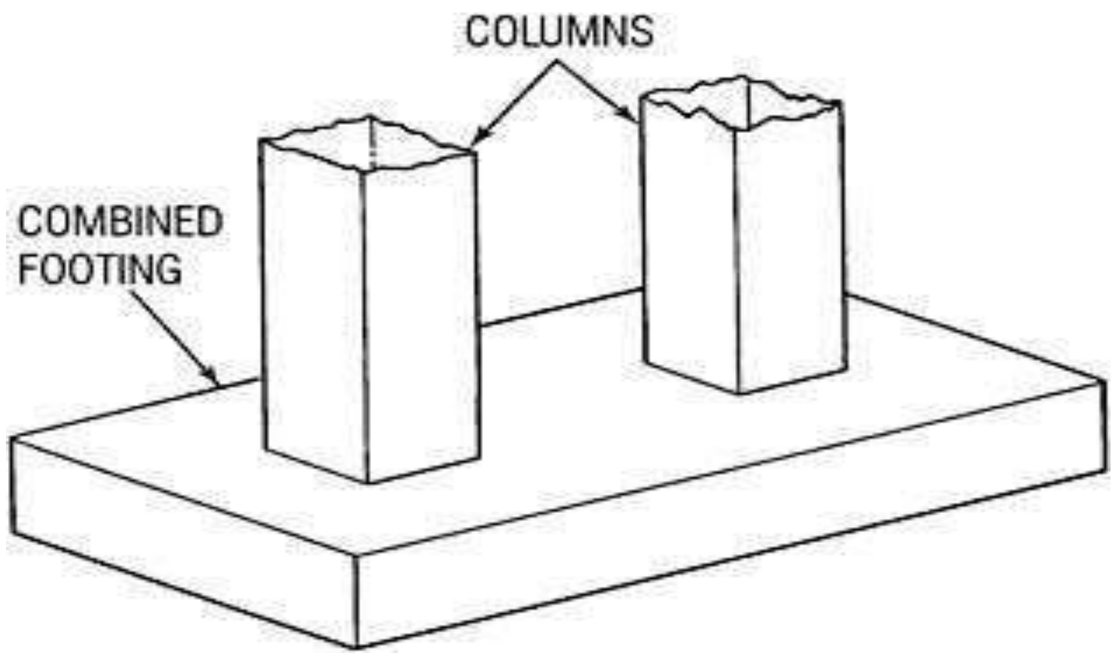
PART A

DESIGN OF RCC STRUCTURES

Chapter 1

DESIGN OF COMBINED FOOTINGS



Combined footings are constructed for two or more columns when they are close to each other and their foundations overlap.

Combined footings are provided only when

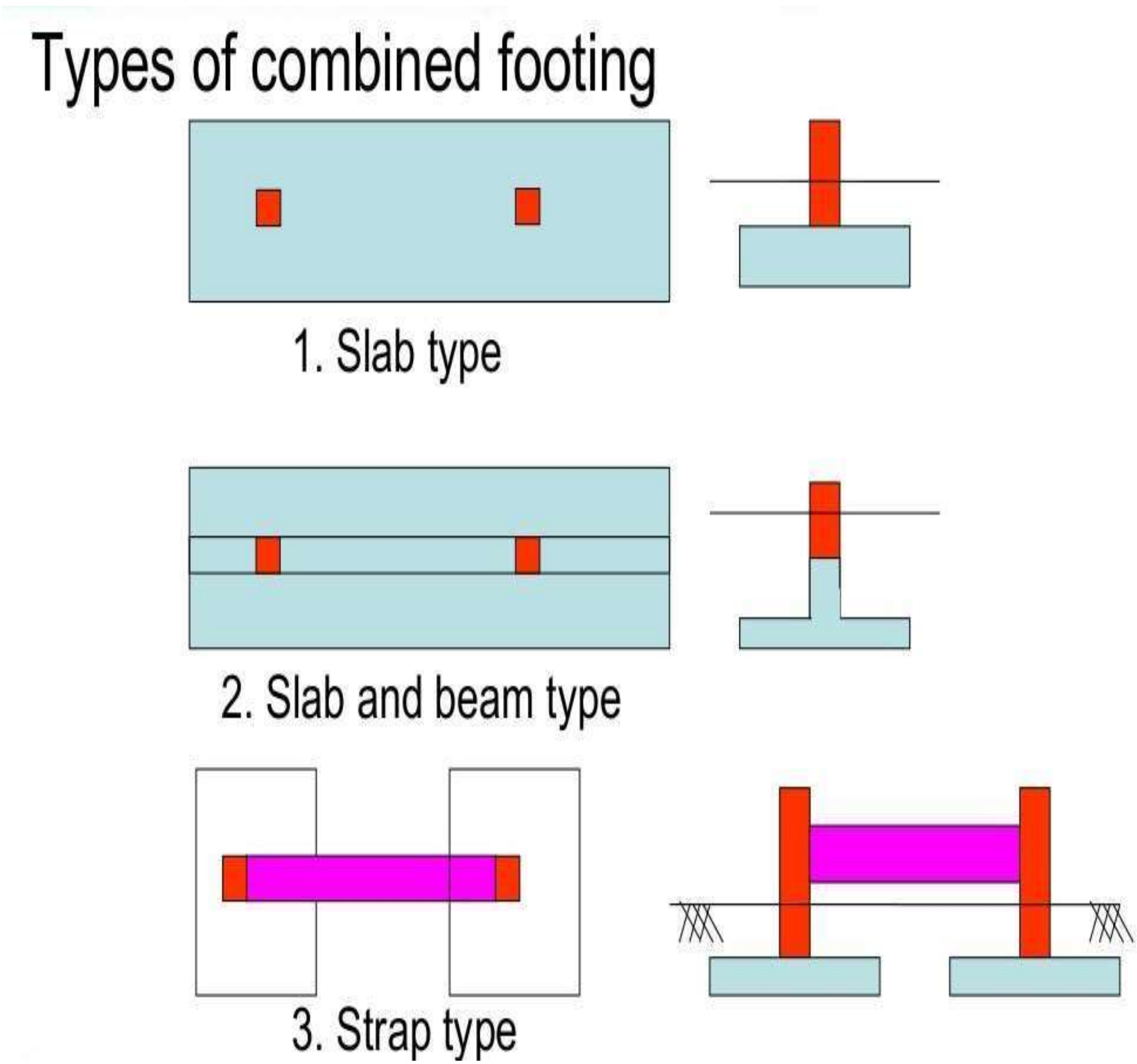
1. When two columns are close together, causing overlap of adjacent isolated footings
2. Where soil bearing capacity is low causing overlap of adjacent isolated footings.
3. Proximity of building line or existing building or sewer, adjacent to building column.

- The main purpose of using **combined footing** is to distribute uniform pressure under the **footing**.

- To achieve this , the center of gravity of the **footing** area should coincide with the center of gravity of the two columns loads.

Following are the different types of combined footing

1. Slab type combined footing
2. Slab and Beam type combined footing
3. Strap type Combined footing.



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1. Design a combined footing for two RCC columns A and B separated by distance of 4 m c/c, Column A is 500 x 500 and carries a load of 1250 KN and Column B is 600 x 600 and carries a load of 1600 KN. Take SBC of soil as 200 KN/m^2 . Use M20 concrete and Fe415 Steel. Draw the rough sketches of the following
   a. Sectional Elevation
   b. Plan of bottom reinforcements
   c. Plan of top reinforcements
   d. Cross Section of two different places to show the maximum details of shear reinforcements.

Data Given :

Size of Column A = 500 x 500 mm

Load on Column A W_1 = 1250 KN

Size of Column B = 600 x 600 mm

Loan on Column B W_2 = 1600 KN

SBC of Soil = 200 KN/m^2

fck = 20 KN/m^2

fy = 415KN/m^2

Soln:

1. Size of the Footing:

Total Column Load = 1250 + 1600 = 2850 KN

Self Wt. of Footing = 10 % of Column Load = 285 KN

Total load = 3135 KN

Area of footing $L \times B = \frac{Total\ Load}{SBC\ of\ Soil} = \frac{3135}{200}$

$A_f = L \times B = 15.675\ m^2$

Assume the Width of the footing between 1.5 m to 2.5 m

Take B = 2.5 m

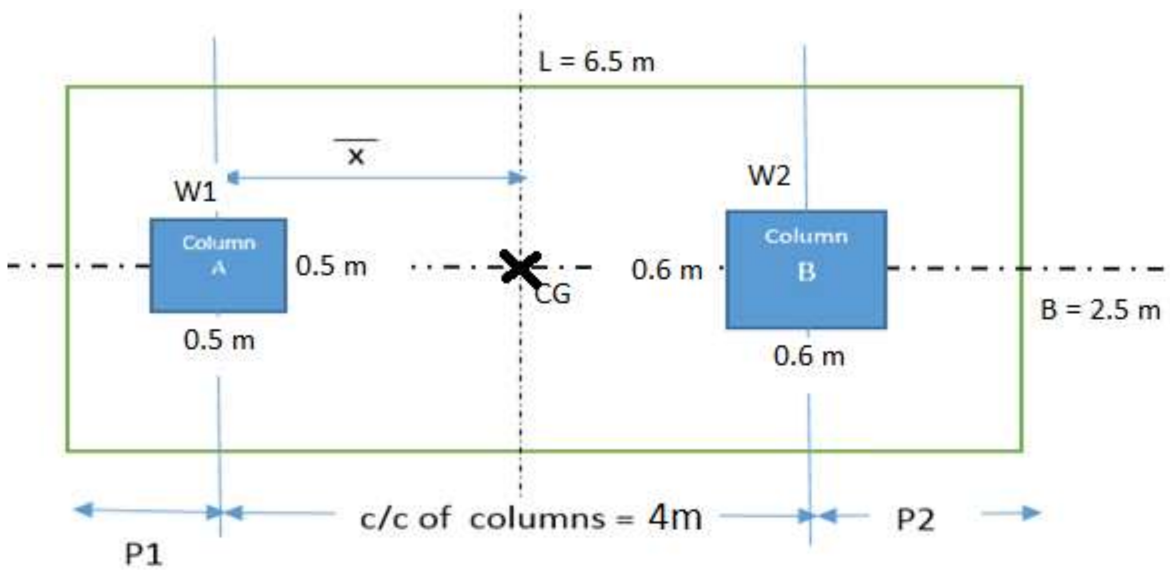
L x B = 15.675

$L = \frac{15.675}{2.5} = 6.27$ m say 6.5 m

$\boxed{\therefore \text{ Provide } L \times B = 6.5\ m \times 2.5\ m}$

2. Projections p_1 & p_2:

Projections should be such that the center of gravity of column loads should coincide with the center of the footing.



$\therefore$ CG of footing from the center of the Column A

$$\bar{x} \quad = \frac{(W1*x1\,) + (w2*x2)}{(W1+W2)}$$

$$= \frac{(1250*0\,) + (1600*4)}{(1250+1600)}$$

$\bar{x} = 2.24$ m

From the above diagram ,we can write

$p_1 + 2.24 = L/2$

$p_1 + 2.4 = 6.5/2$

$p_1 = 1$ m

Also $p_1 + 4 + p_2 = L$

$1 + 4 + p_2 = 6.5$

$p_2 = 1.5$ M

$\therefore$ Projections $p_1 = 1$ m and $p_2 = 1.5$ m

3. Shear force and Bending Moment Diagram: (SFD & BMD):

Net Upward Pressure / m^2 $\quad q = \dfrac{Only\ Column\ load}{area\ of\ footing}$

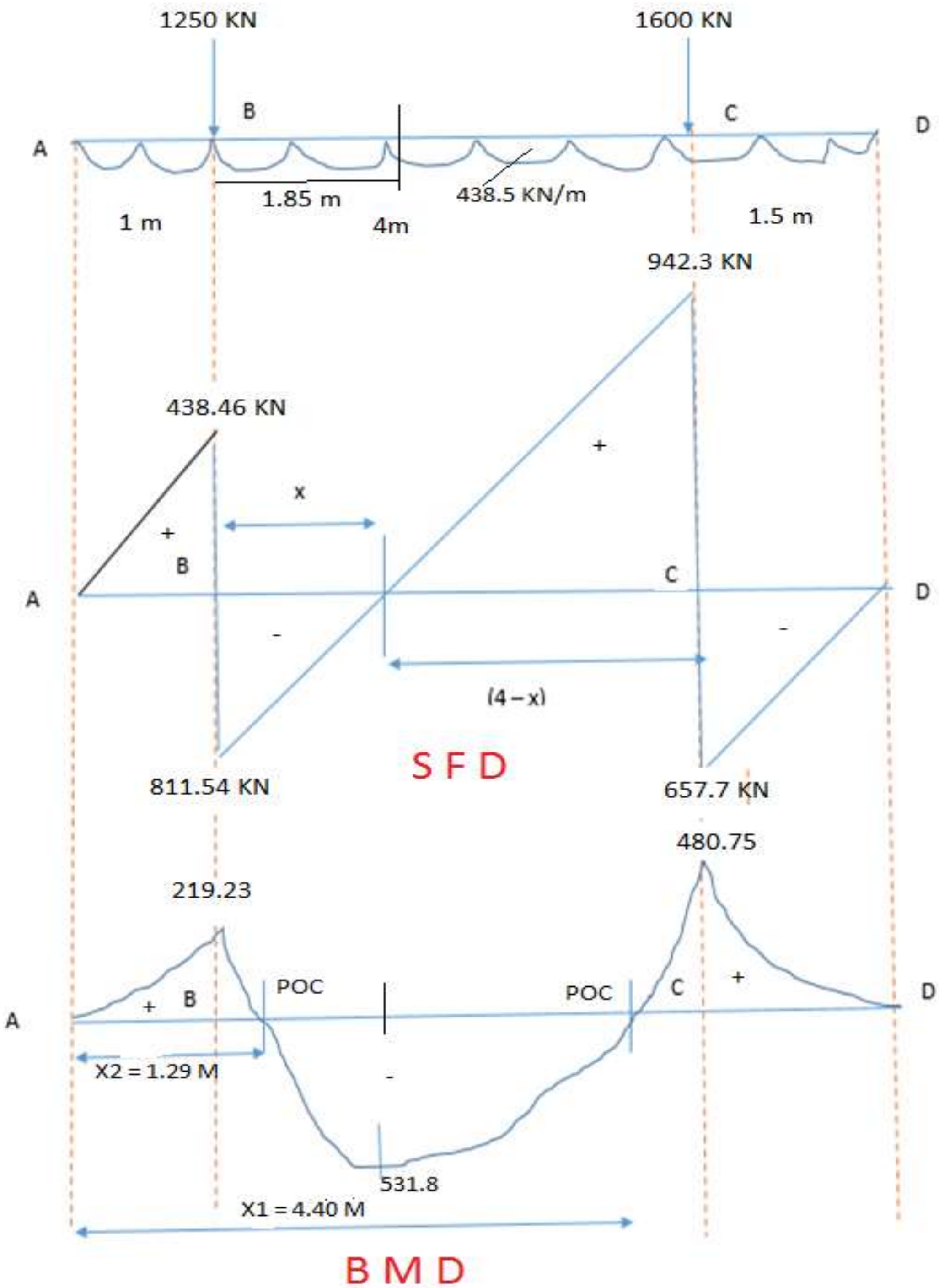
$$= \frac{2850}{6.5*2.5}$$

$q = 175.4\ KN/m^2$

Net Upward Pressure/ m $\quad q_o = q \times B$

= 175.4 * 2.5

= 438.5KN/m



Shear Force Calculation:

SF at A = 0

SF up to B = + 438.5 * 1 = 438.46 KN

SF at B = + 438.5 – 1250 = - 811.54 KN

SF up to C = - 438.5 *1.5 = - 657.7 KN

Sf at C = - 657.7 + 1600 = + 942.3 KN

SF at D = 0

Bending Moment Calculation:

BM at A = 0

BM at B = + 438.46 * 1 *1/2 = + 219.23 KN-m

BM at O = + 438.46*2.85*2.85/2 -1250*1.85 = - 531.80 KN-m

BM at C = + 438.46*5*5/2 – 1250*4 = + 480.75 KN-m

BM at D = 0

Location of Zero Shear Force:

The point where SF=0, the BM is maximum

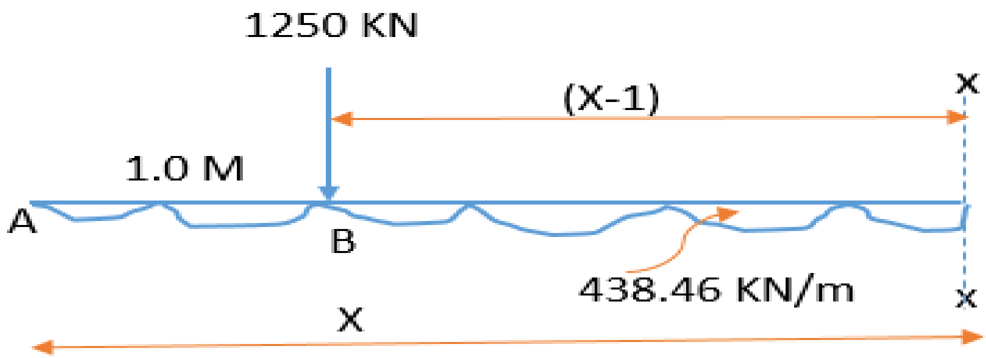
From Shear force diagram, from two similar triangles, we can write

$$\frac{811.54}{x} = \frac{942.3}{(4-x)}$$

$$\therefore x = 1.85 \text{ m}$$

i.e Shear force is zero at a distance of 1.85 m from B.

Location of POC's:



POC is the point where BM changes its sign

Therefore equating BM at x-x = 0

438.46 * x *x /2 – 800 (x-1) = 0

219.23 *x^2 -1250* x + 1250 = 0

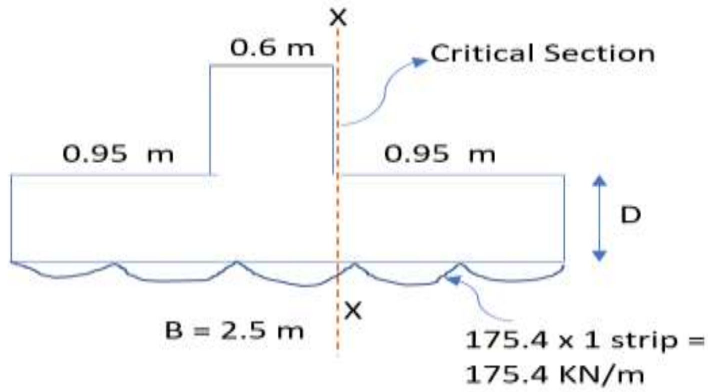
Solving Quadratic equation,

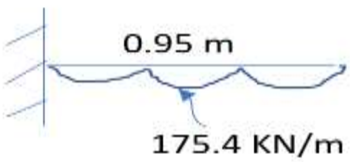
$\boxed{x_1 = 4.10\text{ m} \quad \& \quad x_2 = 1.5\text{ m}}$

4. Design of Slab:

Provide width of Beam is equal to size of bigger column

∴ Beam width = 600 mm





Taking moment about Critical Section x - x

$\therefore$ M = 175.4*0.95*$\frac{0.95}{2}$ = 79.15 KN-m

Mu = 1.5 *M = 1.5 *79.15 = 118.72 KN-m

Thickness or Depth of Slab:

Equating M_u to M_{ulimit}

M_u = $0.36\,\frac{x_{u,max}}{d}\left[1-0.42\,\frac{x_{u,max}}{d}\right] f_{ck}\,b\,d^2$

118.72 * 10^6 = $0.36 * 0.48[1-0.42 * 0.48] * 20 * 1000 * \;d^2$

d = 207.4 mm

Using 60 mm effective cover

Overall depth D = 207.4 + 60 = 267 mm say 270 mm

But from shear consideration, double the above thickness

$\therefore$ D = 540 mm and d = 480 mm

Area of Steel:

$$Mu = 0.87\, fy\, Ast\, d[1 - \frac{Ast\, fy}{fck\, b\, d}]$$

$$118.72 \times 10^6 = 0.87 * 415 * Ast * 480[1 - \frac{Ast*415}{20*1000*480}]$$

$$Ast = 706.62 \ mm^2$$

Providing 12 mm dia bars, Spacing is taken least of the following

i. $\text{Spacing} = \frac{ast}{Ast} * 1000 = \frac{\frac{\pi * 12^2}{4}}{706.62} * 1000 = 160 \text{ mm}$

ii. $\text{Spacing} = 3d = 3 * 480 = 1440 \text{ mm}$

iii. $\text{Spacing} = 300 \text{ mm}$

$\therefore$ Provide 12mm dia bars @ 160mm c/c

Distribution Steel:

$$Ast = 0.12 \% \text{ of Gross Area} = \frac{0.12}{100} * 1000 * 540 = 648 \ mm^2$$

Providing 10 mm dia bar, Spacing is taken as least of the following

i. $\text{Spacing} = \frac{ast}{Ast} * 1000 = \frac{\frac{\pi * 10^2}{4}}{660} * 1000 = 121\, mm \approx 120\, mm$

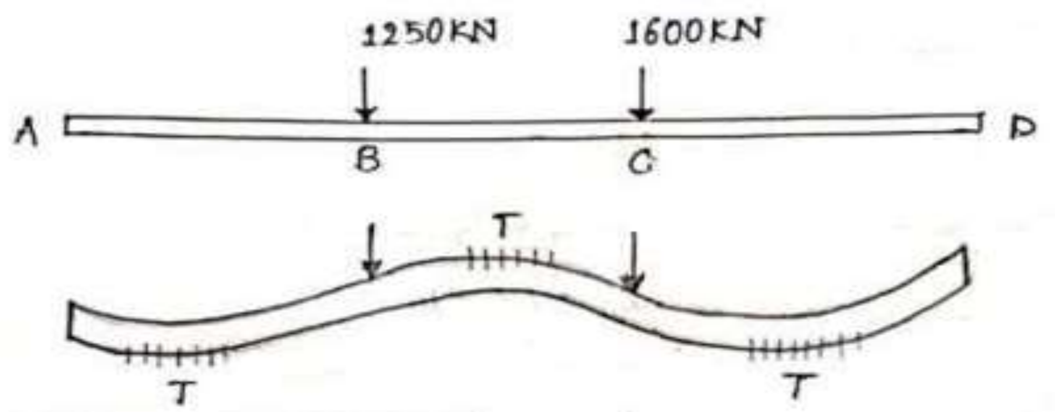
ii. $\text{Spacing} = 5d = 5 * 480 = 2400 \text{ mm}$

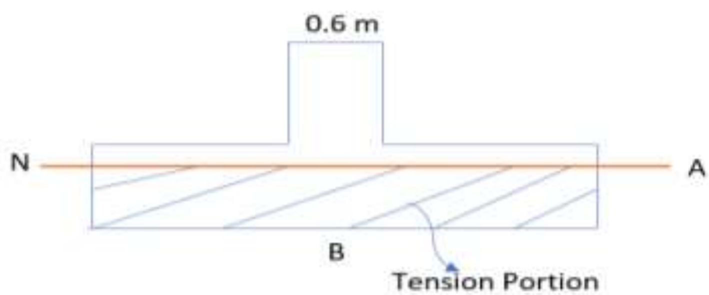
iii. $\text{Spacing} = 450 \text{ mm}$

$\therefore$ Provide 10mm dia bars @ 110 mm c/c

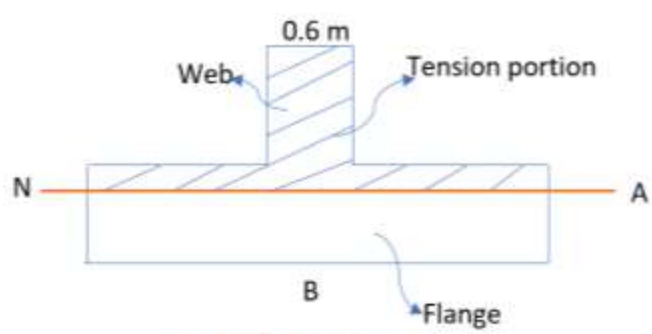
5. Design of Beams:

Beam Width b = 600 mm





In 'AB' & 'CD' Portion



In 'BC' Portion

AB and CD portion are designed as Rectangular Beam and BC portion is designed as T – Beam.

Concrete is very weak in tension, hence neglect the concrete portion in tension zone.

M_{max} = 531.8 KN-m (From BM Diagram)

Mu = 1.5 * 531.8 = 797.7 KN-m

Equating M_u to M_{ulimit}

$M_u = 0.36 \frac{x_{u,max}}{d}\left[1 - 0.42\frac{x_{u,max}}{d}\right] f_{ck}\, b\, d^2$

$797.7 * 10^6 = 0.36 * 0.48[1 - 0.42 * 0.48] * 20 * 600 * d^2$

d = 694.14 say 700 mm

∴ Beam dimensions b = 600 mm, d = 700 mm and D = 760mm

(i) Design of "AB" Portion:

M_{AB} = 219.23 KN-m. (From BM diagaram)

Mu = 1.5 * 219.23 = 328.84 KN-m

In AB portion, tension is in the flange, hence neglecting tension zone, Take b = 600mm

Area of Steel:

$\text{Mu} = 0.87\ \text{fy Ast d}[1 - \frac{\text{Ast fy}}{\text{fck b d}}]$

$328.8 \times 10^6 = 0.87 * 415 * \text{Ast} * 700[1 - \frac{\text{Ast}*415}{20*600*700}]$

Ast = 1397.6 mm^2

Providing 20 mm dia bars,

No. of bars = $\frac{Ast}{ast}$ = $\frac{1397.6}{\frac{3.14*20^2}{4}}$ = 5 bars.

(ii) Design of "CD" Portion:

M_{CD} = 480.75 KN-m. (From BM diagram)

Mu = 1.5 * 480.75 = 721.12 KN-m

Even in CD portion, tension is in the flange, hence neglecting tension zone, Take b = 600mm

Area of Steel:

Mu = $0.87\ \mathrm{fy\ Ast\ d}[1-\frac{\mathrm{Ast\ fy}}{\mathrm{fck\ b\ d}}]$

$721.12 \times 10^6 = 0.87*415*\ \mathrm{Ast}*\ 700[1-\frac{\mathrm{Ast}*415}{20*600*700}]$

Ast = 3437 mm^2

Providing 25mm dia bars,

No. of bars = $\frac{Ast}{ast}$ = $\frac{1397.6}{\frac{3.14*25^2}{4}}$ = 7 bars.

(iii) Design of "BC" Portion:

M_{BC} = 531.8 KN-m. (from BM diagram)

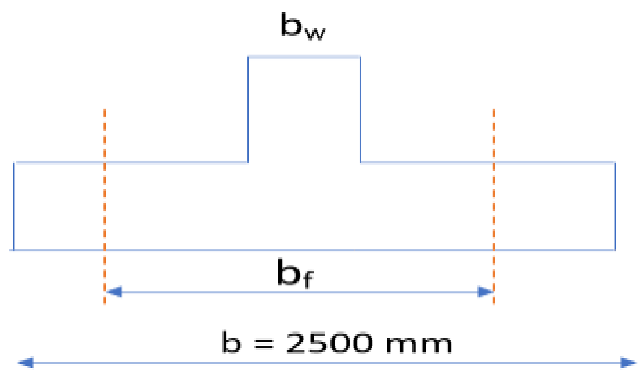
Mu = 1.5 * 531.8 = 797.7 KN-m

In this portion, tension is in WEB, hence neglecting, web portion and considering flange

∴ b = b_f = effective flange width

Hence beam is designed like a **T- Beam**

For Isolated T-Beam



Effective flange width $b_f = \frac{l_o}{(\frac{l_o}{b})+4} + b_w$ ----Page 37, IS 456

b = Actual flange Width = 2500

l_o = Distance between points of zero moments

From BMD l_o = Distance between POC

= $(x_1 - x_2)$ = 4.4 - 1.29 = 3.11 m = 3110 mm

Therefore, $b_f = \frac{3110}{(\frac{3110}{2500})+4} + 600$ = 1193 mm

Area of Steel:

$$\text{Mu} = 0.87 \text{ fy Ast d}[1 - \frac{\text{Ast fy}}{\text{fck } b_f \text{ d}}]$$

$$797.7 \times 10^6 = 0.87 * 415 * \text{ Ast} * 700[1 - \frac{\text{Ast}*415}{20*1193*700}]$$

Ast = 3453.5 mm^2

Providing 25mm dia bars,

No. of bars = $\frac{Ast}{ast}$ = $\frac{3453.5}{\frac{3.14*25^2}{4}}$ = 7 bars.

