UNIT 1 – INTRODUCTION AND CONNECTIONS

1. Find the efficiency of the lap joint shown in fig.1.with the following data: M20 bolts of grade 4.6 and Fe410 plates are used.

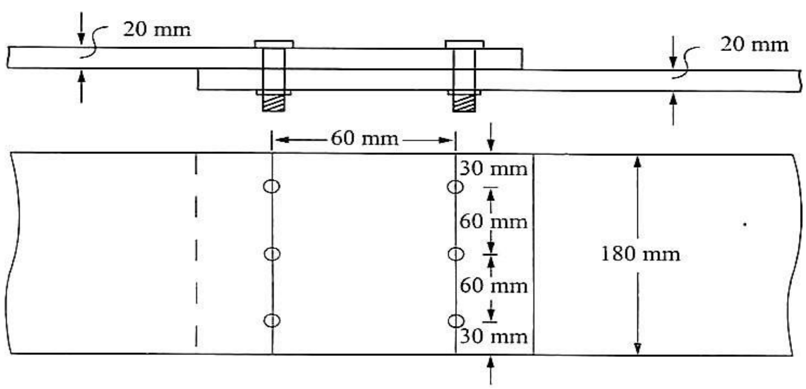


Figure 1

Solution:

For M20 bolts of grade 4.6,

diameter of bolt, $d = 20$ mm

diameter of bolt hole, $d_0 = 22$ mm

Ultimate strength $f_{ub} = 400$ MPa

Partial safety factor, $\gamma_{mb} = 1.25$

For Fe 410 (E 250) plates,

Ultimate stress, $f_u = 410$ MPa

Partial safety factor, $\gamma_{ml} = 1.25$

Strength of plates in the joint:

Thickness of thinner plate, $t = 20$ mm

Width of plate $b = 180$ mm

There is no staggering $p_{si} = 0$

Number of bolt holes in the weakest section = 3

∴ Net area at weakest section

$$A_u = [b - nd_0 + 0]\, t$$

$$= [180 - 3 \times 22] \times 20 = 2280 \text{ mm}^2$$

Design strength of plates in the joint

$$T_{dn} = \frac{0.9\, f_u\, A_n}{\gamma_{ml}} = \frac{0.9 \times 410 \times 2280}{1.25}$$

$$= 673056 \text{ N} = 673.056 \text{ kN}.$$

Strength of Bolts:

Total number of bolts = 6

(i) *Design Strength in Shear:*

Number of shear planes at thread $n_n = 1$ per bolt.

Number of shear planes at shank $n_s = 0$ per bolt.

$\therefore$ Total $n_n = 1 \times 6 = 6$ and total $n_s = 0$.

$$A_{nb} = 0.78 \times \frac{\pi}{4} d^2 = 0.78 \times \frac{\pi}{4} \times 20^2 = 245 \text{ mm}^2$$

There are no reduction factors i.e. $\beta_{lj} = \beta_{lg} = \beta_{pk} = 1$

$\therefore$ Nominal shear strength,

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

$$= \frac{400}{\sqrt{3}} \left(6 \times 245 + 0 \right) = 339482 \text{ N} = 339.482 \text{ kN}$$

$\therefore$ Design strength in shear,

$$V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}} = \frac{339.482}{1.25} = 271.586 \text{ kN}$$

(ii) *Design Strength in Bearing:*

Nominal strength

$$V_{npb} = 2.5\, K_b d t\, f_u$$

where K_b = least of the following:

(a) $\frac{e}{3d_0} = \frac{30}{3 \times 22} = 0.4545$

(b) $\frac{p}{3d_0} - 0.25 = \frac{60}{3 \times 22} - 0.25 = 0.659$

(c) $\frac{f_{ub}}{f_u} = \frac{400}{410} = 0.9756$

(d) 1.0.

Note: Edge distance provided is less. Hence it is critical in this case.

$K_b = 0.4545$

$\therefore\ V_{npb} = 2.5 \times 0.4545 \times 20 \times 20 \times 410 = 186345$ N per bolt

Design strength $= \dfrac{V_{npb}}{\gamma_{mb}} = \dfrac{186345}{1.25} = 149076$ N

$\therefore$ Design strength of joint $= 6 \times 149076 = 894456.8$ N

$= 894.456$ kN

$\therefore$ Design strength of bolts in joint $= 271.586$ kN $< T_{dn}$.

$\therefore$ Strength of joint $= 271.586$ kN.

Efficiency of Joint:

Area of solid plate $= 180 \times 20 = 3600$ mm^2.

$\therefore$ Design strength of solid plate

$$= \frac{f_y}{\gamma_{mb}} \times A_g = \frac{250}{1.1} \times 3600 = 818181.8 \text{ N} = 818.818 \text{ kN}$$

$\therefore$ Efficiency of the joint $= \dfrac{\text{Strength of joint}}{\text{Strength of solid plate}} \times 100$ percent

$$= \frac{271.586}{818.818} \times 100 = 33.19\%$$

2. Find the maximum force which can be transferred through the double covered butt joint shown in fig. Find the efficiency of the joint also. Given M20 bolts of grade 4.6 and Fe410 steel plates are used.

Solution:

For M20 bolts of Grade 4.16,

$d = 20$ mm $\quad d_0 = 22$ mm $\quad f_{ub} = 400$ N/mm^2.

For grade Fe 410 plates, $f_{ub} = 410$ N/mm^2.

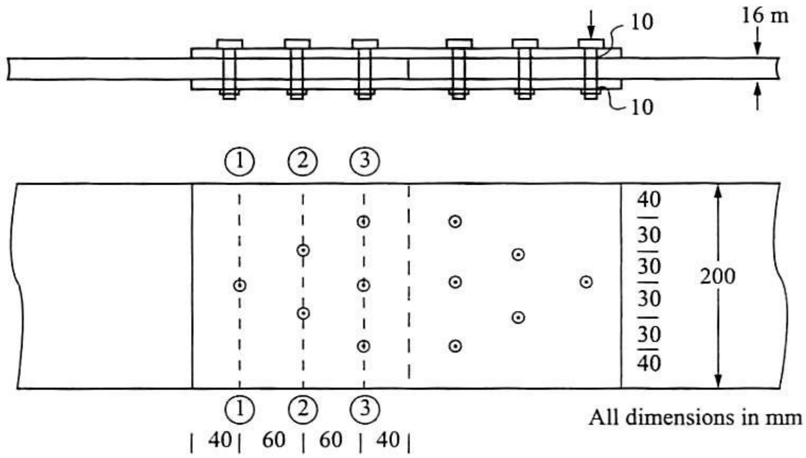


Figure 3.19

Nominal strength of one bolt in shear (double shear)

$$= \frac{f_{ub}}{\sqrt{3}}\left(1 \times \frac{\pi}{4} d^2 + 0.78 \times \frac{\pi}{4} d^2\right)$$

$$= \frac{400}{\sqrt{3}}(1.78) \times \frac{\pi}{4} 20^2$$

$$= 129143 \text{ N}$$

Design strength of one bolt in double shear

$$= \frac{129143}{1.25} = 107619 \text{ N}$$

Design strength of joint in double shear

$$= 6 \times 107619 = 645715 \text{ N} = 645.715 \text{ kN}$$

Strength of bolts in bearing:

K_b is the least of the following:

$$\frac{e}{3d_0}, \frac{p}{3d_0} - 0.25, \frac{f_{ub}}{f_u}, 1.0$$

$\therefore$ For bolts at section (3) – (3), it is least of

$$\frac{40}{3 \times 22}, \frac{60}{3 \times 22} - 0.25, \frac{400}{410}, 1.0$$

i.e. $K_b = 0.6061$

$\therefore$ For bolts on section (2)–(2) and (1)–(1), 'e' in large. Hence

$$K_{b1} = K_{b2} = 0.6591, \text{ which is governed by } \frac{p}{3d_0}$$

$\therefore$ Nominal strength of bolts in bearing

$$= 3\,(2.5 \times 0.6061 \times 20 \times 16 \times 410) + 3\,(2.5 \times 0.6591 \times 20 \times 16 \times 410)$$

$$= 1244957 \text{ N}$$

$\therefore$ Design strength in bearing $= \dfrac{1244957}{1.25}$

$$= 995965 \text{ N}$$

$$= 995.965 \text{ kN} > 645.715 \text{ kN}$$

$\therefore$ Strength of bolts in the joint = 645.715 kN and strength of each bolt = 107619 N

Strength of plate:

It is to be checked along all the three sections.

Now, t = 16 mm (least of the thicknesses of cover plates and main plate)

$f_u = 410 \text{ N/mm}^2$

(a) At section (1) – (1)

$$T_{dn_1} = \frac{0.9\, f_u\, A_n}{1.25} = \frac{0.9 \times 410\,(420 - 22) \times 16}{1.25}$$

$$= 1879833 \text{ N}$$

(b) At section (2) – (2)

When this section fails, bolt in section (1) – (1) also has to fail. Hence strength of plate at section (2) – (2)

$$T_{dn_2} = \frac{0.9 \times 410\,(420 - 2 \times 22) \times 16}{1.25} + 107619$$

$$= 1883542 \text{ N}$$

At section (3) – (3)

$$T_{dn_3} = \text{Plate strength} + \text{strength of 3 bolts}$$

$$= \frac{0.9 \times 410\,(420 - 3 \times 22) \times 16}{1.25} + 3 \times 107619$$

$$= 1994870 \text{ N}$$

Strength of plate in the joint = 1879.833 N

= 1879.833 kN

Strength of joint = 645.715 kN

Maximum design force that can be transferred safely = 645.715 kN.

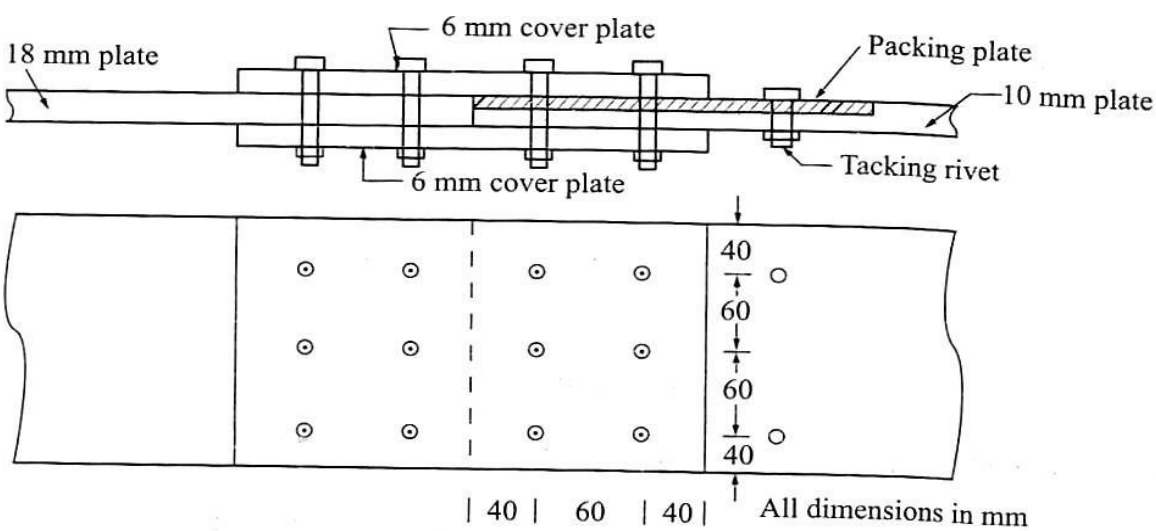
$$\text{Permissible force at working condition} = \frac{645.715}{1.5} = 430.477 \text{ kN}$$

$$\text{Design strength of solid plate} = \frac{250 \times 200 \times 16}{1.1} = 727272 \text{ N}$$

$$= 727.272 \text{ kN}$$

$$\therefore \text{Efficiency of the joint} = \frac{645.715}{727.272} \times 100 = 88.78\%$$

3. Two cover plates, 10mm and 18mm thick are connected by a double cover butt joint using 6mm cover plates as shown in fig. Find the strength of the joint. Given M20 bolts of grade 4.6 and Fe410 plates are used.



Solution:

Note: Strength of tacking rivets are not to be considered in the design.

In this connection packing plate of 8 mm thickness is to be used. Hence there shall be reduction in the shear strength of bolt. The reduction factor is given by

$$\beta_{pk} = (1 - 0.0125\, t_{pk})$$
$$= 1 - 0.0125 \times 8 = 0.9$$

∴ Nominal shear strength of one bolts in double shear

$$= \beta_{pk} \frac{f_{ub}}{\sqrt{3}} \left(1 \times \frac{\pi}{4} d^2 + 0.78 \times \frac{\pi}{4} d^2 \right)$$
$$= 0.9 \times \frac{400}{\sqrt{3}} (1.78) \times \frac{\pi}{4} \times 20^2$$
$$= 116228 \text{ N}$$

Design shear strength of one bolt in shear

$$= \frac{116228}{1.25} = 92982.6 \text{ N}$$

∴ Design shear strength of 6 bolts in the joint

$$= 6 \times 92982.6 = 557896 \text{ N}$$
$$= 557.896 \text{ kN}$$

Strength of bolts in bearing:

K_b is the minimum of

$$\frac{e}{3d_0}, \frac{p}{3d_0} - 0.25, \frac{f_{ub}}{f_u}, 1.0$$

i.e.

$$\frac{40}{3 \times 22}, \quad \frac{60}{3 \times 22} - 0.25, \quad \frac{400}{410}, \quad 1.0$$

$$\therefore \quad K_b = 0.6061.$$

∴ Nominal strength of one bolt in bearing $= 2.5\, K_b\, dt\, f_u$

$$= 2.5 \times 0.6061 \times 20 \times 10 \times 410$$
$$= 124250.5 \text{ N}$$

Note: Thickness of thinner plate $t = 10$ mm

$\therefore$ Design strength of a bolt $= \dfrac{124250.5}{1.25} = 99400$ N

Design strength of 6 bolts $= 6 \times 99400$

$= 59640$ N.

$= 596.4$ kN > 557.896 kN

$\therefore$ Strength of bolts in connection $= 557.896$ kN.

Strength of plates in the joint = Strength of thinner plate at weakest section.

$\therefore$ Design strength of plate

$$= \frac{0.9\, A_n\, f_u}{\gamma_m} = \frac{0.9 \times [200 - 3 \times 22] \times 10 \times 400}{1.25}$$

$= 385920$ N

$= 385.920$ kN < 597.896 kN

Design strength of the joint $= 385.920$ kN.

4. Design a lap joint between the two plates each of width 120mm,if the thickness of one plate is 16mm and the other is 12mm.the joint has to transfer a design load of 160Kn.the plates are of Fe410grade.use bearing type bolts.

Solution:

Using M16 bolts of grade 4.6,

$d = 16$ mm $\quad d_0 = 18$ mm $\quad$ and $\quad f_{ub} = 400$ N/mm^2

Strength of a bolt:

Since it is lap joint bolt is in single shear, the critical section being at the root of bolt.

$\therefore$ Nominal strength of a bolt in shear $= \dfrac{f_u}{\sqrt{3}}\left(1 \times 0 + 0.78 \times \dfrac{\pi}{4} d^2\right)$

$$= \frac{400}{\sqrt{3}} \times 0.78 \times \frac{\pi}{4} \times 16^2$$

$= 36218$ N.

$\therefore$ Design shear strength $= \dfrac{36218}{1.25} = 28974$ N.

Minimum edge distance to be provided $= 1.5 \times 18 = 27$ mm

Minimum pitch to be provided $= 2.5 \times 16 = 40$ mm.

Providing $e = 30$ mm, $p = 40$ mm,

K_b is least of $\dfrac{30}{3\times18}$, $\dfrac{40}{3\times18} - 0.25$, $\dfrac{400}{410}$ and 1.0.

i.e. $K_b = 0.4907$

$\therefore$ Nominal bearing strength $= 2.5\, K_b\, dt\, f_u$

$= 2.5 \times 0.4907 \times 16 \times 12 \times 400$

$= 94222$ N

$\therefore$ Design bearing strength $= \dfrac{94222}{1.25} = 75378$ N

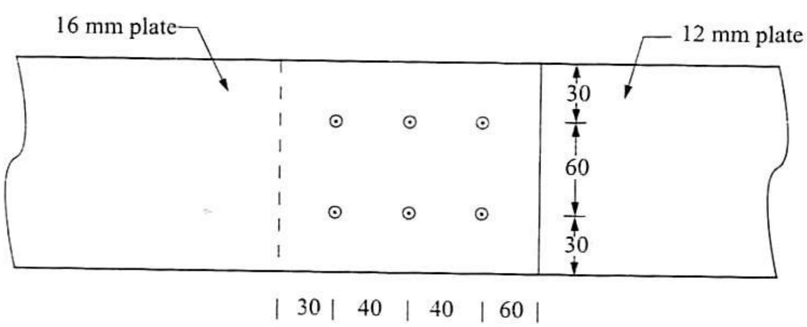
$\therefore$ Design strength of a M16 bolt $= 28974$ N

$= 28.974$ kN

Hence to transfer a design force of 160 kN,

No. of bolts required $= \dfrac{160}{28.974} = 5.5$

$\therefore$ Provide 6 bolts. They may be provided in two rows with a pitch of 40 mm as shown in Fig.



Check for the strength of plate:

$$T_{dn} = \frac{0.9\, A_n\, f_u}{\gamma_m} = \frac{0.9 \times (120 - 2 \times 18) \times 12 \times 400}{1.25}$$

$$= 290304 \text{ N} = 290.304 \text{ kN} > 160 \text{ kN safe.}$$

5. Design a single bolted double cover butt joint to connect boiler plates of thickness 12mm for maximum efficiency. Use M16 bolts of grade 4.6.boiler plates are of Fe 410 grade. Find the efficiency of the joint

Solution:

$d = 16$ mm $\quad d_0 = 18$ mm $\quad f_{ub} = 410$ N/mm^2

$f_u = 410$ N/mm^2 $\quad t = 12$ mm

Since it is double cover butt joint, the bolts are in double shear one section at shank and another at root.

Nominal strength of a bolt in shear

$$= \frac{400}{\sqrt{3}} \left(1 \times \frac{\pi}{4} \times 16^2 + 1 \times 0.78 \times \frac{\pi}{4} \times 16^2 \right)$$

$$= 82651 \text{ N} \tag{a}$$

Design strength in shear $= \dfrac{82651}{1.25} = 66121$ N

Assume bearing strength is more than it. To get maximum efficiency, strength of plate per pitch width should be equated to strength of a bolt.

To avoid failure of cover plates, the total thickness of cover plates should be more than the thickness of main plates. Provide cover plates of 8 mm thicknesses.

Design strength of plate per pitch width

$$= \frac{0.9 \times 410 (p - 18) \times 12}{1.25}$$

$$= 3542.4\ (p - 18) \tag{b}$$

Equating (a) to (b) to get maximum efficiency, we get,

$$3542.5\ (p - 18) = 66121$$

$\therefore \quad p = 36.67$ mm.

Minimum pitch $= 2.5 \times 16 = 40$ mm.

$\therefore$ Provide bolts at $p = 40$ mm.

Check for strength of bolt in bearing:

K_b is the minimum of $\frac{e}{3d_0}, \frac{p}{3d_0} - 0.25, \frac{f_{ub}}{f_u}, 1.0$

Assuming sufficient 'e' will be provided

$$K_b = 0.4907$$

$\therefore$ Design strength of bolt in bearing

$$= \frac{2.5 \times 0.4907 \times 16 \times 12 \times 400}{1.25} = 75372 \text{ N} > 66121 \text{ N}$$

Hence assumption is correct.

Since pitch provided is slightly more than required from strength consideration of the plate, the strength of plate is more than the strength of the bolt.

$\therefore$ Design strength of joint per 40 mm width = 66121 N.

Design strength of solid plate per 40 mm width

$$= \frac{250 \times 40 \times 12}{1.1} = 109091 \text{ N.}$$

$\therefore$ Maximum efficiency of joint $= \frac{66121}{109091} \times 100$

$$= 60.61\%$$

6. A bracket bolted to a vertical column is loaded as shown in fig. If M20 bolts of grade 4.6 are used, determine the maximum value of factored load P which can be carried safely.

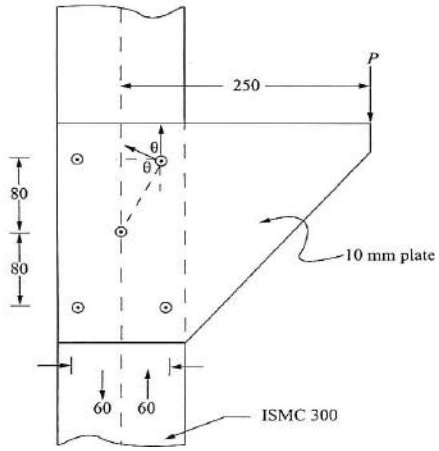
Solution:

For M20 bolts of grade 4.6, $d = 20 \quad d_0 = 22$ mm $\quad f_u = 400$ N/mm^2

For rolled steel sections, $f_u = 410$ N/mm^2

Thickness of web of ISMC 300, is

$$= 7.6 \text{ mm [Refer steel table].}$$



Since this is a lap joint between bracket plate and web of ISMC 300, the bolts are in single shear.

∴ Design strength of bolts in shear

$$= \frac{1}{1.25} \times \frac{400}{\sqrt{3}} \left(0.78 \times \frac{\pi}{4} \times 20^2 \right)$$

$$= 45272 \text{ N}$$

Strength in bearing against 7.6 mm web of ISMC 300:

K_b is the least of

$$\frac{e}{3d_0},\ \frac{p}{3d_0} - 0.25,\ \frac{f_{ub}}{f_u},\ 1.0$$

i.e. least of $$\frac{150-60}{3\times 22},\ \frac{80}{3\times 22} - 0.25,\ \frac{400}{410},\ 1.0$$

i.e. $$K_b = 0.96212$$

∴ Design strength of a bolt in bearing

$$= \frac{1}{1.25} \times 2.5\, k_b\, d\, t\, f_u$$

$$= \frac{1}{1.25} \times 2.5 \times 0.96212 \times 20 \times 7.6 \times 410$$

$$= 119919 \text{ N} > 45272 \text{ N}$$

∴ Design strength of a bolt is = 45272 N

Force in extreme bolt:

Direct shear force $F_1 = \frac{P}{5} = 0.2P$

Centre of gravity of bolted connection is at the centre of central bolt.

For four bolts, $r = \sqrt{80^2 + 60^2} = 100$ mm.

For central bolt $r = 0$

$$\therefore \quad \sum r^2 = 4 \times 100^2 + 0 = 4 \times 100 \times 100$$

For extreme bolt $r = 100$ mm.

$\therefore$ Force due to bending moment in extreme bolt $= \frac{P \times e \times r}{\sum r^2} = \frac{P \times 250 \times 100}{4 \times 100 \times 100} = 0.625P$

Angle between the two forces is given by, θ where $\cos\theta = \frac{60}{100} = 0.6$

$\therefore$ Total shear force on extreme bolt

$$= \sqrt{(0.2P)^2 + (0.625P)^2 + 2 \times 0.2P \times 0.625P \times 0.6}$$

$$= P\sqrt{(0.2)^2 + (0.625)^2 + 2 \times 0.2 \times 0.625 \times 0.6}$$

$$= 0.76199\ P$$

Equating it to strength of bolt we get

$$0.76199\ P = 45272$$

$$\therefore \quad P = 59413 \text{ N}$$

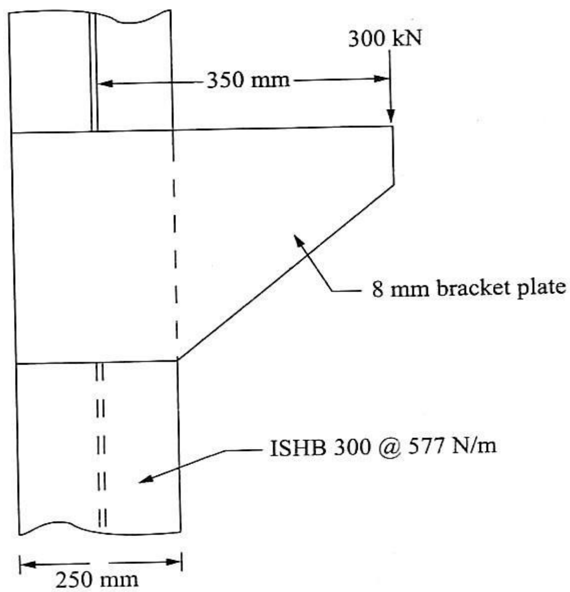
$$P = 59.413 \text{ kN}$$

7. A bracket is bolted to the flange of a column as shown in fig. Using 8mm thick bracket plate. Using M20 bolts of grade 4.6 design the connection.

Solution:

Flange thickness of ISHB 300 @ 577 N/m is 10.6 mm. Thickness of bracket plate is 8 mm. Hence thickness of thinner member in the connection is 8 mm. For M20 bolts of grade 4.6,

$d = 20$ mm $\quad d_0 = 22$ mm $\quad f_{ub} = 400$ N/mm^2



For the rolled section $f_u = 410$ N/mm^2

Bolts are in single shear.

$\therefore$ Design strength of a bolt $= \frac{1}{1.25} \times \frac{400}{\sqrt{3}} \left(0 + 0.78 \times \frac{\pi}{4} \times 20^2 \right)$

i.e $V_{db} = 45272$ N

Strength of bolt in bearing:

K_b is the least of $\frac{e}{3d_0}$, $\frac{p}{3d_0} - 0.25$, $\frac{f_{ub}}{f_u}$ and 1.0

Adopting two rows of bolting, with edge distance of 55 mm and pitch of 50 mm ($\geq 2.5\ d_0$), K_b is the least of

$$\frac{55}{3\times 22},\ \frac{50}{3\times 22} - 0.25,\ \frac{400}{410} \text{ and } 1.0$$

$$K_b = 0.5076$$

$$\therefore \quad V_{dpb} = \frac{1}{1.25} \times 2.5 \times 0.6591 \times 20 \times 8 \times 410$$

∴ Design strength of bolt, $V = V_{db} = 45272$ N

$M = 300 \times 350$ kN/mm $= 300 \times 1000 \times 350$ N-mm.

∴ Number of bolts required per row

$$n = \sqrt{\frac{6M}{2 \times Vp}}$$

$$= \sqrt{\frac{6 \times 300 \times 1000 \times 350}{2 \times 45272 \times 50}} = 11.79$$

∴ Provide 12 bolts in each row as shown in Fig. 3.27.

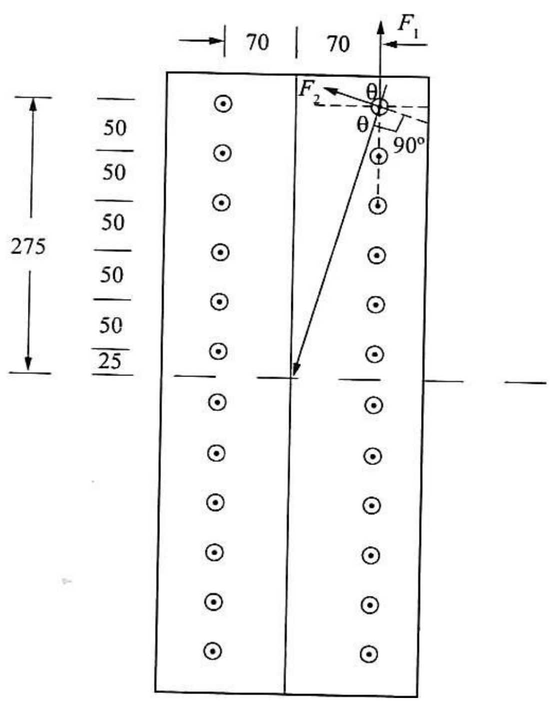
Distance of extreme bolt from centre of gravity of bolts

$$r = \sqrt{70^2 + 275^2} = 283.77 \text{ mm}$$

$$\Sigma r^2 = 4\left[\sum_{i=1}^{6}\left(x_i^2 + y_i^2\right)\right]$$

$$= 4\left[6 \times 70^2 + 25^2 + 75^2 + 125^2 + 175^2 + 225^2 + 275^2\right]$$

$$= 832600 \text{ mm}^2$$



$\therefore$ Force in extreme bolt due to bending

$$= \frac{Per}{\sum r^2} = \frac{300 \times 1000 \times 350 \times 283.77}{832600}$$

$$F_2 = 35786.5 \text{ N}$$

Direct shear force $\quad F_1 = \frac{300 \times 1000}{2 \times 12} = 12500 \text{ N}$

$\therefore$ Resultant force on extreme bolt

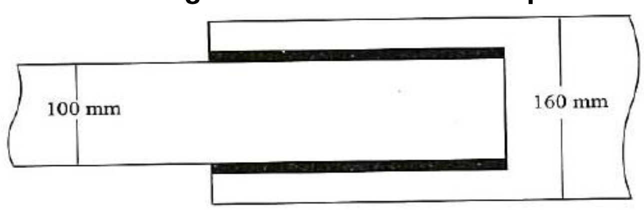
$$= \sqrt{F_1^2 + F_2^2 + 2 F_1 F_2 \cos\theta}$$
$$= \sqrt{12500^2 + 35786.5^2 + 2 \times 12500 \times 35786.5 \times 0.24668}$$

$$= 37907 \text{ N} < V$$

$\therefore$ Design is safe.

