Module – 1

Design of rectangular slab type combined footing.

Contents

1.1.1 Introduction

Whenever two or more columns in a straight line are carried on a single spread footing, it is called a combined footing. Isolated footings for each column are generally the economical. Combined footings are provided only when it is absolutely necessary, as

i) When two columns are close together, causing overlap of adjacent isolated footings
ii) Where soil bearing capacity is low, causing overlap of adjacent isolated footings
iii) Proximity of building line or existing building or sewer, adjacent to a building column.

The combined footing may be rectangular, trapezoidal or Tee-shaped in plan. The geometric proportions and shape are so fixed that the centeroid of the footing area coincides with the resultant of the column loads. This results in uniform pressure below the entire area of footing.

Trapezoidal footing is provided when one column load is much more than the other. As a result, the both projections of footing beyond the faces of the columns will be restricted. Rectangular footing is provided when one of the projections of the footing is restricted or the width of the footing is restricted.

notes4free All in one

Rectangular combined footing

Longitudinally, the footing acts as an upward loaded beam spanning between columns and cantilevering beyond. Using statics, the shear force and bending moment diagrams in the longitudinal direction are drawn. Moment is checked at the faces of the column. Shear force is critical at distance 'd' from the faces of columns or at the point of contra flexure. Two-way shear is checked under the heavier column.

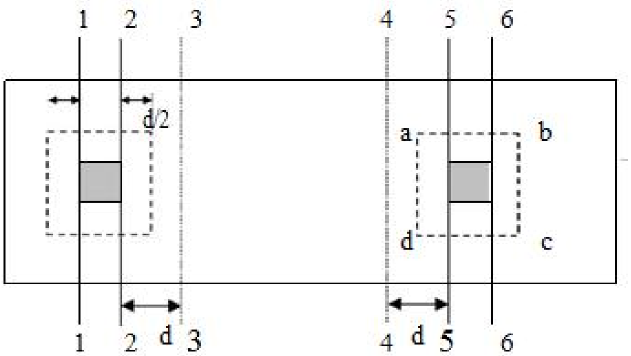
The footing is also subjected to transverse bending and this bending is spread over a transverse strip near the column.

Combined footing may be of slab type or slab and beam type or slab and strap beam type

Design:

1. Locate the point of application of the column loads on the footing.
2. Proportion the footing such that the resultant of loads passes through the centre of footing
3. Compute the area of footing such that the allowable soil pressure is not exceeded.

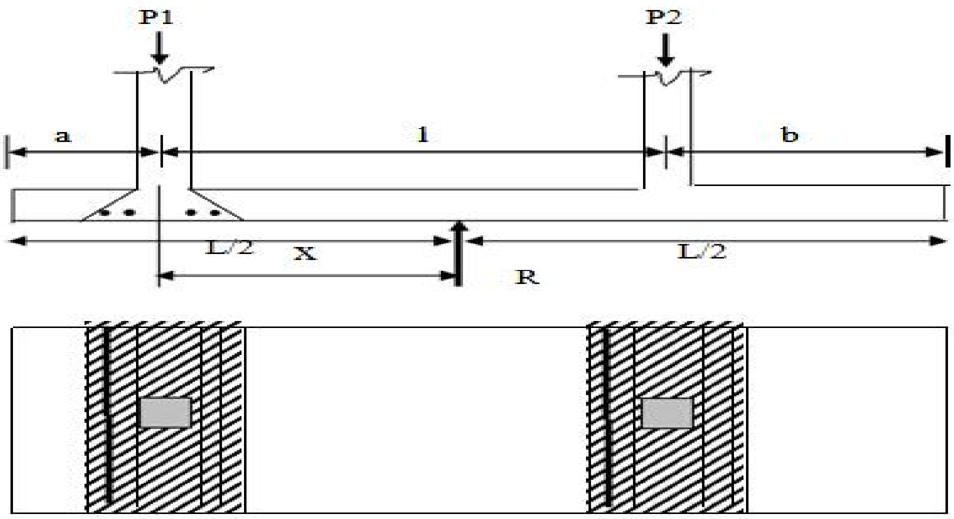
4. Calculate the shear forces and bending moments at the salient points and hence draw SFD and BMD.
5. Fix the depth of footing from the maximum bending moment.
6. Calculate the transverse bending moment and design the transverse section for depth and reinforcement. Check for anchorage and shear.
7. Check the footing for longitudinal shear and hence design the longitudinal steel
8. Design the reinforcement for the longitudinal moment and place them in the appropriate positions.
9. Check the development length for longitudinal steel
10. Curtail the longitudinal bars for economy
11. Draw and detail the reinforcement
12. Prepare the bar bending schedule



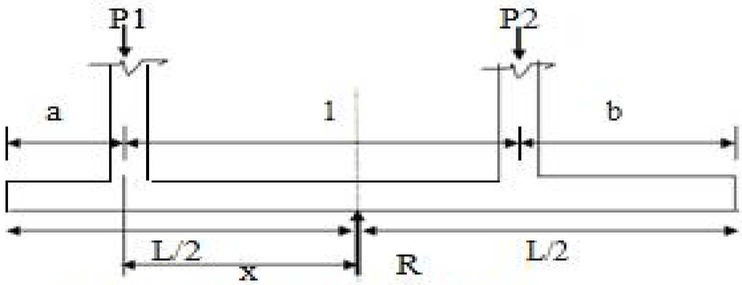
Section 1-1, 2-2, 5-5, and 6-6 are sections for critical moments

Section 3-3, 4-4 are sections for critical shear (one

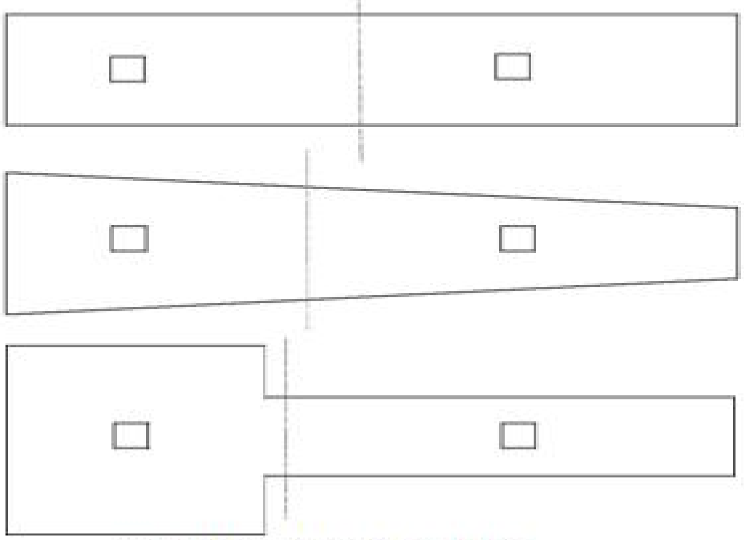
CRITICAL SECTIONS FOR MOMENTS AND



TRANSVERSE BEAM BELOW



Combined footing with loads, (Slab type)



Types of combined footings (Plan)

1.1.2 Objective

1. To provide basic knowledge in the areas of limit state method and concept of design of RC and Steel structures
2. To identify, formulate and solve engineering problems in RC and Steel Structures

1.1.3 Design example

Design of combined footing – Slab and Beam type

Two interior columns A and B carry 700 kN and 1000 kN loads respectively. Column A is 350 mm x 350 mm and column B is 400 mm X 400 mm in section. The centre to centre spacing between columns is 4.6 m. The soil on which the footing rests is capable of providing resistance of 130 kN/m^2. Design a combined footing by providing a central beam joining the two columns. Use concrete grade M25 and mild steel reinforcement.

Solution: Data

f_{ck} = 25 Nlmm^2,

f_y= 250 N/mm^2,

f_b (SBC)= l30 kN/m^2,

Column A = 350 mm x 350 mm,

Column B = 400 mm x 400 mm,

c/c spacing of columns = 4.6 m ,

P_A = 700 kN and P_B = 1000kN

Required: To design combined footing with central beam joining the two columns.

Ultimate loads

P_{uA}= 1.5 x 700 = 1050 kN, P_{uB} = 1.5 x 1000 = 1500 kN

Proportioning of base size

Working load carried by column A = P_A = 700 kN

Working load carried by column B = P_B = 1000 kN

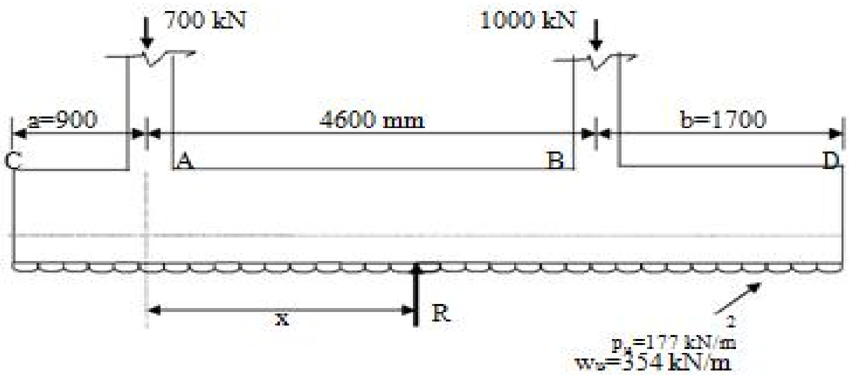
Self weight of footing 10 % x (P_A + P_B) = 170 kN

Total working load = 1870 kN

Required area of footing = A_f = Total load /SBC=1870/130 = 14.38 m^2 Let the width of the footing = B_f = 2m

Required length of footing = L_f = A_f /B_f = 14.38/2 = 7.19m Provide footing of size 7.2m X 2m, A_f = 7.2 x 2 = 14.4 m^2

For uniform pressure distribution the C.G. of the footing should coincide with the C.G. of column loads. Let x be the distance of C.G. from the centre line of column A



Combined footing with loads

Then x = (P_B x 4.6)/(P_A + P_B) = (1000 x 4.6)/(1000 +700) = 2.7 m from column A.

If the cantilever projection of footing beyond column A is ‘a’

notes4free All in one

then, a + 2.7 = L_f /2 = 7.2/2, Therefore a = 0.9 m

Similarly if the cantilever projection of footing beyond B is 'b'

then, b + (4.6-2.7) = L_f /2 = 3.6 m, Therefore b = 3.6 - 1.9 = 1.7 m

The details are shown in Figure

Total ultimate load from columns = P_u = 1.5(700 + 1000) = 2550 kN.

Upward intensity of soil pressure w_u= P/A_f= 2550/14.4 = 177 kN/m^2 1.5 SBC or UBC

Design of slab:

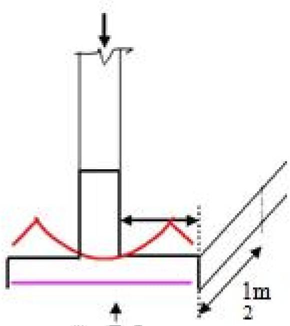
Intensity of upward pressure = p_u =177 kN/m^2

Consider one meter width of the slab (b=1m)

Load per m run of slab at ultimate = 177 x 1 = 177 kN/m

Rectangular Footing with Central Beam:-Design of Bottom slab.

Cantilever projection of the slab (For smaller column) =1000 - 350/2 = 825 mm Maximum ultimate moment = 177 x 0.825^2/2 = 60.2 kN-m



For M25 and Fe 250, $Q_{u\ max}$ = 3.71 N/mm^2

Required effective depth = √ (60.2 x 10^6/(3.71 x 1000)) = 128 mm

Since the slab is in contact with the soil, clear cover of 50 mm is assumed.

Using 20 mm diameter bars

Required total depth = 128 + 20/2 + 50 =188 mm say 200 mm

Provided effective depth = d = 200-50-20/2 = 140 mm

To find steel

M_u/bd^2 =3.07 3.73, URS

P_t=1.7%

A_{st} = 2380 mm^2

Use Φ20 mm diameter bar at spacing = 1000 x 314 / 23 84

say 130 mm Area provided =1000 x 314 / 130 = 2415 mm^2

Check the depth for one - way shear considerations

Design shear force=V_u=177x(0.825-0.140)=121kN

Nominal shear stress=τ_v=V_u/bd=121000/(1000x140)=0.866MPa

Permissible shear stress

P_t = 100 x 2415 /(1000 x 140) = 1.7 %, τ_{uc} = 0.772 N/mm^2 Value of k for 200 mm thick slab = 1.2

Permissible shear stress = 1.2 x 0.772 = 0.926 N/mm^2 $\tau_{uc} > \tau_v$ and hence safe

The depth may be reduced uniformly to 150 mm at the edges.

Check for development length

L_{dt}= [0.87 x 250 / (4 x 1.4)]Φ =39 Φ = 39 x 20 = 780 mm

Available length of bar = 825 - 25 =800mm > 780 mm and hence safe.

