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

Moment of resistance of the web plate,

$$M_w = \text{Area} \times \text{Average Stress} \times \text{Lever arm}$$

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

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

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.

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

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 1.3 of the gross flange area.

Curtailment of Flange Plates

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

This can be done by two methods;

1. Graphical method; and
2. Anayltical method

Stiffeners

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

Web Stiffeners (as per IS: 800)

1. $\frac{d_1}{t_{tw}} \le 85$; no stiffener is required.
2. $\frac{d_2}{t_w} \le 200$; vertical plates are provided.
3. $\frac{d_2}{t_w} \le 250$; vertical stiffeners and one horizontal stiffener are provided.

Spacing of stiffeners, C $\nless d/3$ and $\ngtr 1.5\,d$, and moment of inertia of the stiffener,

$$I_s \nless \frac{1.5d^3}{C^2} t^3$$

where, d = distance between the flange angles.

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

Bearing Stiffeners

The function of the 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.

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

Effective length = 0.7 × actual length

Longitudinal Stiffeners

In addition to the vertical intermediate stiffeners, longitudinal stiffeners should be provided if t_w is less than $\frac{d}{200}$

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

$= \frac{2}{5}$ of the distance from the compression flange to the neutral axis.

Connection of (Flange Plate to Flange Angles)

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

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

Web Plates and Flange Angles

$$\text{Pitch of rivets} = \frac{n \times R \times I}{F.A_{\bar{y}}}$$

where R = rivet value

$A_{\bar{y}}$ = moment of the net area of the flange and flange plates about neurtal axis

Intermediate Stiffeners to Web

The stiffener connection to web plate should be designed to develop a shearing force in tonnes per cm run of not less than $\frac{t_w^2}{2h}$

where t_w = thickness of web plate in cm

h = outstanding leg width of stiffener

Bearing Stiffener to Web

The connection should be designed to transmit the entire concentrated load to the web.

Splices

There are of three types :

(*i*) web splice;

(*ii*) flange plate splice; and

(*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 perforce to be in two or more lengths.

For riveted girders, the 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.

The plates 'A' called moment plates to resist the web's share of the bending moment and the plates 'B' called shear plates to resist the shear are used in this type.

Size of moment plates (A) is decided on the basis of number of rivets to be accomodated.

Size of the shear plates (B) can be determined on the basis of shear area required.

Design of Plate 'A'

$$\text{Moment resisted by web} = \frac{1}{8} A_w f \,.\, h$$

$$\text{Moment of resistance of plates} = F \,.\, h_1$$

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

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

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

where f_1 = allowable stress in bending for the plates

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

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

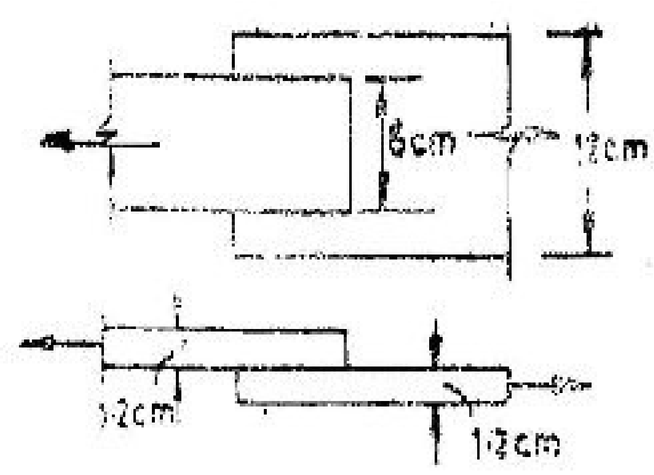
Total number of rivets required on each side of the splice to connect each pair of plates

$$A = \frac{f}{r_1}$$

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

R = least rivet value

Example : Two plates are proposed to be joined by welding as shown in the figure. Find the length of the weld required to develop the full strength of the smaller plates which is 8 cm × 1.2 cm. [Assume the permissible tension in plate as 150 N/mm_2 and the permissible shear stress in fillet weld as 102.5 N/mm^2



Ans. Strength of 80 mm × 120 mm. plate

= 80 × 12 × 150

= 1440 N

= 144 kN

Maximum 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 mm length of the weld

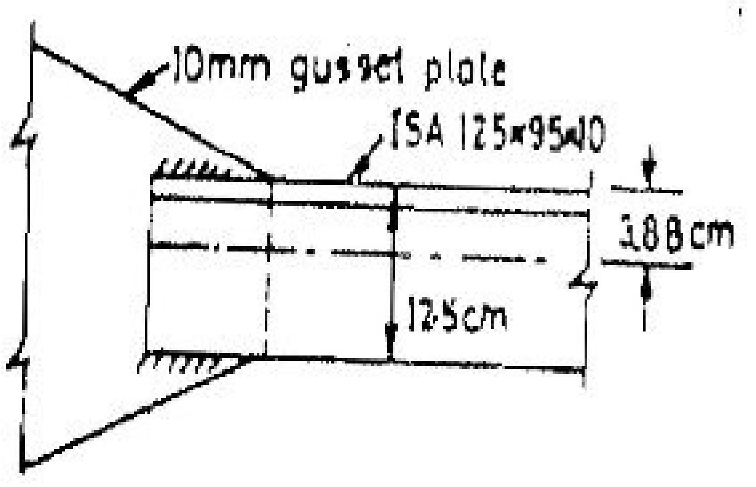
= 10 × 7 × 6 × 102.5

= 43050 N

$$\text{Necessary length of the weld} = \frac{14400}{4305}$$

= 335 mm

Example : An ISA 125 × 95 × 10mm thick steel angle tension member is to be connected by its long leg to a 10 mm thich gusset plate as shown in the figure. In the joint use c mm fillet weld on toe and back and none on the end. Assume the permissible shear stress for weld metal as 1125 kg/cm². The axial pull in the joint



Ans: Net effective area of single angle iron in tension connected by one leg only

$$= a + bk$$

where, a = net sectional area of connected leg

= 125 × 10

= 1250 m²

= 12.50 cm²

b = area of the outstanding leg

= (95 – 19) 10

= 850 mm²

= 8.50 cm²

$$k = \frac{1}{1+0.35\frac{b}{a}}$$

$$= \frac{1}{1+0.35\times\frac{8.50}{12.50}}$$

$$= 850 \text{ cm}^2$$

∴ Net effective area = 12.50 + 8.50 × 0.808

= 19.368 cm²

and Axial pull = 19.368 × 1500

= 29052 kg ≈ 29 T

DESIGN OF ROOF TRUSSES

The truss is a framework in which the members are connected at their ends. The pitch of the roof truss is the ratio of the height of the truss to the span. Common spacing of trusses ranges from 3 to 5 m. Economical spacing varies from $\frac{1}{3}$ to $\frac{1}{5}$ of span. Purlines are spaced such that they are at each node of the truss to avoid bending in the main rafter of the truss.