Hence provide 24 M20 bolts as shown in Fig.

8. Design a suitable longitudinal fillet welds to connect the plates as shown in fig. To transmit a pull equal to the full strength of small plate. Given: plates are 12mm thick, grade of plates Fe410 and welding to be made in workshop.



Solution:

Minimum size to be used = 5 mm

Maximum size = 12 – 1.5 = 10.5 mm

Use s = 10 mm fillet weld

$f_y = 410 \text{ N/mm}^2$, $\gamma_{mw} = 1.25$, thickness of plate = 12 mm, breadth of plate = 100 mm

$\therefore$ Full design strength of smaller plate $= \frac{A_g f_y}{\gamma_{mo}}$

$f_y = 250$ MPa, $\gamma_{mo} = 1.1$

$$= 12 \times 100 \times \frac{250}{1.1} = 272727 \text{ N}$$

Let effective length of welds be L_w

Assuming normal weld, throat thickness

$$t = 0.7 \times 10 = 7 \text{ mm}$$

$\therefore$ Design strength of weld $= L_w t \dfrac{f_u}{\sqrt{3}} \times \dfrac{1}{1.25}$

$$= L_w \times 7 \times \frac{410}{\sqrt{3}} \times \frac{1}{1.25}$$

Equating it to the strength of plate, we get

$$L_w \times 7 \times \frac{410}{\sqrt{3}} \times \frac{1}{1.25} = 272727$$

$L_w = 205.7$ mm

Provide effective length of 105 mm on each side.

9. a tie member of a roof truss consists of 2 ISA 10075, 8mm.the angles are connected to either side of a 10mm gusset plates and the member is subjected to a working pull of 300Kn.design the weld connection. Assume connections are made in the workshop.

Solution:

Working Load = 300 kN

$\therefore$ Factored Load $= 300 \times 1.5 = 450$ kN

Thickness of weld:

(i) At the rounded toe of the angle section, size of weld should not exceed $= \dfrac{3}{4} \times$ thickness

$$s = \frac{3}{4} \times 8 = 6 \text{ mm}$$

(ii) At top (Ref. Fig.) the thickness should not exceed

$$s = t - 1.5 = 8 - 1.5 = 6.5 \text{ mm.}$$

Hence provide $s = 6$ mm, weld.

Each angle carries a factored pull of $\dfrac{450}{2} = 225$ kN.

Let L_w be the total length of the weld required.

Assuming normal weld, $t = 0.7 \times 6$ mm

$\therefore$ Design strength of the weld $= L_w t \dfrac{f_u}{\sqrt{3}} \times \dfrac{1}{1.25}$

$$= L_w \times 0.7 \times 6 \times \frac{410}{\sqrt{3}} \times \frac{1}{1.25}$$

Equating it to the factored load, we get

$$L_w \times 0.7 \times 6 \times \frac{410}{\sqrt{3}} \times \frac{1}{1.25} = 225 \times 10^3$$

$\therefore \quad L_w = 283$ mm.

Centre of gravity of the section is at a distance 31 mm from top.

Let L_1 be the length of top weld and L_2 be the length of lower weld. To make centre of gravity of weld to coincide with that of angle,

$$L_1 \times 31 = L_2 (100 - 31)$$

$$L_1 = \frac{69}{31} L_2$$

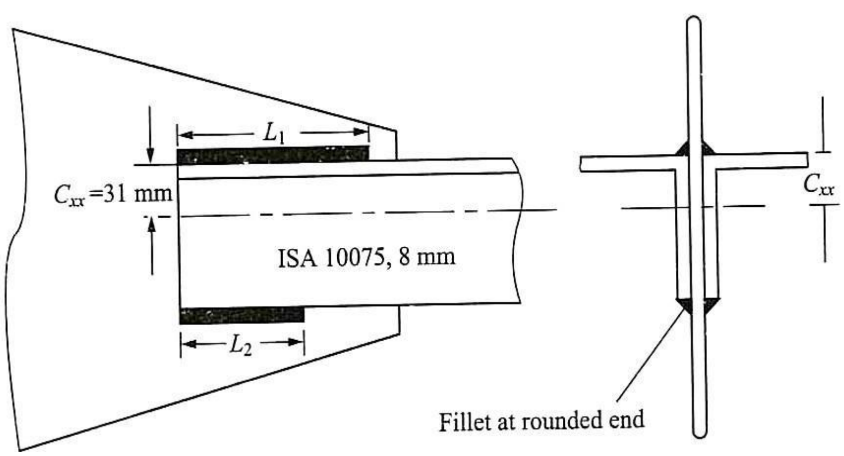
$$L_1 + L_2 = 283$$

$$L_2 \left(\frac{69}{31} + 1 \right) = 283$$

or $\quad L_2 = 87$ mm.

$\therefore \quad L_1 = 195$ mm.

Provide 6 mm weld of $L_1 = 195$ mm and $L_2 = 87$ mm as shown in the Fig. 4.10.



10. Design a welded connection to connect two plates of width 200mm and thickness 10mm for 100 percent efficiency.

Solution:

Strength of plates $= \frac{A_g f_y}{\gamma_{mo}} = \frac{200 \times 10 \times 250}{1.1} = 454545$ N

Minimum size of weld = 5 mm.

Maximum size = 10 – 1.5 = 8.5 mm.

Use s = 8 mm weld.

Effective length of fillet welds = 2 (200 – 2 × 8)

$$L_w = 368 \text{ mm.}$$

Throat thickness $t = 0.7 \times 8$

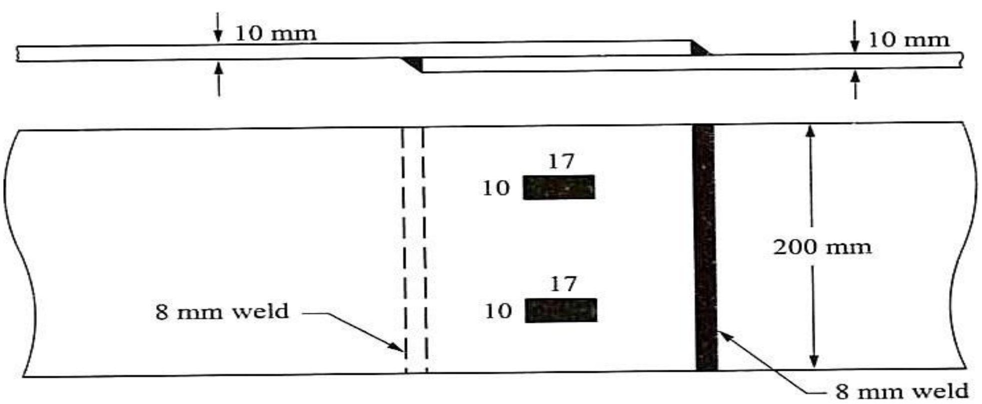
Design strength of fillet welds $= L_w t \frac{f_u}{\sqrt{3}} \times \frac{1}{\gamma_{mw}}$

$$= 368 \times 0.7 \times 8 \times \frac{410}{\sqrt{3}} \times \frac{1}{1.25} = 390256 \text{ N}$$

∴ Slot welds are to be provided to resist a force of = 454545 – 390256 = 64289 N

Strength of the slot weld $= \frac{f_{wn}}{\gamma_{mw}} = \frac{f_u}{\sqrt{3}\gamma_{mw}}$

$$= \frac{410}{\sqrt{3} \times 1.25} = 189.37 \text{ N/mm}^2$$



$$\therefore \quad \text{Area of the slot weld required} = \frac{64289}{189.37} = 339.5 \text{ mm}^2$$

Provide two slot welds of size 10 mm × 17 mm as shown in Fig.

11. A tie member consists of two ISMC 250.the channels are connected on either side of a 12mm thick gusset plate. Design the welded joint to develop the full strength of the tie. However the overlap is to be limited to 400mm.

Solution:

For ISMC 250, [From steel tables]

Thickness of weld = 7.1 mm

Thickness of flange = 14.1 mm

Sectional area = 3867 mm^2

$$\text{Tensile design strength of each channel} = \frac{A_g f_y}{1.1} = \frac{3867 \times 250}{1.1} = 878864 \text{ N}$$

Weld thickness:

Minimum thickness = 3 mm.

Maximum thickness = $0.7t = 0.7 \times 7.1 = 4.97$ mm.

Provide $s = 4$ mm weld.

$\therefore$ Throat thickness, $t = 0.7 \times 4 = 2.8$ mm

$$\text{Strength of weld } = L_w t \frac{f_u}{\sqrt{3}} \times \frac{1}{\gamma_{mw}}$$

$$= L_w \times 2.8 \times \frac{410}{\sqrt{3}} \times \frac{1}{1.25}$$

Equating strength of weld to tensile strength of the channel, we get

$$L_w \times 2.8 \times \frac{410}{\sqrt{3}} \times \frac{1}{1.25} = 878804$$

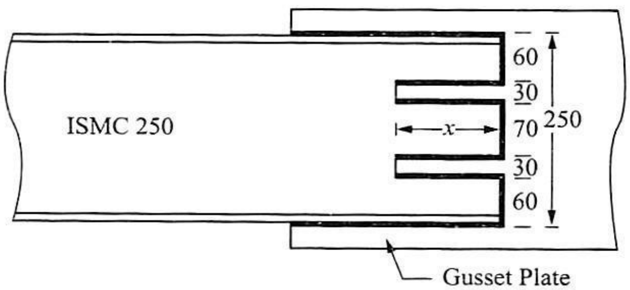
$\therefore \quad L_w = 1658$ mm.

Since allowable length is limited to 400 + 400 mm it needs slot weld. The arrangement can be as shown in the figure with two slots of length 'x'. Then

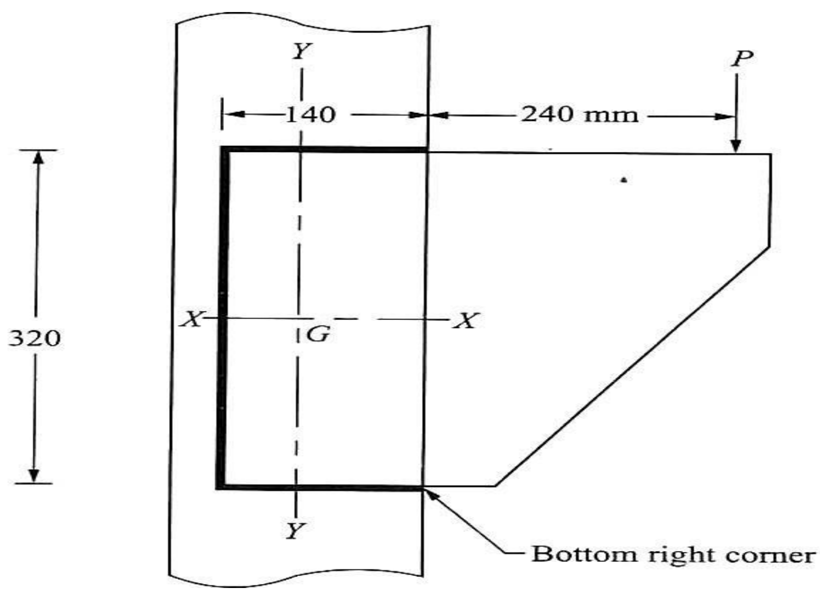
$$400 + 400 + (250 - 2 \times 30) + 4x = 1658$$

$$\therefore \quad x = 167 \text{ mm.}$$

Provide $x = 170$ mm as shown in the Fig. 4.13.



12. Determine the maximum load that can resist by the bracket shown in fig. By fillet weld of size 6mm, if it is shop welding.



Solution:

Size of weld = 6 mm.

$\therefore$ Throat thickness $= 0.7 \times 6 = 4.2$ mm

Consider the area of the weld which has channel shape and has width 4.2 mm throughout.

Total area of weld $= 320 \times 4.2 + 140 \times 4.2 \times 2$

$= 600 \times 4.2 \text{ mm}^2$

Due to symmetry, centroidal x-x axis is at the mid height of vertical weld. Let centroidal y-y axis be at a distance $\bar{x}$ from the vertical weld. Then

$$\bar{x} = \frac{140 \times 4.2 \times 70 \times 2}{600 \times 4.2} = 32.67 \text{ mm}$$

$$I_{xx} = \frac{1}{12} \times 4.2 \times 320^3 + 140 \times 4.2 \times 160^2 \times 2$$

$$= 41574400 \text{ mm}^4$$

$$I_{yy} = 320 \times 4.2 (32.67)^2 + 2\left[\frac{1}{12} \times 140^3 \times 4.2 + 4.2 \times 140 (70 - 32.67)^2\right]$$

$$= 4994080 \text{ mm}^4$$

$$\therefore \quad I_{zz} = I_{xx} + I_{yy} = 46568480 \text{ mm}^4$$

$$r_{\max} = \sqrt{160^2 + (140 - 32.67)^2} = 192.66 \text{ mm.}$$

$$\tan\theta = \frac{160}{140 - 32.67} \qquad \therefore \quad \theta = 56.15°$$

Eccentricity of load from the centre of gravity of weld

$$e = 240 + 140 - 32.67 = 347.33 \text{ mm.}$$

Let P be in kilo newtons.

Direct shear stress $q_1 = \frac{P \times 1000}{\text{Total area}} = \frac{P \times 1000}{600 \times 4.2}$

$$= 0.3968\, P \text{ N/mm}^2$$

Shear stress at extreme edge due to torsional moment

$$q_2 = \frac{P \times e \times r_{\max}}{I_{zz}}$$

$$= \frac{P \times 1000 \times 347.33 \times 192.66}{46568480}$$

$$= 1.4370\, P \text{ N/mm}^2$$

Resultant stress $= \sqrt{q_1^2 + q_2^2 + 2q_1q_2\cos\theta}$

$$= P\sqrt{0.3968^2 + 1.4370^2 + 2 \times 0.3968 \times 1.4370 \times \cos 56.15}$$

$$= 1.69046\, P$$

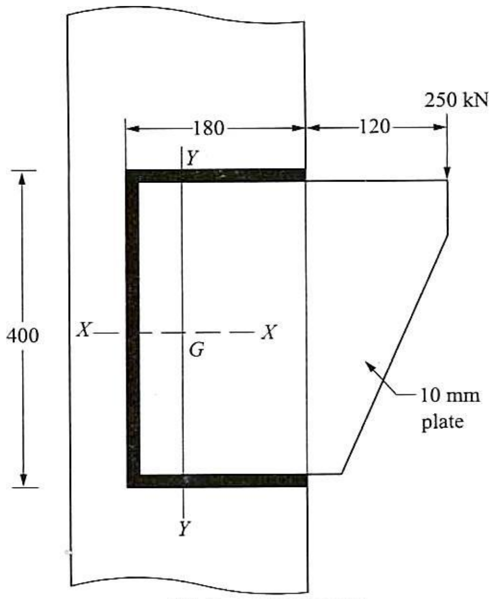
weld can resist a stress of $= \frac{f_u}{\sqrt{3}} \times \frac{1}{1.25} = \frac{410}{\sqrt{3} \times 1.25} = 189.32$

Equating maximum stress to resisting stress, we get,

$$2.69046\, P = 189.32$$

$$\therefore \quad P = 111996 \text{ N} = 111.996 \text{ kN}$$

13. The 10mm thick bracket plate shown in fig. Is connected with the flange of column ISHB 3002577N/mm. find the size of the weld to transmit a factored load of 250Kn.



All dimensions in mm

Solution:

Let 't' be the throat thickness of the weld required and $\bar{x}$ be the distance of c.g. of weld from vertical weld. Then

$$\text{Area of the weld} = 400\,t + 180\,t \times 2 = 760\,t$$

$$\bar{x} = \frac{2 \times 180t \times 90}{760t} = 42.63 \text{ mm.}$$

$$I_{xx} = \frac{1}{12} \times 400^3 \times t + 180t \times 200^2 \times 2 = 19733333t \text{ mm}^4$$

$$I_{yy} = 400t \times 42.63^2 + 2\left[\frac{1}{2} \times t \times 180^3 + 180t \times (90 - 42.63)^2\right]$$

$$= 2506737t \text{ mm}^4$$

$$\therefore\ I_{zz} = 22240070t \text{ mm}^4$$

Distance of extreme point of the weld from c.g.

$$r_{max} = \sqrt{200^2 + (180 - 42.63)^2} = 242.63 \text{ mm}$$

$$\tan\theta = \frac{200}{180 - 42.63} = 1.4559$$

$$\theta = 55.17^\circ$$

Eccentricity $e = 120 + 180 - 42.63 = 257.37$ mm.

$\therefore$ Direct shear stress $= q_1 = \frac{250 \times 10^3}{760t} = \frac{328.95}{t}$ N/mm^2

Maximum shear stress due to twisting moment

$$q_2 = \frac{P \times e \times r_{max}}{I_{zz}} = \frac{250 \times 10^3 \times 257.37 \times 242.63}{22240070t}$$

$$= \frac{701.950}{t} \text{ N/mm}^2$$

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

$$= \sqrt{\left(\frac{328.95}{t}\right)^2 + \left(\frac{701.95}{t}\right)^2 + 2\frac{328.95}{t} \times \frac{701.950}{t}}$$

$$= \frac{1030.9}{t}$$

Resistance of the weld $= \frac{f_u}{\sqrt{3}} \times \frac{1}{1.25} = \frac{410}{\sqrt{3}} \times \frac{1}{1.25}$

$= 189.37$ N/mm^2

Equating maximum shear to it, we get $\frac{1030.9}{t} = 189.37$

$\therefore$ $t = 5.44$ mm

$\therefore$ Size of normal fillet $= \frac{5.44}{0.7} = 7.777$ mm

$\therefore$ Provide 8 mm fillet weld.

14. Design a suitable fillet weld for the bracket shown in fig. If working load P=100Kn and eccentricity e=150mm.thickness of the bracket plate is 12mm and the column used is ISHB 300 @618N/m.

Solution:

Load = 100 kN

$\therefore$ Factored load, $P = 100 \times 1.5 = 150$ kN

Thickness of flange of ISHB 300 @ 618 N/m is 10.6 mm.

Minimum size of weld = 5 mm.

Use 8 mm weld on each side of bracket plate.

Throat thickness, $t = 0.7 \times 8$ mm.

Resistance of weld $= \dfrac{f_u}{\sqrt{3}} \times \dfrac{1}{\gamma_{mw}} = \dfrac{410}{\sqrt{3}} \times \dfrac{1}{1.25} = 189.37\ \text{N/mm}^2$.

Depth of weld required to resist bending alone $= h' = \sqrt{\dfrac{6 \times 150 \times 10^3 \times 150}{2 \times 0.7 \times 8 \times 189.37}} = 252.3$ mm

About 10 percent extra depth is to be provided.

Let $h = 280$ mm.

Check for the stresses:

Direct shear stress $q = \dfrac{P}{2 \times t \times h} = \dfrac{15 \times 10^3}{2 \times 0.7 \times 8 \times 280} = 47.83\ \text{N/mm}^2$

Bending stress $f = \dfrac{M}{Z} = \dfrac{6M}{2t \times h^2} = \dfrac{6 \times 150 \times 10^3 \times 150}{2 \times 0.7 \times 8 \times 280^2}$

$= 153.744\ \text{N/mm}^2$

$$\therefore \sqrt{f^2 + 3q^2} = \sqrt{153.744^2 + 3 \times 47.83^2}$$

$$= 174.6\ \text{N/mm}^2 < 189.37\ \text{N/mm}^2$$

Hence design is safe.

UNIT II – COMPRESSION MEMBERS

1. **in a truss a strut 3m long consists of two angles ISA 100100, 6mm. find the factored strength of the member if the angles are connected on both sides of 12mm gusset plate by**
   i) One bolt
   ii) Two bolt
   iii) Welding, which makes the joint rigid.

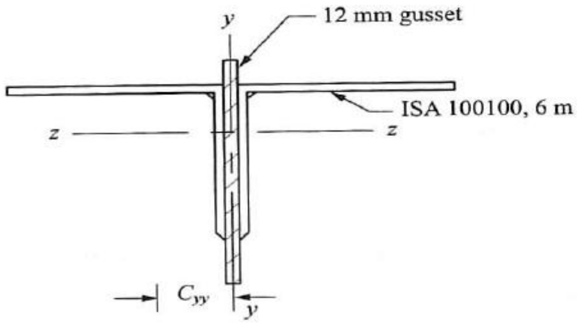
Solution:

From steel table for a ISA 100100, 6 mm,

area = 1167 mm^2; $C_{zz} = C_{yy} = 26.7$ mm.

$r_{zz} = r_{yy} = 30.9$ mm.

Figure shows the details of the member.



Figure

r_{zz} of the member is the same as r_{zz} of single angle, since the z-z axis for both is the same, resulting into doubling of I_{zz} and area.

$$\therefore \quad r_{zz} = 30.9 \text{ mm.}$$

$I_{yy} = 2\,[I_{yy}$ of one angle + Area of one angle $\times (C_{yy} + 6)^2]$.

From steel table, I_{yy} of one angle $= 111.3 \times 10^4$

$$I_{yy} \text{ of the member } = 2\left[111.3\times10^4 + 1167\times(26.7+6)^2\right]$$

$$= 4721723 \text{ mm}^4$$

$$\therefore \quad r_{yy} = \sqrt{\frac{I_{yy}}{A}} = \sqrt{\frac{4721723}{2\times1167}} = 44.98$$

$\therefore \quad r_{zz}$ is governing the strength of member.

Case (i): When a single bolt is used

$r = r_{zz} = 30.9$ mm

$KL = L = 3000$ mm.

$$\therefore \quad \frac{KL}{r} = \frac{3000}{30.9} = 97$$

The member belongs to buckling class c (Ref. Table 6.4)

Hence referring to Table 6.4(c), for $\frac{KL}{r} = 97$, corresponding to $f_y = 250$ MPa,

$$f_{cd} = 121 - \frac{7}{10}(121 - 107)$$

$$= 111.2 \text{ N/mm}^2.$$

$$\therefore \quad P_d = A_e f_{cd}$$

$$= 2 \times 1167 \times 111.2 = 259541 \text{ N.}$$

i.e. $P_d = 259.541$ kN **Answer**

Case (ii): When two bolts are used

The effective length is reduced. It may be taken as 0.85 times actual length.

$$\therefore \quad KL = 0.85 \times 3000 = 2550 \text{ mm.}$$

Hence in this case $\frac{KL}{r} = \frac{2550}{30.9} = 82.5$

From Table 6.4(c), for steel with $f_y = 250$ N/mm^2,

f_{cd} for $\frac{KL}{r} = 80$ is 136 N/mm^2

for $\frac{KL}{r} = 90$ is 121 N/mm^2

$\therefore$ Linearly interpolating, f_{cd} for $\frac{KL}{r} = 82.5$ is

$$f_{cd} = 136 - \frac{2.5}{10} \times (136 - 121)$$

$$= 132.25 \text{ N/mm}^2$$

$$\therefore \quad P_d = 2 \times 1167 \times 132.25$$

$$= 308672 \text{ N} = 308.672 \text{ kN}$$ **Answer**

Case (iii): Rigid joint by welding

Effective length $KL = 0.7 \times L = 0.7 \times 3000 = 2100$ mm

$$\therefore \quad \frac{KL}{r} = \frac{2100}{30.9} = 67.96$$

From the table, f_{cd} values are

for $\frac{KL}{r} = 60 \quad f_{cd} = 168 \text{ N/mm}^2$

$\frac{KL}{r} = 70 \quad f_{cd} = 152 \text{ N/mm}^2$

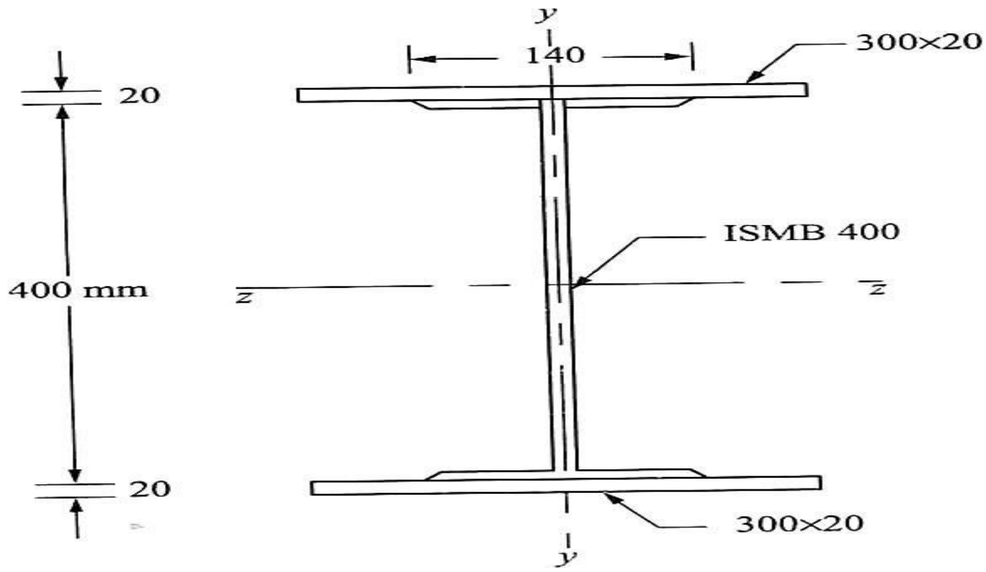
$\therefore$ For $\frac{KL}{r} = 67.96, \quad f_{cd} = 168 - \frac{7.96}{10}(168 - 152)$

$= 155.26 \text{ N/mm}^2$

$\therefore \quad P_d = 2 \times 1167 \times 155.26 = 362386 \text{ N}$

$= 362.386 \text{ kN}$ **Answer**

2. Determine the load carrying capacity of the column section shown in fig. If its actual length is 4.5m.its one end may be assumed fixed and other end hinged. The grade of steel is Fe415.



Solution:

For ISMB 400,

$h = 400$ mm, $\quad b_f = 140$ mm, $\quad t_f = 16$ mm, $\quad I_{zz} = 20458.4 \times 10^4 \text{ mm}^4$

$I_{yy} = 622.1 \times 10^4 \text{ mm}^4$, $\quad$ Area = 7846 mm^2.

$\frac{h}{b_f} = \frac{400}{140} > 1.2$ and $t_f = 16 \text{ mm} \leq 40 \text{ mm}$

Hence it belongs buckling class (Table 6.1) a about z-z axis and to b about y-y axis.