Transverse reinforcement

Required A $_{st}$ = 0.15 bD / 100 = 0.15 x 1000 x 200/100 = 300mm^2

Using Φ 8 mm bars, spacing = 1000 x 50 / 300 = 160 mm Provide distribution steel of Φ 8 mm at 160 mm c/c

(c) Design of Longitudinal Beam:

Two columns are joined by means of a beam monolithic with the footing slab. The load from the slab will be transferred to the beam. As the width of the footing is 2 m, the net upward soil pressure per meter length of the beam

= w_u = 177 x 2 = 354 kN/m

Shear Force and Bending Moment

V_{AC} = 354 x 0.9 = 318.6 kN, V_{AB} = 1050-318.6 = 731.4 kN

V_{BD} = 354 x 1.7 = 601.8kN, V_{BA} = 1500-601.8 = 898.2 kN

Point of zero shear from left end C

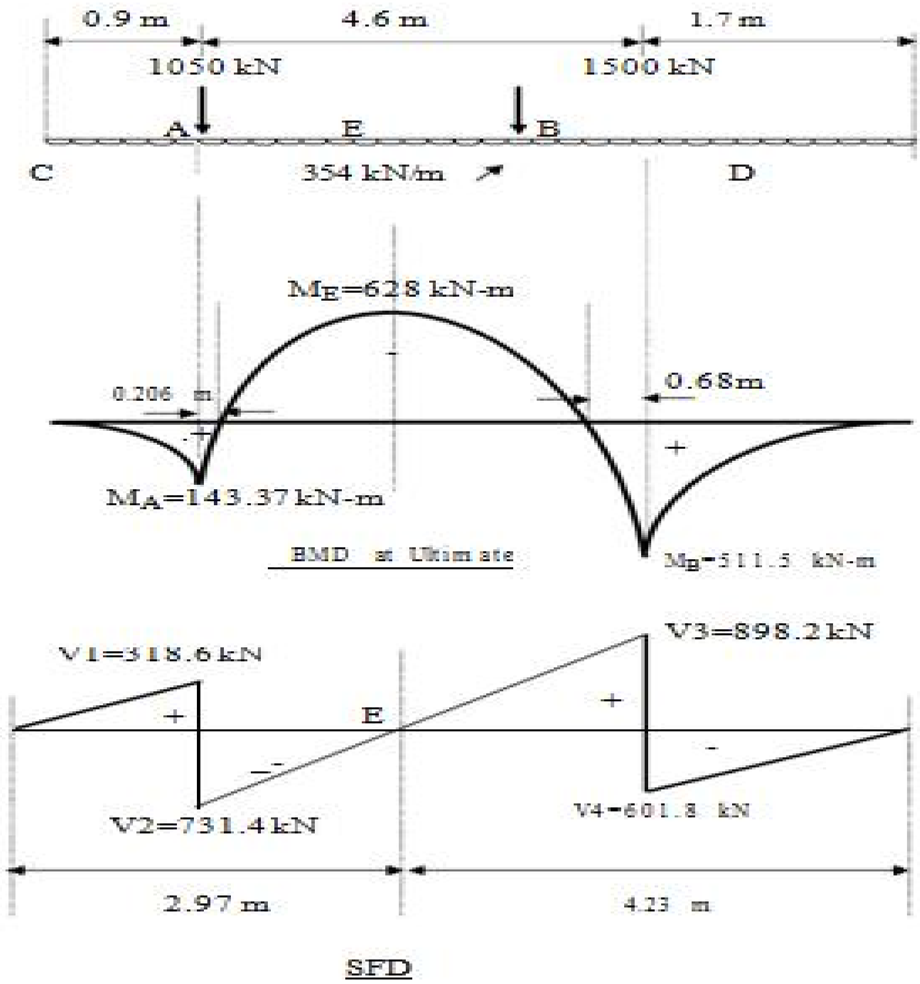
X_1 = 1050/354 = 2.97m from C or X_2 = 7.2-2.97 = 4.23 m from D Maximum B.M. occurs at a distance of 4.23 m from D

M $_{uE}$ = 354 x 4.23^2 / 2 - 1500 (4.23 - 1.7) = -628 kN.m

notes4free All in one

Bending moment under column A = M_{uA} = 354 x 0.9^2 / 2 = 143.37 kN.m Bending moment under column B = M_{uB} = 354 x 1.7^2 = 511.5 kN-m Let the point of contra flexure be at a distance x from the centre of column A Then, M_x= I050x - 354 (x + 0.9 $)^2$/ 2 = 0

Therefore x = 0.206 m and 3.92 m from column A i.e. 0.68 m from B.



Depth of beam from B.M. Considerations

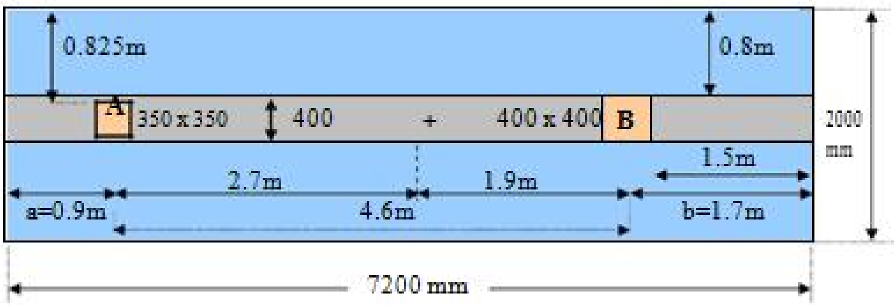
The width of beam is kept equal to the maximum width of the column i.e. 400 mm. Determine the depth of the beam where T- beam action is not available. The beam acts as a rectangular section in the cantilever portion, where the maximum positive moment = 511.5 kN/m.

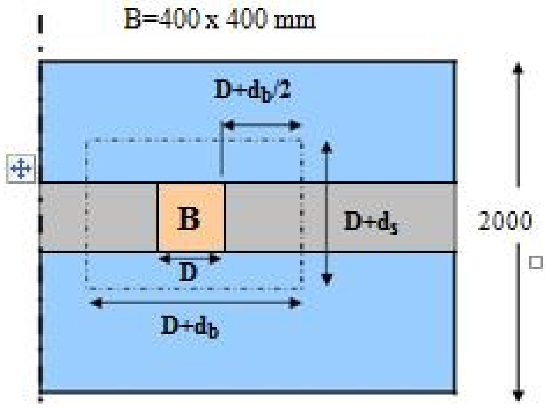
d =√ (511.5 x 10^6/ (3.73 x 400)) = 586 mm

Provide total depth of 750 mm. Assuming two rows of bars at an effective cover of 70 mm. Effective depth provided = d= 750-70 = 680 mm (Less than 750mm and hence no side face steel is needed).

Check the depth for Two-way Shear

The column B can punch through the footing only if it shears against the depth of the beam along its two opposite edges, and along the depth of the slab on the remaining two edges. The critical section for two-way shear is taken at distance d/2 (i.e. 680/2 mm) from the face of the column. Therefore, the critical section will be taken at a distance half the effective depth of the slab ($d_s/2$) on the other side as shown in Fig.





In this case b = D = 400 mm, d_b = 680 mm, d_s = 140 mm Area resisting two - way shear

$=2(b \times d_b + d_s \times d_s) + 2(D + d_b)ds$

$=2(400 \times 680 + 140 \times 140) + 2(400 + 680)\,140$

885600 mm^2

Design shear=P_{ud}= column load – W_u x area at critical section = 1500 - 177 x$(b + d_s)$ x $(D + d_b)$

=1500-177 x (0.400+0.140) x (0.400+ 0.680)

=1377.65kN

$\tau_v = P_{ud}/b_o d$= 1377.65x1000/885600=1.56 MPa

Shear stress resisted by concrete = τ_{uc} =

τ_{uc} x K_s

where, $\tau_{uc} = 0.25\sqrt{f_{ck}} = 0.25\sqrt{25} = 1.25$ N/mm^2

$K_s = 0.5 + d / D = 0.5 + 400/400 = 1.5 \leq$

1 Hence $K_s = 1$

$\tau_{uc} =$ 1 x 1.25 = 1.25 N/mm^2

Therefore unsafe and the depth of slab need to be increased. However the same depth is taken.

Area of Reinforcement

Cantilever portion BD

Length of cantilever from the face of column = 1.7 - 0.4 / 2 = 1.5 m. Ultimate moment at the face of column = 354 x 1.5^2 / 2 = 398.25 kN-m

M_{umax} = 3.71 x 400 x 680^2 x 10^6 = 686 kN.m >398.25 kN-m

Therefore Section is singly reinforced.

M_u/bd =398.25x10 /(400x680) =2.15 3.73, URS

A_{st} =3030 mm^2

Provide 3 - Φ 32 mm + 4 - Φ 16 mm at bottom face, Area provided = 3217 mm^2 L_d = 39 x 32 =1248 mm

Curtailment

All bottom bars will be continued up to the end of cantilever. The bottom bars of 3 - Φ 32 will be curtailed at a distance d (= 680 mm) from the point of contra flexure (λ = 680 mm) in the portion

BE with its distance from the centre of support equal to 1360 mm from B.

Cantilever portion AC

Length of cantilever from the face of column = 900 - 350 / 2 = 725 mm Ultimate moment = 354 x 0.725^2 /2 = 93 kN-m

M_u/bd =93x10 /(400x680) =0.52 3.73, URS

A_{st} =660 mm^2

Provide 4 - Φ 16 mm at bottom face, Area provided = 804 mm^2

Continue all 4 bars of 16 mm diameter through out at bottom.

Region AB between points of contra flexures

notes4free All in one

The beam acts as an isolated T- beam.

b_f=[L $_o$ /(L $_o$ / b +4)] + b $_{w,}$ where,

L $_o$ 4.6 - 0.206 - 0.68 = 3.714 m = 3714 mm

b= actual width of flange = 2000 mm, b $_w$ = 400 mm

b_f = [3714 / (3714 / 2000 + 4)+ 400] =1034mm < 2000mm

D $_f$ = 200 mm, M_u = 628 kN-m

Moment of resistance M_{uf} of a beam for x $_u$ = D $_f$ is :

(M_{uf}) = [0.36 x 25 x 1034 x 200 (680 - 0.42x200)]

x10^{-6} = 1109 kN.m > M_u (= 628 kN-m)

Therefore X_u <D $_f$

$M_u=0.87f_yA_{st}(d-f_yA_{st}/f_{ck}b_f)^2$

Provide 5 bars of Φ 32 mm and 3 bars of Φ 16 mm,

Area provided = 4021+ 603 = 4624 mm^2 >4542

mm^2 p_t= 100 x 4624/(400x680) = 1.7 %

Curtailment

Consider that 2 - Φ 32 mm are to be curtailed

No. of bars to be continued = 3 - Φ16 + 3 - Φ 32 giving area = A_{st} =3016

mm^2 Moment of resistance of continuing bars

M_{ur}= (0.87 x 250 x 3016 (680 – ((250 x 3016) / (25 x 400) x 10^{-6} = 396.6 kN-m Let the theoretical point of curtailment be at a distance X from the free end C, then M_{uc}= M_{ur} Therefore -354 x^2 / 2 + 1050 (x- 0.9) = 396.6

x^2-5.93x + 7.58 =0, Therefore x = 4.06m or 1.86m from C

Actual point of curtailment = 4.06 + 0.68 = 4.74 m from C or 1.86 - 0.68 = 1.18 m from C Terminate 2 - Φ 32 mm bars at a distance of 280 mm (= 1180 - 900) from the column A and 760mm (= 5500 - 4740) from column B. Remaining bars 3 - Φ 32 shall be continued beyond the point of inflection for a distance of 680 mm i.e. 460 mm from column A and up to the outer face of column B. Remaining bars of 3 - Φ 16 continued in the cantilever portion.

Design of shear reinforcement

Portion between column i.e. AB

In this case the crack due to diagonal tension will occur at the point of contra flexure because the distance of the point of contra flexure from the column is less than the effective depth d(= 680mm)

(i) Maximum shear force at B = V_{umax} = 898.2 kN

Shear at the point of contra flexure = V_{uD} - 898.2-354 x 0.68 = 657.48 kN

τ_v=657000/(400x680) =2.42 MPa $\tau_{c,max.}$

Area of steel available 3 - Φ 16 + 3 - Φ 32 , A_{st} = 3016

mm^2 p_t = 100 x 3016 / (400 x 680) = 1.1%

τ_c=0.664MPa

$\tau_v > \tau_c$

Design shear reinforcement is required.

Using 12 mm diameter 4 - legged stirrups,

Spacing = 0.87 x 250 x (4 x 113) /(2.42-0.664)x400 =139 mm say 120 mm c/c

Zone of shear reinforcements between τ_v to τ_c

= m from support B towards A

(ii) Maximum shear force at A = $V_{u\ max}$ = 731.4 kN.

Shear at the point contra flexure = V_{uD} = 731.4 - 0.206

x 354 = 658.5 kN

τ_v=658500/(400x680) =2.42MPa $\tau_{c,max.}$

Area of steel available = 4624 mm^2, p_t= 100 x 4624 / (400 * 680) =

1.7 % τ_{uc} = 0.772 N/ mm^2

$\tau_v > \tau_c$

Design shear reinforcement is required.

Using 12 mm diameter 4 - legged stirrups,

Spacing = 0.87 x 250 x (4 x 113) /(2.42-0.774)x400 =149 mm say 140 mm c/c

Zone of shear reinforcement.

From A to B for a distance as shown in figure

For the remaining central portion of 1.88 m provide minimum shear reinforcement using 12 mm diameter 2 - legged stirrups at

Spacing , s = 0.87 x 250 x (2 x 1 1 3) / (0.4 x 400) = 300 mm c/c< 0.75d

Cantilever portion BD

V_{umax} = 601.8kN, V_{uD} = 601.8 - 354(0.400 / 2 + 0.680) = 290.28kN.

