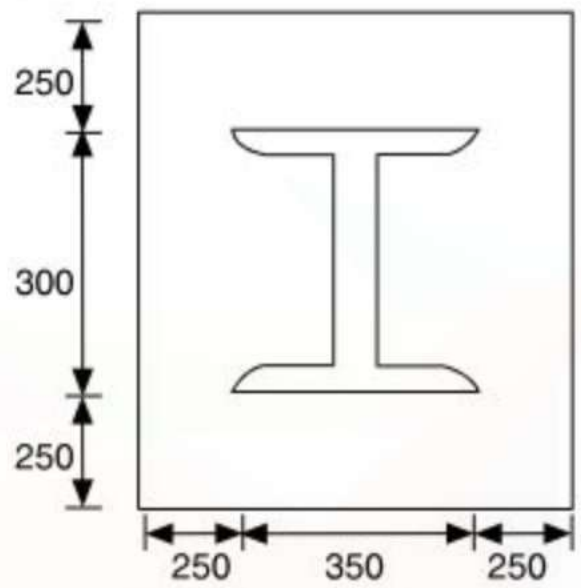
(*a*) moment of inertia about the bending axis larger than the other

(*b*) moment of inertia about the bending axis smaller than the other

(*c*) fully supported compression flange

(*d*) none of these

90. What is the thickness of slab base plate carrying an axial load of 2500 kN as shown in the given figure, safe bearing capacity of soil is 250 kN/m^2 and permissible bearing pressure on concrete is 4000 kN/m^2 ?



(*a*) 2.1 cm (*b*) 4 cm

(*c*) 5.6 cm (*d*) 7.8 cm

91. Which one of the following pair is not correctly matched ?

(*a*) If $\frac{d_1}{t_w} \le$ lesser of $\frac{816}{\sqrt{\tau_{val,cal}}}$ and $\frac{1344}{\sqrt{f_y}}$ and 85

$\rightarrow$ No stiffners are required

(*b*) If $\frac{d_2}{d_w} \le$ lesser of $\frac{3200}{\sqrt{f_y}}$ and 200

$\rightarrow$ Vertical stiffners are provided

(*c*) If $\frac{d_2}{t_w} \le$ lesser of $\frac{4000}{\sqrt{f_y}}$ and 250

$\rightarrow$ Vertical stiffner and one horizontal stiffner at $\frac{2}{5}$ distance from compression flange to neutral axis is provided

(*d*) If $\frac{d_2}{t_w} \le$ less of $\frac{6400}{\sqrt{f_y}}$ and 400

$\rightarrow$ Two vertical stiffners are provided

92. For a standard 45º fillet, the ratio of size of fillet to throat thickness is

(*a*) 1 : 1 (*b*) 1 : $\sqrt{2}$

(*c*) $\sqrt{2}$: 1 (*d*) 2 : 1

93. Which of the following methods of design would be suitable for metal structures subjected to stress reversals and impact ?

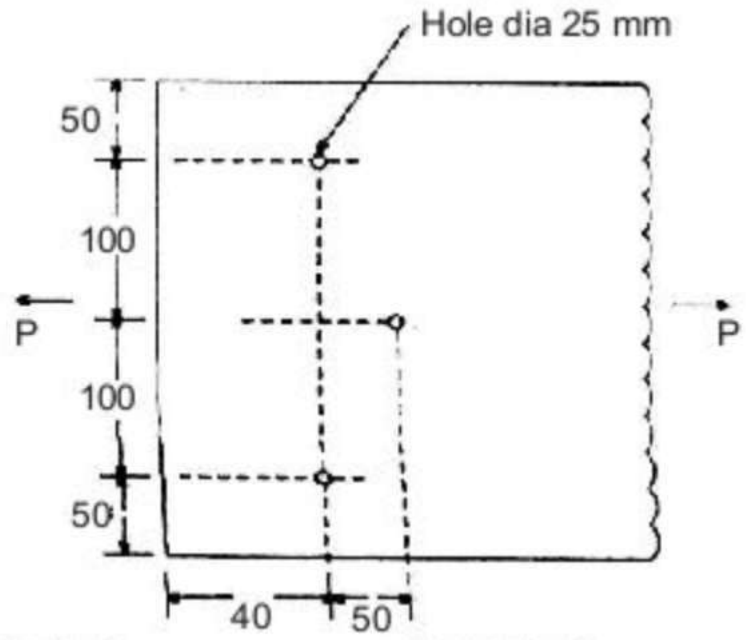
(*a*) Simple working stress design

(*b*) Semi-rigid design

(*c*) Elastic rigid design

(*d*) all of these

94. What is the effective net width of plate shown in the figure for carrying tension ?



(*a*) 212.5 mm (*b*) 237.5 mm

(*c*) 250 mm (*d*) 275 mm

95. High yield deformed bars have a

(*a*) definite yield value.

(*b*) chemical composition different from mild steel.

(*c*) percentage elongation less than that of mild steel.

(*d*) percentage elongation more than that of mild steel.

96. According to IS specifications, maximum pitch of rivets in compression is

(*a*) lesser of 200 mm and 12 t

(*b*) lesser of 200 mm and 16 t

(*c*) lesser of 300 mm and 32 t

(*d*) lesser of 300 mm and 24 t

97. The best arrangement to provide united behaviour in built up steel columns is by

(*a*) lacing

(*b*) battening

(*c*) tie plates

(*d*) perforated cover plates

98. Angle of inclination of the lacing bar with the longitudinal axis of the column should preferably be between

(*a*) 10° to 30° (*b*) 30° to 40°

(*c*) 40° to 70° (*d*) 90°

99. Battening is preferable when

(*a*) column carries axial load only

(*b*) space between the two main components is not very large

(*c*) both (*a*) and (*b*)

(*d*) none of these

100. The bracing between two columns of a steel tank will be designed to resist

(a) horizontal shear due to wind or earthquake only

(b) horizontal shear due to wind or earthquake +2.5% of column loads

(c) column loads +2.5% of horizontal shear dueto wind or earthquake

(d) column loads + full horizontal shear due to wind or earthquake

101. For a compression member with double angle section, which of the following section will give larger value of minimum radius of gyration ?

(a) equal angles back to back

(b) unqual legged angles with long legs back to back

(c) unequal legged angles with short legs back to back

(d) both (b) or (c)

102. Battens provided for a compression member shall be designed to carry a transverse shear equal to

(a) 2.5% of axial force in member

(b) 5% of axial force in member

(c) 10% of axial force in member

(d) 20% of axial force in member

103. Conventional practice is to brace end panels of the side walls of an industrial building. Instead bracing can be provided in the bays near centre of the building. Which one of the following reasons is correct ?

(a) Wind pressure at the mid length is higher compared to ends

(b) Trusses are erected starting from the ends of the building

(c) Fixing the bracings to end gables is convenient

(d) Free change of length between centre and the ends of the building is possible in mid-span bracing

104. A steel beam supporting loads from the floor slab as well as from wall is called

(a) stringer beam (b) lintel beam

(c) spandrel beam (d) header beam

105. Allowable shear stress in the web of mild steel beams decreases with

(a) decrease in h/t ratio

(b) increase in h/t ratio

(c) decrease in thickness

(d) increase in height

where 'h' is height and t is thickness

106. Ratio of the allowable stress in bending compression to that of bending tension in steel beams at

(a) 1 (b) ≤ 1

(c) ≥ 1 (d) greater than 1

107. For a certain longitudinal span, a beam in an industrial building requires section modulus of 423.00 cm^3. Which of the following sets of sections would be most suitable for this purpose?

(a) ISWB 250 @ 40.9 kg/m,
$Z_{xx} = 475.4\ cm^3$, $Z_{yy} = 85.7\ cm^3$

(b) ISLB 300 @ 37.7 kg/m,
$Z_{xx} = 488.9\ cm^3$, $Z_{yy} = 50.2\ cm^3$

(c) ISLB 325 @ 43.1 kg/m,
$Z_{xx} = 607.7\ cm^3$, $Z_{yy} = 61.9\ cm^3$

(d) ISWB 600 @ 145.1 kg/m,
$Z_{xx} = 3854.2\ cm^3$, $Z_{yy} = 423.9\ cm^3$

108. Allowable shear stress in stiffened webs of mild steel beams decreases with

(a) decrease in the spacing of the stiffeners

(b) increase in the spacing of the stiffeners

(c) decrease in the effective depth

(d) increase in the effective depth

109. In the design of framed connections, rivets or bolts connecting the web of the beam with the connecting angles are subject to

(a) single shearing and beaing on the web

(b) double shearing and bearing on the web

(c) double shearing and no bearing on the web

(d) no shearing but only bearing on the web

110. Economical depth of a plate girder corresponds to

(a) minimum weight

(b) minimum depth

(c) maximum weight

(d) minimum thickness of web

111. Web crippling due to excessive bearing stress can be avoided by

(a) increasing web thickness

(b) providing suitable stiffeners

(c) increasing length of the bearing plates

(d) none of these

112. As per IS : 800, for compression flange, the outstand of flange plates should not exceed

(a) 12 t (b) 16 t

(c) 20 t (d) 25 t

113. Intermediate vertical stiffeners in a plate girder need be provided if depth of web exceeds

(a) 50 t (b) 85 t

(c) 200 t (d) 250 t

where t is thickness of web

114. Forces acting on the web splice of a plate girder are

(a) axial forces

(b) shear and axial forces

(c) shear and bending forces

(d) axial and bending forces

115. Rivets connecting flange angles to cover plates in a plate girder are subjected to
(*a*) horizontal shear only
(*b*) vertical load only
(*c*) both (*a*) and (*b*)
(*d*) none of these

116. Intermediate vertical stiffeners are provided in plate girders to
(*a*) eliminate web buckling
(*b*) eliminate local buckling
(*c*) transfer concentrated loads
(*d*) prevent excessive deflection

117. The thickness of web for unstiffened plate girder with clear distance d between the flanges shall not be less than
(*a*) d/200 (*b*) d/85
(*c*) d/100 (*d*) d/160

118. At a section along the span of a welded plate girder, where web is spliced, the bending moment is M. If girder has top flange, web and bottom flange plates of equal area, than share of the bending moment which would be taken by the splice plates would be
(*a*) M (*b*) M/3
(*c*) M/7 (*d*) M/13

119. Generally purlins are placed at the panel points so as to avoid
(*a*) axial force in rafter
(*b*) shear force in rafter
(*c*) deflection of rafter
(*d*) bending moment in rafter

120. The risk co-efficient k_1 depends on
(*a*) mean probable design life of structures
(*b*) basic wind speed
(*c*) both (*a*) and (*b*)
(*d*) none of these

121. The external wind pressure acting on a roof depends on
(*a*) degree of permeability of rood
(*b*) slope of roof
(*c*) both (*a*) and (*b*)
(*d*) none of these

122. As per IS : 875, for the purposes of specifying basic wind velocity, the country has been divided into
(*a*) 4 zones (*b*) 5 zones
(*c*) 6 zones (*d*) 7 zones

123. The portal bracing in a truss bridge is used to
(*a*) transfer load from top of end posts to bearings
(*b*) keep the rectangular shape of the bridge cross-section
(*c*) stiffen the structure laterally
(*d*) prevent the sidesway buckling of top chord

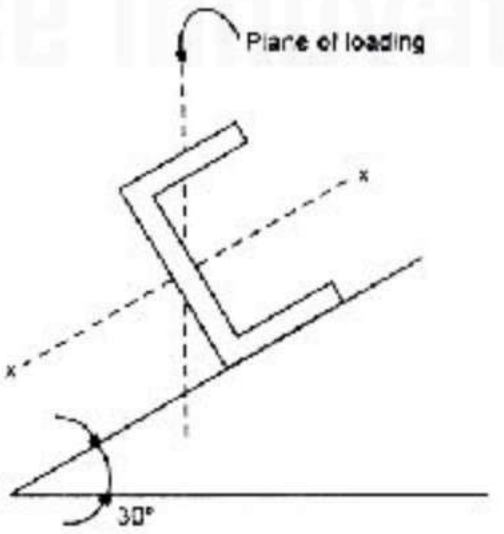
124. The sway bracing is designed to transfer
(*a*) 2½ % of the top panel wind load to bottom bracing
(*b*) 10% of the top panel wind load to bottom bracing
(*c*) 25% of the top panel wind load to bottom bracing
(*d*) 50% of the top panel wind load to bottom bracing

125. The channels or angles in the compression chords of the steel truss girder bridges are turned outward in order to increase
(*a*) cross-sectional area
(*b*) section modulus
(*c*) torsional constant
(*d*) radius of gyration

126. Z-purlins may be fixed in either orientation, A or B as shown in the figure.

Which of the following statements is correct in this regard ?
(*a*) Orientation A is structurally more efficient than orientation B
(*b*) Orientation B is structrually more efficient than orientation A
(*c*) Both orientations are structurally equally efficient
(*d*) Neither of the two orientations is structurally efficient

127. A channel section is placed in an inclined position carrying vertical loads as shown in the figure



If applied moment for the channel is 'M' due to vertical load, then M_{xx} is given by

(*a*) $\left(\frac{\sqrt{3}}{2}\right)M$ (*b*) 1/2 M

(*c*) $\left(\frac{1}{\sqrt{2}}\right)M$ (*d*) 2 M

128. In the simplified design of angle iron purlins, which of the following assumptions would not be valid?

(*a*) Load component acting normal to the slope is considered.

(*b*) Bending moment about the minor axis is considered.

(*c*) Allowable bending stress is not reduced.

(*d*) Slope of the roof should not exceed 30°.

129. Eleastic strain for steel is about

(*a*) 1/12 of strain at the initiation of strain hard ening and about 1/120 of maximum strain

(*b*) 1/2 of strain at the initiation of strain hardening and about 1/12 of maximum strain

(*c*) 1/12 of strain at the initiation of strain hardening and 1/200 of maximum strain

(*d*) 1/24 of strain at the initiation of strain hardening and about 1/200 of maximum strain

130. Mechanism method and the statical method give

(*a*) lower and upper bounds respectively on the strength of structure

(*b*) upper and lower bound respectively on the strength of structure

(*c*) lower bound on the strength of structure

(*d*) upper bound on the strength of structure

131. If a welded plate girder has a web plate 1500 × 6 mm, then

(*a*) web is left unstiffened

(*b*) web is provided with vertical stiffeners only

(*c*) web is provided with vertical stiffeners and a horizontal stiffener at 0.4 d from the top flange

(*d*) web is provided with vertical stiffener and two rows of horizontal stiffeners, one at 0.4 d from the top flange and another at the mid-height of the web

132. A laterally unsupported compression flange of beam has been strengthened by channel, angle and plates as shown. Areas of all the members added are equal. Which one of the following options will yield higher allowable stress ?

