UNIT-II

WATER TANKS.

TOPICS

1. Circular and Rectangular tanks resting on the Ground
2. Elevated tanks
3. Design of Staging
4. Intz Tank.
5. Underground Rectangular Tank (not in the syllabus)

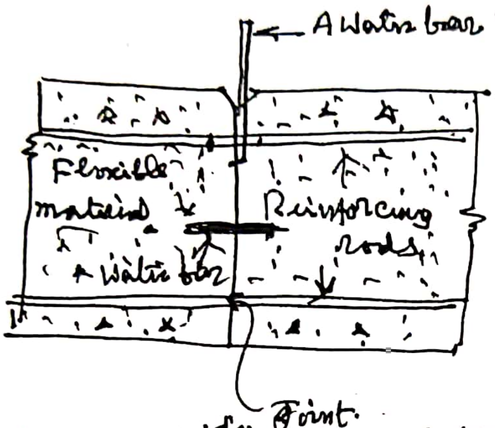
DR. B.L.P.

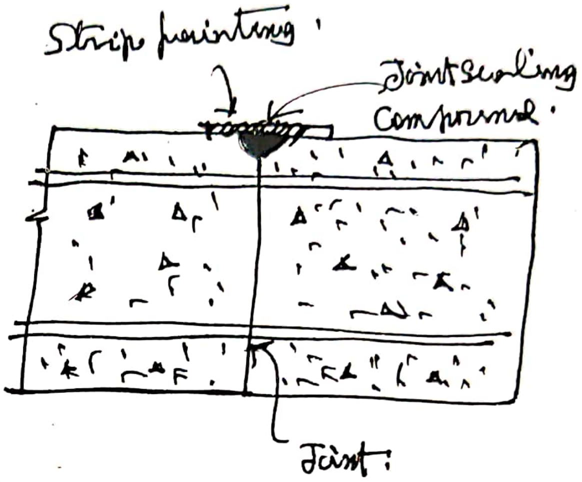
Scanned with CamScanner

Joints in liquid retaining structures

MOVEMENT JOINTS.

1). Contraction Joints.





Reinforcement is discontinuous and concrete is discontinuous. Water bar is of a flexible material like metal or PVC or rubber.

a) Complete contraction joint.

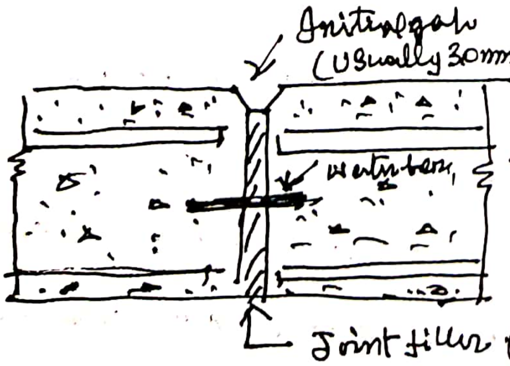
The materials used as fillers are all highly impermeable.

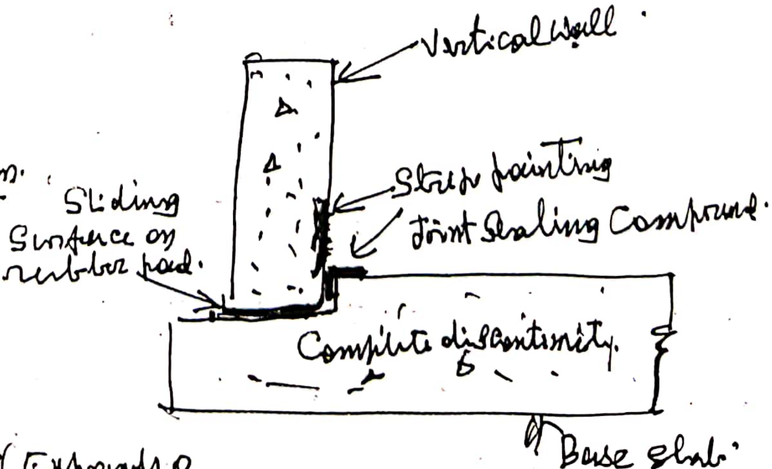
Reinforcement is continuous and concrete is also discontinuous.

Joint Sealing Compounds are asphalt, Bitumen, Coal tar etc. With or without fillers. Fillers are slate dust, lime stone, asbestos fibre etc.

b) Partial contraction joint.

2) Expansion Joints.





a) Typical expansion joint

.Used mostly in cylindrical tanks.

b) Typical Sliding joint

WATER TANKS INTRODUCTION:

A Water tank is used to store water to tide over the daily requirements. In the Construction of Concrete structures for the storage of Water, or other liquids, the imperviousness of Concrete is Very important.

Imperviousness:

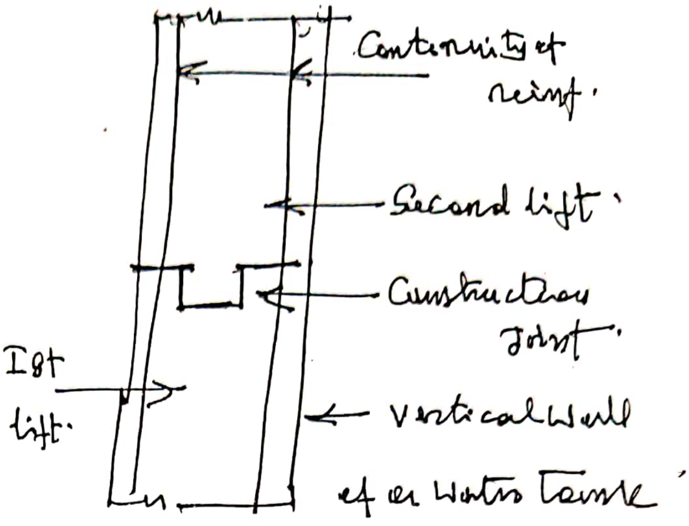
It mainly depends on the mix proportions, Cement Content, water - cement ratio and degree of Compaction. Hence, a lower limit for water cement ratio for given materials of Concrete is to be followed. The Cement Content should not be less than 330 kg and not more than 530 kg/m^3 of Concrete. It should be thoroughly vibrated and Compacted. Generally a rich mix like M30 is preferable. Segregation and honeycombing should be avoided.

Cracking

Cracking of Concrete due to its lower tensile strength should be avoided. Concrete should not crack on its water face. Cracking may also result due to restraint to shrinkage or temperature. Use of small sized bars, cement placing is desirable. Heat of hydration should disperse freely from the forms. Use of flat bottoms and sliding joints would help in preventing cracking. Joints should be minimised.

2) Construction Joints

It is fully continuous. It is provided for convenience of Construction. It is a rigid joint. Care should be taken to avoid percolation of water.



Construction Joint.

OTHER SPECIFICATIONS.

Min. Reinforcement

For thickness of the element upto 100 mm thickness the min reinforcement should be 0.30. For 450 mm thickness it should be 0.20. It is in each direction.

For thickness from 100 mm to 450 mm it is $= 0.30 - 0.1 \frac{(t-100)}{(450-100)}$

't' : Thickness between 100 mm to 450 mm.

If t > 225 mm, then two layers of bars are provided on both faces

Total steel area can be = Min. requirement.

Cover to Reinforcement.

1) For faces in contact with water = 25 mm or dia of the bar whichever is greater.

2) " away from water = Like other RCC Components.

3) For sea water, ground or water of corrosive nature Practical thickness : 12 mm extra (Not in design)

Table : Permissible stresses in Concrete

Concrete Grade.	Permissible stress in tension in N/mm²		Permissible stress in Shear. N/mm²
	Direct.	Bending	
M20	1.2	1.7	1.7
M25	1.3	1.8	1.9
M30	1.5	2.0	2.2
M35	1.6	2.2	2.5
M40	1.7	2.4	2.7

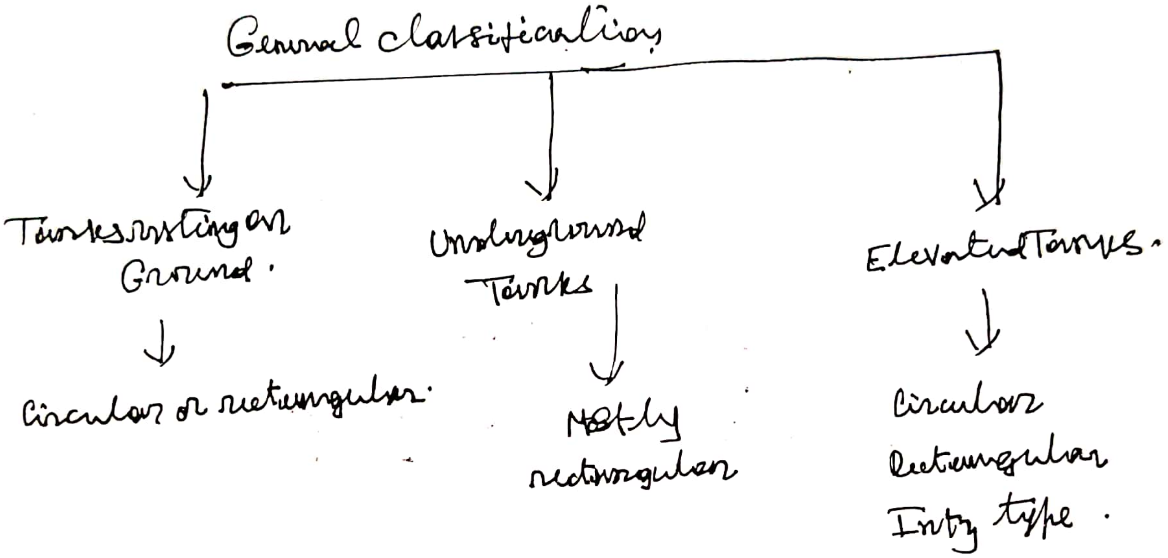
Table : Permissible stress for steel reinforcement.

+. ~~Direct tensile stress~~	Permissible stress in N/mm². Mild Steel	HYSD bars
1. Direct tensile stress.	~~150~~ 115	150.
2. Tensile stress in bending		
a) on liquid retaining force.	115	150.
b) on force away from the liquid		
< ~~225~~ 225 mm thick	115	150.
> 225 mm thick.	120	175
3. Tensile stress in Shear reinf.		
a) < 225 mm thick	115	150
b) > 225 mm thick	125	175

WATER TANKS

INTRODUCTION

To meet the daily requirement of water by industries, Campusses, localities, towns and cities various types of R.C. water tanks.



I.S. CODE

I.S 3370 – 1967 revised in 1997 and 1999 deals with water tanks.

There are four Parts:

- Part I —— General requirements
- Part II —— RCC tanks
- Part III —— Prestressed Concrete structures
- Part IV —— Design Tables.

Method of Design.

To avoid leakage problems LSD Should not be used. ~~old~~ previous IS456 (Working stress Method) is referred for permissible stresses.

Methods of Analysis

Exact methods.

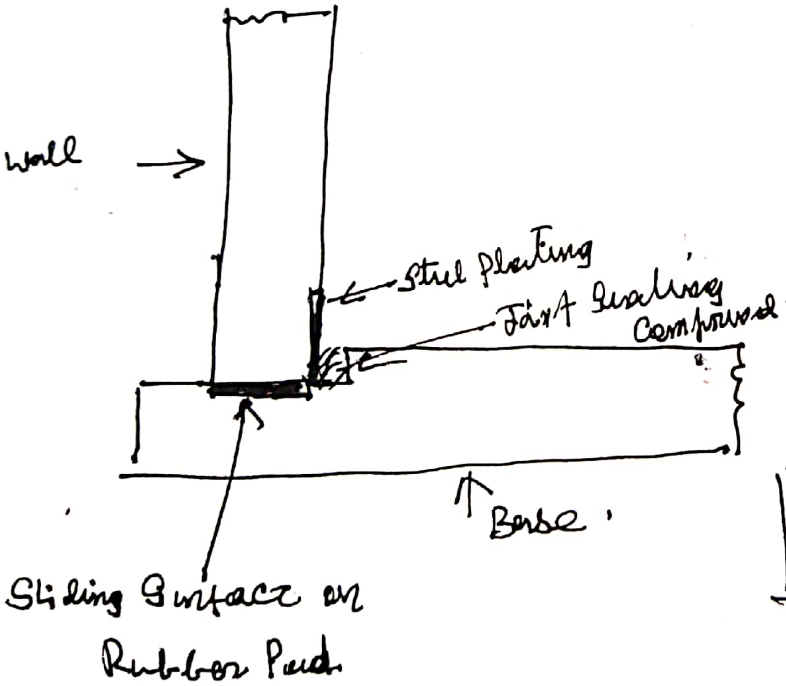
They are somewhat complicated. Coefficients are given based on the analysis conducted by using plate theory or FEM. More economical.

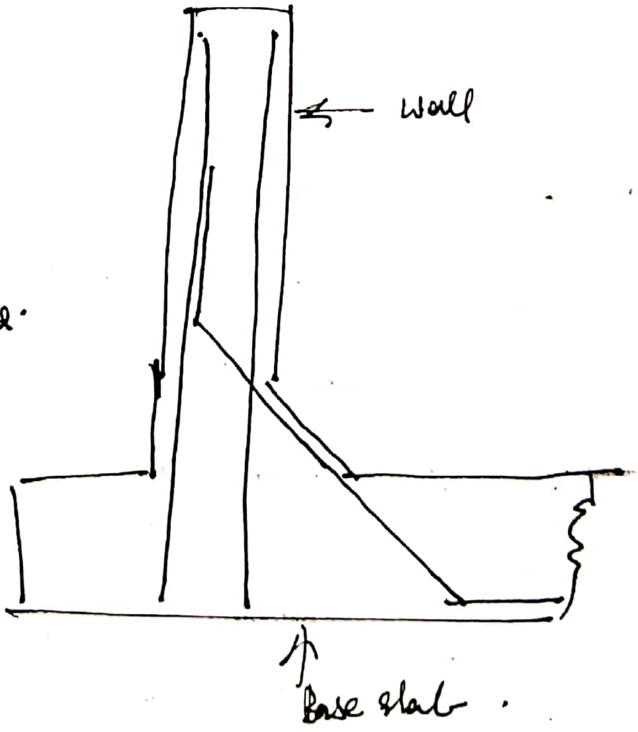
Approximate Methods.

They are safe. These methods are simpler. But not so economical.

I. CIRCULAR TANK RESTING ON GROUND.

a) Flexible Base and b) Rigid Base.



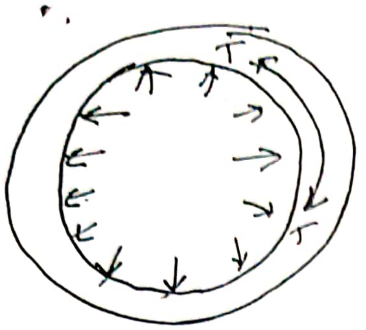


a) Flexible base

The tank wall is subjected to only hoop tension.

b) Rigid Base.

a) Flexible Base



$T = \frac{\gamma H D}{2}$, (Tension).

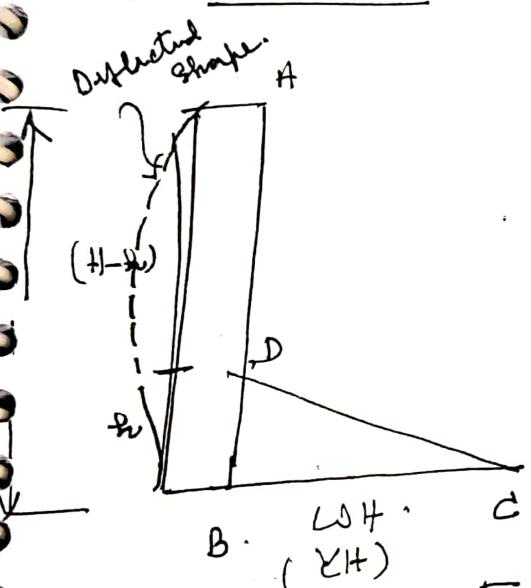
γ : unit wt. of water

D : Dia of the tank

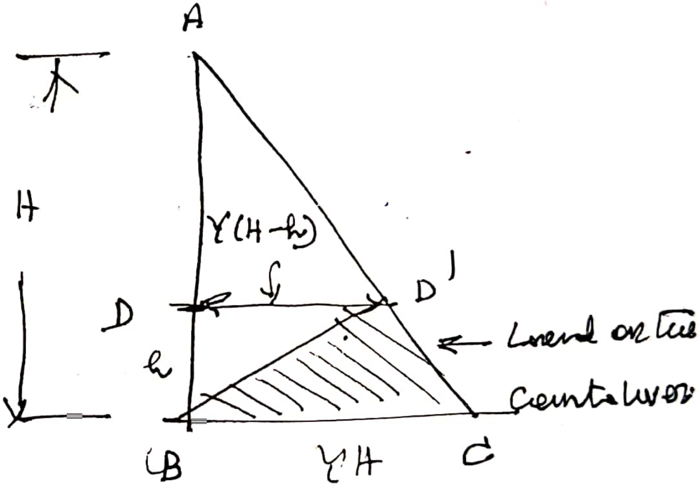
H : Ht. of the tank

In the case of a circular tank with flexible base the complete height of the tank is subjected to Hoop tension. Reinforcement for hoop tension is provided horizontally in the form of hoops. In the vertical direction nominal rings is provided,

b) Rigid Base



1) Load for Cantilever action



Load Sharing

In the approximate method,

Bottom $\frac{h}{3}$ or 1m (Whichever is greater) is treated as Cantilever

The remaining height above the Cantilever is designed for hoop tension

Moment about the base due to the Load taken by the Cantilever $= \frac{1}{2}\gamma H \cdot h \cdot \frac{h}{3} = \frac{\gamma H h^2}{6}$,

In the remaining height above the cantilever,

Max. hoop tension $= \gamma(H-h) \cdot \frac{D}{2}$, The reinforcements are designed accordingly.

EXAMPLE. (Flexible Base.)

Design a Circular water tank with flexible base resting on the ground to store 50,000 lit. of water, The depth of the tank may be kept at 4m. Use M25 Concrete and Fe415 Steel

SOLUTION. .

Capacity of the tank = 50,000 lit. = 50 m^3.

Depth of the tank = 4m.

Hence $\frac{\pi}{4} D^2 \times 4 = 50$, Solving D = 3.989m Say 4.0m.

Allowing 200mm free board, overall depth = 4.2m.

Taking Unit wt. of water = 980kg = 9.8 KN/m^3

Permissible stress in steel Fe415 for direct tension = 150 N/mm^2.

Permissible direct tension in M25 Concrete = 1.3 N/mm^2.

Max. hoop tension $= \frac{pD}{2} = \gamma H \cdot \frac{D}{2} = 9.8 \times 4.2 \times \frac{4}{2}$

= 82.32 kN/m height at the base.

Area of steel required for hoop tension $= \frac{82.32 \times 1000}{150}$

= 548.8 mm^2.

Using 12mm bars, Spacing $= \frac{\pi/4 \times 12^2}{548.8} \times 1000$ = 206 mm.

Provide 12 ϕ hoops @ 200 c/c along the height

Ast (Provided) $= \frac{\pi}{4} \times 12^2 \times \frac{1000}{200}$ = 565.5 mm^2/m. height.

* Towards top, the water pressure and hence the hoop tension get reduced. Hence at 1.5m above the base provide 12ϕ @ 300 c/c in the remaining portion of the wall.

Thickness of the wall

If 't' is the thickness of the wall over 1m ht. the equivalent area of Concrete is

$$1000t + (m-1)A_{st}$$

$$m = \frac{280}{3\times\sigma_{cbc}} = \frac{280}{3\times 8.5} = 11.$$

$A_{st} = 565.5\ mm^2$, For M25, permissible stress in tension $= \sigma_c = 1.3\ N/mm^2$.

Total tensile force T = 82.32 kN

Equating,

$$1.3 = \frac{82.32\times 1000}{1000t + (11-1)565.5}$$

Solving, t = 57.66 mm, Provide t = 100 mm.

Vertical steel

Providing minimum reinforcement at 0.3%.

$$A_{st}/m = \frac{0.3}{100}\times 100\times 1000 = 300\ mm^2$$

Using 8ϕ; Spacing $= \frac{\frac{\pi}{4}\times 8^2}{300}\times 1000 = 167\ mm$

Provide 8ϕ @ 150 c/c Vertically.

Base slab.

As the load is directly transfered from the slab to the soil, nominal thickness with minimum reinf. is sufficient.

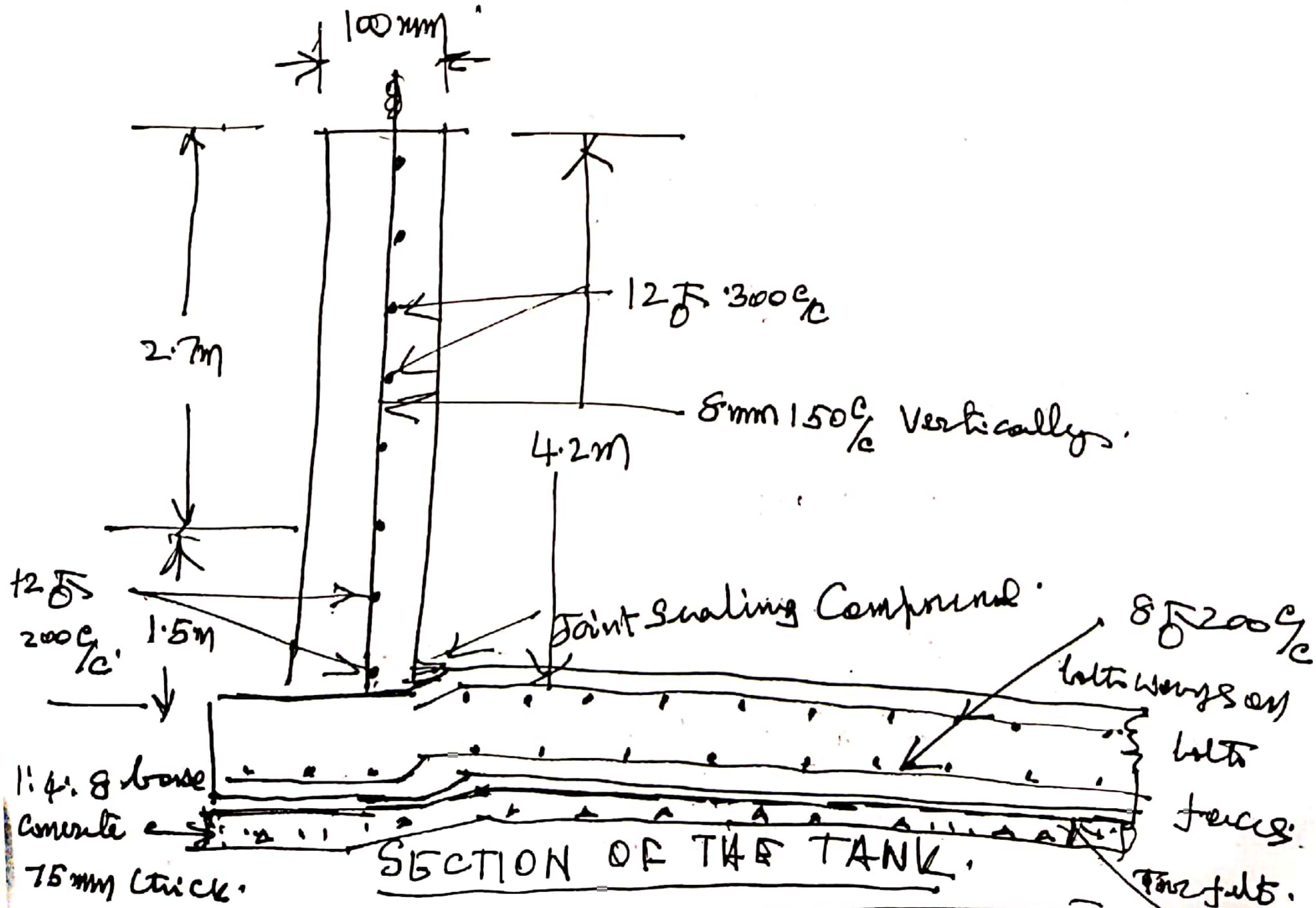
Providing 150 mm thickness,

Steel reinf. (at 0.3%) = $\frac{0.3}{100} \times 150 \times 1000$ = 450 mm²

Providing equally at top and bottom of the slab,

Steel on one face = $\frac{450}{2}$ = 225 mm²

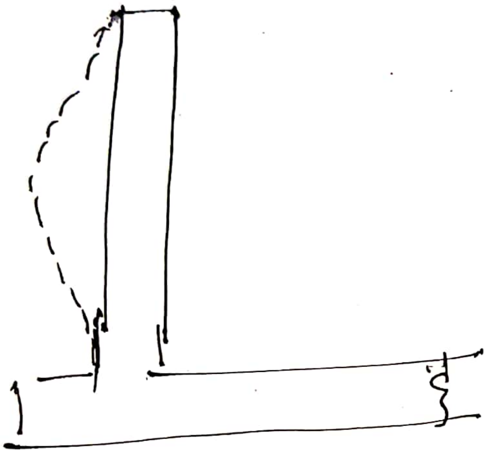
Spacing of 8ϕ = $\frac{\pi/4 \times 8^2}{225} \times 1000$ = 223 mm   Provide 220 c/c on both faces.