τ_v=290280/(400x680) =1.067MPa < $\tau_{c,max.}$

A_{st} = 3217 mm^2 Therefore p_t = 100 x 3217/(400 x 680) = 1.18%

τ_c =0.683N/mm^2 (Table IS:456-2000)

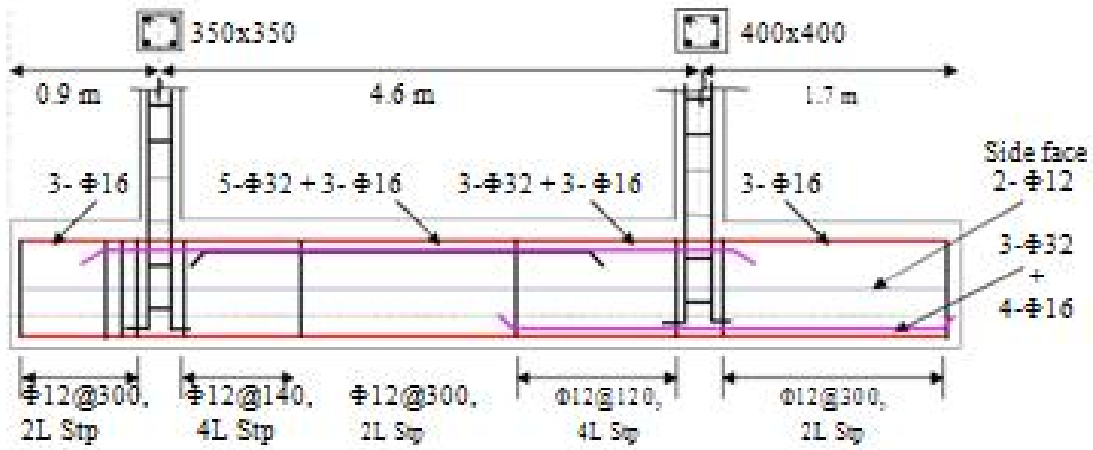
$\tau_v > \tau_c$ and $\tau_v - \tau_c < 0.4$. Provide minimum steel.

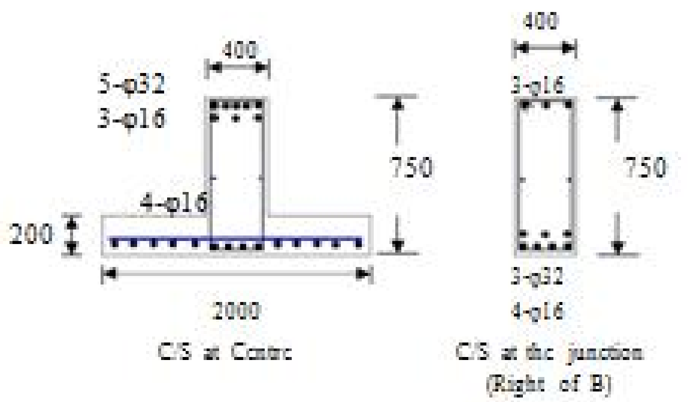
Using 12 mm diameter 2- legged stirrups,

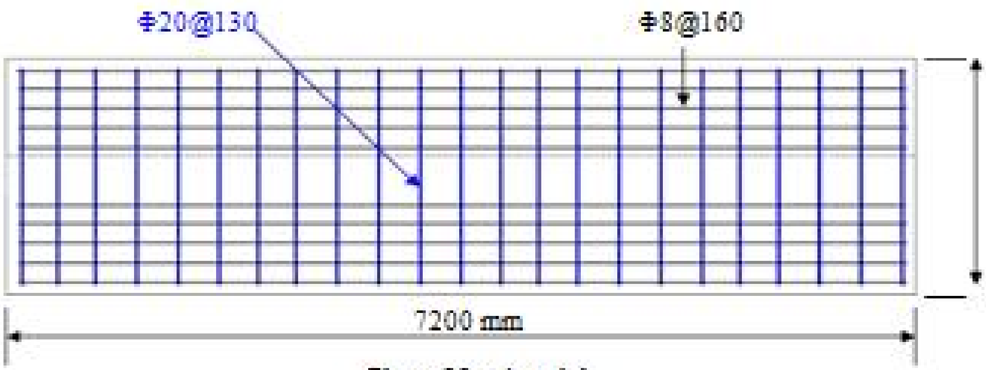
Spacing = 0.87 x 250 x (2 x 113) /(0.4x400) =307.2 mm say 300 mm c/c

Cantilever portion AC

Minimum shear reinforcement of Φ 12 mm diameters 2 - legged stirrups at 300mm c/c will be sufficient in the cantilever portions of the beam as the shear is very less.







Plan of footing slab

1.1.4 Outcome

Able to design and detailing the combined footing as per IS code provisions

1.1.5 Future Study

https://nptel.ac.in/courses/105108069/3

notes4free
All in one

Module – 2

Design of Roof truss

Contents

notes4free
All in one

2.1.1 Introduction

In engineering, a **truss** is a structure that "consists of two-force members only, where the members are organized so that the assemblage as a whole behaves as a single object".[1] A "two-force member" is a structural component where force is applied to only two points. Although this rigorous definition allows the members to have any shape connected in any stable configuration, trusses typically comprise five or more triangular units constructed with straight members whose ends are connected at joints referred to as nodes.

In this typical context, external forces and reactions to those forces are considered to act only at the nodes and result in forces in the members that are either tensile or compressive. For straight members, moments (torques) are explicitly excluded because, and only because, all the joints in a truss are treated as revolutes, as is necessary for the links to be two-force members.

A planar truss is one where all members and nodes lie within a two dimensional plane, while a space truss has members and nodes that extend into three dimensions. The top beams in a truss are called top chords and are typically in compression, the bottom beams are called bottom chords, and are typically in tension. The interior beams are called webs, and the areas inside the webs are called panels

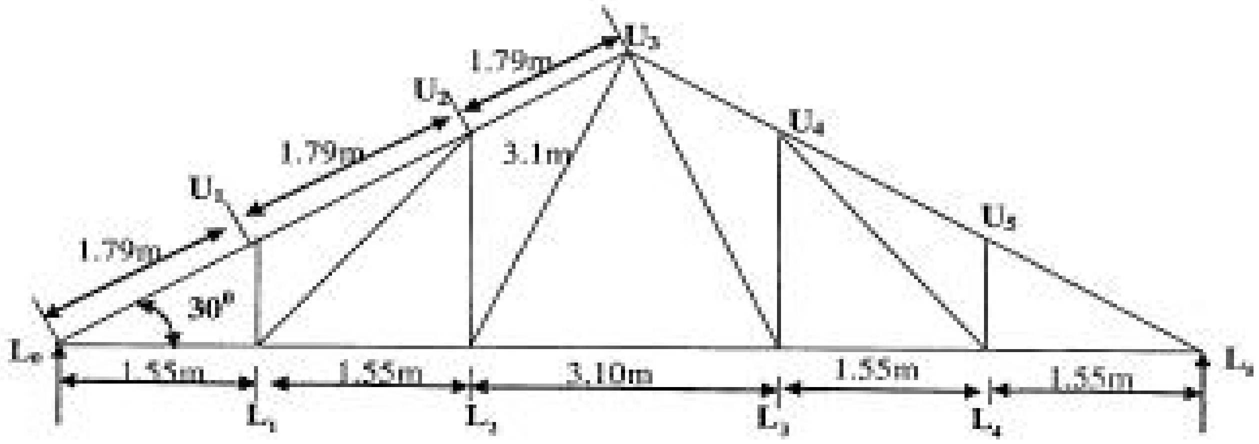
2.1.2 Objective

To study the design criteria of roof truss

notes All in o

2.1.3 Design of Roof truss

The forces in the members of the roof truss of an industrial building are shown in table. The truss is supported on 400mm thick masonry. End reaction due to DL + LL is 10.558kN. Members are to be connected at the joint with 16mm Φ bolts and 8mm thick gusset plate. Design members and base plate. Assume permissible bearing pressure on masonry = 0.8 N/mm² and size of shoe angle ISA 75 × 75 × 6mm on each side of gusset plate. (40 marks)



Members	Force (kN)	
	Compression -ve	Tension +ve
L_0U_1, U_1U_2, U_2U_3	-17.4	20.9
L_0L_1, L_1L_2, L_2L_3	14.9	-14.0
U_3L_2	6.0	-8.7
U_2L_2	-5.3	7.4
U_2L_1	4.6	-6.7
U_1L_1	-3.5	5.0

Solution:

Design of members L_0U_1, U_1U_2, U_2U_3:

Maximum factored compressive force = 1.5 × 17.4 = 26.1 kN

Maximum factored tensile force = 1.5 × 20.9 = 31.35 kN

Length of member, L = 1.79m

Effective length, KL = 0.7 × 1.79 = 1.253 m

Try 2 ISA 50 × 50 × 6mm placed back to back.

The properties of angle section are:

Area, $A = 2 \times 568 = 1136\ mm^2$

Minimum radius of gyration, $r_{min} = 15.1$ mm

Slenderness ratio, $\lambda = \frac{KL}{r_{min}} = \frac{1253}{15.1} = 82.98 < 180$

For $\lambda = 82.98$ and $f_y = 250 N/mm^2$ from Table 8c of IS 800:2007, stress reduction for column buckling class (C) is,

By interpolation, $\chi = 0.6 - \frac{(0.6 - 0.533)}{10} \times (82.98 - 80) = 0.58$

∴ Design compressive stress is given by,

$$f_{cd} = \frac{\chi f y}{\gamma m0} = \frac{0.58 \times 250}{1.25} = 116\ N/mm^2$$

∴ Design compressive force is given by,

$$P_d = Af_{cd} = 1136 \times 116 \times \frac{1}{10^3} = 131.77\ kN > 26.1\ kN$$

Hence the section is safe in compression.

Check for Tension capacity:

Design tensile force is given by,

$$P_d = \frac{131.77}{1.1} = 119.79\ kN > 31.35\ kN$$

Hence the section is safe in tension.

Design of Bottom chord members L_0L_1, L_1L_2, L_2L_3:

Maximum factored compressive force = 1.5 × 14.0 = 21.0 kN

Maximum factored tensile force = 1.5 × 14.9 = 22.35 kN

Length of member, L = 1.55m

Effective length, KL = 0.7 × 1.55 = 1.085m

Try 2 ISA 50 × 50 × 6mm placed back to back. Assume a gusset plate of 8mm thick with 16mm diameter bolts spaced at 50mm c/c.

The properties of angle section are:

Area, $A = 2 \times 568 = 1136\ mm^2$

Design strength due to yielding of gross section:

As per IS800:2007 clause 6.3.2

$$T_{dg} = \frac{A_g f_y}{\gamma m0} = \frac{1136 \times 250}{1.10 \times 10^3} = 258.18 \text{ kN}$$

Design strength governed by tearing or rupture at net section:

As per IS800:2007 clause 6.3.3

Assuming that the 16mm diameter bolts are provide in a single line with spacing of 50mm,

$\alpha = 0.6$ for one or two bolts.

$d = 16$mm

$d_o = 16 + 2 = 18$mm

$A_n = 2 \times 6 \times [50 - 18] = 384 \text{mm}^2$

$$T_{dn} = \frac{\alpha A_n f_u}{\gamma m1} = \frac{0.6 \times 384 \times 410}{1.25 \times 10^3} = 75.57 \text{ kN} > 22.35\text{kN}$$

Hence the section is safe.

Design of member U_7L_1 *:*

Maximum factored compressive force = 1.5 × 8.7 = 13.05 kN

Maximum factored tensile force = 1.5 × 6.0 = 9.0 kN

Length of member, L = 3.10 m

Try a single ISA 60 × 60 × 5mm placed back to back. Assume a gusset plate of 6mm thick with 16mm diameter bolts spaced at 50mm c/c.

The properties of angle section are:

Area, A = 575 mm²

$d = 16$mm

$d_o = 16 + 2 = 18$mm

$A_g = 575 \text{ mm}^2$

$A_{nc} = 5 \times [60 - 18] = 210 \text{ mm}^2$

$A_{go} = 5 \times [60 - 5] = 275 \text{ mm}^2$

Design strength due to yielding of gross section:

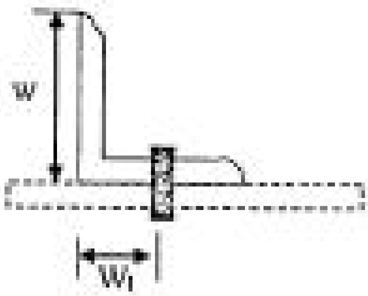
As per IS800:2007 clause 6.3.2

$$T_{dg} = \frac{A_g f_y}{\gamma m0} = \frac{575 \times 250}{1.10 \times 10^3} = 130.68 \text{ kN}$$

Design strength governed by tearing or rupture at net section:

As per IS800:2007 clause 6.3.3

Assuming that the 16mm diameter bolts are provide in a single line with spacing of 50mm,



$b_s = w + w_l - t$

$$T_{dn} = \frac{0.9\,A_{nc} fu}{\gamma m1} + \frac{\beta A_{go} f_y}{\gamma m0}$$

L_c = Length of end connection = 50mm

b_s = shear lag width = w + w_1 - t = 60 + 30 – 5 = 85

$$\beta = 1.4 - 0.076\left(\frac{w}{t}\right)\left(\frac{f_y}{f_u}\right)\left(\frac{b_s}{L_c}\right) = 1.4 - 0.076\left(\frac{60}{5}\right)\left(\frac{250}{410}\right)\left(\frac{85}{50}\right) = 0.4546$$

$$T_{dn} = \frac{0.9 \times 210 \times 410}{1.25} + \frac{0.4546 \times 275 \times 410}{1.10} = 117041.95\,N = 117.04\ kN$$

Strength governed by block shear:

As per IS800:2007 clause 6.4.1

The block shear strength T_{db} shall be smaller of T_{db1} and T_{db2}:

$$T_{db1} = \frac{A_{vg} fy}{\sqrt{3}\,\gamma mo} + \frac{0.9 A_{tn} f_u}{\gamma m1}$$

$$T_{db2} = \frac{0.9 A_{vn} f_u}{\sqrt{3}\,\gamma m1} + \frac{A_{tg} f_y}{\gamma mo}$$

A_{vg} = 5 x [60 + 50] = 550 mm²

A_{vn} = 5 x [60 + 50] – [1.5 x 18] = 523mm²

$$A_{tg} = 5 \times \frac{60}{2} = 150\ mm^2$$

$$A_{tn} = \left[5 \times \frac{60}{2}\right] - [0.5 \times 18] = 141\ mm^2$$

$$T_{db1} = \frac{550 \times 250}{\sqrt{3} \times 1.10} + \frac{0.9 \times 141 \times 410}{1.25} = 113791.98\ N = 113.79\ kN$$

$$T_{db2} = \frac{0.9 \times 523 \times 410}{\sqrt{3} \times 1.25} + \frac{150 \times 250}{1.10} = 123227.78\,N = 123.23\ kN$$

$\therefore T_{db}$ = 113.79kN

The design strength of member under axial tension is the lowest of the above three values:

T_{dg} = 130.68 kN

T_{dn} = 117.04 kN

T_{db} = 113.79 kN

∴ The design tensile strength of the angle = 113.79kN > 9.0 kN

Hence the section is safe against axial compression also.