(*a*) 

(*b*) 

(*c*) 

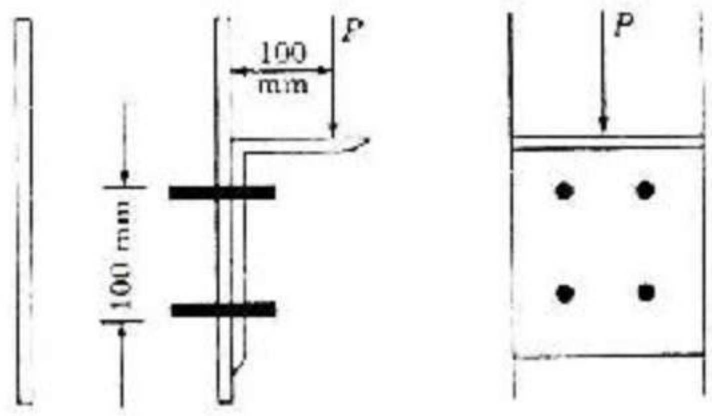
(*d*) 

133. An equal angle of area A has been attached to the support by means of a lug angle. If allowable stress in tension is f, what is the load carrying capacity of the member?

(*a*) 0.5 fA (*b*) 0.85 fA

(*c*) 0.9 fA (*d*) 1.0 fA

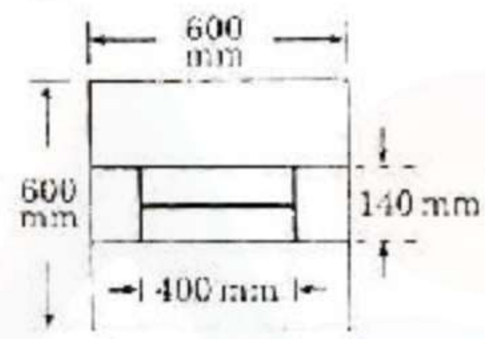
134. Each bolt shown in the given figure is capable of resisting a shear force of 20 kN and tension of 15 kN. The interaction equation between the forces is

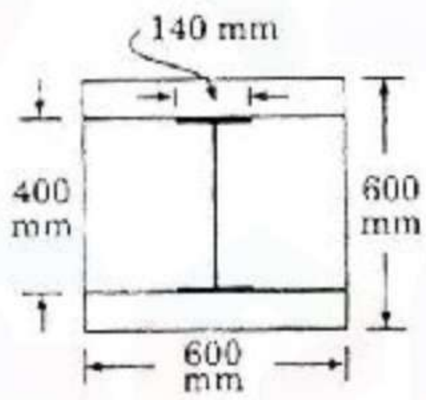


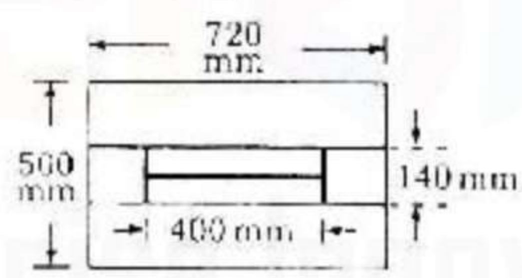
(*a*) $\frac{P}{40}+\frac{P}{30}=1.4$ (*b*) $\frac{P}{80}+\frac{P}{15}=1.4$

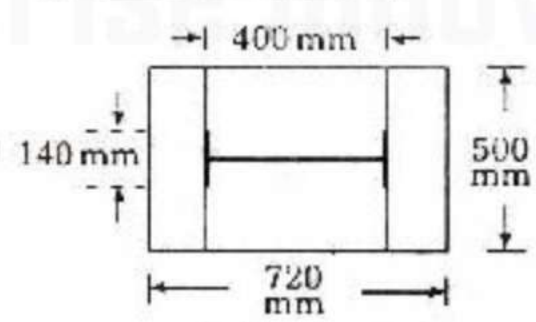
(*c*) $\frac{P}{80}+\frac{P}{30}=1.4$ (*d*) $\frac{P}{40}+\frac{P}{15}=1.4$

135. Which one of the following plan views of a gussetted base plate will result in minimum base plate thickness ?

(*a*) 

(*b*) 

(*c*) 

(*d*) 

136. In an eccentrically loaded bearing type connection, P_{dx} and P_{dy} are components of the direct load. P_{tx} and P_{ty} are components of the ten side load. The maximum load on any connector is given by

(*a*) $\sqrt{(P_{dx}+P_{tx})^2+(P_{dy}+P_{ty})^2}$

(*b*) $\sqrt{(P_{dx}+P_{ty})^2+(P_{dy}+P_{tx})^2}$

(*c*) $\sqrt{(P_{dx}-P_{tx})^2+(P_{dy}-P_{ty})^2}$

(*d*) $\sqrt{(P_{dx}-P_{ty})^2+(P_{dy}-P_{tx})^2}$

137. Economic spacing of a roof truss depends upon the
- (*a*) cost of purlins and cost of roof covering
- (*b*) cost of roof covering and dead loads
- (*c*) dead loads and live loads
- (*d*) live loads and cost of purlins

138. Bearing stiffeness are provided in a plate girder to
- (*a*) to prevent buckling of web
- (*b*) to strengthen the web
- (*c*) both (*a*) and (*b*)
- (*d*) none of these

139. As per IS: 800 in the case of a plate girder with vertical and horizontal stiffeners, the greater and lesser unsupported clear dimension of a web panel in terms of web thickness 't_w' should not exceed respectively
- (*a*) 180 t_w and 85 t_w (*b*) 270 t_w and 200t_w
- (*c*) 270 t_w and 180 t_w (*d*) 400 t_w and 250 t_w

140. Which one of the following is the load factor ?
- (*a*) $\dfrac{\text{Live load}}{\text{Dead load}}$ (*b*) $\dfrac{\text{Failure load}}{\text{Working load}}$
- (*c*) $\dfrac{\text{Total load}}{\text{Dead load}}$ (*d*) $\dfrac{\text{Dynamic load}}{\text{Dead load}}$

141. Which one of the following is the mode of failure in a fillet weld material ?
- (*a*) Tension (*b*) Shear
- (*c*) Bearing (*d*) Crushing

142. Load on connection is not eccentric for
- (*a*) LAP joint
- (*b*) Single cover butt joint
- (*c*) Double cover butt joint
- (*d*) all of these

143. In a plate girder bridge, thickness of web is less than d'/200, where d' is unsupported depth of web. The web plate should be provided with
- (*a*) vertical stiffness
- (*b*) horizontal stiffness
- (*c*) end stiffness
- (*d*) both vertical and horizontal-stiffness

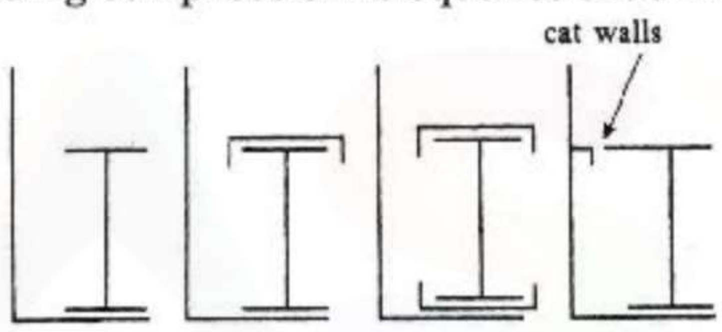
144. Which of the following apply to 'fully rigid design' of steel structures ?
- (*a*) Angle between members at the joint does not change.
- (*b*) structure is assumed to be pin-connected.
- (*c*) A reduction in the maximum bending moment is permitted to provide a degree of direction fixity.
- (*d*) All of these

145. The elements that are normally subjected to combined bending and axial forces are
- (*a*) struts in reinforced concrete members
- (*b*) the members in long span bridges
- (*c*) columns in framed structures
- (*d*) space truss members

146. Which among the following assumptions are made in the design of roof trusses ?
- (*a*) Roof truss is restrained by the reactions
- (*b*) Riveted joints act as friction less hinges
- (*c*) Loads act normal to roof surface
- (*d*) Both (*b*) and (*a*)

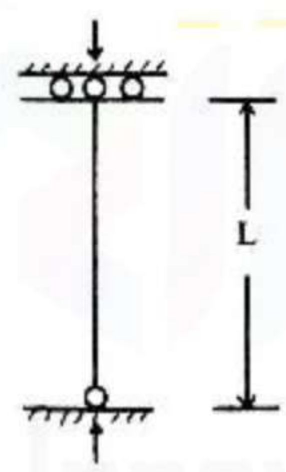
147. For a portal truss column fixed at the base, the point of contraflexure is assumed at
- (*a*) a distance mid-way between base and the foot of the knee brace
- (*b*) a distance mid-way between base and top of the column
- (*c*) the foot of the knee brace
- (*d*) quarter distance between base and top of the column

148. A gantry girder has been provided with the following sections shown in the following figures. In which cases, the allowable stresses in bending compression is equal to that in tension ?



(*a*) (*b*) (*c*) (*d*)

149. Effective length of the member shown in the figure is equal to



- (*a*) 1.2 L (*b*) 1.5 L
- (*c*) 2.0 L (*d*) 3.0 LL

150. In a compression member, plate element may buckle locally before the entire member fails. To avoid this, which of the following recommendations are made ?
- (*a*) Thickness of members is taken in terms of lengths of compression members
- (*b*) Length of element is increased
- (*c*) Length of member is increased
- (*d*) All of these

151. Design of a simple element in steel uses
- (*a*) full area of cross-section.
- (*b*) buckling criterion.
- (*c*) crushing (or yielding) criterion.
- (*d*) All of these

152. The type of stress induced in the foundation bolts fixing a column to its footing is

(*a*) pure compression (*b*) bearing

(*c*) pure tension (*d*) bending

153. Which of the following does not describe a weld type?

(*a*) Butt (*b*) Plug

(*c*) Zig-Zag (*d*) Lap

154. A plate used for connecting two or more structural members intersecting each other is

(*a*) Template (*b*) Base plate

(*c*) Gusset plate (*d*) Shoe plate

155. In the context of ultimate load theory for steel, stress-strain curve for steel is idealized as

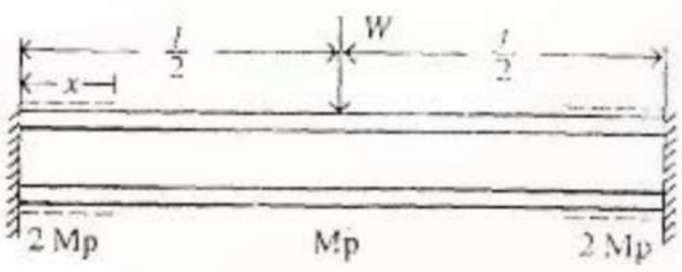
(*a*) a single straight line

(*b*) Bi-linear

(*c*) a quadratic parabola

(*d*) a circular arc

156.



For a fixed beam shown above, it has been decided to weld coverplates at ends so that moment capacity doubles at the ends. If maximum advantage has to be derived, then length x of the plate should be

(*a*) $l/2$ (*b*) $l/3$

(*c*) $l/4$ (*d*) $l/6$

157. For a compresion member having same effective length about any cross-secitonal axis, the most suitable section from the point of view of strength is a/an

(*a*) box (*b*) I-section

(*c*) circular tube (*d*) single angle

158. An angle is connected to the back of the flange of a channel section to be used as a beam as shown in the diagram.This is done to

(*a*) increase the compression flange area

(*b*) increase the moment of inertia about the major axis

(*c*) increase the moment of inertia about the minor axis

(*d*) make the load pass through the shear centre

159. Some steels do not show yield plateau and show continuous curve. For such steels, is the yield strength obtained by drawing

(*a*) 0.2% offset of the strain

(*b*) 0.5% offset of the strain

(*c*) initial tangent

(*d*) initial secant modulus

160. When effect of wind or earthquake load is considered in the design of rivets and bolts for steel structures, by what percentage the permissible stresses may be exceeded ?

(*a*) 15% (*b*) 25%

(*c*) 33.33% (*d*) 50%

161. Horizontal stiffner is provided when

(*a*) depth of webs is small

(*b*) vertical stiffness becomes too close

(*c*) only thin plates are available for web

(*d*) both (*b*) and (*c*)

162. Only a portion of the area of outstanding leg in an angle section serving as tension member is considered in computing effective area of the member. because

(*a*) near the joint, outstanding leg does not take its full stress

(*b*) outstanding leg has a number of rivet holes reducing the net area

(*c*) outstanding leg is susceptible to buckling

(*d*) additional safety is preferred in case of tension failure

163. In a gabled industrial building in order to minimize the wind forces on the roof, the roof slope should be kept close to

(*a*) 5° (*b*) 15°

(*c*) 30° (*d*) 45°

164. Which one of the following statements is correct ? In a crane gantry girder, a channel is provided at the top flange girder. This accounts for bending in

(*a*) horizontal piane only

(*b*) vertical plane only

(*c*) horizontal and vertical planes

(*d*) horizontal and vertical planes and twisting

165. For steel structure proportioned using plastic design, working load (dead load + imposed load) should be multiplied by which of the following minimum load factor ?

(*a*) 1.3 (*b*) 1.5

(*c*) 1.7 (*d*) 2.0

166. A simply supported beam of span 4000 mm is loaded with a uniformly distributed load of 30 kN/m, f_y for the material is 250 MPa. Which rolled steel section is required ?

(*a*) ISMB 500 (*b*) ISMB 400

(*c*) ISMB 300 (*d*) ISMB 600

167. How are the most commonly produced and used structural elements in frames, floor beams, etc. with high a moment of inertia about x-axis, are designated?

(*a*) ISWB - section (*b*) ISLB - section
(*c*) ISMB - section (*d*) ISHB - section

168. Which steel member section among the following combinations can carry maximum load?

(*a*) A pair of angles welded on opposite sides of a gusset plate, but not tack welded
(*b*) A pair of angles welded on opposite sides of a gusset plate, and tack welded along its length
(*c*) A pair of angles back to back on same side of a gusset plate, and tack welded
(*d*) A pair of angles on same side of a gusset plate, but not tack welded

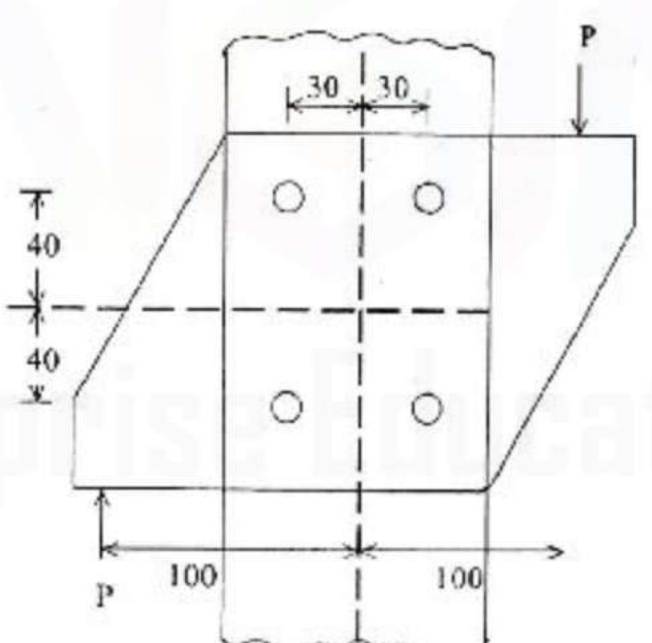
169. In a gusseted base, when end of the column is machined smooth for complete bearing, the axial load is transferred to base slab

(*a*) fully through fastening
(*b*) fully by direct bearing
(*c*) 50% by direct bearing and 50% through fastening
(*d*) 60% by direct bearing and 40% through fastening

170. In a situation where torsion is dominant, which of the following is the desirable section ?

(*a*) Angle section (*b*) Channel section
(*c*) I-section (*d*) Box type section

171.