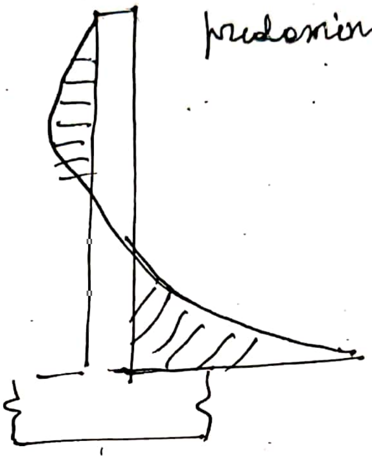
SECTION OF THE TANK.

Circular Tanks with Rigid Base.

When the joint between the wall and the base is rigid, there is no horizontal displacement of the wall. ~~Tanks~~ In the case of tanks with large diameter and small depth, hoop tension is smaller and the cantilever action is predominant.

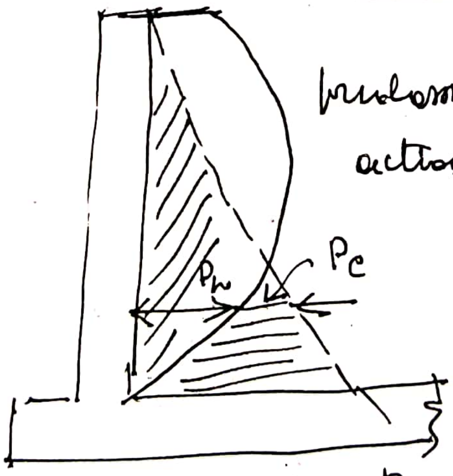


a) Deflected Shape.



b) BMD

otherwise Hoop tension is predominant and the cantilever action is small.



c) Load Distribution.

Methods of Analysis

1) Reissner's Method
2) Carpenter's Simplified Method
3) I.S. Code Method
4) Approximate Method.

1) Reissner's Method.

The max. moment $\underset{T}{\text{'M'}}$ and max tension are given in terms of Coefficients (Tables are given) and a parameter given by,

$$K = \frac{12(H)^4}{\left(\frac{D}{2}\right)^2 T^2} = \frac{48H^4}{D^2T^2} \text{ where}$$

H: Height

D: Diameter

T: Thickness.

Parameter 'k' can be determined.

T' can be found in the beginning by an approximate formula as

$$T = (30\overset{\text{in 'm'}}{H} + 50)\ mm.$$

2) Carpenter's Simplified Method.

- Simplified expressions are given as

1) Position of Max. hoop tension above the base $= KH$

2) Max. hoop tension $= \omega(H - KH)\frac{D}{2} = \omega\frac{D}{2}H(1-K)$.

3) Max. Cantilever moment $= F\omega H^3$.

The values of k and f can be taken from the tables.

3. I.S. Code Method : (3 tables are Considered)

Tables are given for various H^2/DT values for various depths the coefficients are given for max. tension and moments.

Hence Tension = Coeff. × $wH\frac{D}{2}$ per metre

Moment = Coeff × wH^3 Nm / m

Shear = Coeff × wH^2 kN

IS 3370-1967

Part I : General Requirements

Part-II : Reinforced Concrete structures.

Part III : Prestressed concrete structures

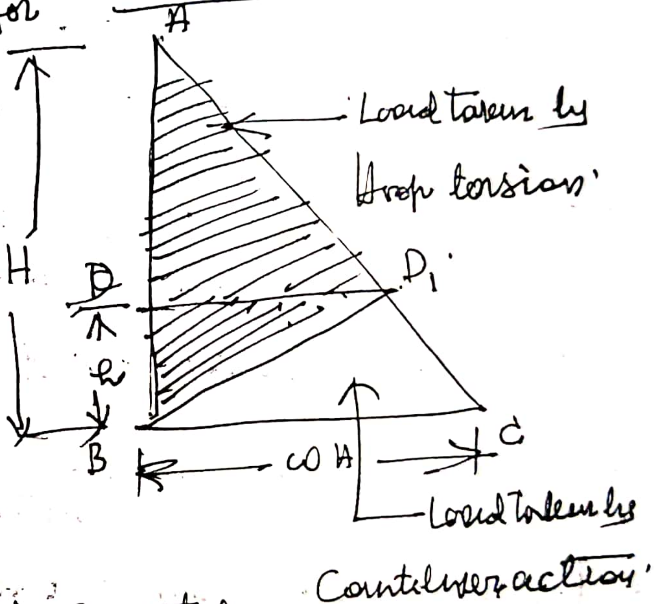
Part IV : Design tables

4. Approximate Method ;

Cantilever action is Considered for bottom $\frac{H}{3}$ or 1 m whichever is greater.

In the I.S Code method we have for $\frac{H^2}{DT}$ between 6 to 12 Cantilever action is predominant.

For $\frac{H^2}{DT}$ between 12 to 40 hoop tension is predominant.



In the figure, Max. hoop tension is Computed at the point 'D'. for a height of $(H-h) = w(H-h)\,D/2$.

The area of the triangle D_1BC gives the load on the Cantilever.

The cantilever reinforcement is provided against the max. moment occurring at the rigid end at the base. The reinforcements is provided on the inner face Vertically. Above the point 'D' the spacing of the reinforcement may be increased

Hoop reinforcement in the form of rings may be provided on both faces. Below the point D, the spacing of the rings may be increased.

Example on IS Code Method

A circular tank has an internal diameter of 10m. Max. height of water in the tank is 4 m. The walls of the tank are restrained at the base. Determine the values of max. hoop tension and its location and max. moment and shear at the base using I. S. Code Method. Assume thickness of wall as 160 mm.

SOLUTION : We get $\frac{H^2}{DT} = \frac{4^2}{10 \times 0.16} = 10$.

From table (1) of IS 3370 - Part IV, for $\frac{H^2}{DT} = 10$; we find that the max. tension at ($0.6H = 2.4m$ from top) as $= 0.608$.

Hence, max. Hoop tension $= 0.608\, \omega H \frac{D}{2} = 0.608 \times 9800 \times 4 \times \frac{10}{2}$

$= 119168$ N.

Now, for $\frac{H^2}{DT} = 10$, we get from table 2, the moment Coefficient for the base (at 1.0H) $=$ ~~0.179~~ -0.0122. (-ve indicates Tension on the inside face. +ve indicates tension on the outside)

Hence, moment at the base $= -0.0122 \times 9800 \times 4^3 = 7652$ Nm/m.

For $\frac{H^2}{DT} = 10$; from the table (3), Coeff. for shear $= 0.158$ at the base.

Hence, Shear $= 0.158\, \omega H^2 = 0.158 \times 9800 \times 4^2 = 24{,}774$ N acting towards inside.

EXAMPLE ON RIGID BASE. (Approximate Method.)

Design the water tank (resting on the ground) assuming that the joint between wall and base slab is rigid. Use Approximate Method. Capacity = 50,000 lit. Depth of the tank = 4m. Use M25 concrete and Fe 415 steel.

SOLUTION: We have kept $D = 4m$, $H = 4.2m$ including the free board.

We have $\sigma_{cbc} = 8.5 N/mm^2$, $\sigma_{st} = 150 N/mm^2$, $m = \frac{280}{3\sigma_{cbc}} = 11$

Design Constants are, $n = \frac{m\sigma_{cbc}}{m\sigma_{cbc}+\sigma_{st}} = \frac{11 \times 8.5}{11 \times 8.5 + 150} = 0.384$

$$j = 1 - \frac{n}{3} = 1 - \frac{0.384}{3} = 0.872$$

$$R = \frac{1}{2}\sigma_{cbc}\, j \cdot n = \frac{1}{2} \times 8.5 \times 0.872 \times 0.384 = 1.428.$$

CANTILEVER ACTION

Height for cantilever action above the base $= \frac{H}{3}$ or $1m = \frac{4.2}{3}$ or $1m$.

Hence $h = \frac{4.2}{3} = 1.4m$.

Cantilever moment $= \frac{1}{2}\gamma H \times h \times \frac{h}{3}$

$$= \frac{1}{2} \times 9.8 \times 4.2 \times 1.4 \times \frac{1.4}{3} = 13.446 kNm.$$

$$d = \sqrt{\frac{M}{Rb}} = \sqrt{\frac{13.446 \times 10^6}{1.428 \times 1000}} = 97.3mm,$$ Increasing by $\frac{1}{3}$

$$d = \frac{4}{3} \times 97.3 = 129.7 mm$$

However, ~~Providing~~ a minimum thickness of 150 mm is normally provided to avoid leakage.

Hence provide $d = 130mm$ with overall thickness $= 165mm$

$$A_{st} = \frac{M}{\sigma_{st}\, j d} = \frac{13.446 \times 10^6}{150 \times 0.872 \times 130} = 790.8\ mm^2.$$

Using 10 ϕ, spacing $= \dfrac{\frac{\pi}{4} \times 10^2}{790.8} \times 1000 = 99.32\ mm.$

Provide 10 ϕ 95 c/c vertically on the inside water face.

Provide a clear cover of 30 mm.

After 1.4m height curtail half the bars and provide 10 ϕ 190 c/c in the remaining height of 2.8 m.

SECTION FOR HOOP TENSION

At 1.4m height above the base, Max. hoop tension $= \gamma (H - h) \frac{D}{2}$ (due to weight)

$$= 9.8\,(4.2 - 1.4) \times \frac{4}{2} = 54.88\ KN.$$

Area of steel $A_{st} = \dfrac{54.88 \times 1000}{150} = 365.8\ mm^2$

Using 10 ϕ, spacing $= \dfrac{\pi/4 \times 10^2}{365.8} \times 1000 =$ ~~392.6~~ 214 mm.

Provide 10 ϕ hoops 200 c/c horizontally.

TENSILE STRESS IN CONCRETE

$T = 54.88$ kN.
$D = 165$ mm
$m = 11$
$A_{st}(act.) = 392.6\ mm^2$

$A_{st}(act.) = \dfrac{\pi/4 \times 10^2}{200} \times 1000 = 392.6\ mm^2.$

$$\underset{(Tensile)}{\sigma_{ct}} = \frac{T}{\text{Equivalent Concrete Area}} = \frac{54.88 \times 1000}{165 \times 1000 + (11 - 1)\,392.6}$$

$= 1.10 < 1.3$ (Permissible direct tensile stress for M25). Safe.

Hoop reinf. is provided @ 200 c/c up to 1.4m above the base. For other remaining, the spacing may be increased to 300 c/c towards bottom

DISTRIBUTION STEEL

In the vertical direction, Min. Steel = $\frac{0.3}{\cancel{1000}\ 100} \times 165 \times 1000 = 495\ mm^2$.

We have already provided vertical steel for Cantilever action. The same reinforcement of 10ϕ 95 c/c up to 1.4m height and 10ϕ 190 c/c above 1.4m. Serves the purpose of distribution steel also.

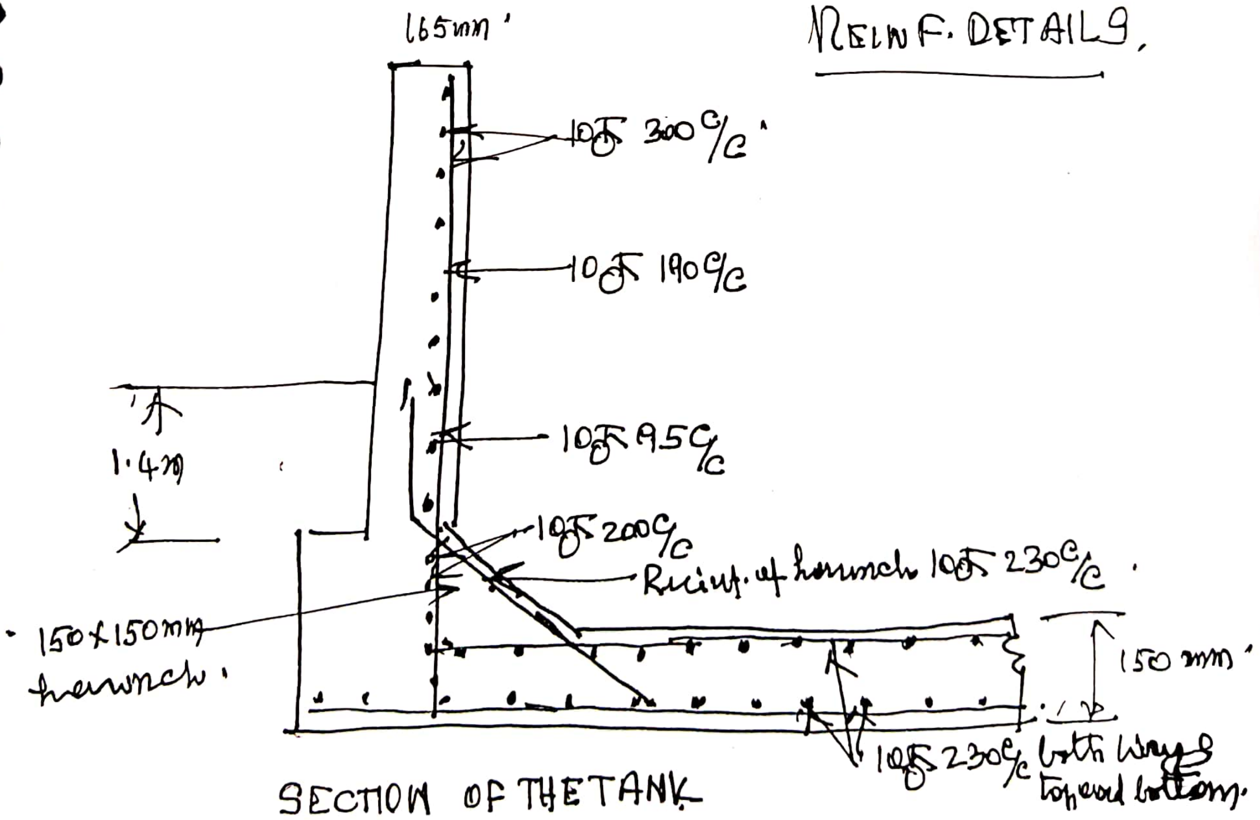
BASE SLAB.

Provide 150 mm thick, 10ϕ 230 c/c both ways top and bottom.

HAUNCHES.

Provide 150 mm x 150 mm haunches with 10ϕ 230 c/c to take up the cantilever moment at the base as development length.

REINF. DETAILS.



SECTION OF THE TANK

RECTANGULAR TANKS RESTING ON GROUND.

Dimensions of the tank are $L \times B \times H$.

APPROXIMATE METHOD.

There are two categories. 1) Tanks with $L/B < 2$ 2) $\frac{L}{B} > 2$.

First Category $\left(\frac{L}{B} < 2\right)$.

Bottom portion is treated as cantilever over $\frac{H}{4}$ or 1m whichever greater.

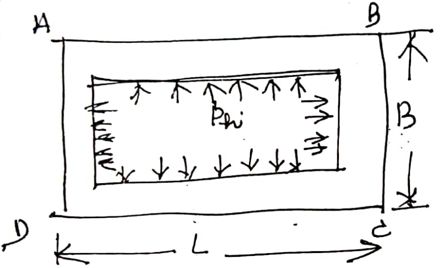
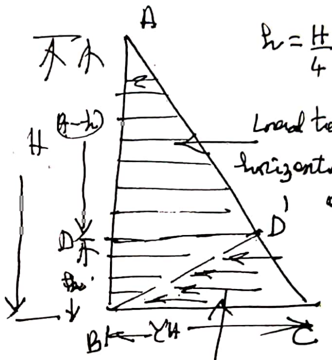


Fig ①



$h = \frac{H}{4}$ or 1m.

Fig. ②

Hence max. cantilever moment on the wall at the base $= \frac{1}{2}\gamma H \cdot h \cdot \frac{h}{3}$

For frame action max. pressure is at $(H-h)$ from top.

Hence horizontal pressure for this $= p_h = \gamma(H-h)$.

Considering this as a UDL, the FEM at 'A' $= p_h \cdot \frac{B^2}{12}$ and $p_h \cdot \frac{L^2}{12}$.

These moments are balanced at the joint by moment distribution.

There is horizontal tensile force developed as long wall supports short wall and short wall supports long wall.

Hence the horizontal forces (Reactions) developed in the walls,

In the long wall = $T_L = \gamma(H-h_1)\cdot\frac{B}{2} = M$
(Max.)

In the short wall = $T_B = \gamma(H-h_1)\cdot\frac{L}{2} = M$
(Max.)

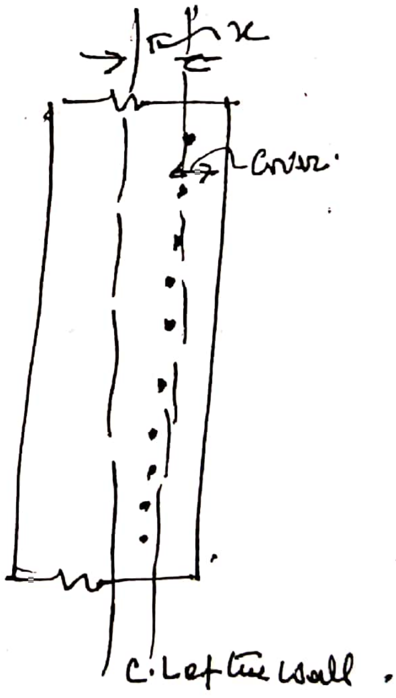
Against the tension, tensile reinforcement is kept in the walls. This tensile force acts at some distance w.r.t the centre of thickness of the wall.

Hence, this develops a moment = Tx.

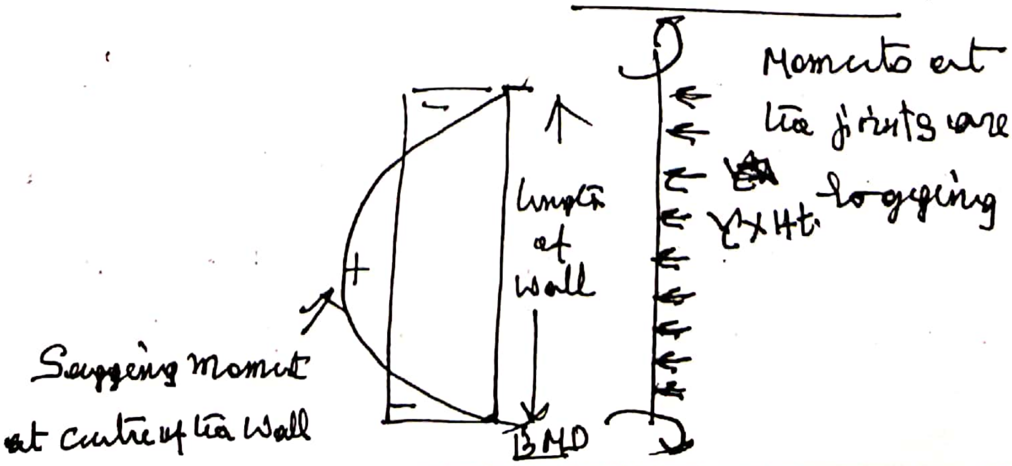
Hence, the net moment at the joint = $M-Tx$

As per the variation of the B.M in the wall the moment is (BMD) hogging at the joints and sagging at centre.

Hence. Reinf. is on the inside at the corners.
Reinf. is on the outside at centre.



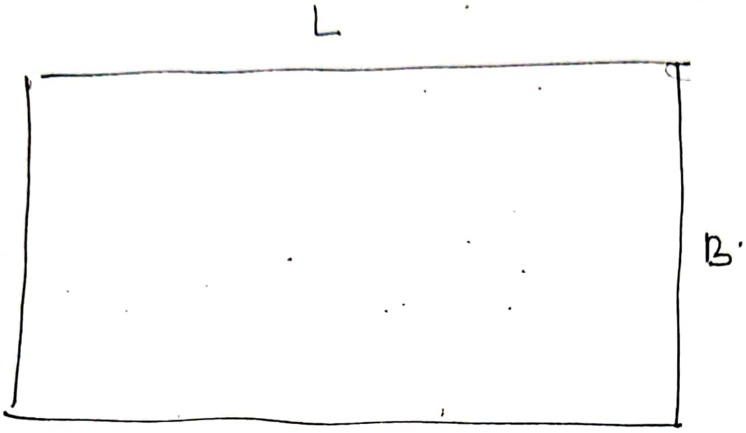
Along the Ht. of the Wall

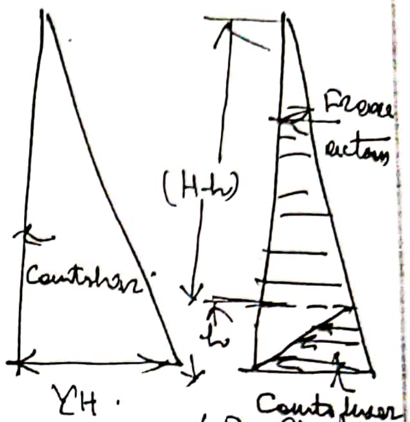


Moments at the joints are hogging

Sagging moment at centre of the wall

Second Category $\left[\frac{L}{B} > 2\right]$





(For long wall) (For Short wall)

Long walls

They are considered as cantilevers over the entire height of 'H'. Hence vertical reinf. against maximum cantilever moment is provided on the inside water face.

Tension in the long wall $= T_L = \gamma(H-h)\frac{B}{2}$.

(because for short walls cantilever action is considered up to 'h' above the base. Hence tension in the long wall is considered at this point.)

Horiz. reinf. or (Minimum 0.3%) whichever is greater is provided in the long wall. ~~Again~~

Short wall.

cantilever moment $= \gamma H \cdot \frac{h^2}{6}$.

Due to frame action, the jt. moment $= \frac{\gamma(H-h)B^2}{16}$ (Inner face)

Same at centre also may be taken (Outer face)

Reinf. is provided accordingly.

For short wall, the tension is produced by the long walls. But the long walls are considered as cantilevers over the entire height. But ~~the~~ 1m. portion of the long wall at the joints is considered for producing tension in the short wall.

Hence, in the short wall, $T_B = \gamma(H-h) \times 1$

Hence the reduction in the ~~joint~~ B.M of the short wall

$$= T_B \times x.$$

Reinf. for B.M and direct tension are calculated separately and provided as total horizontal steel.

EXAMPLE

Design a rectangular water tank of size 5m x 4m and 3m deep resting on firm ground. Use M25 concrete and mild steel.

SOLUTION.

$L = 5m,\ B = 4m,\ H = 3m;\ \frac{L}{B} = \frac{5}{4} = 1.25 < 2$

For Critical Section is at $\frac{H}{4}$ or 1m whichever is more

M25 concrete, $\sigma_{cbc} = 8.5\ N/mm^2$, $m = 11$

σ_{st} (For mild steel) = $115\ N/mm^2$.