Design of members L_1U_2, L_1U_2:

Maximum factored compressive force = 1.5 × 5.3 = 7.95 kN

Maximum factored tensile force = 1.5 × 7.4 = 11.1 kN

Length of member, L = 1.79 for L_1U_2

Effective length, KL = 0.7 × 1.79 = 1.253 m

Try a single ISA 50 × 50 × 6mm.

The properties of angle section are:

Area, A = 568 mm^2

Minimum radius of gyration, r_{min} = 15.1 mm

$$\text{Slenderness ratio, } \lambda = \frac{KL}{r_{min}} = \frac{1253}{15.1} = 82.98 < 180$$

For $\lambda = 82.98$ and $f_y = 250\text{N/mm}^2$ from Table 8c of IS 800:2007, stress reduction for column buckling class (C) is,

$$\text{By interpolation, } \chi = 0.6 - \frac{(0.6-0.533)}{10} \times (82.98-80) = 0.58$$

∴ Design compressive stress is given by,

$$f_{cd} = \frac{\chi f_y}{\gamma m0} = \frac{0.58 \times 250}{1.25} = 116 \text{ N/mm}^2$$

∴ Design compressive force is given by,

$$P_d = Af_{cd} = 568 \times 116 \times \frac{1}{10^3} = 65.89 > 7.95 \text{ kN}$$

Hence the section is safe in compression.

Check for Tension capacity:

Design tensile force is given by,

$$P_d = \frac{65.89}{1.1} = 59.9 \text{ kN} > 11.1 \text{ kN}$$

Hence the section is safe in tension.

Since the other members are not severely loaded, a single ISA 50x50x6mm may be used for U_1L_1 and U_2L_1.

Design of joints:

Joint L_0:

Maximum factored force in member L_0L_1 = 1.5 × 20.9 = 31.35 kN

Maximum factored force in member L_0U_1 = 1.5 × 14.9 = 22.35 kN

Using 16mm diameter bolts and 8mm thick gusset plate,

d = 16mm

$d_o = 16 + 2 = 18$ mm

$$A_{sb} = \frac{\pi \times 16^2}{4} = 201.6 \text{ mm}^2$$

$A_{nb} = 0.8\ A_{sb} = 0.8 \times 201.6 = 160.85\text{mm}^2$ (for ISO threads)

Strength of blot in shear:

AS per IS800:2007 clause 10.3.3,

The design strength of bolt $V_{dsb} = \dfrac{V_{nsb}}{\gamma_{mb}}$

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

For double shear plane, $n_n = 1$, $n_s = 1$

Strength of bolt in double shear,

$$V_{dsb} = 400\,(1 \times 201.6 + 1 \times 160.8) \times \frac{1}{\sqrt{3} \times 1.125 \times 10^3} = 66.86 \text{ kN}$$

Strength of bolt in bearing:

AS per IS800:2007 clause 10.3.4,

The design strength of bolt $V_{dpb} = \dfrac{V_{npb}}{\gamma_{mb}}$

$$V_{npb} = 2.5 k_b\ d\ t\ f_u$$

Where k_b is smaller of $\dfrac{e}{3d_0}, \dfrac{p}{3d_0} - 0.25, \dfrac{f_{ub}}{f_u}, 1.0$

e = end distance = 1.7 d = 1.7 × 16 = 30.6mm ≈ 30mm

p = pitch = 50mm

$$k_b = \frac{30}{3 \times 18} = 0.5555$$

$$k_b = \frac{50}{3 \times 18} - 0.25 = 0.6759$$

$$k_b = \frac{400}{410} = 0.9756$$

$k_s = 1.0$

$\therefore k_b = 0.5555$

Design strength of joint in bearing,

$$V_{dpb} = \frac{2.5 k_b d t f_u}{\gamma mb} = \frac{2.5 \times 0.5555 \times 16 \times 6 \times 410}{1.25} = 43728.96N = 43.72 \text{ kN}$$

Hence design strength of bolt = 43.72kN

$$\text{Number of bolts required} = \frac{31.35}{43.72} = 0.72$$

Therefore provide a minimum number of 2 bolts spaced at 50mm c/c.

Joints U_1, U_2, U_3, L_1 and L_2 joints:

The member forces joining at these joints are very small when compared to joint L_0. Therefore provide a minimum number of 2 bolts spaced at 50mm c/c.

Design of Shoe angle at supports:

Provide 2 ISA 75 × 75 × 6mm shoe angle with 300mm long at support.

Factored reaction at support = 1.5 × 10.558 = 15.84 kN

Provide minimum number of 2 bolts 16mm diameter spaced at 50mm c/c.

Design of Bearing Plate:

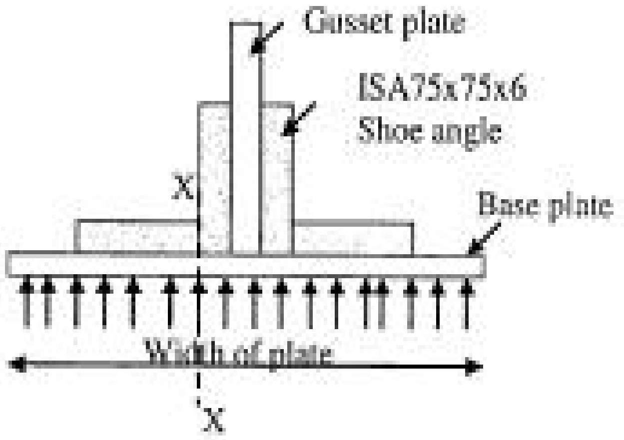
Assume a bearing plate length = 300mm

Width of bearing plate = 75 + 75 + 6 = 156mm

Therefore provide a bearing plate of size 300 × 300 mm at support.

Provide masonry below bearing plate.

Allowable bearing pressure of concrete = 0.8N/mm^2



$$\text{Pressure on concrete bearing pad} = \frac{15.84 \times 1000}{300 \times 300} = 0.176 \ \text{N/mm}^2 < 0.8\text{N/mm}^2$$

Hence safe.

Consider 1mm width of base plate.

$$\text{Bending moment about X} - \text{X} = \frac{0.176 \times (75-6)^2}{2} = 418.97 \ \text{N / mm}$$

$$\text{Design bending stress, } f_{bd} = \frac{\chi_{LT} f_y}{\gamma m0} = \frac{1.0 \times 250}{1.10} = 227.27 \ \text{N / mm}^2$$

$$\text{Therefore thickness of plate, } t = \sqrt{\frac{6M}{f_{bd} b}} = \sqrt{\frac{6 \times 418.97}{227.27 \times 1}} = 3.33 \ \text{mm}$$

$\therefore$ Provide 6mm thick base plate.

Anchor bolts:

Assume Pull in anchor bolt = 30.0 kN

Allowable axial tension in anchor bolt = 150N/mm^2

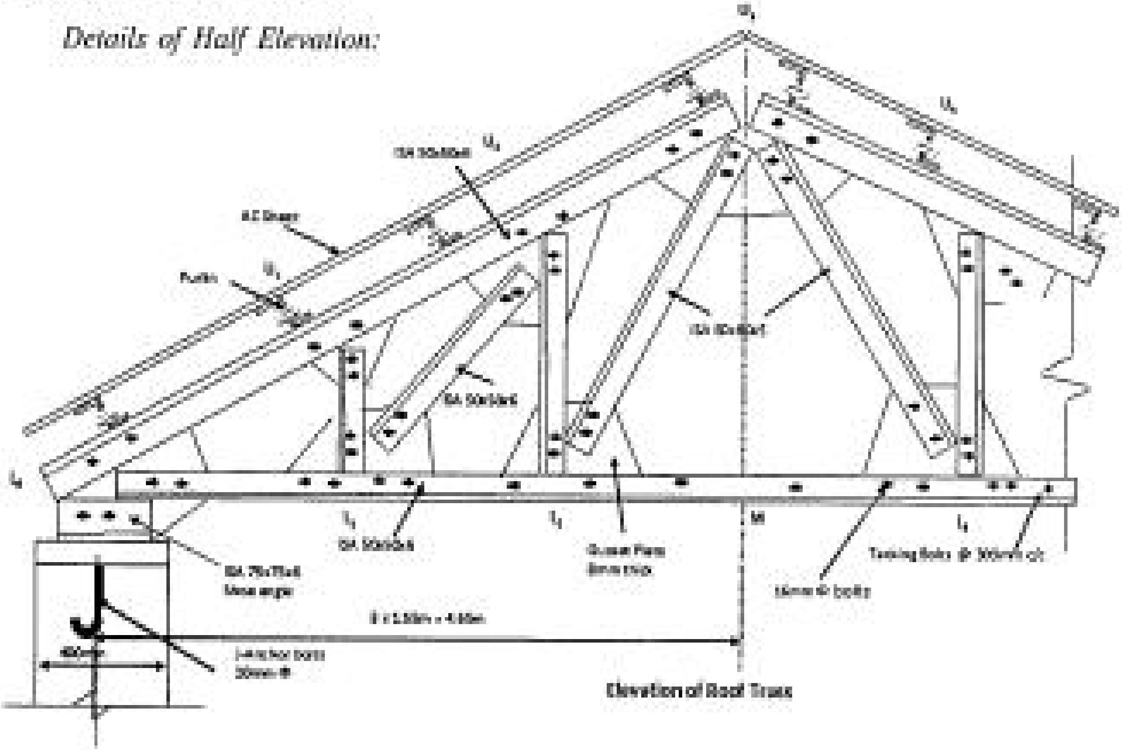
$$\text{Area of anchor bolt required} = \frac{30.0 \times 1000}{150} = 206 \ \text{mm}^2$$

Provide two 20mm J-anchor bolts.

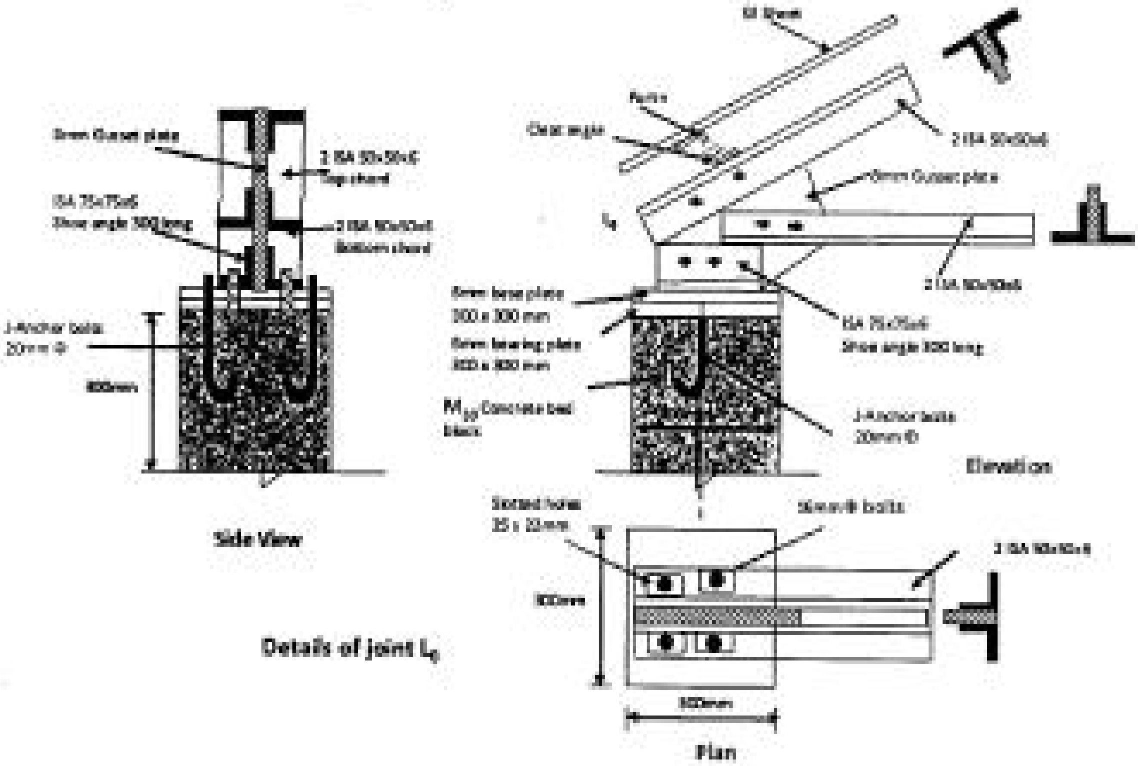
Provide 22 × 50mm slot holes to allow sliding due to temperature variations.