Sectional Properties:

$$I_{zz} = 20458.4 \times 10^4 + 2 \times 300 \times 20 \times (200 + 10)^2$$
$$= 733784000 \text{ mm}^4$$

(*Note:* M.I. of plate about its own axis neglected)

$$I_{yy} = 622.1 \times 10^4 + \frac{1}{12} \times 20 \times 300^3 \times 2$$
$$= 96221000 \text{ mm}^4$$
$$\therefore \quad I_{zz} < I_{yy}.$$

Buckling about z-z axis governs the design.

$$r = r_{zz} = \sqrt{\frac{I_{zz}}{A}}$$
$$A = 7846 + 2 \times 300 \times 20 = 19846 \text{ mm}^2$$
$$\therefore \quad r = r_{zz} = \sqrt{\frac{733784000}{19846}} = 192.28$$

Effective length $KL = 0.8\, L = 0.8 \times 4500 = 3600$ mm.

$$\therefore \quad \text{Slenderness ratio } \frac{KL}{r} = \frac{3600}{192.28} = 18.72$$

Referring to Table 6.4(b), for $\frac{KL}{r} = 18.72$

$$f_{cd} = 227 - \frac{8.72}{10}(227 - 224)$$
$$= 224.384$$
$$\therefore \quad P_d = A\, f_{cd} = 19846 \times 224.384$$
$$= 4453125 \text{ N}$$
$$= 4453.125 \text{ kN}.$$

$\therefore$ Load (working load) carrying capacity of the column $= \frac{4453.125}{1.5} = 2968.75$ kN

3. A column am long has to support a factored load of 6000kN.the column is effectively held at both and restrained in direction at one of the ends. Design the column using beam section and plates.

Solution:

Assuming $f_{cd} = 200$ N/mm^2,

$$\text{Area required} = \frac{6000 \times 10^3}{200} = 30000 \text{ mm}^2$$

Using ISHB 450 @ 907 N/m,

Area provided = 11789 mm^2, width of flange = 250 mm.

$\therefore$ Area to be provided by plates = 30000 − 11789 = 18211 mm^2.

Selecting 20 mm plates, breadth required 'b' is obtained from,

$$2\,b \times 20 = 18211$$

$$b = 455.3$$

Provide 20 mm × 500 mm plate.

Check for overhang:

Overhang $\frac{500-250}{20} = 12.5 < 16t$

Total area provided

$$A_e = 11789 + 500 \times 20 \times 2$$
$$= 31789 \text{ mm}^2$$

ISHB 450 @ 907 N/m

$$I_{zz} = 40349.9 \times 10^4 \text{ mm}^4$$
$$I_{yy} = 3045 \times 10^4 \text{ mm}^4$$

For the section selected,

$$I_{zz} = 40349.9 \times 10^4 + 2 \times 500 \times 20\,(225 + 10)^2$$
$$= 1507.994 \times 10^6 \text{ mm}^4$$

$$I_{yy} = 3045 \times 10^4 + 2 \times \frac{1}{12} \times 20 \times 500^3$$

$$= 447.1167 \times 10^6 \text{ mm}^4$$

$$\therefore \quad r = r_{yy} = \sqrt{\frac{I_{yy}}{A}} = \sqrt{\frac{447.1167 \times 10^6}{31789}}$$
$$= 118.6 \text{ mm}$$

Effective length $KL = 0.8\,L = 0.8 \times 4000 = 3200$ mm.

$$\therefore \quad \frac{KL}{r} = \frac{3200}{118.6} = 26.98$$

$t_f = t_f$ of I section + 20 = 13.7 + 20 = 31.7 < 40 mm.

It belongs to buckling class c for buckling about y-y axis.

$\therefore$ From Table 6.1(c)

$$f_{cd} = 224 - \frac{6.98}{10}(224 - 211)$$
$$= 214.9 \text{ N/mm}^2$$

$$\therefore \quad P_d = A_e f_{cd} = 31789 \times 214.9$$
$$= 6831456 \text{ N}$$
$$= 6831.456 \text{ kN} > \text{Factored load}$$

Hence safe.

4. Design a single angle strut connected to the gusset plate to carry 180Kn factored load. The length of the strut between centre to centre intersections is 3m.

Solution:

Assuming $f_{cd} = 90$ N/mm^2,

$$A = \frac{180 \times 10^3}{90} = 2000 \text{ mm}^2$$

Try ISA 9090, 12 mm, which has $A = 2019$ mm^2

$$r_{\min} = r_{vv} = 17.4 \text{ mm.}$$

Assuming the strut will be connected to the gusset plate with at least 2 bolts (Note: Strength of 20 mm bolt in single shear is about 45 kN)

$$KL = 0.85L = 0.85 \times 3000 = 2550 \text{ mm}$$

$$\therefore \quad \frac{KL}{r} = \frac{2550}{17.4} = 145.55$$

From the Table 6.4(c),

$$f_{cd} = 66.2 - \frac{5.55}{10}(66.2 - 59.2) = 66.32 \text{ N/mm}^2$$

$$\therefore \quad P_d = A f_{cd} = 2019 \times 66.32 = 125813 < 180000 \text{ N}$$

Hence revise the section.

Try ISA 130130, 8 mm.

Area provided $= 2022 \text{ mm}^2$, $r = r_{vv} = 25.5$

$$\therefore \quad \frac{KL}{r} = \frac{2550}{25.5} = 100$$

$$\therefore \quad f_{cd} = 107 \text{ N/mm}^2$$

$$\therefore \quad P_{cd} = 2022 \times 107 = 216354 > 180{,}000 \text{ N.}$$

Provide ISA 130130, 8 mm.

5. Design a laced column with two channels back to back of length 10m to carry an axial factored load of 1400kN.the column may be assumed to have restrained in position but not in direction at both ends.

Solution:

Assuming $f_{cd} = 135 \text{ N/mm}^2$

Area required $= \dfrac{140 \times 1000}{135} = 1037 \text{ mm}^2$

Try 2 ISMC 350 @ 413 N/m.

Area provided $= 2 \times 5366 = 10732 \text{ mm}^2$

$$r_{zz} = 136.6 \text{ mm}$$

Distance will be maintained so as to get $r_{yy} > r_{zz}$.

$$\therefore \quad \text{Actual} \quad \frac{KL}{r} = \frac{1 \times 10000}{136.6} = 73.206$$

Since it is a laced column

$$\frac{KL}{r} = 1.05 \times 73.206 = 76.87$$

From Table 6.4(c),

$$f_{cd} = 152 - \frac{6.87}{10}(156 - 136)$$
$$= 138.26 \text{ N/mm}^2$$

Load carrying capacity $= 10732 \times 138.26$

$= 1483.806 \times 10^3$

$= 1483.806 \text{ kN} > 1400 \text{ kN}$

Spacing between the channels:

Let it be a clear distance 'd',

Now: $I_{xx} = 2 \times 10008 \times 10^4 = 20016 \times 10^4$ mm^4

$$I_{yy} = 2\left[430.6\times10^4 + 5366\left(\frac{d}{2}+24.4\right)^2\right]$$

Equation I_{yy} to I_{xx}, we get

$$2\left[430.6\times10^4 + 5366\left(\frac{d}{2}+24.4\right)^2\right] = 20016\times10^4$$

i.e. $\left(\frac{d}{2}+24.4\right)^2 = 17848.3$

$\therefore \quad d = 218.4$ mm

Provide $d = 220$ mm as shown in Fig. 6.7.

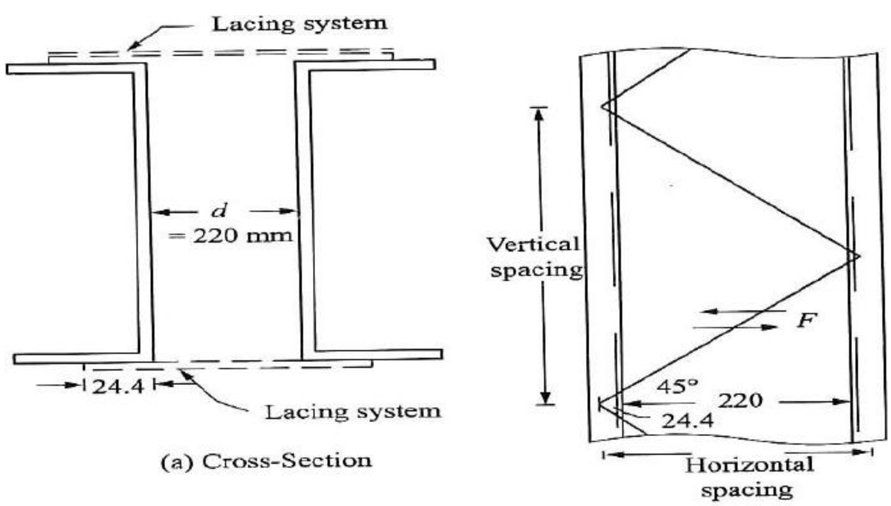
Lacings: Let the lacings be provided at 45° to horizontal.

Horizontal spacing of lacing = 220 + 60 + 60

= 340 mm [Note: $g = 60$ is gauge distance.]

$\therefore$ Vertical spacing = 340 sin 45° × 2

= 680 mm



(a) Cross-Section

(b) Elevation

Least r of one channel $= r_{yy} = 28.3$

$$\therefore \quad \frac{KL}{r} \text{ of channel between lacing } = \frac{680}{28.3} = 24.03 < 50$$

Transverse shear to be resisted by lacing systems $= \frac{2.5}{100} \times 1400 \times 10^3 = 35000$ N .

Shear to be resisted by each lacing systems $= \frac{35000}{2} = 17500$ N .

Length of lacing $= (240 + 60 + 60)\ \frac{1}{\cos 45} = 480.83$ mm .

Minimum thickness of lacing $= \frac{1}{40} \times 480.83$

$= 12.02$ mm.

Use 14 mm flats

Minimum width of lacing, if 20 mm bolts are used $= 3 \times 20 = 60$ mm.

Use 60 ISF 14

Sectional area $= 60 \times 14 = 840\ \text{mm}^2$.

$$r_{\min} = \sqrt{\frac{\frac{1}{12} \times 60 \times 14^3}{60 \times 14}} = 4.041 \text{ mm}$$

$$\therefore \quad \frac{KL}{r} = \frac{480.83}{4.041} = 118.97 < 145$$

Strength of 20 mm shop bolt:

(a) in single shear $= 0.78 \times \frac{\pi}{4} \times 20^2 \times \frac{400}{\sqrt{3} \times 1.25} = 45272$ N

Edge distance $= \frac{60}{2} = 30$

$$\therefore \quad K_b = \frac{30}{3 \times 22} = 0.4545$$

$$\text{Strength in bearing} = \frac{2.5 K_b\, d\, t\, f_u}{1.25}$$

$$= \frac{2.5 \times 0.4545 \times 20 \times 14 \times 400}{1.25}$$

$$= 101808 \text{ N}$$

$\therefore$ Bolt value = 45272.

$$\text{Number of bolt required } = \frac{17500}{45272} = 0.387$$

Provide one bolt.

Check for the strength of lacing:

$$\frac{KL}{r} = 118.97$$

A flat belongs to buckling class c.

$$\therefore \quad f_{cd} = 94.6 - \frac{8.97}{10}(94.6 - 83.7)$$

$$= 84.82 \text{ N/mm}^2$$

Load carrying capacity in compression = $84.82 \times 60 \times 14 = 71251$ N

$$\text{Force in lacing } = \frac{17500}{\sin 45} = 24749 \text{ N} < 71251 \text{ N}$$

$\therefore$ Safe.

Hence provide 60 ISF 10 flats at 45° and connect them to centre of gravity of channels with one bolt of 20 mm nominal diameter.

6. Design the built up section as shown in fig. Using battens.

Solution:

The design of column is same as in the previous example i.e. use 2ISMC 350 @ 413 N/m with clear spacing of 220 mm. $\frac{KL}{r} = 1.1 \times \frac{10000}{136.6} = 80.52$

Distance between centres of channels $S = 220 + 24.4 + 24.4 = 268.8$ mm

Design of battens:

Let C be the spacing of battens, longitudinally.

Radius of gyration of one channel = 28.3 mm

$$\therefore \quad \frac{C}{28.3} < 50 \quad \text{i.e.} \quad C < 1415.$$

It should also satisfy the condition,

$$\therefore \quad \frac{C}{28.3} < 0.7 \times 80.52 \quad \text{i.e. } C < 1595.$$

Let us select $C = 1200$ mm.

$$V_t = \frac{2.5}{100} \times 1400 \times 10^3 = 35000 \text{ N}$$

$$\therefore \quad V_b = \frac{V_t C}{NS} = \frac{35000 \times 1200}{2 \times 268.8} = 78125 \text{ N}$$

$$M = \frac{V_t C}{2N} = \frac{35000 \times 1200}{2 \times 2} = 10500000 \text{ N-mm}$$

Size of battens:

Effective depth of end batten $\not< 268.8$ mm and also $\not< 2 \times 100$ mm.

$\therefore$ Provide 270 mm depth for end battens, overall depth $= 270 + 2 \times 35 = 340$ mm.

For intermediate battens it is $\not< \frac{3}{4} \times 268.8$ mm and $\not< 200$ mm.

Provide depth = 210 mm

Giving edge distance of 35 mm,

Overall depth $= 210 + 2 \times 35 = 280$ mm

$$\text{Thickness of battens} \not< \frac{1}{50} \times 268.8$$

$$\not< 5.36$$

Use 6 mm thick plates.

Check for stresses in batten plates:

$$\text{Shear stress } = \frac{78125}{280 \times 6} = 46.5 \text{ N/mm}^2 < \frac{f_y}{\sqrt{3}} \times \frac{1}{1.1}$$

$$\text{Bending stress } = \frac{6M}{td^2} = \frac{6 \times 10500000}{6 \times 280^2} < \frac{f_y}{1.1}$$

$$= 133.9 < 227.27 \text{ N/mm}^2$$

Obviously end plate satisfies these requirements since it is deeper.

Connections:

It is to be designed to transmit both shear and bending moment.

Using 20 mm bolts,

Strength in single shear $= 0.78 \frac{\pi}{4} \times 20^2 \times \frac{400}{\sqrt{3} \times 1.25} = 45272$ N

Strength in bearing is much higher.

$\therefore$ Bolt value = 45272 N.

Number of bolts required $= \frac{78125}{45272} = 1.72$

Let us provide 3 bolts to take into account stresses due to bending also.

Check:

Force in each bolt due to shear $= \frac{78125}{3} = 26042$ N

Let the pitch be $\frac{210}{2} = 105$ mm

Force due to moment $= \frac{Mr}{\sum r^2}$

$$= \frac{10500000 \times 105}{105^2 + 105^2} = 50000$$

Resultant force $= \sqrt{26042^2 + 50000^2} > 45272$ N

Try 5 bolts as shown in Fig. 6.8.

i.e. Force in each bolt due to shear $= \frac{78125}{5} = 15625$ N

Force due to moment in extreme bolt $= \frac{Mr}{\sum r^2} = \frac{10500000 \times 100}{2\left(50^2 + 100^2\right)} = 42000$ N

$\therefore$ Resultant force $= \sqrt{15625^2 + 42000^2}$

$$= 44812 \text{ N} < 45272 \text{ N}$$

Provide the bolts as shown in Fig.

7. A column section ISHB 300@577 N/m is carrying a factored axial load of 600 kN. A factored moment of 30N-m and a factored shear force of 60 kN. Design a suitable column splice. Assume ends are milled.

Solution:

Since the ends are milled, 50% of axial load is transferred through bearing and splice plates transfer the remaining 50% of the load.

$\therefore$ Load to be transferred by splice plate = 300 kN

i.e. load to be transferred by each splice plate = 150 kN

Assuming the thickness of splice plate 6 mm, for the calculation of lever arm,

$a = 300 + 6 = 306$ mm.

$$\therefore \quad \text{Force in each plate due to moment } = \frac{30 \times 10^3}{306} = 98.04 \text{ kN}$$

$\therefore$ Total load in each splice plate = 150 + 98.04 = 248.04 kN

For rolled steel section, $f_y = 250$ N/mm^2.

$$\text{Area required } = \frac{248.04 \times 10^3}{250/1.1} = 1091.376 \text{ mm}^2$$

Width of splice plate = width of flange = 250 mm.

$$\therefore \quad \text{Thickness required } = \frac{1091.376}{250} = 4.365 \text{ mm}$$

Provide 6 mm plates.

Using 20 mm bolts of grade 4.6,

$$\text{Strength in single shear } = 0.78 \times \frac{\pi}{4} \times 20^2 \times \frac{400}{\sqrt{3} \times 1.25} = 45272 \text{ N}$$

Strength in bearing is much higher, if more than minimum specified edge distance is provided.

$\therefore$ Bolt value = 45272 N

$$\text{Number of bolts required } = \frac{248.04 \times 10^3}{45272} = 5.47$$

Provide 3 bolts on each side of web.

To resist the shear splice plates are provided on each side of web.

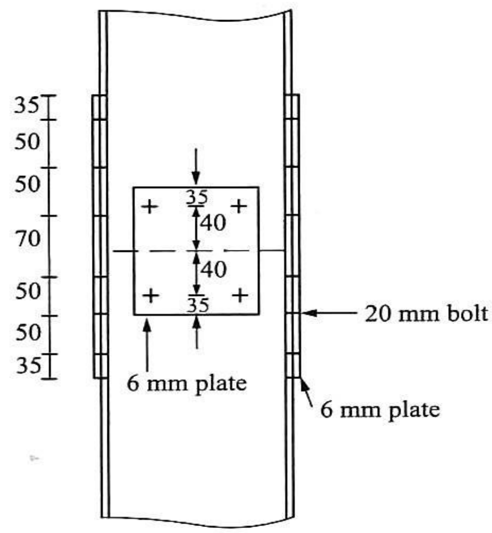
Maximum shear force = 60 kN.

Yield strength of the web [clause 8.4.1 in IS 800] $= \frac{f_{yw}}{\sqrt{3}} = \frac{250}{\sqrt{3}} = 144.3 \text{ N/mm}^2$

Area of plate required $= \frac{60 \times 10^3}{144.3} = 416 \text{ mm}^2$

Number of bolts required $= \frac{60 \times 10^3}{45272} = 1.32$

Provide 2 bolts. Using 6 mm web splice the arrangement may be made as shown in Fig.



8.a upper storey column ISHB 300 @577N/m carries a factored load of 1200 kN and a factored moment of 12 kN-m.it is to be spliced with lower storey column ISHB 400 @806N/m. design a suitable splice.

Solution:

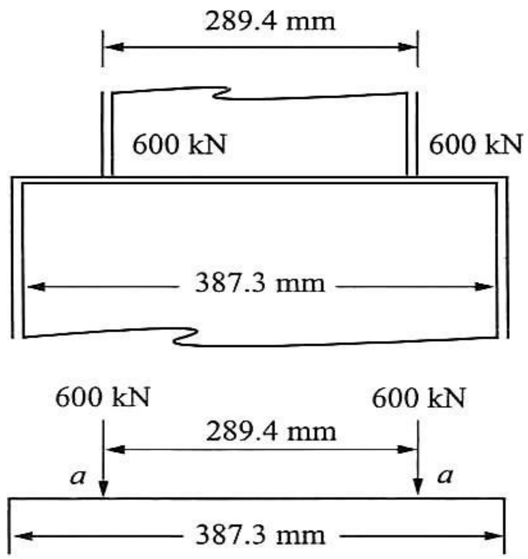
Assuming that column load is transferred by flanges only,

Load on each flange $= \frac{1200}{2} = 600$ kN.

Distance between the flanges of ISHB 300 @ 577 N/m = 300 − 10.6 = 289.4 mm

$\therefore$ Distance between the flanges in ISHB 400 @ 806 N/m = 400 – 12.7 = 387.3 mm.

$$\therefore \quad \text{Distance } a = \frac{387.3 - 289.4}{2} = 48.95 \text{ mm}$$



$\therefore$ Moment in bearing plate = 600×48.95 kN-mm

Width of bearing plate = Width of flange

= 250 mm

Design bending stress $= \dfrac{f_y}{\gamma_{mo}} = \dfrac{250}{1.1} = 227.27 \text{ N/mm}^2$

Equating moment of resistance to bending moment, we get,

$$\frac{1}{6} b t^2 f_{bs} = M$$

$$\frac{1}{6} \times 250\, t^2 \times 227.27 = 600 \times 48.95 \times 10^3$$

$$\therefore \quad t = 55.43 \text{ mm}$$

Adopt 56 mm thickness.

Splice Plate:

Column ends are milled for complete bearing. Hence splice plates are designed for 50% of load

Load on splice plates $= \dfrac{1200}{2} = 600$ kN.

$\therefore$ Load on each splice plate = 300 kN.

Assuming 6 mm thick plates, distance between splice plates = 400 + 6 = 406 mm.

$\therefore$ Load due to bending $= \dfrac{12 \times 10^3}{406} = 29.6$ kN

Hence, Total load = 300 + 29.6 = 329.6 kN

Width of splice plate = Width of column

= 250 mm.

$\therefore$ Thickness of splice plate $= \dfrac{329.6 \times 1000}{250 \times 227.27} = 5.8$ mm

Provide 6 mm plates.

Bolts: Using 20 mm bolts

Strength in single shear $= 0.78 \times \dfrac{\pi}{4} \times 20^2 \times \dfrac{400}{\sqrt{3}} \times \dfrac{1}{1.25} = 45272$ N

Bearing strength is higher as long as minimum end distance is maintained.

$\therefore$ Bolt value = 45272 N.

No. of bolts required $= \dfrac{329.4 \times 1000}{45272} = 7.3$

Provide 8 bolts in two rows on each plate as shown in Fig. 6.13.

Thickness of filler plate = $\dfrac{400 - 300}{2} = 50$ mm > 6 mm.

$\therefore$ According to clause 10.3.3.3,

$$\beta_{pk} = 1 - 0.0125\, t_{pk}$$
$$= 1 - 0.0125 \times 50 = 0.375$$

$\therefore$ Shear capacity of bolt = (0.375) 45272

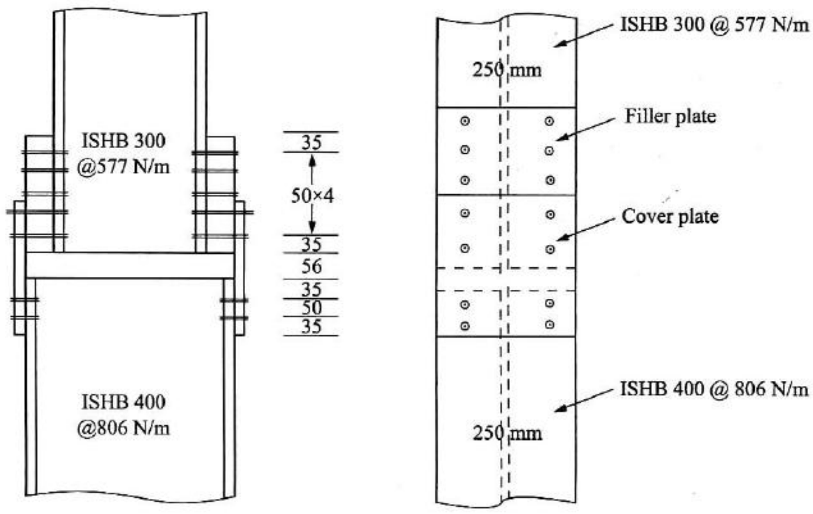
= 16977 N

$\therefore$ Number of bolts required to connect splice plate with ISHB 300 @ 577 N/m

$$= \frac{329.4 \times 1000}{16977} = 19.4$$

$\therefore$ Provide 12 additional bolts, 6 on each side in the filler portion.

The details of connection are shown in :



(a) Elevation (b) Side view

9. Design a slab base for a column ISHB 300 @577N/m carrying an axial factored load of 1000 kN.M20 concrete is used for the foundation. Provide welded connection between column and base plate.

Solution:

Bearing strength of concrete $= 0.45 f_{ck}$

$$= 0.45 \times 20 = 9 \text{ N/mm}^2$$

Factored load $P_u = 1000$ kN.

$$\therefore \quad \text{Area of base plate required } = \frac{1000 \times 10^3}{9} = 111111 \text{ mm}^2$$

Provide 360×310 size plate.

Area provided $= 360 \times 310 = 111600 \text{ mm}^2$.

$$\text{Pressure } = \frac{1000 \times 10^3}{111600} = 8.96 \text{ N/mm}^2$$

Projections are

$$a = \frac{360 - 300}{2} = 30 \text{ mm}$$

$$b = \frac{310 - 250}{2} = 30 \text{ mm}$$

$$\therefore \quad t_s = \left[\frac{2.5 \times 8.96 \left(30^2 - 0.3 \times 30^2 \right) \times 1.1}{250} \right]^{0.5}$$

$$= 7.88 \text{ mm.}$$

Thickness of flange of ISHB 300 @ 577 N/m is 10.6 mm.

Provide 12 mm thick plate.

Connecting 360 × 310 × 12 mm plate to concrete foundation:

Use 4 bolts of 20 mm diameter 300 mm long to anchor the plate.

Welds: Properly machined column is to be connected to base plate using fillet weld.

Total length available for welding (Ref. Fig. 6.16)

$$= 2(250 + 250 - 7.6 + 300 - 10.6) = 1563.6 \text{ mm.}$$

Strength of weld $= \dfrac{410}{\sqrt{3}} \times \dfrac{1}{1.25} = 189.37 \text{ N/mm}^2$

Let t be the size of weld. Then effective area of weld $= 0.7\, t L_e$

where L_e is effective length.

$\therefore$ The design condition is $0.7\, t\, L_e \times 189.37 = 1000 \times 10^3$

$t L_e = 7543.8$

Using 6 mm weld, $L_e = 1257$ mm.

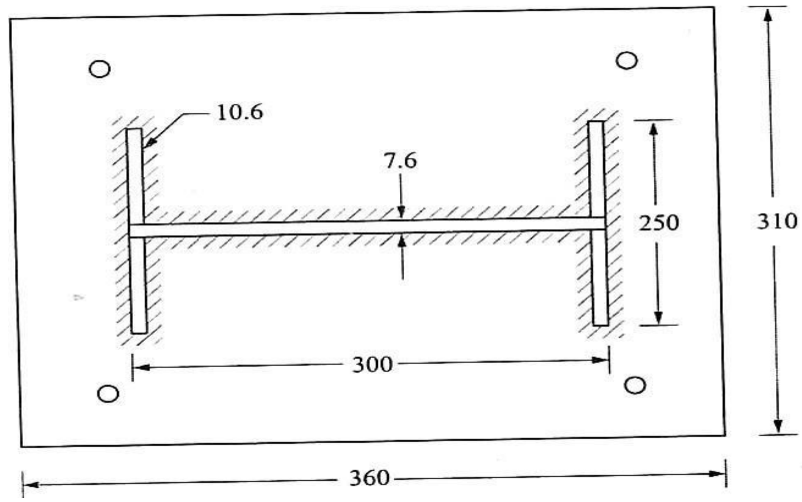
After deducting for end return of the weld at the rate of twice the size of the weld at each end.

Available effective length $= 1563.6 - 2 \times 6 \times$ No. of returns

$$= 1563.6 - 2 \times 6 \times 12$$

$$= 1419.6 > 1257 \text{ mm.}$$

Hence 6 mm weld is adequate.



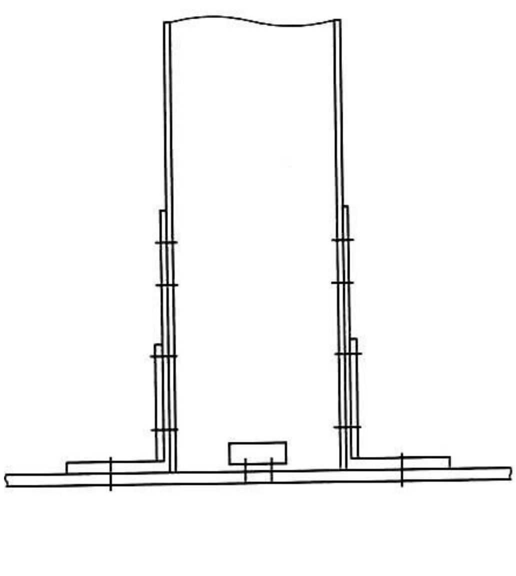
10. Design a gusseted base for a column ISHB 350 @710N/m with two plates 450mmx20mm carrying a factored load of 3600 kN. The column is to be supported on concrete pedestal to be built with M20 concrete.