The design constants are,

$$n = \frac{m\sigma_{cbc}}{(m\sigma_{cbc} + \sigma_{st})} = \frac{11 \times 8.5}{(11 \times 8.5 + 115)} = 0.448$$

$$j = 1 - \frac{n}{3} = 1 - \frac{0.448}{3} = 0.850.$$

$$R = \frac{1}{2}\sigma_{cbc}\, j\, n = \frac{1}{2} \times 8.5 \times 0.83 \times 0.448 = 1.619.$$

Hence in this case, cantilever action for the bottom $h = 1m$ takes place for both 'L' and 'B'. In the remaining $(H-h) = 2m$ horizontal action takes place.

HORIZONTAL FRAME ACTION.

$h = 1m$; Hence $p_h = \gamma(H-h) = 9.8(3-1) = 19.6\ kN/m^2$

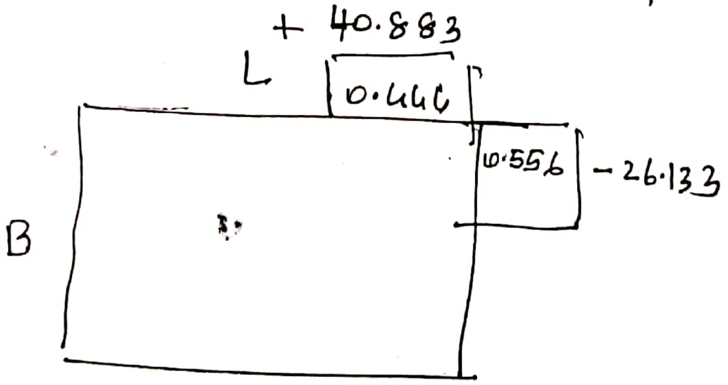
Fixed end moments are, $\frac{p_h L^2}{12} = \frac{19.6 \times 5^2}{12} = 40.833\ kNm/m$ for (Long wall)

$\frac{p_h B^2}{12} = \frac{19.6 \times 4^2}{12} = 26.133\ kNm/m$ (for short wall.)

Maintaining the same thickness for both long and short walls,

The moment distribution table is as follows.

Member	Stiffness	Total joint Stiffness	Distribution factor
Short wall (B)	$\frac{4EI}{L} = \frac{4EI}{4} = EI$	1.8 EI	0.556
Long wall (L)	$\frac{4EI}{l} = \frac{4EI}{5} = 0.8EI$		~~0.556~~ 0.444



Short wall D.F 0.556	D.F 0.444 Long wall
-26.133	+ 40.883
- 8.2	- 6.55
- 34.333	+ 34.333

LONG WALL.

At the corner Moment = 34.333 kNm (Tension on the ~~outside~~ water face)

$$d = \sqrt{\frac{M}{Rb}} = \frac{34.333\times10^6}{1.619\times1000} = 146\,mm.$$

To keep the section under reinforced, keep overall thickness = 200 mm.

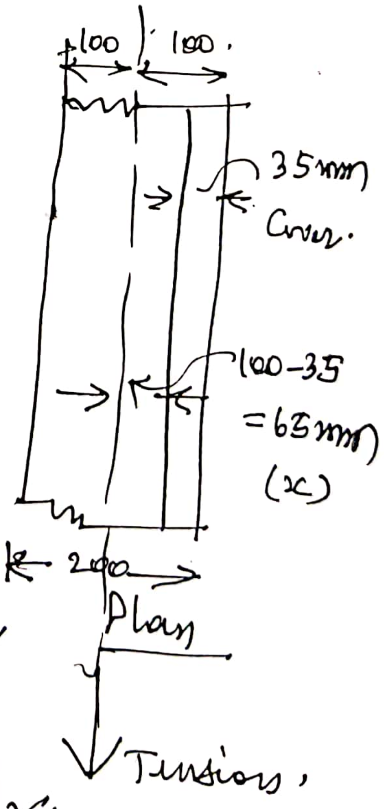
Hence $d = 200 - 35 = 165\,mm$ (Provided).

Direct tensile force on long wall $= T_L = P_w \frac{B}{2} = 19.6\times\frac{4}{2} = 39.2\,kN$

" on short wall $= T_B = P_w \frac{L}{2} = \frac{19.6\times5}{4} = 49\,kN.$

Eccentricity of the reinforcement from the centre is

$$= \frac{200}{2} - 35 = 65\,mm.$$



Hence, design moment at corner = $M - Tx$

$$= 34.333 - 39.2\times0.065 = 31.785\,kNm$$

Hence, at corner, the horiz. steel

$$= \frac{31.785\times10^6}{115\times0.85\times165} = 1970\,mm^2.$$

Steel against direct tension $= \frac{39.2\times10^3}{115} = 341\,mm^2$

Hence, total horizontal steel $= 1970+341 = 2311\,mm^2.$

Reinforcement for short wall

Net moment $M = 34.333 - T_B x = 34.333 - 49 \times 0.065$

$M = 31.148$ kNm.

Hence, $A_{st_1} = \frac{31.148 \times 10^6}{115 \times 0.850 \times 165} = \overset{1931}{\cancel{19.31}}\ mm^2$.

$A_{st_2} = \frac{49 \times 1000}{115} = 426\ mm^2$

(Direct tension) Hence total $A_{st} = A_{st_1} + A_{st_2} = \overset{1931}{\cancel{1931}} + 426 = 2357\ mm^2$.

Using 20ϕ, spacing $= \frac{\pi/4 \times 20^2}{2357} \times 1000 = 133\ mm$.

Hence provide 20ϕ @ 130 c/c horizontally.

At the middle of the wall, net B.M $= \gamma(H-h)\frac{B^2}{8}$ − End moment.

$= 9.8(3-1)\frac{4^2}{8} - 34.333 = 4.867$ kNm (Small value)

Hence min. steel is sufficient for this.

By bending half the bars of the end reinforcement at $B/4$ (=1m) towards the outer face. Remaining is continued straight.

Hence at centre of the wall, reinf. provided is 20ϕ @ 260 c/c horizontally

The end reinf. is ~~to~~ on the inside water face and the central reinf. is on the outside.

Reinf. in the vertical direction (Bottom $h = 1m$)
$H = 3m$

Cantilever moment at the base $= \gamma H . \frac{h^2}{6} = \frac{9.8 \times 3 \times 1^2}{6} = 4.9$ KNm.

$$A_{st.} = \frac{M}{\sigma_{st}\, j\, d} = \frac{4.9 \times 10^6}{115 \times 0.85 \times 165} = 304 \text{ mm}^2 \text{ (Small)}$$

$$\text{Min. steel} = \frac{0.3}{100} \times 200 \times 1000 = 608 \text{ mm}^2.$$

Half the steel is provided on each face.

Hence, on each face, for the vertical distribution steel (using 10ϕ)

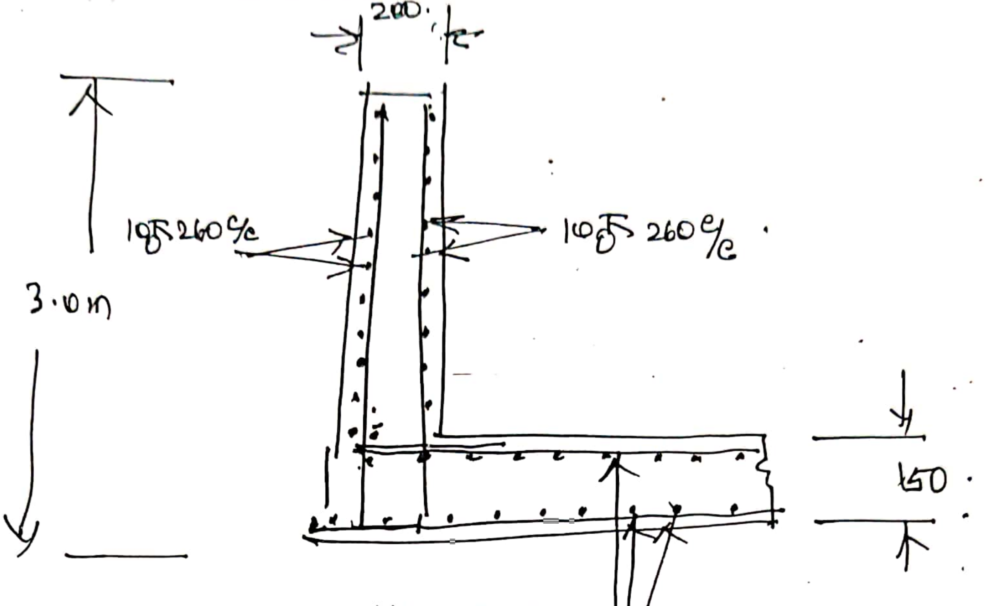
$$\text{Spacing} = S = \frac{\pi/4 \times 10^2}{304} \times 1000 = 258 \text{ mm}.$$

Provide 10ϕ 250 c/c vertically on each face.

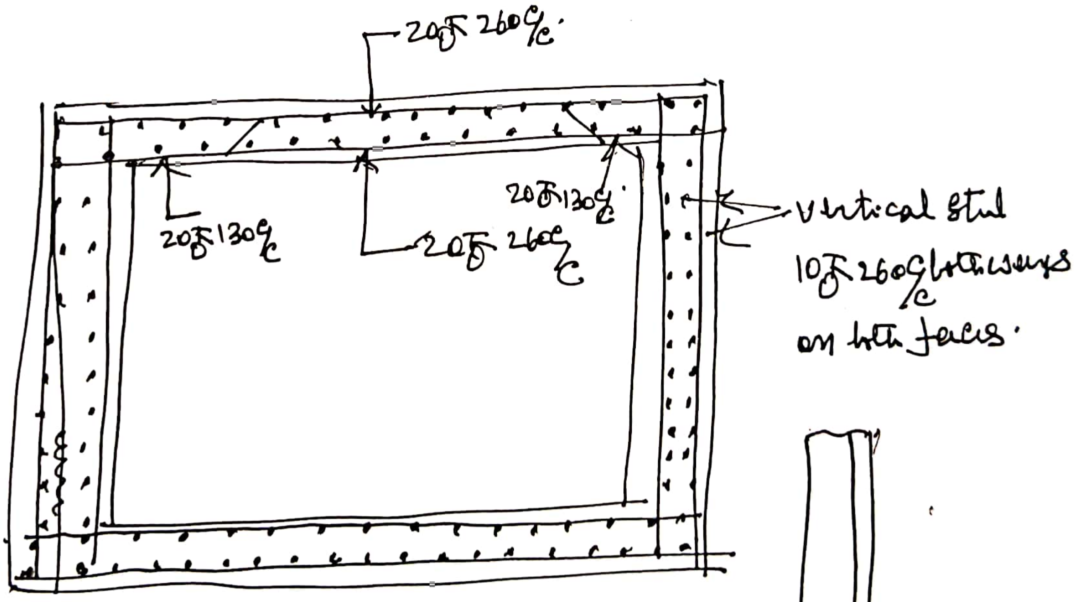
BASE SLAB..

Provide nominal 150mm thickness with 8ϕ 220 c/c both ways at top and bottom. A lean concrete bed with PCC 1:4:8 of 100 mm thickness may be provided below the base slab.

DETAILS OF REINFORCEMENT

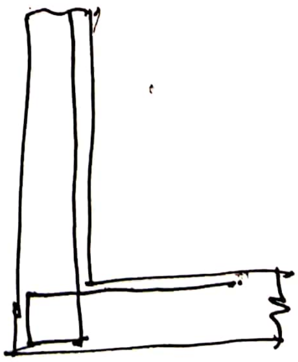


SECTIONAL ELEVATION



SECTIONAL PLAN

NOTE: The inner bars are bent in the form of a loop to avoid bursting at the joint.



LOOP AT THE JOINT

CASE II

RECTANGULAR WATERTANKS [WITH $\frac{L}{B} > 2$]

Long walls.

They behave like Cantilevers of height 'H'.

Hence thickness and area of vertical steel are Calculated.

Steel may be Curtailed towards top.

Short Wall

Lower portion is Considered as Cantilever over 'h'

The remaining portion $(H-h)$ is Considered for horizontal frame action as discussed in the previous case.

Hence Cantilever moment in Short wall $= \left(\frac{1}{2}\gamma H \cdot h\right)\frac{h}{3} = \gamma H \cdot \frac{h^2}{6}$.

Due to horizontal frame action B.M may be taken $= \frac{\gamma(H-h)B^2}{12}$
(Tension on the Waterface) (at the ends)

At centre +Ve B.M $= \frac{\gamma(H-h)B^2}{8} - \frac{\gamma(H-h)B^2}{12} = \frac{\gamma(H-h)B^2}{24}$

(Tension on the outerface).

For short wall, tension is transferred from long wall only from the end 1m length.

Direct tension in the long wall (Horizontal steel)

This is Considered to be transferred from the Short wall. ~~only over 1m end length of the Shortwall.~~

Hence Direct tension in the long wall $= T_L = \gamma(H-h) \times \frac{B}{2}$ m.

Hence the moment due to eccentricity $= T_L \cdot x$; Hence the net

Steel is usually minimum at 0.3%.

~~moment for the long Wall = (Above moment − $T_L x$). Steel in the long wall is provided for the net moment + Direct tension.~~

EXAMPLE: Design an open rectangular water tank of size 3m × 8m and 3 m deep. Use M25 and Fe415.

SOLUTION **Design Constants.**

$\sigma_{cbc} = 8.5\ N/mm^2$, $\sigma_{st} = 150\ N/mm^2$.

$m = 11$, $n = 0.\cancel{364}\overset{384}{}$, $j = 0.872$, $R = 1.423$.

$\frac{L}{B} = \frac{8}{3} > 2$.

LONG WALLS.

Cantilever action for the whole depth.

Hence max moment at the base $= \frac{1}{2}\gamma H \cdot H \cdot \frac{H}{3} = \frac{\gamma H^3}{6} = \frac{9.8\times 3^3}{6} = 44.1\ kNm$.

$d = \sqrt{\frac{M}{Rb}} = \sqrt{\frac{44.1\times10^6}{1.423\times1000}} = 176\ mm$; Provide $D = 220$, $d = 220-35$
$= 185\ mm$.

Hence, vertical reinf. $= \frac{44.1\times10^6}{150\times0.872\times185} = 1823\ mm^2$.

Using 16 mm φ, Spacing $= \frac{201}{1823}\times1000 = 119\ mm$.

Hence provide 16φ 110 c/c on the inner face.

To curtail the steel towards top, Considering any height 'h'

The ratio of steel $= \frac{A_{sth}}{A_{stH}} = \frac{h^3}{H^3}$, As per the code, the point of

curtailment should be at $0.62 + 12\times$ Dia of the bars in m $= 0.62 + \frac{12\times16}{1000} = 0.812$

However, curtail half the bars at 0.9m above the base.

Longwall – Reinf. in horizontal Direction

Direct tension in long wall transferred by the short wall ~~is~~ due to water pressure at $(H-h)$ where 'h' is the height considered above the base for a cantilever action in the short wall).

$T_L = \gamma (H-h) \frac{B}{2}$

$= \gamma (H-1) \cdot \frac{B}{2}$

$h = 1 m$.

Hence $T_L = 9.8 (3-1) \frac{3}{2} = 29.4 KN$.

Hence, horizontal steel $= \frac{29.4 \times 1000}{150} = 196 mm^2$ (very small)

Hence, min. steel $= \frac{0.3}{100} \times 220 \times 1000 = 660 mm^2$.

Hence, provide half ($330 mm^2$) may be provided on each face.

Provide ~~Using~~ 8 mm ϕ 150 c/c horizontally on both faces. Provide both ways on the outer wall.

SHORT WALL: Cantilever action $\left(\frac{H}{3}\right)$

Cantilever moment (in the bottom $\frac{H}{3}$ length)

$= \frac{1}{2} \gamma H \cdot h \cdot \frac{h}{3} = \gamma H h^2 / 6$.

$\frac{H}{3} = h = 1 m$

Hence $M = \frac{9.8 \times 3 \times 1^2}{6} = 4.9 KNm$.

$A_{st}. = \frac{4.92 \times 1000}{150 \times 0.872 \times 185} = 202 mm^2$ (Too Small)

Hence min. steel of 8 ϕ @ 150 c/c is provided vertically on each face

Horizontal steel: water pressure for frame action $= \gamma (H-h)$

$= 9.8 (3-1) = 19.6 kN/m^2$.

B.M at ends $= \frac{p_h B^2}{12} = \frac{19.6 \times 3^2}{12} = 14.7 KNm$.

Actual tension due to 1m. long wall $= T_B = \gamma (H-h) \times 1 = 9.8 (3-1) 1 = 19.6 KN$.

Hence, $A_{st_1} = \frac{14.7\times10^6}{150\times0.872\times185} = 607\ mm^2$.

$A_{st_2} = \frac{19.6\times1000}{150} = 130\ mm^2$.

Hence, total Steel = 607+130= 737 mm^2 (at the ends.)

~~Hence provide~~ 12 ϕ spacing = $\frac{113}{737}\times1000 = 153$ mm

Hence provide 12ϕ @ 150 c/c at the ends horizontally.

At centre, +ve B.M = $\frac{p_h . B^2}{24}$, ~~p_h~~

$p_h = \gamma(H-h) = 9.8\ (3-1) = 19.6\ kN/m^2$.

Hence, +veBM at centre = $\frac{19.6\times3^2}{24} = 7.35$ kNm.

$A_{st}. = \frac{7.35\times10^6}{150\times0.872\times1000} = 303.5\ mm^2$.

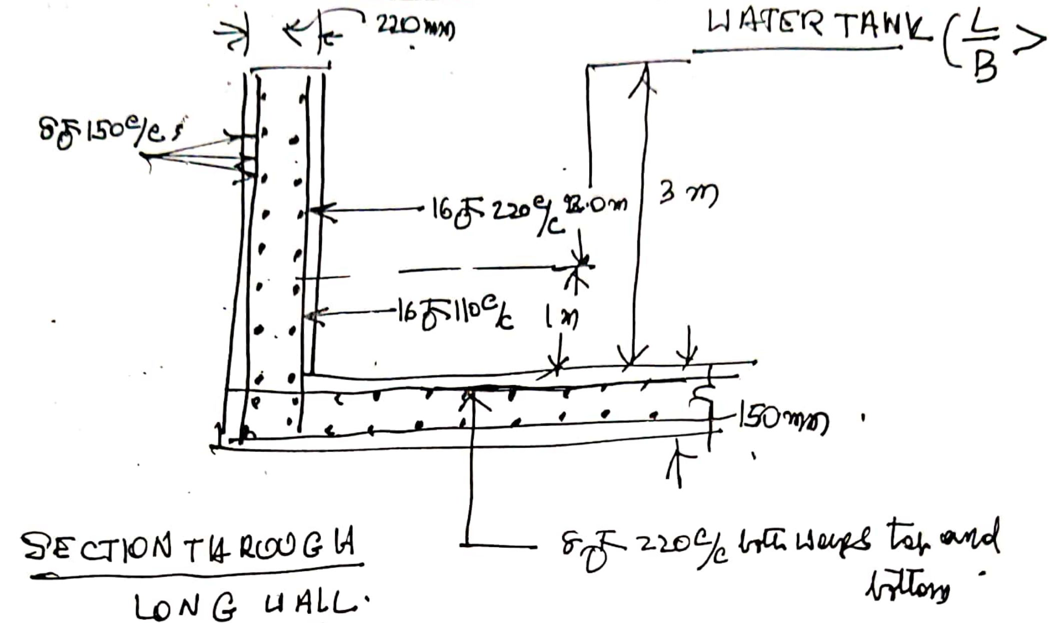
Hence provide 12ϕ @ 300 c/c. (Half of the Steel required at the ends.).

Hence, alternate bars of the end reinf. may be bent to outer face.

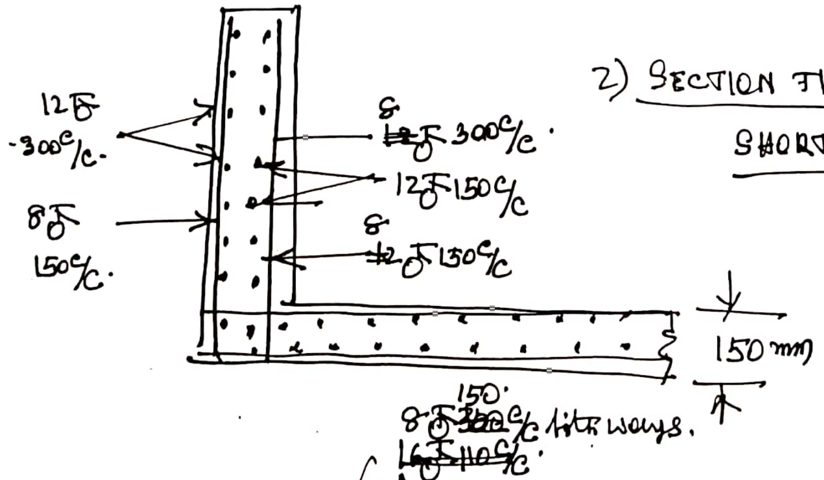
BASE SLAB.

Provide 150 mm thickness. With 8ϕ @ 220 c/c both ways at top and bottom.

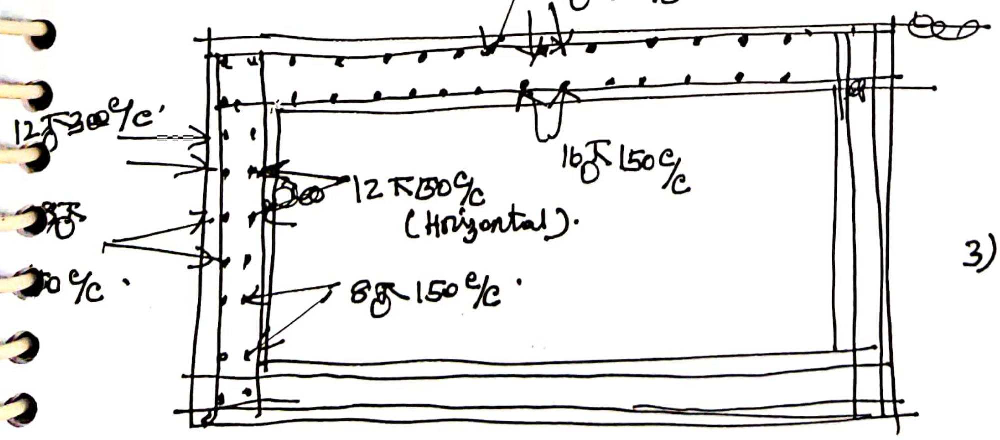
REINFORCEMENT DETAILS OF RECTANGULAR WATER TANK $(\frac{L}{B} > 2)$



1) SECTION THROUGH LONG WALL.



2) SECTION THROUGH SHORT WALL



3) SECTIONAL PLAN AT BASE

UNDERGROUND WATER TANKS.

Underground tanks are provided to store water which is received from the mains. For large capacities circular tanks and for medium to smaller capacities rectangular tanks are preferred. Form work cost is more for circular tanks.

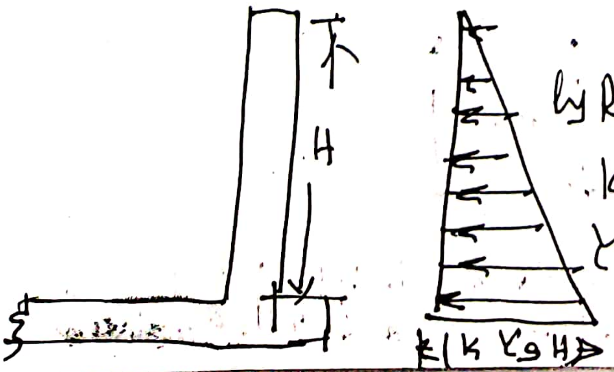
CASE I Tank is full and no earth fill.

Case II Tank is empty and earth pressure from outside.

CASE I: They are similar to tanks resting on ground. Depending upon $\frac{L}{B}$ ratio the tank can be treated accordingly.

CASE II.

a) If the outside of the tank is filled with dry soil or wet cohesionless soil.



The active earth pressure is given by Rankine's theory = $p_h = K\gamma_s H$.

K: Rankine's Coeff.