All dimensions in mm

A bracket has been attached to flange of a column as shown in the figure. What is the maximum force in the bolt ?

(*a*) P/4 (*b*) P/2
(*c*) P (*d*) 2 P

172. Sag rods are provided because

(*a*) Sag rods reduce span length of purlin in the weak direction.
(*b*) Sag rods are installed in the plane of the roof.
(*c*) Ridge purlin is subject to vertical components of sag rods on either side of slope.
(*d*) all of these

173. Purlins are provided, in industrial buildings, over roof trusses to carry dead loads, live loads and wind loads. As per IS code, what are they assumed to be?

(*a*) Simply supported (*b*) Cantilever
(*c*) Continuous (*d*) Fixed

174. In plate girders

(*a*) a large number of cover plates are provided over flange angles so that curtailed flanges match the bending moment diagram exactly,
(*b*) atleast one cover plate should extend over the entire span so that rain water may not enter and corroded the connections.
(*c*) a minimum of one-third of flange area should be provided in flange angles and balance in flange cover plates for stability.
(*d*) all of these

175. Tie plates provided in laced columns to check

(*a*) buckling of column as a whole
(*b*) buckling of the lacing flats
(*c*) buckling of the component columns
(*d*) distortion of the end cross sections

176. Which of the following stresses is independent of yield stress as a permissible stress for steel members?

(*a*) Axial tensile stress
(*b*) Maximum shear stress
(*c*) Bearing stress
(*d*) Stress in slab base

177. In columns, splices should be provided at

(*a*) the floor levels
(*b*) the mid height of columns
(*c*) the beam-column joints
(*d*) one fourth height of columns

178. Which of the following should be employed to provide lateral support to the beams ?

(*a*) Bracing of compression flanges
(*b*) Shear connectors
(*c*) Embeding compression flanges into R.C.C. slab
(*d*) All of these

179. In beam to column connections in steel construction, if torsion is permitted at the ends of simply supported beams by not providing the cleats,

(*a*) effective length of the beam increases by 20%
(*b*) effective length remains same as the actual length
(*c*) permissible bending stresses are increased by around 10%
(*d*) joint has to be designed for torsion

180. For a vertical stiffened web of a plate girder, the lesser clear dimension of the panel should not exceed

(*a*) 85 t (*b*) 180 t
(*c*) 200 t (*d*) 250 t

(where t is thickness of the web)

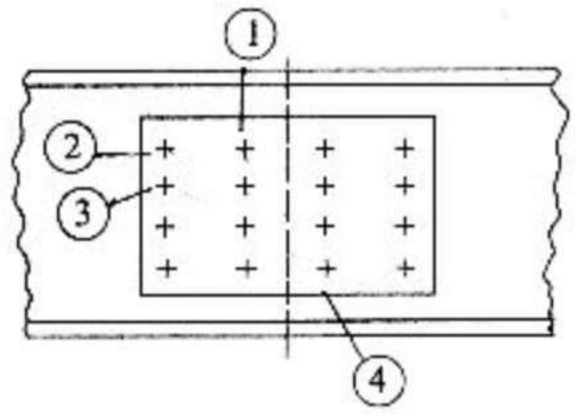
181. What is the permissible tensile stress in bolts used for column bases ?

(a) 120 N/mm^2 (b) 150 N/mm^2

(c) 0.6 f_y (d) 04 f_y

(where f_y is the yield stress of the steel)

182.



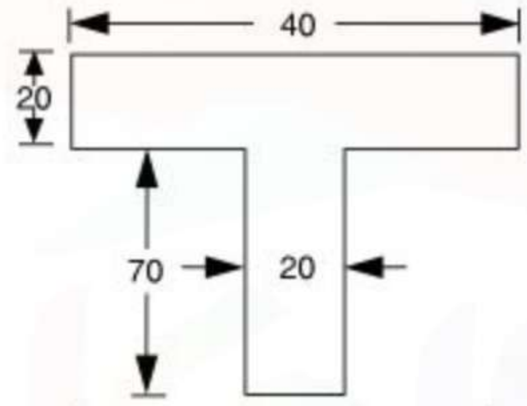
Which one of the bolts in a web splice of a plate girder as shown in the figure is stressed maximum?

(a) Bolt – 1 (b) Bolt – 2

(c) Bolt – 3 (d) Bolt – 4

NUMERICAL TYPE QUESTIONS

1. In the T-section shown in the figure (all dimension in mm), distance of plastic neutral axis from top is ______ mm



2. The shape factor of an isosceles triangle for bending about the axis parallel to the base is____

3. For an I-beam the shape factor is 1.12. Factor of safety in bending is 1.5. If allowable stress is increased by 20% for wind and earthquake loads, then load factor is _______

4. The plastic modulus of a section is 4.8×10^{-4} m^3. The shape factor is 1.2. The plastic moment capacity of the section is 120 kN-m. The yield stress of the material is ______ MPa

5. If a structue is statically indeterminate to second degree, then maximum number of plastic hinges required to render the structure a mechanism is ________

6. Shape factor for the diamond section is _______

7. A 10 mm thick plate is jointed with 12 mm thick thick plate using lap joint by 18 mm diameter rivet. The rivet value when permissible stresses for rivets in shear and bearing are 80 MPa and 250 MPa respectively and for plate in bearing is 250 MPa will be ________ kN

8. A member of a roof truss consists of two angle iron 80 × 50 × 6 mm placed back to back on both side of an 8 mm thick gusset plate. Number of 16 mm power driver field rivet when member carries a 71 kN direct load will be _______

9. Two plates of 16 mm and 14 mm are jointed by fillet, the maximum size of fillet weld may be____

10. If a lacing column is designed to carry 500 kN axial load, then total transverse shear resisted by lacing will be ________ kN

11. If a 55 I.S.F. 12 mm is used as lacing flat, then minimum radius of gyration will be ________ mm

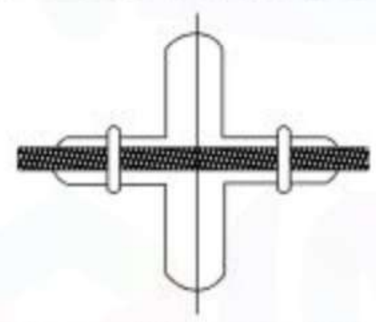
12. Slenderness ratio of lacing bars should not exceed ________

13. If 18 mm rivets are used in lacing bars, then minimum width of lacing bars should be ____ mm

14. Effective length of battened column is increased by _______%

15. According to IS specifications, effective length of a 8 m long column effectively held in position at both ends but not restrained in direction at one end is taken as _______ m

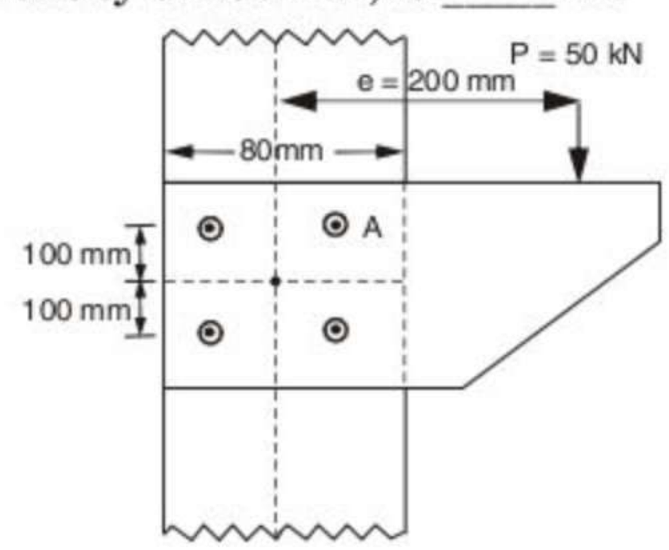
16. The net area of a tension member consisting of 4 ISA 75 × 75 × 8 mm connected by 18 mm dia rivet as shown in the figure is _______ cm^2



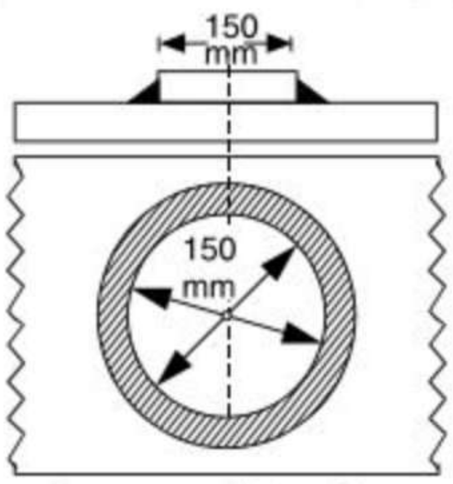
17. A simply supported beam of rectangular section and span L is subjected to a uniformly distributed load at the centre. The length of elastoplastic zone of plastic hinge will be ______ l

18. A steel welded plate girder is subjected to a maximum bending moment of 150 t-m. If maximum permissible bending stress is 1650 kg/cm^2 and use b thickness is 8 mm, then most economical depth of the girder will be ______ cm

19. In the given figure below, the total force on the rivet A due to eccentric loading 50 kN having an eccentricity of 200 mm, is _____ kN



20. A circular plate 150 mm diameter is welded to another plate by means of 6 mm fillet weld. If permissible shearing stress in the weld is 110 N/mm^2, then greater twisting moment that can be resisted by the weld will be ______ kN-m



21. A tension member consists of two angles 60 × 60 × 8, the angles being placed back to back on the same side of the gusset plate. One leg of each angle is connected to the gusset plate. The outstanding legs are also connected by tack rivets. If the permissible stress in tension be 150 MPa and diameter of rivets be 16 mm, then the safe tension for the member will be _____ kN

22. In a truss girder of a bridge, a diagonal consists of mild steel flat 400 ISF and carries a pull of 800 kN. If gross diameter of the rivets is 26 mm, then number of rivets required in the splice is ______

23. If diameter of the reduced end (if any) of the column 400 mm, then minimum cap of base plate will be _______ mm

24. If capacity of the electrically operated crane is 100 kN and weight of the crab is 25 kN, then latecral forces acting on the gantry girder will be _______ kN

25. M60 structural steel tube has a radius of gyration 20 mm. The unbraced length upto which the tube can be used as a compression member is ____ m

26. In a plate girder bridge of the unsupported depth of web is 800 mm, minimum thickness of web required only for vertical stiffner will be _______ mm

27. The effective length of a battened column is increased by _______ %

28. Minimum thickness of web in a plate girder, when plate is accessible and also exposed to weather, is ______ mm

29. To minimise total cost of a roof truss, the ratio of cost of truss to the cost of purlins shall be _____

30. The permissible stresses in rivets under wind load conditions as per IS: 800 can be exceeded by about ______ %

31. In a truss girder of a bridge, a diagonal consists of mild steel flat 400 ISF and carries a pull of 800 kN. If gross diameter of the rivets is 26 mm, then number of rivets required in the splice is _______

32. The slenderness ratio in tension member as per BIS code where reverse of stress is due to loads other than wind or seismic shall not exceed ____

33. The load factor to be used for plastic design of steel structures for dead load and imposed load is ____

34. The order of elongation which specimen of mild steel undergoes before fracture is ______%

35. An electric pole 5 m high is fixed into the foundation. It carries a wire at the top and is free to move sideways. The effective length of the pole is ______ m

36. Working strees for structural steel in tension is of the order of ______ N/mm^2

37. The centre to centre maximum distance between bolts in tension member of thickness 10 mm is _______ mm

38. A steel column in a multi-storeyed building carries an axial load of 125 N. It is built up of 2 ISMC 350 channles, connected by lacing. The lacing carries a load of ______ N

39. A structure has two degrees of indeterminacy. The number of plastic hinges that would be formed at complete collapse is ______

40. A 6 mm thick mild steel plate is connected to an 8 rnm thick 16 mm diameter shop rivets. The number of rivets required to carry an 80 kN load is _______

41. A member is subjected to axial compression Effective length is 3000 mm. Size of the angle used is 100 x 100 x 10 (sectional area 1903 mm^2, 8 mm = 19.4). The maximum capacity (if f_y = 250 MPa) is _______kN

42. What is the maximum slenderness ratio for a steel member carrying compressive loads resulting from dead loads and imposed loads is ______

43. 2-ISMB 300 x 140 section are acting as a compound column. The height of the column is 400 mm. Two sections are spaced 400 mm, centre to centre . In the longitudinal direction there is runner at the top of the column. The maximum capacity of the column in compression (f_y = 250 Mpa) is _______ kN

44. The maximum slenderness ratio of lacing bars in built-up columns is _______

45. The plastic modulus of a section is 5×10^{-4} m^3. Its shape factor is 1.2 and the plastic moment capacity is 120 kNm. The value of the yield stress of the material is _______ N/mm^2

46. A steel rod of 16 mm diameter has been used as a tie in a bracing system, but may be subject to possible reversal of stress due to the wind. The maximum permitted length of the members is ______ mm

EXERCISE – II

(QUESTIONS FROM PREVIOUS GATE EXAMS)

MCQ TYPE QUESTIONS

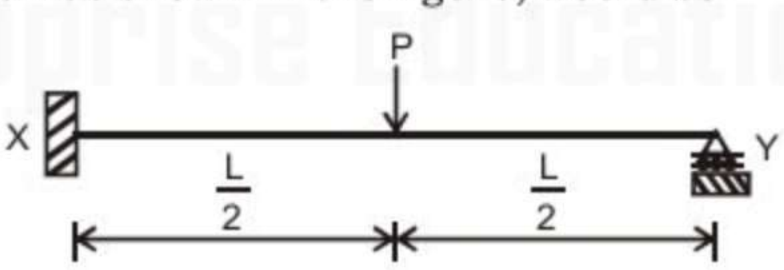
2014

1. Match the information given in Group-I with those in Group-II.

Group-I	**Group-II**
P. Factor to decrease ultimate strength to design strength	1. Upper bound on ultimate load
Q. Factor to increase working load to ultimate load for design	2. Lower bound on ultimate load
R. Statical method of ultimate load analysis	3. Material partial safety factor
S. Kinematical mechanism method of ultimate load analysis	4. Load factor