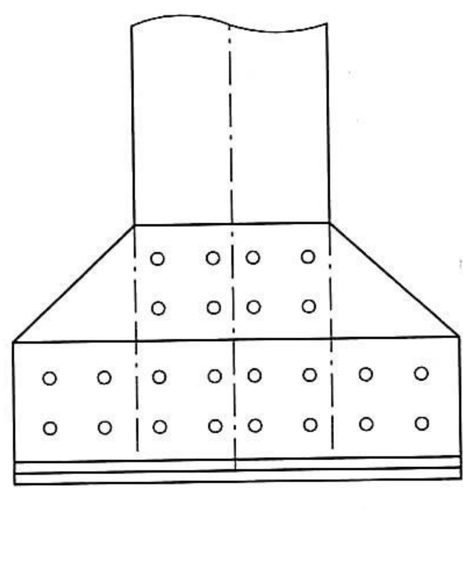
Solution:

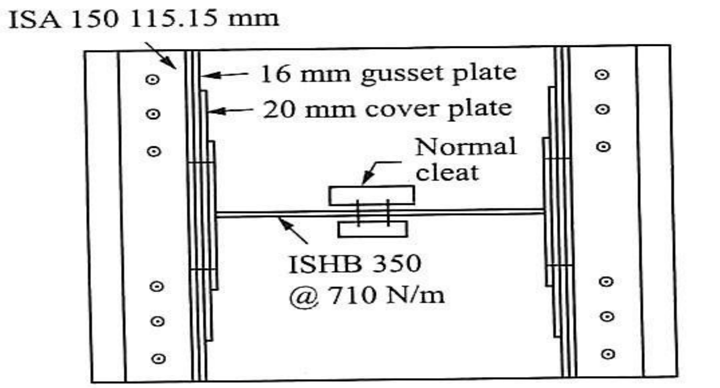
$f_{ck} = 20$ N/mm^2.

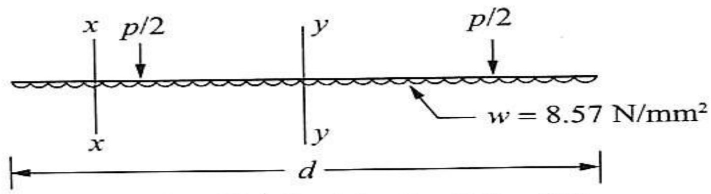
$$A = \frac{P_u}{0.45 f_{ck}} = \frac{3600 \times 10^3}{0.45 \times 20} = 400000 \text{ mm}^2$$

Selecting ISA 150115, 15 mm angle and 16 mm thick gusset plate









$d = 350 + 2 \times 20 + 2 \times 16 + 2 \times 115 = 652$ mm

Minimum width required $= 350 + 2 \times 20 + 2 \times 16 + 2 \times 115$
$= 652$ mm.

Use 700 mm wide plate.

$\therefore$ Length of base plate $= \dfrac{400000}{700} = 571$ mm

Provide 700×600 mm plate.

At section Y-Y, bending moment [Note: Per mm width $P = 8.57 \times 350$ N]

$$M_{yy} = 8.57 \times \frac{350^2}{2} - \frac{700}{2} \times 8.57 \times \left(\frac{350}{2} + 20 + \frac{16+15}{2} \right)$$

$= 106482$ N-mm

$\therefore$ Design moment $= 106482$ N-mm.

Bending strength $= \dfrac{f_y}{\gamma_{mo}} = \dfrac{250}{1.1} = 227.27$ N/mm^2

Equating moment of resistance to bending moment we get,

$$\frac{1}{6} \times 1 \times t^2 \times 227.27 = 10648^2$$

$\therefore$ $t = 53$ mm.

$\therefore$ Use 56 mm base plate of size 700×600 mm.

Assuming ends of columns are faced for complete bearing, the connection between gusset plate and column will be designed for 50 percent of axial load.

Design load $= 0.5 \times 3600 = 1800$ kN.

Load on each splice $= \dfrac{1800}{2} = 900$ kN.

Using 24 mm shop bolts,

Strength of bolt in single shear $= 0.78 \times \dfrac{\pi}{4} \times 24^2 \times \dfrac{400}{\sqrt{3}} \times \dfrac{1}{1.25}$

$= 65192$ N.

Strength in bearing is higher.

$\therefore$ Bolt value $= 65192$ N.

$\therefore$ No. of bolts required $= \dfrac{900 \times 10^3}{65192} = 13.8$

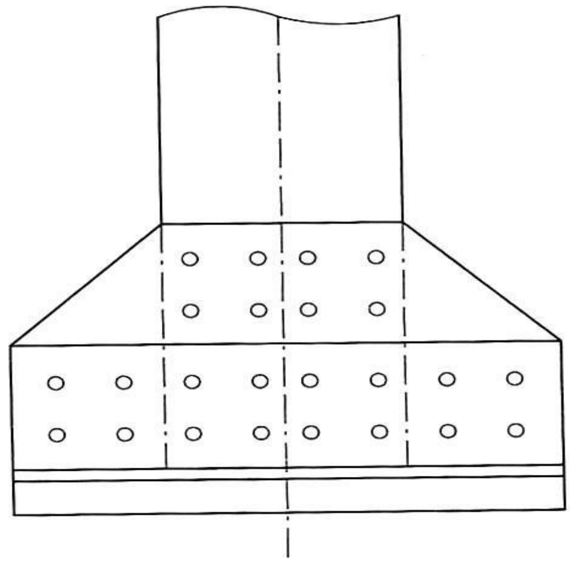
Provide 16 bolts as shown in Fig. , for connecting column to gusset plate. Use another 8 bolts to connect cleat angle to gusset plate.

Strength in bearing is higher.

$\therefore$ Bolt value $= 65192$ N.

$\therefore$ No. of bolts required $= \dfrac{900 \times 10^3}{65192} = 13.8$

Provide 16 bolts as shown in Fig. 6.18, for connecting column to gusset plate. Use another 8 bolts to connect cleat angle to gusset plate.



Estd: 2007

UNIT 3 – DESIGN OF BEAMS

1. A roof of a hall measuring 8mx12m consists of 100mm thick R.C slab supporting on steel I beam spaced 3m apart as shown in fig. The finishing load may be taken as 1.5kN/m^2.design the steel beam.

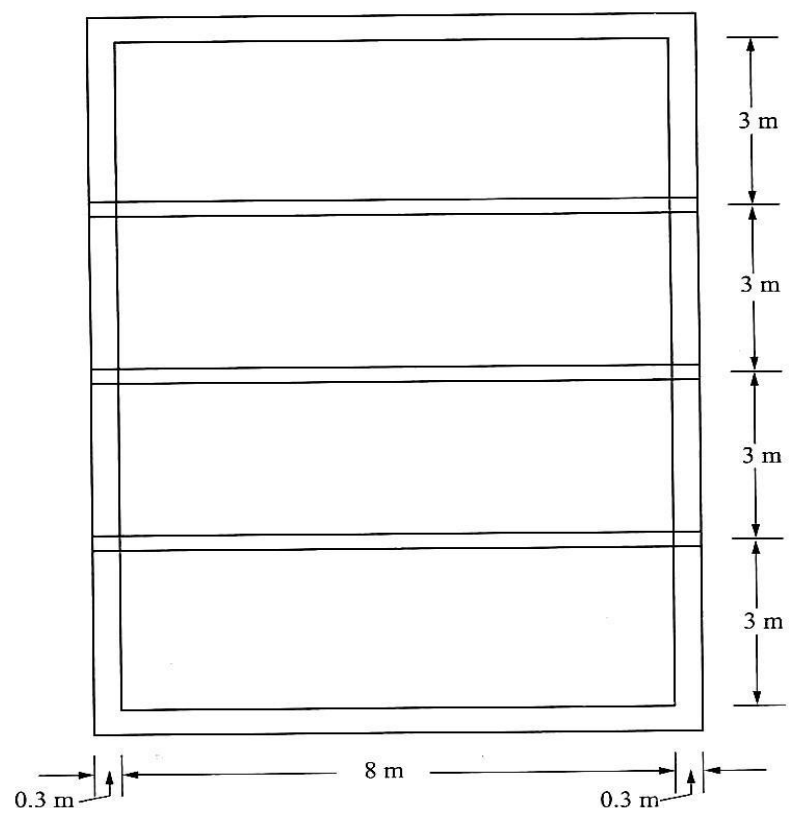
Solution:

Each beam has a clear span of 8 m and takes care of 3 m width of slab. Hence the load per metre length of the beam is as follows:

Weight of R.C. slab $= 0.1 \times 1 \times 3 \times 25 = 7.5$ kN/m

Finishing load $= 1.5 \times 3 = 4.5$ kN/m

Self weight (assumed) $= 0.8$ kN/m



$\therefore$ Total dead load $= 12.8$ kN/m.

Live load $= 1 \times 3 \times 1.5 = 4.5$ kN/m.

$\therefore$ Factored dead load $= 1.5 \times 12.8 = 19.2$ kN/m

Factored live load $= 1.5 \times 4.5 = 6.75$ kN/m

$\therefore$ Total factored load $= 25.95$ kN/m.

Effective span of the simply supported beam = centre to centre distance of supports

Assuming width of support = 0.3 m,

Effective span = 8 + 0.3 = 8.3 m.

$$\therefore \quad \text{Design moment, } M = \frac{wL^2}{8}$$

$$= \frac{25.95 \times 8.3^2}{8} = 223.46 \text{ kN-m}$$

Design shear force $V = \dfrac{25.95 \times 8.3}{2} = 107.61$ kN

$$\therefore \quad \text{Section modulus required} = \frac{M}{f_y} \times \gamma_{mo}$$

$$Z_p = \frac{223.46 \times 10^6 \times 1.1}{250} = 983224 \text{ mm}^3$$

Try ISMB 400 which has $Z_p = 1175.2 \times 10^3$ mm^3.

The properties of the section are as follows:

Depth of section $h = 400$ mm

Width of flange $b = 140$ mm

Sectional area $A = 7846$ mm^2

Thickness of flange $t_f = 16.0$ mm

Thickness of web $t_w = 8.9$ mm

Depth of web $d = h - 2\,(t_y + r_1)$

$$= 400 - 2(16 + 14) = 340 \text{ mm}$$

Moment of inertia about z-z axis

$$I_{zz} = 20458.4 \times 10^4 \text{ mm}^4$$

Elastic section Modulus $Z_e = 1020 \times 10^3$ mm^4

Section Classification:

$$\in = \left(\frac{250}{f_y}\right)^{1/2} = \left(\frac{250}{250}\right)^{1/2} = 1.0$$

$$\frac{b}{t_f} = \frac{140}{16} = 8.75 < 9.4 \in$$

$$\frac{d}{t_w} = \frac{340}{8.9} = 38.2 < 84 \in$$

Hence the section is classified as plastic section check for assumed self weight:

Weight of the section = 0.604 kN/m.

Assumed weight = 0.8 kN/m.

Difference is not much. Hence the design is continued with moments and shears calculated as earlier.

Check for shear strength:

Design shear $V = 107.61$ kN

Design shear strength of the section

$$V_d = \frac{f_y}{\sqrt{3}} \times \frac{1}{1.1} \times \text{shear area}$$
$$= \frac{f_y}{\sqrt{3}} \times \frac{1}{1.1} \times h \times t_w$$
$$= \frac{250}{\sqrt{3}} \times \frac{1}{1.1} \times 400 \times 8.9$$
$$= 467128 \text{ N} = 467.128 \text{ kN} > 107.61 \text{ kN}$$

Hence the section is adequate.

$$0.6\, V_d = 0.6 \times 467.128 = 280.277 > 107.61 \text{ kN}$$

Check for design moment capacity:

$\frac{d}{t_w} = 38.2$ which is less than $67\in$, since $\in = 1$.

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

$\beta_b = 1.0$ since it is plastic section.

$$\therefore \quad M_d = 1.0 \times 1175.2 \times 10^3 \times \frac{250}{1.1} = 267.091 \times 10^6 \text{ N-mm}$$

$$= 267.091 \text{ kN-m} > 223.46 \text{ kN-m}.$$

Hence adequate.

Check for deflection:

Total working load = 12.8 + 4.5 = 17.3 kN/m.

= 17.3 N/mm

Maximum deflection

$$\delta = \frac{5}{384}\frac{wL^4}{EI}$$

$$\therefore \quad \delta = \frac{5}{384} \times \frac{17.3 \times (8300)^4}{2 \times 10^5 \times 20458.4 \times 10^4}$$

$$= 26.127 \text{ mm.}$$

Permissible deflection for a beam in building (Ref. Table 7.2) = $\frac{l_e}{300} = \frac{8300}{300} = 27.67$ mm

Hence deflection is within the permissible limit.

$\therefore$ Provide ISMB 400.

2. Design a simply supported beam of effective span 1.5m carrying a factored concentrated load of 360 kN at mid span.

Solution:

Maximum moment occurs at mid span and is given by

$$M = \frac{WL}{4} = \frac{300 \times 1.5}{4} = 135 \text{ kN-m} = 140 \times 10^6 \text{ N-m}$$

$\therefore$ Z_p required is obtained from the relation $f_y \dfrac{Z_p}{\gamma_{mo}} = M$

or $$Z_p = \frac{135 \times 10^6}{250} \times 1.1 = 594.0 \times 10^3 \text{ mm}^3$$

Select trial section as ISMB 300 which has $Z_p = 651.7 \times 10^3$ mm^3.

The sectional properties of ISMB 300 are

Overall depth $h = 300$ mm.

Width of flange $b = 140$ mm

Thickness of flange $t_f = 12.4$ mm

Depth of web $d = h - 2\,(t_f + r_1)$

$= 300 - 2\,(12.4 + 14) = 247.2$ mm

Thickness of web $t_w = 7.5$ mm

$I_{zz} = 8603 \times 10^4$ mm^4

$Z_e = 573.6 \times 10^3$ mm^4

$Z_p = 651.7 \times 10^3$ mm^4

Self weight of beam = 0.452 kN/m.

$\therefore$ Factored weight = 1.5 × 0.452 kN/m

$\therefore$ Additional factored moment due to self at $= 1.5 \times 0.452 \times \frac{1.5^2}{8} = 0.190$ kN-m

$\therefore$ Total factored moment

$$M = 135 + 0.190 = 135.190 \text{ kN-m.}$$

Factored shear force due to self weight $= 1.5 \times 0.492 \times \frac{1.5}{8} = 0.508$ kN

$\therefore$ Total factored shear force on section $= \frac{350}{2} + 0.508 = 180.508\ kN$

Section Classification:

$$\in = \sqrt{\frac{250}{f_y}} = \sqrt{\frac{250}{250}} = 1$$

$$\frac{b}{t_f} = \frac{140}{12.4} = 11.29 < 15.7 \in$$

$$\frac{d}{t_w} = \frac{247.2}{7.5} = 32.96 < 84 \in$$

It is classified as a semi compact (class 3) section:

Shear capacity of the section:

$$V_d = \frac{f_y}{\sqrt{3}} \times \frac{1}{\gamma_{mo}} \times h \times t_w$$

$$= \frac{250}{\sqrt{3}} \times \frac{1}{1.1} \times 300 \times 7.5$$

$$= 295.235 \times 10^3 \text{ N} = 295.235 \text{ kN}$$

section is adequate to resist shear

$\therefore$ $0.6\ V_d = 0.6 \times 295.235 = 177.145$ kN

$\therefore$ $V > 0.6\ V_d$

Moment capacity of the section:

Since $V > 0.6\ V_d$ and the section belongs to class 3 (semi compact) category

$$M_d = M_{dv} = \frac{Z_e f_y}{\gamma_{mo}} = \frac{573.6\times10^3\times250}{1.1}$$

$$= 130.36 \times 10^6 \text{ N-mm} = 130.36 \text{ kN-m} < \text{M}$$

Hence the section is not safe. Revise the section. Try ISMB 350.

Sectional properties:

Overall depth $h = 350$ mm

Width of flange $b = 140$ mm

Thickness of flange $t_y = 14.2$ mm

Thickness of web $t_w = 8.1$ mm

Depth of web $d = 350 - 2\ (14.2 + 140) = 293.6$ mm

$Z_e = 778.9 \times 10^3\ \text{mm}^3$ $Z_p = 889.6 \times 10^3$ mm

$I_{zz} = 13630.3 \times 10^4\ \text{mm}^4$.

Section Classification:

$\in\ = 1.$

$$\frac{b}{t_f} = \frac{140}{14.2} = 9.86 \text{ between } 9.4\in \text{ and } 10.4\in$$

$$\frac{d}{t_w} = \frac{293.6}{8.1} = 36.24 \text{ between } 33.5\in \text{ and } 42\in$$

$\therefore$ It belongs to class 3 (semicompact) section.

Shear capacity of the section:

$$V_d = \frac{f_y}{\sqrt{3}}\times\frac{1}{\gamma_{mo}}\times h\times t_w$$

$$= \frac{250}{\sqrt{3}}\times\frac{1}{1.1}\times350\times8.1 = 371.997\times10^3 \text{ N}$$

$$= 371.997 \text{ kN} > 180.508 \text{ kN}$$

Section is adequate to resist shear

$$0.6\ V_d = 0.6 \times 371.997 = 223.198 \text{ kN} > 180.508 \text{ kN}$$

Moment capacity of the section:

Since $\frac{d}{t_w} < 67 \in$ and $V < 0.6V_d$,

$$\therefore \quad M_d = \beta_b Z_p f_y \times \frac{1}{\gamma_{mo}}$$

$$\beta_b = \frac{Z_e}{Z_p} = \frac{778.9\times10^3}{889.6\times10^3}$$

$$M_d = \frac{778.9\times10^3}{889.6\times10^3} \times 889.6\times10^3 \times 250 \times \frac{1}{1.1}$$

$$= 194.725\times10^6 \text{ N-mm}$$

$$= 194.725 \times 10^6 \text{ N-mm}$$

$$= 194.725 \text{ kN-m} > 135.19 \text{ ie. M.}$$

$\therefore$ The section is adequate to resist moment.

Maximum deflection corresponding to working load

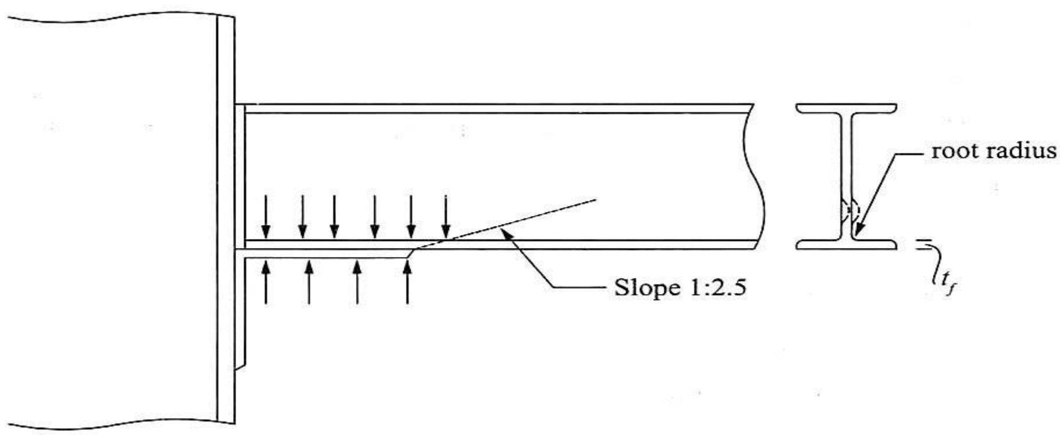
$$\delta = \frac{WL^3}{48EI} = \frac{360\times10^3\times1500^3}{48\times2\times10^5\times13630.3\times10^4}$$

$$= 0.928 \text{ mm} < \frac{1500}{300}$$

Hence section is adequate.

Hence use ISMB 350 as beam.

3.check the section shown in fig for web buckling and web crippling if stiff hearing is over a length b_1=75mm.



Solution:

Section selected was ISMB 400.

End reaction = End shear = 107.61 kN.

Stiff bearing at ends = 75 mm.

From steel table,

$t_w = 8.9$ mm, $t_f = 16.0$ mm,

radius at root = 14.0 mm.

Depth of section $h = 400$ mm.

$\therefore$ Depth of web $= 400 - 2\,(t_f + r_1) = 400 - 2\,(16 + 14)$

$= 340$ mm.

Check for web buckling:

$$\lambda = 2.5\,\frac{d}{t_w} = \frac{2.5 \times 340}{8.9} = 95.5$$

Hence from Table 9.c of IS 800-2007 (Table 6.4) we get,

$$f_c = 121 - \frac{5.5}{10}(121 - 107) = 113.3 \text{ N/mm}^2$$

$$n_1 = \frac{400}{2} = 200 \text{ mm}$$

$\therefore$ Web buckling resistance of the section,

$$F_{cdw} = (b_1 + n_1)\,t_w f_c$$

$$= (75 + 200) \times 8.9 \times 113.3 = 277.302 \times 10^3 \text{ N} = 277.302 \text{ kN} > 107.61 \text{ kN}$$

Hence the section is safe against web buckling.

Check for web crippling:

Flange thickness = 16.0 radius at root = 14.0

$\therefore$ $n_2 = 2.5\,(16 + 14) = 75$ mm.

$\therefore$ Strength of web against web crippling

$$F_w = (b_1 + n_2)\, t_w\, f_{yw} \times \frac{1}{\gamma_{mo}}$$

$$= (75 + 75)\, 8.9 \times 250 \times \frac{1}{1.1} = 303.409 \times 10^3 \text{ N}$$

$= 303.409$ kN > load transferred by bearing in this case (107.61 kN).

Hence safe.

4. Determine the uniformly distributed load carrying capacity of the welded plate girder shown in fig. When it is used as a cantilever beam of 4m effective span and checks it for shear, deflection, web buckling and web crippling. Assuming stiff bearing length as 100mm.

Solution:

Section Moduli

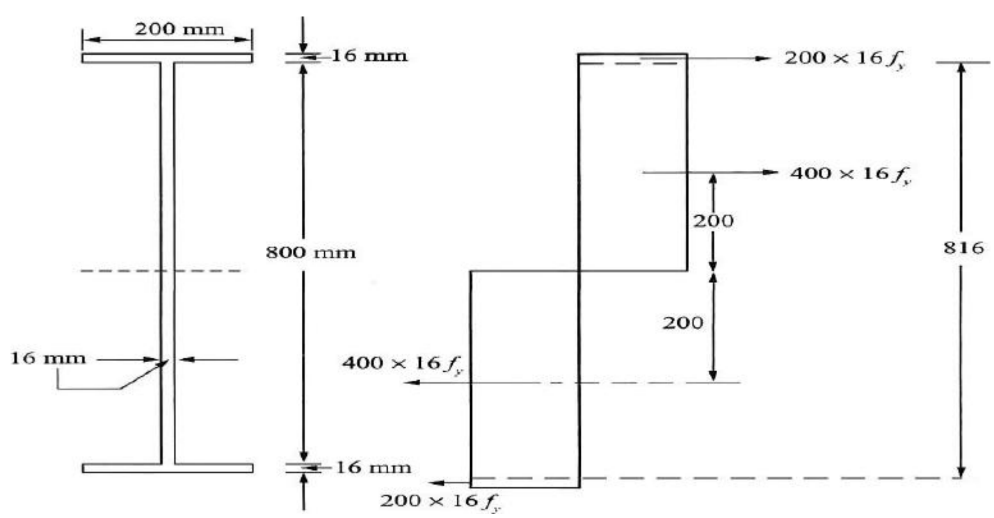
$$I_{zz} = \frac{1}{12}\left[200 \times 832^3 - 184 \times 800^3\right] = 1748.173 \times 10^6 \text{ mm}^4$$

$$\therefore \quad Z_e = \frac{I_{zz}}{Y_{\max}} = \frac{1748.173 \times 10^6}{\left(\frac{832}{2}\right)} = 4202.338 \times 10^3 \text{ mm}^3$$

Plastic N-A is at mid depth since stress is f_y (comp) in top half and f_y (tensile) in bottom half

$$M_p = (200 \times 16 \times 816 + 400 \times 16 \times 400)\, f_y$$

$$\therefore \quad Z_p = \frac{M_p}{f_y} = 5171.199 \times 10^3 \text{ mm}^3$$



Section classification:

$$\in = 1$$

$$\frac{b}{t_f} = \frac{200}{16} = 12.5, \qquad \text{between } 9.4t \text{ and } 13.6t$$

$$\frac{d}{t_w} = \frac{800}{16} = 50 \qquad < 84 \in$$

It belongs to semicompact class of section.

Trial section:

Assuming $V < 0.6\ V_d$,

$$\therefore \quad M_d = \beta_b Z_p f_y \frac{1}{\gamma_{mo}} = \frac{Z_e}{Z_p} Z_p f_y \frac{1}{\gamma_{mo}}$$

$$= Z_e f_y \frac{1}{\gamma_{mo}}$$

$$= \frac{4202.338 \times 250}{1.1} = 955.0768 \times 10^6 \text{ N-mm} = 955.0768 \text{ kN-m}$$

Let factored *udl* be wkN per metre length. Then,

$$M = \frac{wL^2}{2} = w \times \frac{4^2}{2} = 8w$$

Equating it to M_d, we get

$$8w = 955.0768$$

$$w = 119.385 \text{ kN/m.}$$

$$\therefore \quad \text{Shear} \quad V = wL = 119.385 \times 4 = 477.538 \text{ kN.}$$

Check for shear:

$$V_d = \frac{f_y}{\sqrt{3}} \times \frac{1}{\gamma_{mo}} \times h \times t = \frac{250}{\sqrt{3}} \times \frac{1}{1.1} \times 832 \times 16$$

$$= 1746.747 \times 10^3 = 1746.747 \text{ kN} > V \qquad \text{safe.}$$

$$0.6\ V_d = 286.523 \text{ kN} > V$$

Hence calculated M_d is correct.

Check for deflection:

For a cantilever beam

$$\delta = \frac{wL^4}{8EI_{zz}}$$

$$w = \text{working load} = \frac{119.38 \times 1000}{1.5 \times 100} \text{ N/mm}$$

$$= 79.587 \text{ N/mm.}$$

$$\therefore \quad \delta = \frac{79.587 \times (4000)^4}{8 \times 2 \times 10^5 \times 1748.173 \times 10^6}$$

$$= 7.28 \text{ mm} < \frac{4000}{300}$$

Hence safe.

Check for web buckling:

Slenderness ratio $\lambda = 2.5 \dfrac{h}{t_w} = 2.5 \times \dfrac{816}{16} = 127.5$

From Table 9(c), $f_c = 83.7 - \dfrac{7.5}{10}(83.7 - 74.3)$

$$= 76.65 \text{ N/mm}^2.$$

$$\therefore \quad F_{cdw} = (b_1 + n_1)\, t_w f_c = (100 + 416) \times 16 \times 76.65$$

$$= 632.822 \times 10^3 \text{ N}$$

$$= 632.822 \text{ kN} > V$$

$\therefore$ Web is safe.

Check for web crippling:

$$F_w = \left(b_1 + 2.5\, t_f\right) f_y \frac{1}{\gamma_{mo}} t_w$$

$$= \left(100 + 2.5 \times 16\right) \times 250 \frac{1}{1.1} \times 16$$

$$= 509.09 \times 10^3 \text{ N}$$

$$= 509.09 \text{ kN} > V$$

Hence safe.

$\therefore$ We conclude that the *load carrying capacity (factored) is 119.38 kN/m.*

5.design a simply supported beam of 10m effective span carrying a total load of 60 kN/m. the depth of beam should not exceed 500mm.the compression flange of the beam is laterally supported by floor construction. Assume stiff end bearing is 75mm.

Solution:

$L = 10 \text{ m} = 10000 \text{ mm}, w = 60 \text{ kN/m}$

Trial Section:

$$\text{Maximum } BM, M = \frac{wL^2}{8} = \frac{60 \times 10^2}{8} = 750 \text{ kN-m}$$

$$= 750 \times 10^6 \text{ N-mm.}$$

$$\therefore \quad Z_p \text{ required} = \frac{M \gamma_{mo}}{f_y} = \frac{250 \times 10^6 \times 1.1}{250}$$

$$= 3300 \times 10^3 \text{ mm}^3.$$

Since depth restricted is 500 mm, select ISMB 450 and suitable plates over flanges.