γ_s: Unit Wt. of Soil; H: Height.

Rankin's earth pressure Coeff = $K = \dfrac{1 - \sin\phi}{1 + \sin\phi}$,

where 'ϕ' is angle of repose of soil.

b) If the back fill is Saturated, then the unit weight of Cohesionless ~~Soil and also~~ Saturated soil and its pressure are considered. In addition water pressure up to full height is also considered.

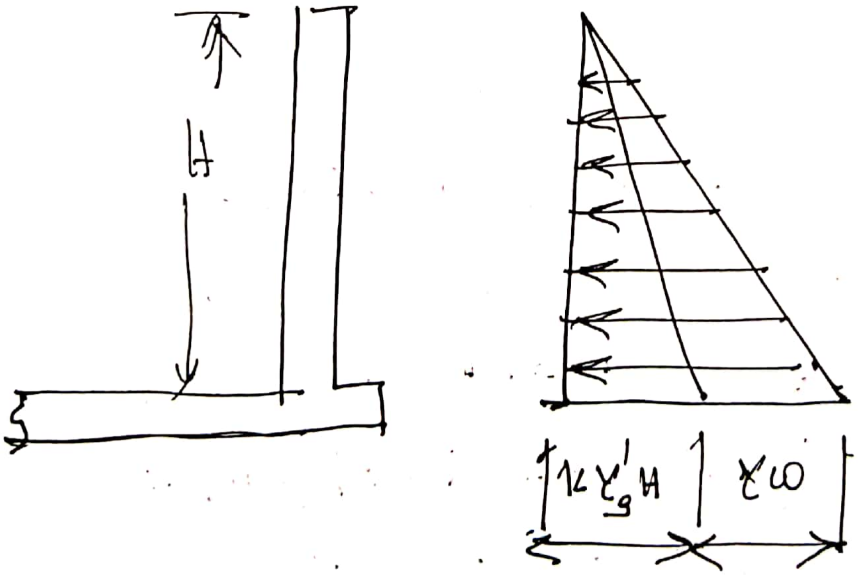
If γ_s' is the unit wt. of Saturated soil and

γ_w is " " water, then the pressure

on the wall from outside is $p_h = K\gamma_s' H + \cancel{K\gamma_\omega H}\ \gamma\omega$.

(Due to Saturated earth) (Due to water)

or ~~$p_h = K$~~



In this Case, the bottom slab is Subjected to uplift Water pressure and hence to balance, projections to the base beyond the walls are given.

In addition to underground tanks also require a roof slab which is designed like any other slab.

DESIGN EXAMPLE Design an underground rectangular water tank of size 3m × 8m × 3m. for the following data. The type of soil is submerged sandy soil. Water table can rise up to ground level. Take $\gamma_s = 16 KN/m^3$, $\phi = 30^0$.

Grade of Concrete : For tank : M25

For roof slab : M20.

Grade of steel : Fe 415

Unit weight of water : 9.8 KN/m^3, Live load on the roof slab. = 2KN/m^2.

SOLUTION.

ROOF SLAB

Roof slab is not in contact with water. Hence the stresses are normal as for building elements. If WSD is followed we can take $\sigma_{st} = 230$ for HYSD or $\sigma_{st} = 140$ for mild steel. For roof slab LSD also may be used.

$d = \frac{L}{25} = \frac{3000}{25} = 120 mm$, Provide D = 150 with d = 120 mm.

Self wt. of Slab = $0.15 \times 1 \times 1 \times 25 = 3.75\ kN/m^2$.

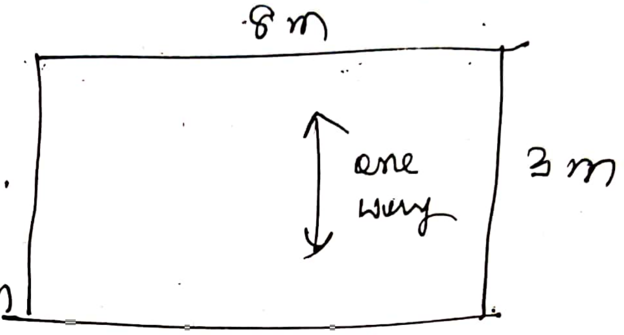
$L.L = 2.0\ kN/m^2$.

$F.L = 0.5\ kN/m^2$.

Total load = $6.25\ kN/m^2$

Since $\frac{L}{B} = \frac{8}{3} > 2$, the slab is 'one way'.

Hence $M = \frac{wl^2}{8}$



$M = \frac{6.25 \times 3^2}{8} = 7.03\ kNm.$

$M_u = 1.5 \times 7.03 = 10.55\ kNm$

(factored) $M_u(Lim) = 0.38\ f_{ck} bd^2 = 0.38 \times 20 \times 1000 \times 120^2$

$= 39.444 \times 10^6\ Nmm = 39.444\ kNm.$

Hence M_u (Lim) > M_u (actual); Hence underreinforced.

We have $M_u = 0.87 f_y A_{st} d \left[1 - \frac{A_{st}}{bd} \cdot \frac{f_y}{f_{ck}}\right]$.

Substituting, $10.55 \times 10^6 = 0.87 \times 415\ A_{st} \times 120 \left[1 - \frac{A_{st}}{1000 \times 120} \times \frac{415}{20}\right]$

This becomes,

$$A_{st}^2 - 5783.13 A_{st} + 243.5 \times 5783.13 = 0.$$

Solving, we get $A_{st} = 254.7\ mm^2$.

Min. steel for slab = $\frac{0.12}{100} \times 1000 \times 120 = 144\ mm^2$ only.

Using 10ϕ, Spacing $= \frac{79}{254.7} \times 1000 = 308$ mm.

Hence, provide 10ϕ 300 c/c along the width.

Distribution steel at 0.12% $= 144\,mm^2$

Using 8ϕ Spacing $= \frac{50}{144} \times 1000 = 349$ mm.

Provide 8ϕ 300 c/c in the length direction.

DESIGN OF WALLS. (By working stress method to avoid leakage problem).

We have $\sigma_{cbc} = 8.5\,kN/mm^2$, $m = 11$, $\sigma_{st} = 150\,N/mm^2$.

$$n = \frac{m\,\sigma_{cbc}}{m\,\sigma_{cbc} + \sigma_{st}} = 0.384, \quad j = 1 - \frac{n}{3} = 0.872$$

$$R = \frac{1}{2}\sigma_{cbc}\,n\,j = \frac{1}{2} \times 8.5 \times 0.384 \times 0.872 = 1.423.$$

~~Lo~~ Long wall

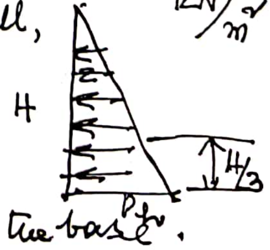
a) When the tank is empty, earth pressure governs the thickness.

p_h (Submerged Soil) $= K\gamma_s' H + \gamma_\omega H$.

$$K = \frac{1 - \sin\phi}{1 + \sin\phi} = \frac{1 - \sin 30^0}{1 + \sin 30^0} = \frac{1}{3}, \quad \gamma_s' = \gamma_s - \gamma_\omega = 16 - 9.8 = 6.2\ kN/m^3$$

Considering the long wall,

Hence $p_h = \frac{1}{3} \times 6.2 \times 3 + 9.8 \times 3 = 35.6\,kN/m^2$.



Hence $M = \frac{1}{2} \times 35.6 \times 3 \times \frac{3}{3} = 53.4$ kNm about the base.

$$d = \sqrt{\frac{53.4 \times 10^6}{1.423 \times 1000}} = 193 \text{ mm}.$$

Provide $d = 195$ mm with $D = 195 + 35 = 230$ mm.

$$A_{st} = \frac{53.4 \times 10^6}{150 \times 0.672 \times 195} = 2094 \text{ mm}^2.$$

20 mm φ, Spacing $= \frac{\pi/4 \times 20^2}{2094} \times 1000 = 150$ mm.

Hence provide [illegible] 20φ 150 c/c on the outer face Vertically.

Alternate ~~bars~~ bars may be curtailed at a point where the B.M is half.

Considering a depth of 'h', We have $\frac{h^3}{H^3} = \frac{1}{2}$, $H = 3$ m.

Hence, We get $h = 2.38$ m., Hence above the base at ~~0.62~~ 0.9 m.
(Above the base $= 0.62$ m)

(Actually adding 12 × dia of the bar $= 0.62 + 12 \times 20 = 0.86$ m, say 0.90 m)
half the bars may be curtailed and the remaining may be continued towards top.

b) When tank is full and no earth pressure

Considering the long wall (Treated as cantilever over the entire height when $\frac{L}{B} \geq 2$).
Cantilever action

Water pressure on the inside over the full height

$= p_w = \gamma_\omega H = 9.8 \times 3 = 29.4 \text{ kN/m}^2.$

Cantilever moment $= \frac{1}{2} \times 29.4 \times 3 \times \frac{1}{3} \times 3 = 44.1$ kNm.

$$A_{st} = \frac{44.1 \times 10^6}{150 \times 0.672 \times 195} = 1729 \text{ mm}^2$$

Spacing of 16φ $= \frac{\pi/4 \times 16^2}{1729}$ = 106 mm, Provide 110 c/c.

Horiz. bars in long walls (Min. steel is usually Sufficient.)

Min. percentage of steel

$$= 0.3 - 0.1\left(\frac{230-100}{450-100}\right) = 0.263$$

[0.3% is provided up to 100 mm.
For 100 mm to 450 mm, it is $0.3 - \frac{(t-100)}{(450-100)}$]

Hence, $A_{st.} = \frac{0.263}{100} \times 230 \times 1000 = 604 \text{ mm}^2$

Steel required on each face = 302 mm^2

Spacing of 8ϕ = $\frac{\pi/4 \times 8^2}{302} \times 1000 = 166 \text{ mm}$.

Provide horizontally on each face @ 160 c/c.

Check for Direct tension

$T_L = \gamma_\omega (H - h) \frac{B}{2}$ [transferred from short wall over a height of $(H - h)$]

$= 9.8 (3-1) \frac{3}{2} = 29.4 \text{ KN}$.

Steel required = $\frac{29.4 \times 1000}{150} = 196 \text{ mm}^2 < 604 \text{ mm}^2$ (Min. Steel)

Hence Same amount of distribution steel as already provided is Sufficient.

Design of Short wall.

Lower $\frac{H}{4}$ or 1m. is treated as Cantilever.

When tank is empty, considering the earth pressure from outside,

we have, p_h = ~~35~~ 35.6 kN/m² (Already calculated due to saturated soil).

$M = \frac{1}{2} \times 35.6 \times 1 \times \frac{1}{3} = 5.933$ kNm.

(over 1m ht.)

$A_{st} = \frac{5.933 \times 10^6}{150 \times 0.872 \times 195} = 232$ mm².

Considering the case, tank is full and no earth pressure,

$p_h = 9.8 \times 3 = 29.4$ kN/m²

$M = \frac{1}{2} \times 29.4 \times 1 \times \frac{1}{3} = 4.833$ kNm. (Bottom 1m is cantilever.)

Hence provide minimum steel of 8 mm ϕ 160 c/c vertically on both faces.

In the remaining $(H - h = 2m)$ portion.

~~P~~ Horizontal pressure from outside $p_h = k\gamma_s'(H-h) + \gamma_\omega(H-h)$

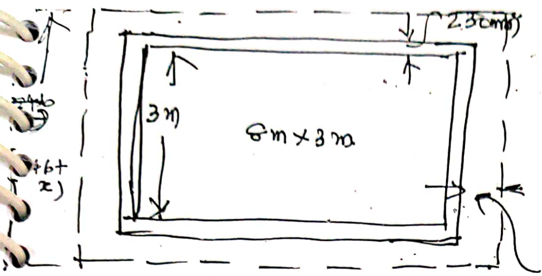
$= \frac{1}{3}(16 - 9.8)(3-1) + 9.8(3-1) = 23.73$ kN/m².

Due to frame action, Moment at support $= \frac{23.73 \times 3^2}{12} = 17.8$ kNm (B = 3m)

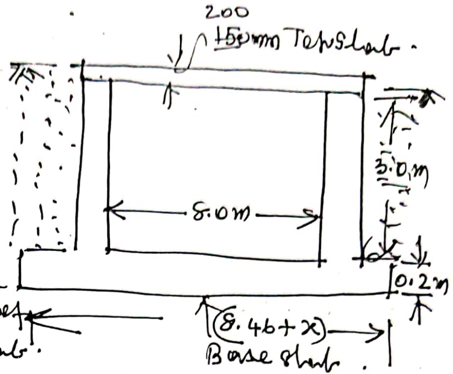
$A_{st} = \frac{17.8 \times 10^6}{150 \times 0.872 \times 195} = 698$ mm², At Mid span $A_{st} = \frac{698}{2} = 349$ mm²

Hence provide 10ϕ @ 110 c/c on the out face. At mid span take 10ϕ (alternate) is bent towards inside.

BOTTOM SLAB.



a) PLAN



b) SECTION.

Assuming a thickness of 0.2m for the bottom slab,

$$H = 3.0 + 0.2 = 3.2m.$$

If the soil below is saturated (sandy), then the upward pressure

$$= 9.8 \times 3.2 = 31.36 \, kN/m^2 \uparrow$$

The bottom slab is given a projection beyond the tank walls allround so that there is an additional downward load of the soil which helps in the stability of the tank against upward floatation.

Downward loads on the base

1) Wt. of top slab (150mm thick) $= 0.15 \, (8 + 2 \times 0.23)(3 + 2 \times 0.23) \times 25$

$= 109.77 \, kN.$

2) Wt. of long walls $= 2 \times 0.23 \, (8 + 2 \times 0.23) \times 3 \times 25 = 291.9 \, kN.$

3) Wt. of short walls $= 2 \times 0.23 \times 3 \times 3 \times 25 = 103.5 \, kN.$

4) Wt. of bottom slab $= (8.46 + x)(3.46 + x) \times 0.2 \times 25 = (146.4 + 59.6x + 5x^2)$

5. Wt. of Soil on the projection $= \left[(8.46+2x)(3.46+2x) - (8.46\times 3.46)\right] 3\times 16$

$= (23.64x + 4x^2)\,48 = 1144.32x + 192x^2$

uplift pressure on the bottom slab $= 31.36\,(8.46+2x)(3.46+2x)$

$= (917.96 + 747.62x + 125.44x^2) \uparrow$

For the stability of the tank, equating downward and upward loads, we get

$$(917.96 + 747.62x + 125.44x^2) = (109.77 + 291.9 + 103.5 + 146.4 + 59.6x + 5x^2 + 1144.32x + 192x^2)$$

Hence finally, $71.56x^2 + 456.3x - 266.39 = 0.$

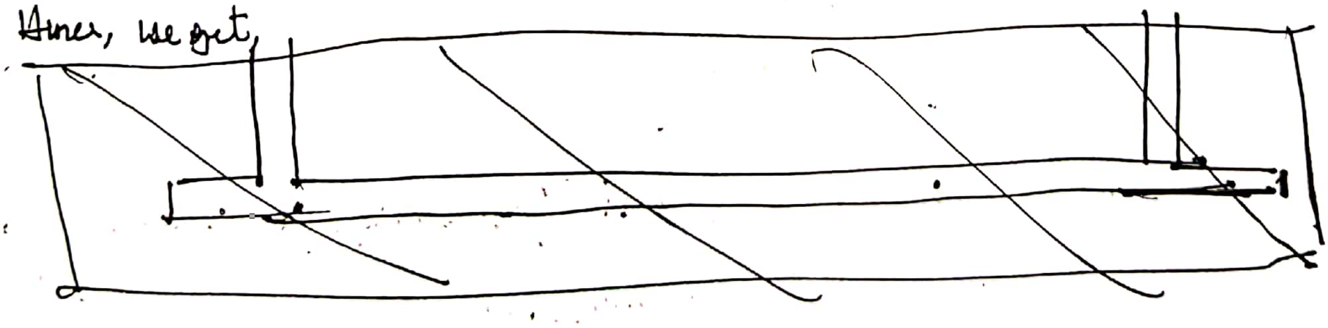
Solving the quadratic,

$$x = \frac{-456.3 + \sqrt{456.3^2 - 4\times 71.56\times 266.39}}{(2\times 71.56)}$$

Hence, $x = 0.538\ m$.

Hence, provide a projection of 0.6 m around the tank below.

Hence, we get,



Loads acting on the base slab.

Self wt : Directly transferred to the soil.

Hence, net upward pressure $= 31 \cdot 26 - 0{\cdot}2 \times 1 \times 1 \times 25 = 26{\cdot}36\ \text{kN/m}^2$

Wt. of soil on the projected portion $= 16 \times 3 = 48\ \text{kN/m}^2$

Horizontal reaction on the wall $= \frac{1}{2} P_h \cdot 3$

$= \frac{1}{2} \times 35{\cdot}6 \times 3 = 53{\cdot}4\ \text{kN}$ acting at $\frac{1}{3}$ ht $= \frac{3}{3} = 1\,\text{m}$ + Thickness of base slab

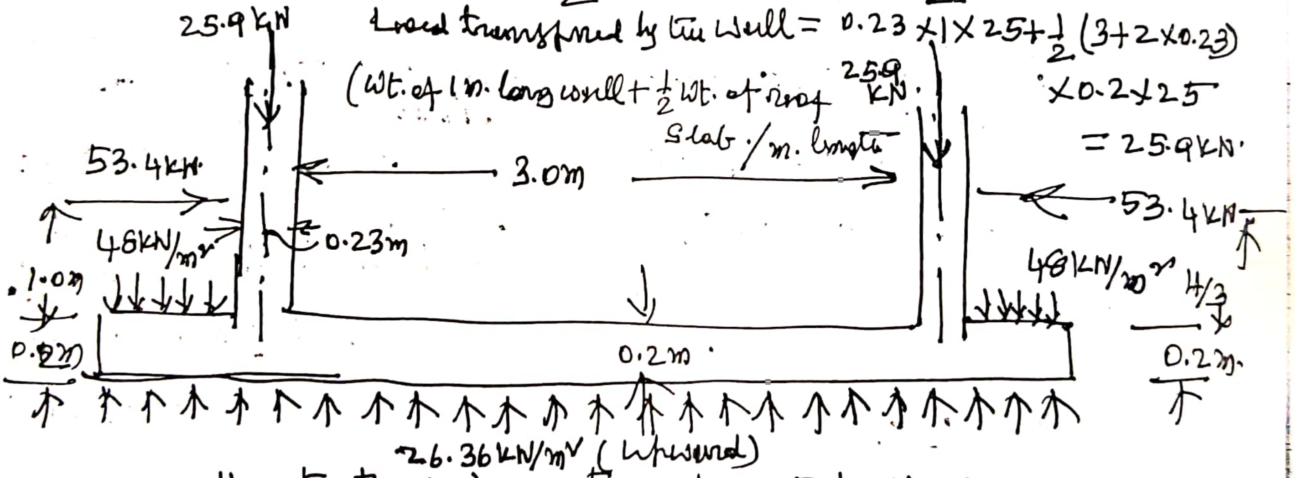
$= 1{\cdot}2\,\text{m}$ above the base.

Cantilever moment due to projection on the face of the wall due to upward pressure $= 26{\cdot}36 \times \frac{0{\cdot}6^2}{2} + 53{\cdot}4 \times 1{\cdot}2 - \frac{48 \times 0{\cdot}6^2}{2} = \underline{60{\cdot}18\ \text{kN m}}$

Load transferred by the wall $= 0.23 \times 1 \times 25 + \frac{1}{2}(3 + 2 \times 0.23) \times 0{\cdot}2 \times 25$

(Wt. of 1 m. long wall + $\frac{1}{2}$ wt. of roof slab / m. length

$= 25{\cdot}9\ \text{kN}$.



When the tank is empty and earth pressure due to submerged sandy soil.

Moment at centre of base slab.

$$= 26.36\,\frac{(3.46+1.2)^2}{\cancel{2}\ 8} + 53.4\times1.2 - 48\times0.6\left(\frac{3.46}{2}+\frac{0.6}{2}\right)$$

$$-25.9\left(1.5+\frac{0.23}{2}\right) = 25.2\text{ kNm}$$ (Produces tension at ~~top~~ bottom

Considering the critical case of tank full condition.

For the design of the base slab, we have

water pressure at $\frac{H}{3}$ = 1 m above the base = $P = \frac{1}{2}(9.8\times3)\times3 = 44.1$ kN.

Moment due to water pressure about the base = ~~44~~ × 1.2

= 44.1 × 1.2 kNm.

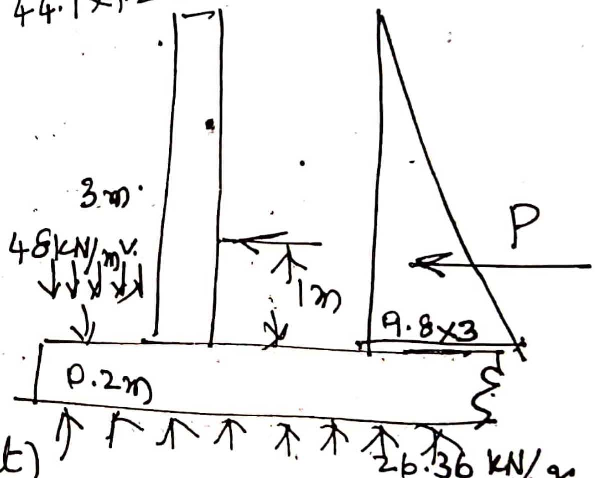
Hence, total moment about the centre

$$= 26.36\,\frac{(3.46+1.2)^2}{\cancel{2}\ 8} - 44.1\times1.2 - 48\times0.6\,\frac{(3.46+0.6)}{2}$$

(Water wt. is directly transmitted to the soil)

= − 50.23 ~~39.63~~ kNm (cantilever moment)

(Tension at top)



$P = \frac{1}{2}\times9.8\times3\times3$

= 44.1 kN.

Design Values

Max. Cantilever moment = 60.18 kNm (Reinf. at bottom)

Max. moment for tension at bottom = 25.2 kNm (Reinf. at top)

Moment at centre of slab requiring reinf. at bottom = 25.2 kNm.

Thickness of the slab required $= \sqrt{\dfrac{60.18 \times 10^6}{1.423 \times 1000}} = 205$ mm

Provide $d = 215$ with $D = 250$ mm

$$A_{st} = \frac{60.18 \times 10^6}{150 \times 0.872 \times 215} = 2139 \text{ mm}^2$$

Using 16ϕ, Spacing $= \frac{\pi/4 \times 16^2}{2139} \times 1000 = 94$ mm

Provide 16ϕ 90 c/c near bottom face for the cantilever moment.

In the middle portion, $A_{st} = \dfrac{50.23 \times 10^6}{150 \times 0.872 \times 215} = 1786$ mm², Provide 16ϕ 90 c/c at top in the middle.

~~In the middle portion~~, ~~Ast~~

At bottom, reinf. required $= \dfrac{\frac{\cancel{50.23}}{25.2} \times 10^6}{150 \times 0.872 \times 215} = 896 \text{ mm}^2$

Using 12 mm ϕ, Spacing $= \dfrac{\pi/4 \times 12^2}{896} \times 1000 = 126$ mm

Provide 12ϕ 120 c/c at ~~top~~ bottom.