	P	**Q**	**R**	**S**
(*a*)	1	2	3	4
(*b*)	2	1	4	3
(*c*)	3	4	2	1
(*d*)	4	3	2	1

2. The ultimate collapse load (P) in terms of plastic moment M_p by kinematic approach for a propped cantilever of length L with P acting at its mid-span as shown in the figure, would be



(*a*) $P = \frac{2M_P}{L}$ (*b*) $P = \frac{4M_P}{L}$

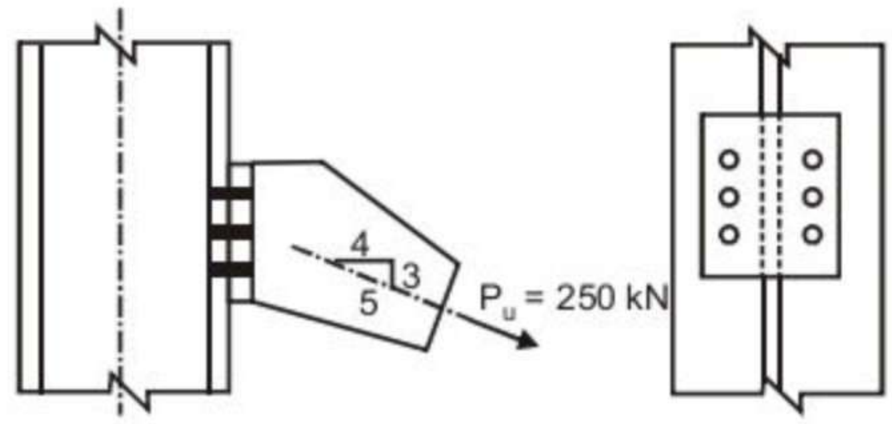
(*c*) $P = \frac{6M_P}{L}$ (*d*) $P = \frac{8M_P}{L}$

3. A steel section is subjected to a combination of shear and bending actions. The applied shear force is V and the shear capacity of the section is V_s. For such a section, high shear force (as per IS: 800-2007) is defined as

(*a*) $V > 0.6\ V_s$ (*b*) $V > 0.7\ V_s$

(*c*) $V > 0.8\ V_s$ (*d*) $V > 0.9\ V_s$

4. The tension and shear force (both in kN) in each bolt of the joint, as shown below, respectively are



(*a*) 30.33 and 20.00 (*b*) 30.33 and 25.00

(*c*) 33.33 and 20.00 (*d*) 33.33 and 25.00

5. A steel member 'M' has reversal of stress due to live loads, whereas another member 'N' has reversal of stress due to wind load. As per IS 800:2007, the maximum slenderness ratio permitted is

(*a*) less for member 'M' than that of member 'N'

(*b*) more for member 'M' than for member 'N'

(*c*) same for both the members

(*d*) not specified in the Code

6. Prying forces are

(*a*) shearing forces on the bolts because of the joints

(*b*) tensile forces due to the flexibility of connected parts

(*c*) bending forces on the bolts because of the joints

(*d*) forces due the friction between connected parts

2013

7. As per IS 800:2007, the cross-section in which the extreme fiber can reach the yield stress, but cannot develop the plastic moment of resistance due to failure by local buckling is classified as

(*a*) plastic section

(*b*) compact section

(*c*) semi-compact section

(*d*) slender section

8. Two steel columns P (length L and yield strength f_y = 250 MPa) and Q (length 2L and yield strength f_y = 500 MPa) have the same cross-sections and end-conditions. The ratio of buckling load of column P to that of column Q is

(*a*) 0.5 (*b*) 1.0

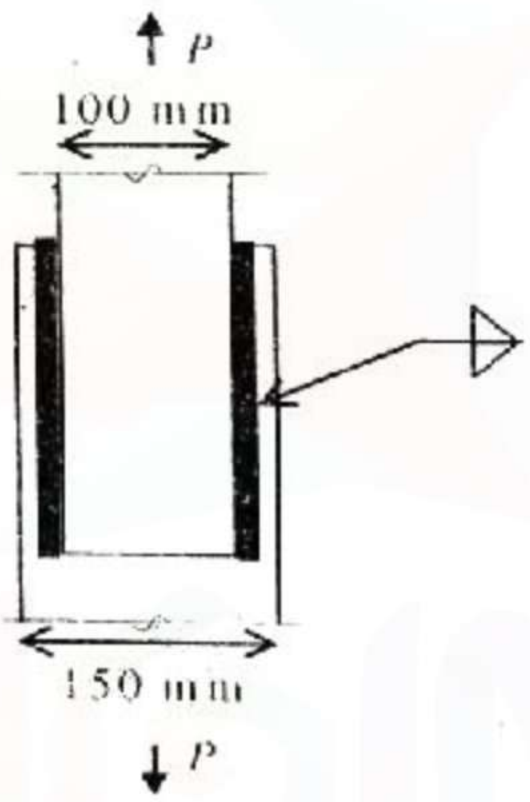
(*c*) 2.0 (*d*) 4.0

2012

9. In a steel plate with bolted connections, the rupture of the net section is a mode of failure under

(*a*) tension (*b*) compression
(*c*) flexure (*d*) shear

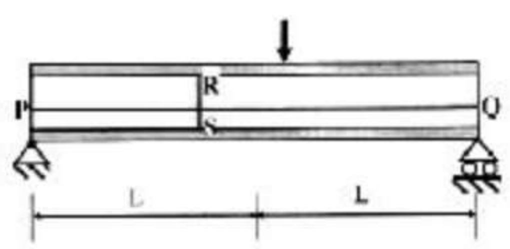
10. Two plates are connected by fillet welds of size 10 mm and subjected to tension, as shown in the sketch. The thickness of each plate is 12 mm. The yield stress and the ultimate tensile stress of steel are 250 MPa and 410 MPa respectively. The welding is done in the workshop ($\gamma_{mw} = 1.25$). As per the Limit State Method of IS 800 : 2007, the minimum length (rounded off to the nearest higher multiple of 5 mm) of each weld to transmit a force P equal to 270 kN is



(*a*) 100 mm (*b*) 105 mm
(*c*) 110 mm (*d*) 115 mm

2011

11. The adjoining figure shows a schematic representation of a steel plate girder to be used as a simply supported beam with a concentrated load. For stiffeners, PQ (running along the beam axis) and RS (running between the top and bottom flanges) which of the following pairs of statements will be TRUE?



(*a*) (*i*) RS should be provided under the concentrated load only.
(*ii*) PQ should be placed in the tension side of the flange.

(*b*) (*i*) RS helps to prevent local buckling of the web.
(*ii*) PQ should be placed in the compression side of the flange.

(*c*) (*i*) RS should be provided at supports.
(*ii*) PQ should be placed along the neutral axis.

(*d*) (*i*) RS should be provided away from points of action of concentrated loads.
(*ii*) PQ should be provided on the compression side of the flange.

2010

12. The effective length of a column of length L fixed against rotation and translation at one end and free at the other end is

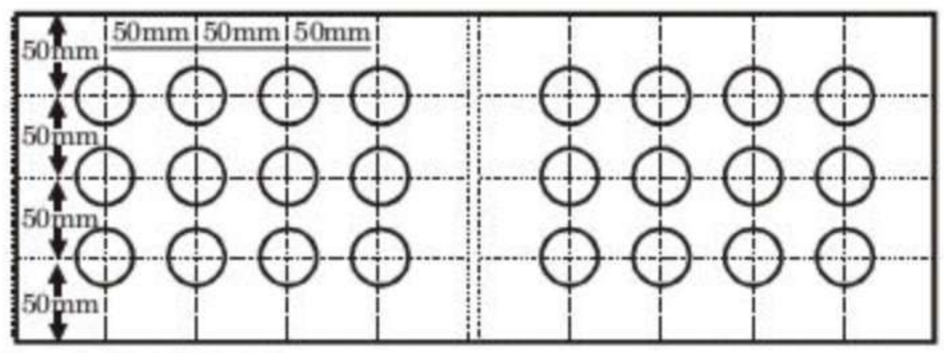
(*a*) 0.5 L
(*b*) 0.7 L
(*c*) 1.414 L
(*d*) 2L

13. A solid circular shaft of diameter *d* and length L is fixed at one end free at the other end. A torque T is applied at the free end. The shear modulus of the material is G. The angle of twist at the free end is

(*a*) $\dfrac{16TL}{\pi d^4 G}$ (*b*) $\dfrac{32TL}{\pi d^4 G}$

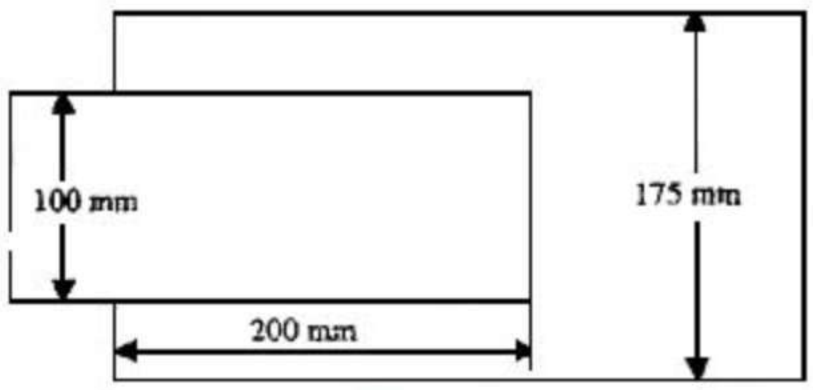
(*c*) $\dfrac{64TL}{\pi d^4 G}$ (*d*) $\dfrac{128TL}{\pi d^4 G}$

14. A double cover butt riveted joint is used to connect two flat of 200 mm width and 14 mm thickness as shown in the figure. There are twelve power driven rivets of 20 mm diameter at a pitch of 50 mm in both directions on either side of the plate. Two cover plates of 10 mm thickness are used. the capacity of the joint in tension considering bearing and shear ONLY, with permissible bearing and shear stresses as 300 MPa and 100 MPa respectively is



(*a*) 1083.6 kN
(*b*) 871.32 kN
(*c*) 541.8 kN
(*d*) 433.7 kN

15. Two plates, subjected to direct ension, each of 10 mm thickness and having widths of 100 mm and 175 mm, respectively are to be fillet welded with an overlap of 200 mm. Given that the permissible weld stress is 110 MPa and the permissible stress in steel is 150 MPa, the length of the weld required using the maximum permissible weld size as per IS : 800-1984 is



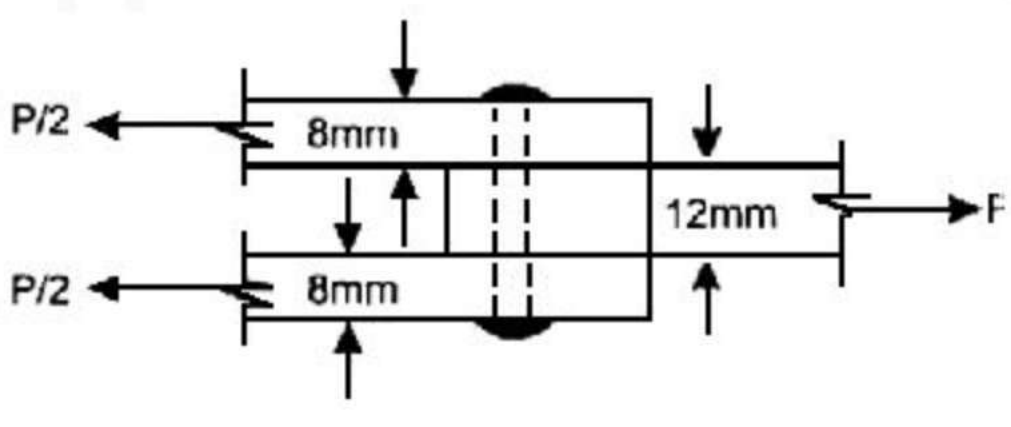
(*a*) 245.3 mm (*b*) 229.2 mm
(*c*) 205.5 mm (*d*) 194.8 mm

2009

16. In the theory of plastic bending of beams, the ratio of plastic moment to yield moment is called

(*a*) shape factor
(*b*) plastic section modulus
(*c*) modulus of resilience
(*d*) rigidity modulus

17. A 12 mm thick plate is conected to two 8 mm thick plates, on either side through a 16 mm diameter power driven field rivet as shown in the figure below. Assuming permissible shear stress as 90 MPa and permissible bearing stress as 270 MPa in the rivet, the rivet value of the joint is



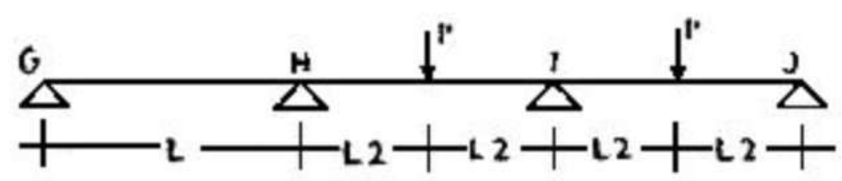
(*a*) 56.70 kN
(*b*) 43.29 kN
(*c*) 36.19 kN
(*d*) 21.65 kN

2008

18. Rivets and bolts subjected to both shear stress ($\tau_{vf,\,cal}$) and axial tensile stress ($\sigma_{tf,\,cal}$) shall be so proportioned that the stresses do not exceed the respective allowable stresses τ_{vf} and σ_{tf} and the value of $\left(\dfrac{\tau_{vf,\,cal}}{\tau_{vf}} + \dfrac{\sigma_{tf,\,cal}}{\sigma_{tf}}\right)$ does not exceed

(*a*) 1.0 (*b*) 1.2
(*c*) 1.4 (*d*) 1.8

19. A continuous beam is loaded as shown in the figure below. Assuming a plastic moment capacity equal to M_p, the minimum load at which the beam would collapse is