Design of Beam for Shear:

Maximum shear force V_{max} = 942.3 KN (From SFD)

Ultimate shear force = 1.5 * 942.3 = 1413.4 KN

b = 600mm, d = 700 mm, Ast = 3453.5 mm^2

Nominal Shear force $\tau_v = \frac{Vu}{B\ d} = \frac{1413.4*10^3}{600*700}$ = 3.36 N/mm^2

Shear Stress in Concrete:(τ_c)

$P_t = \frac{100\ Ast}{b\ d} = \frac{100*3453.5}{600*700}$ = 0.82

Referring to IS 456, Pg 73, for Pt = 0.82 and M_{20} Concrete

$\therefore$ τ_c = 0.58 N/mm^2

Comparing τ_v and τ_c,

$\tau_v > \tau_c$, $\therefore$ Provide Shear Reinforcement

Vertical Stirrups:

Using 4L - #10 mm Vertical stirrups

A_{sv} = $4 * \frac{\pi * 10^2}{4} = 314.16$ mm^2

Shear force to be carried by vertical stirrups

V_{us} = $(Vu - \tau_c * b*D) = (1413.4 * 10^3 - 0.58*600*700)$

= 1169.83 kN

From IS 456, Page 73

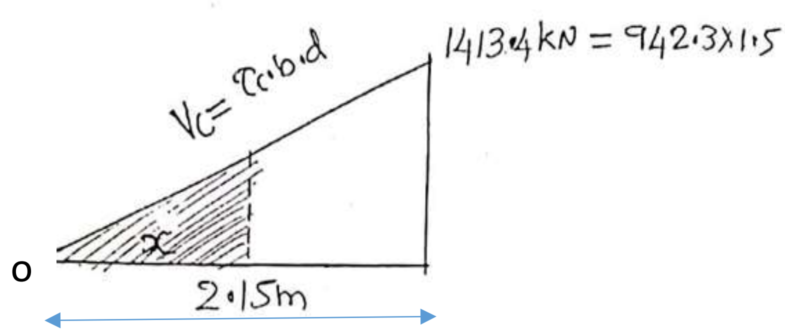
$$V_{us} = \frac{0.87\ fy\ Asv\ d}{Sv}$$

∴ Spacing of Vertical Stirrups from above equation

$$S_v = \frac{0.87\ fy\ Asv\ d}{Vus}$$

$$Sv = \frac{0.87*415*314.15*700}{1169.83*10^3}$$ = 67.87 mm say 65 mm

Provide 4L – #10 mm Vertical Stirrups @ 65 mm c/c below the Column and @ 300 mm c/c in other places



Shear strength of Concrete at distance x from apex (from o) of above triangle

$V_c = \tau_c *b*d = 0.58 *600*700$

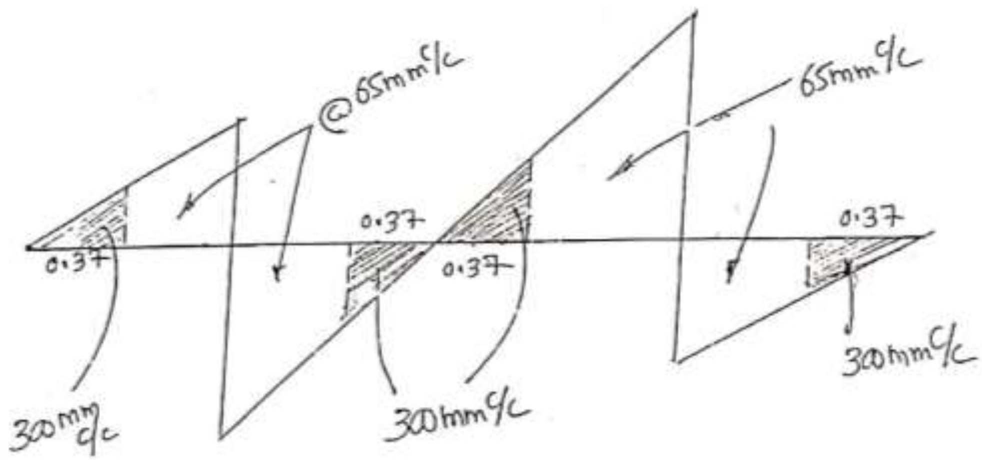
= 243.6 KN

Distance x from above similar triangles

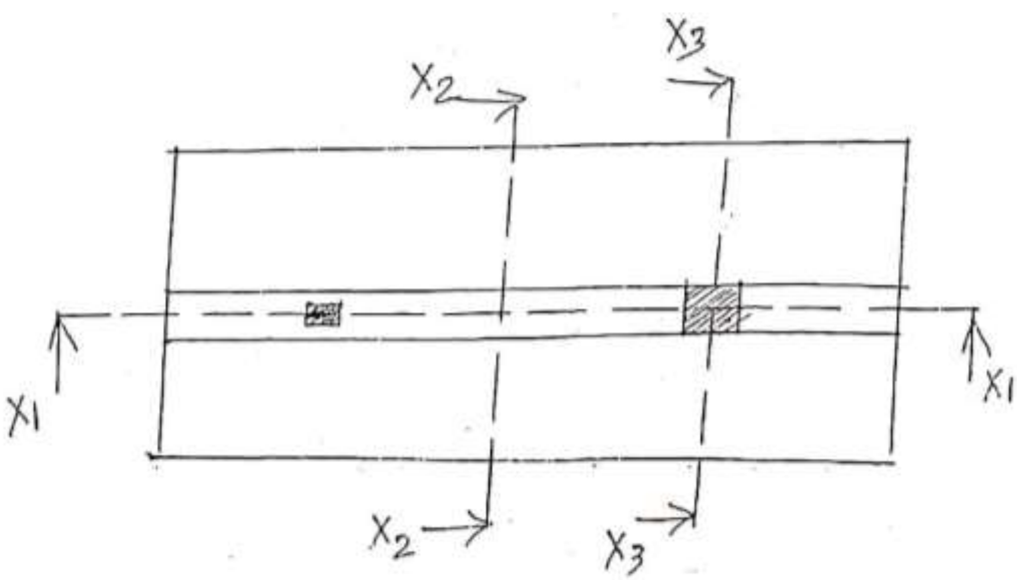
$$\therefore \quad \frac{1413.4}{2.15} = \frac{243.6}{x}$$

$\therefore \quad x = 0.37$ m.

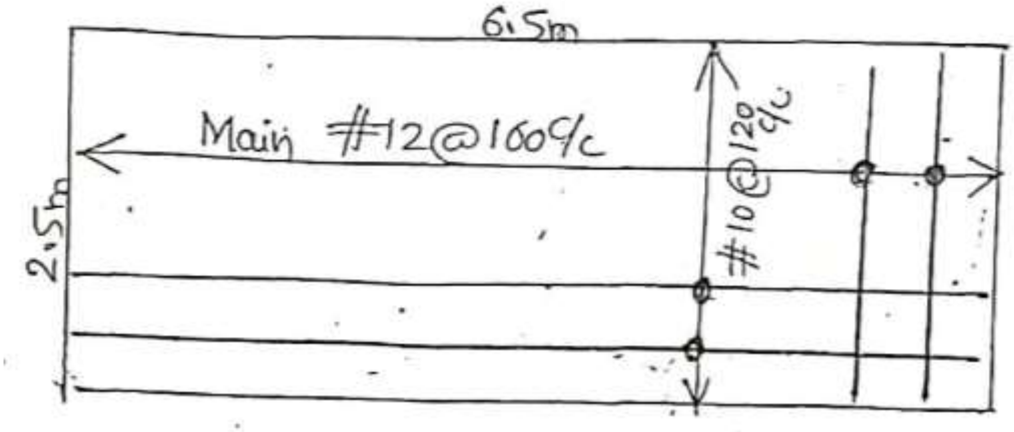
Spacing of Stirrups:



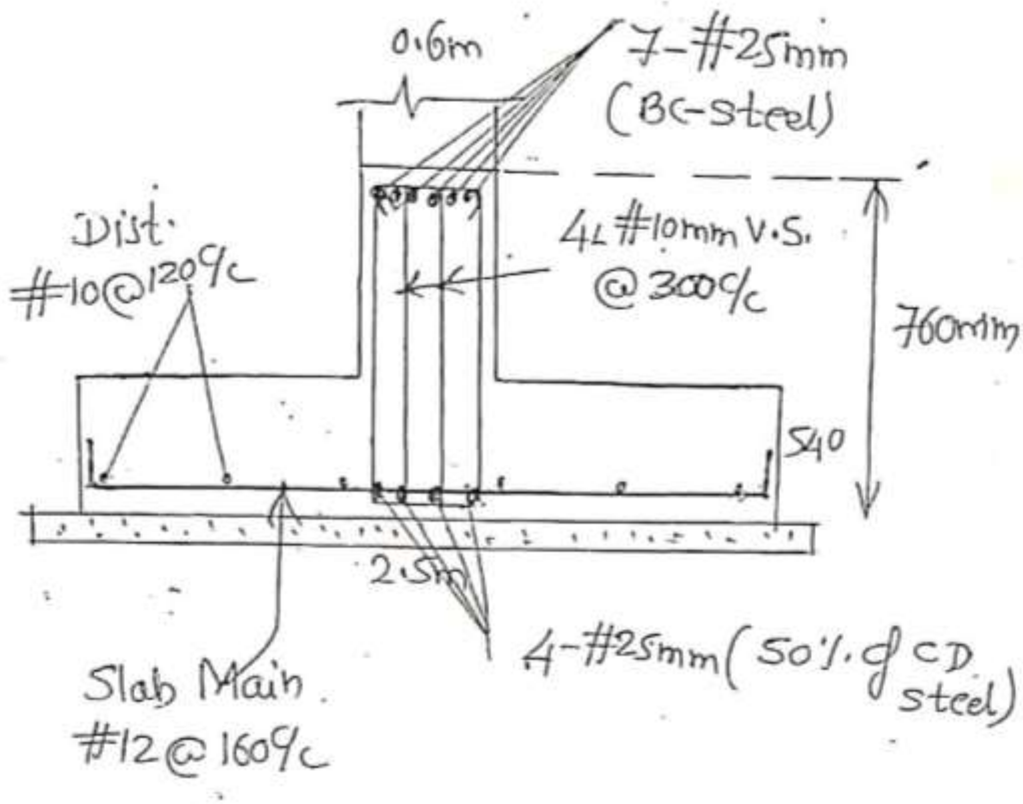
Vertical Stirrups @ 65 mm c/c below the Column and @ 300 mm c/c in other places   (0.37m from apex of the triangles)



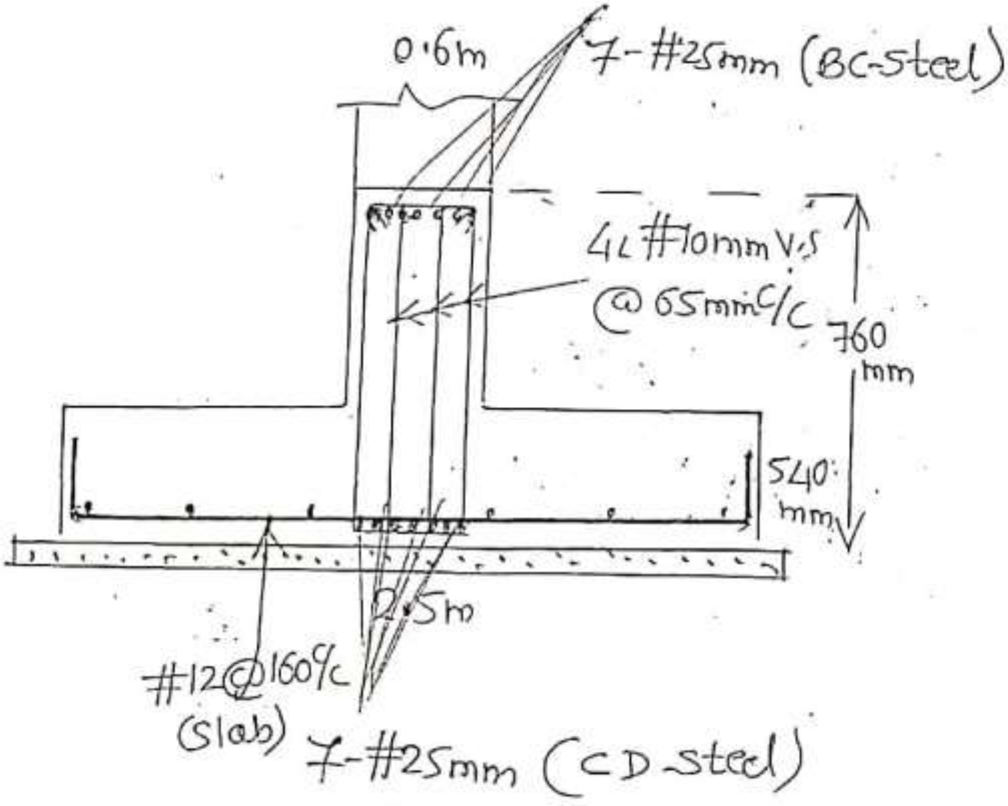
Plan showing Slab, Beam and Column



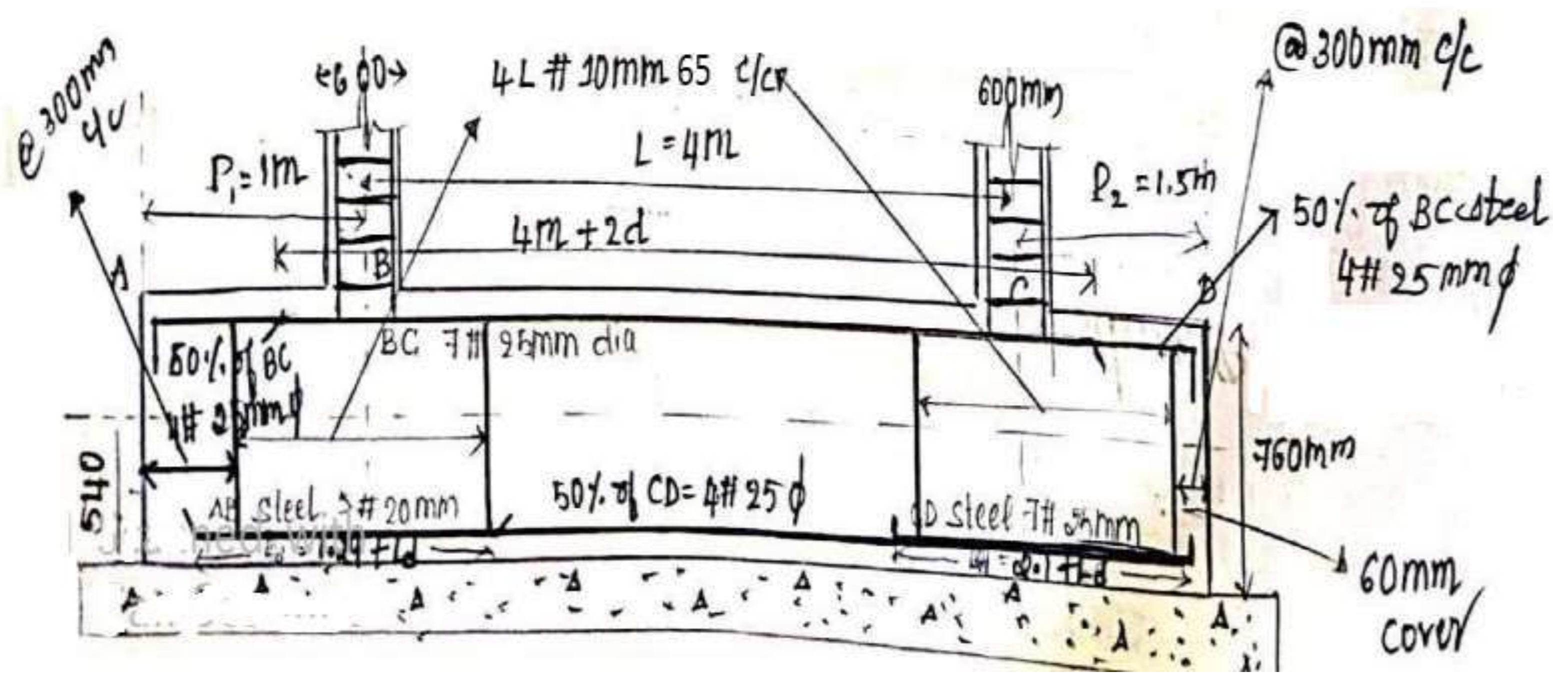
Plan showing reinforcement in slab



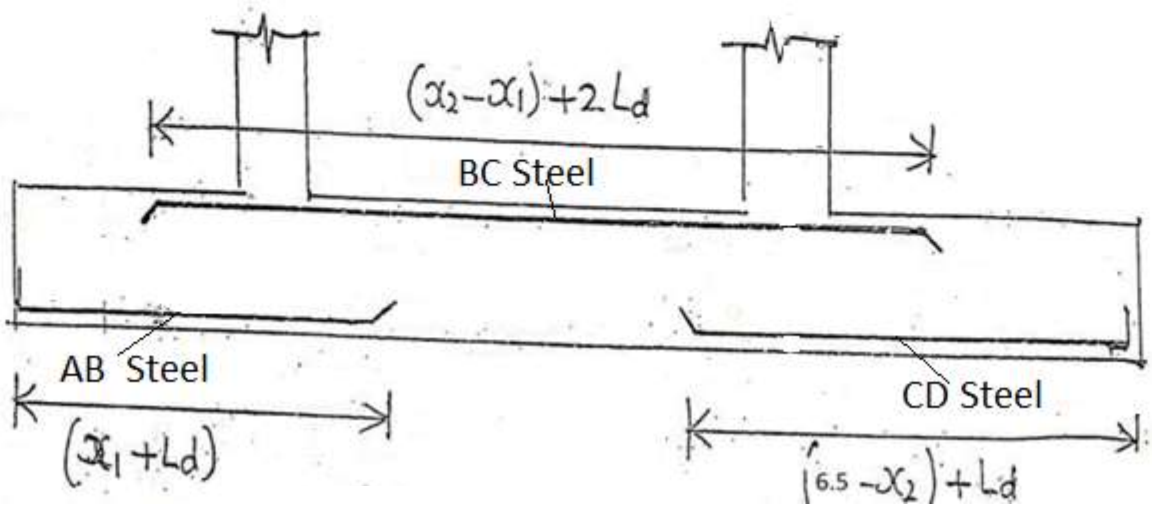
Cross Section at Mid span (X2 – X2)



Cross Section through the Column (X3 – X3)



Longitudinal Cross Section (X1 –X2)



L_d = 47Φ for M25

Longitudinal Section showing reinforcement in beam

DESIGN OF WATER TANKS

(Working stress method using IS 3370 – Code of Practice for Concrete structure for the storage of liquids)

Following are the different types of water tanks to be designed

A. Design of circular water tank with rigid base.
B. Design of circular water tank with flexible base.
C. Design of rectangular water tank.

Working Stress Method:

a. Permissible Stresses in Concrete:

σ_{cbc} = Permissible Bending Compressive Stress in concrete.

σ_{ct} = Permissible Tensile Stress in Concrete.

Grade of Concrete	σ_{cbc}	σ_{ct}
M_{15}	5 N/mm^2	1.10 N/mm^2
M_{20}	7 N/mm^2	1.20 N/mm^2
M_{25}	8.5 N/mm^2	1.30 N/mm^2

b. Permissible Tensile Stresses in Steel:

σ_{st} = Permissible Tensile stress in Steel.

Grade of Steel	Near Water face	Away from Water face
Mild steel or Fe 250	115 N/mm^2	125 N/mm^2
HYSD or Fe 415 or Fe 500	150 N/mm^2	190 N/mm^2

c. Working Stress Constants:

i. Modular ratio m = $\frac{280}{3\sigma_{cbc}}$

ii. Neutral axis c0efficient k = $\frac{m\,\sigma_{cbc}}{m\,\sigma_{cbc}+\sigma_{st}}$

iii. Lever arm constant j = 1 - $\frac{k}{3}$

iv. Moment of Resistance coefficient Q = $\frac{\sigma_{cbc}\,k\,j}{2}$

vi. Effective depth d = $\sqrt{\frac{M}{Q\ x\ b}}$

vii. Area of Steel

If the moment is known A_{st} = $\frac{M}{\sigma_{cbc}\,j\,d}$

If force is known A_{st} = $\frac{Force}{\sigma_{st}}$

d. Specifications for the design of Water Tank:

i. Adopt clear cover = 30 mm

ii. Minimum reinforcement

Up to 100mm thick wall = 0.3% of Gross Area

Between 100 & 450 mm thick wall = 0.2% of Gross Area

iii. Thickness of wall (T)

1. T = 30 H + 50 mm, Where H = Depth of water in m

2. σ_{cbc} = $\frac{Maximum\ Hoop\ Tension}{1000\ T+(m-1)Ast.}$

iv. w = Unit weight of water = 1000 N/m^3

= 1 KN/m^3

Also 1000 lts = 1 m^3

$\therefore$ 1 lt = 1 * 10^{-3} m^3

Design of circular water tank with flexible Base.

1. Design a circular water tank with flexible base for a capacity of 4×10^5 lts. The depth of water tank is to be 4m with a free board of 200 mm. Use M25 and Fe 415 steel.

   Draw the following sketches

   a. Cross section of the tank
   b. Half plan through the wall
   c. Half plan through the base slab.

   Data given:

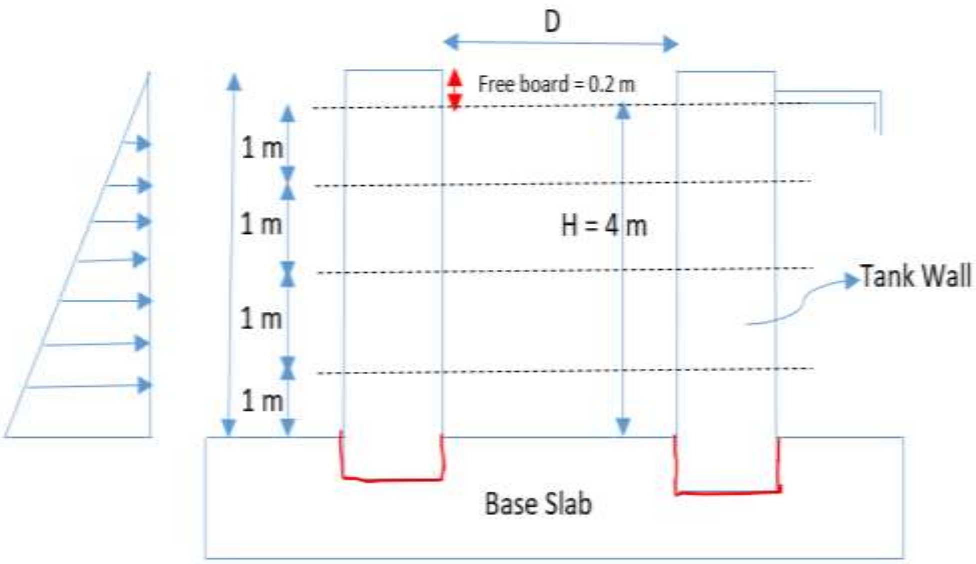
Capacity = 4×10^5 lts.

Depth H = 4m

Free board = 0.2 m

Take Unit weight of water = 9.81 KN/m^3

= 981 N/m^3



1. Design Constants

For M25, σ_{cbc} = 8.5 N/mm^2 , σ_{ct} = 1.31 N/mm^2

For Fe 415, σ_{st} = 150 N/mm^2

Modular ratio m = $\frac{280}{3\sigma_{cbc}}$ = $\frac{280}{3*8.5}$ = 10.98

2. Dimensions of the Water Tank:

Equating capacity to volume

i.e. capacity = Volume

4 x 10^5 lts. = Area * Height

$\frac{4 \text{ x } 105}{1000}$ m^3= $\frac{\pi \ \ D^2}{4}$* 4 m

D = 11.28m say D = 11.3 m.

3. Hoop Tension (in the bottom 1m height)

Here H = 4 m from top

Maximum Hoop tension = W *H * $\frac{D}{2}$

= 9.81 * 4 * $\frac{11.3}{2}$

= 221.70 KN

∴ Area of hoop tension steel, A_{st} = $\frac{Force}{\sigma st}$ = $\frac{max.Hoop\ tension}{\sigma st}$

Ast = $\frac{221.70 * 10^3}{150}$ = 1478 mm^2

Providing 16 mm dia bars

Spacing = $\frac{ast}{Ast} * 1000$ = $\frac{\frac{\pi * 16^2}{4}}{1478} * 1000$ = 136.03 = 130 mm

∴ Provide #16mm hoop tension steel @ 130 mm c/c in the bottom 1m height

4. Hoop Tension in 1m to 2 m from bottom:

Here H = 3 m from top

Hoop tension = W *H_1 * $\frac{D}{2}$

= 9.81 * 3 * $\frac{11.3}{2}$

= 166.27 KN

∴ Area of hoop tension steel, $A_{st} = \frac{Force}{σst} = \frac{Hoop\ tension}{σst}$

$Ast = \frac{166.27 * 10^3}{150}$ = 1108.46 mm^2

Providing 16 mm dia bars

Spacing = $\frac{ast}{Ast} * 1000 = \frac{\frac{\pi * 16^2}{4}}{1108.46} * 1000 =$ 181.3 = 180 mm

∴ Provide #16mm hoop tension steel @ 180 mm c/c between 1m to 2m from bottom

5. Hoop Tension between 2m – 3m from bottom:

Here H = 2 m from top

Hoop tension = W *H_2 * $\frac{D}{2}$

= 9.81 * 2 * $\frac{11.3}{2}$

= 110.85 KN

∴ Area of hoop tension steel, $A_{st} = \frac{Force}{σst} = \frac{Hoop\ tension}{σst}$

$Ast = \frac{110.85 * 10^3}{150}$ = 739 mm^2

Providing 16 mm dia bars

$$\text{Spacing} = \frac{ast}{Ast} * 1000 = \frac{\frac{\pi * 16^2}{4}}{739} * 1000 = \ \ 270 \text{ mm}$$

∴ Provide #16mm hoop tension steel @ 270 mm c/c between 2m to 3m from bottom

6. Wall Thickness: (T)

1. T = 30 H + 50 mm, Where H = Depth of water in m

   = 30 * 4 + 50 mm = 170 mm

2. $$\sigma_{cbc} = \frac{Maximum\ Hoop\ Tension}{1000\ T + (m-1)Ast.}$$

   $$1.30 = \frac{221.70 * 10^3}{1000\ \ T + (10.98 - 1) * 1478}$$

   T = 155.78 mm

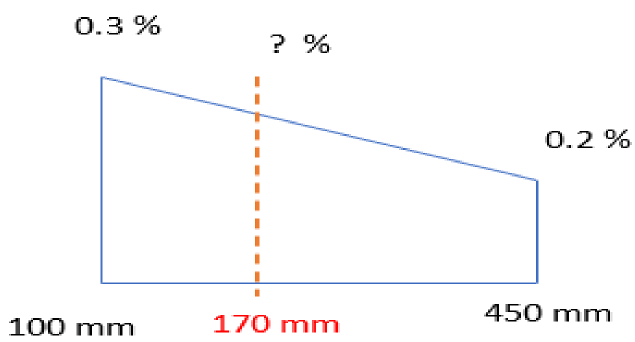
∴ Take T = 170 mm

7. Steel for remaining top 1 m height: (between 3 to 4 m)

For top 1m, provide minimum steel,

For 100 thick wall, Minimum steel = 0.3 % of gross area

For 450 thick wall, Minimum steel = 0.2 % of gross area



∴ For 170 thick wall, Minimum steel = 0.28 % of gross area

= 0.28/100 * (1000 *170)

= 476 mm^2

Using 12 mm dia bars

Spacing = $\frac{ast}{Ast} * 1000 = \frac{\frac{\pi * 12^2}{4}}{476} * 1000$ = 237 say 230 mm

∴ Provide #12 mm hoop tension steel @ 230mm c/c from top 1m

7. Vertical Distribution Steel:

Area of steel = 0.28 % of gross area

= 0.28/100 * (1000 *170)

= 476 mm^2

Using 10 mm dia bars

Spacing = $\frac{ast}{Ast} * 1000 = \frac{\frac{\pi * 10^2}{4}}{476} * 1000$ = 164 say 160 mm

∴ Provide #10 mm @ 160 mm c/c as Vertical Steel.