LOADS

Dead Loads

(*a*) **Sheeting :** GI sheeting–15 kg/m²

AC sheeting–18 kg/m²

(*b*) **Purlins:** Self-weight of purlins with corrugated sheets varies from 6 to 9 kg/m² area covered by purlin.

(*c*) **Truees:** Weight of truss $\left(\frac{L}{3}+5\right)$ kg/m²

where L = span of truss in metres.

Live Loads : For sloping roofs of greater than 10° slop, the live load is 75 kg/m² less 1 kg/m² for every degree increase in slope up to 20°.

Snow Loads: No snow load is considered if slope is greater than 50°. For other slopes 2.5 kg/m² per cm depth of snow may be taken. .

Wind Loads: External wind pressure plus internal air pressure.

Design of Purlins: Depth of purlin < $\frac{L}{45}$, width of purlin < $\frac{L}{60}$,

where L is the spacing of trusses

$$\text{Maximum B.M. in purlin} = \frac{WL}{10}$$

where W = total distributed load on the purlin due to all loads.

Design of Tension Member

1. Double angle sections shall be used for main tie and single angle sections for other members.
2. Minimum size of the angle for main tie is ISA 50 × 50 × 6 mm.
3. For other members, the minimum size is ISA 50 × 50 × 5 mm.
4. Minimum size of gusset plate = 6 mm.
5. Minimum size of rivets is 16 mm.
6. Maximum permissible slenderness ratio of tension members is 350.

(When subjected to reversal of stress)

Design of Compression Members

1. Double angle sections are used for main rafter.
2. Minimum size of the angle = 50 × 50 × 6 mm.
3. Minimum thickness of gusset plate = 6 mm.
4. Minimum number of rivets at ends = 2
5. Maximum slenderness ratio = 180.
6. Effective length of member = 0.7 to 1.0 of the actual length.

Example : Given the purlins of a roof truss of span = 9.0 m. The pitch of truss is L/5. The height of truss at eaves level is 15 m. The spacing of trusses is 4 m. The roof covering will consist ofasbestos sheets. What is the required Z ?

Ans. Given,

$$\text{Span} = 9 \text{ m},$$

$$\text{Pitch} = \frac{L}{5} 1.8 \text{ m},$$

Spacing of roof trusses = 4 m

Slope of the truss is given by

$$\tan\theta = \frac{1.8}{4} = 0.45$$

$$\therefore \quad \theta = 24' \; 13'$$

Maximum slope required for AC sheeting = 20°

Maximum permissible spacing for purlins = 1.68 m

Loads on Purlin

Area covered by each intermediate purlin

$$= 1.68 \times 4 = 6.72 \text{ m}^2$$

Dead Loads

(*i*) Self-weight of AC sheeting per purlin/m at 18 kg/m²

$$= 18 \times 1.68 \text{ m} = 30.2 \text{ kg}$$

(*ii*) Self-weight of purlin at 10 kg/m = 10 kg

$$\therefore \quad \text{Total dead load} = 40.2 \text{ kg}$$

Live loads

$$\text{Slope of the truss} = 24° \; 13'$$

$$\text{Live load on the truss} = 75 - (10 \times 1 + 4.13 \times 2) = 56.74 \text{ kg/m}^2$$

$$\text{Live load per purlin/in} = 56.74 \times 1.68 \cos 24° \; 13' = 86.93 \text{ kg}$$

$$\therefore \quad \text{Design load} = 127.13 \text{ kg/m}$$

Bending Moment : For a continuous purlin,

$$BM = \frac{WL}{10} = \frac{127.13 \times 4^2}{10} = 203.40 \text{ kg/m}$$

$$Z_{\text{required}} = \frac{20340 \times 100}{1650} = 12.3 \text{ cm}^2$$

BUILT UP GIRDERS

1. DESIGN OF PLATE GIRDER

$$\text{Impact factor,} \quad i = \left[0.15 + \frac{8}{6+L}\right] \not> 1.0$$

where, L = span of the girder

Economic depth of web plate

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

Also

$$d = k \cdot \left(\frac{M}{\sigma_{bc}}\right)^{\frac{1}{3}}$$

where, M = maximum bending moment

t_w = thickness of web plate

σ_{bc} = permissible bending strength

k = constant may be = 5 for welded plate girder = 4.5 for riveted girder

Shear at plate,

$$\tau = \frac{V}{I_b}(A\bar{y})$$

$$\tau_{av} = \frac{V}{d.t_w}$$

2. GANTRY GIRDER DESIGN PROCEDURE FOR GANTRY GIRDERS

1. Find out loads.

$$W_1 = \frac{W_c}{4}$$

where W_c = total weight of crane

$$W_2 = \frac{W_t(B-a)}{2B}$$

where B = distance between gantry girders

Then Total load $W = W_1 + W_2$

where a = newest distance.

2. Find Vertical Bending moment.

$$\text{Assume self weight as } w_1 = \frac{2W}{250} \text{kN/m.}$$

$$\text{self weight of rail, } w_2 = 0.3 \text{ kN/m.}$$

$$\therefore \quad \text{Bending Moment, } M_x = M_{x1} + \frac{wL^2}{8}$$

Where M_{x1} = bending moment due to two wheel loads of load W each.

2. Adopt $\sigma_{bt} = 0.66 f_y$

$$\text{Determine } Z = \frac{M_x}{\sigma_{bt}}$$

Choose a built up section whose Z is about 40% more than the calculated.

4. Determine I_n N.A. and σ_{bc}.
5. Check for Bending compressive stress gue to vertical load.
6. Determine compressive stress due to horizontal bending.
7. Check for maximum, bending compressive stress.
8. Check for shear stress.
9. Design the riveted or welded joint.

SOLVED EXAMPLE

1. A plate girder is composed of the following elements:

(*i*) Web plate: 900 mm depth × 10 mm thickness

(*ii*) Two angles: ISA 200 mm × 100 mm × 12.0 mm @ 27.2 kg/m, in each flange

(*iii*) Two flange plates: 500 mm × 16 mm in each flange.