Distribution steel.

$$\text{Percent.} = 0.3 - \frac{(250 - 225)}{(450 - 100)} = 0.229$$

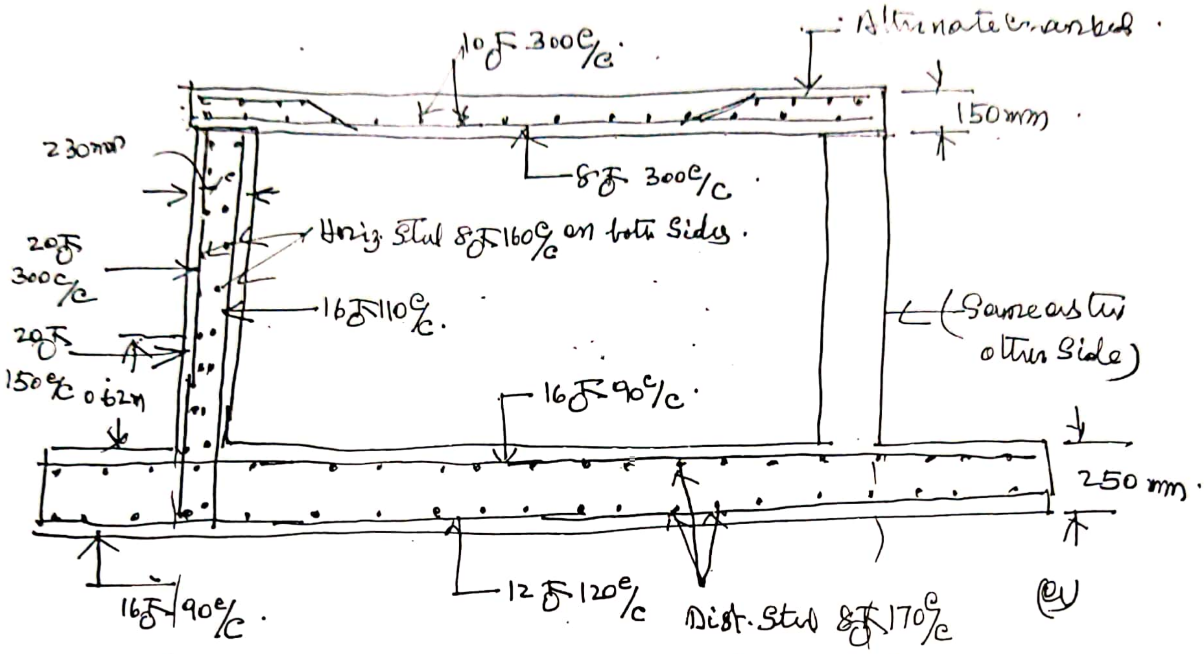
$$A_{st} = \frac{0.229}{100} \times 250 \times 1000 = 571 \text{ mm}^2$$

on each face, $A_{st} = \dfrac{571}{2} = 286 \text{ mm}^2$

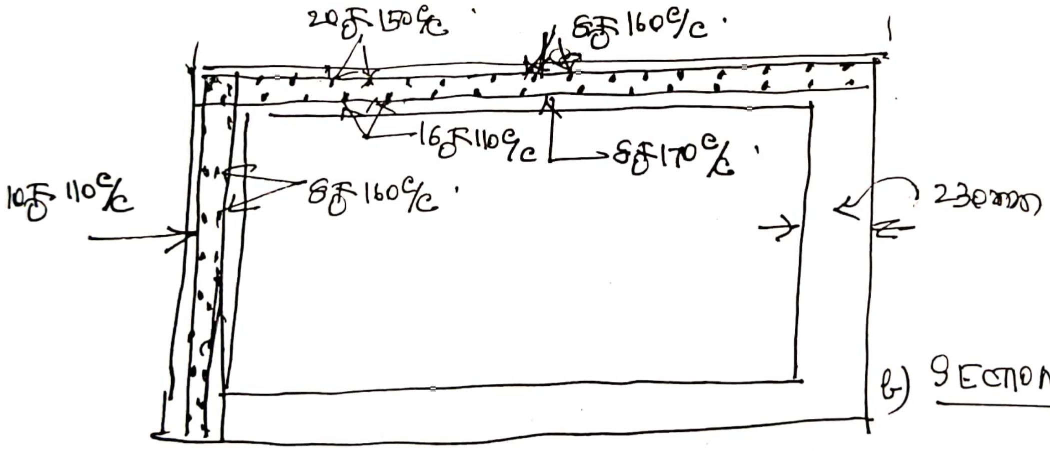
Using 8ϕ, Spacing $= \dfrac{\pi/4 \times 8^2}{286} \times 1000 = 175$ mm.

Hence, provide 8ϕ 170 c/c in the longitudinal direction on both faces.

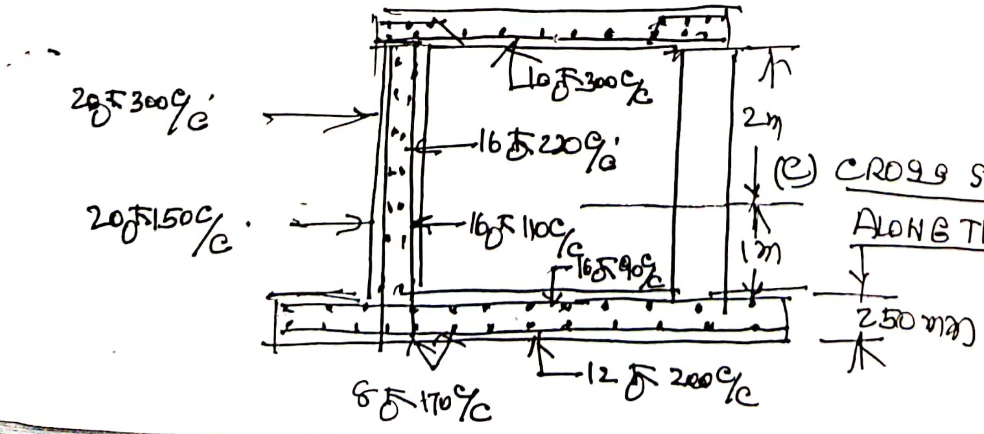
REINF. DETAILS OF UNDERGROUND TANK.



a) SECTION ALONG THE LENGTH.



b) SECTIONAL PLAN



(c) CROSS SECTION ALONG THE SHORT DIRECTION

OVERHEAD WATER TANKS.

Rectangular

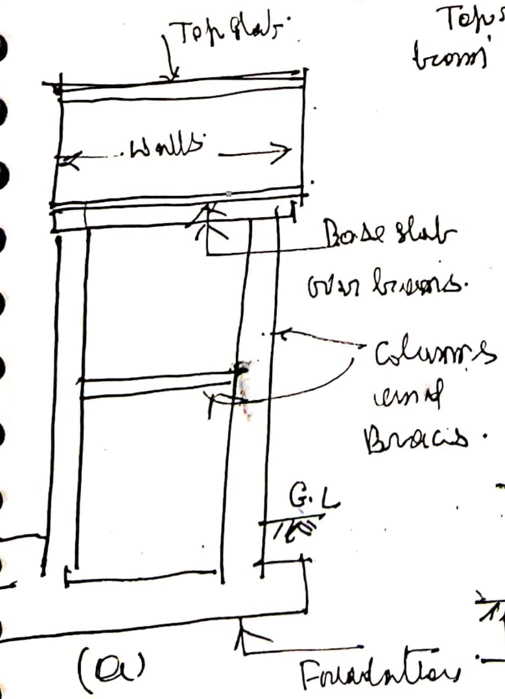
Provided for smaller capacities like 50,000 to 75,000 lit. Convenient for apartmental buildings. Designs can be carried out applying the same principles of tanks resting on ground with rigid base. Top slab may be provided.

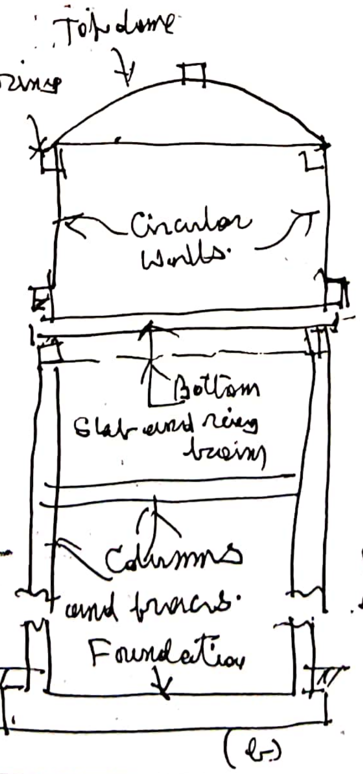
Circular.

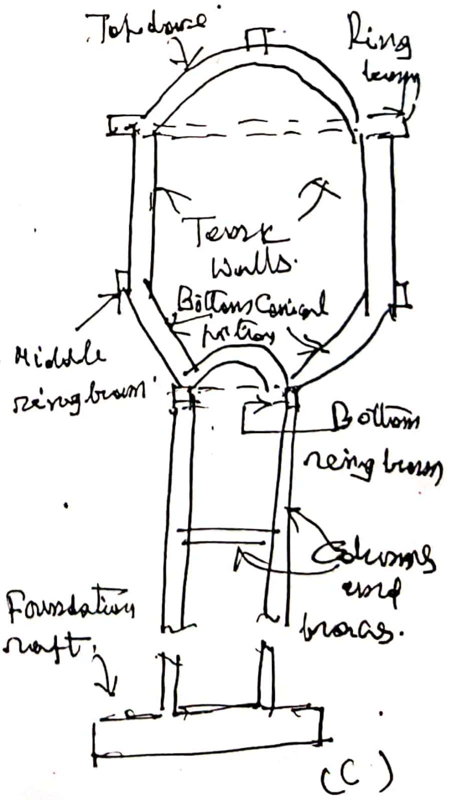
For larger capacities like upto 7,50,000 lit. Provided with a top dome and bottom flat slab. All the elements are continuous. Provided at a height supported over columns, braces and foundation.

Intz.

Very large capacities like more than 1 million lit. Circular in cross section with domes at top and bottom resting on ring beams. Columns, braces and foundations are provided as usual.

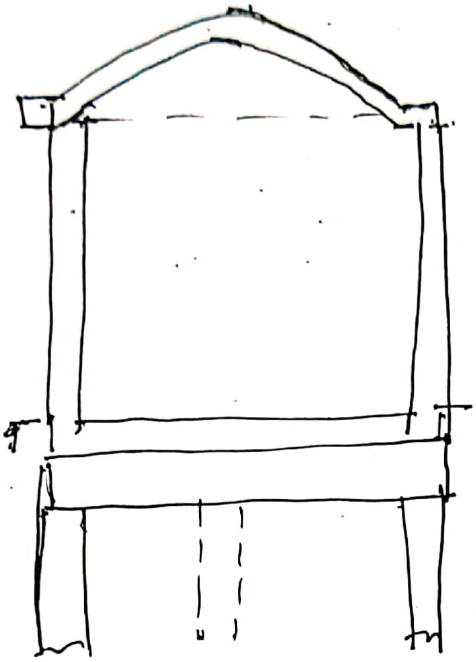






OVERHEAD CIRCULAR TANKS

(ELEVATED) (Approximate Method).



a) Section of the tank

TOP DOME.

D= Diameter

R= Radius of curvature.

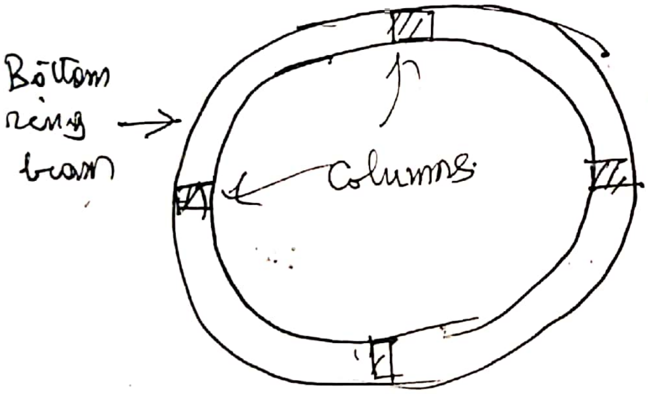
h: Rise of dome, usually = $\frac{D}{7}$

t: Thickness of dome taken from 75 to 100 mm.

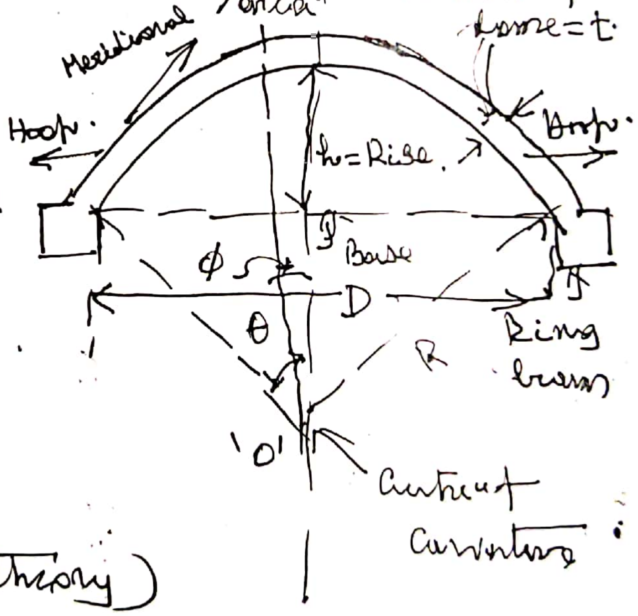
D.L and F.L are considered /m².

L.L may be taken as 1.5 kN/m².

ω: Total load/unit area.



Plan at the base.



For the dome (By membrane theory)

$$T_1 = \text{Meridional thrust} = \frac{\omega R}{1-\cos\phi} \;/\text{unit length}$$

$$T_2 = \text{Circumferential force} = \omega R\left[\cos\phi - \frac{1}{(1+\cos\phi)}\right] \Big/ \text{unit length}$$

Max. values of above forces occur at $\phi = \theta$ (i.e) at the junction of the dome with the ring beam.

Ring beam

It is subjected to load from meridional thrust 'T_1'

Hence, hoop tension in the ring beam = $\cos\theta \cdot \frac{D}{2}$.

Hence, tensile stress produced in concrete, reinforcement required against direct tension are determined

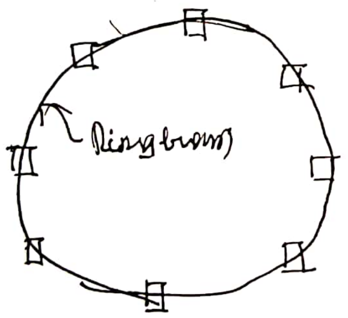
Tank walls

By approximate method, bottom $\frac{H}{3}$ or 1m is designed as cantilever.

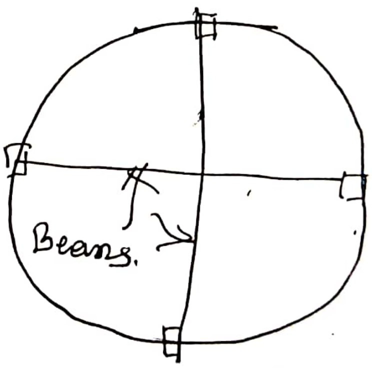
Tank walls are also designed for hoop tension over the entire height

Base slab.

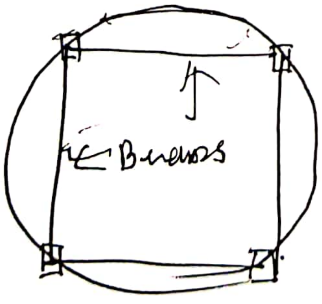
It is a circular slab supported over beams and columns.



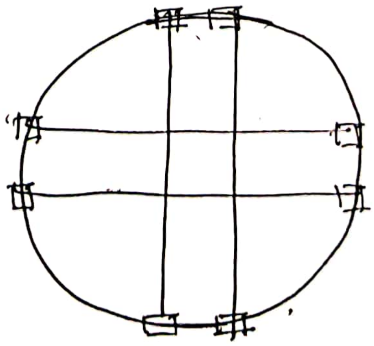
a) Ring beam and columns.



(b) Cross beams.



c) 4 Nos of beams



d) Beams in the form of a Grid.

CIRCULAR TANK SUPPORTS: (ELEVATED).

1) Tank Supported on walls. (For Smaller tanks)

The ~~cir~~ bottom circular slab of the tank is assumed to be simply supported. The moments in the circular slab are,

$$M_r \text{ (Radial)} = \frac{q}{16}(3+\mu)(a^2-r^2) = \frac{3q}{16}(a^2-r^2) \quad (\mu=0)$$

a: Radius of the circular tank

q: UDL on the bottom slab = $\gamma_\omega H$ + Self wt.

μ: Poisson's ratio of Concrete. (It is small and can be assumed as zero.)

r: Any radius of a section, a: Radius of the slab.

$$M_\theta \text{ (Circumferential)} = q\cdot\frac{a^2}{16}(3+\mu)\frac{r^2}{16}(1+3\mu)$$

$$= \frac{3qa^2}{16} - \frac{qr^2}{16} = \frac{q}{16}\left[3a^2 - r^2\right]$$

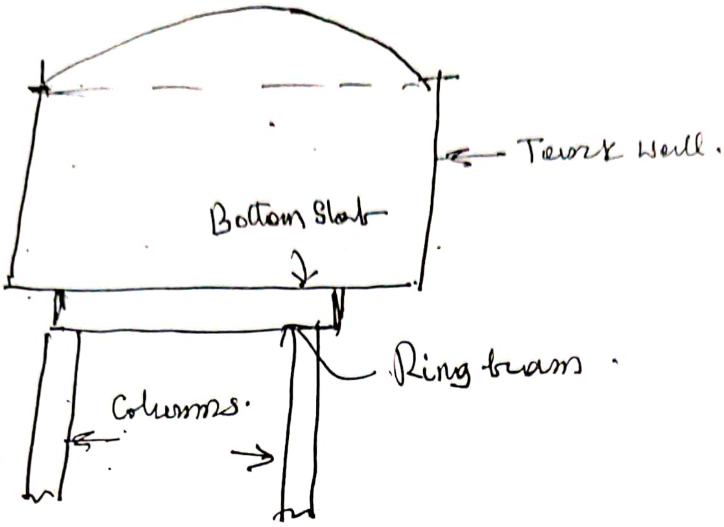
Tank wall is designed as a beam to support its own wt. Additional steel is provided at top and bottom of the tank wall.

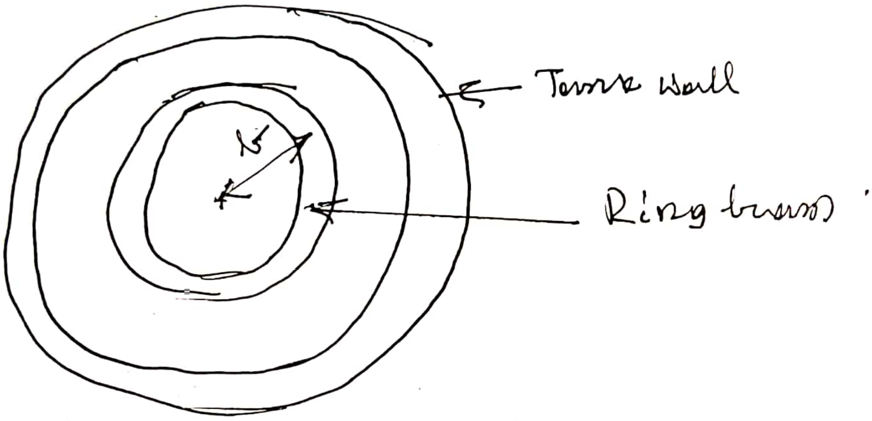
2) Tank Supported on ring beam (For larger tanks)

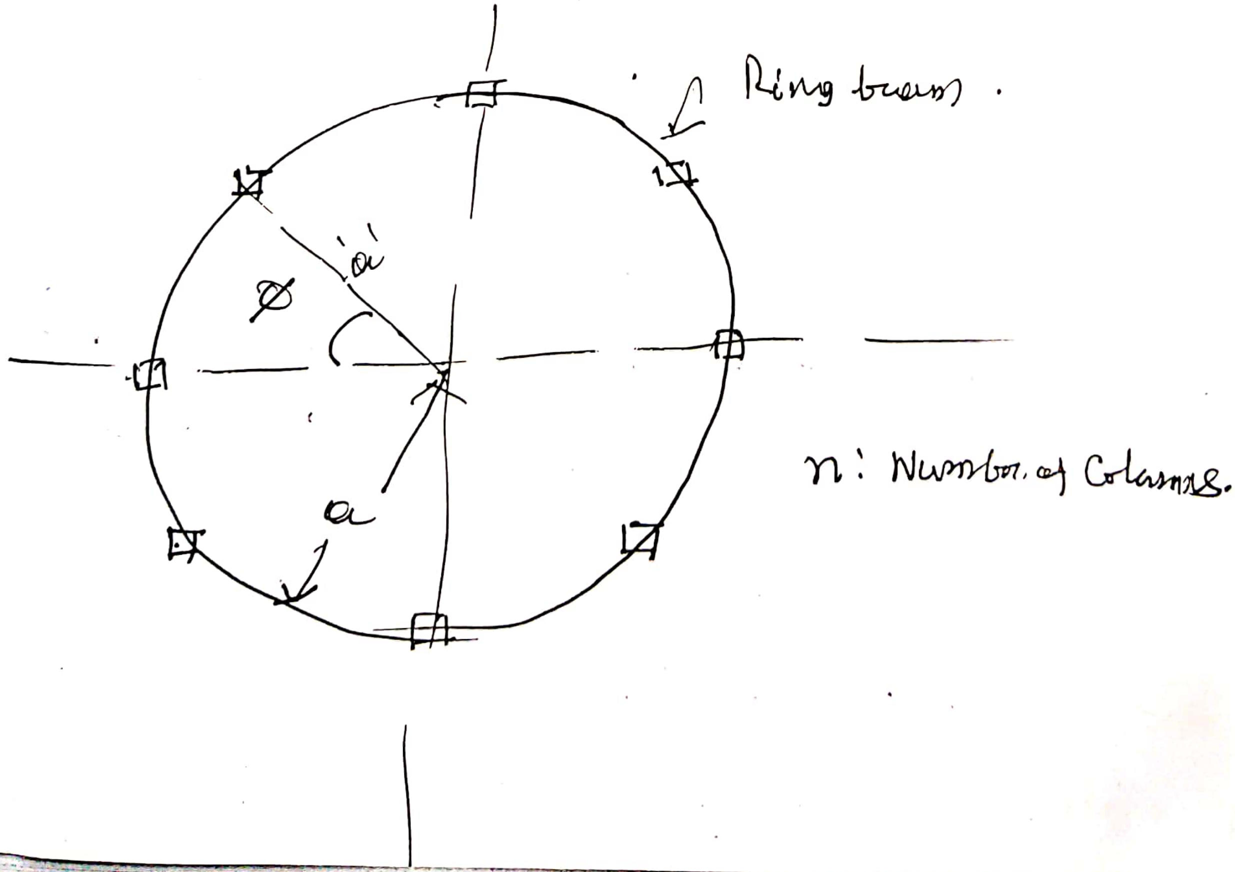
For the ring beam supporting the tank, provide a diameter equal to 0.75 times the dia. of the ~~Slab~~ tank.

The ring beam is supported on number of columns.

'ϕ' is the angle subtended at the centre by the arc in between any two columns. 'n' is the number of columns.


Tank wall.
Bottom Slab
Ring beam.
Columns.


Tank wall
R
Ring beam.


Ring beam.
'a'
Ø
a
n : Number of Columns.

On this, the total load on the ring beam from the slab = 'W'.

b = radius of the ring beam.

~~To~~ For tank bottom slab. (Radius = a)

For ~~r~~ $r < b$

$$M_r = M_\theta = \frac{W}{8\pi}\left[2\log\frac{a}{b} + 1 - \left(\frac{b}{a}\right)^2\right]$$

For $r > b$

$$M_r = \frac{W}{8\pi}\left[2\log\frac{a}{r} - \left(\frac{b}{a}\right)^2 + \left(\frac{b}{r}\right)^2\right] \left(= 0 \text{ if } r = a\right)$$

$$M_\theta = \frac{W}{8\pi}\left[2\log\frac{a}{r} - \left(\frac{b}{r}\right)^2 + 2 - \left(\frac{b}{a}\right)^2\right]$$

FOR ~~the~~ ring beam

No. of Columns	ϕ (Degrees)	K	K'	K''	α for Max. torsion
4	90	0.137	0.070	0.021	19.25
5	72	0.108	0.054	0.150	15.25
6	60	0.089	0.045	0.009	12.75
8	45	0.066	0.030	0.005	9.33
10	36	0.054	0.023	0.003	7.30
12	30	0.045	0.017	0.002	6.25

$\phi = 360/n$.

R: radius of the ring beam.