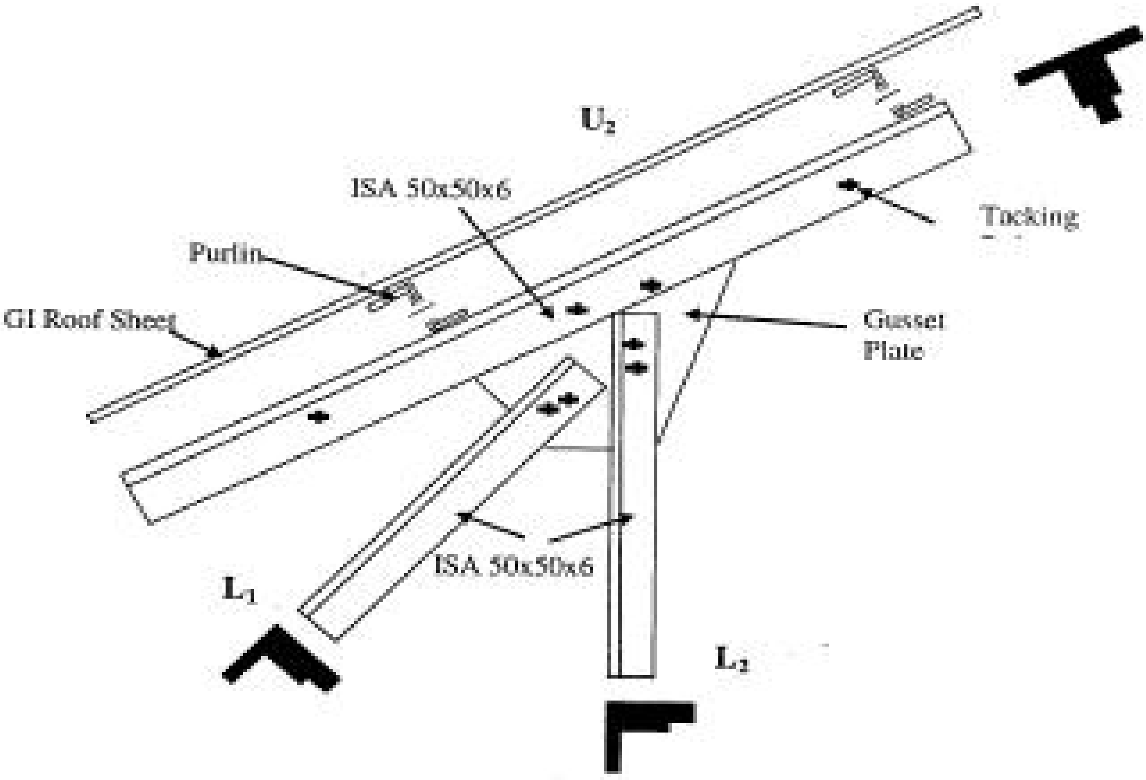
Details of Roof Truss:

Details of Half Elevation:



Connection details at joints L_0:





Details of Joint U_2

2.1.4 Outcome

One can be able to know design criteria of roof truss

2.1.5 Future study

http://nptel.ac.in/courses/105103094/

3.3.1 Introduction

Gantry girders are laterally unsupported beams to carry heavy loads from place to place at the construction sites, mostly these are of steel material.

A girder is a support beam used in construction. It is the main horizontal support of a structure which supports smaller beams. Girders often have an I-beam cross section composed of two load-bearing *flanges* separated by a stabilizing *web*, but may also have a box shape, Z shape and other forms. A girder is commonly used many times in the building of bridges.

Gantry cranes are a type of crane built atop a gantry, which is a structure used to straddle an object or workspace.

Overhead travelling cranes are used in factories and workshops to lift heavy materials equipments, etc and to carry them from one place to the other. These cranes are either hand operated or electrically operated. The crane consists of a bridge spanning the bay of the shop.

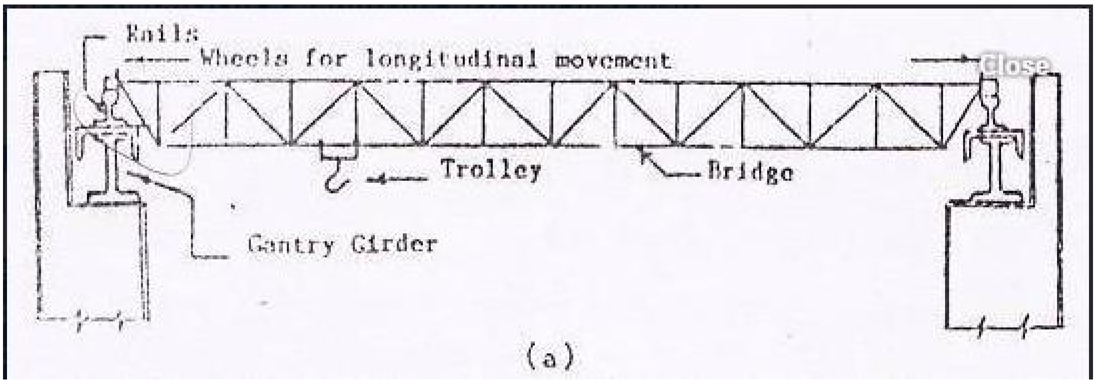
A trolley or a crab is mounted on the bridge. The trolley moves along the bridge. The bridge as a whole moves longitudinally on rails provided at the ends.

The rails on either side of the bridge rest on crane gantry girders. The gantry girders are the girders which support the loads transmitted through the travelling (moving) wheels of the cranes as shown in figure below

notes4free All in one

3.3.2 Objective

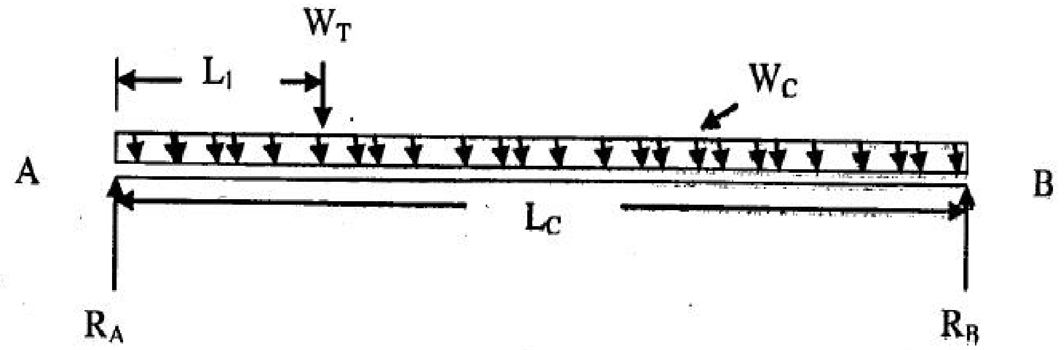
To study the design criteria of gantry girders



3.3.4 Design procedure

The design of a gantry girder is a trial method:

1. Finding the maximum wheel load on the gantry girder: The maximum wheel load occurs when the trolley is closer to the gantry girder.



W_T = Weight of trolley + Capacity of Crane (Hook Load)

W_C = Weight of Crane girder/unit length

L_1 = Minimum approach of Crane hook i.e. distance between CG of gantry girder and Trolley
L_C = Span of Crane girder

Take $\sum M_A = 0$

$$R_B \times L_C = W_C\, L_C \frac{L_C}{2} + W_T L_1$$

Take $\sum V = 0$

$$R_A + R_B = W_C\, L_C + W_T$$

After solving the above equations, we get,

$$R_A = \frac{1}{L_C}\left[\frac{W_C L_C^2}{2} + W_T\,(L_C - L_1)\right]$$

Therefore the wheel load R_A of Crane girder is distributed on Two wheels of a gantry girder

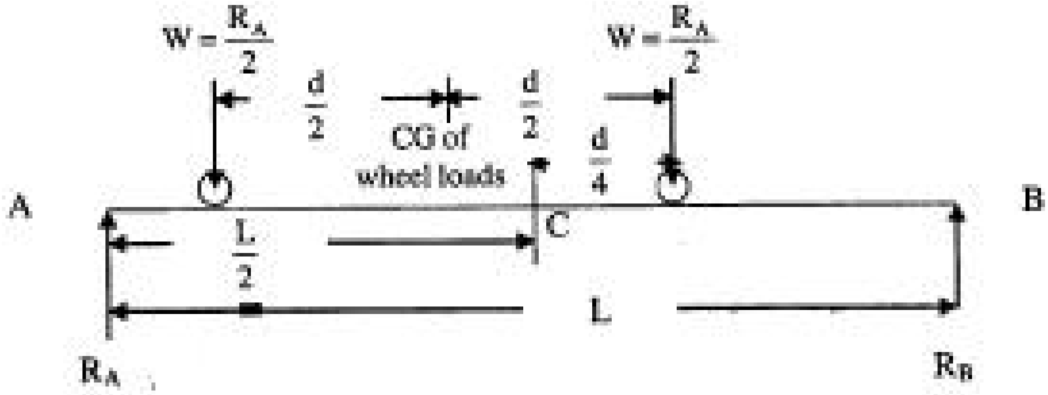
$$W = \frac{R_A}{2}$$

2. Finding the maximum Bending moment in the gantry girder due to vertical loads:

   Maximum Bending moment,

   BM = BM due to (max. wheel load + Impact + DL of girder + Self weight of Rails)

The maximum bending moment due to wheel loads occur when the CG of wheel loads and one of the wheel loads are equidistant from center of gantry girder i.e the quarter distance of the span of wheels must coincide with the center of the girder.



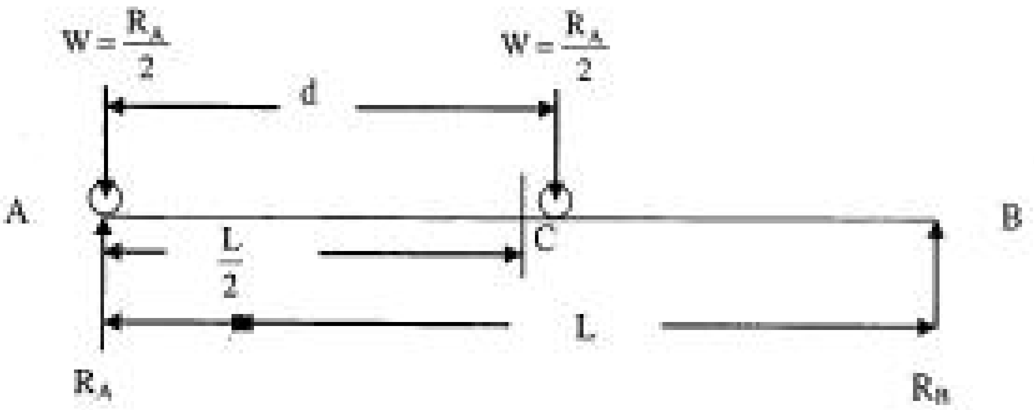
Where L= Span of Gantry girder

3. Finding the maximum Shear force in the gantry girder due to vertical loads:

   Maximum Shear force,

   SF = SF due to (max. wheel load + Impact + DL of girder + Self weight of Rails)

The maximum Shear force occurs when both wheels loads are on the girder and one of the wheels is at support.



Where d= wheel base

4. Finding the maximum Bending moment and Shear force due to Lateral forces are similar to the steps in 2 and 3 and Referring to IS875(Part 2):1987
   - 10% of the weight of the crab and the maximum weight of lifted by the crane For EOT cranes.
   - 5% of the weight of the crab and the maximum weight of lifted by the crane For Hand operated cranes.

   This load should be distributed to all the wheels of the crane.
5. Selection of Preliminary section of the Gantry girder:

   The selection of preliminary section of the Gantry girder is based on trials sections and an I-section with a channel section on its top is the most suitable built-up section for the gantry girder.

   Economical depth of the girder $= \frac{L}{12}$

   Compression flange width $= \frac{L}{30}$ to $\frac{L}{40}$ to prevent excessive lateral deflection
6. Calculate: I_{xx}, I_{yy} and $Z_p = \frac{M_r}{f_y}$

   Z_p of the trial section is 40% to 50% greater than the required to resist combined moment safely.
7. Check the moment capacity of section as per IS800:2007 clause 8.2.1.2.
8. Check for buckling resistance when the top flange (compression flange) is not supported as per IS800:2007 clause 8.2.2.
9. Check for local crushing for concentrated load points (wheel reactions).