The girder is simply supported over an effective span of 12 m. The diameter of rivets used for connecting flange angles to the web and flange plates to flange angles is 20 mm.

Determine the safe uniformly distributed load which the girder can carry, inclusive of its own weight. Assume that the compression flange is not restrained against lateral bending, but the ends are restrained against torsion. Take $f_y = 250$ N/mm²,

Solution.

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

$$= \frac{0.66 \times 372.8 \times 250}{\left[(372.8)^{1.4} + (250)^{1.4}\right]^{\frac{1}{1.4}}}$$

$$= 119.47 \text{ N/mm}^2$$

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

$$= 0.66 \times 250 = 165 \text{ N/mm}^2.$$

Moment of resistance,

$$M = \frac{\sigma_{bc} . I_{xx}}{\frac{D}{2}}$$

$$= \frac{119.47 \times 1011273 \times 10^4}{\frac{964}{2}}$$

$$= 2506 \times 10^6 \text{ N-mm}$$

$$= 2506 \text{ kN-m.}$$

$$\therefore \quad w = \frac{8M}{l^2}$$

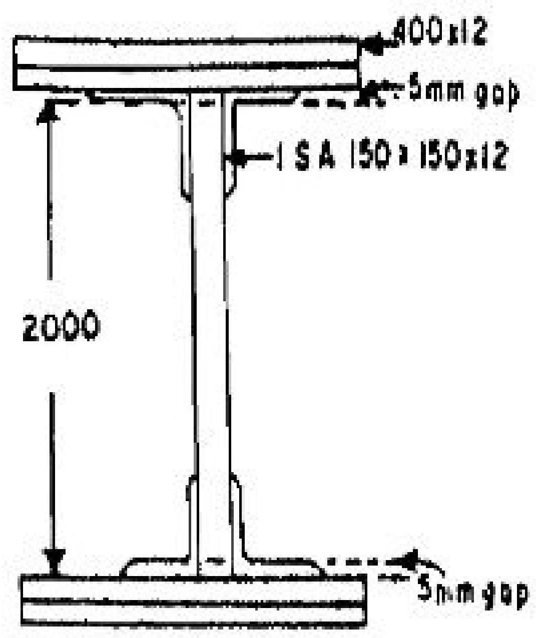
$$= \frac{8 \times 2506}{(12)^2}$$

$$= 139.2 \text{ kN/m.}$$

2. Following are the details of a riveted plate girder web = 2000 × 10 mm; ISA 150 × 150 × 12 mm, two on each flange; plates 4.00 × 12 mm, two on each flange. Bending moment at a particular section = 4400 kNm; shear force at section =135 kN properties of ISA 150 × 150 × 12 mm.

Sectional area = 34.77 cm²; $I_{xx} = I_{yy} = 746.3$ cm⁴; $C_{xx} = C_{yy} = 41.6$ mm. Permissible stresses in rivets: in shear = 100 MPa; in bearing = 300 MPa. Design a suitable web splice for the plate girder using splice plates 350mm wide and 16 mm thick which are available.

Solution.



$$I = \frac{1}{12} \times 1.0 \times 200^3 + 746.3 \times 4$$

$$+ 4 \times 34.77\,(96.34)^2 + 2 \times 40 \times \frac{2.4^3}{12}$$

$$+ 2 \times 40 \times 2.4 \times (100.5 + 1.2)^2$$

$$= 3946435.4 \text{ cm}^4$$

$$I_{web} = \frac{1}{12} \times 1.0 \times 200^3 = 666666.7 \text{ cm}^4$$

Moment resisted by web

$$= \frac{MI_w}{I}$$

$$= \frac{4400 \times 666666.7}{3946435.7}$$

$$= 743.01 \text{ kN}$$

Depth of splice plate,

$$d_s = 200 - 15 - 15 = 170 \text{ cm}$$

$$= 1700 \text{ mm}$$

Thickness of each splice plate

$$t = \frac{t_w d_w^2}{2 d_s^2}$$

$$= \frac{10 \times 2000^2}{2 \times 1700^2} = 6.92 \text{ mm}$$

Hence, thickness of web plate of 16 mm is suficient

Using 20-mm diameter rivets at a pitch $p = 80$ m c/c in two rows.

$$\text{Number of rivets, } n = \sqrt{\frac{6M_w}{m \times p \times R}}$$

Rivet value (R)

$$\text{In double shear} = \frac{2\times 100}{1000}\times\frac{\pi}{4}\times(2.15)^2$$
$$= 72.61 \text{ kN}$$
$$\text{In bearing over web} = \frac{300}{100}\times 21.5\times 10$$
$$= 64.5 \text{ kN}$$
$$\therefore \quad \text{Rivet value, R} = 64.5 \text{ kN}$$
$$n = \sqrt{\frac{6\times 743.01\times 10^6}{6\times 80\times 64.5\times 10^3}}$$
$$= 20.784$$

Provide 21.20 mm dia rivets at a pitch 80 mm c/c and edge distance of 47.5 mm.

Check for stresses

Moment about C.G of rivet-group

$$= \text{M}_w + \frac{\text{V}(47.5+40)}{1000}$$
$$= 743.01 + 135\times\frac{87.5}{1000}$$
$$= 754.82 \text{ kNm}$$

Force on extreme right top rivet

$$\text{F}_m = \frac{\text{M}\times r}{\sum r^2}$$

where, $r = \sqrt{800^2+40^2} = 801$ mm

$$\sum x^2 = 21\times 2\times 40^2 = 67200$$
$$\sum y^2 = 4[80^2+160^2+240^2+320^2+400^2 + 480^2+560^2+640^2+720^2+800^2]$$
$$= 9856000$$
$$\sum r^2 = \sum x^2 + \sum y^2$$
$$= 9923200 \text{ mm}^2$$
$$\therefore \quad \text{F}_m = \frac{754.82\times 10^6\times 801}{9923200}$$
$$= 60929 \text{ N}$$