8. Base slab Design (Floor slab):

Base slab is continuously supported on ground, therefore provide a minimum thickness of 150 mm and reinforcement of 0.3 % of steel in the form of mesh @ top and bottom

∴ Area of steel = 0.3% of Area

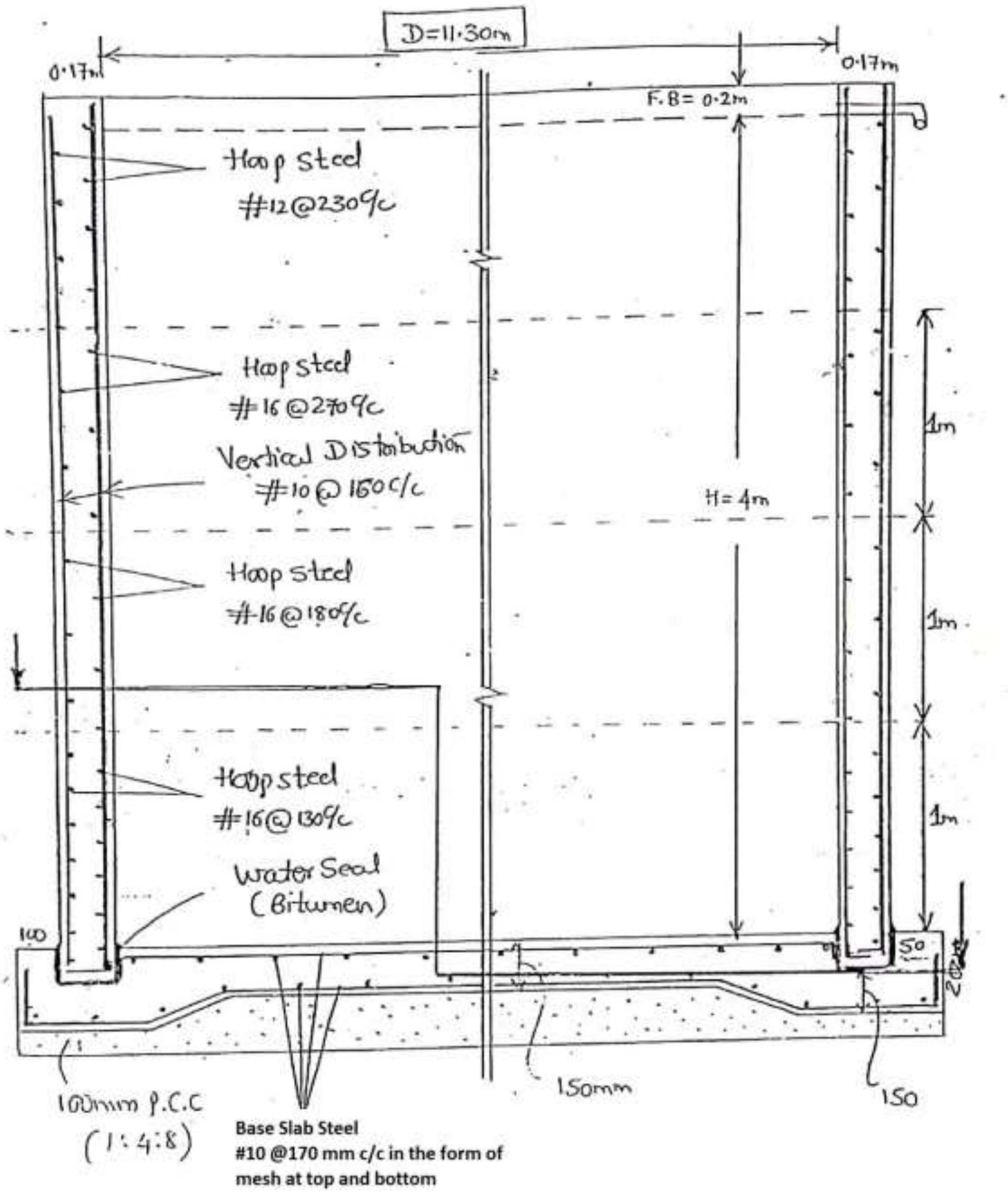
= 0.3/100 *(1000*150)

= 450 mm^2

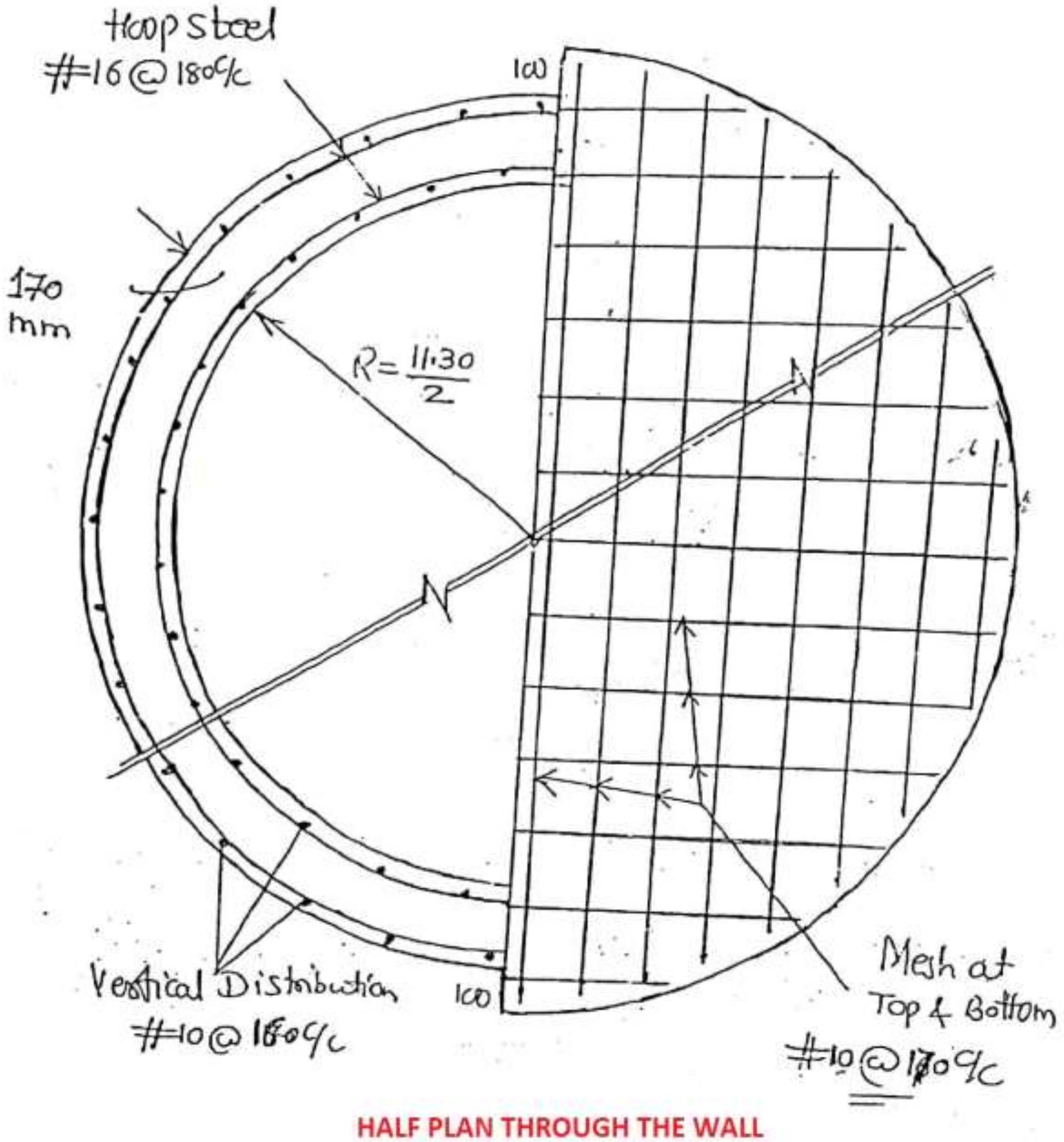
Using 10 mm dia bars

$$\text{Spacing} = \frac{ast}{Ast} * 1000 = \frac{\frac{\pi * 10^2}{4}}{450} * 1000 = 174.53 \text{ say } 170 \text{ mm}$$

∴ Provide #10 mm @ 170 mm c/c



CROSS SECTION OF THE TANK



HALF PLAN THROUGH THE WALL

AND

HALF PLAN THROUGH THE BASE SLAB

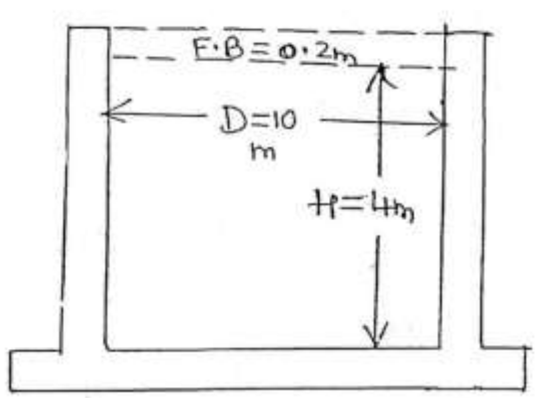
Design of Circular Water Tank with RIGID BASE

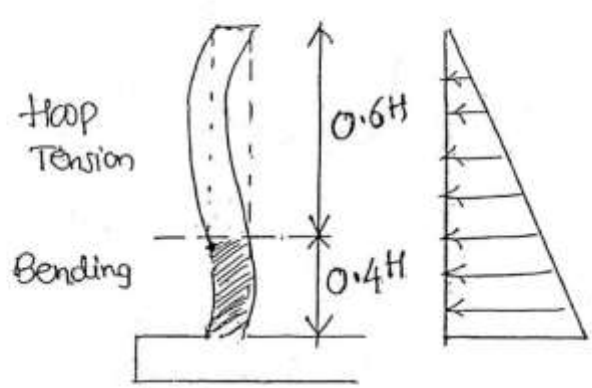
(Fixed or rigid base or restrained at the base)

1. Design a circular water tank of an internal dia 10 m and height 4m, the walls are restrained at the base. Use IS code method. Design the tank for M25 and Fe415. Draw sketches showing reinforcements
   a. Cross section of water tank.
   b. Draw half plan through wall.
   c. Draw half plan through base slab.

Given Data:

Diameter = 10m, height = 4m, Assume free board = 0.2 m





1. Design Constants:

For M25, σ_{cbc} = 8.5 N/mm^2 , σ_{ct} = 1.31 N/mm^2

For Fe 415, σ_{st} = 150 N/mm^2

$$m = \frac{280}{3\sigma_{cbc}} = \frac{280}{3*8.5} = 10.9$$

$$k = \frac{m\,\sigma_{cbc}}{m\,\sigma_{cbc} + \sigma_{st}} = \frac{10.98*8.5}{10.98*8.5 + 150} = 0.383$$

$$j = 1 - \frac{0.383}{3} = 0.872$$

$Q = \frac{8.5 * 0.383 * 0.872}{2}$ = 1.42

2. Thickness of Wall:

T = 30 H + 50 mm = 30*4 +50 = 170 mm

Using 50 mm Effective Cover, d = 170 – 50 = 120 mm

3. Hoop Tension and Bending Moment (Ring Tension):

Hoop Tension:

From IS 3370 (Part IV) , Table-9, Page 35

Hoop Tension = Coefficient * H * $D/2$ *W (Kg/m)

Ratio = $\frac{H^2}{DT} = \frac{4^2}{10 * 170}$ = 9.41

Search for the maximum values, it coincides at 0.6 H

For 8 – 0.578

10 – 0.602

By interpolating for 9.41 – 0.598 $\approx$ 0.6

$\therefore$ Hoop Tension = 0.6 *4 * 10/2 * 9.81 = 117.72 KN

Bending Moment:

From IS 3370, Table 10, page 36

Moment = Coefficient * W *H^3

Ratio = $\frac{H^2}{DT} = \frac{4^2}{10 * 170}$ = 9.41

Search for the maximum values, it coincides at 1.0 H

For 8 – 0.0146

10 – 0.0122

By interpolating for 9.41 – 0.0129

$\therefore$ Moment = Coefficient * W *H^3 = 0.0129*9.81*4^3

= 8.10 KN-m

4. Area of Steel:

a. Hoop Steel for Hoop Tension:

$$Ast = \frac{Hoop\ Tension}{\sigma st} = \frac{118*10^3}{150} = 786.67\ mm^2$$

Providing 12 mm dia bars,

$$Spacing = \frac{ast}{Ast} * 1000 = \frac{\pi*12^2/4}{786.67} * 1000 = 140\ mm$$

$\therefore$ Provide 12mm dia bars @ 140 mm c/c up to a depth = 0.6 H = 2.4 m from bottom and 280 mm c/c for the remaining depth

b. Bending Moment or Cantilever Steel:

$$Ast = \frac{M}{\sigma st*j*d} = \frac{8.10 * 10^6}{150*0.872*135} = 459\ mm^2$$

T = 170 mm, Using 10 mm bar and 30 mm clear cover

d = 170 – 30 – 10/2 = 135 mm

Check for minimum Steel

Min Steel = 0.3 % of Gross area, for T = 100 mm

= 0.2 % of Gross area, for T = 450 mm

For T = 170 mm Min Steel = 0.28 % of Gross area

$\therefore$ Ast = 0.28/100 * 1000 * 170 = 476 mm²

Providing 10 mm dia bars,

$$\text{Spacing} = \frac{ast}{Ast} * 1000 = \frac{\pi * 10^2/4}{476} * 1000 = 160 \text{ mm}$$

$\therefore$ Provide 10mm dia bars @ 160 mm as Cantilever steel up to a height of 0.4*H = 1.6m (or 4 – 2.4 = 1.6 m) from bottom

c. Vertical Distribution Steel:

Area of steel = 0.28 % of Gross Area

$\therefore$ Ast = 0.28/100 * 1000 * 170 = 476 mm²

Providing 8 mm dia bars,

$$\text{Spacing} = \frac{ast}{Ast} * 1000 = \frac{\pi * 8^2/4}{476} * 1000 = 100 \text{ mm}$$

$\therefore$ Provide 8 mm dia bars @ 100 mm

5. Base Slab Design:

Provide 150 mm thick slab with 0.3% of Gross area of steel in the form of mesh @ top and bottom.

Area of steel = 0.3 % of Gross Area

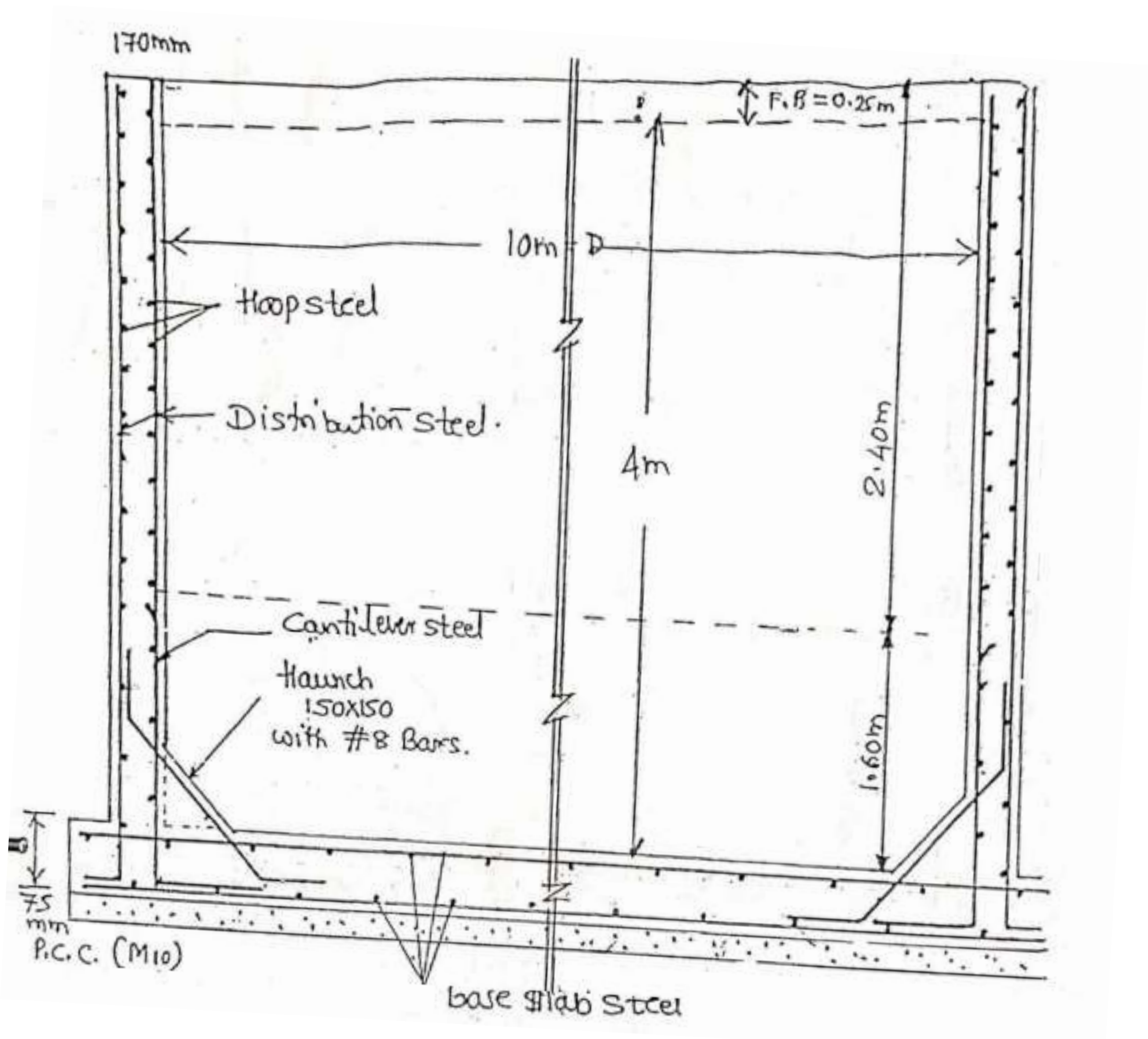
$\therefore$ Ast = 0.23/100 * 1000 * 150 = 450 mm²

Providing 10 mm dia bars,

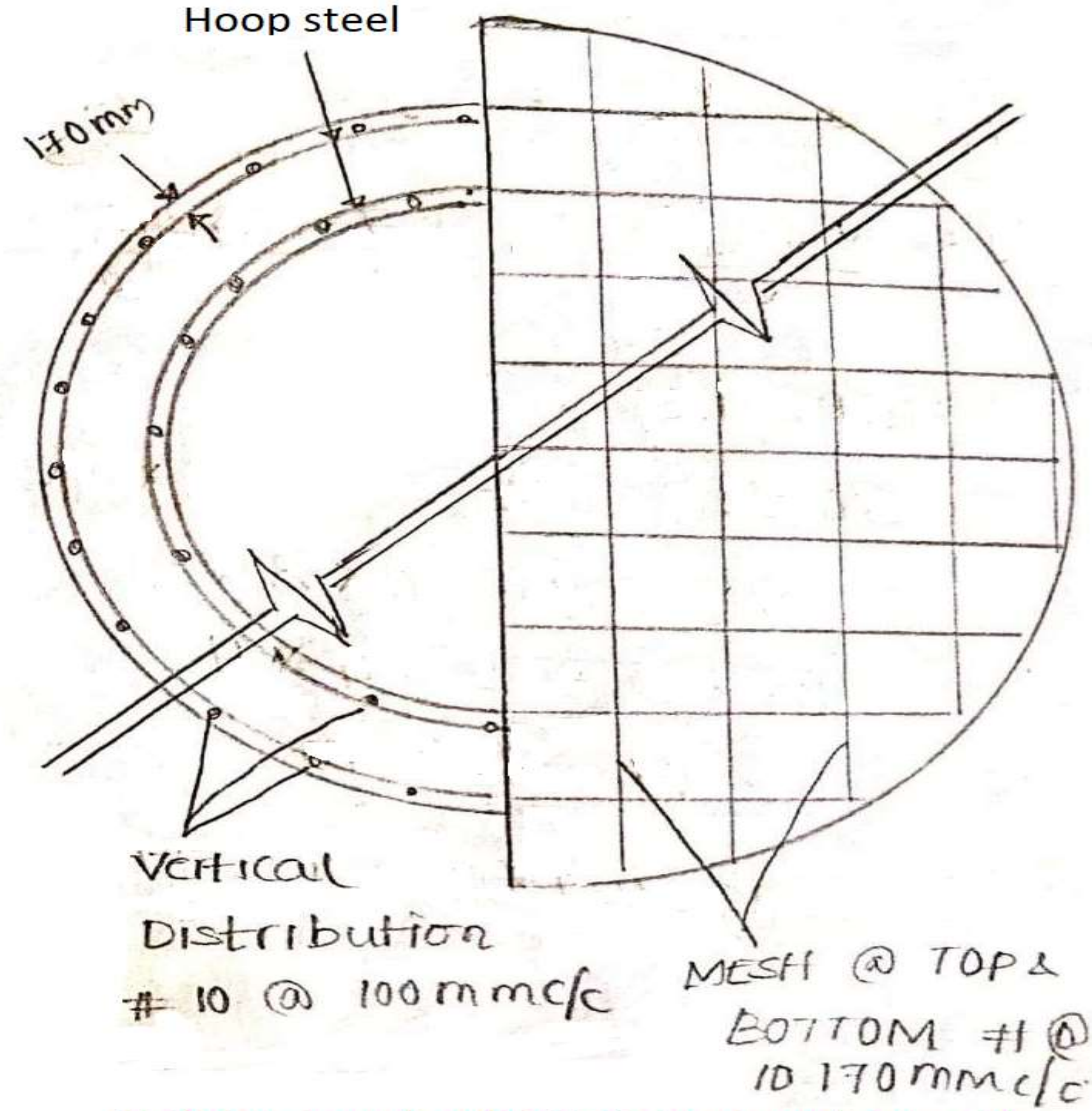
$$\text{Spacing} = \frac{ast}{Ast} * 1000 = \frac{\pi * 8^2/4}{476} * 1000 = 170 \text{ mm}$$

$\therefore$ Provide 10 mm dia bars @ 170 mm c/c in the form of mesh

Also provide haunch 150 mm x 150 mm with 8mm dia bars @ 200 mm c/c



Cross Section of Water Tank (fixed)



Half plan through wall & Half plan through base slab

2. Design a circular water tank 12 m dia, 4 m height, the tank rests on ground, the wall is fixed on a base slab. Use M20 concrete and Fe 415 steel. Adopt working stress method and design as per 3370. Also draw the following sketch

a. Section through the tank.

b. Base slab reinforcement at top and bottom.

Design of Rectangular Water Tank
(By using IS 3370 –Part(4)

1. A rectangular water tank with an open top is required to store 80,000 lts of water. The inside dimension of the tank may be taken as 6 x 4 m. Design the side wall of the tank using M20 Concrete and Fe415 steel. Use IS method. Also draw
   a. Sectional plan of the tank
   b. Longitudinal section of the tank
   c. Cross Section of the tank.

Given Data:

Capacity = 80,000 lts

Inside dimension L = 6m, B = 4m

1. Design Constants:

For M20 σ_{cbc} = 7 N/mm² σ_{ct} = 1.2 N/mm²

For Fe415 σ_{st} = 150 N/mm²

i. Modular ratio $m = \frac{280}{3\sigma_{cbc}} = \frac{280}{3*7} = 13.33$

ii. Neutral axis coefficient $k = \frac{m\,\sigma_{cbc}}{m\,\sigma_{cbc} + \sigma_{st}} = \frac{13.33*7}{(13.33*7 + 150} = 0.383$

iii. Lever arm constant $j = 1 - \frac{0.383}{3} = 0.8723$

iv. Moment of Resistance coefficient $Q = \frac{7*0.38*0.87}{2} = 1.16$

Equating Capacity to volume

i.e. Capacity = Volume

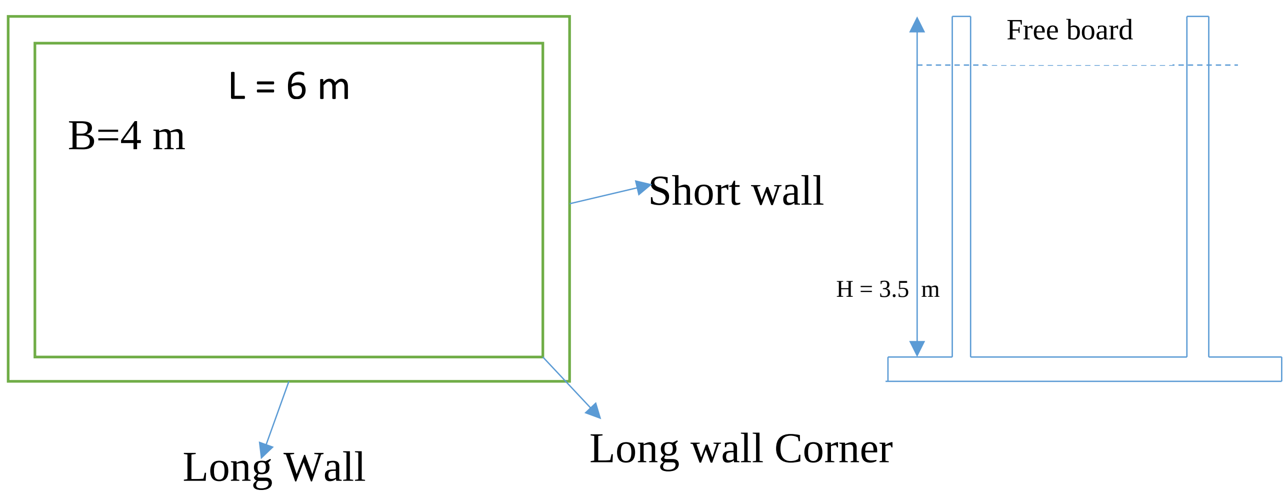
80,000 lts = Area * Height

$80\ m^3 = L * B * H$

$80 = 6 * 4 * H$

$H = 3.33\ m$

Providing 0.17 m as free board, $H = 3.33 + 0.17 = 3.5\ m$



2. Moment Calculation:

i. Moment Calculation for long wall:

As per IS 3370, table 3,

Horizontal moment $= M_y\, w\, a^3$

Vertical moment $= M_x\, w\, a^3$

a = height of the wall $= 3.5\ m$

b = Width of wall = 6 m

∴ Ratio = b/a = 6/3.5 = 1.71

	Mx	My
For 1.75 ……………	0.074	0.052
For 1.71	?	?
For 1.5 ……………	0.060	0.044

Select the Max Coefficients

∴ By interpolating for 1.71 Mx = 0.0719 & My = 0.051

∴ Horizontal moment = 0.051 * 10 * 3.5^3 = 21.86 KN-m

∴ Vertical moment = 0.071 * 10 * 3.5^3 = 30.87 KN-m

ii) Moment calculation for short Wall:

a = height = 3.5 m, b = 4 m

Ratio b/a = 4/3.5 = 1.14

	Mx	My
For 1.25 ……………	0.047	0.037
For 1.14	?	?
For 1.0 ……………	0.035	0.029

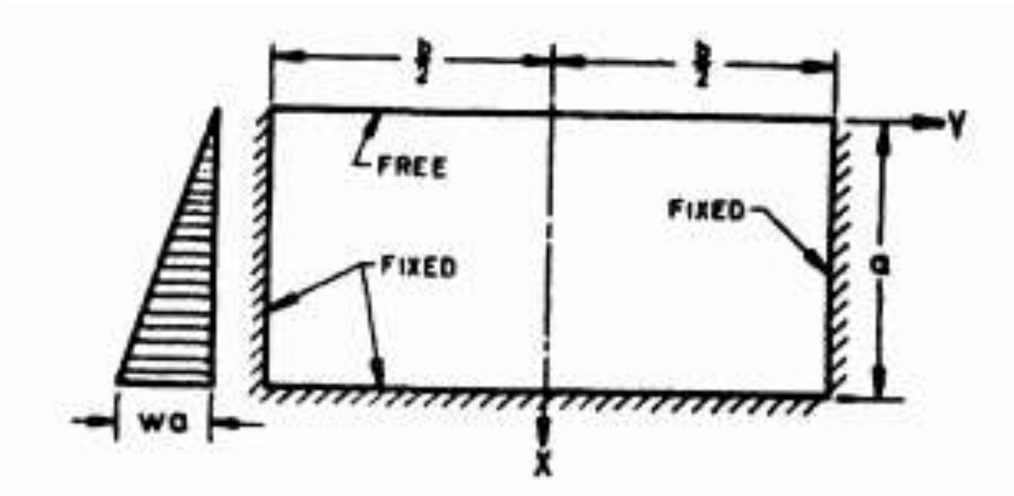
Select the Max coefficients

∴ By interpolating for 1.14, Mx = 0.042 & My = 0.033

∴ Horizontal moment = 0.033 * 10 * 3.5^3 = 14.14 KN-m

∴ Vertical moment = 0.042 * 10 * 3.5^3 = 18 KN-m

iii) Moment for long wall Corner:



a = height = 3.5 m, b = 6 m

Ratio = b/a = 6/3.5 = 1.71

Also $y = b/2$

From IS 3370, table 3,

	Mx	My
For 1.75 ……………	0.01	0.052
For 1.71	?	?
For 1.5 ……………	0.009	0.044

Select the Max coefficients

∴ By interpolating for 1.71, Mx = 0.009 & My = 0.050

Neglect Mx value since coefficient is very small

Horizontal moment = $M_y\, w\, a^3 = 0.050 * 10 * 3.5^3 = 21.437$ KN-m

3. Tank wall thickness:

Maximum Bending moment = 30.87 KN-m

$$\therefore \text{ Effective depth d} = \sqrt{\frac{m}{Q*b}} = \sqrt{\frac{30.87*10^6}{1.16*1000}} = 163.12 \text{ mm}$$

Providing 50mm effective cover D= 63.13 + 50 = 213.13 ≈ 220 mm

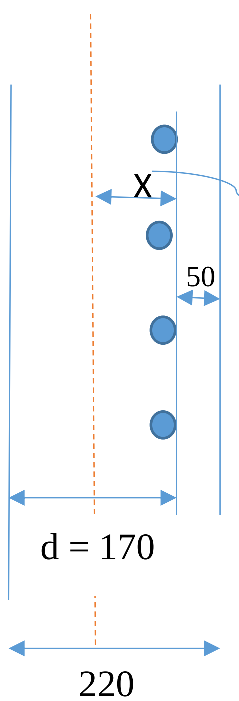
$$\boxed{\therefore \text{ Provide D} = 220 \text{ mm} \ \& \ \ d = 170 \text{ mm}}$$

4. Pull in each Wall:

$$\text{Pull in long wall} = \frac{wHB}{2} = \frac{10*3.5*4}{2} = 70 \text{ KN}$$

$$\text{Pull in short wall} = \frac{wHL}{2} = \frac{10*3.5*6}{2} = 105 \text{ KN}$$

5. Design of long wall:



$$X = (170 - \frac{220}{2})$$

$$= 60 \text{ mm}$$

i. Hoop steel or Horizontal Steel:

$$Ast = \frac{M-T.x}{\sigma st\ .j.d} + \frac{T}{\sigma st}$$

where M = Horizonal Moment = 21.86 KN-m,T = 70 KN

$$Ast = \frac{21.86*10^9\ (70*10^3\ )}{150*0.87*170} + \frac{70*10^3}{150} = 1263\ mm^2$$

Providing 16 mm dia bars,

$$Spacing = \frac{ast}{Ast} * 1000 = \frac{\pi*16^2/4}{1263} * 1000 = 160\ mm$$

∴ Provide 16 mm dia bars @ 160 mm c/c

ii. Vertical Steel

$$Ast = \frac{M}{\sigma st\ .j.d}$$

$$Ast = \frac{30.87*10^9}{150*0.87*170} = 1392\ mm^2$$

Providing 12 mm dia bars,

$$Spacing = \frac{ast}{Ast} * 1000 = \frac{\pi*12^2/4}{1392} * 1000 = 81.24\ mm\ say\ 80\ mm$$

∴ Provide 12 mm dia bars @ 80 mm c/c

6. Design of Short Wall:

i. Hoop steel or Horizontal Steel:

$$Ast = \frac{M-T.x}{\sigma st\ .j.d} + \frac{T}{\sigma st}$$

where M = Horizonal Moment = 14.14 KN-m, T = 105 KN

$$Ast = \frac{14.14*10^6\ (105*10^3\ )}{150*0.87*170} + \frac{105*10^3}{150} = 1054\ mm^2$$

Providing 16 mm dia bars,

$$\text{Spacing} = \frac{ast}{Ast} * 1000 = \frac{\pi * 16^2/4}{1054} * 1000 = 190 \text{ mm}$$

∴ Provide 16 mm dia bars @ 190 mm c/c

ii. Vertical Steel:

$$Ast = \frac{M}{\sigma st\ .j.d}$$

$$Ast = \frac{18 * 10^6}{150*0.87*170} = 811.35 \text{ mm}^2$$

Providing 12 mm dia bars,

$$\text{Spacing} = \frac{ast}{Ast} * 1000 = \frac{\pi * 12^2/4}{811.35} * 1000 = 139.2 \text{ mm say } 140 \text{ mm}$$