Shear force at support $= \frac{\omega R\phi}{2}$ (ω: UDL).

Support moment $= K\omega R^2\phi$

Mid span Moment $= K'\omega R^2\phi$

Max. torsional moment $= K''\omega R^2\phi$.

'α' ~~ϕ~~ : Angle for max. torsion

The values of K, K', K'' and 'α' are taken from the table.

3) If the slab is supported on beams.

Depending upon the arrangement of beams, the slab panel between the beams is designed.

EXAMPLE: Design a flat bottom circular elevated water tank of diameter 10m and total height 4m. It is supported on a ring beam of 7.5m which is supported over 6 Nos. columns equally spaced. Use M25 and Fe415.

Design a) Top dome b) top ring beam c) Cylindrical wall d) bottom slab e) bottom ring beam.

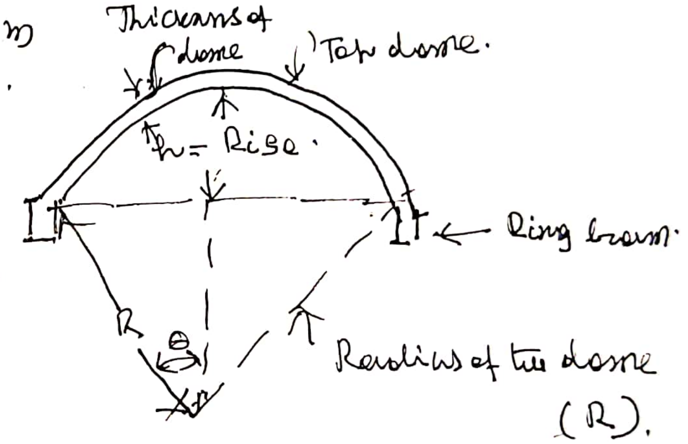
SOLUTION.

EXAMPLE: Design a flat bottom Circular elevated Water tank of diameter 10m and total height 4m Supported by a ring beam of 7.5m diameter: The ring beam is Supported by 6 Nos of Columns equally placed: Use M25 and Fe 415 steel. Design a) Top dome b) Top ring beam c) Cylindrical wall, bottom slab, bottom ring beam.

SOLUTION: **TOP DOME**

Dia. of the tank = 10m

Radius 'a' = 5m (Tank).



R: Radius of dome

h: Rise of dome = $\frac{1}{7}D = \frac{10}{7} = 1.43$ Say 1.5m

Take h = 1.5m.

Hence, $(2R-h)h = \frac{D}{2} \times \frac{D}{2} = \left(\frac{D}{2}\right)^2$

Hence $(2R-1.5)1.5 = 5^2 = 25$, $R = \left[\frac{25}{1.5} + 1.5\right]\frac{1}{2} = 9.083m.$

Semicentral angle θ is given by, $\cos\theta = \frac{5}{9.083}$, $\theta = 33.4^\circ$.

Assuming the thickness of the dome shell as 75mm,

Self wt. of dome = $0.075 \times 1 \times 1 \times 25 = 1.875\ kN/m^2$

L.L = $1.5\ kN/m^2$, F.L = $0.5\ kN/m^2$

Total load on the dome = 'w' = $3.875\ kN/m^2$.

Max. meridional thrust $T_1 = \frac{\omega R}{1+\cos\theta}$

$$= \frac{3.825 \times 9.083}{1+\cos 33.4^\circ} = 19.184\ kN/m$$

Circumferential force $T_2 = \omega R\left[\cos\theta - \frac{1}{1+\cos\theta}\right]$

$$T_2 = 3.875 \times 9.083 \left(\cos 33.4^\circ - \frac{1}{1+\cos 33.4^\circ}\right)$$

$T_2 = 10.202\ kN/m$.

Permissible Comp. stress in M25 Concrete = $6\ N/mm^2$.

Max. stress actually produced $= \frac{19.18 \times 10^3}{75 \times 1000} = 0.256\ N/mm^2 < 6.0$.

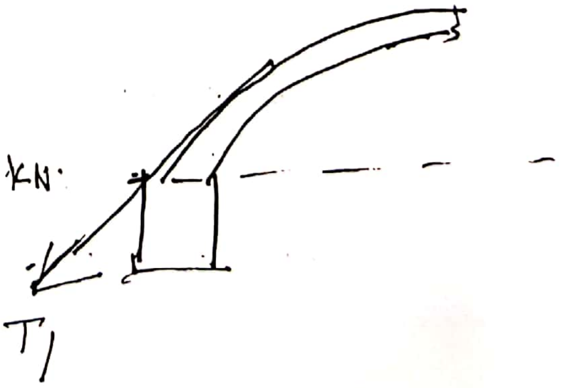
Hence Safe.

Hence, provide only nominal reinf. of 8 Tor 160 c/c for both meridional and circumferential directions.

TOP RING BEAM

Hoop tension $= T_1 \cos\theta \frac{D}{2}$

$= 19.18 \times \cos 33.4 \times 5 = 80.062\ kN$.



Hence, $A_{st} = \frac{80.062 \times 10^3}{150.0}$.

$A_{st} = 533\ mm^2$, Provide 6 Nos 12φ (A_{st} act. $= 678\ mm^2$).

$m = 11$, Area of concrete required $= \frac{80.062 \times 10^3}{(A_c + 11 \times 678)} = 1.3$

Solving, $A_c = 54{,}122\ mm^2$

Provide 250×300 top ring beam with 6 nos 12ϕ.

Nominal Stirrups of 2L 6 mm ϕ 225 c/c are provided uniformaly.

TANK WALL:

SOLUTION DATA.

Dia. of the tank = 10 m , Radius 'a' = 5 m.

H = 4 m ; Dia. of the bottom ring beam = 7.5 m.

Radius of the ring beam 'b' = 3.75 m.

Concrete Mix: M25, Steel : Fe 415.

TOP DOME

$D = 10\,m$, Rise 'h' $= \frac{D}{7}$ Say 1.5 m.

R: radius of dome; Then, $(2R - h) = \left(\frac{D}{2}\right)^2$.

$(2R - 1.5)\,1.5 = 5^2$, Hence $R = \frac{5^2 + 1.5^2}{2 \times 1.5} = 9.083\,m$.

Semicentric angle $= \theta = \cos^{-1}\frac{(R-h)}{R} = \cos^{-1}\frac{(9.083 - 1.5)}{9.083}$

$\theta = 33.4^0$.

Assuming a thickness of 75 mm for the dome,

Self wt. $= 0.075 \times 1 \times 1 \times 25 = 1.875\,kN/m^2$.

$LL = 1.5\,kN/m^2$

$FL = 0.5\,kN/m^2$

Total load 'w' $= 3.875\,kN/m^2$

Max. meridional, ~~moment~~ thrust (at $\cancel{\theta =}\,\phi = \theta$) $= T_1 = \frac{wR}{1 + \cos\phi}$ $(\phi = \theta)$ $= \frac{3.825 \times 9.083}{1 + \cos 33.4} /m$

Hence T_1 (Max.) $= 19.18\,kW/m$.

Max. Circumferential force $= T_2 = wR\left[\cos\theta - \frac{1}{1 + \cos\theta}\right]$

$= 3.875 \times 9.083\left[\cos 334 - \frac{1}{1 + \cos 334}\right] = 10.202\,kN/m$.

~~[illegible]~~

Hence, Merid. stress produced = $\frac{19.18 \times 10^{3}}{1000 \times 75}$ = $0.256\ N/mm^2$.

(As the load is acting on the surface, the nature of the stress is Compressive)

Permissible Comp. stress for M25 = $6\ N/mm^2$, Hence Safe.

This requires only nominal reinf. in both the directions

Hence, provide 8Ø 180 c/c both ways.

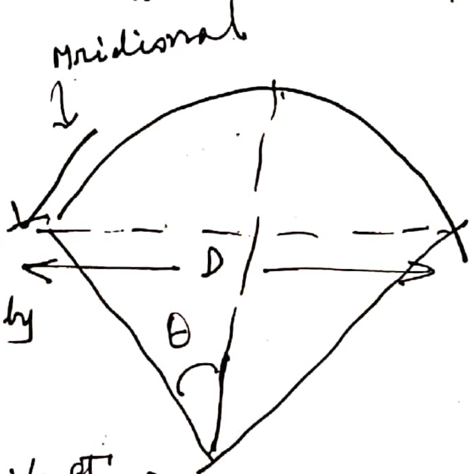
TOP RING BEAM.

Load is transferred from meridional force to the ring beam.
(Causing hoop tension)

$= T_1 \cos\theta \frac{D}{2} = 19.18 \cos 33.4 \times \frac{10}{2} = 80.062\ kN$

Hence $A_g = \frac{80.062 \times 10^{3}}{1500} = 533\ mm^2$

Provide 6 NO S12 Ø.



m = 11, Area of concrete required is given by

$= \frac{80.062 \times 1000}{A_c + 11 \times 678} = 1.3$ (Allowable tensile stress)

Hence, $A_e = 54,122\ mm^2$.

Provide 250 × 300 mm top ring beam with 6 NO S12 Ø reinf.

Stirrups of 6 mm Ø 2L @ 225 c/c are provided.

(c) TANK WALL.

Depth of water tank = 4m, Dia. of water tank = 10m

Max. hoop tension in the wall = $\frac{\gamma h D}{2} = \frac{9.8 \times 4 \times 10}{2} = 196 \text{kN/m}$

A_{st} (Required) = $\frac{196 \times 10^3}{150} = 1306 \text{mm}^2$

A_{st} (on each face) = $\frac{1306}{2} = 653 \text{mm}^2$. Provide 12φ 170 c/c on each face in the form of rings. ($A_{st\,act.} = 665 \text{mm}^2$)

Thickness of the wall is given by,

$$\frac{196 \times 1000}{1000t + 11 \times 665} = 1.3, \text{ Hence } t = 168.7 \text{mm}.$$

Provide t = 200mm ; with effective = 165 mm

Bottom one third = $\frac{4}{3} = 1.333$m (Designed as Cantilever).

Cantilever moment = $\frac{\gamma H h^2}{6} = \frac{9.8 \times 4 \times 1.333^2}{6} = 11.61 \text{kNm}$.

P. ~~Provide 10φ~~ $A_{st} = 538 \text{mm}^2$, $A_{st\,(Min.)}$ at 0.3% = 600 mm^2.

Hence provide 10φ 130 c/c in the lower 1.3m vertically.

Continue half the reinf. beyond this in the remaining height.

On the outer face provide 10φ 260 c/c vertically,

BASE SLAB.

Ar

Load transferred from the top dome = $T_1 \sin\theta$

Total load = $T_1 \sin\theta \cdot 2\pi D/2$

$= 19.18 \times \sin 33.4^0 \times 2\pi \times \frac{10}{2} = 331.7$ kN.

Self wt. of ring beam = $0.25 \times 0.3 \times 2\pi \times \frac{10}{2} \times 25 = 58.90$ kN.

Wt. of wall = $0.20 \times (4 - 0.3) \times 2\pi \times 5.2 \times 25 = 604.4$ kN.

(ring beam) (C/C)

Total = 995 kN.

Wt. of water = $9.8 \times 4 \times \pi \times \frac{10^2}{4} = 3078.8$ kN.

On the edge of slab, Self wt. of slab = $1 \times 1 \times 0.3 \times 25 = \underline{7.5}$ kN/m².

(Assuming slab thickness as $\frac{D}{35} = \frac{10}{35} = 0.29$ m say 0.3 m = 300 mm)

Total bottom slab dia. = $10 + 2 \times 0.2 = 10.4$ m.

(wall thickness)

Total self wt. of slab = $\frac{\pi}{4} \times 10.4^2 \times 7.5 = 637.1$ kN.

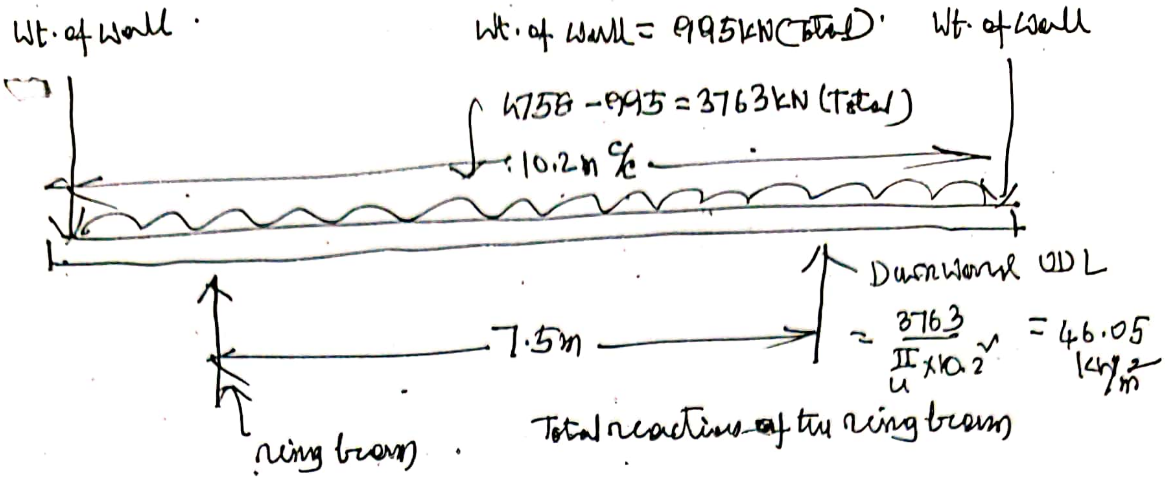
(Self wt/m²)

Finishing load (600 mm waterproofing course) = $0.6 \times \frac{\pi}{4} \times 10^2 = 47.1$ kN.

Hence, total downward load = $995 + 3078.6 + 637.1 + 47.1$

= 4758 kN.

This load is to be carried by the ring beam below the slab.



Total reaction of the ring beam

= 4758 kN.

The bottom slab is analysed by two cases.

Case I: Circular slab simply supported at its periphery by walls and subjected to $\gamma_\omega H$ + Self wt.

r = Radius of the slab = $\frac{10.2}{2}$ (c/c of the walls) = 5.1 m.

We have $M_r = \frac{3q}{16}(a^2 - r^2)$ and $M_\theta = \frac{3qa^2}{16} - \frac{qr^2}{16}$.

We have $a = \frac{10.2}{2} = 5.1$ m., UDL on the slab $= \frac{3763}{\frac{\pi}{4} \times 10.2^2} = 46.05$ kN/m² (q)

At critical points, we obtain

r in m.	0	1.875	3.75	5.1 (a = r)
M_r (kNm)	224.6	194.2	103.2	0
M_θ (kNm)	224.6	214.5	184.1	140.7

In case II. (We consider two sub cases like) (Slab supported on ring beam)

$r < 3.75 = \left(\frac{7.5}{2}\right)$.

$r > 3.75$. , The expressions are applicable.

For $r < 3.75$. (Slab subjected to total ring load of

$M_r = M_\theta =$, $W = 4758$ kN over the periphery with

$= \frac{W}{8\pi}\left[2\log\frac{a}{b}+1-\left(\frac{b}{a}\right)^2\right]$ $r = \overset{b}{D} = \frac{7.5}{2} m$) upward (-ve)

If $r > 3.75$ $M_r = \frac{W}{8\pi}\left[2\log\frac{a}{r} - \left(\frac{b}{a}\right)^2 - \left(\frac{b}{r}\right)^2\right]$

$M_\theta = \frac{W}{8\pi}\left[2\log\frac{a}{r} - \left(\frac{b}{r}\right)^2 + 2 - \left(\frac{b}{a}\right)^2\right]$

Substituting,

$r < 3.75$ (r in m) (Since $b = 3.75 m$, only this case is considered.)

r in m.	0	1.875	3.75	5.1
M_r (kNm)	-201.4	-201.4	-201.4	0
M_θ (kNm)	-201.4	-201.4	-201.4	-99.23

~~r > 375~~ Net moments are obtained by combining Cases I and II as below.
~~Net moments~~.

r	0	1.875	3.75	5.1
M_r	+23.2	-7.2	-98.2	0
M_θ	+23.2	+13.1	-17.3	+50.47

Hence highest design moment $= -98.2$ kNm (M_r)

Hence $d = \sqrt{\frac{98.2\times10^3}{1.423\times1000}} = 262.7$ mm. Provide d = 263 and D = 300 mm. (okay)

$A_{st} = \frac{98.2\times10^3}{150\times0.872\times263} = 2804\ mm^2$.

Using 25 ϕ, spacing $= \frac{\pi/4\times25^2}{2804}\times1000 = 175$ mm.

Hence provide 25 ϕ 175 c/c at Top in the radial direction.

In the circumferential dirn. we have, $M_\theta = 50.47 \text{ kNm}$ (At the edges.

$d = 265 - 25 = 240 \text{ mm}$. (Hogging).

$$A_{st} = \frac{50.47 \times 10^6}{150 \times 0.872 \times 240} = 1654 \text{ mm}^2,$$ Provide 20 ϕ 175 c/c at top at the edges of the slab.

In the central portion provide 20ϕ 300 c/c both ways in the form of a mesh at the bottom.

BOTTOM RING BEAM.

Radius = 3.75m, Total load from the slab = 4758 kN.

$$\text{Load/m} = \frac{4758}{2\pi \times 3.75} = 202 \text{ kN/m}$$

Taking 350mm width and depth $= \frac{1}{15} \times$ Dia. of the ring (for torsion)

D = 600 mm.

Self wt. of the beam $= 0.35 \times 0.60 \times 1 \times 25 = 5.25 \text{ kN/m}$.

(Say) Finishing $= 0.75 \text{ kN/m}$

Total $= 6.0 \text{ kN/m}$

No. of columns = 6

$$\phi = \frac{360}{6} = 60^\circ = \pi/3 \text{ rad}.$$

Total on the ring beam = 202 + 6 = 208 kN/m.

$$\text{Max. S.F at Support} = \frac{wR\phi}{2} = \frac{208 \times 3.75 \times \pi/3}{2} = 408 \text{ kN}.$$

Support moment $= k\,wR^2\phi = 272.6$ kNm (Factored $= 1.5\times272.6$)

Mid span moment $= k'\,wR^2\phi = \overset{137.8}{\cancel{197.8}}$ kNm (Factored $= 1.5\times137.8$)

Max. torsional moment $= k''\,wR^2\phi = 27.68$ kNm. (occurs at $\alpha = 12.75^\circ$ from the columns)

(From the table, $k = 0.089$, $k' = 0.045$, $k'' = 0.009$),

By limit state design procedure, we get.

For the Support moment

$A_{st} = 2566\ mm^2$ (8 Nos 20 ϕ at top)

$A_{sc} = 628\ mm^2$ (2 Nos 20 ϕ at bottom are provided)

For Mid span moment

4 Nos 20 ϕ at bottom.

Design for torsion.

Equivalent $M_t = T_u = T_u\left(\frac{1+D/b}{1.7}\right)$

$M_e = M_u + M_t$, S.F $= V_u$

Equivalent S.F. $V_e = \left(V_u + \frac{\overset{1.60}{\cancel{1.6}}\,T_u}{b}\right)$

Formulae.

Max. torsional moment $\therefore k''\,wR^2\phi$

$= 0.009\times207\times3.75^2\times\frac{\pi}{3} = 27.68$ kNm

Against the Max. -ve moment (272.6 kNm).

The beam (350 x 600 mm) is designed as doubly reinforced.

Using SP-16 tables (Adopting the Cover $d' = 50$ mm)
$d = 550$ mm

$$d'/d = \frac{50}{550} \text{ say } 0.1.$$

$$\frac{M_u}{bd^2} = \frac{1.5 \times 272.5 \times 10^6}{350 \times 550^2} = 3.86.$$

Referring to table 51, of SP-16, we obtain

$p_t = 1.333$, $p_c = 0.146$.

$$A_{st} = \frac{1.333 \times 350 \times 550}{100} = 2566 \text{ mm}^2.$$

$$A_{sc} = \frac{0.146 \times 350 \times 550}{100} = 281 \text{ mm}^2.$$

Provide 8 NOS of 20 mm for tensile steel and 2 NOS 20 ϕ for Comp. steel.

(A_{st} (act) = 2575 mm² at top near the Support.

A_{sc} (act) = 628 mm² at bottom near Support.)

At mid span moment = 137.8 kNm. (Nearly half the steel

Provide 4 NOS 20 ϕ at midspan at bottom. at Support.).

Against torsion, $\alpha = 12.75° = 0.2225$ rad.

Distance from the Support of the torsion Section is

$= 3.75 \times 0.2225 = 0.835$ m.

$T = 27.6$ kNm ; Factored $T_u = 1.5 \times 27.6 =$

B. Mat this Section = $272.6 - 208 \times \frac{0.835^2}{2} = 200$ kNm.

Hence, at the Section, $M_u = 1.5 \times 200 = 300$ kNm.

Equivalent Moment $M_e = M_u + M_t$.

$M_u = 300$ kNm, $M_t = T_u \left[\frac{1 + D/b}{1.7}\right]$

$M_t = (1.5 \times 27.6) \left[\frac{1 + \frac{600}{350}}{1.7}\right]$

Hence $M_e = 300 + 1.5 \times 27.6 \left[\frac{1 + \frac{600}{350}}{1.7}\right] = 366.1$ kNm.

We have M_u (at Support) $= 1.5 \times 272.6 = 408.9$ kNm.

$M_e < M_u$ (Support)

We have provided 8 Nos 20_0 at top and 2 Nos 20_0 at bottom at Support

The Same thing. Can be Continued of up to Max. torsion Section also.

<u>Shear design:</u>

S.F at Support $= 408$ kN, $V_u = 1.5 \times 408 = 612$ kN.

$\tau_v = \frac{612 \times 10^3}{350 \times 550} = 3.16$ N/mm², τ_c (Max.) $= 3.1$ N/mm².

$\tau_v > \tau_c$ (Max) .

DESIGN OF BOTTOM CIRCULAR.

Hence revise the Section. Providing 450 X 600 mm Section,

Now $\tau_v = \frac{612 \times 10^3}{400 \times 650} = 2.78 < 3.1$ $\left(A_{st}\,(\text{act.}) = 2875\ mm^2\right)$

Hence $p = \frac{2875}{400 \times 550.} \times 100 = 1.309\%$

From the tables, τ_c (Allowable) $= 0.70\ N/mm^2$.

Hence Concrete Capacity $= \tau_c b d = 0.7 \times 400 \times 550$.

Balance S.F $= V_u - \tau_c b d = 612 \times 10^3 - 0.7 \times 400 \times 550$.

$V_{us} = 455 \times 10^3\ N$.