Force due to direct shear

$$\text{F}_a = \frac{135\times 10^3}{42}$$
$$= 3214.28 \text{ kN}$$

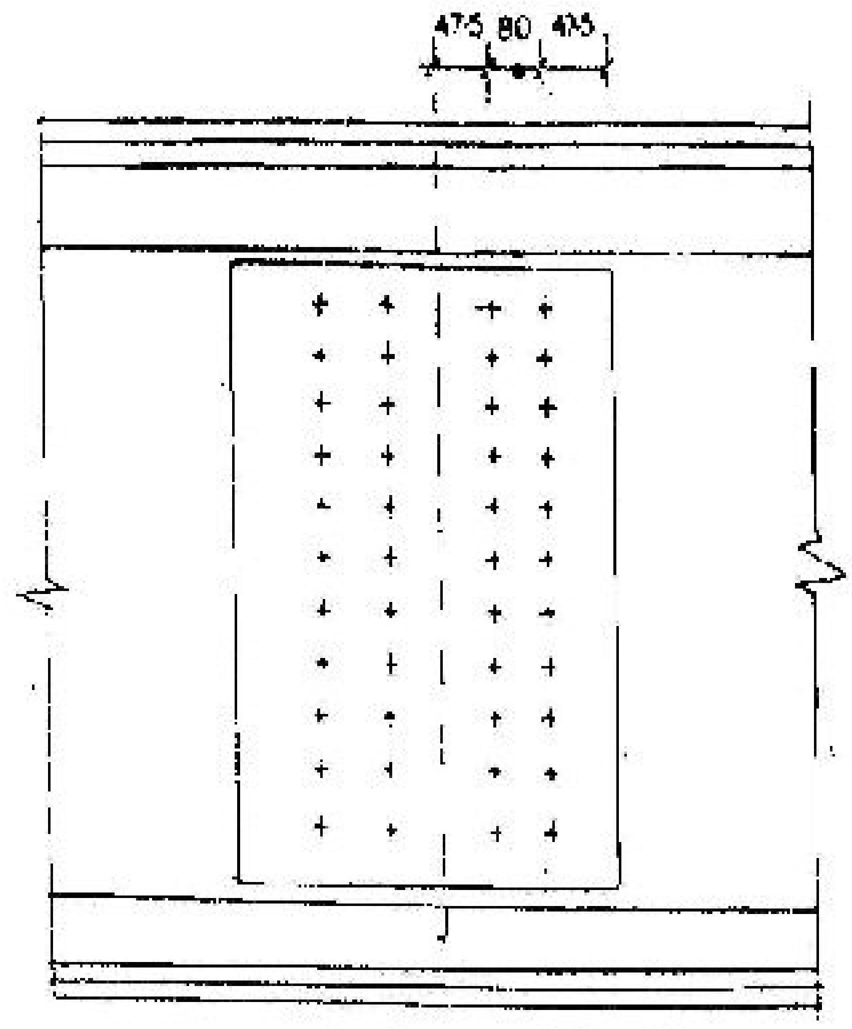
If 'θ' is angle between F_a and F_m then

$$\cos\theta = \frac{40}{801} = 0.05$$

Resultant force, F_r =

$$\sqrt{3214.28^2+60929^2+2\times 60929\times 3214.28\times 0.05}$$
$$= 61.174 \text{ kN} \not> 64.5 \text{ kN}$$

Hence safe



3. Design a gantry girder for a mill building to carry an electric overhead travelling crane, having the following data

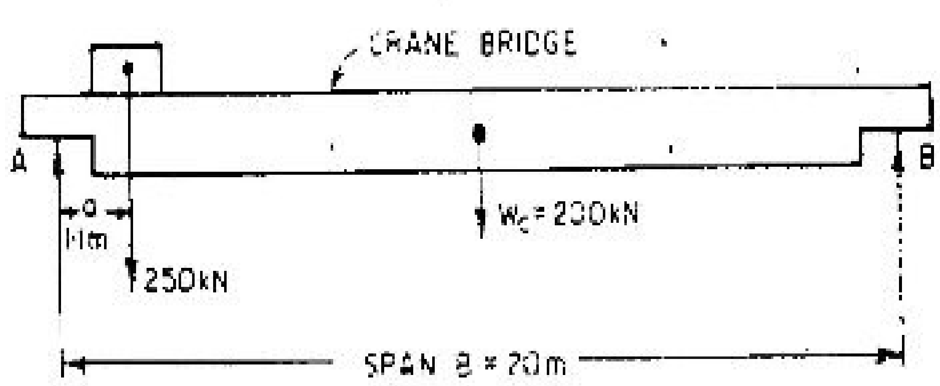
1.	Crane capacity	= 250 kN
2.	Weight of crane excluding crab	= 200 kN
3.	Weight of crab	= 60 kN
4.	Span of crane between rails	= 20 m
5.	Minimum hook approach	= 1.1 m
6.	Wheel base	= 3.4 m
7.	Span of gantry girder	= 7 m
8.	Mass of rail section	= 30 kg/m.
9.	Height of rail section	= 75 mm

Take f_y = 250 N/mm² and E = 2 × 10⁵ N/mm².

Solution.

Step 1. Determination of loads

$$\text{W}_c = 200 \text{ kN}$$
$$\text{W}_t = 60 + 250 = 3.10 \text{ kN}.$$



Maximum static wheel load at A will occur when the crane hook is at a minimum distance of a = 1.1 m.

$$\text{R}_A = \frac{\text{W}_c}{2} + \frac{\text{W}_t(\text{B}-a)}{\text{B}}$$
$$= \frac{200}{2} + \frac{310(20-1.1)}{20}$$
$$= 392.95 \text{ kN}.$$

Since there are two wheel loads at A,

$$W = \frac{1}{2} \times 392.95$$

$$= 196.475 \text{ kN}.$$

Alternatively,

$$W = W_1 + W_2$$

$$= \frac{W_c}{4} + \frac{W_t(B-a)}{2B}$$

$$= \frac{200}{4} + \frac{310(20-1.1)}{2 \times 20}$$

$$= 196.475 \text{ kN}$$

Step 2. Determination of B.M. due to vertical B.M.

$$\text{Static wheel load} = 196.475 \text{ kN}$$

$$\text{Add impact allowance } 25\% = \underline{49.119 \text{ kN}}$$

$$\text{Total W} = 245.594 \text{ kN}$$

$$\text{Wheel base, } b = 3.4 \text{ m}$$

$$\text{span of girder} = L = 7 \text{ m}$$

$$0.586\,L = 0.586 \times 7$$

$$= 4.102 \text{ m}.$$

Since $b < 0.586$ L maximum bending moment will occur when centre of span is midway between C.G. of loads and one wheel load.

Hence distance of one wheel, from centre of span

$$= \frac{1}{4} b$$

$$= \frac{1}{4} \times 3.4 = 0.85 \text{ m}.$$

Let self weight of girder,

$$w_1 = \frac{2W}{250} = \frac{2 \times 245.594}{250}$$

$$\approx 1.96 \text{ kN/m}$$

$$\text{Weight of rail} = w_2$$

$$= 30 \times 9.81 \times 10^{-3}$$

$$= 0.29 \text{ kN/m}.$$

$$\therefore \quad w = 1.96 + 0.29 = 2.25 \text{ kN/m}$$

The positions of wheel loads for maximum bending moment are shown in the figure below.