∴ Provide 12 mm dia bars @ 140 mm c/c

7. Design of Corner Wall :

$$Ast = \frac{M}{\sigma st\ .j.d}$$

$$Ast = \frac{21.43 * 10^6}{150*0.87*170} = 966 \text{ mm}^2$$

Providing 16 mm dia bars,

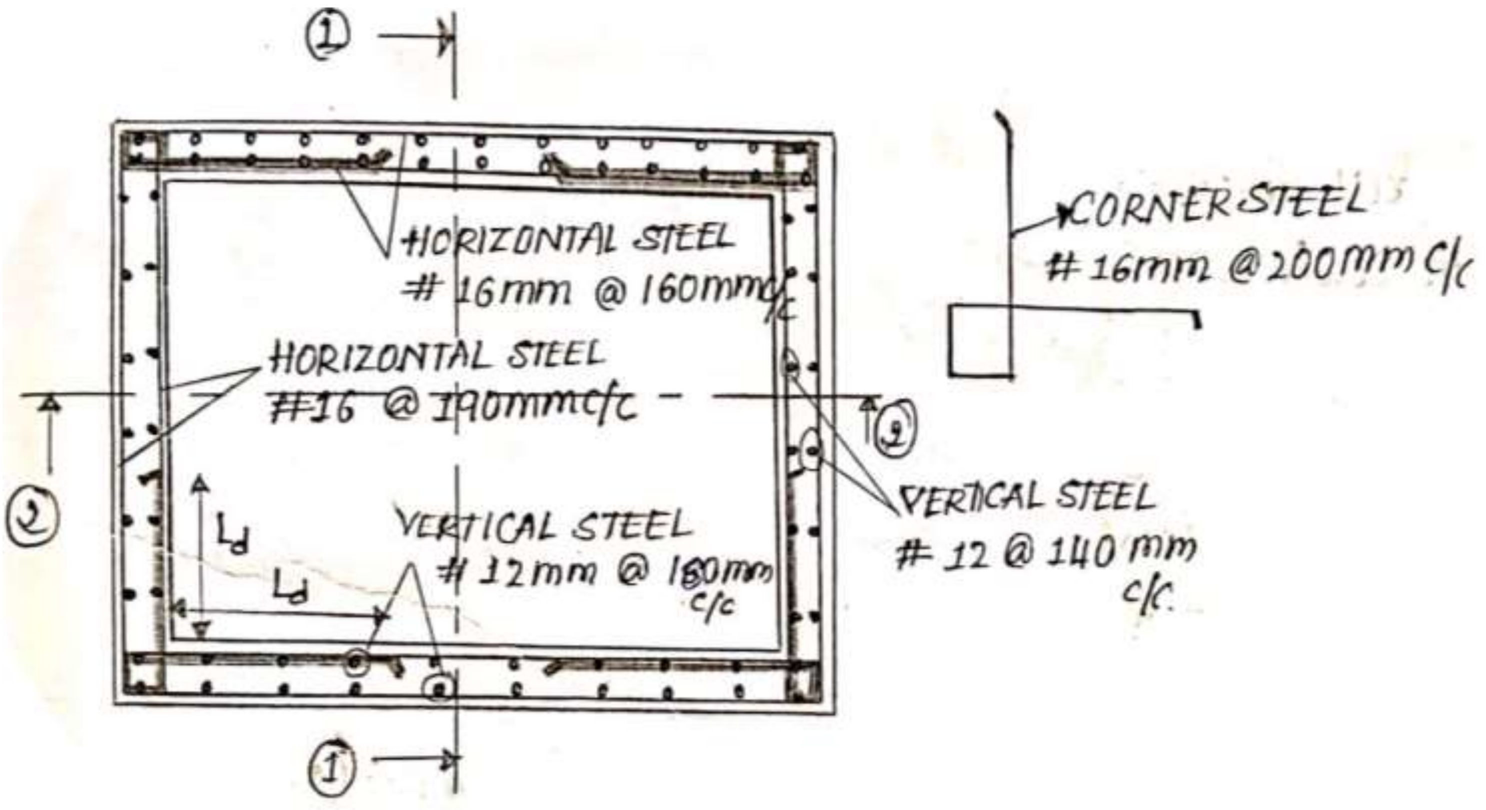
$$\text{Spacing} = \frac{ast}{Ast} * 1000 = \frac{\pi * 16^2/4}{966} * 1000 = 200 \text{ mm}$$

∴ Provide 16 mm dia bars @ 200 mm c/c

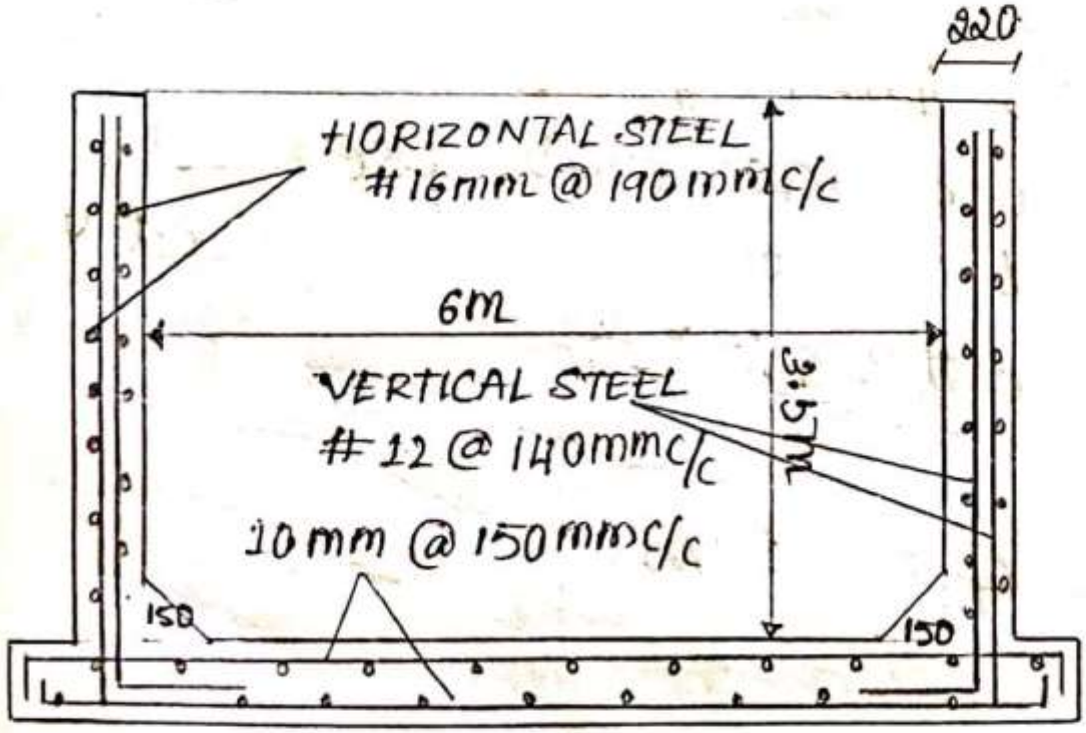
8. Design of Base Slab:

Provide minimum thickness = 150 mm

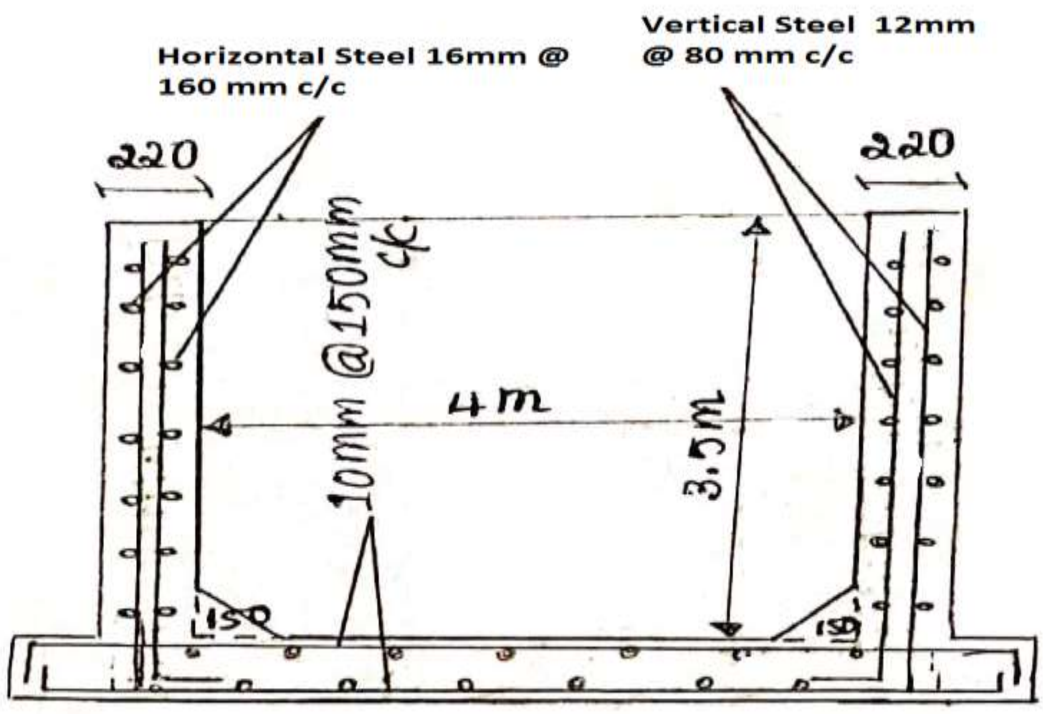
Also provide minimum steel in the form of mesh at the top and bottom = #10 @150 mm c/c.



Sectional Plan of the tank



CROSS SECTION ALONG SHORT WALL



CROSS SECTION ALONG LONGER WALL

2. A rectangular water tank with an open top is required to store 1,00,000 lts of water. The inside dimension of the water tank may be taken as 8 m x 4m. The tank rests on ground. Design the side walls of the tank using the following

Permissible Compressive stress in concrete = 7 N/mm^2

Permissible Tensile stress in steel = 150 N/mm^2

Modular ratio = 13.33

Draw the following sketches.

a. Sectional elevation through short wall.
b. Sectional elevation through long wall.
c. Sectional plan.

Solution:

Following are steps in the design

1. Design Constants and height of the tank
2. Moment Calculation
   i. Moment calculation for long wall
   ii. Moment calculation for short wall
   iii. Moment calculation for long wall corner.
3. Tank wall Thickness
4. Pull in each wall
5. Design of long wall
6. Design of short wall
7. Design of corner wall
8. Design of base slab

3. Design side walls of rectangular reinforced water tank of dimensions 6m x 2 m having a maximum depth of 2.5m using M20 grade concrete and Fe 415 HYSD bars.

Draw a sketch of

a. Sectional plan of the tank
b. Longitudinal section of tank
c. Cross section of the tank.

HINGED PORTAL FRAME

HINGED PORTAL FRAMES

Problem

An RCC portal frame with a hinge base is required to suit the following data

Spacing of portal frames = 4m c/c

Height of columns = 4m

Distance between column centres=10m

Live load on the roof = 1.5kN/m^2

The RC slab is continuous over portal frames

SBC of soil = 200kN/m^2

Materials M20 and Fe 415 steel.

Design the slab, portal frame and foundations.

Draw to a suitable scale

i) Sectional elevation of half frame showing the details of reinforcement in footing, column and beam of portal frame.
ii) Transverse section of beam and column
iii) Sectional plan of footing and column

Design:-

Design of continous slab

Effective depth = $\frac{Span}{26} = \frac{4000}{26}$ = 153.85mm

Assume 0.3% tension reinforcement, modification factor 1.4

Hence effective depth = $\frac{153.85}{1.4}$ = 109.9mm

Assume a clear cover of 20mm and 10mm diameter bars.

Total depth = 109.9+20+$\frac{10}{2}$ = 134.9mm say 150mm

Dead load on the slab = 0.15X24 = 3.6kN/m^2

Roof finishes = 0.756kN/m^2

Ceiling finishes = 0.256kN/m^2

Dead load/m^2	g = 4.6kN/m^2
Live load/m^2	q = 1.5kN/m^2

Maximum Negative BM

$$M = \frac{gl^2}{10} + \frac{ql^2}{9}$$

$$M = 4.6 \text{ x } \frac{4^2}{10} + 1.5 \text{ x } \frac{4^2}{9}$$

$$M = 10.03\text{kN-m}$$

Maximum Positive BM

$$M = \frac{gl^2}{12} + \frac{ql^2}{10}$$

$$M = 4.6 \text{ x } \frac{4^2}{12} + 1.5 \text{ x } \frac{4^2}{10}$$

$$M = 8.53\text{kN-m}$$

Factored design moment = 1.5x10.03=15kN-m

Assuming an effective depth as 0.9 times the total depth,

Ultimate resisting moment = $\frac{0.138x1000x(0.9x150)^2x20}{10^6}$ = 50.3kN-m>15kN-m, Hence ok.

Effective depth provided = 150-20-(10/2) =125mm

Design of reinforcement at top and bottom:-

$15x10^6$=0.87x415xA_{st}x125 (1 - $\frac{Ast*415}{1000*125*20}$)

$1.66x10^{-4}Ast^2 - A_{st} + 332.36 = 0$

Ast=353mm^2

Spacing of #10 = $\frac{\pi*10^2}{4*353}$*1000= 222.5mm

$\boxed{\text{Use \#10 @ 200mmc/c}}$

Distribution Steel:-

Ast== $\frac{0.12}{100}$x 1000x150 = 180mm^2

Spacing of #8 = $\frac{\pi*8^2}{4*180}$*1000= 279mm

$\boxed{\text{Use \#8 @ 250mmc/c}}$

Design of Portal Frame

Effective span of beam = 10m

Effective depth of the beam = $\frac{10000}{12}$ to $\frac{10000}{15}$ = 833.33mm to 666.7mm

Effective depth = 700mm

Overall depth= 750mm

Width of beam = 450mm

Load on frame

Load from the slab = (4.6+1.5)x4x1 = 24.4kN/m

Self-weight of beam = 0.45x0.63x1x25 = 7.1kN/m

Self-weight of finishes = 0.5kN/m

Load/m = 32kN/m

Height of centre line of beam above hinge, h = (4+0.10-0.5x0.75) = 3.72 m

AB = 3.72m; BC = 10m

$I_{AB} = \frac{450*600^3}{12} = 8.1\text{x}10^9 \text{mm}^4$

$I_{BC} = \frac{450*750^3}{12} = 1.58\text{x}10^{10} \text{mm}^4$

$I_{AB} : I_{BC} = 1:1.95$

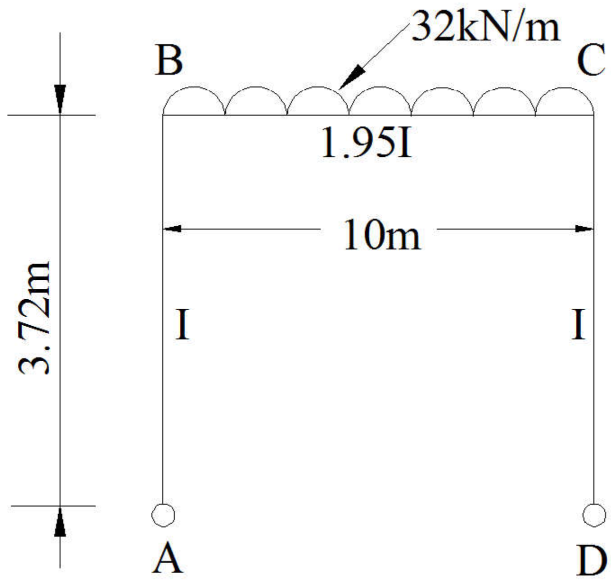


Fig - 1

<u>Relative stiffness values</u>

$$\left.\begin{array}{l} K_{BA} = (\frac{3}{4} \text{ x } \frac{I}{3.72}) = 0.2I \\ K_{BC} = \frac{1}{2} (\frac{1.95I}{10}) = 0.1I \end{array}\right\} \quad 0.3I$$

$d_{BA} = 0.2/0.3 = 0.67$

$d_{BC} = 0.1/0.3 = 0.33$

<u>Fixed End Moments</u>

$M_{FBC} = \frac{-32*10^2}{12} = -266.7 \text{ kNm}$

$M_{FCB} = \frac{-32*10^2}{12} = +266.7\text{kNm}$

	0.67	0.33
AB	BA	BC
		-266.7
	+177.8	+88.9
0	+177.8	-177.8

<u>Design Moments and Shear force</u>

Maximum Negative BM = 177.8 kN-m

Maximum positive moment at centre of span = $\frac{32*10^2}{8} - 177.8 = 222.2\text{kN-m}$

Maximum shear force at B = $\frac{32*10}{2}$ = 160kN

Shear force at the hinge at A = $\frac{177.8}{3.72}$ = 47.8kN

Factored moment at support B = 1.5x177.8=266.7kNm

Factored moment at centre of span= 1.5x222.2=333.3kNm

Factored shear force at hinge at A = 1.5x47.8 = 71.7 kN

Factored shear force at support B = 1.5x160=240kN

DESIGN OF BEAMS

Central section:-

Assume dimensions of the beam are

b_w = 450mm, d = 700mm, D_f = 150mm

$b_f = \frac{l_o}{6} + b_w + 6D_f = \frac{10000}{6} + 450 + (6*150) = 3017mm$

$\gamma = \frac{M_u}{b_f\, d^2 f_{ck}} = \frac{333.33*10^6}{3017x700^2x20} = 0.011$

$\gamma_{lim} = 0.36(\frac{150}{700})(1-0.42x\frac{150}{700}) = 0.07$

$\gamma < \gamma_{lim}$ hence NA is inside the flange.

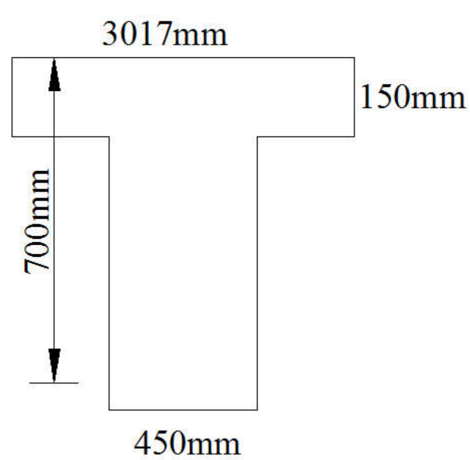


Fig-2

$333.3x10^6 = 0.87x415xAstx700\left(1 - \frac{Ast*415}{3017x700x20}\right)$

Ast = 1336.3mm^2

Use 4# of 25mm dia bars

Support section:-

Mu =266.7kN-m

$266.67x10^6 = 0.87x415xAstx700\left(1 - \frac{Ast*415}{450x700x20}\right)$

A_{st}= 1141mm^2

Use 4# of 20mm dia bars

DESIGN OF SHEAR REINFORCEMENT:-

Vu = 240kN

$\tau_c = \frac{vu}{bd} = \frac{240x1000}{450x700}$ = 0.76N/mm^2

$\frac{100*Ast}{bd} = \frac{100*4*\pi*25^2}{4*450*700} = 0.62$

Interpolation

0.50 0.48

0.62 ?

0.75 0.56

τ_c =0.52N/mm^2

V_{us}=240x10^3-(0.52x450x700) =76200

76200 = 0.87x415x2x$\frac{\pi*8^2}{4*s_v}$*700

S_V =333mm

Use 2L #8 @ 300c/c

Design of Column

Mu = 266.7 kN-m

Vu = 240kN

Assume an effective cover of 50mm, d'/D = 50/600=0.10

$\frac{M_u}{f_{ck}\, b\, d^2} = \frac{266.7 \; X \; 10^6}{20X450X600^2} = 0.082$

$\frac{P_u}{f_{ck} bd} = \frac{240 \; X \; 10^6}{20X450X600} = 0.044$

Referring to the chart given in SP16

$\frac{P}{f_{ck}} = 0.04$

P=0.04x20=0.8

Ast= $P \frac{bD}{100}$ = 0.8x450x $\frac{600}{100}$ = 2160 mm^2

Use 4#20 and #8 @ 300 as ties

DESIGN OF HINGE:-

Permissible bearing stress at the hinge = 0.5fck=10N/mm^2

Area of hinge = $\frac{160*10^3}{10}$ =16000mm^2

Area provided = 450*150 > 16000mm^2

Working shear at the hinge = 47.8kN

Factored shear at the hinge = 71.7kN

A_{sv}sin45x0.87x415 = 71.7x10^3

A_{sv} = 280.9mm^2

Use 4# 12mm dia

Spiral consisting of 10mm dia with 6mm diameter

Design of foundation

Axial load on the column = 160kN

Weight of column = 0.45 x 0.6x 3.72x 24 = 24kN

Self-weight of foundation 10% = 16kN

Total load = 200kN

Moment about the base (M) =47.8*1 = 47.8 kN-m

Eccentricity e = $\frac{M}{P}$ = $\frac{47.8}{200}$ =0.239m

Breadth of foundation = 6X239=1434mm

Provide a foundation of 1mX2m

Intensity of maximum pressure = $\frac{1.5X200}{1*2}$ = 150kN/m^2 < 200kN/m^2

$$p' = \frac{1.3}{2}X150=97.5\ kN/m^2$$

Total pressure on cantilever portion = $\left(\frac{97.5+150}{2}\right) * 0.7$ = 86.6 kN

BM, Mu= $(86.6*\frac{0.7}{2}) * 1.5$ =45kN-m

Effective depth required = $\sqrt{\frac{45X10^6}{0.138X20X1000}}$ = 127.6mm

From the shear considerations; double the effective depth say D= 300mm

<u>Design of main reinforcement</u>

$45X10^6=0.87X415XA_{st}X250\ (1-\frac{Ast}{1000X250}X\frac{415}{20})$

$A_{st}=521mm^2$

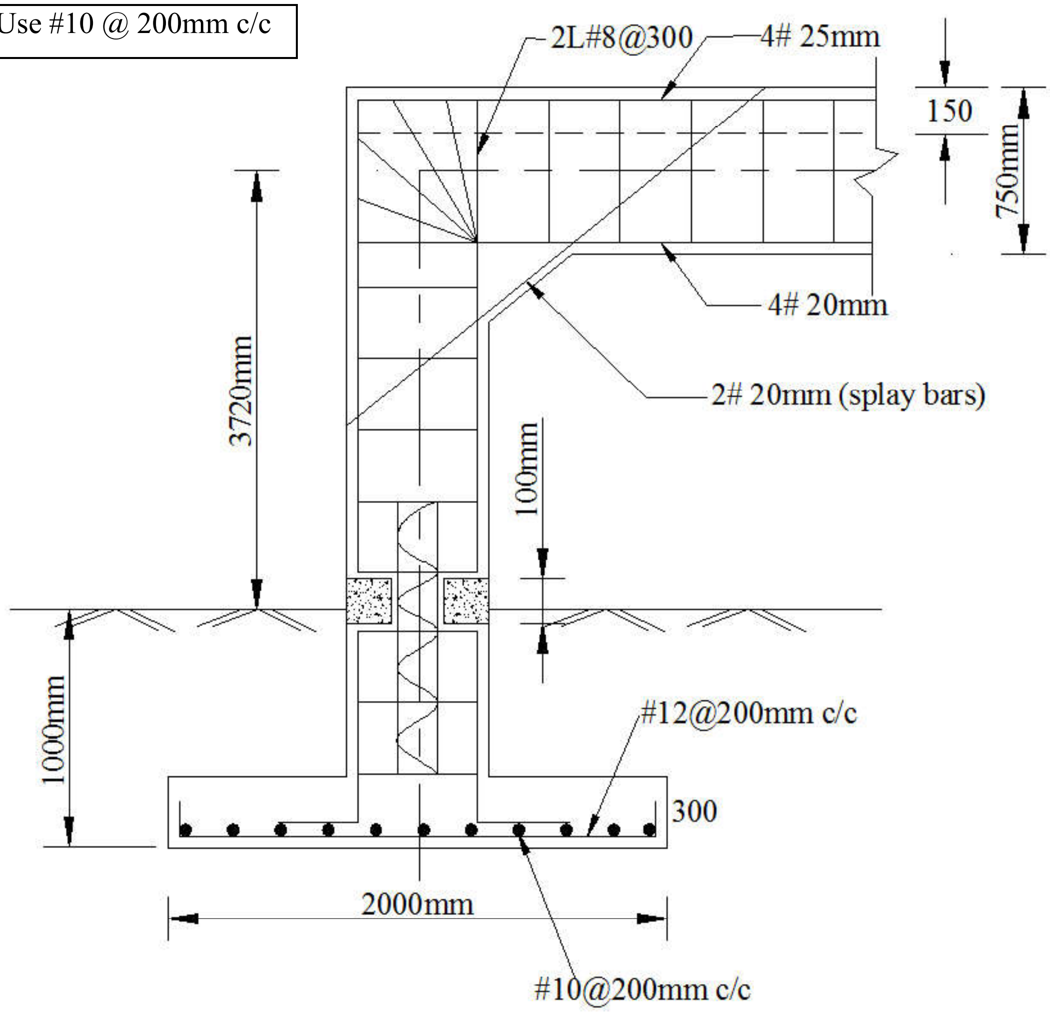
Spacing of #12 = $\frac{\pi*12^2}{4*521}$*1000= 217mm

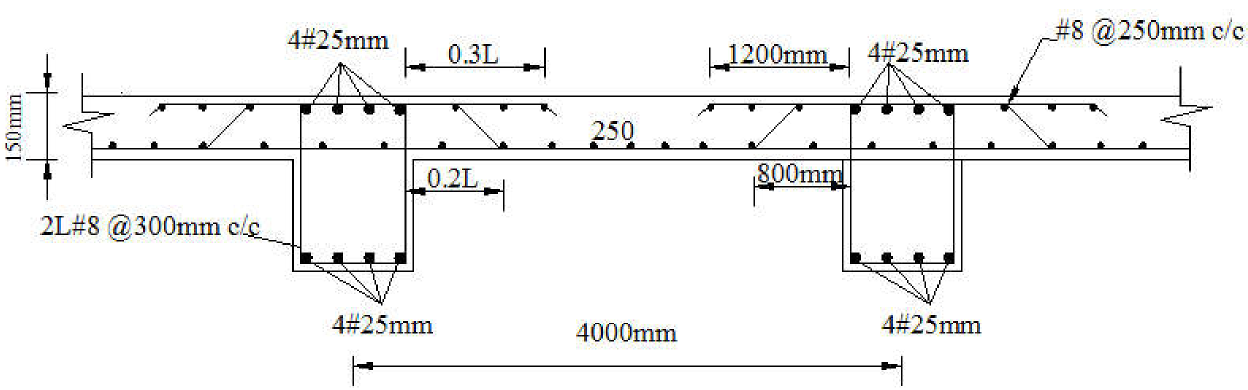
$\boxed{\text{Use \#12 @ 200mm c/c in both ways}}$

Distribution steel, Ast== $\frac{0.12}{100}$x 1000x300 = 360mm^2

Spacing of #10 = $\frac{\pi * 10^2}{4*360}$*1000= 218mm

Use #10 @ 200mm c/c



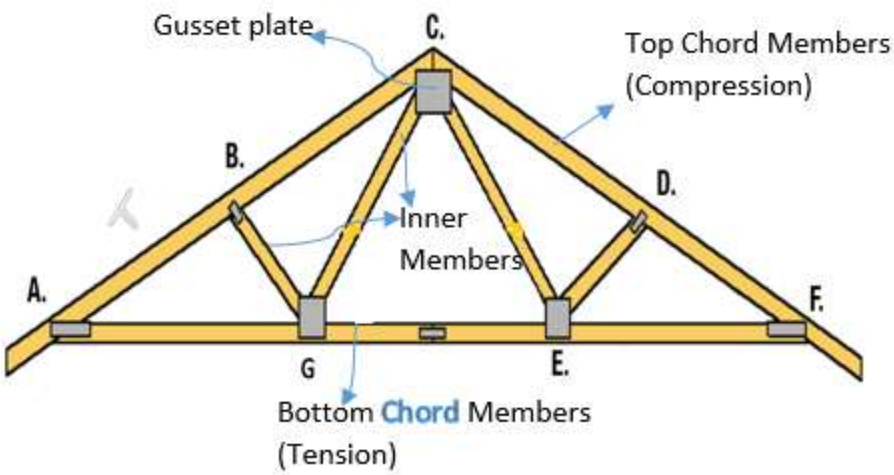


HINGED PORTAL FRAME

Module 2

Design of Steel Structures

Design of Roof Trusses:



Following points are followed while designing the roof truss

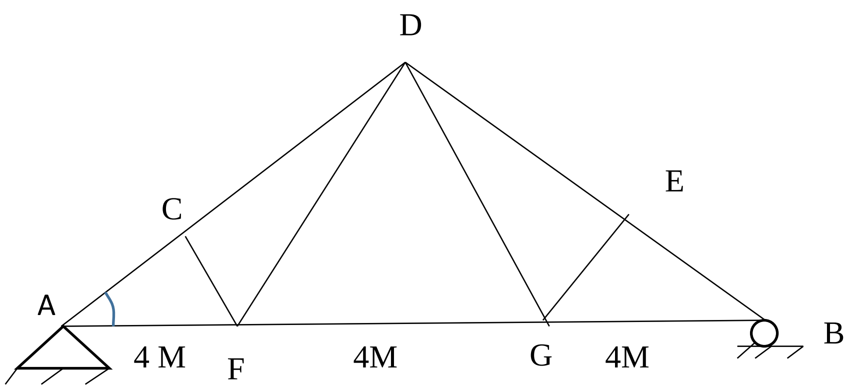
1. Select double angle for top chord and bottom chord members and single angle for inner members.
2. Provide minimum two number of bolts for the connection in Bolted roof truss.
3. Take effective length $\iota_e = 0.85\ \iota$
4. Provide uniform thickness of gusset plate
5. Select minimum size of angle i.e. ISA 50 x 50 x 6 mm
6. Design only four member's i.e.
    a. Outer Maximum Compression Member
    b. Outer Maximum Tension Member
    c. Inner maximum compression member
    d. Inner maximum tension member

1. The Centre line diagram of the steel truss is shown in figure. The magnitude and nature of forces in different members of the truss are given in table. The size of the RC column supporting the truss is 300 x 300 mm. Use M20 concrete for Column.

Design the truss using bolted or welded connection. Also design anchor bolts for an uplift force of 15 KN at each support.

Draw rough sketches of following

a. Elevation of truss greater than half span.
b. Enlarged view of apex joint of the truss.
c. Enlarged view of the left support joint.