10. Check for maximum deflection under service loads.
11. Design the connection details.

Problem 1: (VTU June/July 2011, marks 70)

Design a gantry girder to an industrial shed to support an electric overhead traveling crane using the following data:

Crane capacity = 200kN

Weight of crane(excluding trolley = 300kN

Minimum approach for crane hook = 1m

Span of gantry girder = 6m

Height of rail section = 60mm

Weight of crab(Trolley) = 80kN

Span of crane girder between rails = 18m

Wheel base = 3m

Mass of rail section = 250N/m

Use f_y = 250 MPa

Draw to a suitable scale following: (marks 35)

a. Top view (marks 14)
b. Front view (marks 14)
c. Section of Gantry girder (marks 07)

Solution:

Calculation of loads:

1. Vertical loads:

Self weight of crane girder W_c = 300 kN

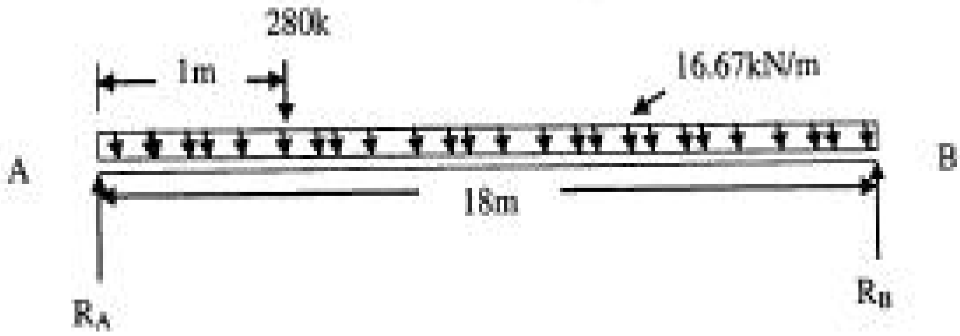
Self weight of crane girder per meter length, W_c = 300/18 =16.67 kN/m

Weight of Trolley = 80 kN

Crane capacity = 200 kN

∴ Crane load, W_T = Crane capacity + Weight of Trolley

= 200 + 80 = 280 kN



Take $\sum M_B = 0$

$$R_A \times 18 = 16.67 \times 18 \times \frac{18}{2} + 280 \times 17$$

$R_A = 414.47$ kN

This load R_A is distributed on two wheels.

$\therefore$ Load on gantry girder from each wheel $= \frac{414.47}{2} = 207.24$ kN

To allow for Impact etc., the above load should be increased by 25% (EOT cranes)

$\therefore$ Design load $= 1.25 \times 207.24 = 259.05$ kN

$\therefore$ Factored Design load on each wheel $= 1.5 \times 259.05 = 388.58$ kN

Lateral Loads:

Horizontal breaking load:

Horizontal force along rails = 5% of the wheel load

$= 0.05 \times 259.05 = 12.95$kN

Factored Horizontal load, $P_g = 1.5 \times 12.95 = 19.43$kN

Horizontal Surge load:

Assuming 4 wheels,

Horizontal Surge load per wheel = 10% (Hook load + Trolley load)/4.0

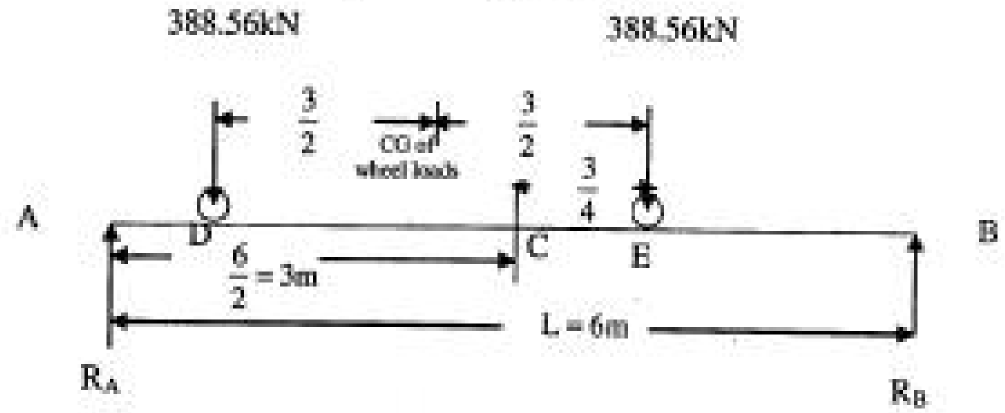
$$= \frac{10 \times (200 + 80)}{100 \times 4} = 7 \text{ kN}$$

Calculation of Maximum Bending moment:

Vertical Bending Moment:

The maximum bending moment due to wheel loads occur when the CG of wheel loads and one of the wheel loads are equidistant from center of gantry girder

d = Wheel base = 3m; Span of Gantry girder = 6m



Take $\sum M_A = 0$

$R_B \times 6 = 388.56 \times 0.75 + 388.56 \times (3 + 0.75) = 1748.52$

$\therefore R_B = 291.42kN$

Maximum Moment will occur at E,

$M_E = 291.42 \times (3 - 0.75) = 655.70 \text{ kN} - m$

BM due to Dead Load of Girder:

Assume Self-weight of the Gantry girder = 2.0kN/m

Given: Self-weight of Rails = 300N/m = 0.3kN/m

Total Dead Load of Girder = 2.0 + 0.3 = 2.3 kN/m

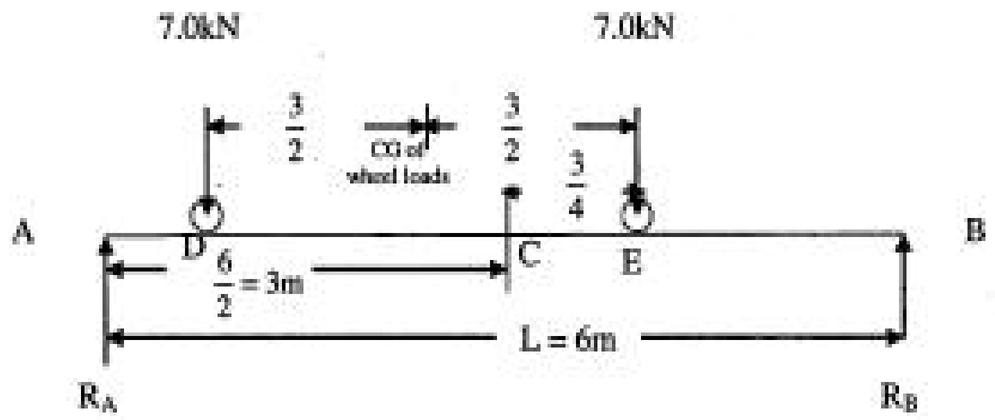
Factored Dead Load of Girder = 1.5 × 2.3 = 3.45 kN/m

BM due to Deal load of Girder $= \frac{WL^2}{8} = \frac{3.45 \times 6^2}{8} = 15.53 \text{ kN} - m$

$\therefore$ Factored moment due to Vertical loads = 655.70 + 15.53 = 671.23kN-m

Horizontal Bending Moment:

Since the procedure is same as that of Vertical BM, we can calculate the BM for horizontal loads by proportioning of vertical loads



$M_y = \frac{655.70}{388.56} \times 7.0 = 11.81 \text{ kN} - m$

BM due to Horizontal drag force:

Height of rail section = 60mm = 0.06m

Assume depth of girder = 0.6m

Eccentricity, e = depth of girder/2.0 + Height of rail $= \frac{0.6}{2.0} + 0.06 = 0.36$ m

L= 6.0m

Reaction due to drag force, $R = \frac{P_g.e}{L} = \frac{19.43 \times 0.36}{6.0} = 1.17$ kN

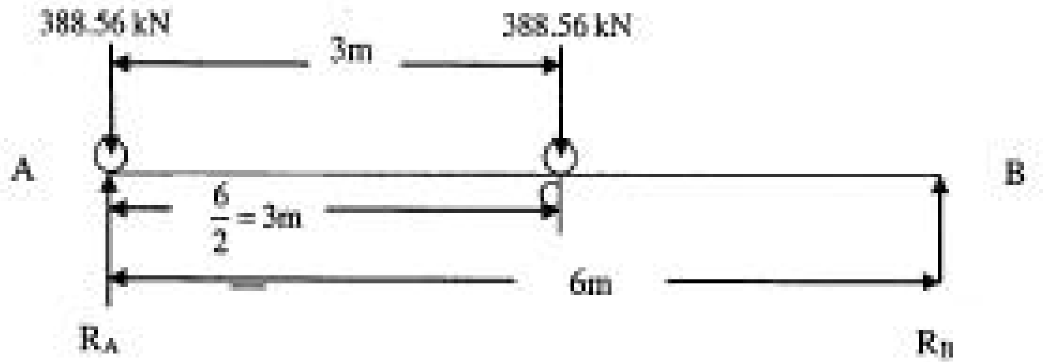
Moment due to drag force $= R \times \left(\frac{L}{2} - \frac{d}{4}\right) = 1.17 \times \left(\frac{6.0}{2} - \frac{3.0}{4}\right) = 2.63$ kN – m

Total design moment, $M_z = 671.23 + 2.63 = 674.35$kN-m

Calculation of Maximum Shear Force:

Vertical Shear force:

The maximum Shear force occurs when both wheels loads are on the girder and one of the wheels is at support.



Take $\sum M_B = 0$

$R_A \times 6 = 388.56 \times 6.0 + 388.56 \times 3.0 = 3497.4$

$\therefore R_A = 582.84$kN

Vertical Shear due to Dead load of the Gantry girder $\frac{WL}{2} = \frac{3.45 \times 6.0}{2} = 10.35$ kN

Horizontal Shear force due to surge load:

$V_y = W_{Surge}\left(2 - \frac{d}{L}\right) = 7.0 \times \left(2 - \frac{3}{6}\right) = 10.50$ kN

$\therefore$ Maximum Ultimate Reaction, V_z = SF (Crane load +DL of Girder + Drag)

$= 582.84 + 10.35 + 1.17 = 594.36$kN

Selection of Preliminary Section:

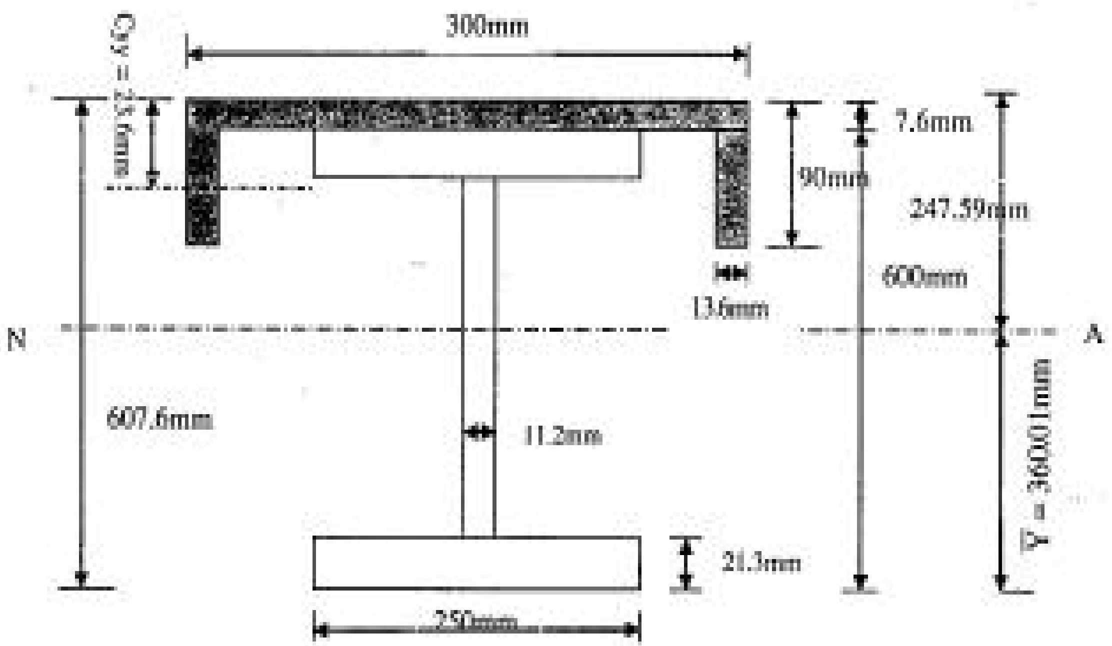
Economical depth of the girder $= \frac{L}{12} = \frac{6000}{12} = 500$ mm

Compression flange width $= \frac{L}{30} = \frac{600}{30} = 200$ mm

Approximate Section modulus required is 40% more than the actual.

$$Z_{PZ} = 1.4\frac{M_Z}{f_y} = 1.4 \times \frac{674.35 \times 10^3}{250} = 3.78 \times 10^6 \, mm^3$$

Referring to Steel Table, Select ISWB600@1.311kN/m and ISMC300@3.512kN/m on compression flange as shown in figure below.