Maximum B.M. will occur under wheel load (F) which is nearer to the centre of span.

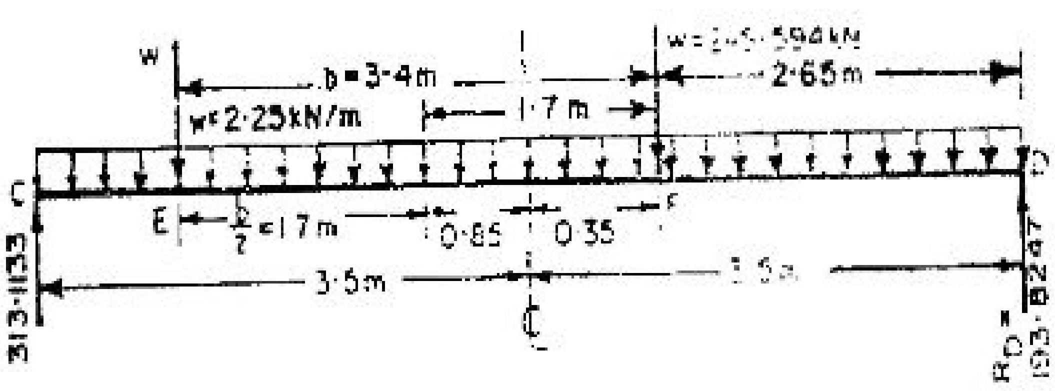


Fig. Vertical Loads For Maximum Bending Moment.

$$R_C = \frac{1}{7}\left(\left(2.25 \times 7 \times \frac{7}{2}\right) + 245.594(2.65 + 6.05)\right)$$

$$= 313.1133$$

Check : Total $= 506.938$

$$M_X \text{ (at F)} = (193.8247 \times 2.65) - \frac{2.25(2.65)^2}{2}$$

$$= 505.735 \text{ kN-m}.$$

Step 3. Selection of section for gantry girder

Take

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

$$= 0.66 \times 250 = 165 \text{ N/mm}^2$$

$$\therefore \quad Z = \frac{M_X}{\sigma_{bt}}$$

$$= \frac{505.735 \times 10^6}{165} = 3065.1 \times 10^3$$

Trial

$$Z = 1.15 \times 3065.1 \times 10^3$$

$$= 3525 \times 10^3 \text{ mm}^3$$

Try ISWB 600 @ 133.7 kg/m as the basic beam having the following properties (Fig. a) below.

$$Z_{xx} = 3540.0 \times 10^3 \text{ mm}^3$$

$$I_{xx} = 106198.5 \times 10^4 \text{ mm}^4$$

$$I_{yy} = 4702.5 \times 10^4 \text{ mm}^3$$

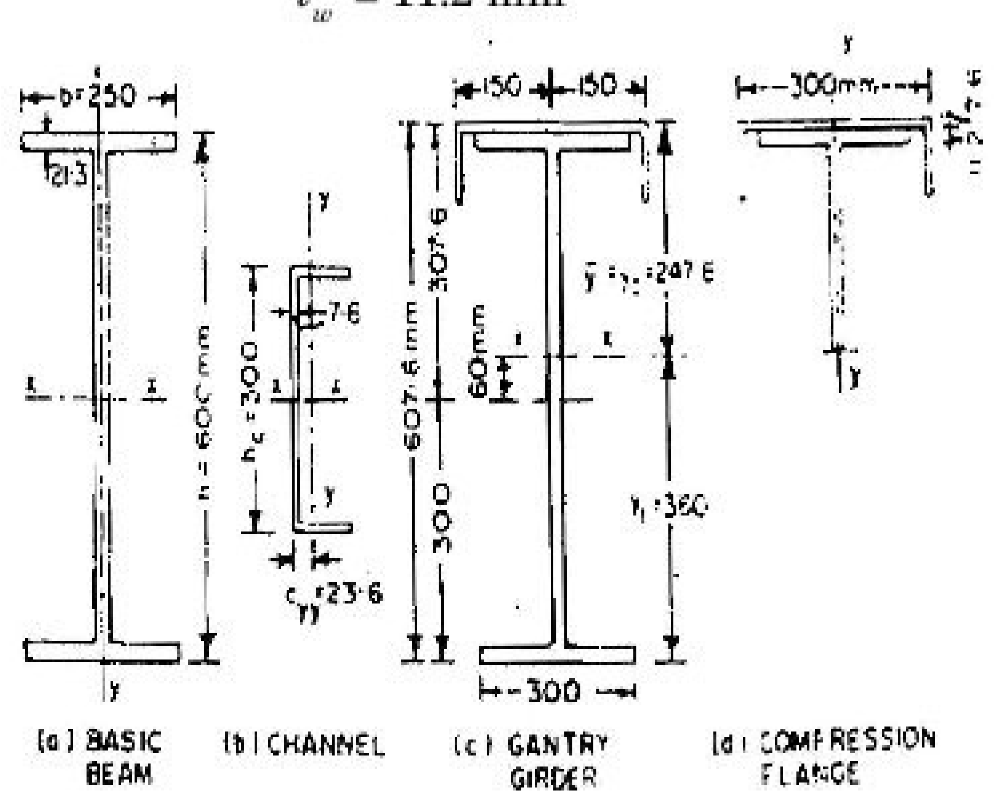
$$a_b = 17038 \text{ mm}^2$$

$$h = 600 \text{ mm}$$

$$b = 250 \text{ mm}$$

$$t_f = 21.3 \text{ mm}$$

$$t_w = 11.2 \text{ mm}$$



Provide ISMC 300 @ 35.8 kg/m on the top flange of the beam, having the following properties (Fig. *b* below).

$$(I_{yy})_c = 310.8 \times 10^4 \text{ mm}^3$$

$$(I_{xx})_c = 6362.6 \times 10^4 \text{ mm}^4$$

$$a_c = 4564 \text{ mm}^2$$

$$C_{yy} = 23.6 \text{ mm}$$

$$t_{wc} = 7.6 \text{ mm}$$

Step 4. Determination of N.A. and I_{xx}

The composite section is shown in Fig. (*c*).