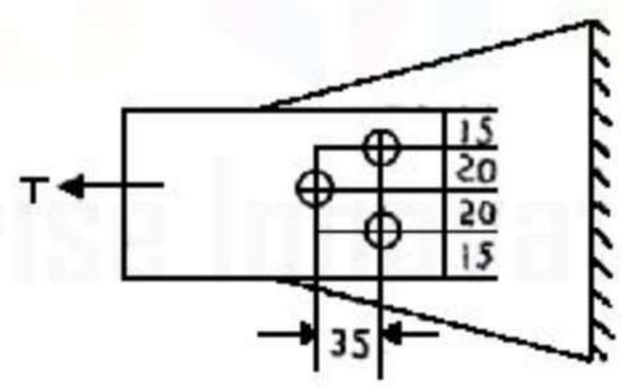


(*a*) $\dfrac{4M_p}{L}$ (*b*) $\dfrac{6M_p}{L}$

(*c*) $\dfrac{8M_p}{L}$ (*d*) $\dfrac{10M_p}{L}$

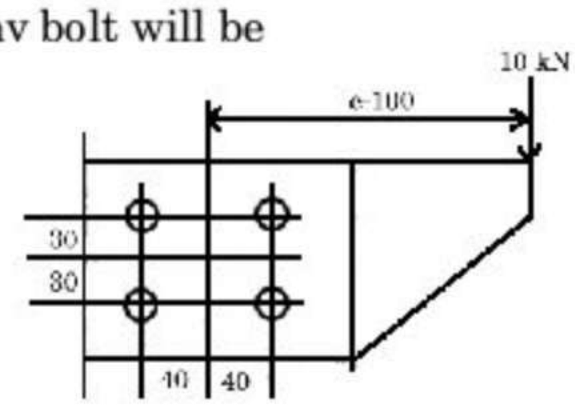
2007

20. A steel flat of rectangular section of size 70 × 6 mm is connected to a gusset plate by three bolts each having a shear capacity of 15 kN in holes having diameter 11.5 mm. If allowable tensile stress in the flat is 150 MPa, maximum tension that can be applied to the flat is



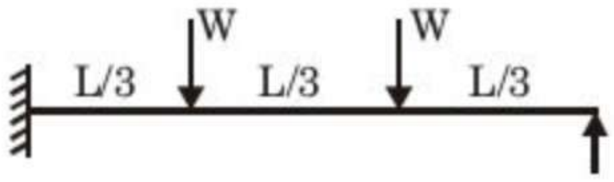
(*a*) 42.3 kN (*b*) 52.65 kN
(*c*) 59.5 kN (*d*) 63.0 kN

21. A bracket connection is made with four bolts of 10 mm diameter and supports a load of 10 kN at an eccentricity of 100 mm. The maximum force to be resisted by anv bolt will be



(*a*) 5 kN
(*b*) 6.5 kN
(*c*) 6.8 kN
(*d*) 7.16 kN

22. The plastic collapse load W_p for the propped cantilever supporting two point loads as shown in figure in terms of plastic moment capacity, M_p is given by



(a) $\frac{3M_p}{L}$ (b) $\frac{4M_p}{L}$

(c) $\frac{5M_p}{L}$ (d) $\frac{6M_p}{L}$

2006

23. In the design of welded tension members, consider the following statements :

I. The entire cross-sectional area of the connected leg is assumed to contribute to the effective area in case of angles.

II. Two angles back-to-back and tack-welded as per the codal requirements may be assumed to behave as a tee section.

III. A check on slenderness ratio may be necessary in some cases.

The TRUE statements are

(a) only I and II (b) only II and III

(c) only I and III (d) I, II and III

24. Consider the following statements

I. Effective length of a battened column is usually increased to account for the additional load on battens due to the lateral expansion of columns.

II. As per IS: 800-1984, permissible stress in bending compression depends on both Euler buckling stress and the yield stress of steel.

III. As per IS: 800-1984, the effective length of a column effectively held in position at both ends but not restrained against rotation, is taken to be greater than that in the ideal end conditions.

The TRUE statements are

(a) only I and II

(b) only II and III

(c) only I and III

(d) I, II and III

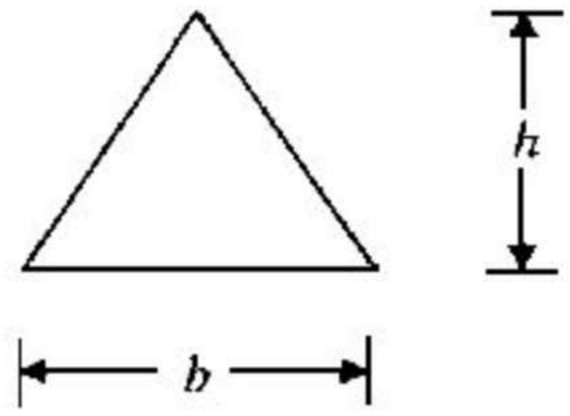
25. When triangular section of a beam as shown below becomes a plastic hinge, then compressive force acting on the section (with σ_y denoting the yield stress) becomes

(a) $\frac{bh\sigma_y}{4}$

(b) $\frac{2bh\sigma_y}{9}$

(c) $\frac{bh\sigma_y}{2}$

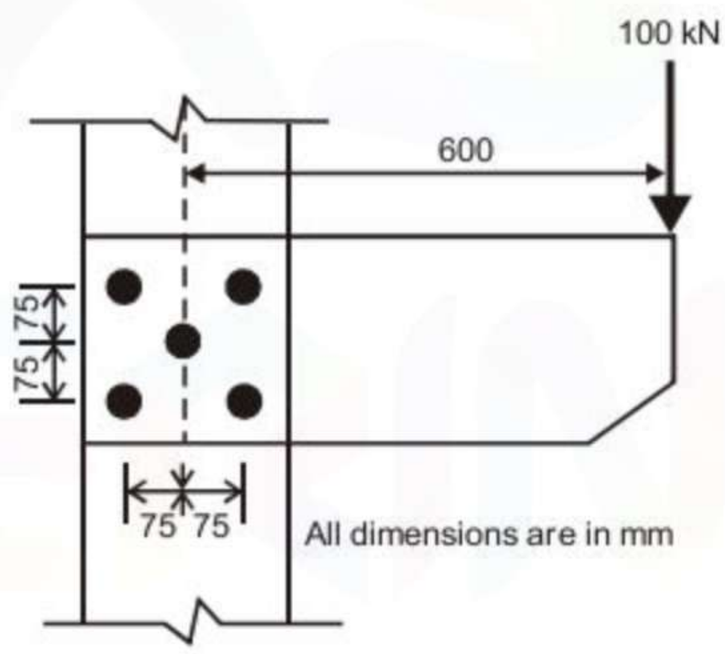
(d) $\frac{bh\sigma_y}{3}$



NUMERICAL TYPE QUESTIONS

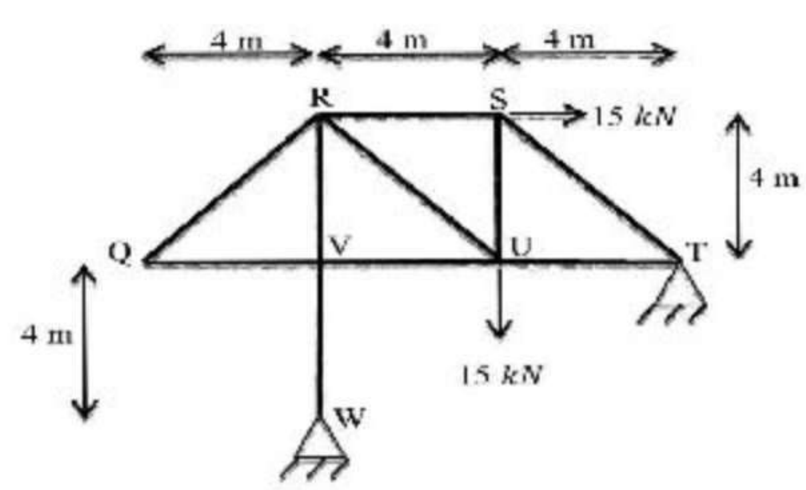
2014

1. A bracket plate connected to a column flange transmits a load of 100 kN as shown in the following figure. The maximum force for which the bolts should be designed is _________ kN.



2013

2. The pin-jointed 2-D truss is loaded with a horizontal force of 15 kN at joint S and another 15 kN vertical force at joint U, as shown. Find the force in member RS (in kN) and report your answer taking tension as positive and compression as negative. _____



ANSWERS

EXERCISE – I

MCQ Type Questions

1. (*c*)	**2.** (*c*)	**3.** (*a*)	**4.** (*c*)	**5.** (*b*)	**6.** (*d*)	**7.** (*c*)	**8.** (*b*)	**9.** (*d*)	**10.** (*c*)
11. (*b*)	**12.** (*c*)	**13.** (*c*)	**14.** (*d*)	**15.** (*d*)	**16.** (*b*)	**17.** (*a*)	**18.** (*a*)	**19.** (*b*)	**20.** (*c*)
21. (*a*)	**22.** (*a*)	**23.** (*b*)	**24.** (*d*)	**25.** (*b*)	**26.** (*a*)	**27.** (*d*)	**28.** (*d*)	**29.** (*d*)	**30.** (*a*)
31. (*b*)	**32.** (*c*)	**33.** (*c*)	**34.** (*c*)	**35.** (*a*)	**36.** (*d*)	**37.** (*b*)	**38.** (*b*)	**39.** (*b*)	**40.** (*d*)
41. (*c*)	**42.** (*a*)	**43.** (*c*)	**44.** (*c*)	**45.** (*a*)	**46.** (*c*)	**47.** (*d*)	**48.** (*b*)	**49.** (*d*)	**50.** (*b*)
51. (*d*)	**52.** (*a*)	**53.** (*d*)	**54.** (*a*)	**55.** (*c*)	**56.** (*a*)	**57.** (*b*)	**58.** (*a*)	**59.** (*d*)	**60.** (*d*)
61. (*b*)	**62.** (*c*)	**63.** (*b*)	**64.** (*a*)	**65.** (*a*)	**66.** (*c*)	**67.** (*c*)	**68.** (*c*)	**69.** (*a*)	**70.** (*c*)
71. (*b*)	**72.** (*d*)	**73.** (*d*)	**74.** (*b*)	**75.** (*c*)	**76.** (*d*)	**77.** (*c*)	**78.** (*c*)	**79.** (*c*)	**80.** (*d*)
81. (*b*)	**82.** (*b*)	**83.** (*b*)	**84.** (*c*)	**85.** (*c*)	**86.** (*a*)	**87.** (*b*)	**88.** (*c*)	**89.** (*a*)	**90.** (*c*)
91. (*d*)	**92.** (*c*)	**93.** (*d*)	**94.** (*b*)	**95.** (*c*)	**96.** (*a*)	**97.** (*a*)	**98.** (*c*)	**99.** (*c*)	**100.** (*b*)
101. (*b*)	**102.** (*a*)	**103.** (*a*)	**104.** (*c*)	**105.** (*b*)	**106.** (*b*)	**107.** (*b*)	**108.** (*b*)	**109.** (*b*)	**110.** (*a*)
111. (*c*)	**112.** (*b*)	**113.** (*b*)	**114.** (*c*)	**115.** (*a*)	**116.** (*a*)	**117.** (*b*)	**118.** (*c*)	**119.** (*d*)	**120.** (*c*)
121. (*b*)	**122.** (*c*)	**123.** (*a*)	**124.** (*d*)	**125.** (*d*)	**126.** (*c*)	**127.** (*a*)	**128.** (*b*)	**129.** (*c*)	**130.** (*b*)
131. (*c*)	**132.** (*a*)	**133.** (*d*)	**134.** (*c*)	**135.** (*c*)	**136.** (*a*)	**137.** (*a*)	**138.** (*c*)	**139.** (*c*)	**140.** (*b*)
141. (*b*)	**142.** (*c*)	**143.** (*a*)	**144.** (*d*)	**145.** (*c*)	**146.** (*c*)	**147.** (*a*)	**148.** (*d*)	**149.** (*c*)	**150.** (*a*)
151. (*d*)	**152.** (*c*)	**153.** (*c*)	**154.** (*c*)	**155.** (*b*)	**156.** (*b*)	**157.** (*c*)	**158.** (*b*)	**159.** (*a*)	**160.** (*b*)
161. (*a*)	**162.** (*a*)	**163.** (*c*)	**164.** (*a*)	**165.** (*c*)	**166.** (*c*)	**167.** (*d*)	**168.** (*b*)	**169.** (*b*)	**170.** (*d*)
171. (*c*)	**172.** (*d*)	**173.** (*c*)	**174.** (*d*)	**175.** (*d*)	**176.** (*d*)	**177.** (*d*)	**178.** (*d*)	**179.** (*a*)	**180.** (*b*)
181. (*a*)	**182.** (*b*)								

Numerical Type Questions

1. (35)	**2.** (2.34)	**3.** (1.40)	**4.** (250)	**5.** (3)	**6.** (2)	**7.** (23.89)
8. (2)	**9.** (12.5)	**10.** (12.5)	**11.** (3.46)	**12.** (145)	**13.** (55)	**14.** (10)
15. (6.4)	**16.** (32)	**17.** (0.577)	**18.** (120)	**19.** (30.17)	**20.** (16.33)	**21.** (196.5)
22. (9)	**23.** (715)	**24.** (12.5)	**25.** (3.6)	**26.** (4)	**27.** (10)	**28.** (8)
29. (2)	**30.** (25)	**31.** (7)	**32.** (350)	**33.** (1.7)	**34.** (1)	**35.** (10.0)
36. (150)	**37.** (200)	**38.** (3.125)	**39.** (3)	**30.** (4)	**41.** (81.7)	**42.** (180)
43. (2002)	**44.** (145)	**45.** (240)	**46.** (1400)			

EXERCISE – II

MCQ Type Questions

1. (*d*)	**2.** (*c*)	**3.** (*a*)	**4.** (*d*)	**5.** (*a*)	**6.** (*b*)	**7.** (*c*)	**8.** (*d*)	**9.** (*a*)	**10.** (*b*)
11. (*b*)	**12.** (*d*)	**13.** (*b*)	**14.** (*b*)	**15.** (*b*)	**16.** (*a*)	**17.** (*b*)	**18.** (*c*)	**19.** (*b*)	**20.** (*c*)
21. (*d*)	**22.** (*d*)	**23.** (*b*)	**24.** (*a*)	**25.** (*a*)					

Numerical Type Questions

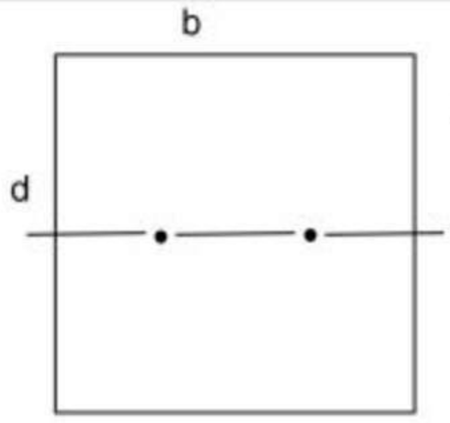
1. (156.20)	**2.** (0)