AC = CD = DE =EB

Member	Force (KN)	Length (m)
AC,EB	-80 KN	3.46
CD,DE	-70 KN	3.46
AF,GB	+70 KN	4
FG	+50 KN	4
CF,EG	-24 KN	2
DF,DG	+24 KN	4

Reaction, Ra = Rb = 50 KN

-ve = Compession, +ve = Tension

Soln:

1. Design of outer Compression Member (Top Chord Member)

The members are AC, CD,DE and EB

Member AC $\Rightarrow$ 80 KN (C) $\Rightarrow$ L = 3.46 m

Member CD $\Rightarrow$ 70 KN (C) $\Rightarrow$ L = 3.46 m

Select maximum force = 80 KN

$\therefore$ Factored force = 1.5 x 80 = 120 KN

Maximum length = 3.48 m

Design the top chord as the compression member using double angle and bolts

i. Selection of Section:

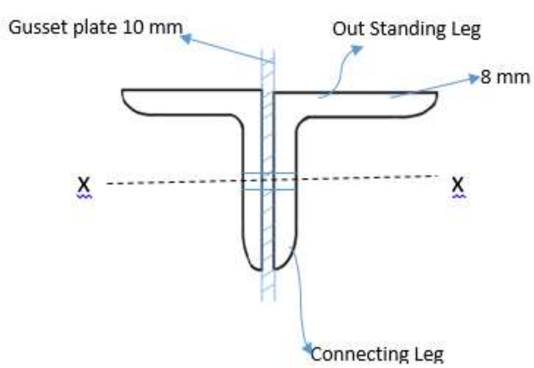
Assume f_{cd} = 60 N/mm^2

Using $P_d = A_c f_{cd}$ ....Page no. 34, IS 800

$120 \times 10^3 = A_c \times 60$

Ac = 2000mm^2 or 20 cm^2

From Steel tables select suitable double angle section.



Let us try 2ISA 70 x 70 x 8mm

$\therefore$ Ac = 2116 mm^2

Assume gusset plate thickness = 10 mm

From steel table r_{xx} = 21.2mm

r_{yy} = 32.9mm (For 10 mm gap)

$\therefore r_{min}$ = 21.2 mm

Length of the member L = 3.46 M = 3460 mm

$\therefore$ Effective Length Le = 0.85 L = 0.8 * 3460

= 2941 mm

$\therefore$ Slenderness ratio = ƛ = $\frac{Le}{r_{min}} = \frac{2941}{21.2}$

ƛ = 138.72

From table 9C (Page 42 IS 800)

For 130 -- fcd = 74.2

For 140 -- fcd = 66.2

$\therefore$ For 138 – fcd = 67.23 N/mm^2

$\therefore$ Design Compressive Strength Pd = Ac * fcd

= 2116 * 67.23

= 142.25 x 10^3 N > 120 KN

Hence Safe.

ii. Design of Connection:

Using m-16 Bolts and Grade 5.6 black bolts

d = 16 mm, do = 16 + 2 = 18 mm and fu = 500 N/mm^2

fu for plate = 410 N/mm^2

e = 1.5 * do = 1.5 * 18 = 27 mm say e = 30 mm

p = 2.5 *d = 2.5 * 16 = 40 mm

From IS 800, Page 75

Shear strength of Bolt

$$V_{dsb} = \frac{1}{\gamma_{mb}} \left[\frac{fub}{\sqrt{3}} (n_n A_{nb} + n_s A_{sb})\right]$$

Assume fully threaded bolts and double shear

n_n = 2 and n_s = 0

$V_{dsb} = \frac{1}{1.25}\ [\frac{500}{\sqrt{3}}\ (2* \frac{\pi\ 16^2}{4} * 0.78 + \ 0\ \ )\ ]$

= 72.43 KN

Similarly Bearing Strength

$V_{\ dpb} = \frac{1}{\gamma_{mb}}$ [2.5*k_b*d*t*fu) … Page 75 IS 800

k_b is taken as least of the following

$k_b\ = \ \frac{e}{3do} = \ \ 0.55$

$k_b\ = \ \frac{p}{3do}\ - 0.25 = 0.49$

$k_b\ = \frac{fub}{fu}\ = 1.21$

$k_b = 1$

$\therefore\ k_b = 0.49$

$\therefore\ V_{dpb} = \frac{1}{1.25}$ [2.5*0.49*16* 8*410) = 51.43 KN or 51.43 x 10^3 N

$\therefore$ Bolt value = 51.43 KN (Least of shear and Bearing Strength)

No. of Bolts = $\frac{Force}{Bolt\ Value}\ \ = \ \frac{120*10^3}{51.43*10^3}\ = 2.3 \approx 3$ Nos

Hence adopt 2ISA 70 x 70 x8 mm for top chord.

2. Design of Outer Tension Member (Bottom Chord Member)

The bottom chord members are AF = FG = GB

AF = 70 KN (T), L = 4 m

FG = 50 KN (T), L = 4 m

GB = 70 KN (T), L = 4 mα

Maximum force = 70 KN

∴ Factored force = 1.5 * 70 = 105 KN

Maximum Length = 4m

i. Selection of Section:

Using $T_{dn} = \frac{\alpha\ An\,fu}{\gamma_{ml}}$ ………… Page 33 IS 800

Here T_{dn} = Factored Load = 105 x 10^3 N

$\alpha = 0.7 \ \ and \ \ \gamma_{ml} = 1.25$

105 x 10^3 = $\frac{An\,fu}{\gamma_{ml}}$

$= 105 \times 10^3 \Rightarrow \frac{0.7 A_n \times 410}{1.25}$

$\boxed{A_n = 457.3 \text{ mm}^2}$

Increase the above area by 30% approximately

$\therefore (Area)_{G10} = 1.3 \times 457.3$

$\boxed{\text{Gross Area} = 594.49 \text{mm}^2}$

From steel table, try double angle

select minimum size $\boxed{2\,ISA\ 50 \times 50 \times 6 mm}$

$\boxed{\therefore \text{Area} = 1136 \text{ mm}^2}$

ii) Connections.

Providing m-16 grade 8.8 HSFG bolts (P-76)

$\therefore d = 16mm; d_o = 18mm; f_u = 800; \mu_f = 0.55, K_h = 1, n_e = 2$

shear strength $= V_{dsf} = \frac{1}{\gamma_{mf}} [\mu_f n_e K_h F_o]$

$F_o = A_{nb} f_u$

$= 0.78 \pi \frac{d^2}{4} \times \cancel{410}\, 800 \times 0.7$

$= \frac{1}{1.25} [0.55 \times 2 \times 1 \times 87.823 \times 10^3]$

$\boxed{F_o = 87.823 \times 10^3}$ $\boxed{V_{dsf} = 77.28 \times 10^3 N}$

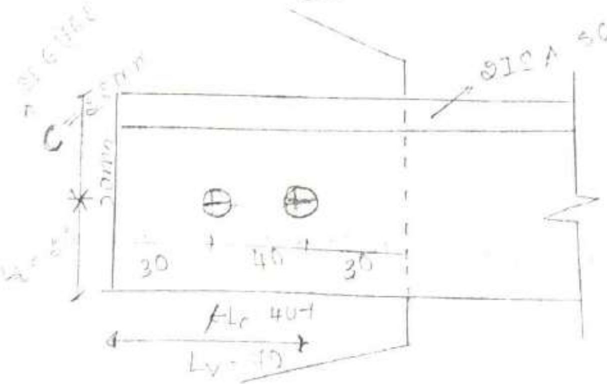
$\boxed{\text{Bolt Value} = 77.28 \times 10^3 N}$

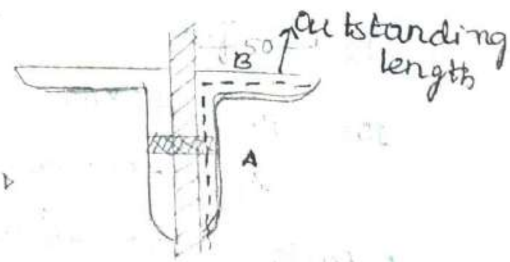
No of bolts $= \frac{\text{Force}}{\text{Value Bolt}} = \frac{105 \times 10^3}{77.28 \times 10^3} = 1.35$

$\boxed{\text{say 2 no of bolts}}$

iii) Check for rupture (Pg-33)

$T_{dn} = \left[\frac{0.9 A_{nc} f_u}{\gamma_{ml}} + \beta A_{go} \frac{f_y}{\gamma_{mo}}\right] \times 2$ → Due to double angle.





$\beta = 1.4 - 0.076 \left[\frac{L_v}{t}\right] \left[\frac{f_y}{f_u}\right] \left[\frac{b}{L_c}\right]$

$W = 50mm,\ T = 6mm\ f_y = 250 N/mm^2;\ f_u = 410.$

$b_s = W + (W_1 - T)$

$= 50 + 28 - 6 \qquad b_s = 72$

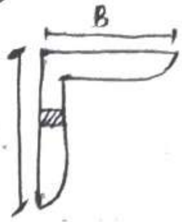
$= 1.4 - 0.076 \left[\frac{70}{6}\right] \left[\frac{250}{410}\right] \left[\frac{72}{40}\right]$

$\boxed{\beta = 0.704}$

$A_{go} = (B - t/2) * t = (50 - 6/2) * 6 \Rightarrow \boxed{A_{go} = 282\ mm^2}$

~~Adn~~ $A_{nc} = (A - d_0 - t/2) * t = (50 - 18 - 6/2) * 6$

$\boxed{A_{nc} = 174\ mm^2}$



$T_{dn} = \left[\frac{0.9 * 174 * 410}{1.25} + \frac{0.704 * 282 * 250}{1.10}\right] * 2$

$\boxed{T_{dn} = 192 * 10^3 N \simeq 193\ KN}$

$\therefore$ $T_{dn} > 105KN$ Hence safe.

iv) Check for block shear [P-33]

$T_{db_1} = 2\left[\frac{A_{vg} f_y}{\sqrt{3} * \gamma_{m0}} + \frac{0.9 A_{tn} f_u}{\gamma_{mL}}\right]$

$A_{vg} = L_v * t$
$= 70 * 6$
$A_{vg} = 420 mm^2$

$T_{db_2} = 2\left[\frac{0.9 A_{vn} f_u}{\sqrt{3} * \gamma_{mL}} + \frac{A_{tg} f_y}{\gamma_{m0}}\right]$

$A_{vn} = 420 - 1.5 * 18 * 6$
$A_{vn} = 258 mm^2$

$A_{tg} = L_t * t = 22 * 6 \Rightarrow A_{tg} = 132 mm^2$

$A_{tn} = 132 - (0.5 * 18 * 6) \Rightarrow A_{tn} = 78 mm^2$

$T_{db_1} = 2\left[\frac{420 * 250}{\sqrt{3} * 1.10} + \frac{0.9 * 78 * 410}{1.25}\right]$

$T_{db_1} = 156.27 * 10^3 N > 105 * 10^3 N$

$T_{db_2} = 2\left[\frac{0.9 * 258 * 410}{\sqrt{3} * 1.25} + \frac{132 * 250}{1.10}\right]$

$T_{db_2} = 147.94 * 10^3 N > 105 * 10^3 N$ Hence safe

$\therefore$ adopt 2ISA 50 * 50 * 6mm for bottom chord

3] Design of Inner Tension Member.

DF = 24 KN (T) ; L = 4m

DG = 24 KN (T) ; L = 4m

$\therefore$ Factored force = 24 * 1.5 ⇒ W = 36 KN (T)

i) selection of section Assume $\alpha = 0.6$

$T_{dn} = \frac{\alpha A_n f_u}{\gamma_{mL}} \Rightarrow 36 * 10^3 = \frac{0.6\ A_n * 410}{1.25}$

$A_n = 182.9\ mm^2$

Increase the area 30% approximately

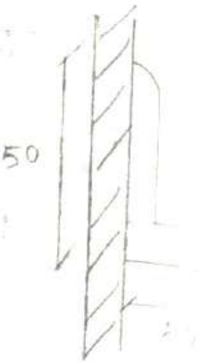
Area = 1.3 * 182.9

Gross Area = $237.77\,mm^2$

From steel table, select single angle

Try minimum size ISA 50×50×6mm

∴ Area = $568\,mm^2$

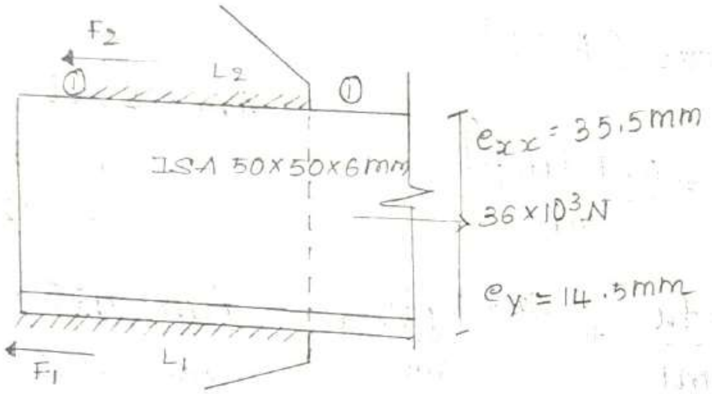


ii) Welded Connection

Take size of weld, $S = \frac{3}{4} \times$ angle thickness

$= \frac{3}{4} \times 6$

$S = 4.5\,mm \simeq S = 4\,mm$



Equating, Force = strength of the weld

$$36\times10^3 = \frac{0.7\, S.L\, f_u}{\sqrt{3}\ \gamma_{mw}}$$

shop Weld, γ_{m1} ⇓ 1.25

$$36\times10^3 = \frac{0.7\times4\times L\times410}{\sqrt{3}\times1.25}$$

$L = 67.89 \simeq 70\,mm \longrightarrow ①$

Take moment about 1-1.

$(F_1\times50) - (36\times10^3)\times35.5 = 0$

$$F_1 = \frac{0.7\times4\times L\times410}{\sqrt{3}\times1.25} - (36\times10^3\times35.5) = 0$$

$L_2 = 48.2\,mm \simeq L_1 = 50\,mm$

$L_2 = 20\,mm.$

iii) Check for rupture.

$$T_{dn}\left[\frac{0.9\,A_{nc}\,f_u}{\gamma_{m1}} + \frac{\beta\,A_{go}\,f_y}{\gamma_{m0}}\right]$$

$$\beta = 1.4 - 0.076\left[\frac{W}{t}\right]\left[\frac{fy}{fu}\right]\left[\frac{b_s}{L_c}\right]$$

$$= 1.4 - 0.076\left[\frac{50}{6}\right]\left[\frac{250}{410}\right]\left[\frac{50}{70}\right]$$

$$\beta = 1.124$$

$b_s = W = 50mm$

L_c = Weld length $= L_1 + L_2 = 70mm$

$A_{go} = (B - t/2) * t = (50 - 6/2) * 6 \Rightarrow A_{go} = 282mm^2$

$A_{nc} = [A - d_o - t/2]t = [50 - 0 - 6/2] * 6 \Rightarrow A_{nc} = 282mm^2$.

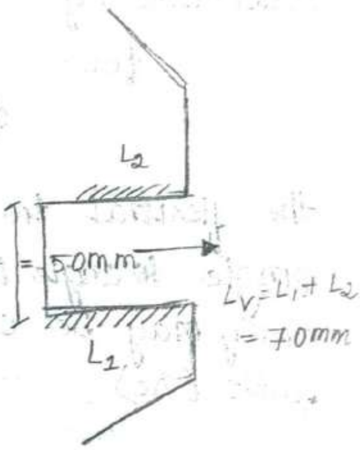
$$T_{dn} = \left[\frac{0.9 * 282 * 410}{1.25} + \frac{1.124 * 282 * 250}{1.10}\right]$$

$T_{dn} = 155.34 \times 10^3 N > 36 KN$ Hence safe

iv) Check for block shear.

$$T_{db_1} = \left[\frac{A_{vg} f_y}{\sqrt{3} * \gamma_{mo}} + \frac{0.9 A_{tn} f_u}{\gamma_{mL}}\right]$$

$$T_{db_2} = \left[\frac{0.9 A_{vn} f_u}{\sqrt{3} * \gamma_{mL}} + \frac{A_{tg} f_y}{\gamma_{mo}}\right]$$



$A_{vg} = A_{vn} = L_v \times t \Rightarrow 70 \times 6 = 420mm^2$

$A_{tg} = A_{tn} = L_t \times t \Rightarrow 50 * 6 = 300mm^2$

$$T_{db_1} = \left[\frac{420 * 250}{\sqrt{3} * 1.10} + \frac{0.9 * 300 \times 250}{\sqrt{3} * 1.25}\right] \quad \boxed{T_{db_1} = 109.110 KN > 36 KN}$$

$$T_{db_2} = \left[\frac{0.9 * 420 * 410}{\sqrt{3} * 1.25} + \frac{300 * 250}{1.10}\right] \Rightarrow \boxed{T_{db_2} = 139.76 KN > 36 KN}$$

Hence safe

∴ adopt ISA 50 * 50 * 6mm for inner tension member

4] Design of Inner Compression Member [Pg - 48]

Loaded through one leg.

CF = -24 KN (C), L = 2m

GE = -24 KN (C); L = 2m

∴ Factored force - 36 KN

$L = 0.85 * L = 0.85 * 2 \Rightarrow \boxed{L = 1.7m = 1700mm}$

i) Selection of section:

Assume, $f_{cd} = 20 N/mm^2$

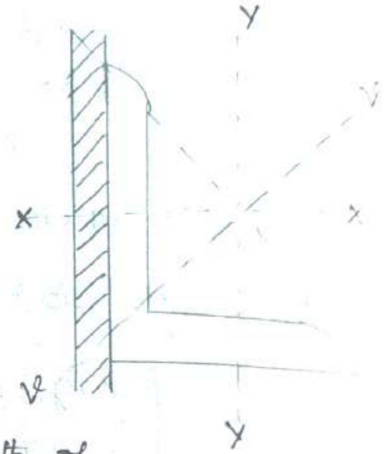
Using $P_d = A_c f_{cd}$ → [Pg 34]

$36 \times 10^3 = A_c \times 20$

$$\boxed{A_c = 1800 mm^2}$$

From steel table select single angle. try ISA 70×70×8mm

$$\boxed{\begin{array}{l} Area = 1058 mm^2 \\ r_x = r_y = 21.2 \\ r_{vv} = 13.5 \;;\; r_{uu} = 26.7 \end{array}}$$



ii) f_{cd} Calculation [Pg. ~~48~~ ~~35~~ 34]

design compressive stress, f_{cd} is Calculated by

$$f_{cd} = \frac{f_y/\gamma_{m0}}{\phi + [\phi^2 - \lambda_c^2]^{0.5}}$$

the flexural ~~to~~ torsion buckling strength of single ~~to~~ angle loaded in compression through one of its leg away may be evaluated using the equivalend slenderness ratio (λ_e)

$$\lambda_e = \sqrt{K_1 + K_2 \lambda_{vv}^2 + K_3 \lambda_\phi^2} \quad [Pg. 48]$$

$l = l_e = 1.7m$; $b_1 = b_2 = 70mm$ (CBK) ; $r_{vv} = 13.5mm$; $t = 8mm$

$\varepsilon = 1$; $E = 2 \times 10^5 N/mm^2$

Take no of bolts greater than or equal to 2 & fixed condition ∴ from table 12 [Pg 48]

$K_1 = 0.2$; $K_2 = 0.35$; $K_3 = 20$

(From Code book) (Pg. 48) $\lambda_{vv} = \dfrac{l/r_{vv}}{\varepsilon\sqrt{\frac{\pi^2 E}{250}}} = \dfrac{1700/13.5}{1 \times \sqrt{\frac{\pi^2 \times 2 \times 10^5}{2500}}}$

$$\boxed{\lambda_{vv} = 1.417}$$

$$\lambda_\phi = \frac{(b_1+b_2)/2t}{\varepsilon\sqrt{\frac{\pi^2 E}{250}}} = \frac{(70+70)/2 \times 8}{\sqrt{\frac{\pi^2 \times 2 \times 10^5}{250}}} = 0.098$$

∴ equivalence slenderence ratio : $(\lambda_c) = \sqrt{K_1 + K_2 \lambda_{vv}^2 \ldots}$

$$= \sqrt{0.2 + 0.35 \times (1.417)^2 + 20 \times (0.098)^2}$$

$$\boxed{\lambda_c = 1.046 \simeq 1.05}$$

[Pg 35] Taking α = imperfection factor for buckling class C

$\boxed{\alpha = 0.49}$

[From Pg no 34]

$$\phi = 0.5[1+\alpha \times (\lambda_c - 0.2) + \lambda_e^2]$$

$$= 0.5[1+0.49\times(1.05-0.2)] + 1.05^2]$$

$\boxed{\phi = 1.26}$

$$f_{cd} = \frac{f_y/\gamma_{mo}}{\phi[\phi^2-\lambda_c^2]^{0.5}} = \frac{250/1.10}{1.26[(1.26)^2 - 1.05^2]^{0.5}} = \boxed{116.20\ N/mm^2}$$

∴ Design compressive force.

$$P_d = A_e f_{cd} = 1058 * 116.20$$

$\boxed{P_d = 122.93 * 10^3\ N > 36\times10^3 N}$

Hence safe.

iii) Design of connection.

Using M 16, Grade 8.8, HSFG bolts

∴ Bolt value / shear strength $= V_{dsf} = \frac{1}{\gamma_{mf}}[\mu_f n_e k_h F_o]$

$F_o = A_{nb} f_u$

$= 0.78 \frac{\pi d^2}{4} \times 0.7 * 800$

$\boxed{F_o = 87.823 \times 10^3}$; $\mu_f = 0.55$; $K_h = 1$

$n_e = 1$

$$= \frac{1}{1.25}[0.55\times1\times1\times 87.823\times10^3]$$

$\boxed{V_{dsf} = 38.64\ kN}$

No of bolts $= \frac{\text{Force}}{\text{Bolt value}} = \frac{36\times10^3}{38.64\times10^3} = 0.931$

∴ Provide min no of bolts = 2 no's.

5] Design of Supports.

1) Design of base slab.

Area of base slab

Given reaction = ~~10KN~~ 50 KN

Factored Reaction = 750 KN

For M-20 concerete, $f_{ck} = 20 N/mm^2$

Bearing Capacity of concrete = 45% of f_{ck}

$= 0.45 * 20$

$= 9\ N/mm^2$

Area of base slab $= \frac{\text{load}}{0.45 f_{ck}} = \frac{75\times10^3}{0.45 * 20}$

$\boxed{\text{Area} = 8333.33\ mm^2}$

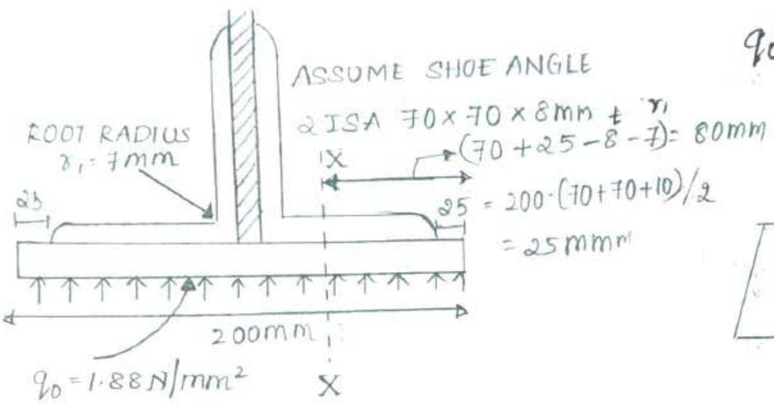
Providing square plate.

size of plate $= \sqrt{8.33 \times 10^3}$

$= 91.28\ mm$

$\boxed{\therefore \text{ Provide minimum } 200mm \times 200mm}$

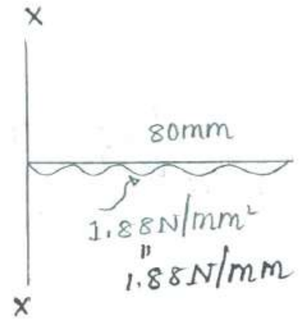
Thickness of Base plate (depends upon B.M)



$q_0 = \frac{Load}{Area}$

$= \frac{75 \times 10^3}{200 \times 200}$

$\boxed{q_0 = 1.875\ N/mm^2 \simeq 1.88\ N/mm^2}$



consider 1m strip

$= 1.88\ N/mm^2 \times 1mm$

$= 1.88\ N/mm$

Moment along $x-x$.

$(M_{x-x}) = 1.88 \times 80 \times 80/2$

$\boxed{M_x = 6016\ N\text{-}mm}$

[M_d = Pg.no 53]

Equating $M_x = M_d$

$6016\ N\text{-}mm = 1.2\ Z_e \frac{f_y}{\gamma_{m0}}$ [Pg no 53]

$6016 = 1.2 \times Z_e \times \frac{250}{\gamma_{m0}}$ where $Z_e = \frac{I}{y} = \frac{bd^3/12}{d/2}$

$6016 = 1.2 \times \frac{1 \times (t+8)^2}{6} \times \frac{250}{1.10}$

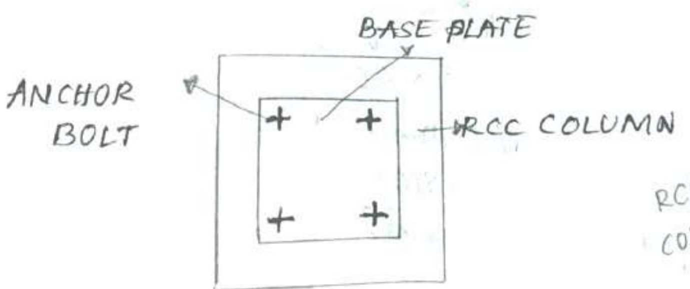
$Z_e = \frac{bd^2}{6}$

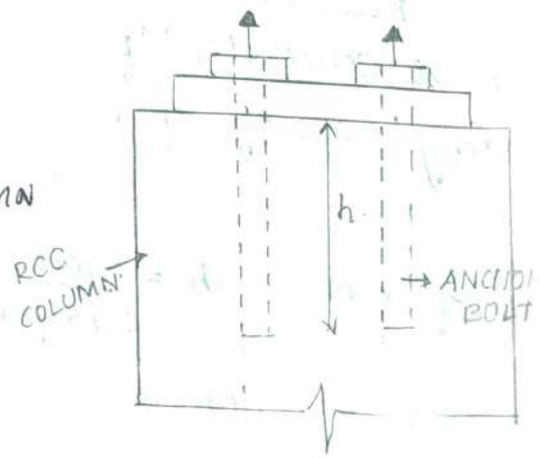
$\boxed{Z_e = \frac{1mm \times (t+8)^2}{6}}$

$\boxed{t = 3.50\ mm \simeq \text{ Say } t = 5mm}$

$\boxed{\therefore \text{ Provide base slab } 200 \times 200 \times 5mm}$

b) Anchor Bolt





Given uplift force = 15 KN

$\therefore$ Force in bolt $= \frac{15}{4} = \boxed{3.75\ KN}$

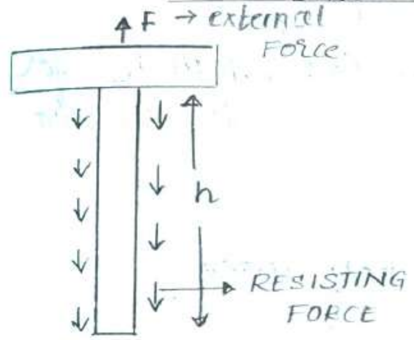
[There are 4 bolts at each end

$\therefore$ Ultimate Force $= 1.5 \times 3.75$

$\boxed{= 5.63\ KN}$

From IS 456 for M-20 concrete

$\boxed{(\tau_{bd})_{\text{Bond stress}} = 1.2\ N/mm^2 \times 1.60}$ [Pg no. 42 & 43]



Equating, External force = Resisting force

5.63 KN = Circumference of Bolt × height × Bond stress

$5.63 \times 10^3 N = \pi D \times h \times \tau_{bd}$

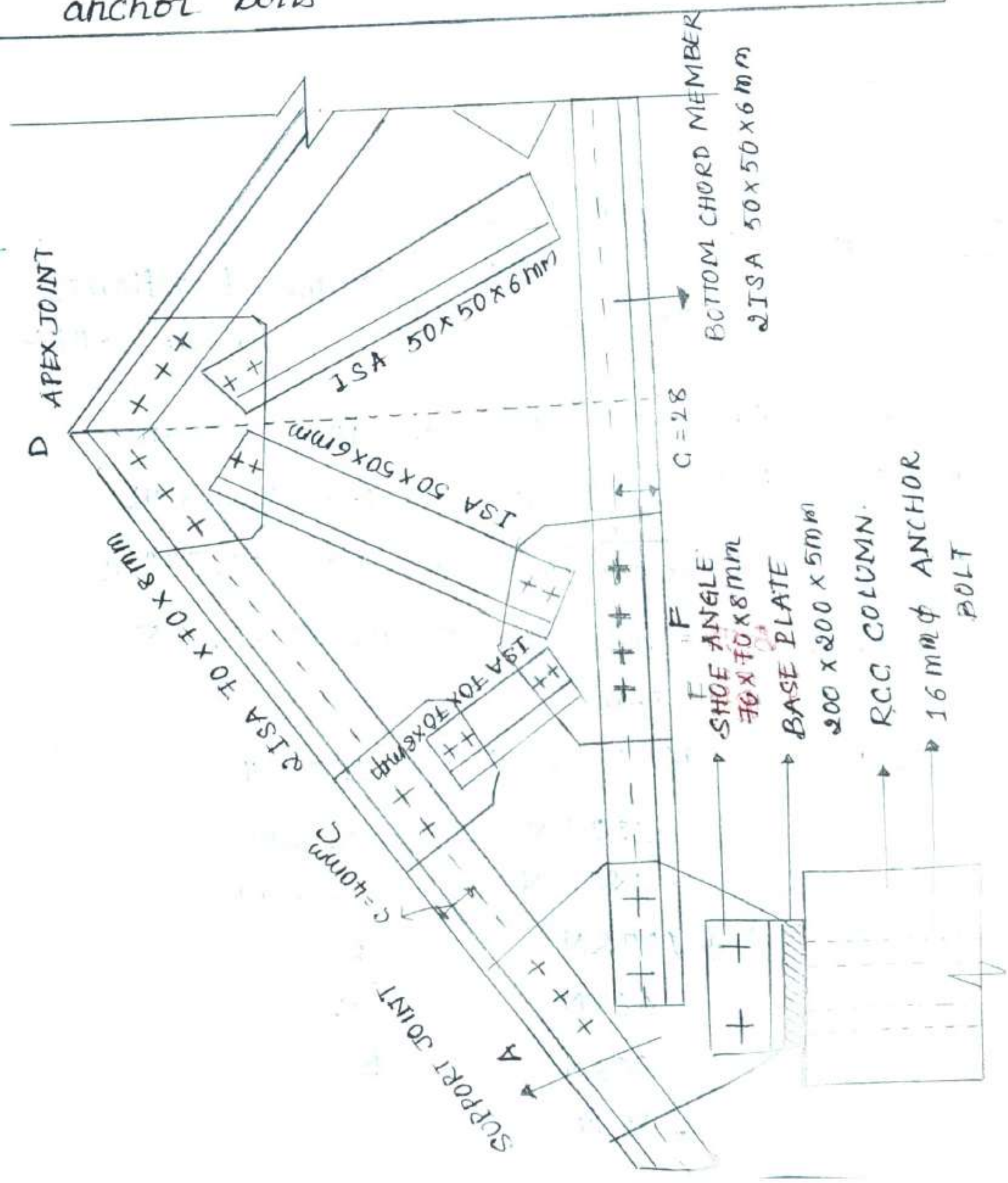
$5.63 \times 10^3 = \pi (16) \times h \times 1.92$

$h = 58.33$ say

$\boxed{h = 60mm}$

Assuming 16mm ϕ anchor bolts

$\boxed{\text{Hence provide @ each end } 4 - 16\ mm\ \phi,\ 60mm\ \text{Length anchor bolts}}$



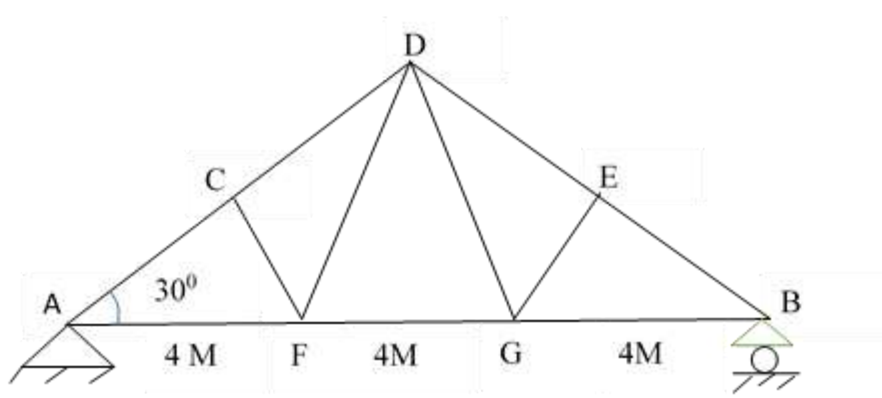
ROOF TRUSS TYPE 2

1. Force in a members as dead load and live load and also wind load is given below. Take tension as positive and compression as negative. Design the truss and support, given upward reaction at support is equal to 180 KN. Uplift pressure 50 KN. Use M-16 bolt for connection.