The N.A. (*i.e.* $x-x$ axis) is situated at $\bar{y}$ below the top flange,

$$\bar{y} = \frac{a_c C_{yy} + a_b\left(\frac{h}{2} + t_{wc}\right)}{a_c + a_b}$$

$$= \frac{(4564 \times 23.6) + 17038(300 + 7.6)}{4564 + 17038}$$

or $\quad \bar{y} = y_c = 247.6$ mm

$\therefore \quad y_t = (h + t_{wc}) - y_c$

$= 600 + 7.6 - 247.6 = 360$ mm

$I_X = [106198.5 \times 10^4 + 17038$
$(307.6 - 247.6)^2] + [310.8 \times 10^4$
$+ 4564\,(247.6 - 23.6)^2]$

$= 135543 \times 10^4$ mm^4

$$(\sigma_{bt,cal})_v = \frac{M_X}{I_X} \cdot y_t = \frac{505.735 \times 10^6}{135543 \times 10^4} \times 360$$

$= 134.32$ N/mm^2

Permissible $\sigma_{bt} = 0.66 f_y$

$= 0.66 \times 250 = 165$ N/mm^2.

Hence safe.

Step 5. Determination of σ_{bc}

I_Y for the whole section $= (4702.5 \times 10^4) + 6362.6 \times 10^4$

$= 11065.1 \times 10^4$

Moment of inertia of compression flange about $y-y$ axis is

$$I_{YCF} = \left(\frac{1}{2} \times 4702.5 \times 10^4\right) + 6362.6 \times 10^4$$

$= 8713.85 \times 10^4$ mm^4

$$\omega = \frac{I_{YCF}}{I_Y} = \frac{8713.85 \times 10^4}{11065.1 \times 10^4} = 0.788$$

$k_2 = 0.288$

$$r_Y = \sqrt{\frac{I_Y}{A}} = \sqrt{\frac{11065.1 \times 10^4}{17038 + 4564}}$$

$= 71.57$ mm

Assuming the ends of the girder to be restrained against torsion (at bracket support) but no lateral restraint.

$l = L = 7$ m

$$\therefore \quad \lambda = \frac{l}{r_Y} = \frac{7000}{71.57} = 97.8$$

Now, $\quad c_1 = y_c = 247.6$ mm

and $\quad c_2 = y_t = 360$

$\Psi = 1$ and hence $k_1 = 1$

Now,
$$Y = \frac{26.5 \times 10^5}{\left(\frac{l}{r_Y}\right)^2} = \frac{26.5 \times 10^2}{(97.8)^2}$$

$= 277.06$ N/mm^2

D = overall depth of the section

$= 607.6$ mm.

T = mean thickness of flange .

$$= \frac{(250 \times 21.3) + (300 \times 7.6)}{300} = 25.35 \text{ mm}$$

Now
$$X = Y\sqrt{1 + \frac{1}{20}\left(\frac{1}{r_y} \times \frac{T}{D}\right)^2}$$

$$= 277.06\sqrt{1 + \frac{1}{20}\left(97.8 \times \frac{23.35}{607.6}\right)^2}$$

$= 375.05$ N/mm^2

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

$$= 1(375.05 + 0.288 \times 277.06)\frac{360}{247.6}$$

$= 661.32$ N/mm^2

Now
$$\frac{T}{t_w} = \frac{25.35}{11.2} = 2.26 > 2.0$$

Hence f_{bc} cannot be increased by 20°.

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

$$= \frac{0.66 \times 661.32 \times 250}{\left[(661.32)^{14} + (250)^{1.1}\right]^{\frac{1}{1.4}}}$$

$= 140.2$ N/mm^2

Step 6. Check for bending compressive stress due to vertical load

$$(\sigma_{bc.cal})_v = \frac{M_X}{I_X} y_c = \frac{505.735 \times 10^6}{135543 \times 10^4} \times 247.6$$

$= 92.38$ N. /mm$^2 < \sigma_{bc}$.

Step 7. Determination of B.M. due to horizontal loading

$$F_H = \frac{1}{10}[60 + 250] = 31 \text{ kN}$$

$$W_H = \frac{F_H}{2} = \frac{31}{2} = 15.5 \text{ kN}$$

The arrangement of wheel loads for max. B.M. due to horizontal loads will be the same as that for vertical loads.

$$R_{HD} = \frac{1}{7}[15.5\,(0.95 + 4.35)]$$
$$= 11.736 \text{ kN}$$
$$\therefore \quad M_Y \text{ at F} = 11.736 \times 2.65$$
$$= 31.1 \text{ kN-m}$$
$$= 31.1 \times 10^6 \text{ N-mm}$$

Step 8. Determination of compressive stress due to horizontal loading

$$I_{YCF} = 8713.85 \times 10^4 \text{ mm}^4,$$

as found earlier.

$$(\sigma_{bc.cal}) = \frac{M_Y}{I_{YCF}} \times \frac{h_c}{2}$$
$$= \frac{31.1 \times 10^6}{8713.85 \times 10^4} \times \frac{300}{2}$$
$$= 53.53 \text{ N/mm}^2$$

Step 9. Check for maximum bending compressive stress

$$(\sigma_{bc.cal})_V + (\sigma_{bc.cal})_H = 92.38 + 53.53$$
$$= 145.91 \text{ N/mm}^2$$
$$\text{Permissible value} = 1.1\,\sigma_{bc}$$
$$= 1.1 \times 140.2$$
$$= 154.22 \text{ N/mm}^2$$

Hence safe.

Step 10. Determination of stresses due to longitudinal force

$$F_{LH} = 5\% \text{ of total wheel loads} = 55 \text{ of } 2W$$
$$= 0.05 \times 2 \times 245.594 = 24.56 \text{ kN}$$
$$M_{LH} = F_{LH}\,(h_r + y_c) = 24.56\,(75 + 247.6)$$
$$= 7923 \text{ kN-mm} = 7.923 \times 10^6 \text{ N-mm}$$