Properties of Built-up Section:

Properties	ISWB600@1.311kN/m	ISMC300@3.512kN/m
Area	17038 mm²	4564 mm²
b_f	250mm	90mm
t_f	21.3mm	13.6mm
t_w	11.2mm	7.6mm
I_{ZZ}	106198.5 x 10⁴ mm⁴	6362.6 x 10⁴ mm⁴
I_{YY}	4702.5 x 10⁴ mm⁴	310.8 x 10⁴ mm⁴
C_{YY}	-	23.6mm

Calculating Moment of Inertia of Gantry Girder:

Let the distance of NA of built-up section from the tension flange (bottom) be $\overline{Y}$.

$$\overline{Y} = \frac{\Sigma AY}{\Sigma A} = \frac{17038 \times 300 + 4564\ (600 + 7.6 - 23.6)}{17038 + 4564} = 360.01 \text{ mm}$$

$$I_{ZZ\ gross} = I_{Z\ beam} + I_{Z\ channel}$$

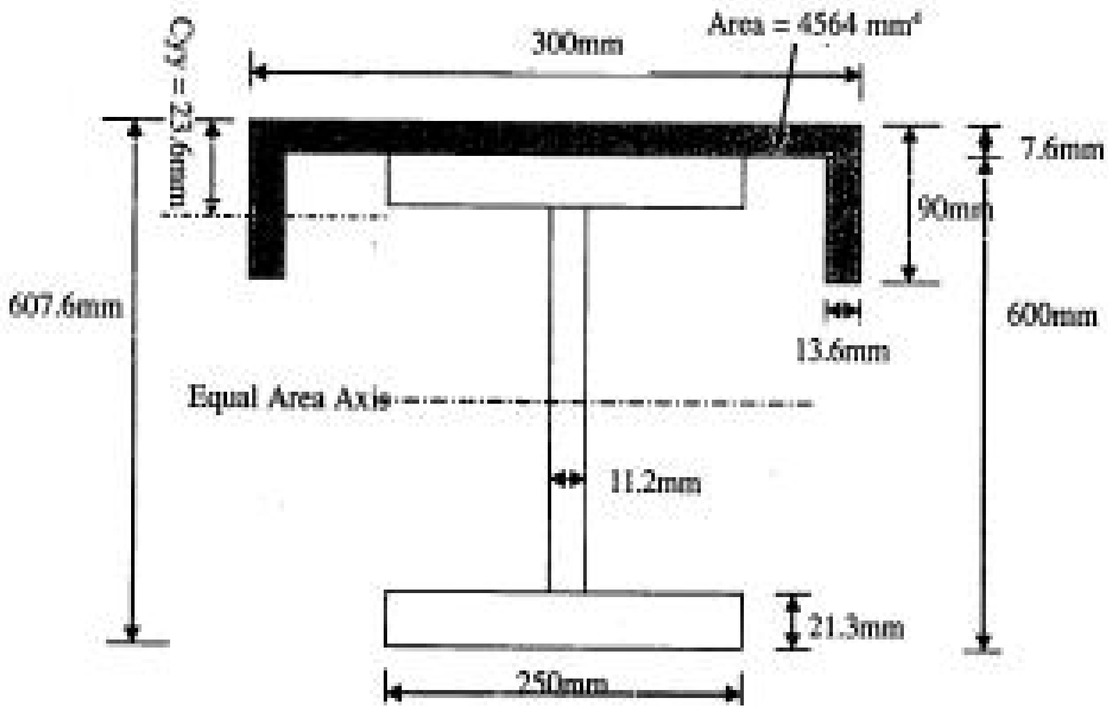
$$I_{ZZgross} = [106198.5 \times 10^4 + 17038 \times (307.6 - 247.59)^2]$$
$$+ [310.8 \times 10^4 + 4564 \times (247.59 - 23.6)^2] = 135542.8 \times 10^4 \text{ mm}^4$$

$$Z_{eZ} = \frac{I_{Z\ gross}}{Y_{max}} = \frac{135542.8 \times 10^4}{(600 + 7.6 - 247.59)} = 376.49 \times 10^4 \text{ mm}^4$$

$$I_{Y\ gross} = I_{Y\ beam} + I_{Y\ channel}$$

$$I_{Y\ gross} = 4702.5 \times 10^4 + 6362.6 \times 10^4 = 11065.10 \times 10^4 \text{ mm}^4$$

Calculating Plastic modulus of section:



$$4564 + 250 \times 21.3 + \overline{Y} \times 11.2 = 250 \times 21.3 + \left(600 - 2 \times 21.3 - \overline{Y}\right) \times 11.2$$

After solving the above equation, we get,

$\overline{Y} = 74.95$ mm from lower surface of compression flange

Plastic section modulus of section above equal area axis:

$$Z_{P\ top} = Z_{channel\ web} + Z_{channel\ flange} + Z_{I\ flange} + Z_{I\ web}$$

$$Z_{P\,top} = \left[300 \times 7.6 \times \left(74.95 + 21.3 + \frac{7.6}{2}\right)\right]$$

$$+ \left[2 \times (90 - 7.6) \times 13.6 \times (74.95 + 21.3) - \frac{(90 - 7.6)}{2}\right]$$

$$+ \left[250 \times 21.3 \times \left(74.95 + \frac{21.3}{2}\right)\right] + \left[74.95 \times 11.2 \times \frac{74.95}{2}\right]$$

$Z_{ptop} = 838.775 \times 10^3\ mm^3$

Plastic section modulus of section below equal area axis:

$Z_{P\,bottom} = Z_{1\,flange} + Z_{1\,web}$

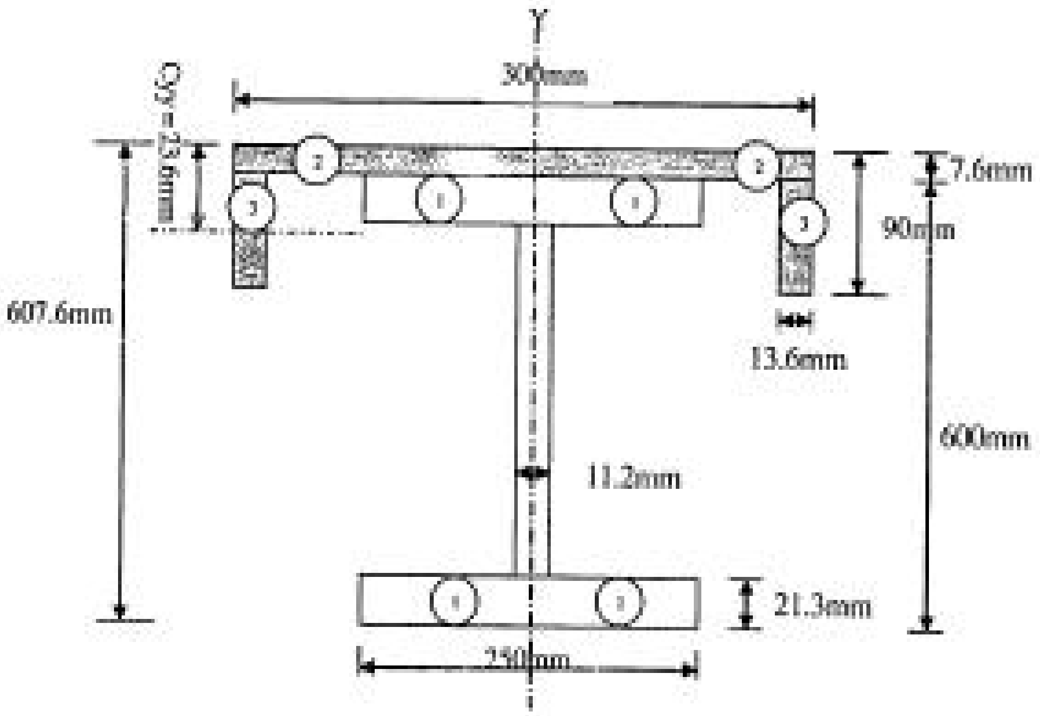
$$Z_{P\,bottom} = \left[250 \times 21.3 \times \left(600 - 21.3 - 74.95 - \frac{21.3}{2}\right)\right]$$

$$+ \left[11.2 \times \frac{(600 - (2 \times 21.3) - 74.95)^2}{2}\right]$$

$Z_{P\,bottom} = 3929.20 \times 10^3\ mm^3$

$\therefore$ **$Z_{pg} = 838.775 \times 10^3 + 3929.20 \times 10^3 = 4767.98 \times 10^3\ mm^3$**

Plastic section modulus of compression flange about Y-Y axis:



$$Z_{PY} = 2Z_1 + 2Z_2 + 2Z_3$$

$$Z_{PY} = 2\left[125 \times 21.3 \times \frac{125}{2}\right] + 2\left[150 \times 7.6 \times \frac{150}{2}\right] + 2\left[13.6 \times (90 - 7.6) \times \left(150 - \frac{13.6}{2}\right)\right]$$

$$\mathbf{Z_{PY} = 824.76 \times 10^3 mm^3}$$

Classification of Section:

$$\text{Outstand of flange of I Section, } b = \frac{b_f}{2} = \frac{250 - 11.2}{2} = 119.4 \text{ mm}$$

$$\varepsilon = \frac{250}{f_y} = \frac{250}{250} = 1.0$$

Referring to clause 3.7.2 and 3.7.4 Table 2 of IS800:2007,

$$\frac{b}{t_f} = \frac{119.4}{21.3} = 5.61 < 8.4\,\varepsilon$$

$$\frac{b}{t_w} \text{ of web of I-section} = \frac{h - 2t_f}{t_w} = \frac{600 - 2 \times 21.3}{11.2} = 49.77 < 84\,\varepsilon$$

Hence it is a plastic section.

For plastic section $\beta_b = 1.0$

Check for Moment capacity of section:

Local moment capacity:

Referring to clause 8.2.1.2 of IS800:2007, the local moment capacity for bending in vertical plane is given by,

$$M_{dz} = \frac{\beta_b Z_{PZ} f_y}{\gamma mo} \le 1.2 Z_e \frac{f_y}{\gamma mo}$$

$$M_{dz} = \frac{1.0 \times 376.49 \times 10^4 \times 250}{1.10 \times 10^6} = 1083.63 \text{ kN-m}$$

$$1.2 \times 376.49 \times 10^4 \times \frac{250}{1.10} = 1026.79 \text{ kN-m}$$

$$\therefore \quad M_{dz} = 1026.79 \text{ kN-m} > 674.35 \text{ kN-m}$$

Hence the section is safe.

Moment capacity of top flange (compression) about Y-axis:

$$Z_e = \frac{I_y}{y}$$

$$I_y = \frac{21.3 \times 250^3}{12} + 63626 \times 10^4 = 9136.04 \times 10^3 \ mm^4$$

$$y = \frac{300}{2} = 150 \ mm$$

$$Z_y = \frac{9136.04 \times 10^4}{150} = 609.07 \ \times 10^3 \ mm^3$$

$$M_{dz} = \frac{\beta_b Z_{Pyf} f_y}{\gamma mo} \leq 1.2 \ Z_{cyf} \frac{f_y}{\gamma mo}$$

$$M_{dz} = \frac{1.0 \times 824.76 \times 10^3 \times 250}{1.10 \times 10^6} = 187.45 \ kN - m$$

$$1.2 \ Z_{cyf} \cdot \frac{f_y}{\gamma mo} = 1.2 \times 609.07 \times 10^4 \times \frac{250_y}{1.10 \times 10^6} = 166.11 kN - m$$

$$\therefore \ M_{dyf} = 166.11 \ kN - m$$

Check for Combined Bending Capacity:

$$\frac{M_Z}{M_{dz}} + \frac{M_{yf}}{M_{dyf}} \leq 1.0$$

$$\frac{674.35}{1026.79} + \frac{11.81}{166.11} \leq 1.0$$

$0.73 < 1.0$

Hence OK.

Check for Buckling Resistance:

Referring to clause 8.2.2 of IS800:2007,

The design bending strength, $M_d = \beta_b Z_p F_{bd}$

Referring to clause 8.2.2.1 of IS800:2007, The elastic lateral buckling strength is given by,

$$F_{crb} = 1.1 \frac{\pi^2 E}{\left(\frac{L_{LT}}{r_y}\right)^2} \left[1 + \frac{1}{20}\left(\frac{\frac{L_{LT}}{r_y}}{\frac{h_1}{t_f}}\right)^2\right]^{0.5}$$

Where,

Overall depth of the section, $h_t = 600 + 7.6 = 607.6mm$

Effective length, $L_{LT} = 6m = 6000mm$

$t_f = 21.3 + 7.6 = 28.9mm$

$I_y = 4702.5 \times 10^4 + 6362.6 \times 10^4 = 11065.1 \times 10^4 \ mm^4$

Area, $A = 17038 + 4564 = 21602 \ mm^2$

$E = 2 \times 10^5 \ N/mm^2$

$$\text{Radius of gyration, } r_y = \sqrt{\frac{I_Y}{A}} = \sqrt{\frac{11065.1 \times 10^4}{21602}} = 71.57mm$$

$$F_{crb} = 1.1 \times \frac{\pi^2 \times 2 \times 10^5}{\left(\frac{6000}{71.57}\right)^2}\left[1 + \frac{1}{20}\left(\frac{\frac{6000}{71.57}}{\frac{607.6}{28.9}}\right)^2\right]^{0.5} = 554.55 \ N/mm^2$$

Referring to Table 13.9 of IS800:2007 for $F_{crb} = 554.55 \ N/mm^2$ and $f_y = 250 \ N/mm^2$

$$\text{From interpolation, } F_{bd} = 197.7 - \frac{197.7 - 188.8}{600 - 500} \times (600 - 554.55) = 193.56 \ N/mm^2$$

The design bending strength,

$$M_d = 1.0 \times 4767.98 \times 10^3 \times 193.56 \times \frac{1}{10^6} = 922.89 \ kN-m > 674.35 \ kN-m$$

Hence the section is safe against buckling.