   Draw the sketches of the following

   a. Half elevation of the truss
   b. Enlarged view of apex joint
   c. Enlarged view of end joint & Intermediate joint.

Member	Dead load (KN)	Live load(KN)	Wind load (KN)	Length (m)
AC	+9.4	-30	+50.4	3.46
CD	-15.7	-23.8	+43.4	3.46
CF	-6.93	-10.4	+19.9	2
DF	+3.74	+5.64	-11.4	4
AF	+17.35	+26.4	-42.8	4
FG	+10.39	+15.8	-21.5	4



Soln:

1. Load Calculation [DL + LL and DL + WL]

Member	Dead load + Live load (KN)	Dead Load + Wind Load (KN)	Final Design Load (KN)	Length (m)
AC	-20.6	+59.80	Top chord +59.80 (T) -39.5(C)	3.46
CD	-39.5	+27.7		
CF	-17.33	+12.97	Inner Member -17.33 (C) +12.97 (T)	4
DF	+9.38	-7.60		
AF	+43.75	-25.45	Bottom Chord +43.75 (T) -25.45 (C)	4
FG	+26.19	-11.11		

2. Design of Top Chord Member:

After the load calculation, select the maximum value

Tension force = 59.8 KN , Factored Force = 1.5 * 59.8 = 89.70 KN

Compressive Force = 39.5 KN, Factored Force = 1.5 * 39.5 = 59.25 KN

Maximum length = L = 3.46 m

∴ Effective length = L_e = 0.85 *3.46 = 2.941 m = 2941 mm

Since tension force is more than compressive force, start the design like a tension member and then check for compressive force.

a. Design of Tension Member:

Using $T_{dn} = \frac{\alpha \ An\, fu}{\gamma_{ml}}$ ………… Page 33 IS 800

Here T_{dn} = Factored Load = 89.70 x 10^3 N

$\alpha = 0.6 \ and \ \gamma_{ml} = 1.25$

89.70 x 10^3 = $\frac{0.6 * An * 410}{1.25}$

$\therefore$ An = 455.792 mm^2

Increase the area approximately by 30 %

V_{dsf} Gross area Ag = 1.3 * 455.792 = 592.54 mm^2

From Steel table, Select double angle

Taking minimum size that is 2ISA 50 x 50 x 6 mm

Area = 1136 mm^2

Also r_{xx} = 15.1 mm and r_{yy} = 24.6 mm (For a gap of 10 mm)

b. Design of Connections:

Given M 16 bolts, Assume grade 8.8 HSFG bolts

$V_{dsf} = \frac{1}{\gamma_{mf}} [\mu_f \, n_e \, K_h \, F_o]$, μ_f = 0.55, n_e = 2, K_h = 1 and

F_o =0.7 *fu * An = 0.7*800*$\frac{\pi * 16^2}{4}$ = 87.82 x 10^3 N

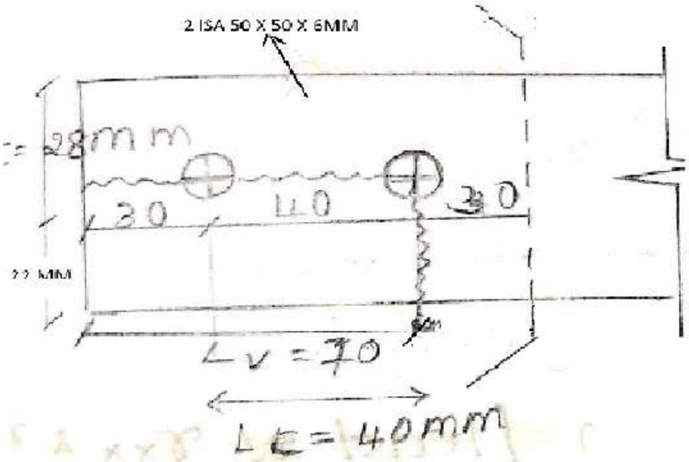
$V_{dsf} = \frac{1}{1.25}$ [0.55 * 2 * 1 *87.823 x10^3]

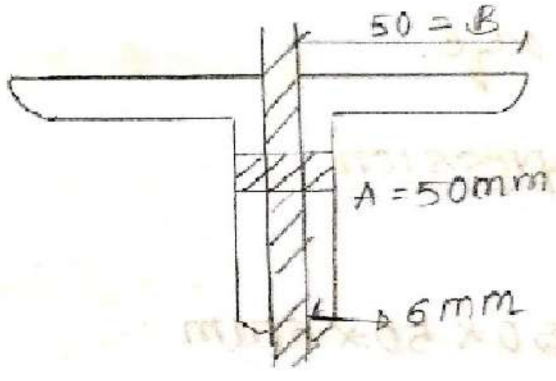
V_{dsf} = 77.284 KN

Therefore Bolt Value = 77.284 KN

No. of Bolts = $\frac{Force}{Bolt\ Value}$ = $\frac{89.71}{77.284}$ = 1.16 Say 2 nos

$\therefore$ Number of bolts = 2 nos.





iii] Check for Rupture

$W = 50mm$; $t = 6mm$; $W_1 = 28mm$; $L_c = 40mm$; $L_t = 22mm$

$L_v = 70mm$. $b_s = W + (W_1 - t)$

$= 50 + (28 - 6)$

$\boxed{b_s = 72\ mm}$

$\beta = 1.4 - 0.076 \left[\frac{L_v}{t}\right] \left[\frac{f_y}{f_u}\right] \left[\frac{b}{L_c}\right]$

$= 1.4 - 0.076 \left[\frac{70}{6}\right] \left[\frac{250}{410}\right] \left[\frac{72}{40}\right]$

$\boxed{\beta = 0.704}$;

$A_{go} = (B - t/2) * t = [50 - 6/2] * 6 = 282mm^2 = A_{go}$

$A_{nc} = (A - d_0 - t/2) * t = [50 - 18 - 6/2] * 6 = A_{nc} = 134 mm^2$

$T_{dn} = \left[\frac{0.9 A_{nc} f_u}{\gamma_{mL}} + \frac{\beta A_{go} f_y}{\gamma_{m0}}\right] * 2$

$= \left[\frac{0.9 * 134 * 410}{1.25} + \frac{0.704 \times 282 * 250}{1.10}\right] * 2$

$\boxed{T_{dn} = 192.96\ kN > 89.70\ kN}$

iv] Check for Block shear.

$A_{vg} = L_v * t = 70 * 6 = A_{vg} = 420 mm^2$

$A_{vn} = (420 - 1.5 * 18 * 6) = A_{vn} = 258 mm^2$

$A_{tg} = L_t * t = 22 * 6 = A_{tg} = 132 mm^2$

$A_{tn} = [132 - 0.5 * 18 * 6] = A_{tn} = 78 mm^2$

$\therefore T_{db1} = 2 * \left[\frac{A_{vg} f_y}{\sqrt{3} * \gamma_{m0}} + \frac{0.9 A_{tn} f_u}{\gamma_{m1}}\right]$

$\boxed{T_{db1} = 156.27\ kN > 89.70\ kN}$

$$T_{db_2} = 2 * \left[\frac{0.9\ A_{vn}\ f_u}{\sqrt{3}\ \gamma_{ml}} + \frac{A_{tg}\ f_y}{\gamma_{mo}} \right]$$

$$T_{db_2} = 147.74\ KN > 89.70\ KN$$

Hence safe.

V] Check for compression

Force = 59.25 KN

Knowing 2ISA 50 × 50 × 6mm

Area = 1136 mm² ; r_{min} = 15.1 mm [least of r_{xx} & r_{yy}]

l_e = 2941 mm

∴ Slenderness Ratio, $\lambda = \frac{l_e}{r_{min}} = \frac{2941}{15.1}$

$$\lambda = 194.76$$

From table 9.c [Pg. no. 92]

for 190 → 39.70

200 → 36.30

fcd for 194.76 → 38.08 N/mm²

$$f_{cd} = 38.08\ N/mm^2$$

∴ Design compressive strength

$P_d = A_c * f_{cd}$

= 1136 * 38.08

$$P_d = 43.25\ KN \not> 59.25\ KN \quad \therefore \text{Unsafe}$$

Hence unsafe under compression

Hence revise section with more area.

∴ try 2ISA 60 × 60 × 8

Area = 1792 mm²

r_{xx} = 18 mm ; r_{yy} = 28.9 mm

$\lambda = \frac{l_e}{r_{min}} = \frac{2941}{18} \Rightarrow \boxed{\lambda = 163.39}$

From table 9(c) table 42

160 →

170 →

For 163.39 = 51.53 N/mm²

$$f_{cd} = 51.53\ N/mm^2$$

$P_d = A_c * f_{cd} = 1792 * 51.53$

$$P_d = 92.34\ KN > 59.25\ KN \quad \text{Hence Safe}$$

Hence adopt 2ISA 60×60×8 mm for top chord member

3] Design of bottom chord member

Max tension force = 43.75 ⇒ $\boxed{\text{Factored force} = 65.625\,kN}$

compress. force = 25.45 ⇒ $\boxed{\text{Factored force} = 38.175\,kN}$

Max length = 4m

$L_e = 0.85 \times 4 \Rightarrow \boxed{L_e = 3.4\,m = 3400\,mm}$

Since tension force is more than compressive force, start the design like a tension member & then check for compression force.

i] Design of tension member : Selection of section

$$T_{dn} = \frac{\alpha A_n f_u}{\gamma_{m1}} \qquad \alpha = 0.6$$

$$65.625 \times 10^3 = \frac{0.6\, A_n \times 410}{1.25}$$

$$\boxed{A_n = 333.46\,mm^2}$$

Increase approximately by 30%.

$\therefore$ Gross Area = 1.3×333.46

$$\boxed{(\text{Area})_{Gross} = 433.498\,mm^2}$$

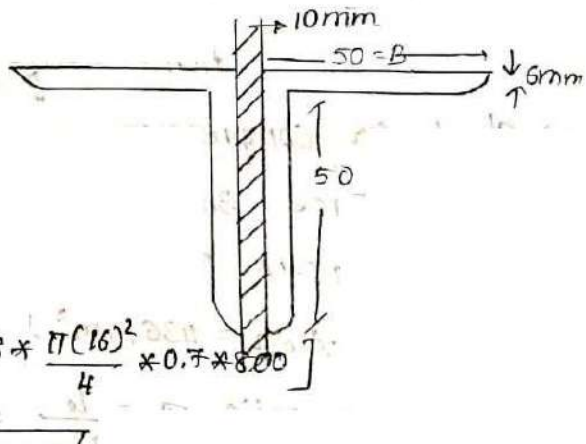
From steel table, select double angle.

Taking min size. i.e., 2ISA 50×50×6 mm

$r_{xx} = 15.1\,mm$, $r_{yy} = 24.0\,mm$ [gap 10]

Area = $1136\,mm^2$ $\qquad r_{min} = 15.1\,mm$

ii] Connection



Given M-16 bolts

Assume Grade 8.8 HSFG bolts

$$V_{dsf} = \frac{1}{\gamma_{mf}}\left[\mu_f\, n_e\, k_h\, F_o\right]$$

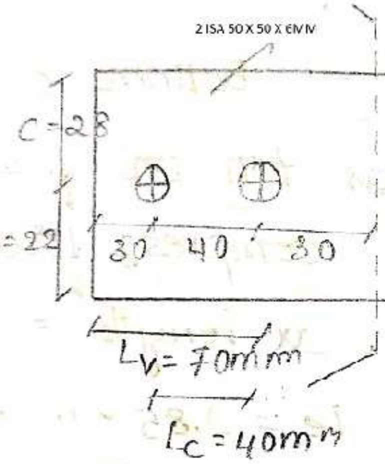
$$= \frac{1}{1.25}\left[0.55 \times 2 \times 1 \times 0.78 \times \frac{\pi (16)^2}{4} \times 0.7 \times 800\right]$$

$$\boxed{V_{dsf} = 77.28\,KN > 65.625\,KN}$$

No of Bolts = $\dfrac{\text{Force}}{\text{Bolt value}} = \dfrac{65.625}{77.28} = 0.849 \simeq 2$

Provide min 2 no's of bolts

iii] Check for Rupture.



$W = 50mm$; $t = 6mm$; $L_t = 28mm$.

$L_c = 40mm$;

$b_s = W + (W - t) = 50 + (28 - 6) = 72mm$

$$\beta = 1.4 - 0.076\left[\frac{W}{t}\right]\left[\frac{f_y}{f_u}\right]\left[\frac{b_s}{L_c}\right]$$

$$= 1.4 - 0.076\left[\frac{50}{6}\right]\left[\frac{4250}{410}\right]\left[\frac{72}{40}\right]$$

$\boxed{\beta = 0.704}$

$A_{go} = (B - t/_2) * t = (50 - 6/_2) * 6 \Rightarrow \boxed{A_{go} = 28mm^2}$

$A_{nc} = (A - d_0 - t/_2) * t = (50 - 18/_1 - 6/_2) * 6 \Rightarrow \boxed{A_{nc} - 134 mm^2}$

$$T_{dn} = 2 * \left[\frac{A_{nc}\, 0.9 f_u}{\gamma_{ml}} + \frac{\beta A_{go} f_y}{\gamma_{mo}}\right]$$

$\boxed{T_{dn} = 192.96KN > 65.63KN}$; Hence.

iv) Check for Block shear

$L_v = 70mm$; $L_t = 22mm$, $t = 6mm$; $d_0 = 18mm$

$A_{vg} = l_v * t = 70 * 6 \Rightarrow A_{vg} = 420 mm^2$

$A_{vn} = 420 - 1.5 * 18 * 6 \Rightarrow A_{vn} = 258 mm^2$

$A_{tg} = l_t * t = 22 * 6 \Rightarrow A_{tg} = 132 mm^2$

$A_{tn} = 132 - 0.5 * 18 * 6 \Rightarrow 78 mm^2$

$$T_{db_1} = 2 * \left[\frac{A_{vg} f_y}{\sqrt{3} * \gamma_{mo}} + \frac{0.9 A_{vn} f_u}{\gamma_{m1}}\right] \qquad T_{db_2} = 2 * \left[\frac{0.9 A_{vn} f_u}{\sqrt{3}\ \gamma_{ml}} + \frac{A_{tg} f_y}{\gamma_{mo}}\right]$$

$\boxed{T_{db_1} = 156.73 KN > 65.63 KN}$ $\boxed{T_{db_2} = 147.94 KN > 65.63KN}$

Hence safe.

v) Check for compression.

Force = 38.18 KN

knowing 2ISA 50 * 50 * 6mm.

Area = $1136 mm^2$; $L_e = 3400 mm$; $r_{min} = 15.1 mm$

Slenderness ratio, $\lambda = \frac{L_e}{r_{min}} = \frac{3400}{15.1} \Rightarrow \boxed{f_{cd} = 29.41 N/mm^2}$

From [T - 9(C) Pg. 42]

220 → 30.6

230 → 28.3

For 225.16, $\boxed{f_{cd} = 29.41 N/mm^2}$

Design compression strength

$P_d = A_c * f_{cd}$

$= 1136 * 29.41 \Rightarrow \boxed{P_d = 33.40\ KN < 38.18\ KN}$

Hence under compression, it is unsafe. Revise the section with more area, Try 2ISA 60 * 60 * 8mm.

Area = $1792 mm^2$; r_{min} = 18mm

$\lambda = \frac{L_e}{r_{min}} = \frac{3400}{18} = 188.88$

180 → 43.6

190 → 39.7

$\boxed{\therefore f_{cd} = 40.17 N/mm^2}$

$P_d = A_c * f_{cd} \Rightarrow 1136 * 40.17 \Rightarrow \boxed{P_d = 71.98\ KN > 38.18\ KN}$.

Hence safe.

Hence adopt 2ISA 60 * 60 * 8mm for Bottom chord member

4] Design of Inner Member

Tensile Force = ·12.97 ⇒ F.F = 19.455 KN

compress Force = −17.33 ⇒ F.F = 25.995 KN

Max Length = 4m ⇒ L_e = 3400mm.

Since compressive force is more than tensile force, start the design like a compression member & then check for tensile force

i] Selection of Section

Assume f_{cd} = 50 N/mm^2

Using $P_d = A_c f_{cd}$

$26 * 10^3 = A_c * 50$

$A_c = 1733 mm^2$

from steel table select single angle. try ISA (60 * 60 * 8)mm

∴ Area = $896 mm^2$

$r_x = r_y$ = 18mm ; r_{uu} = 22.7mm ; r_{vv} = 11.5mm

ii] f_{cd} Calculation

l = 3400 mm = L_e

$b_1 = b_2$ = 60mm (outstanding & connection leg)

thickness = 8mm

r_{min} = 11.5 mm

Design Compressive stress

$$f_{cd} = \frac{f_y/\gamma_{m0}}{\phi + [\phi^2 - \lambda_e^2]^{0.5}}$$

$$\lambda_e = \sqrt{K_1 + K_2 \lambda_{vv}^2 + K_3 \lambda \phi^2}$$

Taking no of bolts ≥ 2 & fixed condition

$\therefore$ from table 12, Pg. 48

$K_1 = 0.2$; $K_2 = 0.35$; $K_3 = 20$

$$\Rightarrow \lambda_{vv} = \frac{l/r_{vv}}{\epsilon\sqrt{\frac{\pi^2 E}{250}}} = \frac{3400/11.5}{1 * \sqrt{\frac{\pi^2 \times 2 * 10^5}{250}}} \Rightarrow \lambda_{vv} = 3.33$$

$$\Rightarrow \lambda\phi = \frac{(b_1+b_2)/2t}{\epsilon\sqrt{\frac{\pi^2 E}{250}}} = \frac{(60+60)/(2*8)}{1 * \sqrt{\frac{\pi^2 * 2 * 10^5}{250}}} \Rightarrow \lambda\phi = 0.084$$

$$\lambda_e = \sqrt{0.2 + (0.35 * (3.33)^2) + (20 * (0.084)^2)}$$

$\lambda_e = 2.05$

From IS 800 page no 34

$$\phi = 0.5 [1 + \alpha(\lambda_e - 0.2) + \lambda_e^2]$$

taking $\alpha = 0.49$ Pg. 35 for buckling class c.

$$\phi = 0.5 [1 + 0.49 (2.05 - 0.2) + 2.05^2]$$

$\phi = 3.05$

$$\Rightarrow f_{cd} = \frac{f_y/\gamma_{m0}}{\phi + (\phi^2 - \lambda_e^2)^{0.5}} \Rightarrow f_{cd} = 42.82 \ N/m^2$$

$\therefore$ Design compressive force (strength)

$P_d = f_{cd} * A_c = 896 * 42.82$

$P_d = 38.36$ KN > 26 KN ; Hence safe

iii) Connections

Using M16 - Grade 8.6 HSFG bolts

$$V_{dsf} = 1/\gamma_{mf} [\mu_f \, n_e \, K_h \, F_o]$$

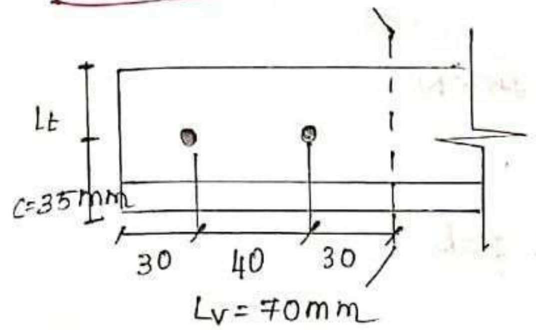
$$= 1/1.25 [0.55 * 2 * 1 * 0.78 * \pi * \frac{16^2}{4} * 0.7 \ 800]$$

$V_{dsf} = 38.64\,kN$

$$\text{No of bolts} = \frac{\text{Force}}{\text{Bolt value}} = \frac{26 \times 10^3}{38.64 \times 10^3} = 0.672$$

Hence, provide min bolts = 2

check for Tension



Sp-6 for c page in (8)

$L_t = 25\,mm$

$L_v = 70\,mm$

$L_c = 40\,mm$

$C_1 = W = 28$

$$\beta = 1.4 - 0.076\left[\frac{W}{t}\right]\left[\frac{f_y}{f_u}\right]\left[\frac{b_s}{L_c}\right]$$

$$= 1.4 - 0.076\left[\frac{60}{8}\right]\left[\frac{250}{410}\right]\left[\frac{80}{40}\right]$$

$\beta = 0.704$

$b_s = W + W_1 - t$

$= 60 + 28 - 8$

$b_s = 80\,mm$

$\Rightarrow A_{nc} = (A - D_0 - t/2) * t$

$= (60 - 18 - 8/2) * 8$

$A_{nc} = 304\,mm^2$

$\Rightarrow A_{go} = (B - t/2) * t$

$= (60 - 8/2) * 8$

$A_{go} = 448\,mm^2$

$$T_{dn} = \left[\frac{0.9 * 304 * 410}{1.25} + 0.704 * 448 * \frac{250}{1.1}\right]$$

$T_{dn} = 191.55 * 10^3\,N > 26\,kN$

Check for Block shear

$L_v = 70$; $L_t = 25$; $d_o = 18$; $t = 8$

$A_{vg} = L_v * t = 560\,mm^2$

$A_{vn} = 560 - (1.5 * 18 * 8) = 344\,mm^2$

$A_{tg} = L_t * t = 200\,mm^2$

$A_{tn} = 200 - (0.5 * 18 * 8) = 120\,mm^2$

$$T_{db1} = \left[\frac{A_{vg} f_y}{\sqrt{3}\gamma_{m0}} + \frac{0.9 A_{tn} f_u}{\gamma_{m1}}\right] = \left[\frac{560 * 250}{\sqrt{3} * 1.10} + \frac{0.9 * 410 * 120}{1.25}\right]$$

$T_{db1} = 111.26\,kN$

$$T_{db_2} = \left[\frac{0.9 A_{vn} f_u}{\sqrt{3}\,\gamma_{m1}} + \frac{A_{tg} f_y}{\gamma_{m0}}\right] = \left[\frac{0.9 * 410 * 344}{\sqrt{3} * 1.25} + \frac{200 * 250}{1.10}\right]$$

$$T_{db_2} = 104.08 \text{ KN} > 26 * 10^3 N,$$

Hence safe

Design of supports

a) Design of Base slab

Given, reaction = 180 KN

factored Load = 180 x 1.5 = 270 KN

For M-20 concrete, $f_{ck} = 20 \text{ N/mm}^2$;

Bearing Capacity = 0.45 % f_{ck}

= 0.45 * 20

= 9 N/mm2

Area of Base slab = $\frac{\text{Load}}{0.45 f_{ck}} = \frac{75}{9} = 8.3 \text{ mm}^2$

⇒ Providing square plate, size of plate = $\sqrt{\text{Area}}$

$= \sqrt{8.33 * 10^3}$

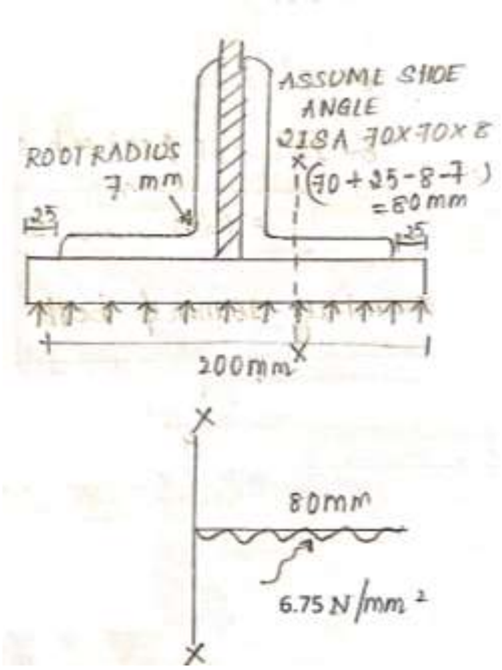
= 91.28 mm

Provide minimum (200 * 200) mm

b. Thickness of Base Plate:

$$q_o = \frac{Load}{Area} = \frac{270 * 10\hat{}3}{200\ x\ 200} \qquad = 6.75\ mm^2$$

Considering 1 m strip, q_o / meter = 2.25 x 1 = 6.75 N/mm



Moment along x -x

$M_{x\text{-}x}$ = 6.75 *80*80/2 = 21,600 N-mm

Equating $M_{x\text{-}x} = M_d$

$$= 1.2\ Ze \frac{fy}{\gamma mo} \text{ .... Page 53 IS – 800}$$

$$21{,}600 = 1.2\ Ze \frac{250}{1.10}$$

Where Ze = $\frac{I}{y}$ = $(bd^3/12)/(d/2)$

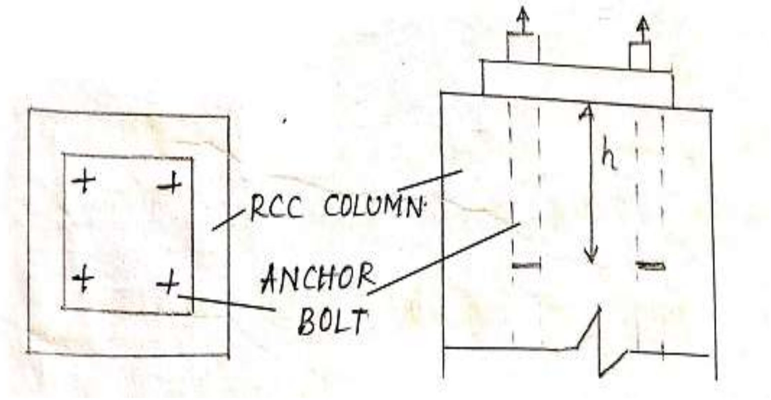
$= bd^2/6 = (1mm * (t+8)^2)/6$

$$21600 = 1.2\ (1mm * (t+8)^2)/6 * \frac{250}{1.10}$$

t = ……… mm . … . mm

∴ Provide base slab 200 mm x 200mm x ………mm

c. Anchor Bolts:



Given uplift force = 50 KN

Provide 4 bolts at each end.