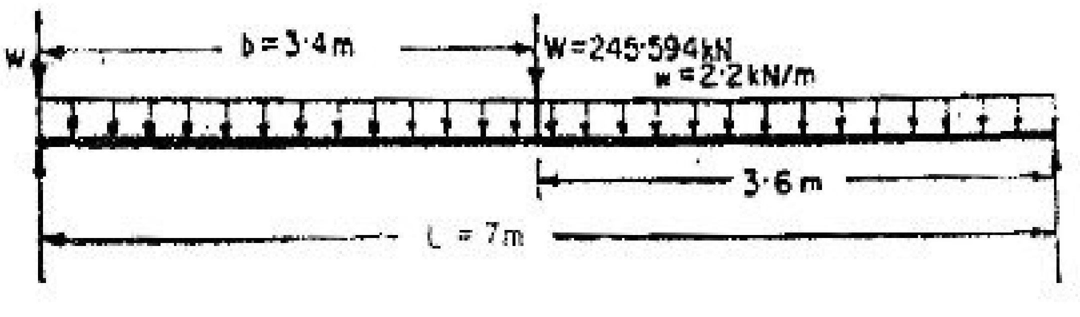
$\therefore$ Stress in longitudinal direction

$$= \frac{F_{LH}}{A} + \frac{M_{LH}}{I_X} y_c$$
$$= \frac{24.56 \times 10^3}{4564 + 17038} + \frac{7.923 \times 10^6}{135543 \times 10^4} \times 247.6$$
$$= 2.58 \text{ N/mm}^2$$

which is negligibly small.

Step 11. Check for shear

Maximum live load shear (V_1) will occur when one of the wheel loads is on the support, as shown in the figure below.



$$R_C = V = 245.594 + \frac{245.594 \times 3.6}{7} + \frac{2.25 \times 7}{2}$$
$$= 397.77 \text{ kN}$$
$$\therefore \quad \tau_{va,cal} = \frac{379.77 \times 10^3}{600 \times 11.2}$$
$$= 56.51 \text{ N/mm}^2$$
$$\text{Allowable} \quad \tau_{va} = 0.4\,f_y$$
$$= 0.4 \times 250$$
$$= 100 \text{ N/mm}^2$$

Hence safe.

Step 12. Design of riveted joint

$$q = \frac{V}{I_X}(A_{fc}\,.\,\bar{y}_1)$$

where A_{fc} = area of cross section of flange of channel, located above the plane of riveted joint

$$= 300 \times 7.6 = 2280 \text{ mm}^2$$

$\bar{y}_1$ = distance of flange of channel from N.A.

$$= y_c - \frac{t_{uc}}{2} = 247.6 - \frac{7.6}{2} = 243.8 \text{ mm.}$$
$$\therefore \quad q = \frac{379.77 \times 10^3}{135543 \times 10^4}(2280 \times 243.8)$$
$$= 155.74 \text{ N/mm.}$$

Hence use 20 mm diameter power driven rivets.

Strength of rivet in single shear

$$= 100 \times \frac{\pi}{4}(21.5)^2 \times 10^{-3}$$
$$= 36.3 \text{ kN}$$

Strength of rivet in bearing

$$= 300 \times 21.5 \times 7.6 \times 10^{-3}$$
$$= 49.02 \text{ kN}$$
$$\therefore \quad \text{Rivet value } R = 36.3 \text{ kN}$$
$$\therefore \quad \text{pitch } p = \frac{2R}{q} = \frac{2 \times 36.3 \times 10^3}{155.74}$$
$$= 466.2 \text{ mm.}$$
$$\text{Maximum permissible spacing} = 12 \times 7.6$$
$$= 91.2 \text{ mm.}$$

Hence provide 20 mm diameter rivets, in two rows, at a staggered pitch of 90 mm.

4. The trusses for a factory building are spaced at 4.5 m centre to centre and the purlins are spaced at 1.8 m centre to centre. The pitch of truss is 1/4 and the span of the roof is 10 m. The vertical load from roof sheets etc. are equal to 180 N/m^2 while the wind load on roof surface normal to the roof is equal to 1200 N/m^2.

Design

(*a*) I-section purline,

(*b*) angle purlins. Take σ_{bt}=165 N/mm².

Solution.

$$\text{Pitch of truss} = \frac{1}{4}.$$

Hence slope, $\theta = \tan^{-1}\frac{1}{2} = 26.56°$

$$\sin\theta = 0.4472;$$

$$\cos\theta = 0.8944$$

(*i*) Design of I-section purlin

(*i*) Computation of dead load on purlin.

Load from roof sheeting = 180 × 1.8 = 324 N/m

Self weight (assumed) = 120 N/m (say)

$$\text{Total, } w_d = \overline{444 \text{ N/m}}.$$

(*ii*) Wind pressure/m run

$$w_w = 1200 \times 1.8 = 2160 \text{ N/m}.$$

(*iii*) Design bending moment for I-section purlin

It is assumed that (dead load + wind load) combination would produce more effect on the purlins than the combination (dead load + live load).

Now, for I-section purlin,

$$M_{UU} = M_{XX} = (w_w + w_d\cos\theta)\frac{l^2}{10}$$

$$= (2160 + 444 \times 0.8944) \times \frac{(4.5)^2}{10}$$

$$= 5178 \text{ N-m} = 5178 \times 10^3 \text{ N-mm}$$

$$M_{VV} = M_{YY} = (w_d \sin\theta)\frac{l^2}{10}$$

$$= 444 \times 0.4472\frac{(4.5)^2}{10}$$

$$= 420.1 \text{ N-m}$$

$$= 420.1 \times 10^3 \text{ N-mm}$$

(*iv*) Selection of I-section

$$\text{Taking } \frac{Z_{XX}}{Z_{YY}} = 6$$

and $\sigma_{bt} = 165$ N/mm²,

$$\therefore \quad Z_{XX} = \frac{M_{XX}}{\sigma_{bt}}\left(1 + \frac{Z_{XX}}{Z_{YY}}.\frac{M_{YY}}{M_{XX}}\right)$$

$$= \frac{5178 \times 10}{165}\left(1 + 6 \times \frac{420.1}{5178}\right)$$

$$= 46.66 \times 10^4 \text{ mm}^3 = 46.66 \text{ cm}^4$$

Hence provide ISLB 125 @ 11.9 kg/m. having

$$Z_{XX} = 65.1 \text{ cm}^3$$

and $Z_{YY} = 11.6 \text{ cm}^3$.

Check for stresses.

$$f_b = \frac{M_{XX}}{Z_{XX}} + \frac{M_{YY}}{Z_{YY}}$$

$$= \frac{5178 \times 10^3}{65.1 \times 10^3} + \frac{420.1 \times 10^3}{11.6 \times 10^3}$$

$$= 115.76 \text{ N/mm}^2 < 165 \text{ N/mm}^2.$$

Hence OK.

(*b*) Design of angle purlin (IS : 800 – 1984)

(*i*) Computation of loads

Assume self weight = 100 N/m (say)

$\therefore \quad w_d = 324 + 100 = 424$ N/m.