EXPLANATIONS

EXERCISE – I

MCQ TYPE QUESTIONS

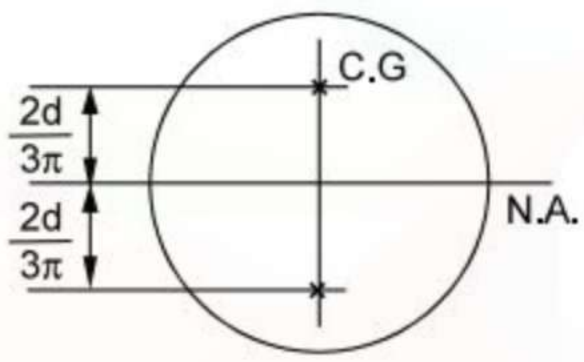
1.



$$Z_p = A_f . \bar{y}_f + A_c \bar{y}_c$$

$$Z_p = \frac{bd}{2}\left[\frac{d}{4}+\frac{d}{4}\right] = \frac{bd^2}{4}$$

2. Plastic section modulus for circular section is



$$Z_p = \frac{\pi d^2}{4 \times 2}\left[2 \times \frac{2d}{3\pi}\right]$$

$$= \frac{\pi d^2}{8} \times \frac{4d}{3\pi}$$

$$= \frac{\pi d^3}{6\pi} = \frac{d^3}{6}$$

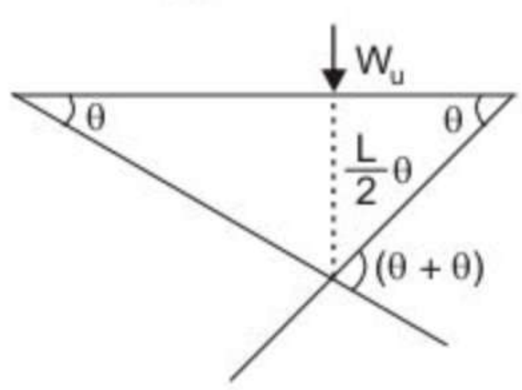
$$Z = \frac{\pi d^4}{64} \times \frac{2}{d}$$

$$= \frac{\pi d^3}{32}$$

$$\therefore \frac{Z_p}{Z} = \frac{d^3}{6} \times \frac{32}{\pi d^3}$$

$$= \frac{16}{3\pi}$$

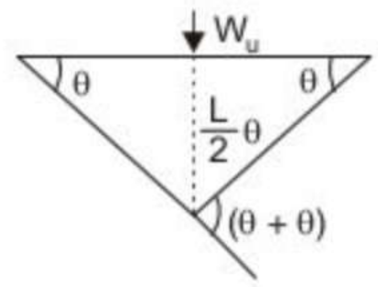
5.



$$M_p (2\theta) = W_u \frac{L}{2} \times \theta$$

$$W_u = \frac{4M_p}{L}$$

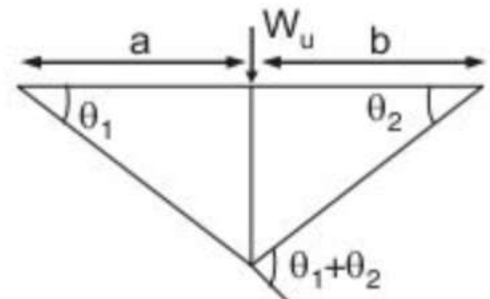
6.



$$4 M_p \theta = W_u \frac{L}{2} \theta$$

$$W_u = \frac{8 M_p}{L}$$

8.



$$M_p \theta_1 + M_p \theta_2 + M_p (\theta_1 + \theta_2) = W_u a \theta_1$$

$$\therefore \quad M_p [2\theta_1 + 2\theta_2] = W_u a \theta_1$$

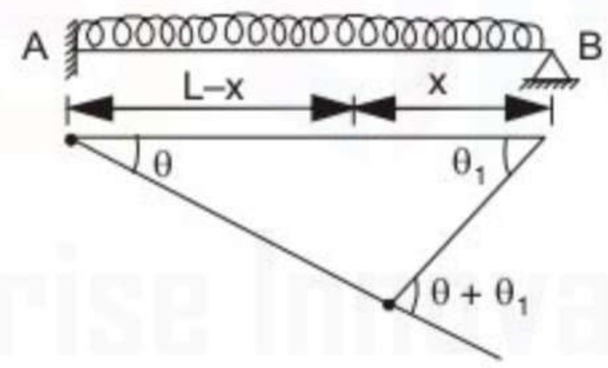
$$M_p 2\left[1+\frac{a}{b}\right] \theta_1 = W_u a \theta_1$$

$$\left[\text{since } a\theta_1 = b\theta_2 \text{ or } \theta_2 = \frac{a}{b}\theta_1\right]$$

$$\therefore \quad M_p 2\left[1+\frac{a}{b}\right]\theta_1 = W_u a \theta_1$$

$$\therefore \quad W_u = \frac{2M_p(a+b)}{ab} = \frac{2M_pL}{ab}$$

10.



$$\theta_1 = \frac{L-x}{x}\theta$$

$$\therefore \quad M_p\theta + M_p(\theta + \theta_1) = \frac{1}{2} W_u L (L-x)\theta$$

$$\therefore \quad M_p = \frac{W_u L}{2} \frac{x(L-x)}{(L+x)}$$

$$\frac{dM_p}{dx} = 0$$

$$\Rightarrow \quad x = 0.414 \text{ L from B}$$

24.

$$P = ftb_1$$

and $b_1 = b + h_2\sqrt{3} = 0.5\,d + \sqrt{3}\,d$

$$\approx 2.0\,d$$

$$\therefore \quad P = 2.0\,dtf$$

35. At location of plastic hinge, radius is zero. So curvature 1/R is infinite.

42. $$P_1 = \frac{Ph_2}{h_1 + h_2}$$

$$P_2 = \frac{Ph_1}{h_1 + h_2}$$

$$\frac{P_1}{P_2} = \frac{h_2}{h_1}$$

44. When both ends fixed,

$$L_{e1} = 0.6\ L$$

$$\therefore \quad L = \frac{L_{e1}}{0.6}$$

When one end hinged,

$$L_{e2} = 0.85\ L$$

$$\therefore \quad L = \frac{L_{e2}}{0.85}$$

$$\frac{L_{e1}}{0.6} = \frac{L_{e2}}{0.85}$$

$$\Rightarrow \quad L_{e2} = \frac{0.85}{0.6}$$

$$L_{e1} = \mathbf{1.41}\ l$$

49. $$t = \left[\frac{3w}{\sigma_{bs}}\left(a^2 - \frac{b^2}{4}\right)\right]^{\frac{1}{2}}$$

where, $w = \dfrac{3500 \times 10^3}{900 \times 900} = 4.32\ \text{N/mm}^2$

a = bigger projection $= \dfrac{900 - 250}{2} = 325$

b = smaller projection $= \dfrac{900 - 350}{2} = 275$

σ_{bs} = allowable bending stress in slab (base) = 185 N/mm

$$\therefore \quad t = \left\{\frac{3 \times 4.32}{185}\left[325^2 - \frac{275^2}{4}\right]\right\}^{\frac{1}{2}} = \mathbf{77.94\ mm}$$

$$\simeq 78\ \text{mm}$$

54. $$P_{cr} = \frac{\pi^2 EI}{l_e^2}$$

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

$$\therefore \quad P_{cr} \propto \frac{1}{\lambda^2} \text{ not } \frac{1}{\lambda} \text{ or } \lambda$$

57. $$T_1 = T_3$$

$$\Rightarrow \quad 2T_1 + T_2 = W$$

$$\therefore \quad \Delta_1 = \Delta_2$$

$$\Rightarrow \quad \frac{T_1 L}{AE} = \frac{T_2(l/2)}{AE}$$

$$\Rightarrow \quad T_2 = 2T_1$$

Let T_2 is collapse first

$$\therefore \quad T_2 = \sigma_Y A = W$$

$$\Rightarrow \quad T_1 = \frac{T_2}{2} = \frac{W_1}{2}$$

$$W_c = T_1 = \frac{W}{2}$$

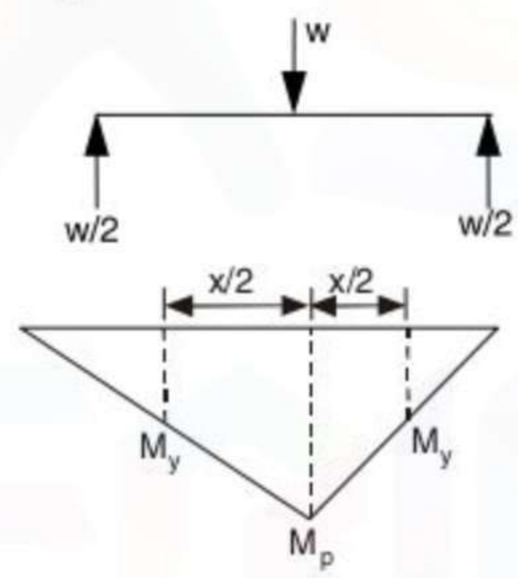
Let at yielding point

$$T_1 = T_2 = \sigma_y A = W$$

$$\Rightarrow \quad W_y = \frac{W}{2}$$

$$\therefore \quad \frac{W_c}{W_y} = \frac{3}{2}$$

76.



$$\because \quad \frac{M_p}{\frac{L}{2}} = \frac{M_y}{\frac{L}{2} - \frac{x}{2}}$$

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

$$\frac{M_p}{M_y} = \text{shape factor} = 1.5$$

$$\Rightarrow \quad \frac{L}{L - x} = 1.5$$

$$\Rightarrow \quad 1.5\ L - 1.5\ x = L$$

$$\Rightarrow \quad x = \frac{0.5\ L}{1.5} = \frac{1}{3}$$

77. $$M = \left[\frac{bh}{4} \cdot \frac{3h}{8} \sigma_y + \frac{1}{2} \cdot \frac{bh}{4} \cdot \frac{2}{3} \cdot \frac{h}{4} \cdot \sigma_y\right] \times 2$$

$$= \frac{bh^2}{32} \sigma_y \left[3 + \frac{2}{3}\right] \times 2 = \frac{11}{48}\ bh^2 \sigma_y$$

79. Bending stress $= \dfrac{M}{I_{xx}} \times y$

$$= \frac{\left(\frac{20 \times 9}{8}\right) \times 10^3 \times 10^3}{1696.6 \times 10^4} \times 100$$

$= 132.62$ N/mm^2

Shear stress $= \dfrac{V}{t_w.D} = \dfrac{\left(\frac{20 \times 3}{2}\right) \times 10^3}{5.4 \times 100}$

$= 27.78$ N/mm^2

82. $\dfrac{d}{t_w} = \dfrac{1000}{6} = 166.67$

$\Rightarrow \dfrac{d_1}{t_w} < \underline{\underline{200}}$ but $> \underline{\underline{85}}$, vertical stiffness provided.

86. Shear stress, $f_s = \dfrac{80 \times 10^3}{4 \times \frac{\pi}{4} \times 19.5^2}$

$= 67$ N/mm^2

Tensile stress, $f_d = \dfrac{Pe \cdot y / \sum y^2}{\frac{\pi}{4}(19.5)^2}$

$= 27.9$ N/mm^2

87. The angle will act in pairs on both side of gusset plate

A_{net} = gross area – deduction for holes

$= 2 \times 858 - 2 \times 17.5 \times 8$

$= 1436$ mm^2

Safe tension $= \sigma_{af} \times A_{net}$

$= \dfrac{150 \times 1436}{1000}$

$= 215.4$ kN

90. $t = \sqrt{\dfrac{3w}{\sigma_{bs}}\left(a^2 - \dfrac{b^2}{4}\right)}$

take w is in mPa

$= \sqrt{\dfrac{3 \times 4}{185}\left(250^2 - \dfrac{250^2}{4}\right)}$

$= 55.14$ mm

≈ 5.6 cm

91. One vertical stiffner and two horizontal stiffeners (one at 2/5th distance from compression flange and another one at neutral axis) are provided

94. Effective net width

$= b - nd + \dfrac{S_1^2}{4_g} + \dfrac{S_2^2}{4_g}$

$= 300 - 3 \times 25 + \dfrac{50^2}{4 \times 100} + \dfrac{50^2}{4 \times 100}$

$= 237.5$ mm

100. 2.5% of column load is added because braces support the column laterally.

111. Bearing of minimum 100 mm length should be provided.

113. They are provided to check the diagonal buckling of the web.

121. Internal air pressure depends upon the degree of permeability of roof or wall.

127. $M_{xx} = M \cos(30) = M\left(\dfrac{\sqrt{3}}{2}\right)$

130. Upper Bound = $P \geq P_u$

- Subjected to a set of loads (P).

Lower Bound = $P \leq P_u$

- Subjected to distribution of bonding moment.

166. Maximum B.M. $= \dfrac{wl^2}{8} = \dfrac{30 \times 4^2}{8} = 60$ kN-m

σ_{bc} or $\sigma_{bc} = 0.66 f_y = 0.66 \times 250 = 168$

So Z_{xx} required $= \dfrac{60 \times 10^6}{165} = 363.63 \times 10^3$ mm^3

$\because \quad Z_{xx} = 363.63$ cm^3

See from steel table,

the section ISMB 300 have the

$Z_{xx} = 599$ cm$^3 > 363.63$ cm

So the ans is ISMB 300

171. $F_1 = \dfrac{2P}{4} = \dfrac{P}{2}$

$F_2 = \dfrac{P \times 100 \times 50}{4(50)^2} = \dfrac{P}{2}$

$\therefore \quad C_e = \sqrt{\left(\dfrac{p}{2}\right)^2 + \left(\dfrac{p}{2}\right)^2 + 2 \times \left(\dfrac{p}{2}\right)\left(\dfrac{p}{2}\right)}$

182. Maximum stress is occured in the bolt of fartest distance from neutral axis

NUMERICAL TYPE QUESTIONS

1. M.A divides the total areas into two equal parts

$40 \times 20 + 20 \times x = 20 \times (70 - x)$

$\Rightarrow \quad 800 + 20x = 1400 - 20x$

$\Rightarrow \quad 40x = 600$

$$x = \frac{600}{40} = 15$$

$\therefore$ Distance from top = 20 + 15 = 35 mm

3. Load factor = Shape factor × F.O.S.

$= 1.22 \times 1.5 = 1.68$

$\therefore$ Reduced load factor $= \frac{1.68}{1.2}$ (as 20%) = 1.4

4. $M_p = Z_p f_y$

$$\Rightarrow \quad f_y = \frac{120 \times 10^6}{4.8 \times 10^{-4} \times 10^9} = 250 \text{ MPa}$$

5. Plastic hinges required = degree of indeterminacy + 1

$= 2 + 1 = 3$

7. Gross diameter = 19.5 mm

Strength of rivet is bearing $= \sigma_{pf}\, dt = 48.75$ kN

Strength in single shear $= \frac{\pi}{4} d^2 \, \tau_{vf} = 23.89$

Lesser of these two is rivet value.