Z_p of ISMB 450 = 1553.4×10^3

Z_p to be provided by cover plates

$$= (3300 - 1553.4) \times 10^3$$

$$= 1746.6 \times 10^3 \text{ mm}^3$$

If A_p is the area of cover plate on each side tensile force and compressive forces developed at the time of hinge formation = $A_p f_y$

If the distance between the two plates is 'd', plastic moment resisted = $A_p f_y d$.

Hence the additional Z_p provided by the cover plates may be taken as

$$Z_p \text{ of plates} = \frac{A_p f_y d}{f_y} \times \frac{1}{\gamma_{mo}} = \frac{A_p d}{1.1}$$

$$\therefore \quad \frac{A_p d}{1.1} = 1746.6 \times 10^3$$

Taking $d = 450 + t \approx 450$ mm.

we get $A_p = \dfrac{1746.6 \times 10^3 \times 1.1}{450} = 4269.5 \text{ mm}^2$

Provide 220×20 mm plates on either side.

Check for shear:

$$V_d = \frac{f_y}{\sqrt{3}} \times \frac{1}{\gamma_{mo}} \times h \times t_w = \frac{250}{\sqrt{3}} \times \frac{1}{1.1} \times 450 \times 9.4$$

$$= 555.044 \times 10^3 \text{ N} = 559.044 \text{ kN}$$

$$0.6 V_d = 333.026 \text{ kN}$$

$$V = 60 \times \frac{10}{2} = 300 \text{ kN}$$

Hence section is safe in shear.

Section classification: Internal element of compression flange:

$$\frac{b}{t_f} = \frac{150}{17.4} = 8.6 < 29.3 \in \text{and}$$

$$\frac{d}{t_w} = \frac{450 - 2(17.4 + 15)}{9.4} = 40.9 < 84 \in$$

Hence plastic section.

$$\therefore \quad M_d = 1.0 \times \frac{Z_p f_y}{\gamma_{mo}}$$

$$= \left(Z_p \text{ of I section} + Zp \text{ of plates}\right) \frac{f_y}{\gamma_{mo}}$$

$$= \left[1553.4 \times 10^3 + 220 \times 20 \times (450 + 20)\right] \times \frac{250}{1.1}$$

$$= 823.0455 \times 10^6 \text{ N.mm}$$

$$= 823.0455 \text{ kN.m} > \text{M}. \quad \text{Hence safe.}$$

Check for deflection:

Working load $= \dfrac{60}{1.5}$ kN/m = 40 kN/m = 40 N/mm

$$\therefore \quad \delta = \frac{5wL^4}{384EI_{zz}} = \frac{5\times 40\times (10000)^4}{384\times 2\times 10^5\times I_{zz}}$$

where

$$I_{zz} = I_{zz} \text{ of ISMB 400} + I_{zz} \text{ due to plates}$$

$$= 30390.8\times 10^4 + 2\times A_p\left(\frac{d}{2}\right)^2$$

$$= 30390.8\times 10^4 + 2\times 20\times 220\,(225+10)^2$$

$$= 789.888\times 10^6 \text{ mm}^4$$

$$\therefore \quad \delta = \frac{5\times 40\times (10000)^4}{384\times 2\times 10^5\times 789.884\times 10^6} = 32.97 \text{ mm}$$

δ permissible, if elastic cladding is assured

(Table 6 of IS 800), $\dfrac{L}{240} = \dfrac{10,000}{240} = 41.67$ mm

Hence safe.

Check for web buckling:

$h = 450$ mm.

$$\therefore \quad \lambda = 2.5\,\frac{h}{t_w} = 2.5\times\frac{450}{9.4} = 119.68$$

From Table 9.c of IS 800-2007

$$f_{cd} = 94.6 - \frac{9.68}{10}(94.6 - 83.7) = 84.13 \text{ N/mm}^2$$

$$\therefore \quad F_{cdw} = (b_1 + n_1)\,t_w\,f_{cd} = \left(75 + \frac{450}{2}\right)9.4\times 84.13$$

$$= 237.246\times 10^3 \text{ N} < 300 \text{ kN}.$$

Hence needs increased effective bearing length b. Provide $b = 175$ mm Then

$$F_{cdw} = \left(175 + \frac{450}{2}\right) \times 9.4 \times 84.13$$

$$= 316.94 \times 10^3 > 300 \text{ kN.} \quad \therefore \quad \text{Safe.}$$

Check for web crippling:

$$F_w = \left(b_1 + 2.5 t_f\right) f_y \frac{1}{\gamma_{mo}} t_w$$

$$= \left[175 + 2.5 \times (17.4 + 20)\right] 250 \times \frac{1}{1.1} \times 9.4$$

$$= 573.614 \times 10^3 \text{ N} > 300 \text{ kN} \quad \text{Hence safe.}$$

Design of connection between flange plates and flange:

Bolts/welds joining the plates and flange are to be designed for the horizontal shear at that level.

Shear stress at the level of plates and flanges

$$= \frac{F}{bI_{zz}}(a\bar{y})$$

$$= \frac{300 \times 10^3}{150 \times 789.884 \times 10^6}(220 \times 20 \times 225)$$

$$= 2.506 \text{ N/mm}^2$$

If bearing type bolts are used, strength in single shear = $\frac{f_u}{\sqrt{3}} \times 0.78 \frac{\pi}{4} d^2 \times \frac{1}{1.25}$

Using 16 mm bolts

Strength in single shear = $\frac{400}{\sqrt{3}} \times 0.78 \times \frac{\pi}{4} \times 16^2 \times \frac{1}{1.25} \quad = 28974 \text{ N}$

Strength in bearing is more, if minimum specified edge distances are provided. There are two bolts in a pitch distance (one on either side of web). Hence shear force per pitch length = shear strength of 2 bolts

i.e. $\quad p \times (150 \times 2.506) = 2 \times 28974$

$$p = 154.16 \text{ mm}$$

Provide 20 mm bolts at 150 mm c/c.

[*Note*: As per clause 10.2.3.2 in IS 800-2007, the distance between the centres of any two adjacent fasteners in a line lying in the direction of stress shall not exceed $16t$ or 200 mm whichever is less in tension members and $12t$ or 200 mm whichever is less, in compression members].

6. An ISMB 500 section IA used as a beam over a span of 6 m, with simply supported ends. Determine the maximum factored uniformly distributed load that the beam can carry if the ends are restrained against torsion but compression flange is laterally unsupported.

Solution:

For ISMB 500,

overall depth $h = 500$ mm

width of flange $b = 180$ mm

Thickness of flange $t_f = 17.2$ mm

Thickness of web = 10.2 mm

$r_{yy} = 35.2$ mm

Effective length for torsional buckling = 6 m.

$$\therefore \quad \frac{KL}{r} = \frac{6\times1000}{35.2} = 170.45$$

$$\frac{h}{t} = \frac{500}{17.2} = 29.06$$

From Table 14 of IS 800 (Table 7.3), we get

f_{crb} values as shown below:

$\frac{h}{t} \rightarrow$ / $\frac{kL}{r} \downarrow$	25	29.6	30
170	136.7	X	121.3
170.45	$\cdots$	O	
180	127.1	Y	112.2

To get the value for $\frac{h}{t_f} = 29.6$ and $\frac{KL}{r} = 170.45$ it needs double linear interpolation.

First get the values at X and Y corresponding to $\frac{h}{t} = 29.6$

To get the value at $X\left(\frac{KL}{r} = 170, \frac{h}{t_f} = 29.6\right)$

$$f_{crb} = 136.7 - \frac{4.6}{5}(136.7 - 121.3) = 124.16 \text{ N/mm}^2$$

To get the value at $Y\left(\frac{KL}{r} = 180, \frac{h}{t_f} = 29.6\right)$

$$f_{crb} = 127.1 - \frac{4.6}{5}(127.1 - 112.2) = 114.57 \text{ N/mm}^2$$

$\therefore$ The value of f_{cbr} at $\frac{h}{t_f} = 29.6$ and $\frac{KL}{r} = 170.45$.

To get the value at O.

$$f_{crb} = 124.16 - \frac{0.45}{10}(124.16 - 114.57) = 123.5 \text{ N/mm}^2$$

Referring to Table 13(a) in IS 800 [7.4(a)], for $f_y = 250$ N/mm^2,

we find $f_{bd} = 77.3$ N/mm^2 $f_{crb} = 100$

and $f_{bd} = 106.8$ N/mm^2 for $f_{crb} = 150$.

$\therefore$ For $f_{crb} = 123.5$,

$$f_{bd} = 77.3 + \frac{23.5}{50}(106.8 - 77.3)$$
$$= 91.165 \text{ N/mm}^2$$

Section classification:

$$\in = \sqrt{\frac{250}{250}} = 1.0$$

$$\frac{b}{t_f} = \frac{180}{17.2} = 10.46 < 10.5 \in$$

$$d = h - 2(t_f + R) = 500 - 2(17.2 + 17) = 431.6 \text{ mm}$$

$$\therefore \quad \frac{d}{t_w} = \frac{431.6}{10.2} \le 105t$$

Hence it belongs to class 2 (compact) category.

$$\therefore \quad M_d = \beta_b Z_p f_{bd}$$

$$\beta_b = 1, \quad Z_p = 2074.7 \times 10^3 \text{ mm}^3, \quad f_{bd} = 91.165 \text{ N/mm}^2$$

$$\therefore \quad M_d = 1 \times 2047.7 \times 10^3 \times 91.165 = 189.14 \times 10^6 \text{ N-mm}$$

$$= 189.14 \text{ kN-m}$$

If *udl* w is in kN/m, then $\frac{wL^2}{8} = M_d$

$$w \times \frac{6^2}{8} = 189.14$$

$$w = 42.03 \text{ kN/m.}$$

Self weight = 86.9 kg/m = 86.9 × 9.81 = 852 N/m

= 0.852 kN/m.

∴ Factored self weight = 1.5 × 0.852 = 1.278 kN/m.

∴ Super imposed *udl* that beam can carry

= 42.03 − 1.278 = 40.752 kN/m

7. Symmetric trusses of span 20m and height 5m are spaced at 4.5m centre to centre. Design the channel section purlins to be placed at suitable distances to resist the following loads:
Weight of sheeting including bolts=171 kN/m^2
Live load=0.4 kN/m^2
Wind load=1.2 kN/m^2
Spacing of purlins=1.4m

Solution:

Height of truss = 5 m   Span of truss = 20 m

∴ Slope of main rafter of symmetric truss

$$\tan\theta = \frac{5}{10} \quad \text{or} \quad \theta = 26.565$$

Design for DL + LL:

D.L from sheeting = 171 N/m^2

Self weight of purlins = 125 N/m^2.   (assumed)

∴ Total dead load = 171 + 125 = 296 N/m^2 = 0.296 kN/m^2

Live load = 0.4 kN/m^2

∴ Factored DL + LL is

= 1.5 (0.296 + 0.4)

= 1.044 kN/m^2

= 1.044 × 1.4 = 1.46 kN/m, vertically downward

$\therefore$ Load normal to sheeting

$$= 1.46 \cos\theta = 1.46 \cos 26.965$$

$$= 1.306 \text{ kN/m}$$

Load in the direction parallel to sheeting

$$= 1.46 \sin\theta = 0.653 \text{ N/m.}$$

Bending moments are:

$$M_z = 1.306 \times \frac{4.5^2}{8} = 3.306 \text{ kN-m}$$

$$M_y = 0.653 \times \frac{4.5^2}{8} = 1.653 \text{ kN-m}$$

Shear forces are:

$$F_z = 1.306 \times \frac{4.5}{2} = 2.939 \text{ kN}$$

$$F_y = 0.653 \times \frac{4.5}{2} = 1.469 \text{ kN}$$

Try ISMC 100 section.

$$d = 100 - 2(7.5 + 4.5) = 76 \text{ mm} \qquad b = 50 \text{ mm}$$

$$Z_{pz} \text{ required} = \frac{3.306 \times 10^6}{250} \times 1.1 + 2.5 \times \frac{76}{50} \times \frac{1.653 \times 10^6}{250} \times 1.1$$
$$= 42.185 \times 10^3 \text{ mm}^3$$

Z_{pz} of ISMC 100 is 43.8×10^3 mm^3. Hence adequate.

Check for shear:

$$V_{dz} = \frac{f_y}{\sqrt{3}} \times \frac{1}{\gamma_{mo}} \times ht_w = \frac{250}{\sqrt{3}} \times \frac{1}{1.1} \times 100 \times 4.7 = 61671 \text{ N} > F_z$$

$$V_{dy} = \frac{f_y}{\sqrt{3}} \times \frac{1}{\gamma_{mo}} \times \left(2b\, t_f\right) = \frac{250}{\sqrt{3}} \times \frac{1}{1.1} \times 2 \times 50 \times 7.5$$
$$= 94412 \text{ N} > F_y$$

Section is adequate to resist shear.

Design capacity of the section:

Section classification:

$$\frac{b}{t_f} = \frac{50}{7.5} = 6.67 < 9.4$$

$$\frac{d}{t_w} = \frac{76}{4.7} = 16.17 < 42$$

Hence the section is plastic.

$$\therefore \quad M_{dz} = \frac{Z_{pz} f_y}{\gamma_{mo}} = \frac{43.8 \times 10^3 \times 250}{1.1} = 9.955 \times 10^6 \text{ N-m}$$

$$= 9.955 \text{ kN-m}$$

$$M_{dy} = \frac{Z_{py} f_y}{\gamma_{mo}}$$

Z_{py} = It may be taken as contribution by flanges only

$$= b t_f \times \frac{b}{2} = \left(\frac{b^2 t_f}{2} \right)$$

$$= \frac{1}{2} \times 7.5 \times 50^2 = 9375 \text{ mm}^3$$

$$\therefore \quad M_{dy} = Z_{py} \frac{f_y}{\gamma_{mo}} = 9375 \times \frac{250}{1.1} = 2.131 \times 10^6 \text{ N-mm}$$

$$= 2.131 \text{ kN-m}$$

$$\therefore \quad \frac{M_z}{M_{dz}} + \frac{M_y}{M_{dy}} = \frac{3.306}{9.955} + \frac{1.653}{2.131} = 1.108 > 1$$

Section fails to satisfy interaction formula.

Try ISMC 125.

$$Z_{pz} = 77.2 \times 10^3$$

$$M_{dz} = 77.2 \times \frac{250}{1.1} = 17.545 \times 10^6 \text{ N-mm}$$

$$= 17.545 \text{ kN-m}$$

$$M_{dy} = \frac{1}{2} \times 8.1 \times 65^2 \times \frac{250}{1.1} = 3.889 \times 10^6 \text{ N-mm}$$

$$= 3.889 \text{ kN-m}$$

$$\therefore \quad \frac{M_z}{M_{dz}} + \frac{M_y}{M_{dy}} = \frac{3.306}{17.545} + \frac{1.653}{3.889} = 0.613 < 1.0$$

Hence adequate.

Check for wind condition:

At this stage live load is not to be considered since wind force is suction and hence critical load condition is when there is no live load.

For this condition of loading:

Factored DL = 1.5 × 0.4135 = 0.620 kN/m, vertically downward.

[*Note:* Though DL reduces net moment about z-axis, it increases moment about y-axis. Hence load factor should be taken as 1.5.]

Wind load = 102 kN/m^2 = 1.2 × 1.4 = 1.68 kN/m

∴ Factored wind load = 1.5 × 1.68 = 2.52 kN/m, suction wind load acts normal to sheeting.

∴ Load normal to sheeting = –2.52 + 0.620 cos 26.565°

= 1.965 kN/m.

Load parallel to sheeting = 0.620 sin 26.965°

= 0.277 kN/m

$$M_{zz} = 1.965 \times \frac{4.5^2}{8} = 4.974 \text{ kN-m}$$

$$\therefore \quad M_{yy} = 0.277 \times \frac{4.5^2}{8} = 0.701 \text{ kN-m} .$$

M_{dz} for laterally unsupported compression flange is to be found.

Effective length of simply supported beam with destabilizing loading (since load do not act through shear centre) = 1.2 L = 1.2 × 4500 = 5400 mm.

$$r_y = 19.2 \text{ mm}$$

$$\therefore \quad \lambda = \frac{kL}{ry} = \frac{5400}{19.2} = 281.25$$

$$\frac{h}{t_f} = \frac{125}{8.1} = 15.43$$

From Table 14 of IS 800 (Table 7.3), f_{crb} is to be found by double linear interpolation. From the table

$\frac{h}{t_f} \longrightarrow$ / $\lambda \downarrow$	14	15.43	16
280	126.9	X	111.9
281.25		O	
290	122.3	Y	107.8

$$\text{At } X,\ f_{crb} = 126.9 - \frac{1.43}{2}(126.9 - 111.9) = 116.18 \text{ N/mm}^2$$

$$\text{At } Y,\ f_{crb} = 122.3 - \frac{1.43}{2}(122.3 - 107.8) = 111.93 \text{ N/mm}^2$$

$$\therefore \quad \text{At } O,\ f_{crb} = 116.18 - \frac{1.25}{10}(116.18 - 111.93) = 115.65 \text{ N/mm}^2$$

From Table 13.a in IS 800 (Table 7.4)

$$f_{bd} = 77.3 + \frac{15.65}{50}(106.8 - 77.3) = 86.53 \text{ N/mm}^2$$

$$\therefore \quad M_{dz} = \beta_b\, 2_{pz} f_{bd} = 1 \times 77.2 \times 10^3 \times 86.53$$

$$= 66.8 \times 10^6 \text{ N-mm} = 66.8 \text{ kN-m}$$

$M_{dy} = 2.593$ kN-m (as found earlier).

$$\therefore \quad \frac{M_z}{M_{dz}} + \frac{M_y}{M_{dy}} = \frac{4.974}{66.8} + \frac{0.701}{2.593} < 1.0$$

Hence the section ISMC 125 is adequate.

Check for Deflection

$I_z = 416.4 \times 10^4 \text{ mm}^4$; $\quad w = 1.306$ kN/m = 1.306 N/mm.

$$\therefore \quad \delta = \frac{5}{384} \times \frac{wL^4}{65} = \frac{5}{384} \times \frac{1.306 \times 4500^4}{2 \times 10^5 \times 416.4 \times 10^4}$$

$$= 6.5 \text{ mm.}$$

Permissible deflection $= \dfrac{L}{150} = \dfrac{4500}{150} = 30$ mm

Hence safe.

Provide ISMC 125 as purlin.

8.An ISLB 300 carrying udl of 50 kN/m has effective span of 8m.this is to be connected to the web of girder ISMB 450.Design the framed connection using 20mm black bolts.

Solution:

1. *Connection of cleat angle with the web of secondary beam*:

Strength of M20 bolts in double shear

$$= \frac{f_{ub}}{\sqrt{3}} \times \frac{1}{\gamma_{mb}} \times (1 + 0.78)\frac{\pi}{4} \times 20^2$$

$$= \frac{400}{\sqrt{3}} \times \frac{1}{1.25} \times 1.78 \times \frac{\pi}{4} \times 20^2$$

$$= 103.314 \times 10^3 \text{ N} = 103.314 \text{ kN.}$$

Strength in bearing over web of ISLB 300:

Providing an edge distance $e = 40$ mm and pitch $p = 60$ mm, we find

K_b as the minimum of $\frac{40}{3\times22}$, $\frac{60}{3\times22} - 0.25$, $\frac{400}{410}$, 1.0.

$\therefore$ $K_b = 0.606$, Now $t = t_w = 6.7$ mm

$$\text{Strength in bearing } = 2.5\, K_b\, dt\, f_u \times \frac{1}{\gamma_{mb}}$$

$$= 2.5\times0.606\times20\times6.7\times410\times\frac{1}{1.25}$$

$$= 66594 \text{ N} = 66.594 \text{ kN}$$

$\therefore$ Bolt value = 66.594 kN

$$\text{End reaction} = 50\times\frac{8}{2} = 200 \text{ kN}$$

$\therefore$ Factored reaction $V = 1.5 \times 200 = 300$ kN

$$\therefore \quad \text{No. of bolts required} = \frac{300}{66.594} = 4.50$$

Provide 6 bolts in two rows.

2. *Connection of angle with web of girder:*

Thickness of web of girder (ISMB 450) = 9.4 mm

Strength of bolt in single shear

$$= \frac{f_{ub}}{\sqrt{3}}\times\frac{1}{1.25}\times0.78\times\frac{\pi}{4}d^2$$

$$= \frac{400}{\sqrt{3}}\times\frac{1}{1.25}\times0.78\times\frac{\pi}{4}\times20^2 = 45272 \text{ N}$$

$$= 45.272 \text{ kN}$$

Strength in bearing is more than it.

$\therefore$ Bolt value = 45.272 kN.

$$\therefore \quad \text{No. of bolts required} = \frac{300}{45.272} = 6.6$$

Provide 4 bolts in each angle at 50 mm spacing [Note even with this spacing strength of bolt in bearing is more than strength in single shear].

Design of cleat angle:

To keep the bearing strength on cleat angle greater than strength in single shear, thickness of cleat angle is given by

$$2.5\, K_b\, dt\, f_u \times \frac{1}{\gamma_{mb}} = \frac{f_{ub}}{\sqrt{3}} \times \frac{1}{\gamma_{mb}} \times 0.78 \times \frac{\pi}{4} \times d^2$$

$$2.5 \times 0.606 \times 20 \times t \times 410 \times \frac{1}{1.25} = \frac{400}{\sqrt{3}} \times \frac{1}{1.25} \times 0.78 \times \frac{\pi}{4} \times 20^2$$

$$t = 4.56 \text{ mm.}$$

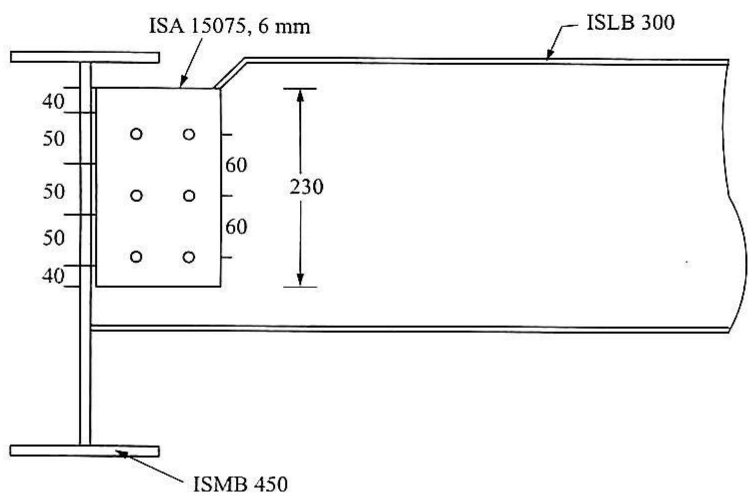
Use 6 mm thick angle.

Provide ISA 15075, 6 mm angle with 150 mm leg on secondary beam so that, two rows of bolts may be provided in it.

Depth of angle required on secondary beam = 40 + 60 + 60 + 40 = 200 mm

Depth of angle required on main beam (girder) = 40 + 50 + 50 + 50 + 40 = 230 mm.

Provide 230 mm long cleat angle as shown in Fig. 8.7.



9. An ISMB 400 beam is to be connected to an ISHB 250@537 N/m to transfer a end force of 140 kN. Design the double plated welded connection.

Solution:

Factored $V = 140 \times 1.5 = 210$ kN

Using 50 mm wide plates, factored moment on weld connecting plate and web of beam (weld B)

$M = 210 \times 50 = \text{kN-mm} = 210 \times 50 \times 10^3$ N-mm.

Thickness of plate should be 1.5 mm more than the web thickness of the beam.

$\therefore$ Thickness of plate $= t_w + 1.5 = 8.9 + 1.5 = 10.4$ mm.

Use 12 mm plates.

Since one weld is shop weld and the other is field weld, design is made for field weld and the same is adopted for shop weld also. For field weld partial safety factor $V_{mw} = 1.5$. Hence

$$\text{Strength of weld} = \frac{f_u}{\sqrt{3}} \times \frac{1}{1.5} = \frac{410}{\sqrt{3}} \times \frac{1}{1.5}$$

$$f_{wd} = 157.81 \text{ N/mm}^2$$

Design of Weld B:

$V = 210$ kN-m; $M = 210 \times 50 \times 10^3$ N-mm. Assuming 6 mm as the size of weld, throat thickness of weld $= 0.7 \times 6$. Since there are two rows of welds,

$$d = \sqrt{\frac{6M}{2 \times t \times f_{wd}}} = \sqrt{\frac{6 \times 210 \times 50 \times 10^3}{2 \times 0.7 \times 6 \times 157.81}}$$

$$= 218 \text{ mm.}$$

The above depth is required to resist bending alone. Since the weld has to resist shear also, try 15 to 20% additional depth.

Trial depth $h = 1.2 \times 218 = 261.6$ mm

Try $h = 260$ mm

Selected h is between $\frac{1}{2}$ to $\frac{2}{3}$ rd of depth of beam.

$$\therefore \quad \text{Bending stress } q_2 = \frac{M}{2} = \frac{6M}{2th^2}$$

$$\text{i.e.} \quad = \frac{6 \times 210 \times 50 \times 10^3}{2 \times 0.7 \times 6 \times 260^2} = 110.95 \text{ N/mm}^2$$

$$\text{Direct shear stress } q_1 = \frac{p}{2th} = \frac{210 \times 10^3}{2 \times 0.7 \times 6 \times 260}$$

$$= 96.15 \text{ N/mm}^2$$

$$\therefore \quad \text{Resultant stress } q = \sqrt{110.95^2 + 96.15^2}$$

$$= 146.8 \text{ N/mm}^2 < 157.81 \text{ N/mm}^2$$

Hence adequate. Provide 6 mm size fillet welds, 260 mm long.

Design of Weld A:

This weld carries only shear.

$$V = 210 \text{ kN}$$

The length of this weld is also kept 260 mm.

Let size of the weld be s.

$\therefore$ Throat thickness of weld $t = 0.7\, s$. Since there are two weld lines equating strength of welds to shear, we get

$$2\, t\, d f_{wd} = V$$

$$2 \times 0.7\, s \times 260 \times 157.81 = 210 \times 10^3$$

$\therefore \quad s = 3.65$

Provide 5 mm welds.

10. An ISMB 400 transfers an end reaction of 160 kN to the flange of an ISHB 300 @577N/m. design an unstiffened welded seat connection. take f_b=185N/mm^2

Solution:

For ISMB 400,

Width of the flange, $b_f = 140$ mm, $t_f = 16.0$

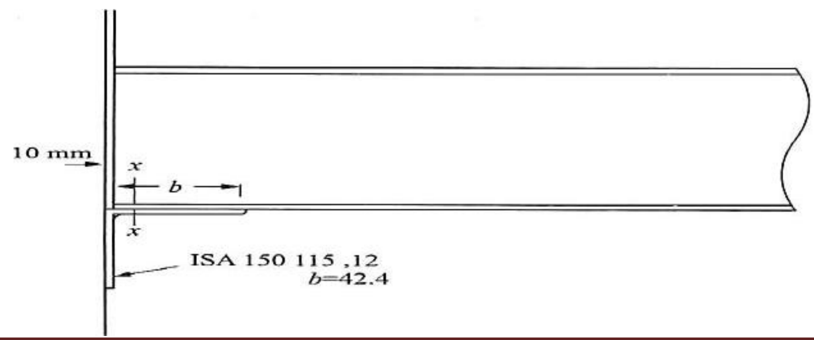
$t_w = 8.9$ mm $r_1 = 14$ mm.

$$B = \frac{F}{f_b t_w} = \frac{160 \times 10^3}{185 \times 10.2} = 84.8 \text{ mm}$$

$$b = B - \sqrt{3}\left(t_f + r_1\right) = 84.8 - \sqrt{3}\,(16 + 14)$$

$$= 32.8 \text{ mm} \qquad \not< \frac{B}{2}$$

$$b = \frac{84.8}{2} = 42.4 \text{ mm}$$



(a) *Design of Seating Angle:*

Assuming 12 mm thick angle of size 150115, the distance of end reaction from critical section

$$= 10 + \frac{1}{2} \times 42.4 - (12 + 13.5)$$

$$= 5.7 \text{ mm}$$

$\therefore$ Bending moment at the critical section in the angle $= 160 \times 10^3 \times 5.7$ N-mm

Length of seating angle = 140 mm

$\therefore$ Moment of resistance

$$M_d = \frac{f_y Z_p}{\gamma_m} = \frac{250 \times \frac{1}{4} \times 140 \times t^2}{1.25} = 7000 t^2$$

Equating it to the applied moment, we get

$$7000t^2 = 160 \times 10^3 \times 5.7$$

$$\therefore \quad t = 11.41 \text{ mm}$$

Hence 12 mm thick angle is adequate.