Also, $w_w = 2160$ N/m. as before.

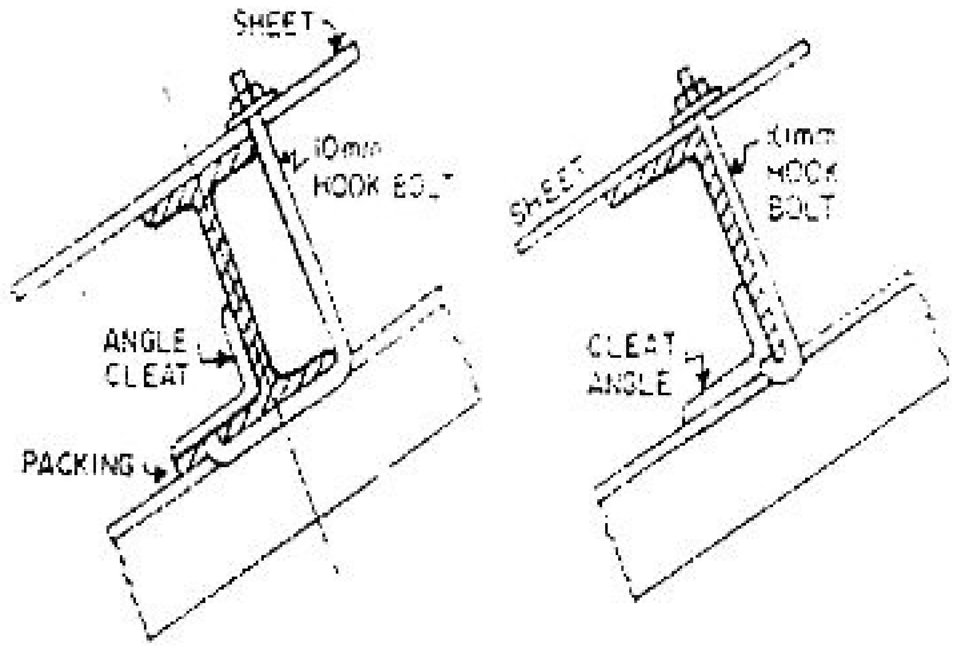
$\therefore$ Total load $w = w_d + w_w$

$$= 424 + 2160 = 2584 \text{ N/m}$$

(*ii*) Computation of bending ment

$$M = \frac{wl^2}{10} = \frac{2584(4.5)^2}{10}$$

$$= 5232.6 \text{ N-m} = 5232.6 \times 10^3 \text{ N-mm}$$



(a) I-SECTION PURLIN (b) ANGLE PURLIN

(*iii*) Design of section

$$\text{Required, } Z = \frac{M}{\sigma_{bt}} = \frac{5232.6 \times 10^3}{165}$$

$$= 31713 \text{ mm}^3 = 31.713 \text{ cm}^3$$

Minimum depth of purlin

$$\frac{l}{45} = \frac{4.5 \times 1000}{45} = 100 \text{ mm}$$

$$\text{Minimum width of purlin} = \frac{l}{60}$$

$$= \frac{4.5 \times 1000}{60} = 75 \text{ mm}.$$

Select ISA 125 × 75 @ 14.9 kg/m with thickness = 10 mm.

Section modulus = 36.3 cm³ > 31.713 cm³

Depth of purlin = 125 mm > 100 mm

Width of purlin = 75 mm ≥ 75 mm

For this case, the I-section purlin (having mass of 11.9 kg/m) is more economical than the angle purlin which has a mass of 14.9 kg/m.

EXERCISE - I

1. The conventional flexure theory is not applicable to fasteners when the length to diameter ratio of the fasterner is less than

(*a*) 2.5 (*b*) 5
(*c*) 10 (*d*) 15

2. As compared to field rivets the shop rivets are

(*a*) stronger (*b*) weaker
(*c*) equally strong (*d*) any of the above

3. Which of the following is in violation of assumptions made in riveted joint analysis ?

(*a*) Deformation of plates is neglected
(*b*) Rivets are rigid
(*c*) Stress concentration is neglected
(*d*) None of the above

4. When the load line coincides with the e.g. of the rivet group, then the rivets are subjected to

(*a*) only shear
(*b*) only tension
(*c*) only bending
(*d*) both shear and tension

5. In a riveted joint subjected to axial tension, the assumption that the rivets are stressed equally implies that

(*a*) Rivets are rigid
(*b*) Plates are rigid
(*c*) Rivets and plates both are rigid
(*d*) Rivets are flexible and plates are rigid

6. In fillet weld cross-section, throat is the

(*a*) minimum dimension
(*b*) average dimension
(*c*) maximum dimension
(*d*) leg length

7. A butt weld is specified by

(*a*) Effective throat thickness
(*b*) Leg length
(*c*) Plate thickness
(*d*) Penetration thickness

8. The reinforcement of butt weld

(*a*) increases the efficiency of the joint
(*b*) is ignored in calculating stresses
(*c*) should not exceed 3 mm
(*d*) all of the above

9. The effective length of intermittent fillet weld should not be less than

(*a*) 2 × size of weld (*b*) 4 × size of weld
(c) $\frac{1}{100}$ × length of weld (*d*) none of the above

10. Design of pins is primarily governed by

(*a*) shear (*b*) bearing
(*c*) flexure (*d*) all of the above

11. Bolts are most suited for

(*a*) shear
(*b*) tension
(*c*) bencthing
(*d*) both (*a*) and (*c*) above

12. For reversal of stress, the most suited bolt is

(*a*) black (*b*) turned
(c) friction grip (*d*) none of the above

13. A high strength bolt connection may be used as

(*a*) friction type connection for reversal where slip may take place between the plates
(*b*) bearing type connection for static loads
(*c*) a shear connection only
(*d*) both (*a*) and (*b*) above

14. High strength bolts are designed on the basis of

(*a*) friction (*b*) tension
(*c*) compression (*d*) shear

15. When a tension member is made of four angles with the plate as a web, the allowance for hole is made as

(*a*) one hole for each angle
(*b*) two holes for each angle and one hole for web
(*c*) one hole for each angle and one hole for web plate
(*d*) none of the above

16. Maximum slenderness ratio of ties premissible in steel is

(*a*) 250 (*b*) 350
(*c*) 450 (*d*) No limit

17. Allowable direct tensile stress in rolled mild steel sections is about (in MPa)

(*a*) 120 (*b*) 150
(*c*) 180 (*d*) 200