Check for Biaxial Bending:

The bending strength about Y-axis will be provided by the top flange only as the lateral loads are act there only.

$$M_{dy} = \frac{f_y Z_{yt}}{\gamma mo}$$

$$Z_{yt} = \frac{I_y}{Y} = \frac{11065.1 \times 10^4}{\left(\frac{300}{2}\right)} = 737.67 \times 10^3 \ mm^3$$

$$\therefore \ M_{dy} = \frac{250 \times 737.67 \times 10^3}{1.10 \times 10^6} \ 167.65 \ kN-m$$

$$\frac{M_z}{M_{dz}} + \frac{M_y}{M_{dy}} \leq 1.0$$

$$\frac{674.35}{1026.79} + \frac{11.81}{167.65} \leq 1.0$$

$0.727 < 1.0$

Hence OK.

Check for Shear Capacity:

Maximum shear force due to wheel load, $V_z = 594.36kN$

$$\text{Shear capacity} = \frac{A_v f_{yw}}{\sqrt{3}\ \gamma mo} = \frac{(600 \times 11.2) \times 250}{\sqrt{3} \times 1.10 \times 10^3} = 881.77 \text{ kN} > 594.36 \text{ kN}$$

Maximum Shear, $V = 594.36 < 0.6\ V_d$

$< 0.6 \times 881.77$

< 529.06 kN

Since $V = 0.6\ V_d$, it is the case of high stress, hence no reduction in moment capacity.

Check for Web buckling under wheel load:

Maximum Wheel load = 388.56kN

Buckling resistance = $(b_1 + n_1)\ t_w\ F_{cd}$

Bearing length, b_1 = 150mm (Assume wheel diameter)

$$n_1 = \frac{600}{2} + 2 \times 7.6 = 315.2 \text{ mm}$$

Depth of I-section, h = 600mm

Slenderness ratio of the web, $\lambda_w = 2.42\ \frac{d_1}{t_w}$

$d_1 = 600 - 2(21.3+17) = 523.4$mm

$t_w = 11.2$ mm

$$\lambda_W = 2.42 \times \frac{523.4}{11.2} = 113.09$$

Referring to Table 9.6C of IS800:2007,

for $\lambda_W = 113.09$ and $f_y = 250N/mm^2$,

$F_{cd} = 110.80N/mm^2$

$$\therefore \text{Buckling Resistance} = F_{cd}\ A = \frac{110.80 \times [(150 + 315.2) \times 11.2]}{10^3} = 577.29kN > 388.56kN$$

Hence OK.

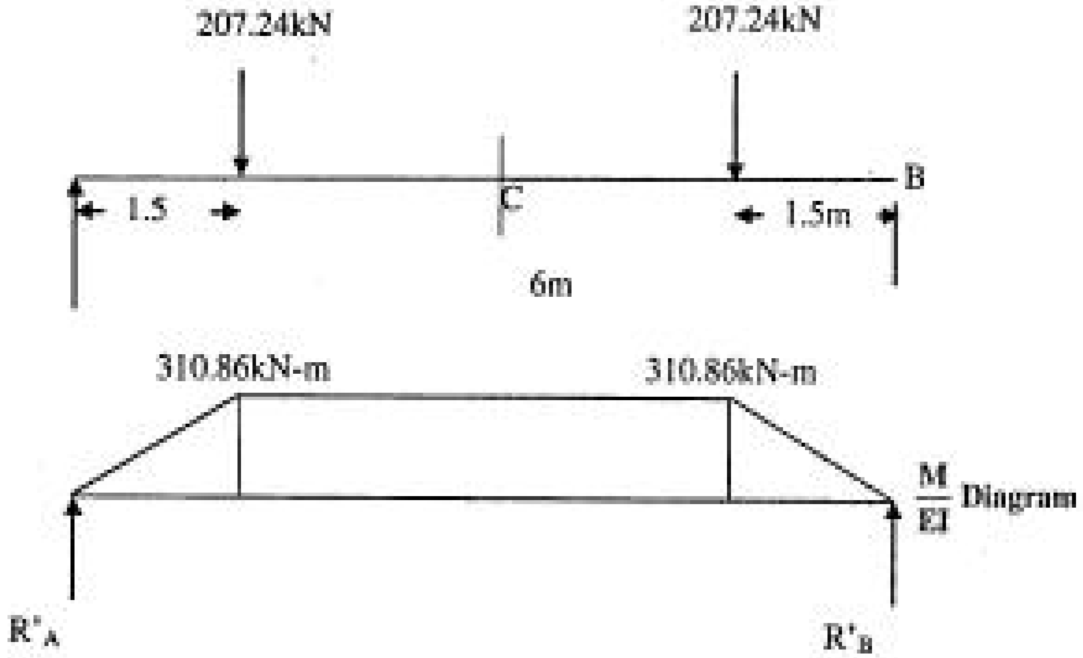
Check for Deflection:

For EOT crane up to 500kN capacity, the deflection limit is $\frac{L}{750}$.

Static Load on each wheel on gantry girder = 207.24kN

Wheel base = 3m

Maximum bending moment occurs when the wheel is at 1.5m from support i.e when they symmetrically placed.



From Conjugate beam method, due to symmetry,

$$R_A' = \frac{\text{Area of } \frac{M}{EI} \text{ diagram}}{2EI} = \frac{2\left[\frac{1}{2}\times1.5\times310.86\right]+[3\times310.86]}{2EI} = \frac{466.29+932.58}{2EI}$$

$$= \frac{699.44}{EI}$$

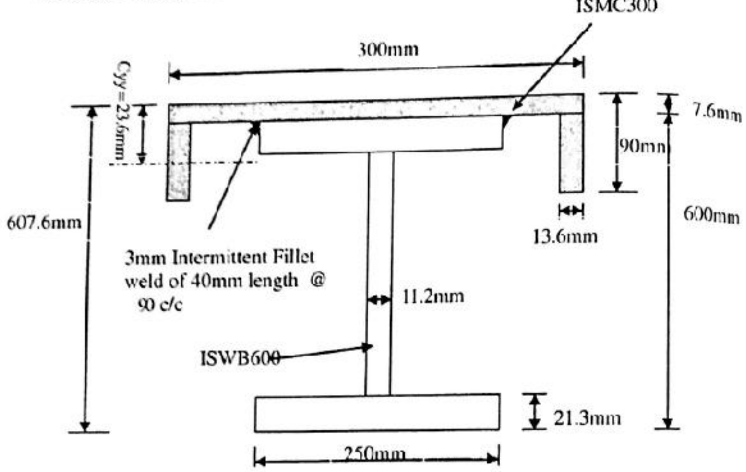
$$\therefore R_A' = \frac{699.44}{EI}$$

Maximum deflection occurs at mid span.

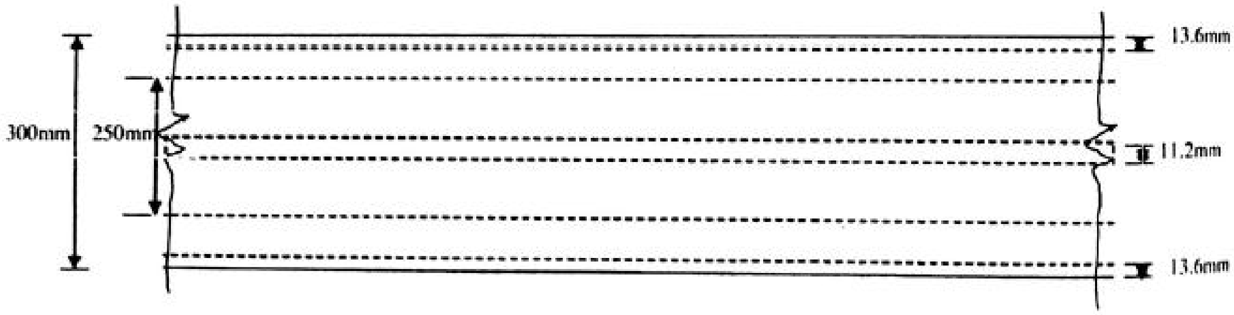
$$\Delta_c = \text{Moment of } \frac{M}{EI} \text{ diagram about C}$$

$$\Delta_C = \frac{1}{EI}\left[(699.44\times3.0)-\left(\frac{1}{2}\times310.86\times1.5\times\left(\frac{2}{3}\times1.5+1.5\right)-\left(1.5\times310.86\times\frac{1.5}{2}\right)\right]\right.$$

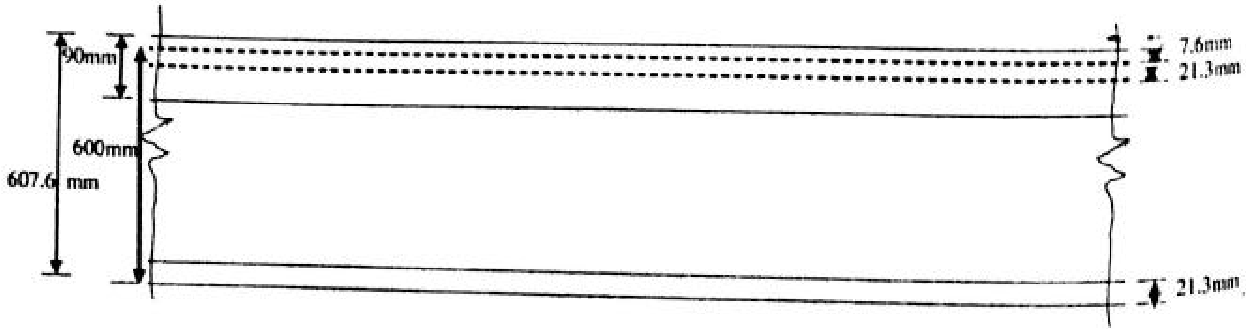
Section of Gantry Girder:



Section of Gantry Girder



Top View



Front View

Outcome

One can be able to know design criteria of gantry girders

Future study

http://nptel.ac.in/courses/105103094/

Design of plate girders

Introduction

Objective

Design of bolted plate girder without stiffener

Design of welded plate girder without stiffener

Design of welded plate girder with end stiffeners

Design of welded plate girders with intermediate stiffeners

Outcome

Future Study

notes4free
All in one

Introduction

In a plate girder bridge, the plate girders are typically I-beams made up from separate structural steel plates (rather than rolled as a single cross-section), which are welded or, in older bridges, bolted or riveted together to form the vertical web and horizontal flanges of the beam.

3.1.2 Objective

- To study the design criteria of plate girder with bolted, welded- with and without stiffeners

Plate girders

Plate girders became popular in the late 1800's, when they were used in construction of railroad bridges. The plates were joined together using angles and rivets to obtain plate girders of desired size. By 1950's welded plate girders replaced riveted and bolted plate girders in developed world due to their better quality, aesthetics and economy. Fig.7.15 shows the cross sections of two common types of plate girder bridges. The use of plate girders rather than rolled beam sections for the two main girders gives the designer freedom to select the most economical girder for the structure. If large embankment fills are required in the approaches to the bridge, in order to comply with the minimum head-room clearance required, the half through bridge is more appropriate [Fig.7.15 (a)]. This arrangement is commonly used in railway bridges where the maximum permissible approach gradient for the track is low. In this case the restraint to lateral buckling of compression flange is achieved by a moment resisting U-frame consisting of floor beam and vertical stiffness, which are connected together with a moment resisting joint. If the construction depth is not critical, then a deck-type bridge, as shown in Fig.7.15 (b) is a better solution, in which case the bracings provide restraint to compression flange against lateral buckling.

Design procedure for bolted plate girder

1. Computing the Factored load W in kN
   $W = 1.5\ DL + 1.50\ LL$
   Where DL = Dead Load
   LL = Live Load
2. Computing the Self- Weight w in kN/m
   $w = W_f L / 200$ to $W_f L / 300$
   where W_f = Factored load W + Factored Self-weight
3. Compute factored bending moment and Shear Force
   $M_d = W_f L^2 / 8$ kN-m
   $V_d = W_f L / 2$ kN
   Compute and Add BM due to Impact if required:
   $M = 0.372\ W L^2 / 8$ kN-m
4. Finding the economical depth of plate girder

$$d = \sqrt[3]{\frac{M_d k}{f_y}}$$

5. Finding the optimum value of thickness of web

   $t_w = (M/ k^2 f_y)^{1/3}$

   where $k = d_w / t_w$

   $f_y = 250$ N/mm^2

   For unstiffened web: $(d_w / t_w) = 67$

   To avoid buckling of the compression flange into web:

   $(d / t_w) \leq 345\, \varepsilon^2{}_f$

   When transverse stiffeners are not provided:

   $(d / t_w) \leq 200\, \varepsilon$

   ε = yield stress ratio of web = $\sqrt{250/f_y}$

   For Stiffened web with End Stiffeners only:

   $(d / t_w) = 100\, \varepsilon$ to $110\, \varepsilon$

   For Stiffened web with End Stiffeners and Intermediate Stiffeners:

   $(d / t_w) > 200\, \varepsilon$

6. Design of Flanges:

   Width of flange $b_f = 0.3\, d$

   Thickness of flange:

   For Plastic section: $t_f \leq 8.4\, b_f$

   For Compact section: $t_f \leq 9.4\, b_f$

   For Semi-Compact section: $t_f \leq 13.6\, b_f$

7. Check the moment capacity of section: $M_d > M$
8. Check for Shear strength of the section
9. Provide End and Intermediate stiffeners if required in case of thin webs.
10. Design the connections.

Outcome

- One can know the design criteria of plate girder with bolted, welded- with and without stiffeners

Future study

http://nptel.ac.in/courses/105103094/