(b) *Design of weld:*

Length of vertical weld = 150 mm

If 't' is the throat thickness of weld,

$$q_v = \frac{160 \times 10^3}{2 \times 150 \times t} = \frac{533.3}{t}$$

The distance of end reaction from this weld

$$= \frac{42.4}{2} + 10 = 31.2 \text{ mm}$$

M at weld $= 160 \times 10^3 \times 31.2 = 4992 \times 10^3$ N

Hence horizontal shear

$$q_h = \frac{4992 \times 10^3}{2 \times \frac{1}{6} \times t \times 150^2} = \frac{665.6}{t}$$

$\therefore$ Resultant shear stress $= \sqrt{\left(\frac{533.3}{t}\right)^2 + \left(\frac{665.6}{t}\right)^2}$

$$= \frac{852.9}{t}$$

Strength of weld $= \frac{f_u}{\sqrt{3}} \times \frac{1}{1.25} = 189.37$ N/mm

Equating resultant shear stress to it, we get

$$\frac{852.9}{t} = 189.37$$
$$t = 4.504$$

$\therefore$ Size of weld $s = \frac{t}{0.7} = \frac{4.504}{0.7}$
$= 6.43$ mm

Provide 8 mm weld.

Since angle thickness is 12 mm, 8 mm weld can be provided at the edge of angle $\left(s \ngtr \frac{3}{4} 12\right)$. Hence selected angle is ISA 150 × 115, 12 mm and is connected by 8 mm welds. At the top cleat angle ISA 100100, 6 mm may be field welded with 6 mm weld.

UNIT 4 –PLATE GIRDER

Plate girders are typically used as long-span floor girders in buildings, as bridge girders, and as crane girders in industrial structures.

Commonly term girder refers to a flexural cross section made up of a number of elements. They are generally considerably deeper than the deepest rolled sections and usually have webs thinner than rolled sections. Plate girders are at their most impressive in modern bridge construction where main spans of well over 200m are feasible, with corresponding cross- section depths, over the supports, in the range of 5-10m.

Need:

1. Large Spans (above 20m)
2. Heavy Loads
3. Road or Rail Bridges
4. When rolled I-Sections are not available (i.e., above 500mm depth)

Elements of Plate Girder: (Welded)

1. Web
2. Flanges
3. Stiffeners (to avoid Web Buckling & Web Crippling Failure)

a. Transverse Stiffener (Vertical)

i. End Bearing Stiffener

ii. Intermediate Stiffener

b. Longitudinal Stiffener (Horizontal)

i. 1st stiffener at 0.20d from top

ii. 2nd stiffener at 0.50 d from top

Selection of Stiffeners:

Case	$k = d/t_w$	End Bearing Stiffener	Intermediate Stiffener	Longitudinal Stiffer	b/t_f
I	<67	NO	NO	NO	8.4
II	100-110	YES	NO	NO	13.6
III	200	YES	YES	NO	
IV	>250	YES	YES	YES	

Case I: Design is Simple (similar to beam). But, uneconomical

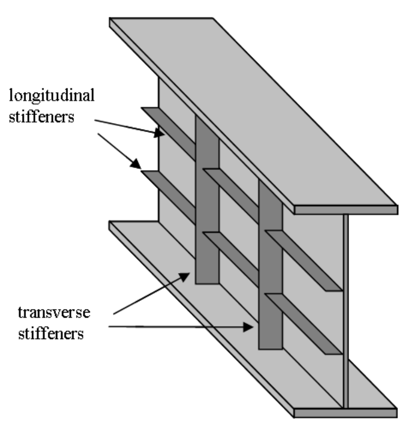
Case II: Economical. Example: For k=67,

depth= 67 x tw and b= 8.4 x tf

Stiffeners are provided to transfer transverse concentrated compressive force on the flange into the web and are essential for desired performance of web panels. These are referred to as bearing stiffeners. Intermediate web stiffeners are provided to improve its shear capacity. Design of these stiffeners is discussed below.

Load bearing stiffeners Whenever there is a risk of the buckling resistance of the web being exceeded, especially owing to concentrated loads, load-bearing stiffeners are provided. Normally a web width of 20 t on both sides as shown in Fig. is assumed to act along with the stiffener provided to resist the compression as an equivalent cruciform shaped strut of effective length 0.7 times its actual length between the top and bottom flanges. The bearing stress in the stiffener is checked using the area of that portion of the stiffener in contact with the flange through which compressive force is transmitted.

Intermediate stiffeners The intermediate stiffeners are provided to prevent out of plane buckling of web at the location of stiffeners. The buckling resistance Pq of the stiffener acting as a strut (with a cruciform section as described earlier) should be not less than (Vt -Vs) where Vt is the maximum shear force in the panel and Vs is the buckling resistance of web without considering tension field action. In its limit Vs will be equal to Vcv of the web without stiffeners. Sometimes the stiffeners are provided for more than one of the above purposes. In such cases stiffeners are considered for their satisfactory resistance under combined load effects. Such combined loads are common.



Flange splices A joint in the flange element provided to increase the length of flange plate is known as flange splice. The flange splices should be avoided as far as possible. Generally, the flange plates can be obtained for full length of the plate girder. In spite of the availability of full length of flange plates, sometimes it becomes necessary to make flange splices. Flange joints should not be located at the points of maximum bending moment.

DESIGN PROCEDURE FOR PLATE GIRDER

Design of Plate Girder:

Step 1: Assume Self Weight of Beam @ WL/2000

Where,

W = Superimposed Load on

beamL = Effective Span

Step 2: Calculate Bending Moment & Shear Force

Step 3: Find economical depth, d

$$d = [M.\ k / fy]^{1/3}$$

Where, k = d/tw = 67 or 100 or 200 or 250 (based on case 1, 2, 3, 4)

tw = k.d

Select suitable Web Plate

Step 4: Select suitable Flange

Equate M = C x Z or T x Z and find Af

Where,

C = Compressive Force = Area of Flange x Design Stress

= Af x (fy/mo)

Z = Lever Arm = C/c of two flanges

Step 5: Equate Af = b. tf and find tf and b

Where,

tf = thickness of flange plateb

= 8.4 tf for Case I

b = 13.6 tf for Case II, III, IV

Step 6: Check for moment carrying capacity of beam

Md > Mu

$Md = Z.\ (fy/\gamma mo)$

Where,

$Z = Ixx / Ymax$

$Ixx = 2\ [b.tf^3/12 + (b.\ tf)\ Y^2\ ]$ (Note: Ixx of web is negligible)

Ymax = Neutral Axis to Top of the flange

Step 7: Check for Shear Resistance (clause 8.4 of IS 800:2007)

$Vd > Vu$

Step 8: Check for web crippling (at supports)

$Fw > Vu$

$Fw = [(b1+n2)\ tw].[(fyw/\ \gamma mo)]$

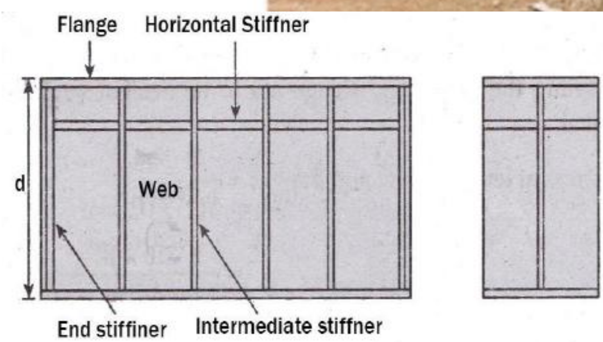


Fig : Plate Girder

1. Design a welded plate girder of span 30m to carry on superimposed load of 35kN/m.avoid use of bearing and intermediate stiffeners. Use Fe415 steel.

Solution:

1. *Moment and shear force:*

Span = 24 m.

Super-imposed load = 35 kN/m

$\therefore$ Factored load = $35 \times 1.5 = 52.5$ kN/m

Self weight $= \dfrac{52.5 \times 24}{200} = 6.3$ kN/m.

$\therefore$ Total factored load = $52.5 + 6.3 = 58.8$ kN/m.

$\therefore$ Maximum moment $M = \dfrac{wL^2}{8} = \dfrac{58.8 \times 24^2}{8}$

$= 4233.6$ kN-m.

Maximum shear force = End reaction

$$M = \frac{wL}{2} = \frac{58.8 \times 24}{2}$$

$$= 705.6 \text{ kN}$$

2. *Depth of web plate:*

If stiffeners are to be avoided,

$$k = \frac{d}{t_w} \leq 67$$

$\therefore$ Economical depth of web

$$d = \sqrt[3]{\frac{Mk}{f_y}} = \left(\frac{4233.6 \times 10^6 \times 67}{250}\right)^{1/3}$$

$$= 4043 \text{ mm.}$$

Use 1000 mm plates.

$$t_w \geq \frac{1000}{67} \geq 14.92$$

Select $t_w = 16$ mm.

Thus web plate selected is 1000 mm × 16 mm.

3. *Selection of Flange:*

Neglecting the moment capacity of web, area of flange required is

$$\frac{A_f\, f_y\, d}{1.1} \geq M$$

$$\frac{A_f \times 250 \times 1000}{1.1} \geq 4233.6 \times 10^6$$

$$\therefore \quad A_f = 18628 \text{ mm}^2$$

To keep the flange in semi compact category $\dfrac{b}{t_f} \leq 13.6$

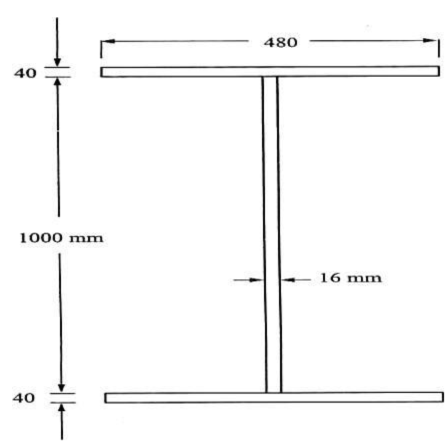
Assuming $t_f = \dfrac{b}{12}$

we get, $A_f = 12\, t_f\, t_f = 18628$

$$\therefore \quad t_f = 39.33 \text{ mm}$$

Select 40 mm plates. Width of plate required = 12 × 40 = 480 mm.

Hence use 480 mm wide and 40 mm plates. Section selected is shown in Fig.



4. *Check for the moment capacity of the girder:*

Since it is assumed that only flanges resist the moment and flange is a semi compact section, (clause 8.2.1.2).

$$M_d = \frac{Z_e f_y}{\gamma_{mo}}$$

Now
$$I_{zz} = 2\left[\frac{1}{12}\times 480\times 40^3 + 480\times 40\times\left(\frac{1000+2\times 40}{2}\right)^2\right]$$

$$= 2 \times 5196.24 \times 10^6 \text{ mm}^4$$

$$Z_e = \frac{I_{xx}}{y_{\max}} = \frac{2\times 51994.24\times 10^6}{540} = 19.238\times 10^6 \text{ mm}^3$$

$$\therefore \quad M_d = \frac{19.238\times 10^6 \times 250}{1.1} = 4372.256\times 10^6 \text{ N-mm}$$

$$= 4372.256 \text{ kN-m} > M \quad \text{Hence section is adequate.}$$

5. *Shear resistance of web [clause 8.4]*

$$V_d = \frac{V_n}{\gamma_{mo}} = \frac{A_v f_{yw}}{\gamma_{mo}\sqrt{3}} = \frac{d\, t_w f_{yw}}{\gamma_{mo}\sqrt{3}}$$

$$\therefore \quad V_d = \frac{1000\times 16\times 250}{1.1\sqrt{3}} = 2099\times 10^3 \text{ N}$$

$$= 2099 \text{ kN} > 705.6 \text{ kN}$$

Hence section is adequate.

No stiffeners are required.

6. *Check for end bearing:*

Bearing strength of web

$$F_w = (b_1 + n_2)\, t_w \frac{f_{yw}}{\gamma_{mo}}$$

Assuming that the minimum stiff bearing provided support = 100 mm.

Dispersion length $n_2 = 2.5 \times 40 = 100$ mm

$$F_w = (100+100)\times 16\times \frac{250}{1.1} = 727\times 10^3 \text{ N} = 727 \text{ kN}.$$

$$> 705.6 \text{ kN}$$

Hence adequate.

End stiffener is also not required.

7. *Design of weld connecting web plate and flange:*

Maximum shear force = 705.6 kN.

Shear stress in flange at the level of junction of web and flange

$$q = \frac{F}{bI}(a\bar{y})$$

$$= \frac{705.6\times10^3}{480\times2\times5601.28\times10^6}\left[480\times16\times\left(500+\frac{16}{2}\right)\right]$$

$$= 0.512 \text{ N/mm}^2$$

$\therefore$ Shear force per mm length in the junction

$$= 0.512 \times 480 = 245.76 \text{ N}$$

If s is the size of shop weld, threat thickness is $0.7s$. Providing weld on both sides of web strength per unit length

$$= 2\times0.7s\times\frac{410}{\sqrt{3}}\times\frac{1}{1.25} = 265.1s$$

Equating it to shear force, we get

$$265.1\,s = 245.76$$

$$\therefore \quad s = 0.92 \text{ mm.}$$

But a minimum of 5 mm is to be provided since thickness of web is 16 mm. Intermittent welds may be provided [clause (0.5.5)].

$$\therefore \quad \% \text{ of weld length} = \frac{0.92}{5}\times100 = 18.4$$

Use 40 mm long welds with a gap of 160 mm which satisfies the clauses that

(a) Minimum weld length 40 mm
(b) Maximum unwelded length $12 \times 16 = 192$ mm

and also the required percentage welding.

Final Design:

Web: 1000×16 mm.

Flange: 480×40 mm.

No stiffeners are required.

Weld: 5 mm intermittent of length 40 mm and a gap of 160 mm.

2. Design a welded plate girder of span 30m to carry on superimposed load of 35kN/m. use intermediate stiffeners. Use Fe415 steel.

Solution:

1. *Moment and shear*

Maximum moment = 4233.6 kN-m

Maximum shear force = 705.6 kN

2. *Depth of web*

If stiffener spacing 'c' is between 'd' and '$3d$' where 'd' is depth of web, then serviceability requirement is $k = \frac{d}{t_w} \le 200$

Taking $k = \frac{d}{t_w} = 190$, we get economical depth as

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

$$= \left[\frac{4233.6 \times 10^6 \times 190}{250}\right]^{1/3}$$

$$= 1476 \text{ mm}$$

Use 1500 mm wide plates.

$$t_w = \frac{1500}{190} = 7.89 \text{ mm}$$

$\therefore$ Use 1500 mm wide, 8 mm thick plates.

Provide stiffeners at every 2 m interval ($3d \ge c \ge d$).

3. *Flange*

Assuming flange alone resists the moment,

$$\frac{A_f \times f_y \times d}{1.1} \ge M$$

$$\frac{A_f \times 250 \times 1500}{1.1} \ge 4233.6 \times 10^6$$

$$\therefore \quad A_f \ge 12418 \text{ mm}^2.$$

To keep flange in semi plastic class,

$$b_f \leq 13.6\, t_f$$

Taking $b_f = 13.6\, t_f$,

we get, $13.6\, t_f \times t_f \geq 12418$

i.e. $t_f \geq 30.2$

Provide 32 mm plates.

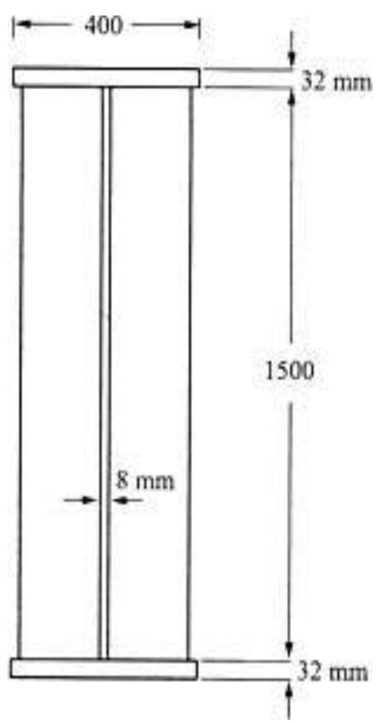
$$\therefore \quad b_f = \frac{12418}{32} = 388 \text{ mm}$$

Use 400 mm wide, 32 mm thick plates.

The trial section selected is shown in Fig.

4. *Check for shear buckling:*

Using simple post critical method (clause 8.4.2.2 a)



For $\frac{c}{d} \geq 1.0$

$$K_v = 5.35 + \frac{4}{\left(\frac{c}{d}\right)^2} = 5.35 + \frac{4}{\left(\frac{2000}{1500}\right)^2} = 7.6$$

$$\tau_{cr} = \frac{K_v \pi^2 E}{12\left(1-a^2\right)\left(\frac{d}{t_w}\right)^2} = \frac{7.6 \times \pi^2 \times 2 \times 10^5}{12\left(1-0.3^2\right)\left(\frac{1500}{8}\right)^2}$$

$$= 39.08$$

$$\lambda_w = \sqrt{\frac{f_{yw}}{\sqrt{3}\,\tau_{cr}}} = \sqrt{\frac{250}{\sqrt{3} \times 39.08}} = 1.92$$

Since $\lambda_w > 1.2,$

$$\tau_b = \frac{f_{yw}}{\left(\sqrt{3}\,\lambda_w^2\right)} = \frac{250}{\sqrt{3} \times 1.92^2} = 39.15 \text{ N/mm}^2$$

$$\therefore \quad V_n = V_{cr} = A_v \tau_b$$

$$= 1500 \times 8 \times 39.15$$

$$= 469.800 \times 10^3 \text{ N} = 469.8 \text{ kN} < 705.6 \text{ kN}.$$

Hence intermediate stiffeners are to be used to improve buckling strength of the slender web and shear capacity of end panel should be checked.

5. *Check for the end panel*

Since it is going to be stiffened web panel, it should be checked as per clause 8.5.3 of IS 800.

$$V_p = \frac{d\,t\,f_y}{\sqrt{3}} = \frac{1500 \times 8 \times 250}{\sqrt{3}} = 1732.05 \times 10^3 \text{ N}$$

$$= 1732.05 \text{ kN}$$

$$H_q = 1.25 V_p \left(1 - \frac{V_{cr}}{V_p}\right)^{0.5}$$

$$= 1.25 \times 1732.05 \left(1 - \frac{469.8}{1732.05}\right)^{0.5}$$

$$= 1848.26 \text{ kN}$$

$$\therefore \quad R_{tf} = \frac{H_q}{2} = 924.13 \text{ kN}$$

$$M_{tf} = \frac{H_q d}{10} = \frac{1848.26 \times 1500}{10} = 277239 \text{ kN-mm}$$

$$= 277.239 \text{ kN-m.}$$

The end panel is to be checked as a beam spanning between the flanges to resist R_{tf} and M_{tf}.

Area resisting shear = $t_w d = 8 \times 1500 = 12000 \text{ mm}^2$

$$V_d = \frac{A_v f_{yw}}{\sqrt{3}\,\gamma_{mo}} = \frac{12000 \times 250}{\sqrt{3} \times 1.1} = 1574.59 \times 10^3 \text{ N}$$

$$= 1574.59 \text{ kN} > 705.6 \text{ kN.}$$

End panel can safely carry the shear due to the anchoring forces.

$$I = \frac{1}{12} t_w c^3 = \frac{1}{12} \times 8 \times 2000^3 = 5333.3 \times 10^6 \text{ mm}^4$$

$$y_{\max} = \frac{c}{2} = \frac{2000}{2} = 1000 \text{ mm}$$

$$M_q = \frac{I}{y_{\max}} \times \frac{f_y}{\gamma_{mo}} = \frac{5333.3 \times 10^6}{1000} \times \frac{250}{1.1}$$

$$= 1212.12 \times 10^6 \text{ N-mm}$$

$$= 1212.12 \text{ kN-m} > M_{tf}$$

Hence the end panel can carry the bending moment due to anchor forces.

6. *Design of end stiffeners*

Reaction at end = 705.6 kN.

Compressive force due to the moment M_{tf}

$$= \frac{M_{tf}}{c} = \frac{277.239 \times 10^6}{2000} = 138.62 \times 10^3 \text{ N}$$

$$= 138.62 \text{ kN}$$

$\therefore$ Total compression = 705.6 + 138.62

$$= 844.22 \text{ kN.}$$

Strength of the stiffener (clause 8.7.5.2):

$$F_{psd} = \frac{A_q f_{yq}}{0.8\gamma_{mo}} = \frac{A_q \times 250}{0.8 \times 1.1}$$

Equating strength to the force to be resisted we get,

$$\frac{A_q \times 250}{0.8 \times 1.1} = 844.22 \times 10^3$$

$$\therefore \quad A_q = 2972 \text{ mm}^2$$

Provide 200 mm wide, 10 mm thick flats on either side of web. Then Aq provided

$$= 2 \times 200 \times 10 = 4000 \text{ mm}^2 > 2972 \text{ mm}^2.$$

Check for outstand:

It should not be more than $20\, t_q = 20 \times 10 = 200$ mm

This requirement is satisfied.

Since it is more than 14×10, the core section is based on the width $14 \times 10 = 140 \text{ mm}^2$

$\therefore$ Core area of each stiffener $= 140 \times 10 = 1400 \text{ mm}^2$.

Buckling check for stiffeners

Considering stiffeners only,

$$I_s = \frac{1}{12} \times 10 \times (400 - 8)^3 - \frac{1}{12} \times 10 \times 8^3$$
$$= 50.196 \times 10^6 \text{ mm}^4.$$

Effective area $= 2 \times 140 \times 10 = 2800 \text{ mm}^2$

$$r = \sqrt{\frac{I}{A}} = \sqrt{\frac{50.196 \times 10^6}{2800}} = 133.89$$

$kL_c = 0.7 \times d = 0.7 \times 1500 = 1050$ mm.

$$\lambda = \frac{k L_c}{r} = \frac{1050}{133.89} = 7.84$$

$\therefore$ From Table 9c of IS 800 (Table 6.4 in this book)

$$f_{cd} = 227 \text{ N/mm}^2.$$

Assuming $20 \times$ web thickness on only one side, effective area $= 2 \times 140 \times 10 + 20 \times 20 \times 8$
$= 6000 \text{ mm}^2$.

$\therefore$ Buckling resistance of stiffener $= 6000 \times 227 = 1362 \times 10^3$ N $= 1362$ kN

This is more than 844.22 kN.

Hence stiffener is adequate.

Checking stiffener for load bearing (clause 8.7.4):

Taking stiff bearing $b_1 = 0$

$$n_2 = 2.5 \times t_f = 2.5 \times 32 = 80 \text{ mm.}$$

Local capacity of web

$$F_w = \frac{(b_1 + n_2) t_w f_{yw}}{\gamma_{mo}} = \frac{(0+80) \times 8 \times 250}{1.1}$$

$$= 145454 \text{ N}$$

$$= 145.454 \text{ kN}$$

$\therefore$ The stiffener is to be designed for a force = 844.22 – 145.454

= 698.766 kN.

$$f_{cd} = 227 \text{ N/mm}^2$$

Area of stiffener alone = $2 \times 200 \times 10 = 4000$ mm^2.

$\therefore$ Bearing capacity of stiffener alone

$$= \frac{227 \times 4000}{1.1} = 825.454 \times 10^3 \text{ N}$$

$$= 825.454 \text{ kN} > 698.766 \text{ kN.}$$

Hence the stiffener is safe.

Thus end stiffeners of size 200 mm × 10 mm are adequate.

7. *Design of intermediate stiffeners*

As the shear force goes on reducing towards mid span, the first stiffener from end is critical. Since first intermediate stiffener is at $c = 2$ m from end,

shear on this stiffener = $R - 2w$

$$= 705.6 - 2 \times 58.8 = 588 \text{ kN.}$$

In this case $c = 2000$ mm

$d = 1500$ mm.

$$\therefore \quad \frac{c}{d} = \frac{2000}{1500} = 1.33 \quad < \sqrt{2}$$

Hence minimum $I_s = \dfrac{1.5 d^3 t_w^3}{c^2}$

$$= \frac{1.5 \times 1500^3 \times 8^3}{2000^2} = 648000 \text{ mm}^4$$

Try intermediate stiffeners of size 120 × 10 mm on each side. This is not violating outstand clause ($< 20t_q$)

$$I_s = \frac{1}{12} \times 10 \times (120 + 8 + 120)^3 - \frac{1}{12} \times 10 \times 8^3$$

$$= 12.71 \times 10^6 \text{ mm}^4 > I_s \text{ required.}$$

Hence adequate.

Check for buckling

Shear buckling resistance of the web alone

$$V_{cr} = 469.8 \text{ kN (as found in 4)}$$

∴ Shear strength of stiffeners alone required $= \dfrac{V - V_{cr}}{\gamma_{mo}} = \dfrac{588 - 469.8}{1.1} = 107.45$ kN

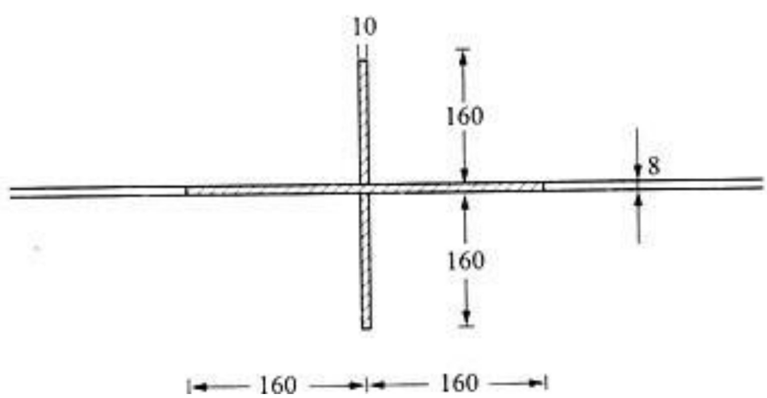
Buckling resistance of intermediate stiffener (clause 8.7.15):

Considering $20 \times t_w = 20 \times 8 = 160$ mm width of web on both side along with stiffeners [Ref. Fig.

$$I_x = I_s + 2 \times \frac{1}{12} \times 160 \times 8^3$$

$$= 12.71 \times 10^6 + 13653$$

$$= 12.73 \times 10^6 \text{ mm}^4.$$



Area = $2 \times 120 \times 10 + 2 \times 160 \times 8 = 4960 \text{ mm}^2$

$$\therefore \quad r = \sqrt{\frac{12.73 \times 10^6}{4960}} = 50.66$$

$kL = 0.7 \times 1500 = 1050$ mm.

$$\lambda = \frac{1050}{50.66} = 20.73$$

From Table 9c in IS 800 (Table 6.4c in this book)

$$f_{cd} = 224 \text{ N/mm}^2.$$

$\therefore$ Buckling resistance = $224 \times 4960 = 1111 \times 10^3$ N = 1111 kN

This is more than required resistance of 107.45 kN. Hence the stiffener is safe.

3. Design a truss of span 15m spacing 4m to be built near Visakhapatnam with the following details:
Class of building=general with life of 50 years
Terrain: category 2
Max.dimension:40m Width of building: 15m Height at eve level: 8m Topography: less than 30^0

Solution:

1. *Selection of Configuration*:

Let a pitch of $\frac{1}{5}$ be provided.

$\therefore$ Height of truss $= \frac{1}{5} \times 15 = 3$ m

$\therefore$ Slope of top chord $= \tan^{-1} \frac{3}{7.5} = 21.8°$

If purlins are to be placed on top panel point only, panel length should be around 1.4 m so that sufficient lap can be provided when 1.65 m A.C. sheets are used.