Using 12 ф 2L, Spacing $S_v = \frac{0.87\ f_y\ A_{sv}\ d}{V_{us}}$

$$S_v = \frac{0.87 \times 415 \times 2 \times \frac{\pi}{4} \times 12^2 \times 550}{455 \times 10^3} = 98.7\ mm.$$

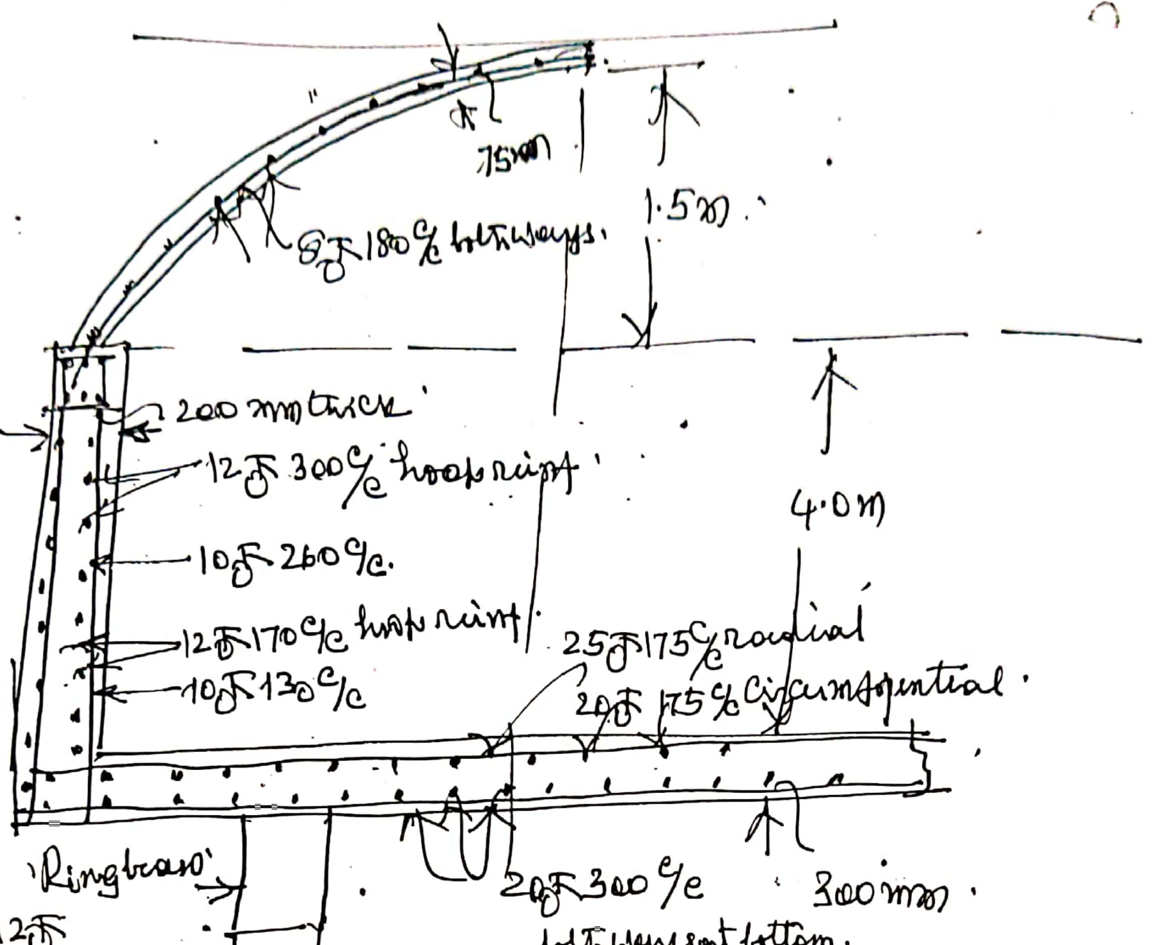
Hence provide. 12 ф 2L 95 c/c

Spacing may be increased to ~~150~~ 160mm after 1 m from Support.

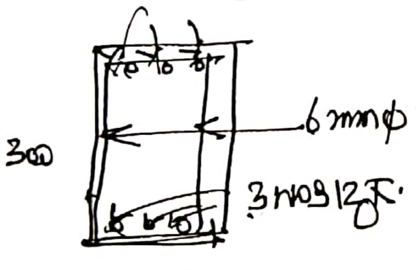
Side face reinf. @ 0.1% $= \frac{0.1}{100} \times 400 \times 550 = 229.8\ mm^2$.

Provide 1 No. 16 ф at mid depth on both Side faces.

REINFORCEMENT DETAILS.

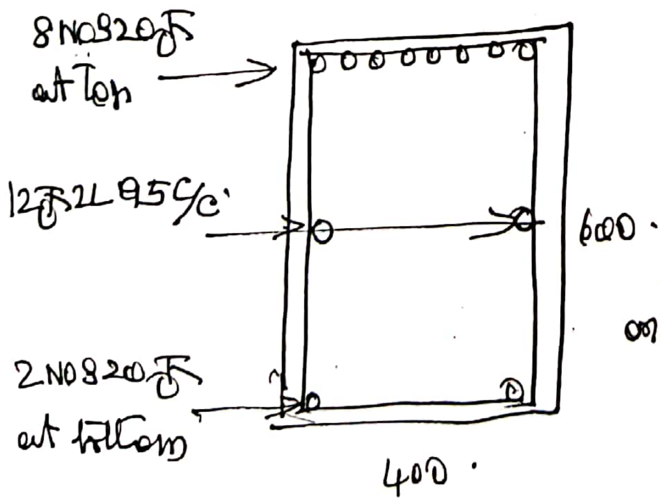


CROSS SECTION OF TANK.

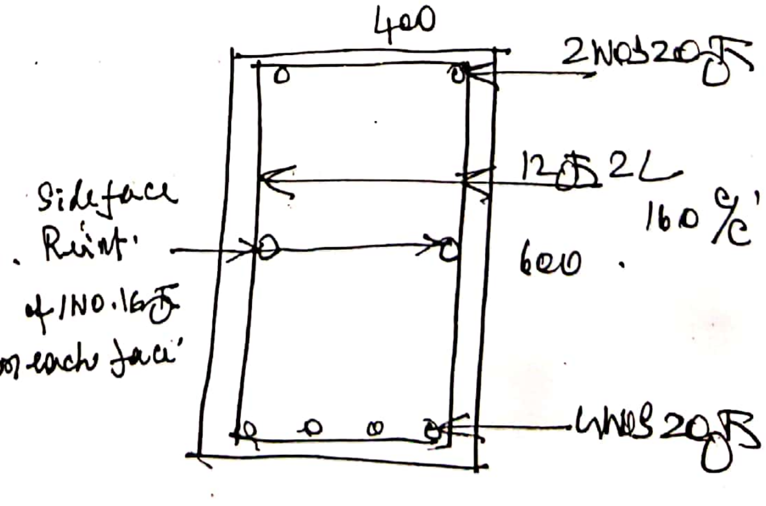


TOP RING BEAM

BOTTOM RING BEAM DETAILS.

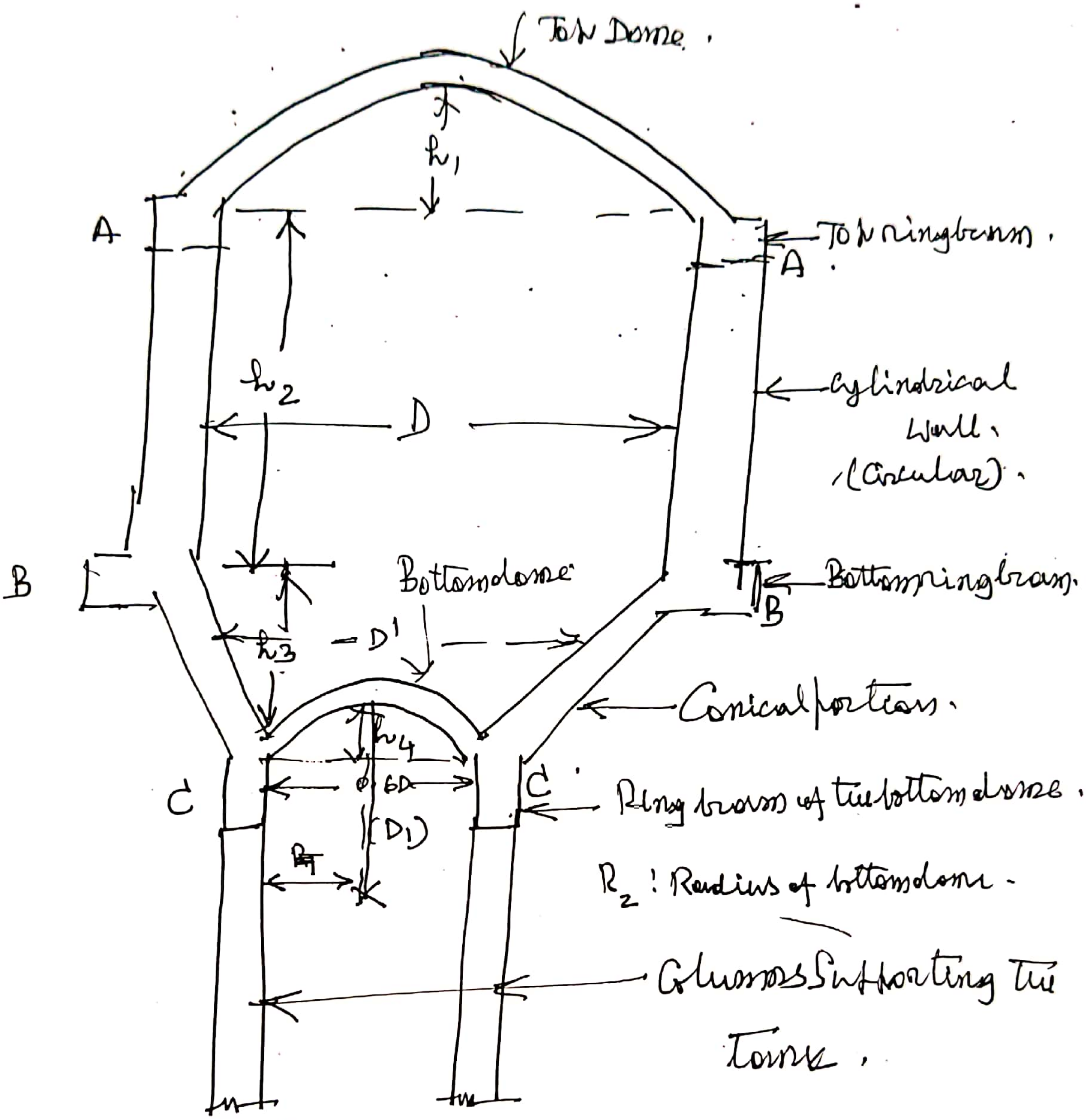


SECTION AT COLUMN SUPPORT



SECTION AT MIDSPAN

INTZ TANK.



DIFFERENT COMPONENTS OF INTZ TANK.

Usual Proportions.

Rise of top dome $h_1 = D/7$.

Ht. of cylindrical portion $h_2 = 0.4D$.

Height of Conical dome $= h_3 = 0.2D$.

Rise of the bottom dome $= h_4 = 0.4D$.

Diameter of the bottom ring beam $= 0.6D$.

DESIGN OF VARIOUS COMPONENTS:

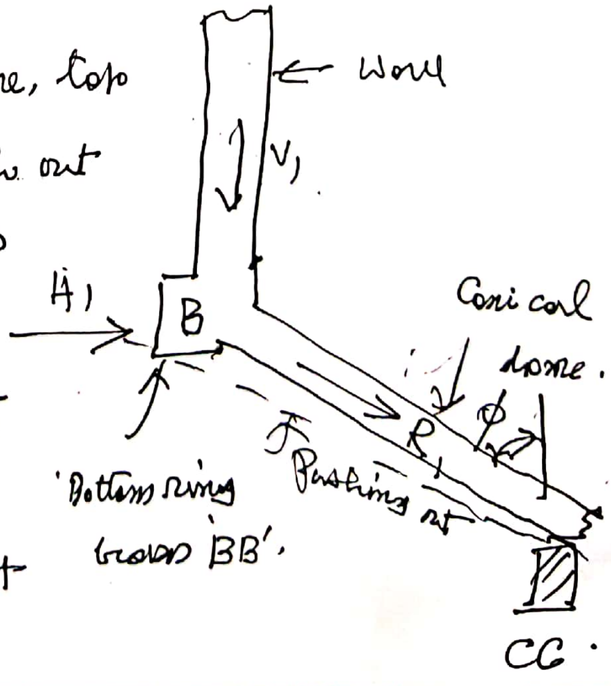
1. Design of top dome : (75 to 100mm thickness. Rise: $D/7$).

2. Top Ring beam. ; Like the previous problem.

3. Cylindrical wall : Against hoop tension completely.

4. Bottom ring beam.

V_1 = The load from the wall, top dome, top ring beam together tries to push out the ring beam 'BB'. ~~Hence it~~ It has also to ~~assist by a horiz reaction~~ resist the rotation caused by the ring beam CC. Hence the ~~[illegible]~~ ring beam is treated against V_1 and H_1.



$R_1 \sin\phi = H_1$, $R_1 \cos\phi = V_1$.

$\tan\phi = H_1/V_1$; $H_1 = V_1 \tan\phi$.

where R_1 : Resultant of V_1 and H_1

Hence, tension in the ring beam $= H_1 \frac{D}{2}$

$= V_1 \tan\phi \times \frac{D}{2}$.

Hoop tension due to horiz. water pressure $= \gamma h_2 \frac{D}{2}$

Hence total hoop tension in the ring beam B-B

$= (V_1 \tan\phi + \gamma h_2)\ D/2$

Hence, the reinf. is designed against ~~that~~ tension and the cross sectional dimensions are decided.

CONICAL DOME

This is subjected to meridional thrust as well as hoop tension.

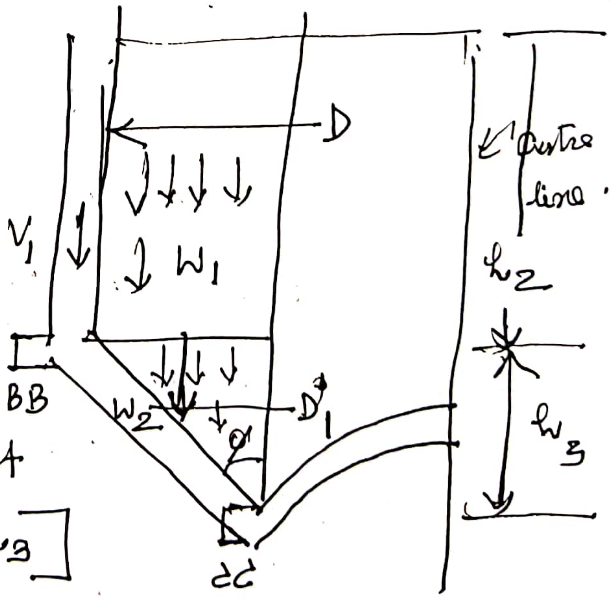
Load due to $V_1 = V_1 \pi D$.

Water load on the conical dome

$= W_1 + W_2$

$= \pi \left(\overset{(D^2 - D_1^2)}{\cancel{D^2 - D_1}}{}^{2} \right) h_2 + \gamma \times$ [Volume of frustrum of cone − volume of Cylinder of dia. D_1 and ht. h_3]

$= W_w$



$$= \gamma \left[\frac{\pi}{4} \left(D^2 - D_1^2 \right) h_{v_2} \right] + \gamma \left(D^2 + D_1^2 + DD_1 \right) - \gamma \cdot \frac{\pi}{4} D_1^2 h_{w_3}$$

Self wt. of Conical dome = $W = \pi \times$ Ave. dia. $\times$ Slanting Length
(W_s) $\times$ thickness $\times$ Unit wt. of Concrete.

$$= \pi \left(\frac{D + D_1}{2} \right) l \, \gamma_c$$
(Concrete)

Total vertical load = $\pi D V_1 + W_w + W_s$

Hence, vertical load/unit on the ring beam 'CC'

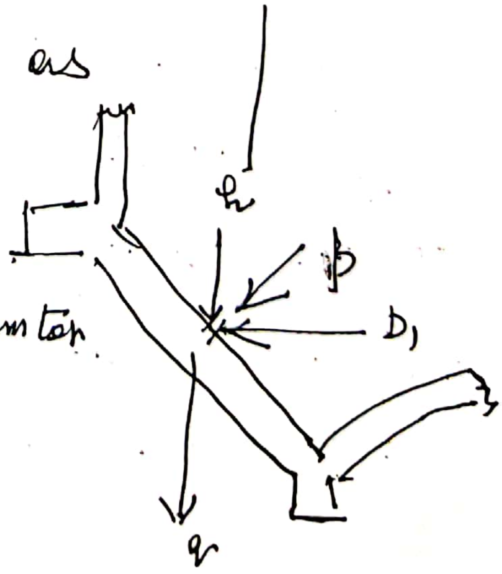
$$V_2 = \left[\frac{\pi D V_1 + W_w + W_s}{\pi D_1} \right] ~~\cos\phi~~ .$$

Hence, Meridional thrust/unit. = $V_2 \cos\phi$

$$= \frac{\left(\pi D V_1 + W_w + W_s \right)}{\pi D_1} \cos\phi .$$

Hoop tension is calculated as

$$T = \left(p \cos\phi + q \tan\phi \right) \frac{D_1}{2}$$



p : Due to Water pressure over a height of 'h' from top

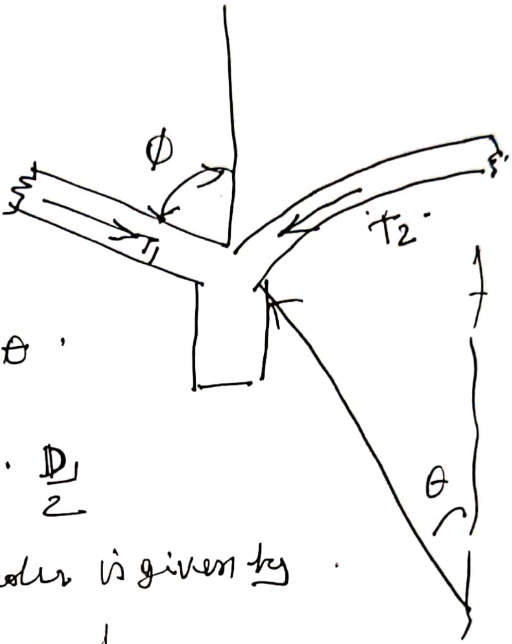
q : Self wt/unit area.

Thickness is Checked ..

BOTTOM SPHERICAL DOME

Designed for meridional and circumferential forces produced due to wt. of water and self wt.

BOTTOM RING GIRDER



Net horizontal thrust on the bottom ring girder =

$$p_h = T_1 \sin\phi - T_2 \cos\theta$$

Hoop compression = $p_h \cdot \frac{D_1}{2}$

Vertical pressure on the girder is given by

$$p_v = T_1 \cos\phi + T_2 \sin\theta \,/\, \text{Unit length}$$

Hence, the ~~+~~ ring girder is designed against these forces.

CONTINUITY.

Membrane Theory — Bending Theory

Continuity.

INTZ TANK :

DESIGN EXAMPLE : Design an intz type water tank of capacity 1 million lit. It is supported on symmetrically located 8 Nos of columns. Use M25 Concrete with Fe 415 Steel.

DESIGN DATA 1000000 lit. = 1×10^6 lit = $1000 \, m^3$.

$\sigma_{cbc} = 8.5 \, N/mm^2$, $\sigma_{ct} = 1.3 \, N/mm^2$; $\sigma_{cc} = 6 \, N/mm^2$.

$\sigma_{st} = 150 \, N/mm^2$, $m = 11$, $n = 0.384$, $j = 0.872$, $R = 1.423$.

DIMENSIONS OF THE TANK :

Let the diameter of the cylindrical portion be 'D'.

Height of the cylindrical portion = $h_2 = 0.4D$.

Dia. of the bottom ring girder = $0.6D = D_1$

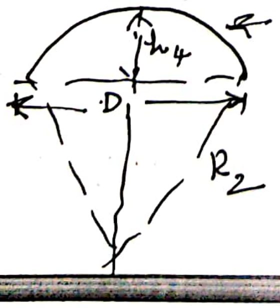
Ht. of the Conical Shell = $h_3 = 0.2D$.

Rise of the bottom dome = $h_4 = \frac{D}{7}$

Volume of the tank = Cylindrical portion + Conical portion − Volume of bottom dome.

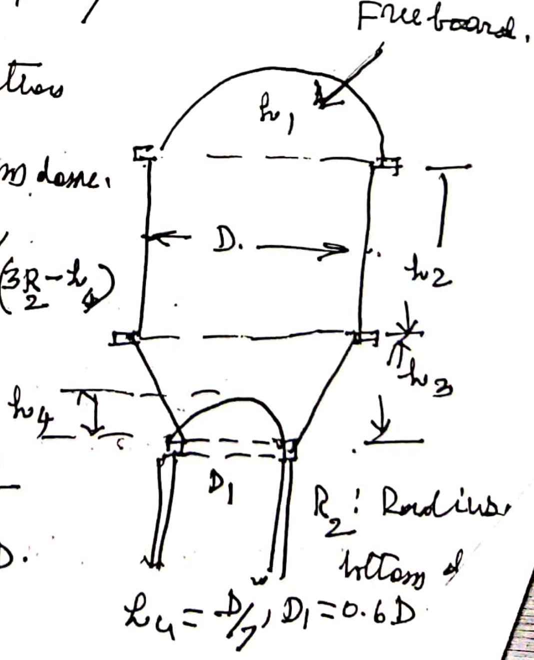
$$= \frac{\pi}{4} D^2 h_2 + \frac{\pi}{12} h_3 \left(D^2 + D_1^2 + D_1 D_2 - \frac{\pi h_4^2}{3}(3R_2 - h_4)\right.$$

$$\frac{D}{7}\left(2R_2 - \frac{D}{7}\right) = (0.6)^2 \frac{D^2}{4} = \left(\frac{0.6}{2} D_1 \times \frac{0.6 D_1}{2}\right)$$



Solving, we get

$R_2 = 0.386D$.



Substituting in the equations of volume, we get

$$V = \frac{\pi}{4}D^2(0.4D) + \frac{\pi}{12} \times 0.2D\left[D^2 + 0.36D^2 + 0.6D^2\right)$$

$$-\pi \times \frac{(0.2D)^2}{3}\left(3 \times 0.36D - D/7\right) = 0.374D^3$$

Hence $0.374D^3 = 1000$, $D = 13.88\,m$. Provide $D = 14\,m$.

$D_1 = 0.6D = 0.6 \times 14 = 8.4\,m$. Say $8.5\,m$.

$h_2 = 0.4D = 5.5\,m$, $h_3 = 0.2D = 2.8\,m$, $h = \frac{D}{4.7} = 2\,m$.

$2(2R_2 - 2) = \frac{8.5^2}{4}$, $R_2 = 5.52\,m$

Actual volume provided $= V$

$$V = \frac{\pi}{4} \times 14^2 \times 5.5 + \frac{\pi}{12} \times 2.8\left(14^2 + 8.5^2 + 14 \times 8.85\right) - \frac{\pi \times 2^2}{3}(3 \times 5.52 - 2)$$

$= 1069\,m^3$

TOP DOME.

Diameter $= 14\,m$. rise $= \frac{D}{7} = \frac{14}{7} = 2\,m$.

Radius of the arc of the dome is given by,

$2(2R_1 - 2) = 7^2$, $R_1 = 13.25\,m$.

Semicentric angle is $\phi = \cos^{-1}\left(\frac{13.25 - 2}{13.25}\right) = 31.89^\circ$.

Provide 75 mm thickness.

Self wt. $= 0.075 \times 1 \times 1 \times 25 = 1.875\,kN/m^2$

$L.L = 1.5\,kN/m^2$

$3.375\,kN/m^2$

Adding F.L, Say the total UDL on the dome $= 4\,kN/m^2$.

Meridional thrust, $T_1 = \frac{wR_1 \cdot 1}{(1+\cos\phi)} = \frac{4\times13.25}{1+\cos 31.89} = 23.66\ \text{kN/m}$.

Hence, meridional stress $= \frac{28.66\times1000}{75\times1000} = 0.382\ \text{N/mm}^2 < 6\ \text{N/mm}^2$

∴ Hence okay.

Circumferential stress $= T_2 = wR_1\left[\cos\phi - \frac{1}{(1+\cos\phi)}\right]$

$T_2 = 4\times13.25\left[\cos 31.89 - \frac{1}{1+\cos 31.89}\right] = 16.34\ \text{kN/m}$.

Stress $= \frac{16.34}{75\times1000} = 0.288\ \text{N/mm}^2 < 6\ \text{N/mm}^2$, okay.

2) Provide nominal reinf. of $0.3\% = \frac{0.3}{100}\times75\times1000 = 225\ \text{mm}^2$

Provide 8 Ø 200 c/c both ways.

TOP RING BEAM (BEAM A-A).

Hoop tension $= T_1 \cos\phi \frac{D}{2} = 28.66\times\cos 31.89\times\frac{14}{2} = 170.33\ \text{kN}$.

Hence, $A_{st} = \frac{170.33\times1000}{150} = 1136\ \text{mm}^2$.

Provide 6 Nos 16 Ø (A_{st} act $= 1206\ \text{mm}^2$).

By limiting direct tension in concrete, we get

$1.3 = \frac{170.33\times1000}{A_c + 11\times1206}$, Solving $A_c = 11{,}757\ \text{mm}^2$.