$\because \quad R_v = 23.89$ kN

8. Strength of rivet in bearing on 8 mm gusset

$$= \sigma_{pf}\, dt = \frac{270 \times 17.5 \times 8}{1000} = 37.8 \text{ kN}$$

Strength of rivet in double shear

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

$$= 2 \times \frac{\pi}{4} \times 17.5^2 \times \frac{90}{1000} = 43.3 \text{ kN}$$

$\therefore$ Rivet value = 37.8 kN

Number of rivets $= \frac{71}{37.8} \approx 2$

9. Maximum size = 14 – 1.5 = 12.5

10. $500 \times 2.5\% = 12.5$ kN

11. Minimum radius of gyration $= \frac{t}{\sqrt{12}}$

$t = 12$

$$r = \frac{12}{\sqrt{12}} = 3.46 \text{ mm}$$

15. $l_{\text{eff}} = 0.8\, L = 0.8 \times 8$

$= 6.4$ m

16. $A_{\text{net}} = 4\,(A_1 + K\,A_2)$

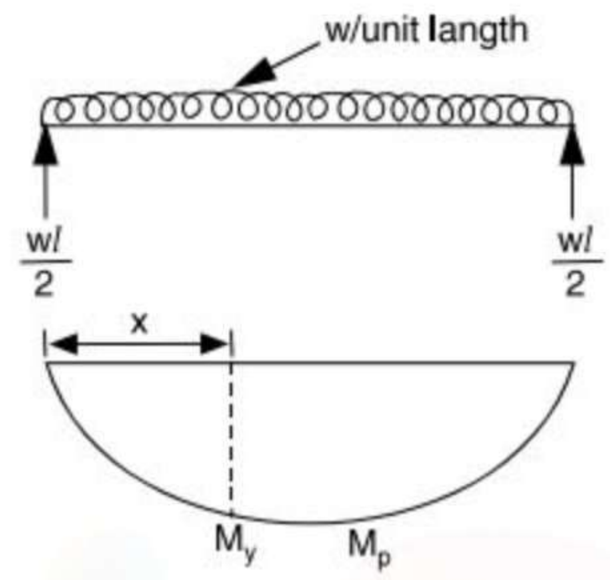
$$A_1 = 0.8\left[7.5 - \frac{0.8}{2} - 1.95\right] = 4.12 \text{ cm}^2$$

$$A_2 = 0.8\left[7.5 - \frac{0.8}{2}\right] = 5.68 \text{ cm}^2$$

$$A_{\text{net}} = 4\left[4.12 + 5.68 \times \frac{3 \times 4.12}{3 \times 4.12 + 5.68}\right]$$

$= 32.04 \text{ cm}^2$

17.



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

$$M_y = \frac{wl}{2}x - \frac{wx^2}{2}$$

$$\frac{M_p}{M_y} = \frac{\frac{wl^2}{8}}{\frac{wlx}{2} - \frac{wx^2}{2}}$$

$$\Rightarrow \quad \frac{l^2/8}{\frac{lx - x^2}{2}} = 1.5$$

$$\Rightarrow \quad \frac{l^2}{lx - x^2} \times \frac{2}{8} = 1.5$$

$\Rightarrow \quad l^2 = 6\,l\,x - 6\,x^2$

$\Rightarrow \quad 6x^2 - 6\,l\,x + l^2 = 0$

$$x = \frac{+6l \pm \sqrt{36\,l^2 - 24\,l^2}}{12}$$

$$= \frac{+6l \pm \sqrt{12\,12}}{12}$$

$x = 0.211\,l$

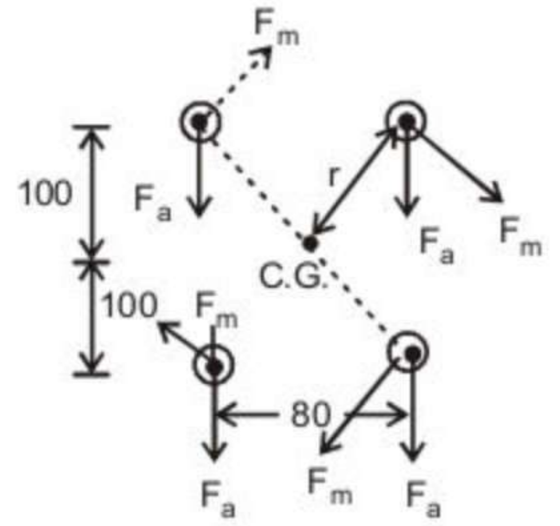
$\therefore$ Length of plastic hinge $= l - 2 \times 0.211\,l$

$= 0.578\,l$

18. $$d = 1.1\sqrt{\frac{M}{f_b \cdot t_w}} = 1.1\sqrt{\frac{150 \times 10^5}{1650 \times 0.8}} = 117.26 \text{ cm}$$

= Sag 117 cm

19.



Force due to axial load on each rivet

$$F_u = \frac{W}{n} = \frac{50}{4} = 12.5 \text{ kN}$$

Force due to moment

$$F_m = \frac{M \times r}{\Sigma r^2} = \frac{50 \times 200 \times \sqrt{100^2 + 40^2}}{4 \times (100^2 + 40^2)}$$

$$= 23.21 \text{ kN}$$

$$\theta = \tan^{-1}\frac{100}{40} = 68.19°$$

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

$$= \sqrt{12.5^2 + 23.21^2 + 2 \times 12.5 \times 23.21^2 \times \cos 68.19}$$

$= 30.17$ kN

20. Greatest twisting moment

$$= (0.7 \times 6 \times 110) \times \pi \times 150 \times \frac{150}{2} \text{ N-mm.}$$

$= 16.33$ kN-m

21. Safe tension $= 150 \times [A_1 + K_2\ A_2]$

Here, $A_1 = 2\left[\left(60 - \frac{8}{2}\right) \times 8 - 17.5 \times 8\right] = 616 \text{ mm}^2$

$$A_2 = 2\left[\left(60 - \frac{8}{2}\right) \times 8\right] = 896 \text{ mm}^2$$

$$\therefore \quad K_2 = \frac{5A_1}{5A_1 + A_2} = 0.77$$

Safe tension $= 150 \times [616 + 0.77 \times 896]$

$= 196.5$ kN

22. $\tau_{vf} = \frac{\pi}{4} \times (26 + 2)^2 \times 150 = 92.36 \text{ kN}$

$$n = \frac{800}{92.36} = 8.66 \approx 9$$

23. Minimum, $Q_p \geq 1.5\,(d_o + 75) = 1.5\,(400 + 75)$

$= 712.5$ mm

24. Lateral thrust on gantry girder

= 10% of (capacity of crane + weight of crab)

$= 0.1 \times 125 = 12.5$ kN

25. $\lambda < 180$

$\Rightarrow \quad \frac{l}{r} \leq 180$

$\Rightarrow \quad l = 180 \times 20 = 3600$ mm

26. $\frac{d}{t_w} < 200$

$\Rightarrow \quad t_w \geq \frac{d}{200} \geq \frac{800}{200} \geq 4$ mm

31. No. of revet required $= \frac{\text{Axial Load}}{\text{Rivet Value}}$

$d_{gross} = 26 + 2 = 28$

R.V. $= 2 \times \frac{\pi}{4} \times (28)^2 \times 100 = 123.15 \times 10^3$

No. of rivet $= \frac{800 \times 10^3}{123.15 \times 10^3} = 6.49 = 7$ rivets.

35. Effective length = 2H

where, H = height of pole = 5 m

38. Load carried by lacing = 2.5 % of axial load

$= 0.025 \times 125 = 3.125$ N

39. Number of plastic hinge

= Number of indeterminacy + Number of collapse

= 2 + 1 = 3

45. Yield stress, $(f_b) = M_P/Z_P$

$= (120 \times 1000 \times 1000)/(5 \times 10^{-4} \times 10^9)$

$= (120 \times 10^2)/5 = 240 \text{ N/mm}^2$

EXERCISE – II

MCQ TYPE QUESTIONS

1. Static method → Upper bound method of ultimate load analysis

Kinematic method → Lower bound on ultimate load

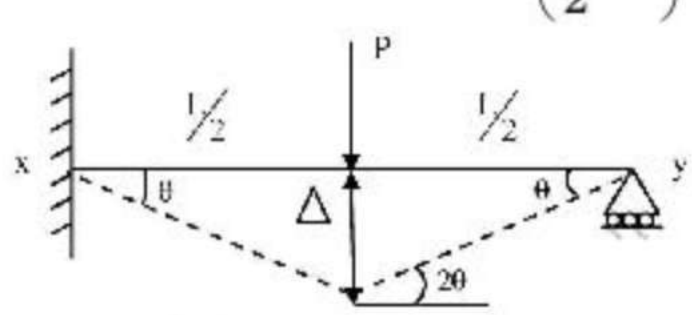
$Q_{Design} = Q_w \times$ load factor

$M_0 = M_u \times \gamma_m$, γ_m = material partial safetyfactor

2. $D_s = 1$

So, no. of plastic hinges $= D_s + 1 = 2$

External work done $= P.\Delta = P.\left(\frac{L}{2}.\,\theta\right)$



Internal work done

$M_p.\theta + M_p.2\theta = 3M_p.\theta$

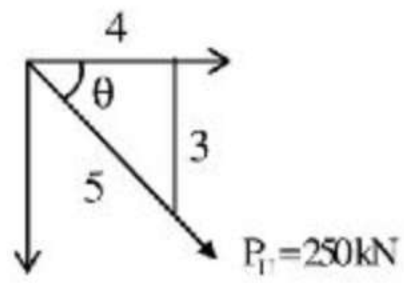
By principal of virtual work $= P.\frac{L}{2}.\theta = 3M_p.\theta$

$$\Rightarrow \quad P = \frac{6M_p}{L}$$

3. As per clause 9.2.1 (IS: 800-2007) for combined shear and bending: Factored value of applied shear force is greater than or equal to shear strength for high shear.

i.e., $V > 0.6V_s$

4. $\sin\theta = \dfrac{3}{5}$, $\cos\theta = \dfrac{4}{5}$



$P_U \cos\theta = \dfrac{4}{5}.P_U$

$P_U \sin\theta = \dfrac{3}{5}.P_U$

$$\text{Tension in each bolt} = \frac{P_U\cos\theta}{6} = \frac{4P_U}{5\times 6}$$

$$= \frac{4}{30}\times 250 = 33.33 \text{ kN}$$

$$\text{Shear in each bolt} = \frac{P_U\sin\theta}{6} = \frac{3}{5}\times\frac{P_U}{6}$$

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

5. According to IS : 800 – 1984

Under Clause 3.7 and in Table 3.1,

Maximum slenderness ratio in M due to live loads = 180

And Maximum slenderness ratio in N due to windload = 350

Hence, λ for M < λ for N.

6. Prying forces are tensile forces due to the flexibility of connected parts.

8. There are two column P and Q

Column 'P'	**Column 'Q'**
Length = L	Length = 2L
Yield stress f_Y = 250 Mpa	f_Y = 250 Mpa
Cross-section = A	Cross-section = A

End condition is same assume that both ends are pissed.

Left = L   Left = 2L

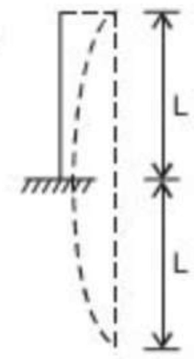
$$P_{\text{buckling}} = \frac{\pi^2 EI}{l_{eff}^2} \qquad Q_{\text{buckling}} = \frac{\pi^2 EI}{l_{eff}^2}$$

$$P_{\text{buckling}} = \frac{\pi^2 EI}{L^2} \qquad Q_{\text{buckling}} = \frac{\pi^2 EI}{(2L)^2}$$

$$\text{Ratio of } \frac{P_{\text{buckling}}}{Q_{\text{buckling}}} = \frac{\dfrac{\pi^2 EI}{L^2}}{\dfrac{\pi^2 EI}{4L^2}} = \frac{1}{1}\times\frac{4}{1}$$

$$\frac{P_{\text{buckling}}}{Q_{\text{buckling}}} = 4$$

12.



Effective length = 2 L

13. We know, $\dfrac{T}{I_P} = \dfrac{G\theta}{L}$

where, I_P = polar moment of inertia = $\dfrac{\pi}{32}d^4$

$$\therefore \qquad \theta = \frac{TL}{I_P G} = \frac{32TL}{\pi d^4 G}$$

14. Gross diameter of rivet, $d = 20 + 1.5 = 21.5$ mm

There are two rivets per pitch length.

Since each rivet is in double sheal, therefore strength of rivet in double shear

$$= 2\times\left(2\times\frac{\pi}{4}\times 21.5^2\times 100\right)$$

$$= 145.220 \text{ kN}$$

Strength of rivet in bearing

$$= 2\times 21.5\times 14\times 300 = 180.6 \text{ kN}$$

$\therefore$ Rivet value = 145.220 kN

Since joint is strengthened by 6 rivets, therefore capacity of the joint = $145.22 \times 6 = 871.32$ kN

Alternately

Strength of rivet in shear,

$$P_S = 2\times\frac{\pi}{4}\times d^2\times\tau$$

$$= 2\times\frac{\pi}{4}\times 21.5^2\times 100 = 72.61 \text{ kN}$$

Strength of joint in bearing,

$$P_b = d.tp_f$$

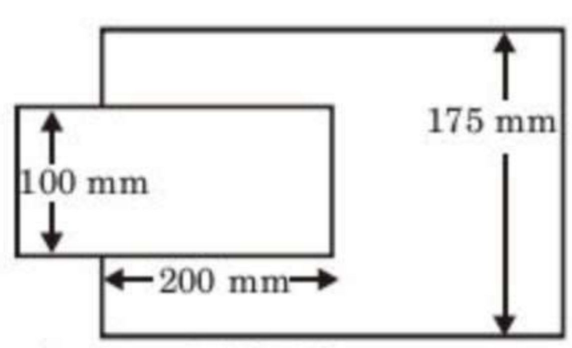
$$= 21.5\times 14\times 300 = 90.3 \text{ kN}$$

$\therefore$ Rivet value, R_v = 72.61 kN

$\therefore$ Capacity of joint = $NR_V = 72.61 \times 12$

$= 871.32$ kN

15.



Let length of the weld is L.

Size of weld = Thickness of thinner plate – 1.5

$= 10 - 1.5 = 8.5$ mm

Throat thickness, $t = 0.7 \times 8.5 = 5.95$ mm

The joint will be designed on the basis of strength of small plate.