Length of top chord $\sqrt{7.5^2 + 3^2} = 8.078$ m.

If we select 6 panels, length of panel $= \frac{80.78}{6} = 1.346$ m say 1.35 m.

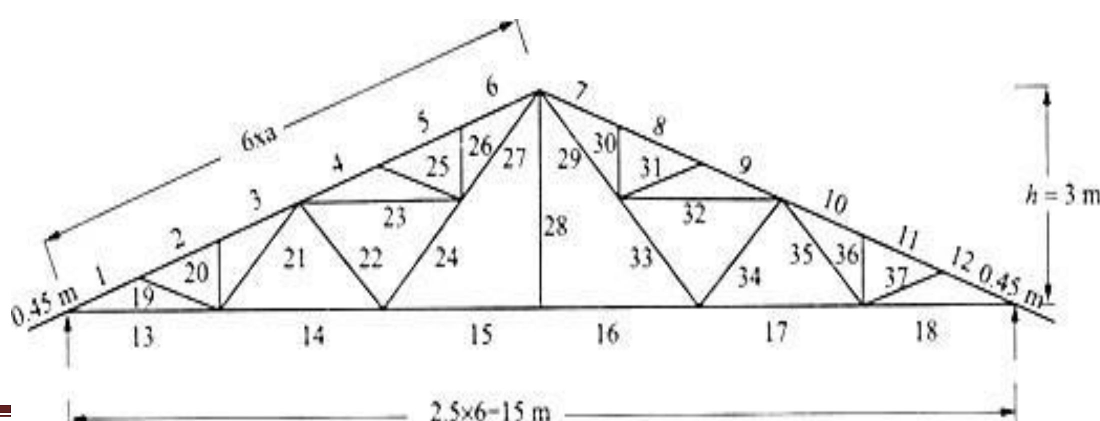
Hence Fan-Type truss shown in Fig. 12.12 is selected. [*Note:* It is not absolutely necessary to provide purlins always on panel point. When they are not on panel points, top chord members are to be designed for bending also].

2. *Loads*:

DL: As in the example 12.3.

Wt. of sheeting including laps and connections = 170 N/m²

Wt. of purlins = 120 N/m²



Self wt. of truss = 20 + 6.6 L

$$= 20 + 6.6 \times 15 = 120 \text{ N/m}^2$$

$\therefore$ Total dead load = 170 + 120 + 120 = 410 N/m^2

Each purlin takes care of an area = 1.35×4 m^2.

$\therefore$ Load on each intermediate panel point = $410 \times 1.35 \times 4$

$$= 2214 \text{ N}$$

$$= 2.214 \text{ kN.}$$

Load on shoe: Taking 450 mm roof projection load = $410 \times \left(\frac{1.35}{2} + \frac{0.45}{2}\right) \times 4 = 1476 \text{ N} = 1.476 \text{ kN}$

Live load:

LL = 750 – (21.8 – 10) × 20 = 514 N/m^2

$\therefore$ LL on intermediate panel point = $514 \times 1.35 \times 4 = 2776$ N = 2.776 kN

LL on shoe = $514 \times \left(\frac{1.35}{2} + \frac{0.45}{2}\right) \times 4 = 1850 \text{ N} = 1.850 \text{ kN}$

Wind Load:

As in example 12.3,

Wind pressure on windward side = –1.872 kN/m^2

and wind pressure on leeward side = –1.636 kN/m^2.

$\therefore$ Wind load on panel points on windward side:

(a) Intermediate panels = $-1.872 \times 1.35 \times 4 = -10.110$ kN
(b) At crown joint = – 5.050
(c) At shoe $= -1.872\left(\frac{1.35 \times 0.450}{2}\right) \times 4$

$$= -6.74 \text{ kN}$$

Wind load on leeward side:

(a) Intermediate panel = $-1.636 \times 1.35 \times 4 = -8.83$ kN
(b) At crown joint = – 4.415 kN
(c) Shoe $= -1.636\left(\frac{1.35 + 0.450}{2}\right) \times 4$

$$= -5.9 \text{ kN.}$$

3. *Analysis*:

The truss is analysed for the dead loads as shown in Fig. 12.13 and dead load forces in various members are entered in column 3 of Table 12.5. Since live loads are in direct preportion of live load in the ratio $\frac{514}{410}$, the forces in various members due to live load are found by the ratio $\frac{514}{410}$ and are listed in column 4 of Table 12.5.

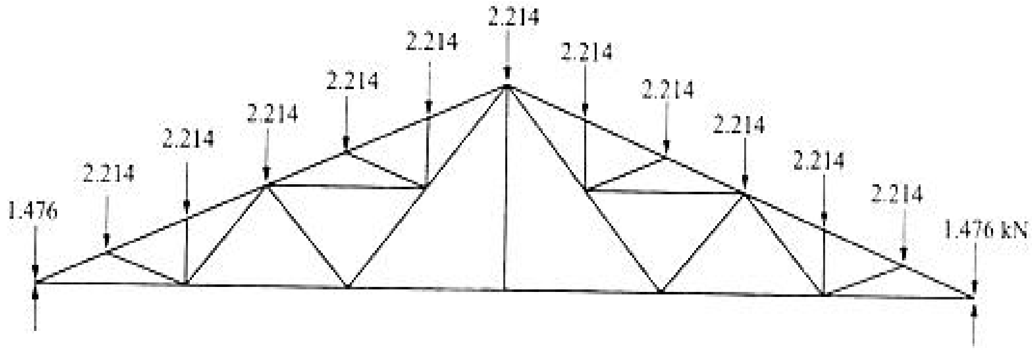


Figure 12.13

Wind load analysis is carried out for the loads as shown in Fig. 12.14 and the member forces are entered in Table 12.5.

The above analyses were carried out using software STAAD PRO 2005. They may be carried out using method of joint clubbed with method of section also.

4. *Design Forces*:

It may be observed that in a member dead loads and live load produce forces of same nature while wind load produces force of opposite nature. Hence for getting design forces the following combinations are to be considered:

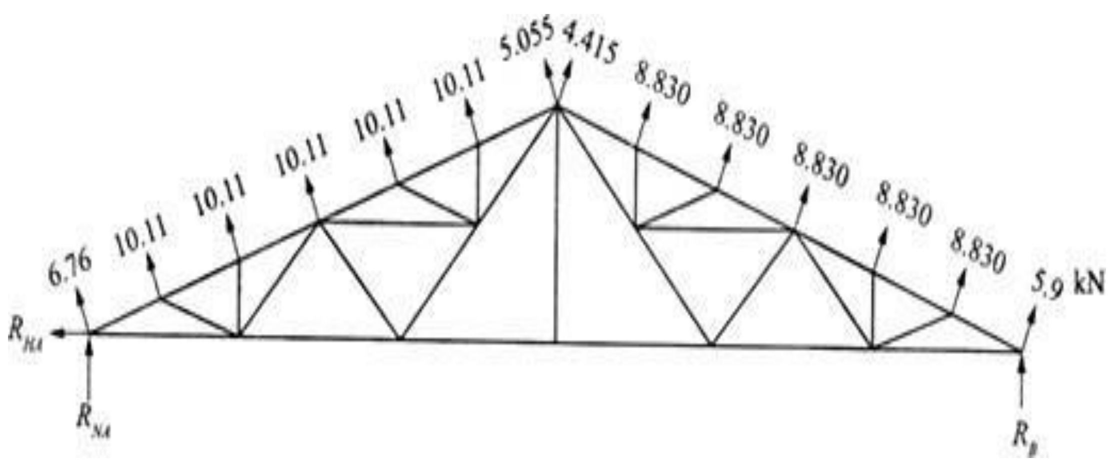


Table 12.5 Member forces [Tension +Ve, compression −Ve]

Group	Member	D.L in kN	L.L. in kN	Wt. in kN
I	1	−41.105	−41.105	133.438
	2	−29.807	−37.368	122.823
	3	−29.807	−37.368	126.867
	4	−28.317	−35.500	123.581
	5	−25.336	−31.763	112.966
	6	−25.336	−31.763	117.010
	7	−25.336	−31.763	112.621
	8	−25.336	−31.763	109.084
	9	−28.317	−44.500	118.360
	10	−29.807	−37.368	121.230
	11	−29.807	−37.368	117.698
	12	−32.784	−41.105	126.970
II	13	30.443	38.165	−124.466
	14	24.908	31.226	−97.244
	15	16.605	20.817	−56.410
	16	16.605	20.817	−56.410
	17	24.908	31.226	−92.071
	18	30.443	38.165	−115.845
III	27	10.807	13.548	−53.154
	24	6.484	8.041	−31.892
	29	10.807	13.548	−46.420
	33	6.484	8.041	−27.852
IV	22	−6.484	−8.041	31.892
	34	−6.484	−8.041	27.852
V	23	5.535	6.939	−27.223
	32	5.535	6.939	−23.774
	21	4.323	5.420	−21.261
	35	4.323	5.420	−18.568
VI	19	−2.981	−3.757	14.660
	20	−2.214	−2.776	10.889
	25	−2.981	−3.737	14.660
	26	−2.214	−3.737	14.660
	37	−2.981	−2.776	10.889
	36	−2.214	−3.737	12.809
	31	−2.281	−3.737	12.803
	30	−2.244	−2.813	9.510
	28	0	0	0

(i) DL + LL
(ii) DL + WL

From Table 4 of IS 800-2007, we find load factor is 1.5 for load case (i) whereas for load case (ii) it is 0.9* for DL and 1.5 for WL. Hence the factored force in a member is to be found for

(i) 1.5 × (Force due to DL + Force due to LL)
(ii) 1.5 × Force due to DL + 1.5 Force due to WL.

In each group combination of design forces are checked for various members and the one which gives maximum +ve and maximum –ve force is picked up.

For *group I*: (Top chord members) the design forces are

$1.5\,(-32.788 - 41.105) = -110.840$ kN

and $1.5\,(-32.788) + 1.5 \times 133.438 = 150.975$ kN

For group II: (Bottom chord members)

$1.5\,(30.443 + 38.165) = 102.912$ kN

and $1.5 \times 30.443 - 1.5 \times 124.466 = -141.035$ kN

For group III: [Main slings]

$1.5\,(10.807 + 13.548) = 36.533$ kN

and $1.5 \times 10.807 - 1.5 \times 53.154 = -63.520$ kN

For group IV:

$1.5\,(-6.484 - 8.041) = -21.788$ kN

and $1.5\,(-6.484) + 1.5 \times 31.892 = 38.112$ kN

For group V:

$1.5\,(5.535 + 6.939) = 18.711$ kN

$1.5 \times 5.535 - 1.5 \times 23.774 = -27.359$ kN

For group VI:

$1.5\,(-2.881 - 3.737) = -9.927$ kN

$1.5 \times (-2.981) + 1.5 \times 14.660 = 17.519$ kN

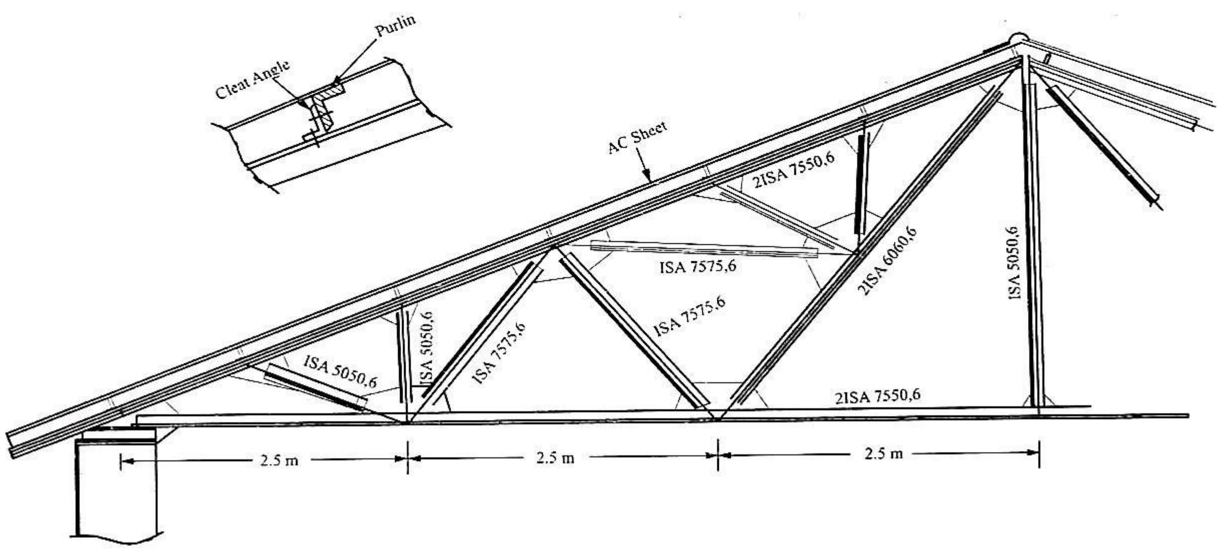
5. *Design of Members*:

A member in each group is designed for the major force and checked for minor force. The design procedure for tension is already explained in chapter 5 and the procedure for the design of compression member has been explained in chapter 6.

6. *Design of Connections*:

Using gusset plates of thickness more than the thickness of members and connections are designed using bolts or welding. Bolted connection design has been already explained in chapter 3 and welded connection design procedure has been explained in chapter 4.

7. Suitable end bearings are designed, which depend whether the truss is supported on steel column or concrete column or on masonry.
8. Drawing: Figure 12.15 shows the detailed drawing.



Design details of trusses.

UNIT 5 – DESIGN OF GANTRY GIRDER

In manufacturing plant it is essential to provide overhead travelling crane to transport heavy components of machines from one place to another. The movement of the load is of three dimensional nature. The crane is required to lift heavy mass vertically and horizontally, also the crane with load is required to move along the length of the shed. The cranes are either hand-or-electrically operated. The crane moves on rails which are at its ends. The rails are provided on a girder known as a gantry girder. The gantry girder spans over gantry columns. If capacity of crane is moderate, the gantry girders rest on brackets connected to roof column of industrial shed.

Characteristics

- Design of gantry girder is a classic example of laterally unsupported beam
- It is subjected to in addition to vertical loads and horizontal loads along and perpendicular to its axis
- Loads are of dynamic nature and produce vibration
- Compression flange requires critical attention

Codal Provisions

Partial safety factor for both dead load and crane load is 1.5 (Table 4, p.29)
Partial safety factor for serviceability for both dead load and crane load is 1 (Table 4, p.29)
Deflection Limits (Table 6, p.31)

Category	Maximum Deflection
Vertical deflection	Manually Operated – Span/500 Electric operated- Span/750 upto 50t capacity Electric operated- Span/1000 over 50t capacity
Lateral deflection	Relative displacement between rails supporting 10 mm or crane- span/400

Other Considerations

Diaphragm must be provided to connect compression flange to roof column of industrial building to ensure restraint against lateral torsional buckling at ends.
Span is considered to be simply supported to avoid bumping effect.

Design Steps

The design of the gantry girder subjected to lateral loads is a trial-and-error procedure. It is assumed that the lateral load is resisted entirely by the compression top flange of the beam and any reinforcing plates, channels, etc. and that the vertical load is resisted by the combined beam. Various steps involved in the design are as follows:

1. Maximum wheel load is to be calculated. The wheel load is maximum when the trolley is closest to the gantry girder. This load is to be correspondingly increased for the impact.

2. Maximum bending moment in the gantry girder due to vertical loads is to be computed. This consists of the bending moment due to maximum wheel loads (including impact) and the bending moment due to dead load of the gantry and rails. The bending moment due to dead loads is maximum at the centre of the girder, whereas the bending moment due to wheel load is maximum below one of the wheels. For simplicity, the maximum bending moment due to dead load is directly added to the maximum wheel load moment.

3. Maximum shear force is to be calculated. This consists of the shear force due to wheel loads and dead loads from the gantry girder and rails.

Generally an I-section with a channel section is chosen, though an I-section with a plate at the top flange may be used for light cranes.

When the gantry is not laterally supported, the equation to be used to select a trial section is as follows:

Zp = Mu/fy(1)

Zp (trial) = kZp, (k = 1.4-1.5) (2)

Generally, the economic depth of a gantry girder is about (1/12)th of the span. The width of the flange is chosen to be between (1/40) and (1/30)th of the span to prevent the excessive lateral deflection.

4. The plastic section modulus of the assumed combined section is found out by considering a neutral axis which divides the area in two equal parts, at distance y to the area centroid from the neutral axis. Thus,

Mp = 2fyA/2y = Ayfy, where Ay = plastic modulus Zp (3)

5. When lateral support is provided at the compression (top) flange, the chosen section should be checked for the moment capacity of the whole section (clause 8.2.1.2 of IS800):

Mdz = BbZpfy/γmo ≤ 1.2Zefy/γmo (4)

6. Above value should be greater than applied bending moment. The top flange should be checked for bending in both the axes using the following interaction equation:

(My/Mndy) + (Mz/Mndz) ≤ 1 (5)

7. If the top (compression) flange is not supported, the buckling resistance is to be checked in the same way as in step 4 but replacing fy with the design bending compressive stress fbd (calculated using Section 8.2.2 of the code).

8. At points of concentrated load (wheel load or reactions) the web of the girder must be checked for local buckling and, if necessary, load carrying stiffeners must be introduced to prevent local buckling of the web.

9. At points of concentrated load (wheel load or reactions) the web of the girder must be checked for local crushing. If necessary, bearing stiffeners should be introduced to prevent local crushing of the web.

The maximum deflection under working loads has to be checked.

10. The gantry girder is subjected to fatigue effects due to moving loads. Normally, light-and medium-duty cranes are not checked for fatigue effects if the number of cycles of load is less than 5 x 106. For heavy-duty cranes, the gantry girders are to be checked for fatigue loads (see IS 1024 and IS 807). Refer section 13 of the code for design provisions for fatigue effects. The fatigue strength is to be checked at working loads.

1. Design principles of gantry girder.

The step by step design procedure of a gantry girder is as follows:

- The maximum wheel load is determined.
- The maximum bending moment in the gantry girder due to vertical loads.The maximum shear force is computed.

This consists of shear force due to wheel load and dead loads from the gantry girder andrails. The shear due to the wheel load is maximum when one of the wheels is at the support.

- The lateral forces on the girder and the maximum bending moments and shear due to these are calculated. The position of the wheels should be same as that in step 1 and step3.
- The plastic section modulus of the trial section is determined by $M_p=f_y x2XA/2Xy=Z_P f_y$
  $Z_{P,REQ}=(1.4 \text{ to } 1.5)M_u/f_y$

- Section is classified.
- The girder is checked for moment capacity.
- Local moment carrying capacity of the girder is checked.The
- girder is checked for buckling resistance.
- The section is checked for shear capacity. Buckling of
- the web under wheel load is checked.The girder is
- checked for bearing.
- Rivets/bolts or welds connecting the channel to the I section are designed.Deflection of the
- gantry girder is checked.
- The girder is checked for fatigue strength.
- The bracket and the connection with column are designed.

2. Design a simply supported gantry girder to carry one electric overhead travellingcrane,
Given Data:

Span of the gantry crane=6.5m
Span of the crane girder=16m
Crane capacity=250kN
Self weight of crane excluding trolley=280 kN
Self weight of the trolley=50 kN
Minimum hook approach=1.0m
Distance between wheels=3.5m
Self weight of rails=0.3 kN/m

Solution:

1. *Moments and Shears*

Load for Maximum Moment

Weight of trolley + lifted load = 250 + 50 = 300 kN

Self weight of crane girder = 200 kN.

For ma y as possible.
Figure

$\therefore$ Dead load due to self weight + rails = 2 + 0.3 = 2.3 kN/m

$\therefore$ Factored DL = 2.3 × 1.5 = 3.45 kN/m.

$$\text{Moment due to DL} = 3.45 \times \frac{6.5^2}{8} = 18.22 \text{ kN-m.}$$

Factored moment due to vertical loads

$$M_z = 496.375 + 124.094 + 18.22 = 638.689 \text{ kN-m.}$$

Maximum moment due to horizontal force (surge):

Horizontal force transverse to rails = 10% of weight of trolley plus load lifted

$$= \frac{10}{100}(250 + 50) = 30 \text{ kN}.$$

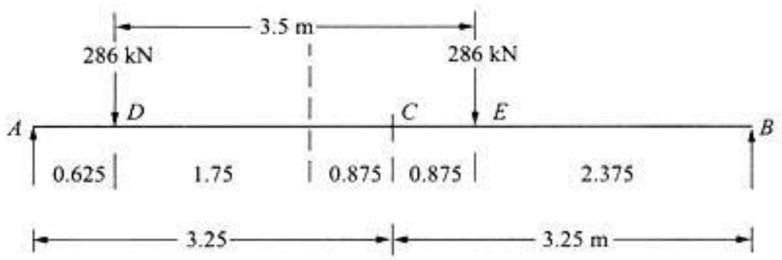
$$R_A = \frac{300 \times 15 + 200 \times 8}{16} = 381.25 \text{ kN}$$

This load is transferred to gantry girder through two wheels, the wheel base being 3.5 m.

$$\therefore \quad \text{Load on gantry girder from each wheel} = \frac{381.25}{2} = 190.63 \text{ kN}$$

Factored wheel load = 190.63 × 1.5 = 286 kN.

Maximum moment due to moving loads occur under a wheel when the c.g. of wheel load and the wheel are equidistant from the centre of girder. This is shown in Fig.



$$R_B = \frac{286 \times 0.625 + 286(3.25 + 0.875)}{6.5} = 209 \text{ kN}.$$

Max moment $M_E = 209 \times 2.375 = 496.375$ kN-m.

Moment due to impact = 0.25 × 496.375 = 124.094 kN-m.

Assume self weight of girder = 2 kN/m.

This is distributed over 4 wheels

$\therefore$ Horizontal force on each wheel $= 7.5$ kN

Factored horizontal force on each wheel $= 1.5 \times 7.5$

$= 11.25$ kN

For maximum moment in gantry girder the position of loads is same as shown in Fig. 11.7 except that it is horizontal. Hence by proportioning we get,

$$M_y = \frac{11.25}{286} \times 496.375 = 19.525 \text{ kN-m}$$

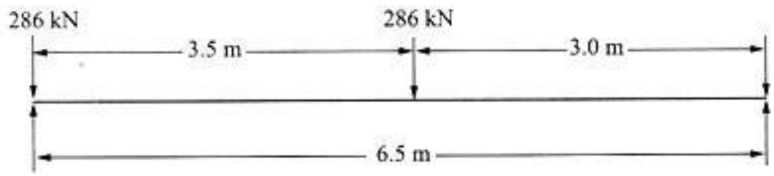
Shear Forces:

For maximum shear force on the girder, the trailing wheel should be just on the girder as shown in Fig. 11.8.

$\therefore$ Vertical shear due to wheel loads $= 286 + \dfrac{286 \times 3.0}{6.5} = 418$ kN.

Vertical shear due to impact $= 0.25 \times 418$

$= 104.5$ kN.



Vertical shear due to self weight $= 3.45 \times \dfrac{16}{2} = 27.6$ kN.

$\therefore$ Total vertical shear $= 418 + 104.5 + 27.6 = 549.9$ kN.

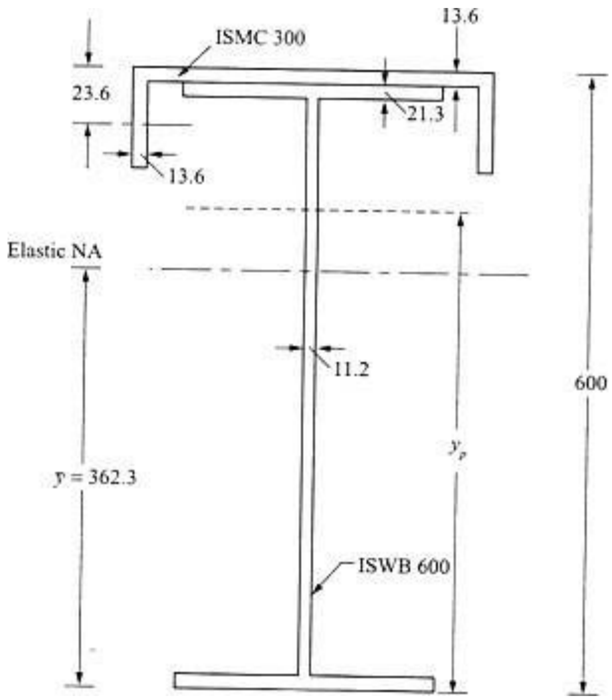
By proportioning lateral shear due to surge $= \dfrac{11.25}{286} \times 148.4 = 16.44$ kN.

Preliminary Section:

$$\frac{L}{12} = \frac{6500}{12} = 541.7 \text{ mm}$$

$$\frac{L}{25} = \frac{6500}{25} = 260 \text{ mm}$$

Let us try ISWB 600 with ISMC 300 on compression flange as shown in



Properties of ISWB 600 @ 1.312 kN/m

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

$b = 250 \text{ mm}$

$t_f = 21.3 \text{ mm}$

$t_w = 11.2 \text{ mm}$

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

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

Properties of ISMC 300

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

$b = 90 \text{ mm}$

$t_f = 13.6 \text{ mm}$

$t_w = 7.6 \text{ mm}$

$I_{zz} = 6362.6 \times 10^4 \text{ mm}^4$

$I_{yy} = 310.8 \times 10^4 \text{ mm}^4$

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

Let distance of *N-A* from the tension flange be $\bar{y}$.

Then,

$$\bar{y} = \frac{17038 \times 300 + 4564 \times (600 + 13.6 - 23.6)}{17038 + 4564} = 362.3 \text{ mm}$$

$$I_{zz} = 106198.5 \times 10^4 + 17038\,(362.3 - 300)^2 + 310.8 \times 10^4 + 4564 \times (590 - 362.3)^2$$

$$= 1207.28 \times 10^6 \text{ mm}^4$$

$$Z_e = \frac{I_{zz}}{y_{\max}} = \frac{1207.28\times10^6}{362.3} = 333.227\times10^4 \text{ mm}^4$$

For compression flange about y-y axis,

$$I = \frac{1}{12}\times21.3\times250^3 + 6362.6\times10^4 = 9136.04\times10^4 \text{ mm}^4.$$

Z_y for compression flange $= \dfrac{9136.04\times10^4}{150} = 609.069\times10^3 \text{ mm}^3$

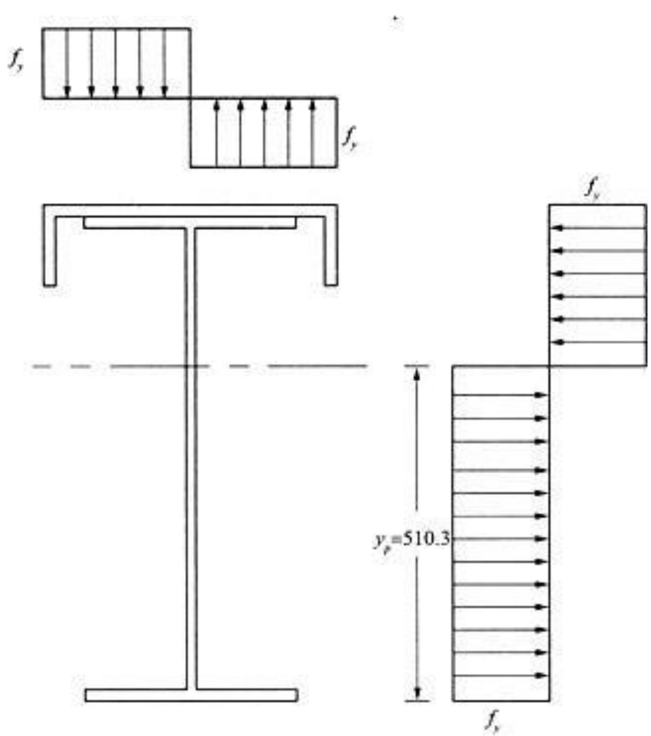
Plastic Modulus of Section: [Ref. Fig. 11.10]

Total area of the section = 17038 + 4564 = 21602

Let plastic *N-A* be at a distance Y_p from tension flange. Then

$$\left(Y_p - 21.3\right)\times11.2 + 250\times21.3 = \frac{A}{2} = \frac{21602}{2}$$

$\therefore \quad Y_p = 510.2$ mm.