∴ Force in each bolt = 50/4 = 12.5 KN

Ultimate force = 1.4 * 4 = 18.75 KN

From IS 456, for M20 Concrete

$(\tau_{bd})_{\text{bond stress}}$ = 1.2 N/mm^2 * 1.60 …….Page No. 43

Equating external force = Resisting force

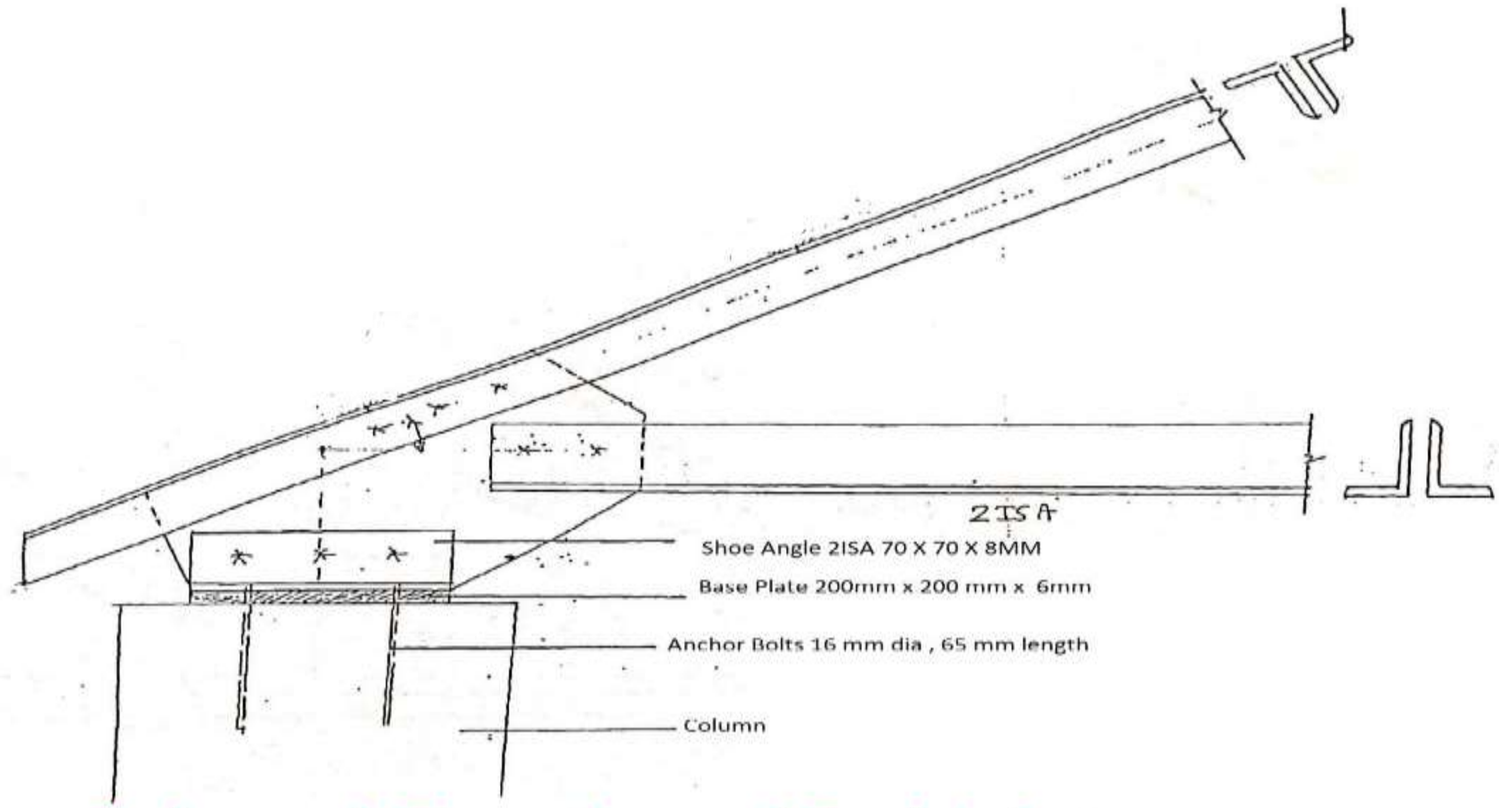
18.75 KN = Circumference of bolt * Height * Bond Stress

$18.75 = \pi D * h * \tau_{bd}$

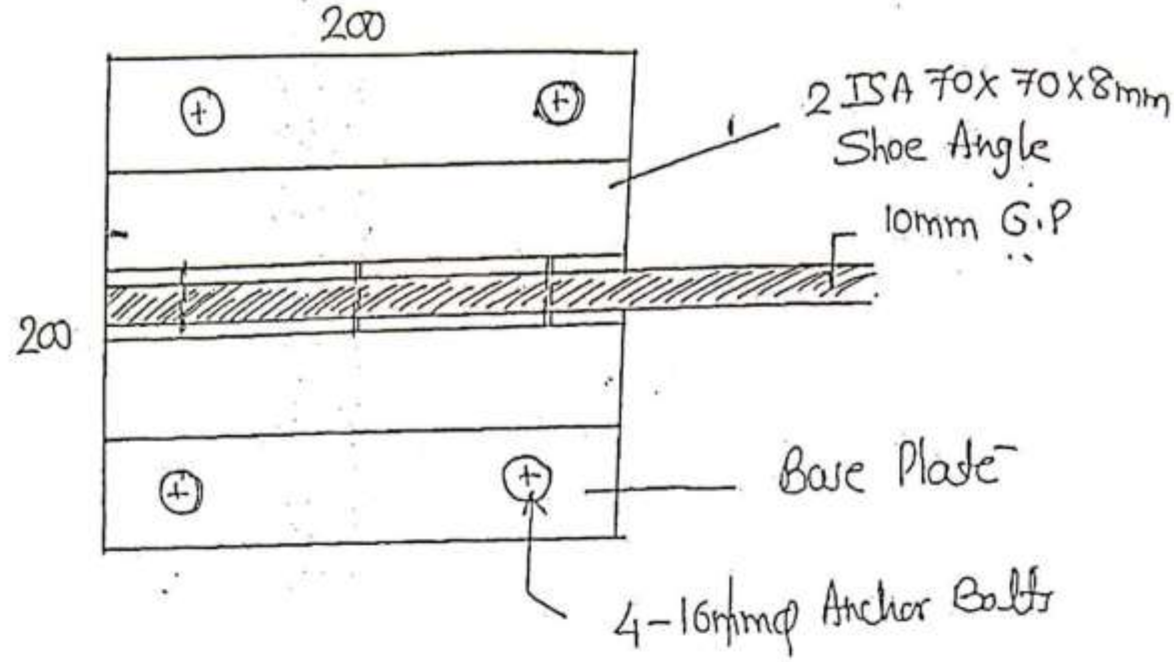
$18.75 * 10^3 = \pi * 16 * h * 1.2 * 1.6$

h = ……… mm $\approx \cdots \ldots \ldots . mm$

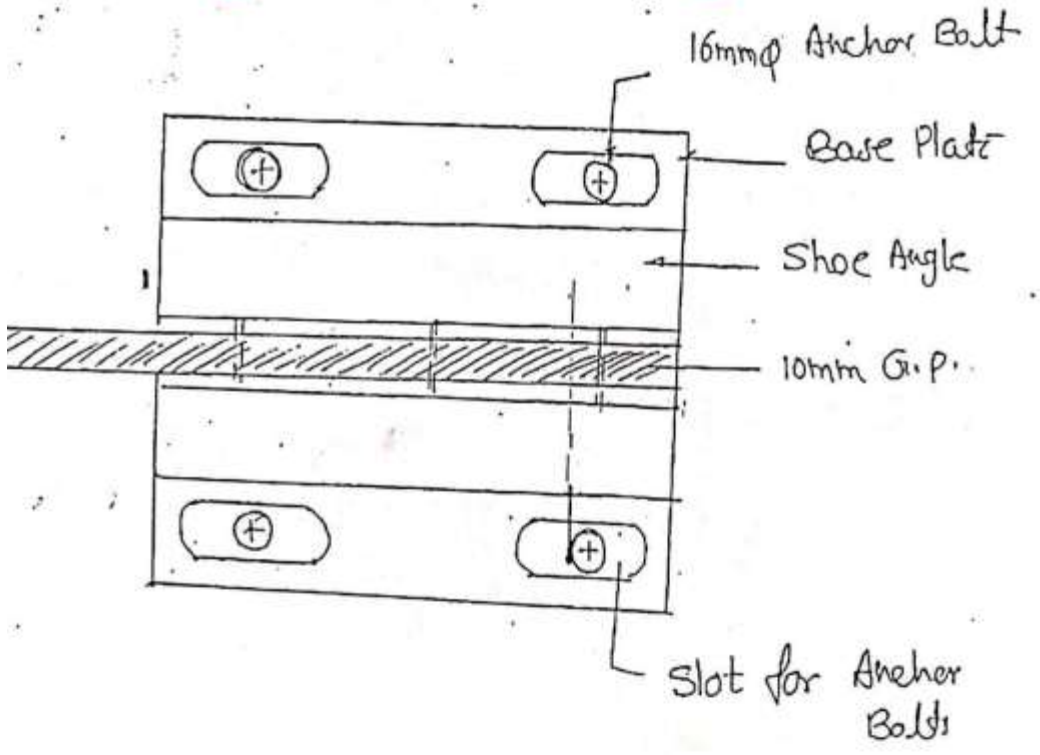
Hence Provide at each end 4-16mm dia,…….. mm length anchor bolts



Enlarged Elevation of End Joint



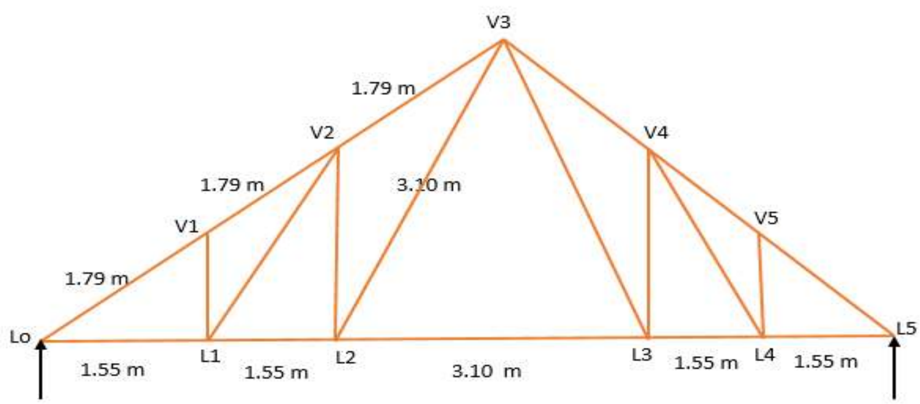
EnLarged View of Hinged End



Enlarged View of Roller End

Roof Truss type 2 – 2nd Problem

2. **The forces in the members of the roof truss of an industrial building are as shown in table. The truss is supported on 400mm thick masonry. End reaction due to dead load + live load is 10.556 KN. Members are to be connected at the joints with 16 mm dia bolts and 8mm thick gusset plate. Design the members and base plate. Assume permissible bearing pressure on masonry = 0.8 Kn/mm^2 and size of the Shoe angle is 2ISA 75 x 75 x 6mm on each side of the gusset plate.**



Members	Force	
	Compression (-ve) KN	Tension (+ve) KN
LoV1,V1V2,V2V3	-17.4	20.9
LoL1, L1L2, L2L3	-14	14.9
V3L2	-8.7	6
V2L2	-5.3	7.4
V2L1	-6.7	4.6
V1L1	-3.5	5

Soln:

1. Load Calculation:

Members	Force		Final Design Force	Length in m
	Compression (-ve) KN	Tension (+ve) KN		
LoV1,V1V2,V2V3	-17.4	20.9	Top Chord +20.9 (T) -17.4 (C)	1.79
LoL1, L1L2, L2L3	-14	14.9	Bottom Chord +14.9 (T) -14 (C)	3.10
V3L2	-8.7	6	Inner Member +8.7 (C) -7.4 (T)	3.10
V2L2	-5.3	7.4		
V2L1	-6.7	4.6		
V1L1	-3.5	5		

2. Design Top Chord Member:

Since tension force is more than compressive force, Design the member as tension member and check for Compressive load Carrying Capacity

3. Desing of Bottom Chord Member:

Since tension force is more than compressive force, Design the member as tension member and check for Compressive load Carrying Capacity

4. Desing of Inner Member:

Since Compressive force is more than tension force, Design the member as Compressive member and check for Tension load Carrying Capacity

5. Design of Base Plate: (Design of Support)

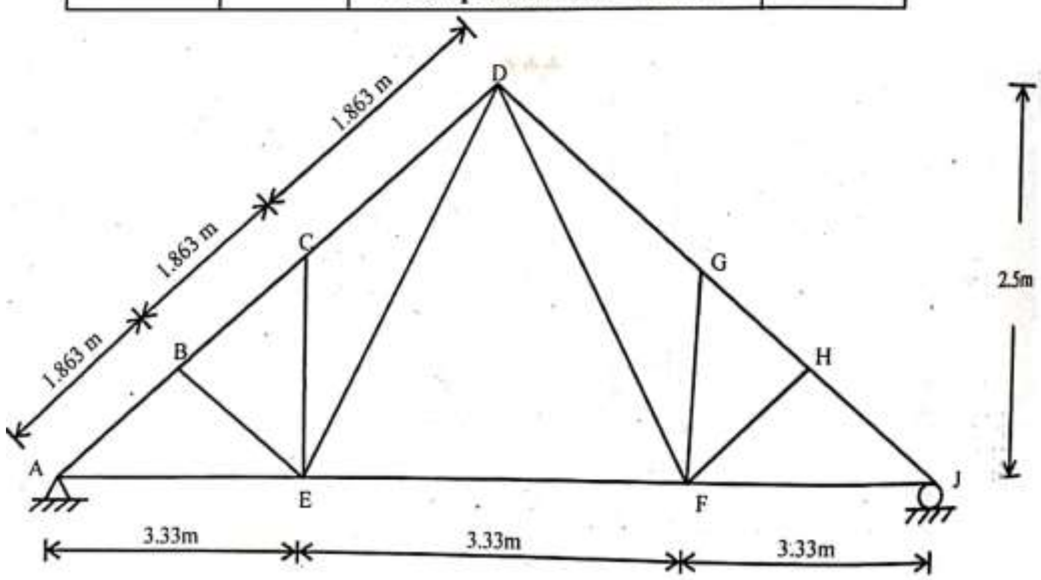
a. Area of Base Plate

b. Thickness of Base Plate

c . Anchor Bolts

3. Design a roof truss shown in figure. The forces in the members of the truss due to dead load, live load and wind load are tabulated below.

Member	DL (kN)	LL (kN)	WL (kN)	DL+LL (kN)	DL+WL (kN)
AB	+14.37	+21.80	−37.32	+36.17	−22.95
BC	+11.64	+17.60	−32.08	+29.24	−20.44
CD	+12.05	+18.26	−35.90	+30.31	−23.85
DE	−5.13	−7.70	+14.70	−12.83	+9.57
EC	+2.77	+4.18	−8.42	+6.95	−5.65
EB	+2.77	+4.18	−9.15	+6.95	−6.38
EA	−12.85	−19.36	+31.69	−32.21	+18.84
EF	−7.69	−11.61	+15.63	−19.30	+7.94
		+ Compression and − Tension			



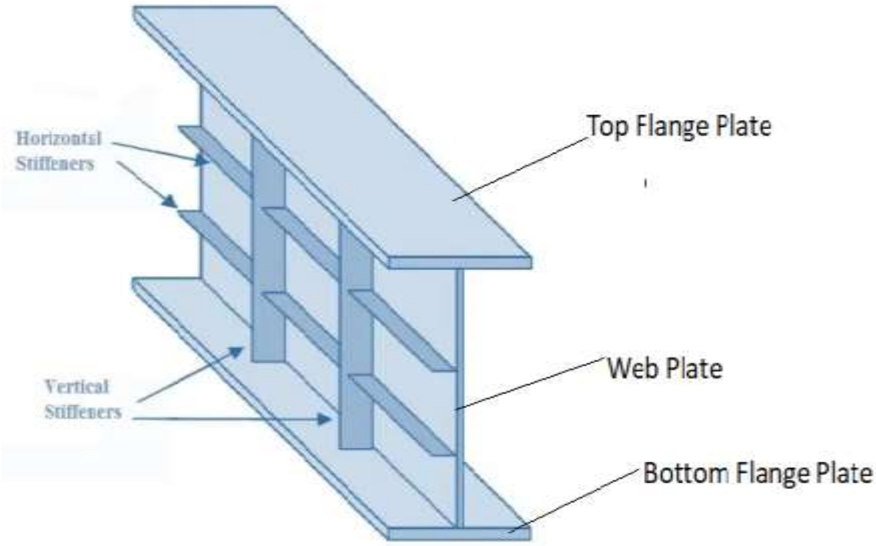
Soln:

1. Load Calculation:

Members	DL + LL (KN)	DL+WL (KN)	Design Force (KN)	Length in M
AB	+36.17	-22.95	Top Chord +36.17 (C) -23.85 (T)	1.863
BC	+29.24	-20.44		
CD	+30.31	-23.85		
DE	-12.83	+9.57	Inner Member -12.83 (C) +9.57 (T)	3.33
EC	+6.95	-5.65		
EB	+6.95	-6.38		
EA	-32.21	+18.84	Bottom Chord - 32.21 (C) + 18.84 (T)	3.33
EF	-19.30	+7.94		

2. Design of Top Chord Member
3. Design of Bottom Chord Member
4. Design of Inner Member

Welded Plate Girder and its Components:



Module 2

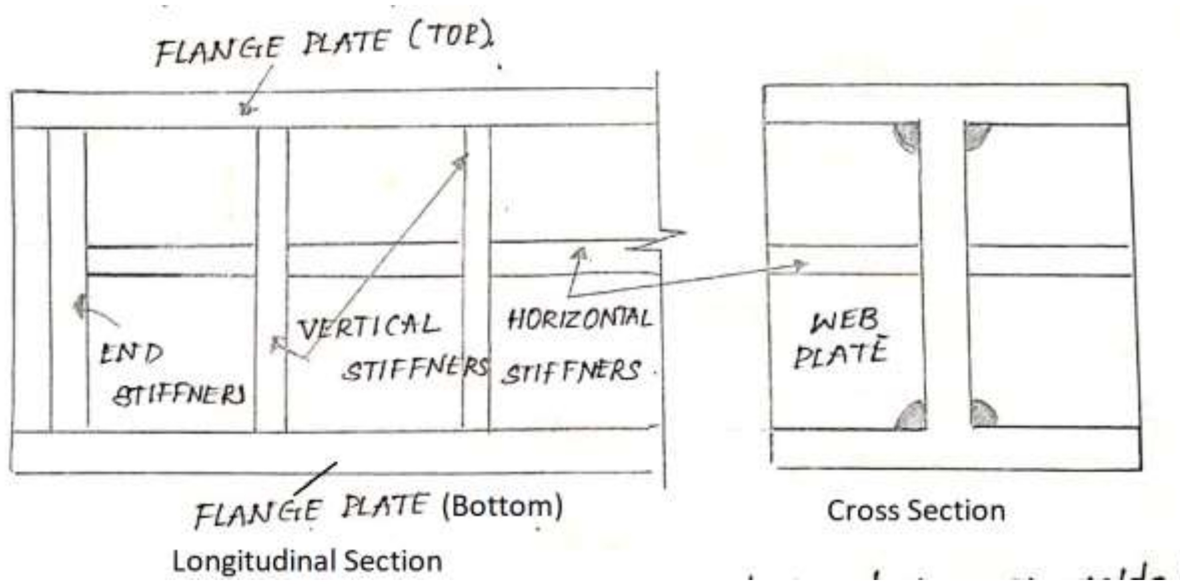
Design of Welded Plate Girder

Plate girders are deep built-up beams used in bridges, building and industrial structure. For heavy loads and long spans, plate girders are substituted for rolled beams. A Plate girder consists of web plate with stiffeners if required and top & bottom flanges.

Components of Welded Plate Girders:

Following are the various components of plate girder as shown in the figure.

1. Flange plates: Top and Bottom Plates to take the bending moment.
2. Web Plate: To take the shear force.
3. Vertical or Transverse Stiffeners: Provided along the span to increase web buckling strength.
4. Horizontal Stiffeners or Longitudinal stiffeners: Provided in areas of very high moments.
5. End or Bearing Stiffeners: Provided at Concentrated loads and reactions points to transfer the loads.
6. Splices: They are provided if necessary continuity required in the web & flanges.



Following are the different steps used in design of Welded plate Girder:

1. Design of mid span
    a. Load Calculation
    b. Girder Dimensions
        i. Web Depth
        ii. Web Thickness
        iii. Flange width
        iv. Flange Thickness
    c. Check for moment of resistance
    d. Check for Shear
    e. Welded connection between flange and Web
2. Curtailment of flange plate
3. Design of Intermediate Stiffeners (IS)
4. Design of End Bearing Stiffeners (EVS)

DESIGN OF WELDED PLATE GIRDER.

Plate Girders are deep built-up beams used in bridges, building & industrial structure. For heavy loads & long spans, plate girders are substituted for rolled beams. A plate girder consists of web plate with stiffners if required & top & bottom flanges.

Components of Welded Plate Girder.

Following are the various components of plate girder as shown in figure

i) Flange plates - top & bottom. to take the bending moment

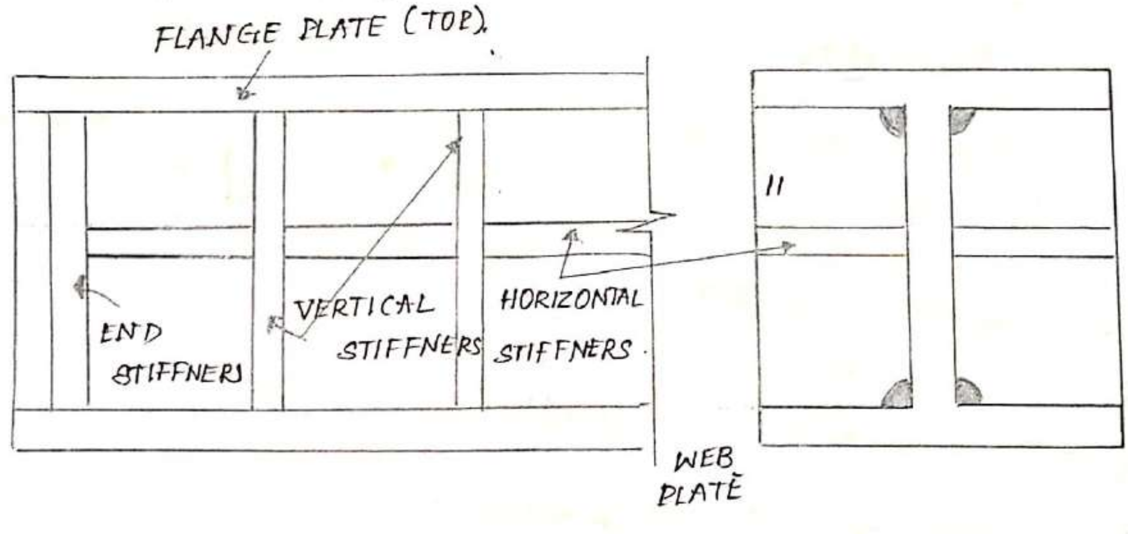
ii) Web plate - to take the shear force.

iii) Verticle or transverse stiffnerss - provided along the span to increase web buckling strength.

iv) Horizontal stiffners or longitudinal stiffners. - provided in areas of very high moments.

v) End or Bearing stiffnerss - Provided at concentrated loads and reactions points to transver the loads.

vi) Splices - These are used to ? - they provided the necessary continuity required in the web & flanges.



Following are the different steps used in design of welded Plate Girder.

1) Design of midspan

a) Load Calculation b) Girder Dimensions

i) Web depth
ii) Web thickness
iii) Flange width
iv) Flange Thickness

c] check for moment of resistance.

d] Check for shear

e] Welded connection between flange & web.

2] Curtailment of flange plate.

3] Design of intermediate stiffners (IS)

4] Design of end bearing stiffners (EVS)

Problems.

1] Design a welded plate girder for an effective span of 18m to support a UDL of 60 KN/m in addition to a pair of point loads of magnitude 600 KN each at 1/3 span. Design the central section (midspan), bearing stiffners, intermediate stiffners, their connection, curtailment of flange plate.

Draw sketches of i) Half - longitudinal section.
ii) Cross - section at centre & support.
iii) Sectional plan support bearing stiffner to an enlarged scale.

soln: 1) Design of midspan.

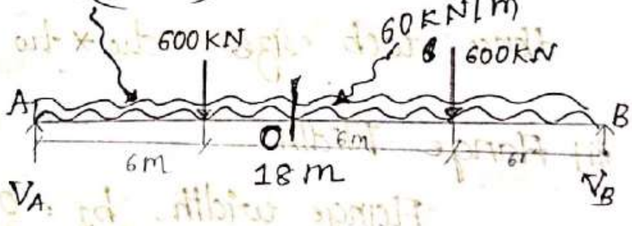
a] Load calculation.

Self Weight of the girder

$= \frac{\text{Live load}}{250}$

$= \frac{60 \times 18 + 2(600)}{250}$

$\boxed{\text{self Wt of Gr} = 9.12 \text{ KN/m} \simeq 10 \text{ KN/m}}$



Span, L_e = 18m
UDL = 60KN/m
Point load = 600KN @ 1/3 of span

Assume $f_y = 250$ N/mm²
$f_{yw} = 250$ N/mm²
$\gamma_{mo} = 1.10$
$\gamma_{mw} = 1.25$

Reaction $= V_A = V_B = \frac{(70 \times 18) + (2 \times 600)}{2}$

Shear force, $V_A = V = 1230$ KN

Ultimate shear force = 1845 KN

Maximum bending moment occurs at 0.

$\therefore\ M_0 = (1230 \times 9) - (70 \times 9 \times 9/2) - (600 \times 3)$

$M_0 = 6435$ KN-m.

$\boxed{\therefore \text{ Ultimate moment } \cdot M_u = 9652.5 \text{ KN-m}}$

b) Girder Dimension.

i) Depth of Web.

$d = d_w = \left[\dfrac{K M_u}{f_y}\right]^{0.33}$; Assume $K = 150$ to 200 & take $K = 150$

$= \left[\dfrac{(15 \times 9652.5 \times 10^6)}{250}\right]^{0.33}$

$d_w = 1666.176$ mm

$\boxed{\therefore \text{ Take, } d_w = d = 1700 \text{mm}}$

ii) Web Thickness

For elastic or compact condition [from table 2 Pg 18]

$\dfrac{d}{t_w} \leq 84\,\epsilon$ or $105\,\epsilon$

$d/t_w = 105\,\epsilon$, where ϵ = Yield stress ratio $= \left[\dfrac{250}{f_y}\right]^{1/2} = \left[\dfrac{250}{250}\right]^{1/2} = 1$.

$\dfrac{1700}{t_w} = 105 \times 1$

$t_w = 16.19 \text{ mm} \simeq 18 \text{mm}$

$\boxed{\text{Hence web size} = d_w \times t_w = 1700 \text{mm} \times 18 \text{mm}}$

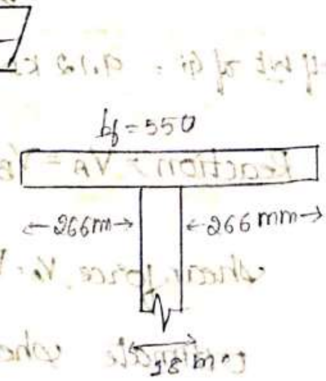
iii) Flange Width

Flange width, $b_f = 0.3\, d_w$

$= 0.3 \times 1700$

$\boxed{b_f = 510 \text{mm} \simeq 550 \text{mm}}$

iv) Flange thickness



Flange thickness, t_f

For, elastic or compact cond^n (Pg. 18)

$\dfrac{b}{t_f} \leq 8.4t$ or $9.4t$

$\dfrac{266}{t_f} = 8.4 \Rightarrow t_f = 31.67$ say $t_f = 32$mm

4

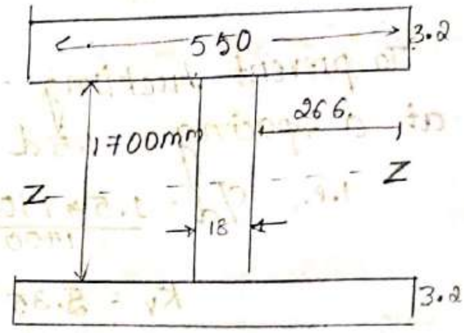
$\therefore$ Flange plate $= b_f \times t_f = 550mm \times 32mm$

For standard thickness of flange, refer steel tables [88 to 90]

c] Check for moment of resistance [Pg. 53]

Design bending strength, $M_d = \frac{\beta_b Z_p f_y}{\gamma_{mo}}$

where, $\beta_b = 1$, Z_p = plastic modulus



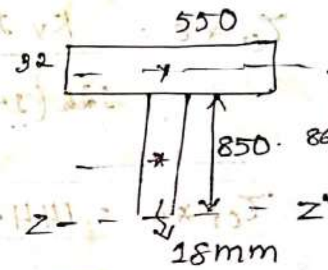
MI about Z-Z axis.

$= \frac{bd^3}{12}$

$= \frac{550 * (1700 * 3.2 + 3.2)^3}{12} -$

$2 \left[\frac{266 * (1700)^3}{12}\right]$

$\boxed{I_{zz} = 3.37 * 10^{10} mm^4}$



Similarly Plastic Modulus (Z_p)

$Z_p = \Sigma a\bar{y} = a_1 y_1 + a_2 y_2$.