$\therefore$ Total load taken by small plate

$$= 150 \times 100 \times 10 = 150 \text{ kN}$$

Allowable load per line

$$= 5.95 \times 110 = 654.5 \text{ N}$$

$$\therefore \quad L = \frac{150 \times 10^3}{654.5} = 229.2 \text{ mm}$$

17. Here, $d = 16 + 1.5 = 17.5$

$$\text{Shear strength} = \frac{\pi}{4} \times d^2 \times 2 \times 90$$

$$= \frac{\pi}{4} \times (17.5)^2 \times 2 \times 90 = 43.29 \text{ kN}$$

Bearing strength $= p \times d \times T$

$$= 270 \times 17.5 \times 12 = 56.7 \text{ kN}$$

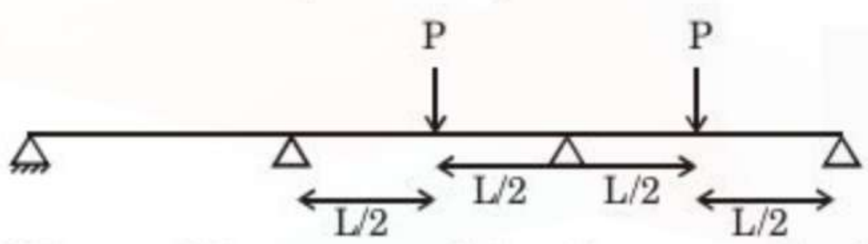
So, minimum strength = 43.29

Rivet value = 43.29 kN

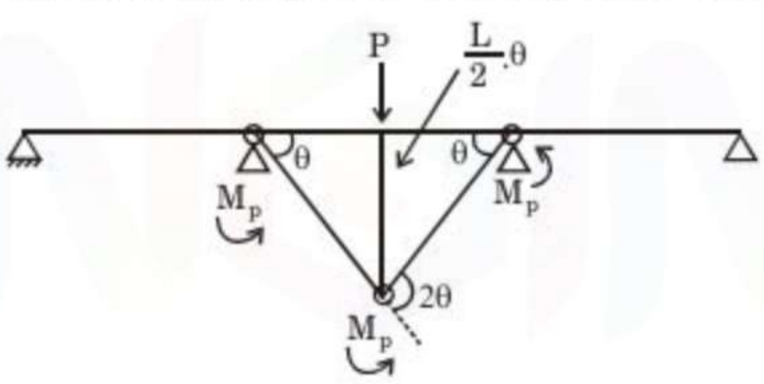
18. Refer IS 800 : 1984 code for General steel construction clause 8.9.4.5

$$\frac{\tau_{vf,cal}}{\tau_{vf}} + \frac{\sigma_{tf,cal}}{\sigma_{tf}} \not> 1.4$$

19.

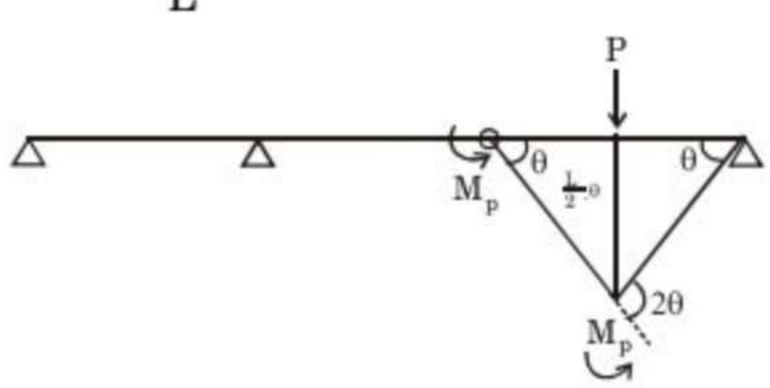


There will be two possible collapse mechanisms



$$\text{Here, } M_p.\theta + M_p.2\theta + M_p\theta = p.\frac{L}{2}.\theta$$

$$\Rightarrow \quad \frac{8M_p}{L} = p$$



$$\text{Here,} \quad M_p.\theta + M_p.2\theta = p.\frac{L}{2}.\theta$$

$$\Rightarrow \quad p = \frac{6M_p}{L}$$

The lower of two callapse values will be the collapse load.

$$\therefore \quad p = \frac{6Mp}{L}$$

20. Size of steel flat = 70 × 6mm

Diameter of bolts, $t = 11.5$ mm

Net minimum area will be at the setion $a\ b\ c\ d\ e$

s = 35,

$g = 20$

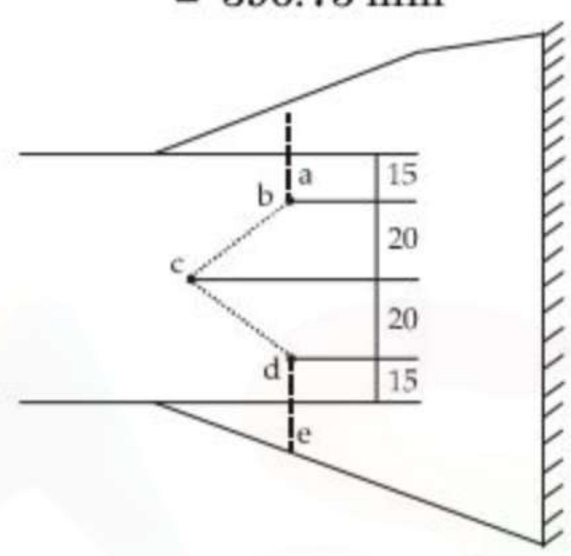
$n = 3$

$m = 2$

$$\text{But} \qquad A_{net} = t\left(L - nd + \frac{ms^2}{4g}\right)$$

$$\therefore \qquad A_{net} = 6\left[70 - 3 \times 11.5 + \frac{2 \times 35^2}{4 \times 20}\right]$$

$$= 396.75 \text{ mm}^2$$



Strength of plate = 150 N/mm^2

Maximum tension in flat, $P = A_{net} \times \sigma_{at}$

$$P = 796.75 \times 150$$

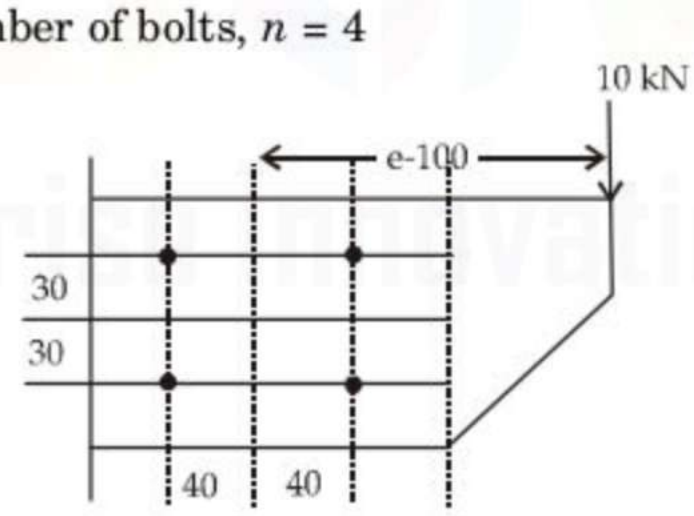
Where, $\sigma_{at} = 150 \text{ N/mm}^2 = 59.5$ kN

21. Diameter of bolt, $d = 10$ mm

Load, P = 10 kN

$e = 100$ mm

Number of bolts, $n = 4$



Bending moment, M = P.e

$$= 10 \times 10^3 \times 100$$

$$= 10 \times 10^5 \text{ N-mm}$$

$$F_1 = \frac{P}{n} = \frac{10 \times 100}{n} = 2.5 \text{ kN}$$

$$F_2 = \frac{M.r}{\sum r^2}$$

$$r = \sqrt{(30)^2 + (40)^2} = 50 \text{ mm}$$

$$\therefore \qquad \sum r^2 = 4 \times (50)^2 = 10,000 \text{ mm}$$

$$\therefore \qquad F_2 = \frac{10 \times 10^5 \times 50}{10,000} = 5000 = 5 \text{ kN}.$$

$$\cos \alpha = \frac{30}{50} = 0.6,$$

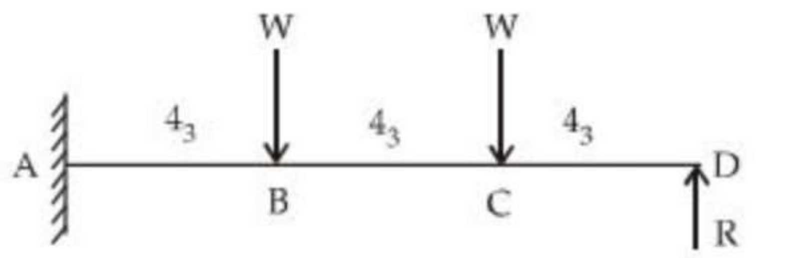
$$\sin \alpha = 0.8$$

Maximum load resisted by bolt,

$$R = \sqrt{(F_1 + F_2 \sin \alpha)^2 + (F_2 \cos \alpha)^2}$$

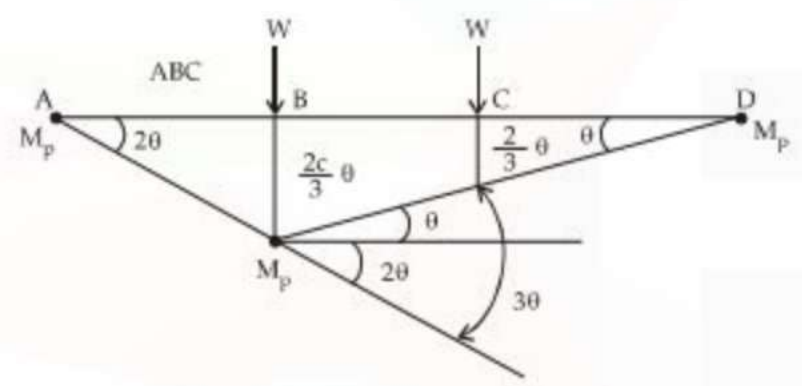
$$= \sqrt{(2.5 + (5 \times 0.81))^2 + (5 \times 0.6)^2} = 7.16 \text{ kN}$$

22.



Collapse mechanism I

Hinge formed at B



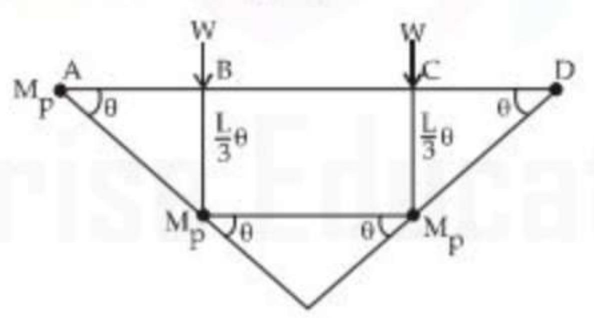
$$W_p \times \frac{2L}{3}\theta + W_p \times \frac{L}{3}\theta = M_p.\, 2\theta + M_p\, 3\theta + M_p\, \theta$$

$$\Rightarrow \qquad W_p.\, L.\, \theta = \Sigma M_p.\, \theta$$

$$\Rightarrow \qquad W_p = \frac{\Sigma M_p}{L}$$

Collapse mechanism II

Hinge formed at A, B, C, D



$$W_p \times \frac{L}{3}\theta + W_p\, \frac{L}{3}\theta = M_P\theta + M_p\theta + M_p\theta + M_p\theta$$

$$\Rightarrow \qquad \frac{2L}{3} W_p\, \theta = 4\, M_p\theta$$

$$\Rightarrow \qquad W_p = \frac{6M_p}{L}$$

25. Compressive force $= \frac{1}{2} \times$ area of triangle $\times \sigma_y$

$$= \frac{1}{2} \times \frac{1}{2} \times b \times h \times \sigma_y = \frac{bh\sigma_y}{4}$$

NUMERICAL TYPE QUESTIONS

1. $$F_D = \frac{P}{n} = \frac{100}{5} = 20\text{kN}$$

$$F_t = \frac{(P.d)r}{\sum r^2} = \frac{100 \times 600 \times 75\sqrt{2}}{4 \times \left(75\sqrt{2}\right)^2} = 141.42 \text{ kN}$$

$$\theta = 45°$$

$$\cos\theta = \frac{1}{\sqrt{2}}$$

$$F_R = \sqrt{F_D^2 + F_t^2 + 2 \times F_D \times F_t \cos\theta}$$

$$F_R = \sqrt{(20)^2 + (141.42)^2 + 2 \times 20 \times 141.42 \frac{1}{\sqrt{2}}}$$

$$= 156.20 \text{ kN}$$

2. First of all find the reaction at support R_T and R_W

$$R_W + R_T = 15 \qquad ...(i)$$

$$\Sigma M_W = 0$$

$$R_T \times 8 = 15 \times 4 + 15 \times 8$$

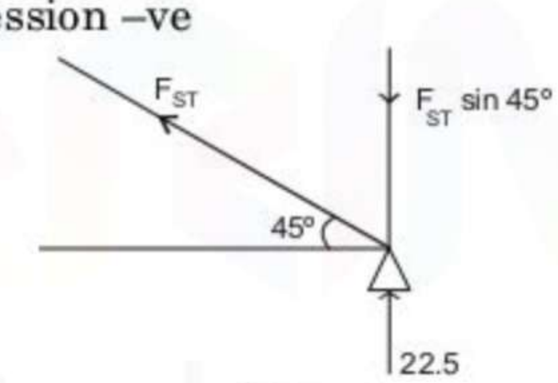
$$R_T = \frac{180}{8} = 22.5 \text{ kN}(\uparrow) \text{upward direction}$$

$$R_W = 15 - 22.5$$

$$= -7.5 \text{ kN } (\downarrow) \text{ downward direction}$$

Consider the joint T

(assume initially tension) tension +ve and compression –ve



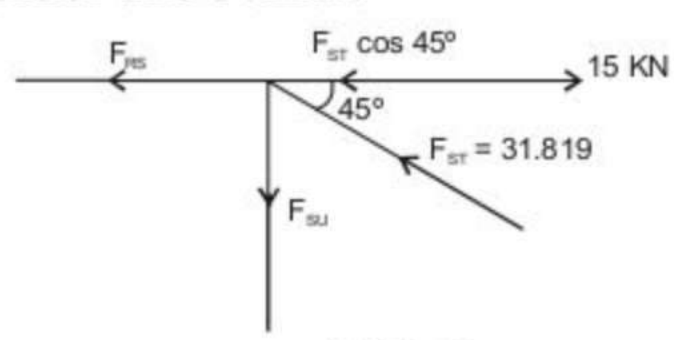
$$\Sigma Y = 0$$

$$F_{ST} \sin 45° + 22.5 = 0$$

$$F_{ST} = \frac{22.5}{\sin 45°} = -31.819 \text{ kN}$$

$$F_{ST} = 31.819 \text{ (comp.)}$$

Consider the Joint S



$$\Sigma X = 0$$

$$F_{RS} + F_{ST} \cos 45° = 15$$

$$F_{RS} + 31.819 \times \cos 45° = 15$$

$$F_{RS} = 15 - 22.5 = -7.5 \text{ (comp.)}$$

$\because$ Force in member RS = 7.5 (comp.)