$\therefore \quad M_p = \Sigma$ Moment of forces at yield about plastic N-A

$$= 21.3 \times 250\left(510.3 - \frac{21.3}{2}\right) f_y + \frac{(510.3 - 21.3)^2}{2} \times 11.2\, f_y$$
$$+ \frac{(600 - 21.3 - 510.3)^2}{2} \times 11.2\, f_y + 21.3 \times 250\left(600 - \frac{21.3}{2} - 510.3\right) f_y$$
$$+ 4564\,(600 + 13.6 - 23.6 - 510.3)\, f_y$$

$$\therefore \quad Z_p = \frac{M_p}{f_y} = 4810.07 \times 10^3 \text{ mm}^3.$$

For top flange,

$$Z_{py} = \frac{M_p}{f_y} = \frac{1}{4} \times 21.3 \times 250^2 + \frac{1}{4}(300 - 2 \times 13.6)^2 \times 7.8 + 2 \times 90 \times 13.6\left(150 - \frac{13.6}{2}\right)$$
$$= 828.485 \times 10^3 \text{ mm}^3.$$

Check for Moment Capacity:

$$\frac{b}{t} \text{ of flange of ISWB } 600 = \frac{250 - 11.2}{2 \times 21.3} = 5.6 < 9.4$$

$$\frac{d}{t} \text{ of web of ISWB } 600 = \frac{600 - 2 \times 21.3}{11.2} = 49.76 < 83.9$$

$$\text{and } \frac{b}{t} \text{ of flange of channel} = \frac{90 - 7.6}{13.6} = 6.06 < 9.4$$

Hence it is a plastic section.

Local moment capacity for bending in vertical plane:

$$M_{dz} = \frac{f_y z_p}{1.1} = \frac{250}{1.1} \times 4810.07 \times 10^3 = 1093.2 \times 10^6 \text{ N-mm}$$
$$= 1093.2 \text{ kN-m.}$$

$$\frac{1.2\, z_e\, f_y}{1.1} = \frac{1.2 \times 333.227 \times 10^3 \times 250}{1.1} = 908.8 \times 10^6 \text{ N-mm}$$
$$= 908.8 \text{ kN-m.}$$

$\therefore \quad M_{dz} = 908.8$ kN-m.

For top flange:

$$M_{dz} = \frac{f_y z_p}{1.1} = \frac{250}{1.1} \times 828.485 \times 10^3 = 188.29 \times 10^6 \text{ N-mm}$$

$$= 188.29 \text{ kN-m}$$

$$\frac{1.2 z_e f_y}{1.1} = \frac{1.2 \times 609.069 \times 10^3 \times 250}{1.1} = 166.11 \times 10^6 \text{ N-mm}$$

$$= 166.11 \text{ kN-m}$$

$\therefore$ For top flange, $M_{dz} = 166.11$ kN-m.

Check for Combined Local Capacity:

$$\frac{638.689}{908.8} + \frac{19.525}{166.11} = 0.82 < 1$$

Hence adequate.

Check for Buckling Resistance [clause 8.2.2]:

$M_d = \beta_b Z_p f_{bd}$

For plastic section $\beta_b = 1.0$

$\therefore \quad M_d = Z_p f_{bd}$

$$f_{cr} = \frac{1.1\pi^2 E}{\left(L_{LT}/r_y\right)^2}\left[1 + \frac{1}{20}\left(\frac{L_{LT}/r_y}{h_f/t_f}\right)^2\right]^{0.5}$$

$L_{LT} = 6500$ mm $\quad E = 2 \times 10^5$ N/mm^2 $\quad h_f = 600 + 7.6 = 607.6$ mm

$I_y = 4702.5 \times 10^4 + 6362.6 \times 10^4 = 11065.1 \times 10^4$ mm^4

$A = 17038 + 4564 = 21602$ mm^2.

$$\therefore \quad r_y = \sqrt{\frac{I_y}{A}} = \sqrt{\frac{11065.1 \times 10^4}{2160^2}} = 71.56 \text{ mm}$$

$$\therefore \quad f_{crb} = \frac{1.1 \times \pi^2 \times 2 \times 10^5}{(6500/71.56)^2}\left[1 + \frac{1}{20}\left(\frac{6500/71.56}{607.6/21.3}\right)^2\right]^{0.5}$$

$$= 323.06 \text{ N/mm}^2.$$

(*Note:* Table 14 of IS 800 also may be used to find f_{crb}).

From Table 13(a),

$$f_{bd} = 167.8 \text{ N/mm}^2$$

$$\therefore \quad M_{dz} = 1.0 \times 167.8 \times 4810.07 \times 10^3 = 807.13 \times 10^6 \text{ N-mm}$$

$$= 807.13 \text{ kN-m} > 638.689 \text{ kN-m}$$

Hence the section is adequate.

Check for Biaxial Bending:

$$M_{dy} = \frac{f_y Z_y}{1.1}$$

$$Z_y = \frac{I_y}{150} = \frac{11065.1 \times 10^4}{150} = 737.67 \times 10^3 \text{ mm}^3.$$

$$\therefore \quad M_{dy} = \frac{250}{1.1} \times 737.67 \times 10^3 = 167.65 \times 10^6 \text{ N-mm}$$

$$= 167.65 \text{ kN-m}$$

$$\therefore \quad \frac{M_z}{M_{dz}} + \frac{M_y}{M_{dy}} = \frac{638.689}{807.13} + \frac{19.525}{167.65}$$

$$= 0.908 < 1$$

Hence adequate.

Check for Shear:

$$V_z = 549.9 \text{ kN}.$$

$$\text{Shear capacity} = \frac{A_v f_{yw}}{\sqrt{3} \times 1.1} = \frac{600 \times 11.6 \times 250}{\sqrt{3} \times 1.1}$$

$$= 913 \times 10^3 \text{ N} = 913 \text{ kN} > 549.9 \text{ kN} \qquad \textbf{O.K.}$$

$0.6 \times 913 = 547.8$, slightly less than V. Considering it as high shear case may be ignored.

Hence there is no reduction in moment capacity. Hence moment capacity is adequate as found earlier.

Weld Design:

Shear stress $= q = \dfrac{V}{bI}(a\bar{y})$

$\therefore$ Shear per unit length $= \dfrac{V}{I}(a\bar{y})$

$V = 549.9$ kN

$a =$ Area of channel $= 4564$ mm^2

$I = I_z = 1207.28 \times 10^6$ mm^4

$\bar{y}$ = Distance of channel from N-A = 600 + 7.6 – 362.3 = 254.3 mm

$$\therefore \quad \text{Shear force per unit length } q = \frac{549.9 \times 10^3}{1207.28 \times 10^6}(4564 \times 245.3)$$

$$= 559.3 \text{ N/mm}$$

If 's' is the size of weld provided on each side, then shear strength of weld

$$= 25 \times 0.7 \times \frac{410}{\sqrt{3}} \times \frac{1}{1.25} = 265.12\, s \text{ N/mm}$$

Equating it to shear force, we get

$$265.12\, s = 559.2$$

$$s = 2.10 \text{ N/mm}$$

Hence provide 3 mm fillet weld (which is minimum) on both sides.

Check for Web Buckling:

Assuming $b_1 = 150$ mm,

$n_1 = 300 + 7.6 = 307.6$ mm

$d = 600 - 2(21.3 + 17) = 523.4$ mm, $t = 11.2$ mm

$$\lambda = 2.42\frac{d}{t} = 2.42 \times \frac{523.4}{11.2} = 113.$$

$\therefore$ From Table 9(a) in IS 800,

$$f_{cd} = 110.8 \text{ N/mm}^2$$

$\therefore$ Buckling resistance $= (b_1 + n_1)\, t f_{cd}$

$= (150 + 307.6)\ 11.2 \times 110.8$

$= 443.6 \times 10^3$ N

$= 443.6$ kN > 286 kN.

Hence adequate.

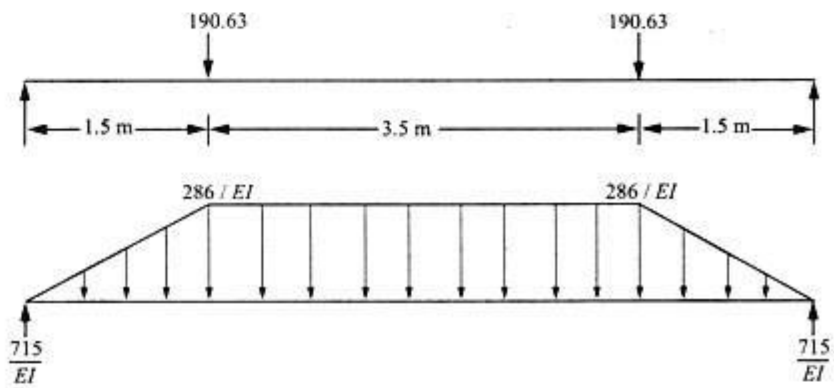
Check for Deflection:

At working load, deflection is to be limited to $\frac{L}{750}$. At this condition wheel load is as shown in Figures 11.1(a) and (b) shows the conjugate beam with $\frac{M}{EI}$ diagram.

Reaction in conjugate beam

$$= \frac{1}{2} \text{ total } \frac{M}{EI} \text{ diagram}$$
$$= \frac{1}{2} \times 1.5 \times \frac{286}{EI} + 286 \times 1.75 = \frac{715}{EI}$$

Maximum deflection occurs at mid span = Moment of $\frac{M}{EI}$ load in conjugate beam



$$EI\Delta = 715 \times \frac{6.5}{2} - \frac{1}{2} \times 286 \times 1.5 \times 2.75 - 1.75 \times 286 \times \frac{1.75}{2}$$
$$= 1295.9$$

Taking EI in kN-m^2 unit,

$$EI = 2 \times 10^5 \times 1207.8 \times 10^6 \times \frac{1}{10^9} = 200 \times 1207.8 \text{ kN-m}^2.$$

$$\therefore \quad \Delta = \frac{1295.9}{200 \times 1207.8} = 0.0054 \text{ m} = 5.4 \text{ mm}$$

$\therefore$ Deflection due to self weight $= \frac{5}{384} \times \frac{wL^3}{EI} = \frac{5}{384} \times \frac{200 \times 6.5^3}{200 \times 1207.8} = 0.003 \text{ m} = 3 \text{ mm}$

$\therefore$ Total deflection = 8.4 mm.

Permissible $\Delta = \frac{L}{750} = \frac{6500}{750} = 8.66$ mm.

$\therefore$ Deflection requirement is satisfied.

Hence use ISWB 600 with ISMC 300 on compression flange as shown in

3. A roof truss shed is to be built in lucknow for an industry. The size of shed is 24mx40m.the height of building is 12m at the eves. Determine the basic wind pressure.

Solution:

From wind zone map of country (IS 875 part 3) the basic wind speed in Lucknow is

$$V_b = 47 \text{ m/sec.}$$

Risk Coefficient k_1: From Table 12.1, for all general buildings with probable design life of structure 50 years,

$$k_1 = 1.0$$

Terrain, Height and Structure Size Factor k_2:

Since the shed is in an industrial area, it may be considered belonging to *category* 3. Its greatest dimension being 40 m, it belongs to *class B structure*. For category 3, class B building

$$k_2 = 0.88 \quad \text{if} \quad h = 10 \text{ m.}$$

$$= 0.94 \quad \text{if} \quad h = 15 \text{ m.}$$

$\therefore$ For $h = 12$ m,

$$k_2 = 0.88 + (0.94 - 0.88)\frac{2}{5} = 0.904.$$

Topography Factor k_3: In Lucknow, the ground near shed may be assumed plain.

$$\therefore \quad k_3 = 1 + Cs$$

where $c = \frac{z}{L} = 0$

$$\therefore \quad K_3 = 1.0$$

Design wind speed

$$V_2 = K_1 K_2 K_3 V_b$$

$$= 1.0 \times 0.904 \times 1.0 \times 47$$

$$= 42.488 \text{ m/sec.}$$

Hence basic wind pressure

$$p_z = 0.6 V_z^2 = 0.6 \times 42.488^2$$

$$= 1083 \text{ N/m}^2$$

$p_z = 1.083 \text{ kN/m}^2$ **Answer**

4. Design a hand operated travelling crane simply supported by gantry girderfor the given data:

Span of gantry girder = 5m Span

of crane girder = 15m

Crane capacity = 200 KN

Self weight of crane girder excluding trolley = 200 KNSelf

weight of trolley = 30 KN

Minimum hook approach = 1m

Distance between wheels = 3.5m

c/c Self weight of rails = 0.3

KN/m

Solution:

Step : 1 For calculating maximum moment

(a) Weight of trolley + load lifted by crane

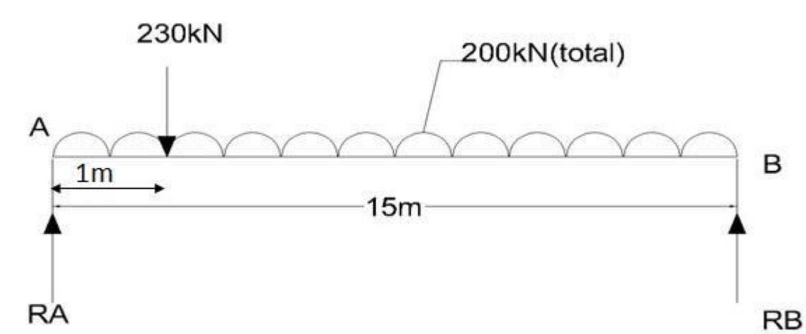
= 30+200

=230 KN

(b) Self weight of crane girder

= 200 KN

For maximum reaction on gantry girder, the moving load should be placedclose to Gantry girder .



$\sum M_B = 0$ ie) taking moment of all forces about BR_A

X 15 – 230X7.5 =0

R_A = 4720/15

= 314.67 KN

This load is transferred to gantry girder through two wheels base which are at 3.5m c/c.

∴ Load on gantry from each wheel

= 314.67/2

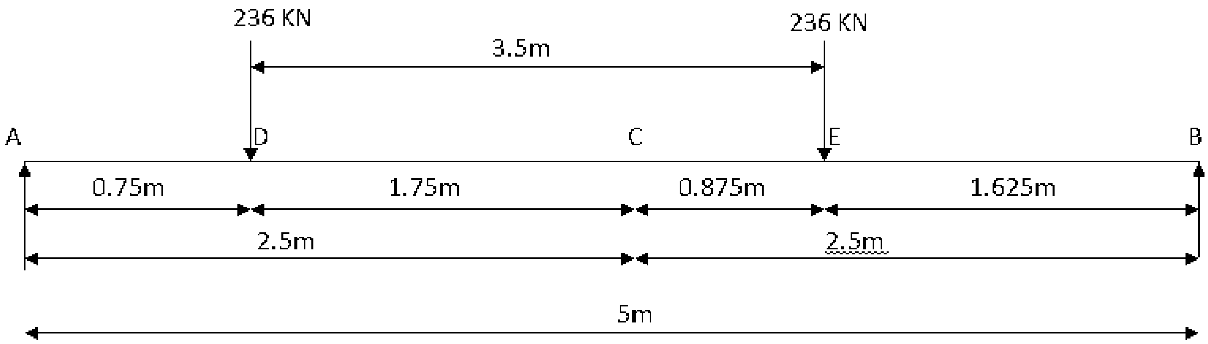
= 157.33 KN

Factored wheel load = 157.33x 1.5

= 236.00KN

The maximum moment due to moving loads occur under a wheel when the C.G (Centerof gravity)

Wheel load and the wheel are equidistance from the center of girder.



R_B = [236X0.75 + 236 x (2.5+0.875)]/5

= 194.7 kN

Maximum moment at E = 194.7 x 1.625 (from support B)

= 316.38 KNm

Moment due to impact = 20% of max moment

= 0.2 x 316.38

= 63.277 KNm

Assume self weight of girder = 2 KN/m

∴ Dead load due to self weight + rails(given)

= 2 + 0.3

= 2.3 KN/m

Factored dead load = 2.3 x 1.5

= 3.45 KN/m

Moment due to dead load = WL^2/8

= 3.45 x 5^2/8

= 10.781 kNm

∴ Factored moment due to vertical load (M_Z)

= 10.781+63.277+316.38

= 390.438 kNm

Maximum moment due to horizontal force,

Horizontal force transverse to rails = 10% of weight of trolley + load lifted

= 10/100 (200+30)

= 23 KN

Assuming double framed wheels, this is distributed over 4 wheels.

∴ Horizontal force on each wheel = 23/4

= 5.75kN

Factored horizontal force on each wheel

= 1.5 x 5.75

= 8.625 Kn

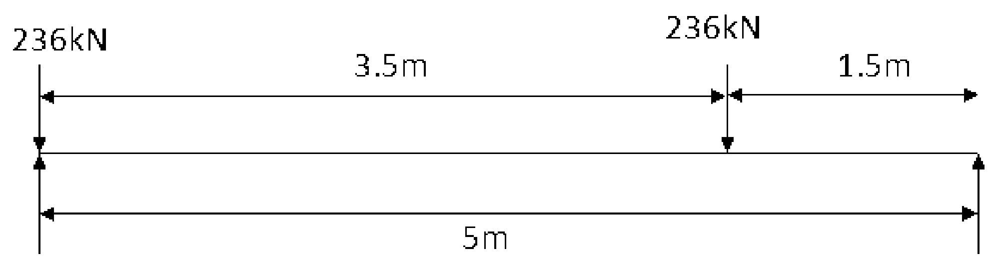
For max bending moment in gantry the position of loads is same except in horizontal,

M_Y = 8.625/236 x 316.38

= 11.56 kNm

Step 2 Shear force

For max shear force on the girder the trailing wheel should be just on the girder.



∴ Vertical shear due to wheel loads = 236 + 236x1.5/5

= 306.8 KN

Vertical shear due to impact = 0.2x306.8

= 61.36 kN

Vertical shear due to self weight = WL/2

= 3.45X5/2

= 8.625kN

Total vertical shear = 306.8+61.36+8.625

= 376.785 KN

By proportioning lateral shear due to surge

= (8.625/236)x306.8

= 11.21kN

Step 3 Selection of section

(i) Economic depth of section = L/12

= 5000/12

= 416mm ≈ 500mm

(ii) Compression flange width = L/25

= 5000/25

= 200mm

Let us try ISWB 500 with ISMC 300 on compression flange,

Properties of ISWB 500 @ 9.52N/m

A = 12,122mm^2

h = 500mm

b_f = 250mm

t_f = 14.7mm

t_w = 9.9mm

I_{zz} = 522.90x10^6mm^4

I_{yy} = 29.878x10^6mm^4

Properties of ISMC 300

A = 4564mm^2

h = 300mm

b_f = 90mm

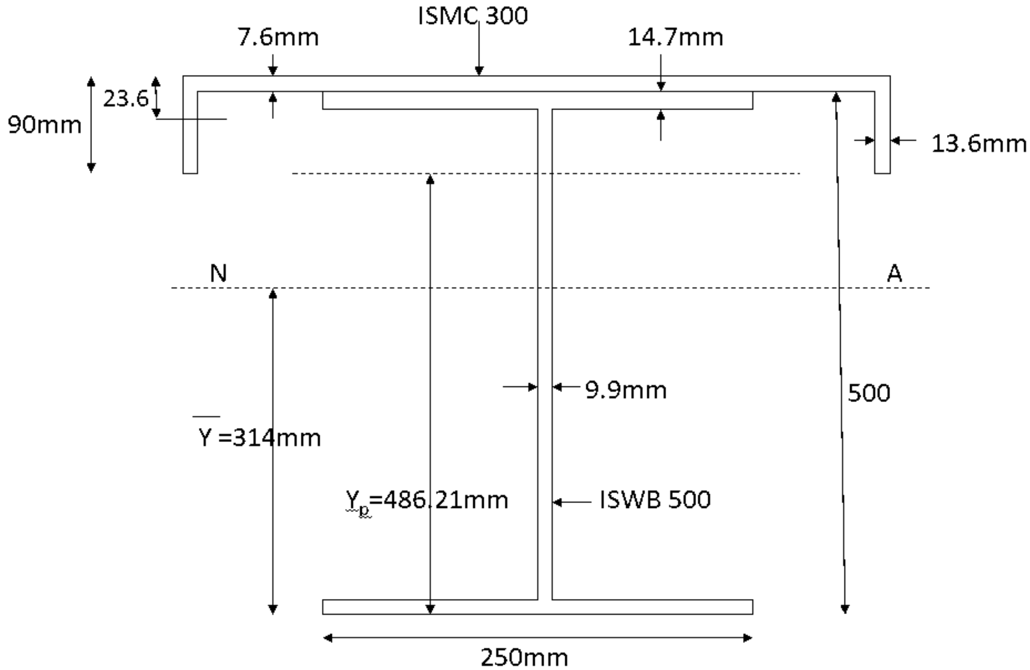
t_f = 13.6mm

t_w = 7.6mm

I_{zz} = 6362.6x10^4mm^4

I_{yy} = 310.8x10^4mm^4

C_{yy} = 23.6mm



To find N.A of the section from bottom flange (tension)

$\bar{Y}$ = [12122x250 + 4564x(500+7.6-23.6)]/(12122+4564)

$\bar{Y}$ = 314.00mm (from bottom flange)

I_{zz} = 522.90x10^6 + 12122(314-250)^2 + 310.8x10^4 + 4564(484-314)^2

= 1.015x10^9 mm^4

Z_e = I_{zz}/y

= 1.015x10^9/314

=

3.233x10^6mm^3For compression flange about yy

axis,

I_{yy} = 14.7x250^3 /12 +6362.6x10^4

= 82.766x10^6mm^4

Z_{ey} for compression flange = 82.766x10^6/150

= 551.77x10^3 mm^3

Now, plastic modulus of section

Total area of the section = 12122 + 4564

= 16686mm^2

Let plastic N.A be a distance Y_p from bottom flange,

∴ $(Y_p - 14.7)$x9.9+250x14.7 = A/2

9.9Y_p – 145.53+3675 = 686/2

Y_p = 4813.53/9.9

=486.21mm

Plastic moment capacity of section M_p = ∑Moment of forces at yield about plastic N.AM_p

= 147x250[486.21 – 14.7/2]x fy + [(486.21-14.7)/2]x9.9 fy+[(500-14.7-486.21)^2/2] x 9.9 fy + 14.7x250[500-(14.7/2)-486.21]fy + 4564(500+13.6-23.6-486.21) fy

= 1.75x10^6 fy + 2.34x10^3 fy + 23.66x10^3 fy +17.29x10^3 fy

= 1.793x10^6 fy

∴ Zp = Mp/fy

= 1.793x10^6 mm^3

For top flange,

Zpy = Mp/fy = ¼ x 14.7x250^2 + ¼(300-2x13.6)^2 x 7.6 +2x90x13.6[150 – 13.6/2]

= 229.68x10^3 + 141.39x10^3 + 350.55x10^3

= 721.62x10^3mm^3

Check for moment capacity,

b/t of flange of ISWB 500 = (250-9.9)/(2x14.7)

= 8.16 < 8.4ϵ

d/t of web of ISWB 500 = (500-2 x 14.7)/9.9

= 47.53<84ϵ

b/t of flange of channel section ISMC 300

= (90-7.6)/13.6

= 6.06<8.4ϵ

∴ The section is plastic.

Local moment capacity for bending in vertical plane,

Md_z =(fy . Zp)/1.1

= 250x1.793x10^6 / 1.1

= 407.5x10^6 Nmm

= 407.5 kNm

1.2 x Ze fy /1.1 = 1.2x5.233x10^6x250 / 1.1

= 881.72x10^6 Nmm

= 881.721 kNm

Lesser of two values Md_z =

407.5kNmFor top flange, Md_z= fy.

Zp/1.1

= (250x721.62x10^3)/1.1

= Md_y
=150.48kNm

∴ For top flange, Md_z = 150.48kNm

=

Md_yCheck for combined local capacity,

$M_z/M_{dz} + My/M_{dy} \leq 1$

390.438/407.5 + 11.56/150.48 = 0.95 +

0.076

= 1.02 ≈1

∴ Section is adequate and economic

Check for buckling resistance,

M_d = $ß_b . Z_p .f_{bd}$

ßb = 1 (for plastic section)

$f_{cr,b}$ = 1.1π^2E/(L_{LT}/r_y)^2 [1+1/20[(L_{LT}/r_y)/(h_f/t_f)]^2]^0.5L_{LT}

= 5.0m

= 5000mm

E = 2X10^5

N/mm^2h_f = 500+7.6

= 507.6mm

t_f =14.7mm

I_{yy} = 29.878x10^6 + 6362.6x10^4

= 93.50x10^6mm

$f_{cr,b}$ =(1.1xπ^2x10^5)/(5000/74.85)^2{1+1/20

[(5000/74.85)/(507.6/14.7)]^2}^0.5

= 486.59 [1+1/20(66.80/34.53)^2]^0.5

= 530.165 N/mm^2

IS code:800-2007, clause 8.2.2

For $f_{cr,b}$ = 530.16

∴ f_{bd} = 191.34 N/mm^2 (by linear interpolation)

∴M_{dz} = 1.0x191.34x1.793x10^6

= 343.07x10^6 Nmm < 407.5x10^6

Check for biaxial bending,

M_{dy} =

fy.Zy/1.1Z_y =

Iyy/150

= 93.50x10^6/150

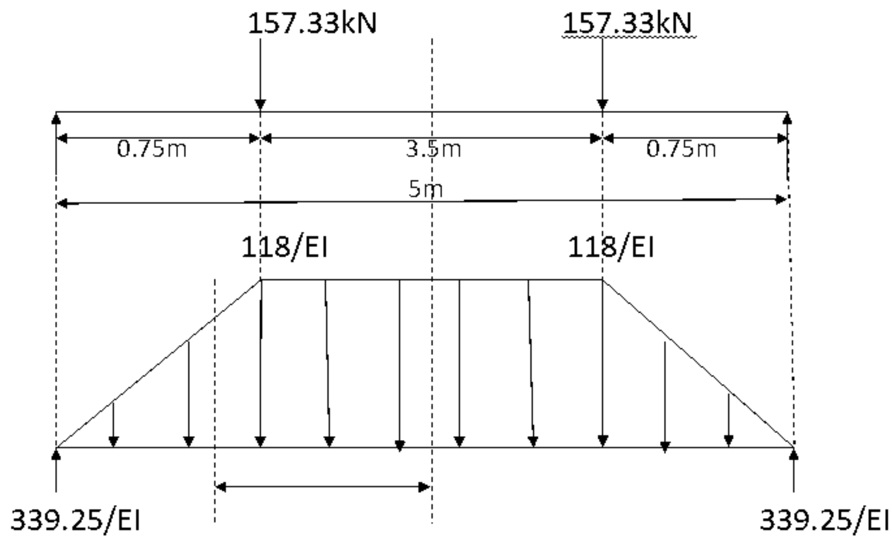
= 623.33x10^3 mm^3

M_{dy} = 250x623.33x10^3/1.1

= 141.67x10^6Nmm

= 141.67 kNm

At working load, deflection is limited to L/750



Maximum deflection occurs at midspan = moment of M/EI load in conjugate beam

Reaction in conjugate beam = ½ x total M/EI diagram

= ½ x0.75x118/EI + 118/EI x5/2

= 339.25/EI

EIΔ = 339.25X5/2 – 1/2X118X0.75X2 – 1/2x3.5x118x1.75/2

= 848.125 – 88.8 – 180.68

= 578.94

EI = (2x10^5x1.015x10^9)/(1000x1000x1000)

= 203x10^3 kNm^2

Δ = 578.94/203x10^3

= 2.85x10^-3 m

= 2.85mm

Permissible deflection Δ $= L/750$

$= 5000/750$

$= 6.67$ mm

$\therefore$ Deflection requirement is satisfied

Hence, ISWB 500 with ISMC 300 can be suitable for gantry girder.