Hence, provide 400×300 mm top ring beam with 6 Nos 16 Ø

Provide nominal shear reinf. of 6 mm Ø 2L 300 c/c.

CYLINDRICAL WALL

Hoop tension at the base = $9.8 \times 5.5 \times \frac{14}{2} = 377.3\,kN$.

$$A_{st} = \frac{377.3 \times 1000}{150} = 2515\ mm^2.$$

$$16\phi \text{ on each face} = \frac{2 \times \pi/4 \times 16^2}{2515} = 159\ mm.$$

Hence, provide 16φ 150 c/c on each face. ($A_{st\ act.} = 2680\ mm^2/m$)

To determine the thickness, we have

$$1.3 = \frac{377.3 \times 1000}{A_c + 11 \times 2680}, \text{ Hence } A_c = 260750\ mm^2.$$

$$\text{Hence, } t = \frac{260750}{1000} = 260.75\ mm,$$ Provide t = 275 mm with the const. over the entire ht.

$$\text{Percentage of dist. steel} = 0.3 - 0.1\frac{(275-100)}{(450-100)} = 0.25\%$$

$$\text{Area of dist. steel} = \frac{0.25}{100} \times 275 \times 1000 = 687.5\ mm^2$$

$$8\phi \text{ on each face} = \frac{2 \times \pi/4 \times 8^2 \times 1000}{687.3} = 146\ mm.$$ Provide 8φ 140 c/c on each face ~~on~~ Vertically.

· Vertical bars of outer face are extended into the conical portion to provide continuity.

Spacing of hoop reinf. may be increased towards top.

Provide 12φ 140 c/c from 2m. above the base. This can further be increased to 240 mm towards top.

BOTTOM RING BEAM (B−B).

Load from top dome = $T_1 \sin\phi = 26.66 \sin 81.89^\circ = 15.14\,kN/m$.

Top ring beam = $0.4 \times 0.3 \times 1 \times \overset{25}{\cancel{2.5}} = 3\,kN/m$

Wt. of wall = $0.275 \times 5.5 \times 1 \times 25 = 37.81\,kN/m$

Self wt. of the ring beam (keeping 1.2m width and 600mm depth)

$= 1.2 \times 0.6 \times 1 \times 25 = 18\,kN/m$

Hence $V_1 = 15.14 + 3 + 37.81 + 18 = 73.95\,kN/m$

Slope of Conical dome with vertical is given by

$$\tan\phi = \frac{(D-D_1)/2}{h_3} = \frac{\frac{(14-8.5)}{2}}{2.8} = 0.982, \text{ Hence } \phi = 44.48^\circ.$$

Hoop tension due to V_1 = $V_1 \tan\phi \frac{D}{2} = 73.95 \times 0.982 \times \frac{14}{2} = 508.4\,kN.$

Hoop tension due to Water pressure = $9.8 \times 5.5 \times \frac{14}{2} = 377.3\,kN.$

Hence, total hoop tension in the ring beam = $508.4 + 377.3 = 885.7\,kN$

$$A_{st} = \frac{885.7 \times 1000}{150} = 5905\,mm^2.$$

Providing 20 Nos of 20ϕ, $A_{st(act.)} = \frac{\pi}{4} \times 20^2 \times 20 = 6283\,mm^2.$

Area of concrete is given by, $\frac{885.7 \times 1000}{A_c + 11 \times 6283} = 1.3$.

Hence, $A_c = 612195\,mm^2$, Hence 1200×600 mm is adequate and safe.

Provide 12ϕ 150 c/c nominal stirrups throughout.

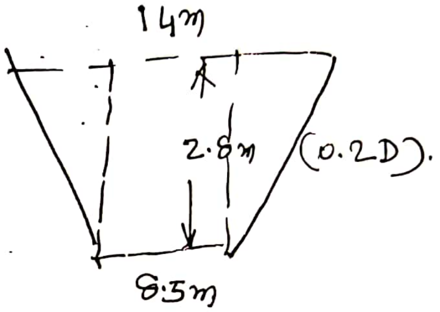
CONICAL DOME

1) Load from Cylindrical Wall = $V_1 \pi D = 73.95 \times \pi \times 14 = 3252.49\,kN$ (Total)

2) Wt. of Water = $V\gamma = \frac{\pi}{4} D^2 h_2 \gamma = 1069 \times 9.8 = 10,476.2\,kN$.

3) Self wt. of Conical dome = Slanting length $\times \pi \times$ Dia. at mid ht. $\times 0.4 \times 25$.

Slanting length = $\sqrt{\left(\frac{14-8.5}{2}\right)^2 + 2.8^2} = 3.925\,m.$



Dia. at mid ht. = $\frac{(14+8.5)}{2} = 11.25\,m$.

Hence, Self wt. = $3.925 \times \pi \times 11.25 \times 0.4 \times 25 = 1387.2\,kN$.

(400 mm thickness is assumed.)

Hence, total load = $3252.49 + 10476.2 + 1367.2 = 15115.9\,kN$.

Hence $V_2 = \frac{15115.9}{\pi \times 8.5} = 566.06\,kN/m$.

Meridional thrust = $V_2 \cos\phi = 566.06 \cos 44.46 = 403.68\,kN/m$.

Hence stress = $\frac{403.68}{400 \times 1000} = 1.01\,N/mm^2 < 6\,N/mm^2$. Okay.

At the top, hoop tension = $T = (p \cos\phi + q \tan\phi)\, D/2$.

$q = 0.4 \times 1 \times 1 \times 25 = 10\,kN/m^2$ (Self wt.)

$p = 9.8 \times 5.5 = 53.9\,kN/m$ (Due to water.)

$\phi = 44.48^0$.

Hence, $T = (53.4 \cos 44.48^0 + 10 \tan 44.48^0)\frac{14}{2} = 337.94\,kN$.

$$A_{st} = \frac{337.94\times1000}{150} = 2253\,mm^2.$$

Half on each face, $A_{st} = 1126.5\,mm^2$, Provide 12 Φ 100 c/c on both faces.

Concrete area is given by, $\frac{337.94}{A_c+11\times2253} = 1.3$.

Hence $A_c = 233.17\,mm^2$, Thickness may be finally kept at 300 mm (hrs).

At the inner edge: Provide 8 Φ 100 c/c on both faces.

BOTTOM SPHERICAL DOME.

Designed against Self wt. and Water load; $\phi = 50.38^\circ$.

Provide 250 mm thickness with 8 Φ 150 c/c both ways on each face.

BOTTOM RING BEAM

T_1: Hoop tension at the bottom edge of the Conical dome.

$$= \left(\overset{\downarrow\text{water load}}{P}\cos 44.48^\circ + \overset{\downarrow\text{Self wt.}}{10}\tan 44.48\right)\cancel{9.8}\frac{D}{2}$$

$p = 9.8(5.5+2.8) = 81.34\,kN/m$, Self wt. $q' = 0\ 4\times1\times1\times25 = 10\,kN/m$

$$T_1 = \left(81.34\cos 44.48 + 10\tan 44.48\right)\cancel{\times}\frac{8.5}{2} = 288.38\,kN/m.$$

T_2 = Meridional force coming from the bottom dome at the joint

$T_2 = 217.7\,kN/m$.

Hence, Vertical load on the beam $= (T_1\cos\phi\ T_2\sin\theta) =$

$= 288.38\cos 44.48 + 217.7\sin 50.38 = 373.5\,kN/m$.

Assuming a Section of 600×1200 mm,

Self wt. = $0.6 \times 1.2 \times 1 \times 25 = 18 \text{ kN/m}$.

Total vertical load on the ring beam = 391.4 kN/m.

There are 8 Nos. Columns. Hence, $\phi = \frac{360}{8} = 45^\circ = \pi/4$.

Hence, Max. S.F = $\frac{391.4 \times 8.5 \times \pi/4}{2} = 1306.5 \text{ kN}$.

From the tables, the various Coeff. are,

for $\phi = 45^\circ$, $K = 0.066$, $K' = 0.030$, $K'' = 0.005$, $\alpha = 9.33^\circ$.

Hence, we obtain the

Max. Support moment = 366.5 kNm, Mid span moment = 166.6 kNm.

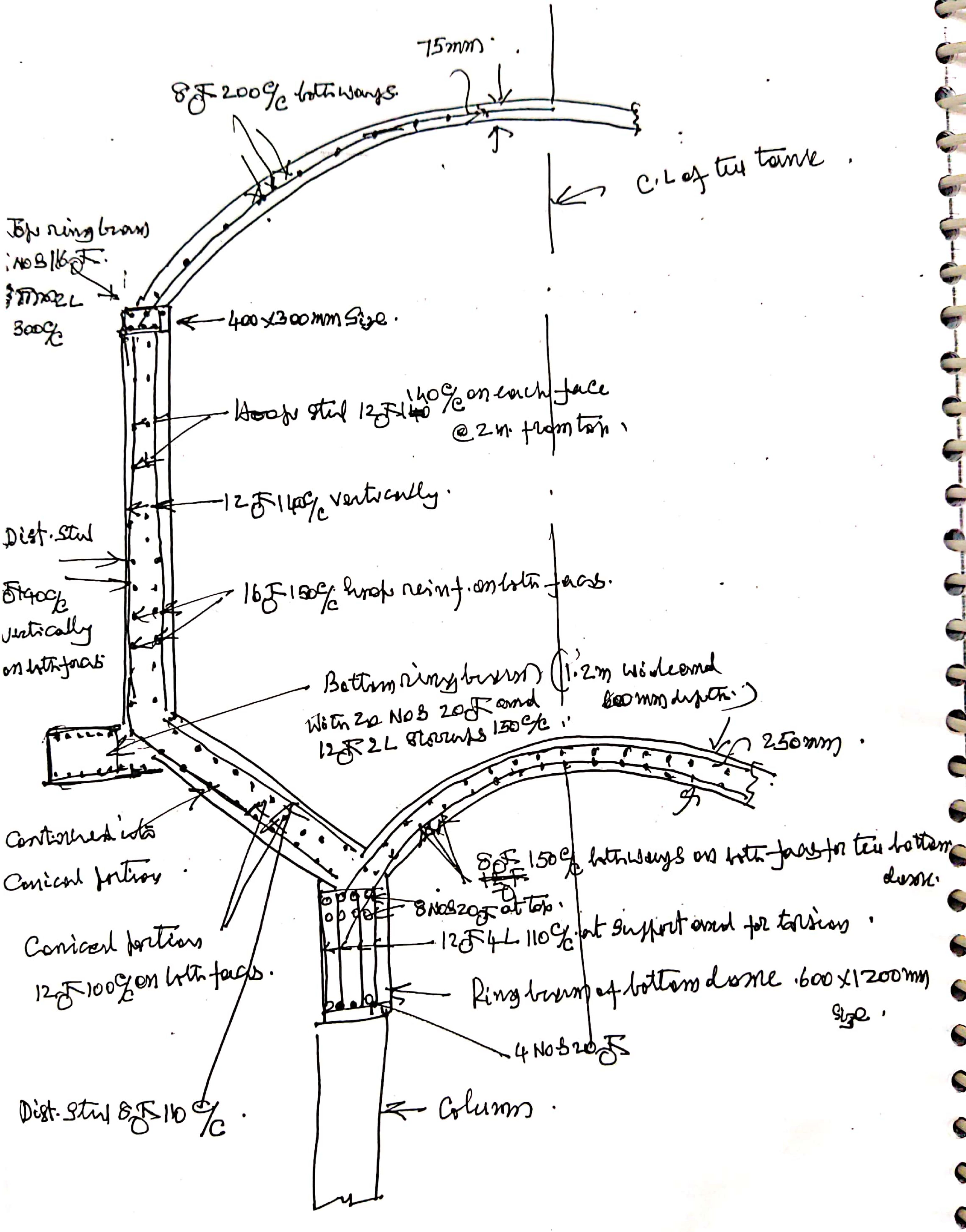
Torsional moment = 27.8 kNm at 9.33° from the Support.

Provide 8 Nos 20 mm at Support, 4 Nos 20 mm (Half) at mid span.

8 Nos. 20 mm are continued for torsion also.

12 φ 2L 100 c/c stirrups are provided. It is increased to 220 c/c towards middle.

Side face reinf. (at 0.1%): Provide 3 Nos 12 φ on each face.


75mm
8ϕ 200 c/c both ways.
C.L of the tank.
Top ring beam
NOS 16ϕ.
ϕ 2L
300 c/c
400 x 300mm Size.
Hoop steel 12ϕ 140 c/c on each face
@ 2m. from top.
12ϕ 140 c/c vertically.
Dist. Steel
ϕ 400 c/c
vertically
on both faces
16ϕ 150 c/c hoop reinf. on both faces.
Bottom ring beam (1.2m wide and 600mm depth.)
With 20 NOS 20ϕ and
12ϕ 2L stirrups 150 c/c.
250mm.
Continued into
Conical portion.
8ϕ ~~12ϕ~~ 150 c/c both ways on both faces for the bottom dome.
8 NOS 20ϕ at top.
12ϕ 4L 110 c/c at support and for torsion.
Conical portion
12ϕ 100 c/c on both faces.
Ring beam of bottom dome. 600 x 1200mm size.
4 NOS 20ϕ
Dist. Steel 8ϕ 110 c/c.
Columns.

DESIGN OF STAGING:

EXAMPLE. Design a tower of 12m height to support an intz tank (designed already), Assume wind pressure as $1.5\,kN/m^2$. Provide 8 Nos of columns symmetrically. Assume the following data:

SOLUTION.

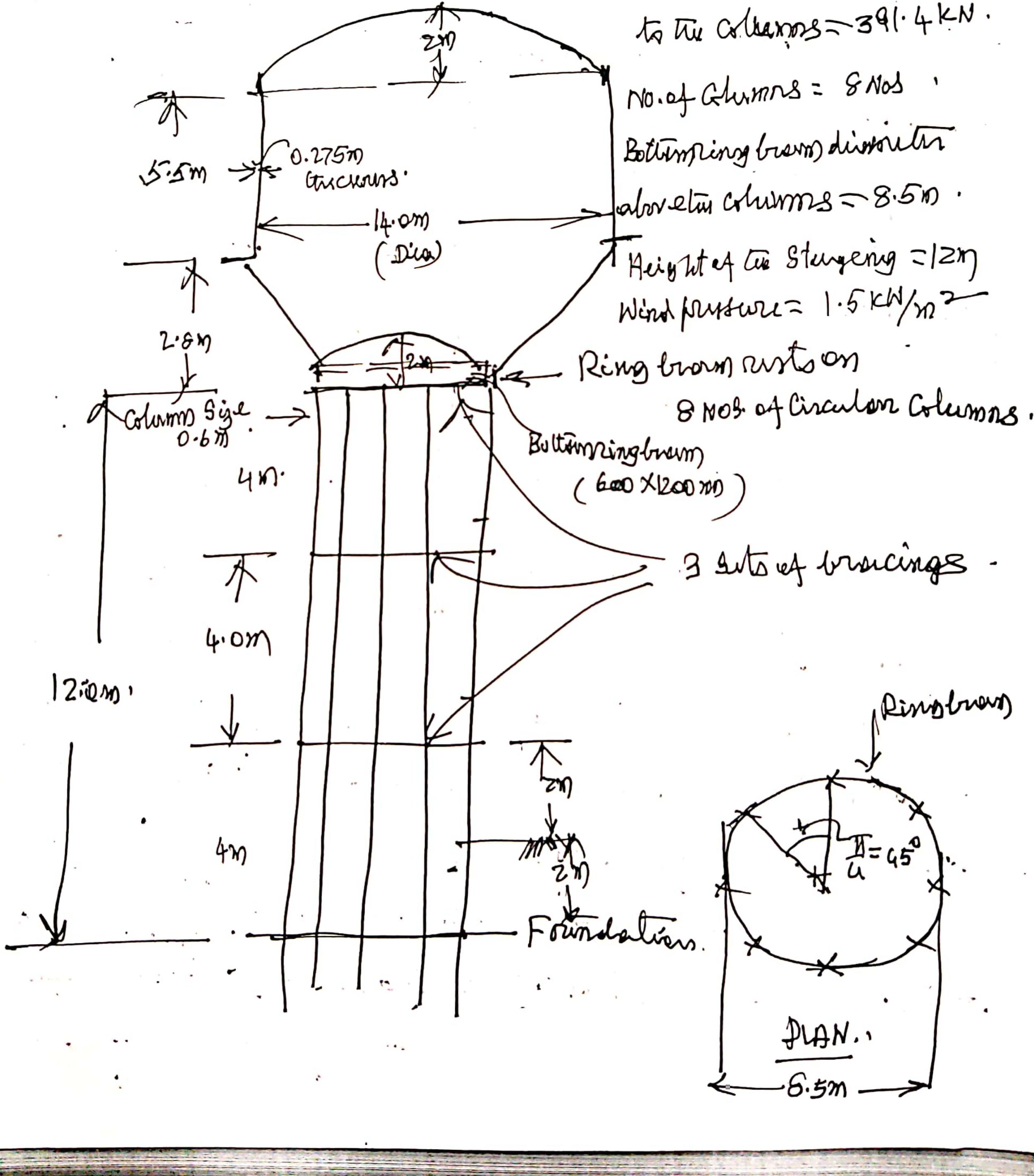
Total load from the tank on to the columns = 391.4 kN.

No. of columns = 8 Nos.

Bottom ring beam diameter above the columns = 8.5m.

Height of the staging = 12m

Wind pressure = $1.5\,kN/m^2$



Scanned with CamScanner

VERTICAL LOADS ON EACH COLUMN :

Total load from the tank = 391.4 kN/m ~~[illegible]~~ (Taken for design of ring beam).

Load on one segment (between two columns) of ring beam

$\pi/8 \times 2\pi R$ = Length of arch

= $= 391.4 \times \frac{8.5}{2} \times \pi/4 = 1306.47$ kN.

Self wt. of column (Assuming 700 mm dia) $= \frac{\pi}{4} \times 0.7^2 \times 12 \times 25 = 115.45$ kN.

Wt. of braces (3 Nos) (for 3 storeys) (Assuming 300 × 600 mm brace and length of each brace $= \frac{8.5}{2} \times \frac{\pi}{4} = 3.33$ m)

$= 3 \times 0.6 \times 0.6 \times 3.33 \times 25 = 45$ kN. ✓

Total vertical load on each column = 1306.47 + 115.45 + 45 = 1467 kN.

WIND LOADS.

Intensity = 1.5 kN/m² (given).

Take shape factor for curved surface = 0.7

Wind load on dome, cylindrical wall and conical dome

$$= 1.5 \times 0.7 \left[\frac{2}{3} \times 14 \times 2 + \underset{\text{(Ext. dia)}}{(14 + 2 \times 0.275)} \times 5.5 + \underset{\text{(Ave. dia)}}{\frac{(14.6 + 9.1)}{2}} \times 2.8 \right]$$

$= \underline{138.46}$ kN.

Centroid of all the above projected areas is found. Assuming that the total wind load is acting at 0.52D from top of bottom ring girder = 7.3 m above the top of the bottom ring girder.

Assuming the ~~ht. of Contraflexure to~~ base of the column at ~~10~~ 12 m below the bottom ring girder, the total ht. = 7.3 + 1.2 + 12 = ~~18.5~~ 20.5 m ~~above~~ above the ~~point of Contraflexure~~, base of the column.

Wind load on the bottom ring girder = 1.5×0.7×1.2(8.5+2×0.6)

= 12.2 kN (0.7 is the shape factor)

Wind load on circular columns (8 Nos) = 1.5×0.7×0.7×12×8 = 70.4 kN.
(No 8)

(0.7 is the shape factor and 0.7 is the diameter of the column.).

Wind load on the bracings (straight) = 1.5×0.3×0.6×8.5×2 = 4.6 kN.

(Projected length of 8.5m is taken for each brace for windward and leeward directions)

Hence, wind load on columns and braces = 70.4+4.6 = 75 kN

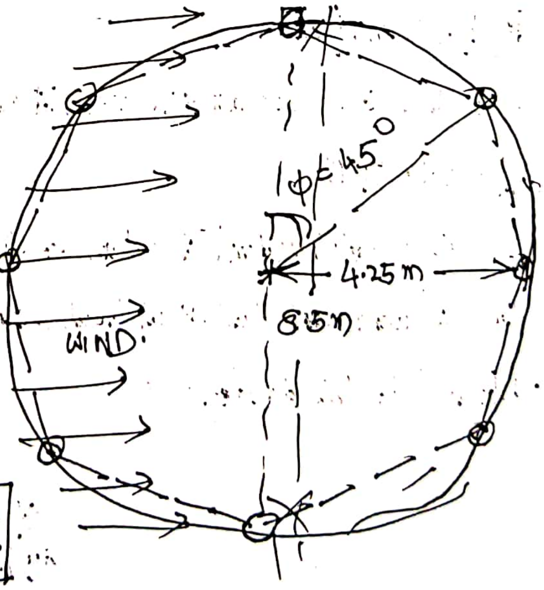
acting at the c.g taken at mid height at 6m above the base.

Hence, total wind moment at the base of all the columns of staging

= 138.46×20.5 + 12.2×12.60 + 75×6 = 3434.83 kNm.

(Due to tank) (Bottom ring beam) (columns and braces)

If 'V' is the vertical ~~force~~ produced in each column and taking moments about the c.L of the column group.



$$\text{Total moment} = \frac{V}{r}\Sigma a^2$$

Where 'a' is the distance of the centre of the column from the c.L. in the horiz. direction

Hence,

$$3434.83 = \frac{V}{\frac{8.5}{2}}\left[2\times4.25^2 + 4\times\left(\frac{4.25}{\sqrt{2}}\right)^2 + 2\times0\right]$$

Hence, force in one column V = 202 kN.

Hence, max. load on each column = 1467+202 = 1669 kN.

Total wind load = 135.46 + 12.2 + 75 = 225.61 kN.

COLUMN DESIGN.

Total vertical load = 1467 kN ; Axial force due to wind = 202 kN.

Hence, total load = 1669 kN

Factored load for DL and LL = 1.5 × 1467 = 2200.5 kN.

Factored load including wind = 1.2 × 1669 = 2002.8 kN.

Hence D.L and L.L are critical, Factored design load = 2200.5 kN.

Using (1% reinforcement) $P_u = 0.4 f_{ck} A_c + 0.67 f_y A_{sc}$ (Comp.)

Hence, $2200.5 \times 10^3 = 0.4 \times 25 \times \overset{0.99A_c}{\cancel{A_c}} + 0.67 \times 415 \times 0.01 A_c$.

Solving, $A_c = 173534 \text{ mm}^2 = \frac{\pi}{4} D^2$, Hence $D = 470$ mm

Provide D = 500 mm (Diameter for the circular columns).

Hence steel $A_{sc} = 0.01 \times \frac{\pi}{4} \times 500^2 = 1435 \text{ mm}^2$

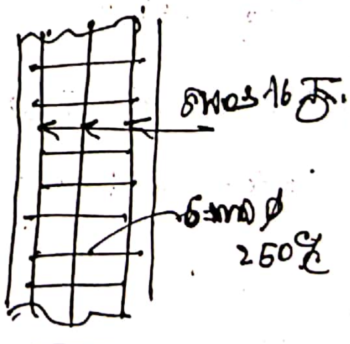
Provide 8 Nos 16ϕ with lateral ties of 6 mm 250 c/c.

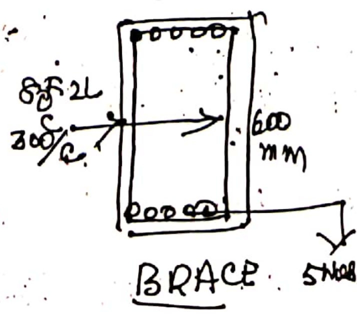
BRACING

Provide 300 × 600 mm size with 5 Nos 16ϕ on each face with 8ϕ 2L 300 c/c

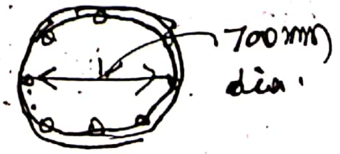
FOUNDATION

Provide independent circular footings for columns as the wind load is not critical in the design.





BRACE   5 Nos 16ϕ on each face



COLUMN