$= 2 \left[\cancel{550 * 32}\right] \cancel{+ 2}[$

$= 2 \left[(550 * 32) * \left(\frac{32}{2} + 850\right)\right] + 2 \left[(850 * 18) * \frac{850}{2}\right]$

$\boxed{Z_p = 43.48 * 10^6 mm^4}$

$\therefore M_d = \frac{1 * 43.48 * 10^6 * 250}{1.10}$

$\boxed{\begin{aligned} M_d &= 9.88 * 10^9 \ N\text{-}mm \\ &= \cancel{9.88 * 10^6 \ kN\text{-}mm} \\ &= 9880 * 10^6 \ N\text{-}mm \end{aligned}}$ $> M_u \ (9652.5 \ kN\text{-}mm)$

If $M_d < M_u$, then increase flange dimension. [NOTE]

d] Check for shear. [Pg. 59 & 60]

From IS 800, design shear strength,

$V_d = \frac{V_n}{\gamma_{mo}} = \frac{V_{cr}}{\gamma_{mo}} = \frac{A_v \tau_b}{\gamma_{mo}}$

where, A_v = shear area

$= d_w t_w = 1700 * 18$

$\boxed{A_v = 30600 mm^2}$

$\tau_b = [1 - 0.8(\lambda_w - 0.8)] * \frac{f_{yw}}{\sqrt{3}}$

$\lambda_w = \sqrt{\frac{f_{yw}}{\sqrt{3}\, \tau_{cr}{*}e}}$; $\tau_{cr}{*}e = \frac{K_v \pi^2 E}{12(1-\mu^2)\left(d/t_w\right)^2}$

$K_v = 5.35 + \frac{4.0}{(c/d)^2}$

To prevent buckling of web, provide intermediate stiffners at a spacing = 1.5d [max spacing allowed]

i.e., $c/d = \frac{1.5 * 1700}{1700}$

$K_v = 5.35 + \frac{4.0}{\left[\frac{(1.5)^{}*1700}{1700}\right]^2} \Rightarrow \boxed{K_v = 7.12}$

$\mu = 0.3$

$\tau_{cr}{*}e = \frac{K_v \pi^2 E}{12(1-\mu^2)\left(d/t_w\right)^2} = \frac{7.12 * \pi^2 * 2\times10^5}{12(1-0.3^2)\left(\frac{1700}{18}\right)^2}$

$\boxed{\tau_{cr}{*}e = 144.35}$

$\lambda_w = \sqrt{\frac{f_{yw}}{\sqrt{3}\, \tau_{cr}{*}e}} = \sqrt{\frac{250}{\sqrt{3} * 144.35}} \Rightarrow \lambda_w = 0.99$

$\tau_b = [1 - 0.8(\lambda_w - 0.8)] * \frac{f_{yw}}{\sqrt{3}} = 1 - 0.8(0.99 - 0.8) * \frac{250}{\sqrt{3}}$

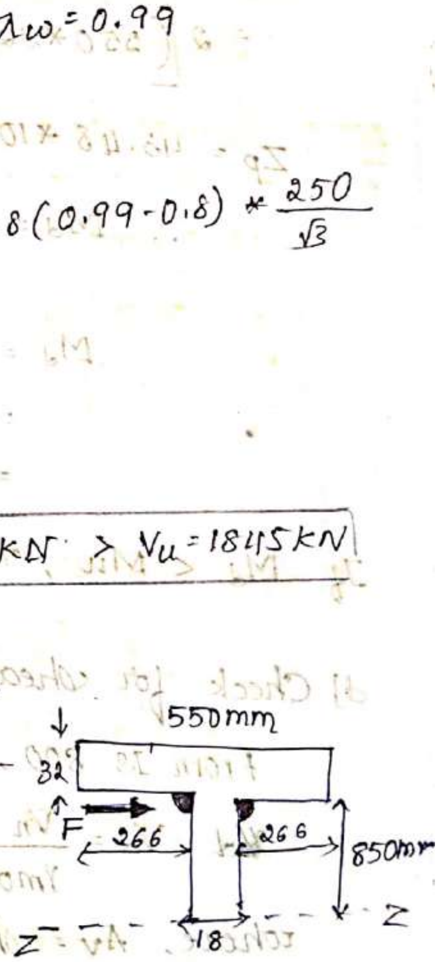
$\boxed{\tau_b = 121.24}$

$\therefore V_d = \frac{A_v \tau_b}{\gamma_{m0}} = \frac{30{,}600 * 121.24}{1.10}$

$\boxed{V_d = 3372.67 \times 10^3\, N = 3372.67\, KN > V_u = 1845\, KN}$

e) Welded connection b/w Flange & Web

Force in the top junction,



$F = \frac{V * a\bar{y}}{I_{zz}}$ where $V = V_u = 1845\,KN$

$= \frac{1845\times10^3 * 15.24\times10^6}{3.37 * 10^{10}}$

$I_{zz} = 3.37 \times 10^{10}\, mm^4$

$a\bar{y} = (550 * 32) * \left(850 + \frac{32}{2}\right) = 15.24 \times 10^6$

$\boxed{F = 834.44\, N/mm}$

Now, equating above force to the strength of the weld 6

i.e., $F = \left[0.75 * S * L * \frac{f_u}{\sqrt{3}\, \gamma_{mw}}\right] * 2$ take $L = 1mm$

$$834.44 = \left[0.75 * S * 1 * \frac{410}{\sqrt{3} * 1.25}\right] * 2$$

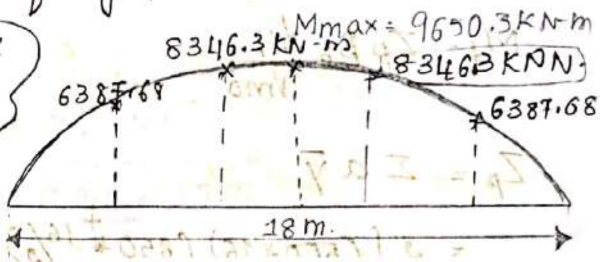
$$\boxed{S = 3.147\ mm}$$

∴ Provide minimum 5mm size of weld for web & flange connection

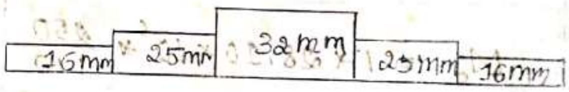
2] Curtailment of Flange Plate.

Bending moment is maximum only in the midspan & decreases towards the support. Hence from economical point of view as moment decreases, decrease the thickness of flange & it is called as curtailment of flange plate.

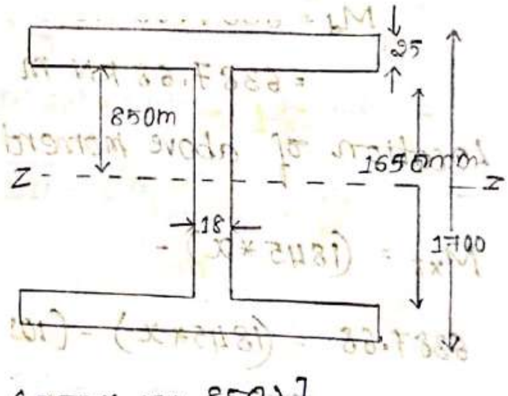
Available size thickness of flange
32, 25, 20, 18, 16, 12, 10, 8, 6mm



1] Let us decrease the plate thickness from 32mm to 25mm



NOTE: The standard thickness of plate are 32, 25, 20, 18, 16, 12, 10, 8, 6mm



Using moment of resistance

$M_d = \frac{\beta_b Z_p f_y}{\gamma_{m0}}$ → Pg no 53.

$\beta_b = 1$

Plastic modulus, $Z_p = \Sigma a\bar{y}$

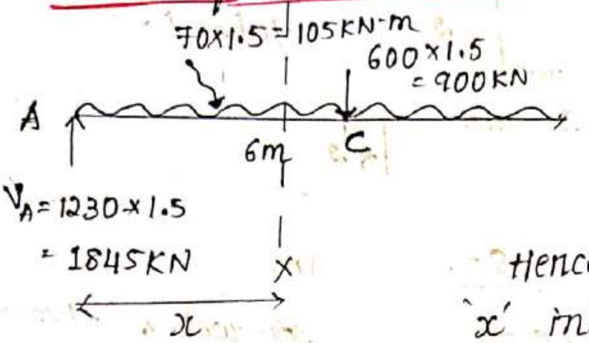
$$= 2\left[\left(550 * 25 * 850 + \frac{25}{2}\right) + \left(850 * 18 * \frac{850}{2}\right)\right]$$

$$\boxed{Z_p = 36.723 * 10^6\ mm^4}$$

$$M_d = \frac{1 * 36.723 * 10^6 * 250}{1.10}$$

$$\boxed{\begin{aligned} M_d &= 8346.3 * 10^6\ N\text{-}mm \\ &= 8346.3\ KN\text{-}m \end{aligned}}$$

Location of the above moment



$\Sigma M_c =$

$(1845 * 6) - (105 * 6 * 6/2) =$

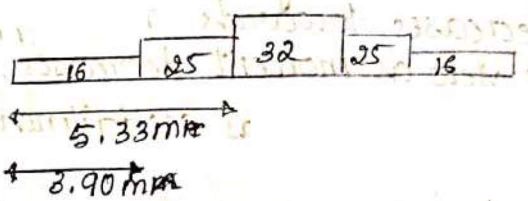
$\therefore M_c = 9180$ KN-m

Hence consider section x-x @ a distance 'x' inbetween A & C

$M_{x-x} \Rightarrow 8346.3 \times 10^6 = (1845 \times x) - (105 * x * x/2)$

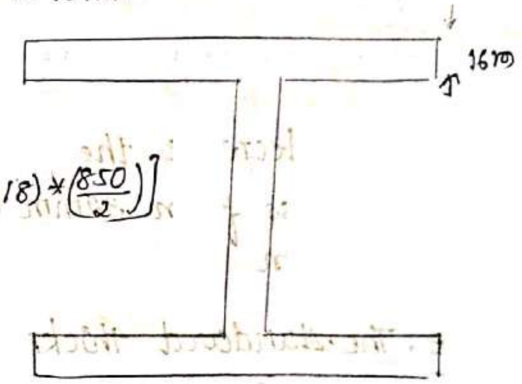
$\boxed{x = 5.33\text{ m from A}}$

ii) In the 2nd curtiment, reducing the thickness from 25 to 16mm



using moment of resistance

$M_d = Z_p \beta_b \frac{f_y}{\gamma_{m0}}$



$Z_p = \Sigma a \bar{y}$

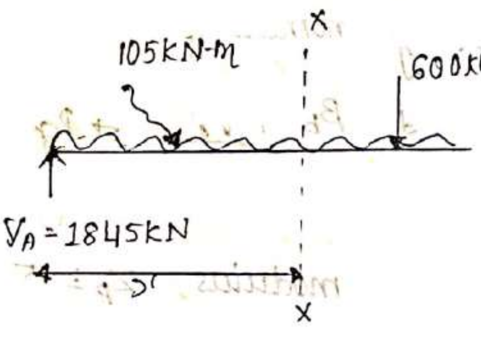
$= 2\left[(560 * 16)(850 + 16/2) + (850 * 18) * \left(\frac{850}{2}\right)\right]$

$\boxed{Z_p = 28.10 * 10^6 mm^4}$

$M_d = 1 * 28.10 * 10^6 * \frac{250}{1.10}$

$\boxed{\begin{array}{l} M_d = 6387.68 * 10^6 \text{ N-mm} \\ = 6387.68 \text{ KN-m} \end{array}}$

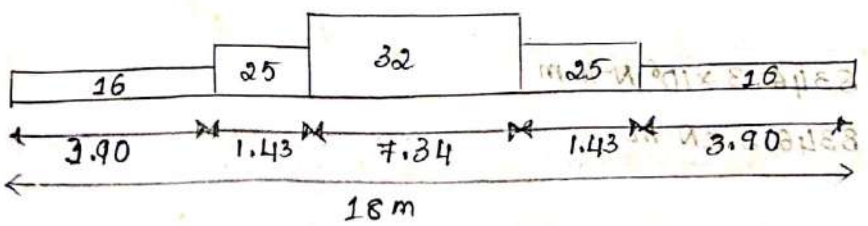
Location of above moment



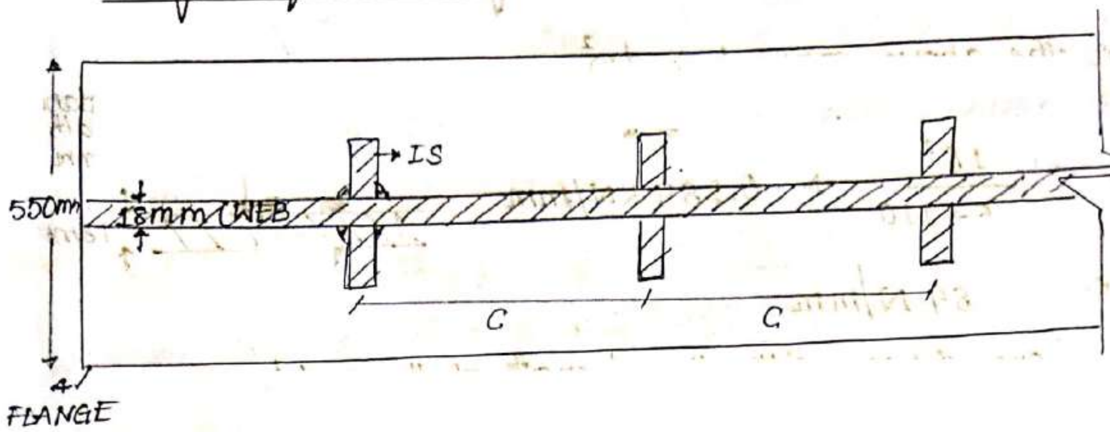
$M_{xx} = (1845 * x) - (105 * x * x/2)$

$6387.68 = (1845 * x) - (105 * x^2/2)$

$\boxed{x = 3.90\text{m from A}}$



3] Design of intermediate stiffners (IS)



i] Check for requirement

Ratio = d/t_w = d_w/t_w = $\frac{1700}{18}$ = $\boxed{94.44\ mm > 67}$

As per code, if the ratio is > 67, then intermediate stiffners is required. Hence provide IS to prevent buckling of web

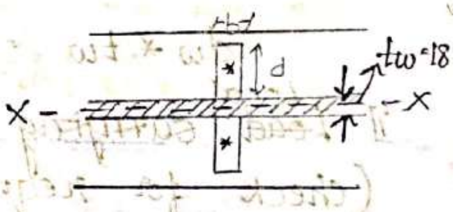
ii] Spacing of I.S [C]

Provide I.S at a maximum spacing $C = 1.5d$

$\therefore C = 1.5(1700) \rightarrow \boxed{C = 2550\,mm = 2.55\,m\ c/c}$

iii] Size of I.S.

Now the ratio. $C/d = 1.5 > \sqrt{2}$



As per code, if $C/d > \sqrt{2}$, the following eq^n should be used. class 8.7.2.4 of IS 800, hence use $\boxed{MI = I_s = 0.75 * d * t_w^3}$ → [Pg. no 66]

$I_s = 0.75 * 1700 * (18)^3 \rightarrow \boxed{I_s = 7.44 \times 10^6\,mm^4}$

MI for the above IS

$$I_s = 2\left[\frac{bd^3}{12} + ay^2\right]$$

$$7.44\times10^6 = 2\left[\frac{bd^3}{12} + (b*d)*\left(d/2 + 18/2\right)^2\right]$$

Let. Using 8mm thick plate, ∴ b = 8 mm

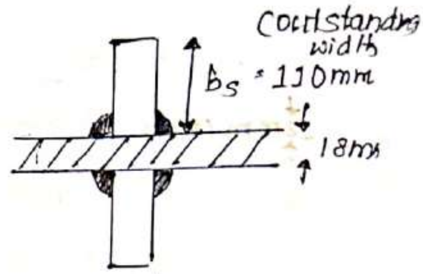
$$7.44\times10^6 = 2\left[\frac{8*d^3}{12} + (8*d)*(d/2 + 9)^2\right]$$

$d = 102.75\,mm$ say $\boxed{110\,mm = d}$

$\boxed{\therefore \text{Provide IS of size } 8 * 110\,mm}$

iv) Connection of ISs with Web [Pg. No. 67] → 8.7.2.6

Force for the above connect = $\frac{t_w^2}{5 \times b_s}$



$\therefore = \frac{18^2}{5 \times 110} \Rightarrow 0.589$ KN/mm

$\boxed{\text{Force} = 589 \text{ N/mm}}$

Equating above force with the strength of the weld

$F = 4\left[0.75 \times s \times L \times \frac{f_u}{\sqrt{3}\,\gamma_{mw}}\right]$ Take $L = 1$m

$0.589 \times 10^3 = 4 \times \left[0.75 \times s \times 1\text{mm} \times \frac{410}{\sqrt{3} \times 1.25}\right]$

$s = 1.03$

$\therefore$ Provide minimum size of weld = 3mm

4] Design of End Bearing Stiffners [EBS]

Due to reactions at the end, the web plate may buckle, To prevent this, provide stiffners at the ends called EBS.

Reactions = SF = V_u = 1845 KN

$d_w \times t_w$ = 1700 × 8mm

i] ~~Load~~ Local carrying capacity of the web. [Pg. No 67] - 8.7.4 (check for requirements)

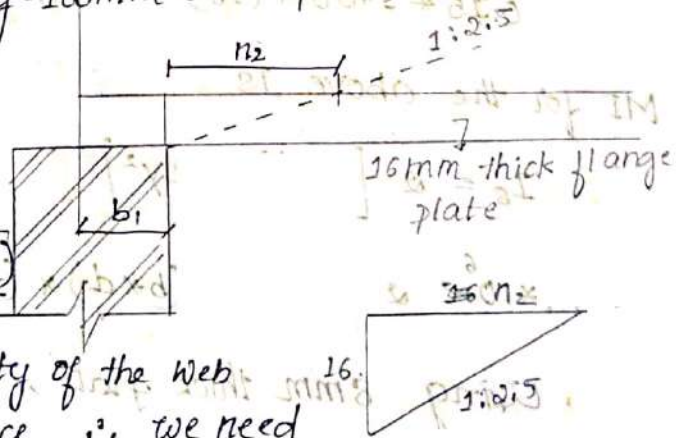
$F_w = (b_1 + n_2)\, t_w \frac{f_{yw}}{\gamma_{m0}}$

where, b_1 = Width of bearing = 100mm (Assumption)

$n_2 = 16 \times 2.5 = 40$mm

$F_w = (100+40) \times 18 \times \frac{250}{1.10}$

$\boxed{F_w = 572.72 \text{ KN} < 1845 (V_u)}$



Since load carrying capacity of the web is < external rea^n (or) Force, $\therefore$ we need to provide bearing stiffeners at the ends.

ii) Area of stiffners [Pg.no 68] - clause 8.7.5.2

$\frac{A_q f_{yq}}{0.8\,\gamma_{m0}} \geq F_x$

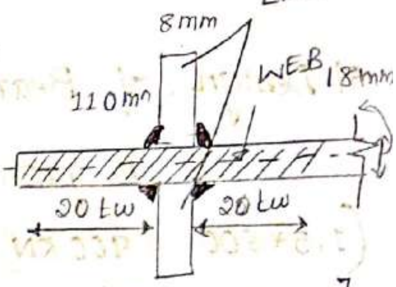
where F_x = Reaction & A_q = Area required for EBS

$$\frac{A_q \times 250}{0.8 \times 1.10} = 1845 \times 10^3 \Rightarrow \boxed{A_q = 6.494 m^2 = 6494 mm^2}$$

Provide approximately the size of EVS, same as that of IS size i.e., $\boxed{110mm \times 8mm}$

Also along with EBS plates, the part of the web ($20 \times t_w = 20 \times 18 = 360$) on either side of stiffners also acts like EBS.

The EBS is designed like a compression member for a effective height $= 0.7 \times d$



$\therefore L_e = 0.7 \times d = 0.7 \times 1700$

$\Rightarrow L_e = 1190 mm$

The c/s area of above EBS $\Rightarrow A = 2[(110 \times 8)] + [(360 + 360) \times 18]$

$A = 14720 mm^2$

$$I_{zz} = \Sigma a y^2$$

$$2\left[\frac{8 \times 110^3}{12} + (8 \times 110) \times \left[\frac{110}{2} + \frac{18}{2}\right)^2\right] + \left[\frac{720 \times 18^3}{12} + (720 \times 8) \times 0\right]$$

$I_{zz} = 9.33 \times 10^6 mm^4$

$\boxed{I_{min} = I_{zz} = 9.33 \times 10^6 mm^4}$

$\therefore$ min radius of gyration $= r_{min} = \sqrt{\frac{I_{min}}{A}} = \sqrt{\frac{9.33 \times 10^6}{14720}}$

$r_{min} = 25.17 mm$

slenderness ratio, $\lambda = \frac{L_e}{r_{min}} = \frac{1190}{25.17} \Rightarrow \lambda = 47.27$

From page 42, table 9C, $40 \rightarrow 198$

$47.27 \rightarrow ?$

$50 \rightarrow 183$

$\boxed{\therefore f_{cd} = 187.09 N/mm^2}$

$\therefore$ Design compressive strength

$P_d = f_{cd} A_c$

$= 187.09 \times 14720$

$\boxed{P_d = 2753.96 KN > V_u (1845 KN)}$

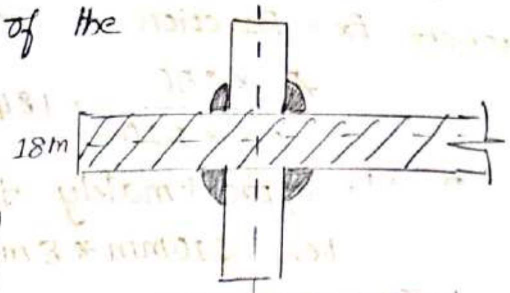
$\therefore$ It is hence

iii) Connections, Welded connection b/w WEB & EBS.

Force/mm height of web $= \frac{\text{Reaction}}{d_w} = \frac{1845 \times 10^3}{1700}$

$\boxed{F = 1086 N/mm}$

Equating above force with strength of the weld



$F = 4\left[0.75 \times S \times 1 \times \frac{f_u}{\sqrt{3} \times \gamma_{mw}}\right]$

$1086 = 4\left[0.75 \times S \times 1 \times \frac{410}{\sqrt{3} \times 1.25}\right]$

$S = 1.68\ mm \simeq 1.91\ mm$ $\boxed{\therefore \text{ provide minimum } 3\ mm \text{ size of weld}}$

5] <u>Design of Bearing stiffners.</u>

It is provided under the external point load ($1.5 \times 600 = 900\ KN$). The design procedure of B.S & EB are one & the same. Replace reaction by point load

$F_w = (b_1 + n_2) \times t_w \frac{f_{yw}}{\gamma_{m0}} = (100 + 40) \times 18 \times \frac{250}{1.10}$

$\boxed{F_w = 572.72\ KN < 900\ KN}$ ∴ since load carrying capacity is < external force ∴ we need to provide bearing stiffners @ the ~~intermediate~~ point / concentrated load.

ii) Area of stiffners -

$A_q f_{yq} / 0.8\ \gamma_{m0} \geq F_x \Rightarrow 900 \times 10^3 = \frac{A_q \times 250}{0.8 \times 1.10}$

$\boxed{A_q = 3168\ mm^2}$

Provide approximately the size of EBS i.e., $110 \times 8mm$

Also along with EBS plate, the part of web ($20 \times t_w = 360\ mm$) on either side of stiffners also acts like EBS

The BS are designed like a compressive member for a effective length $= 0.7 \times d$

$L_e = 0.7 \times 1700 \Rightarrow \boxed{L_e = 1190mm}$

C/s of above BS $= 2 \times [(110 \times 8)] + [(360 + 360) \times 18]$

$\boxed{Area = 14720\ mm^2}$

$I_{zz} = \Sigma a \bar{y}^2$

$= 2\left[\left(\frac{8 \times 110^3}{12}\right) + (8 \times 110) \times \left(\frac{110}{2} + \frac{18}{2}\right)^2\right] + \left[\frac{720 \times 18^3}{12} + (720 \times 8) \times 0\right]$

$\boxed{I_{zz} = 9.33 \times 10^6\ mm^4}$

min radius of Gyration, $r_{min} = \sqrt{\frac{I_{zz}}{Area}} = \sqrt{\frac{9.33 \times 10^6}{14720}}$

$r_{min} = 25.17$

$\lambda = L_e / r_{min} = \frac{11900}{25.17}$ $\lambda = 47.27$ ∴ $f_{cd} = 187.09\ N/mm^2$

Compressive strength, $P_d = f_{cd} * A_c \Rightarrow P_d = \boxed{2753.96\ KN > 900\ KN.}$

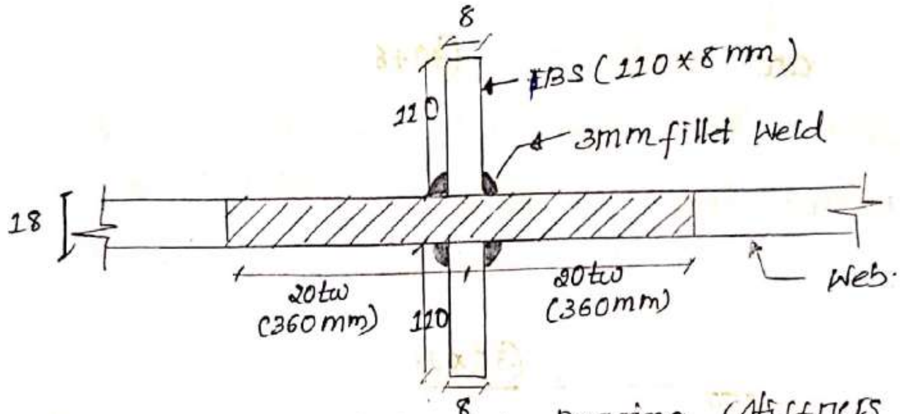
iii) Connections

$$\text{Force/mm length} = \frac{\text{Reaction}}{d_w} = \frac{900}{1700} = \boxed{529.411\ KN}$$

Equating above force with strength of the weld

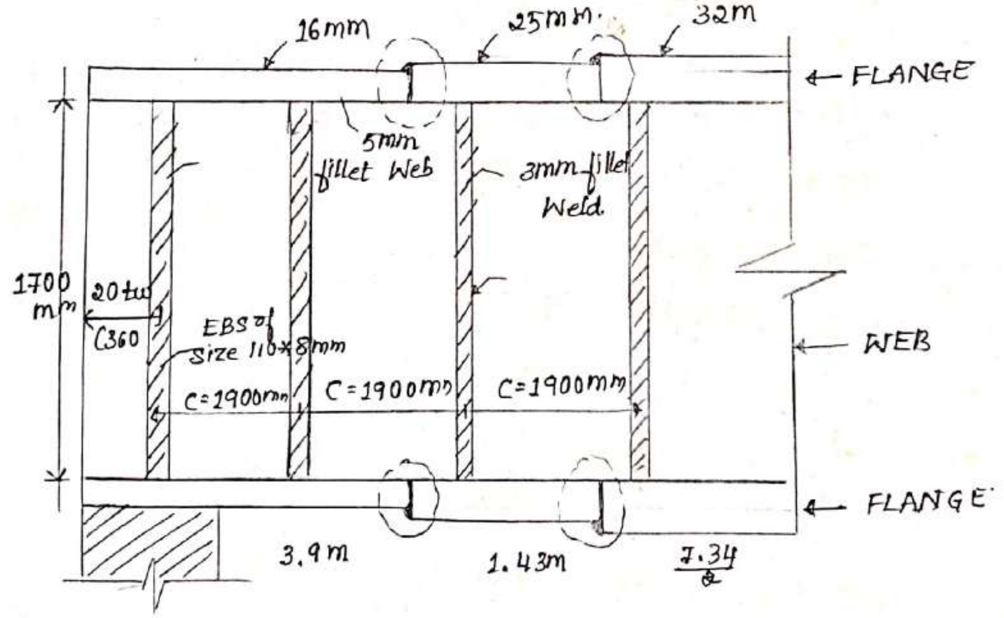
$$529.411 * 10^3 = \left[0.75 * s * 1 * \frac{410}{1.25 * \sqrt{3}}\right]$$

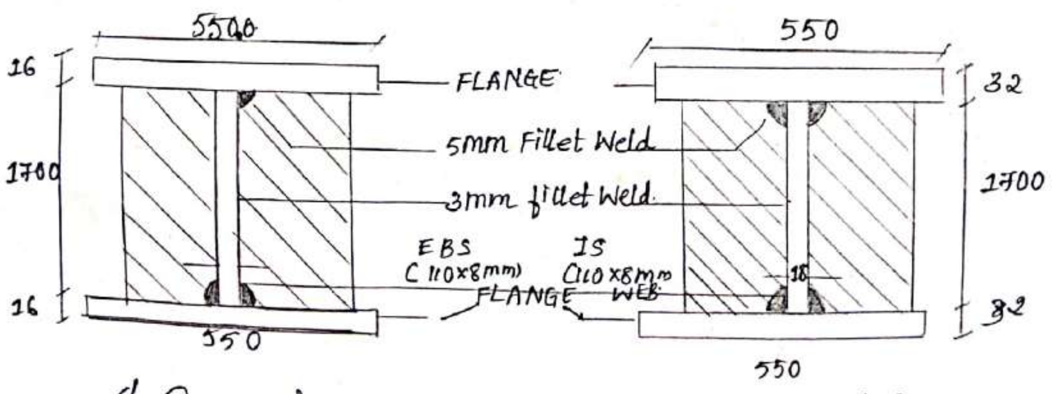
$s = 3.727\ mm$ $\boxed{\therefore \text{ Provide } 4\text{mm size of Weld}}$



Sectional plan & support Bearing stiffners

WELDED PLATE GIRDER [Half Longitudinal sec]





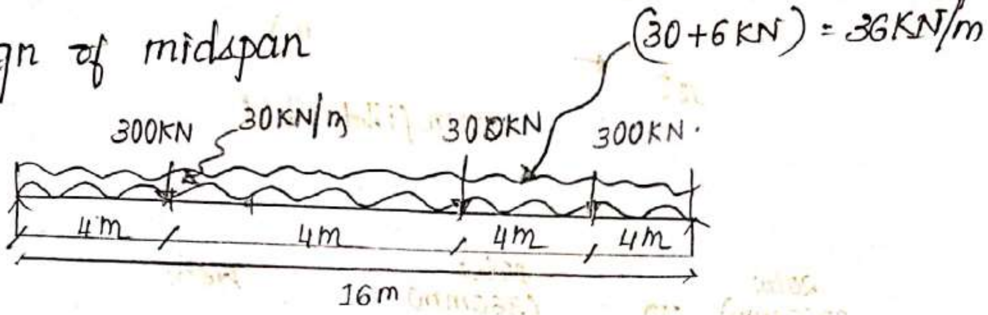
c/s @ suppor

c/c @ centre.

Design a welded plate girder of span 16m subjected to an UDL of 30KN/m throughout the span along with 3 concentrated loads of magnitude 300 KN @ a spacing of 4m. Design

i) Midspan cross-section
ii) Curtailment of flange plate.
iii) Intermidate stiffners
iv) End bearing stiffners
v) Bearing stiffners

1) Design of midspan



$$\text{Self Weight} = \frac{\text{Live load}}{250} = \frac{(30 \times 16) + 3(300)}{250} = 5.52 \text{ KN/m}$$

Self Wt = 6 KN/m

$$\text{Reaction, } V_A = V_B = \frac{(36 \times 16) + 3(300)}{2} \Rightarrow V_A = V_B = 738 \text{ KN}$$