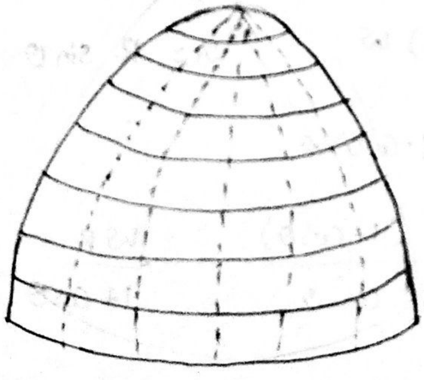
UNIT - IV

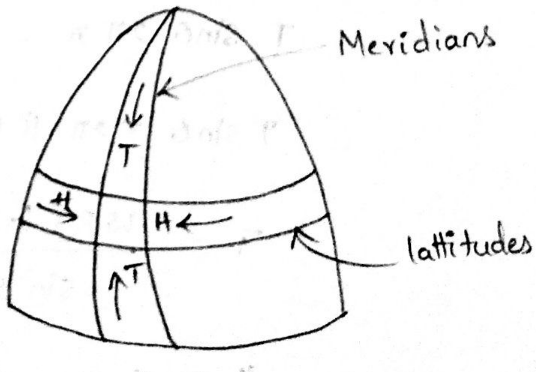
DESIGN OF WATERTANKS - II

Spherical Domes:-

- A spherical dome may be imagined to be made up of a number of horizantal rings placed one over the other. The diameter of successive rings increases in the downward directions.
- The equilibrium is maintained independently of the rings above it.
- The circle of each ring is called as lattitude.
- The circle drawn through two diametrically opposite points on a horizantal diameter and the crown is known as "meridian circle".
- All the meridian circles Convages at the crown of the spherical dome.



(a)



(b)

- If a load is applied on the dome, it gets resisted by the horizantal rings.
- Every horizantal ring supports the load of the rings above it and

transmits the same along with its own weight to the next ring below it.

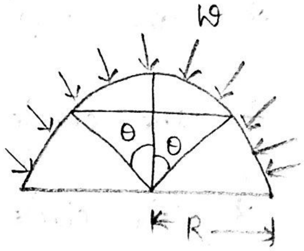
* Thus there will be a thrust of one ring on the other.

* There are two types of stresses induced in the spherical domes.

(i) Meridonial thrust (T) - Along the direction of meridians.

(ii) Hoop thrust (H) - Along the lattitudes.

Case (i) :- when a dome is subjected to U.D.L :-



Meridonial thrust (T) = $\frac{wR}{1+\cos\theta}$

$$T = \frac{wR(1-\cos\theta)}{\sin^2\theta}$$

Hoop ~~thrust~~ stress ~~(H)~~ = $\frac{wR(\cos^2\theta + \cos\theta - 1)}{t\,(1+\cos\theta)}$

Hoop thrust (H) = T cosθ R sinθ = $\frac{wR^2}{1+\cos\theta}\sin\theta\cos\theta$ ✓

$$T\sin\theta\, 2\pi r = 2\pi R^2(1-\cos\theta)\,w \quad , \quad r = R\sin\theta$$

$$T\sin\theta \times 2\pi R\sin\theta = 2\pi R^2(1-\cos\theta)w$$

$$T = \frac{wR(1-\cos\theta)}{\sin^2\theta} = \frac{wR(1-\cos\theta)}{1-\cos^2\theta} = \frac{wR}{1+\cos\theta}$$

Hoop stress = $f = \frac{1}{Rt}\frac{dH}{d\theta}$

* If the value of 'H' is positive, the hoop stress is Compressive,

* If the value of 'H' is negative, the hoop stress is tensile.

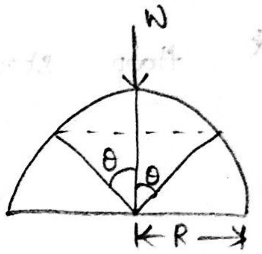
From Figure, we can observe that $\theta = 0$ at the crown

$$T_{crown} = \frac{wR}{2}, \quad H_{crown} = \frac{wR^2}{2} 0, \; f = \frac{wR}{2t}$$

* This is the maximum value of hoop stress.

* The hoop stress goes on decreasing, as θ value increases and solving the equation, we get hoop stress will be zero at $\theta = 51.8°$

Case (ii) :- when the dome is subjected to Concentrated load at crown :-



* load transfered per unit run = $\frac{w}{2\pi R \sin\theta}$

* $T \sin\theta = \frac{w}{2\pi R \sin\theta}$

* ~~Horizontal~~ Meridonial thrust (T) = $\frac{w}{2\pi R \sin^2\theta}$

* Hoop thrust (H) = $T \cos\theta \, R \sin\theta = \frac{w}{2\pi} \cot\theta$

* Hoop stress = $\frac{1}{Rt} \frac{dH}{d\theta} = \frac{-w}{2\pi R t \sin^2\theta}$

* Any Concentrated load at the crown should be distributed over a sufficient area

* It is also desirable to thickned the dome to spread the load over the greater area.

Case (iii):- when the dome is subjected to U.D.L and point load at the crown:-

- Concentrated load = W

- Distributed load = w per unit area

- Meridonial thrust $(T) = \dfrac{wR}{1+\cos\theta} + \dfrac{W}{2\pi R\sin^2\theta}$

- Hoop thrust $(H) = \dfrac{wR^2 \sin\theta\cos\theta}{1+\cos\theta} + \dfrac{W}{2\pi}\cot\theta$

- Hoop stress, $f = \dfrac{wR(\cos^2\theta+\cos\theta-1)}{t(1+\cos\theta)} \cancel{+} - \dfrac{W}{2\pi Rt\sin^2\theta}$

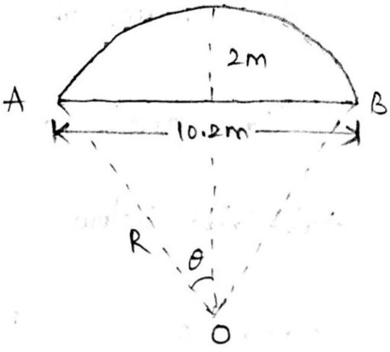
1) Design a spherical dome over a circular room 10m in diameter with 200 mm thick side walls. Live load due to wind and snow may be 2.6 KN/m^2 of the surface area. Assume the rise of dome as 2m. Use M_{15} grade Concrete and mild steel reinforcement.

Sol:- Step 1:- Geometry of the beam:-

Let 'R' be the radius of the dome

Diameter of base AB = 10.2 m

From the geometry of spherical surface and from the property of chords



$$(2R - \text{Rise}) \times \text{Rise} = \left(\frac{\text{span}}{2}\right)^2$$

$$(2R-2) \times 2 = \left(\frac{10.2}{2}\right)^2$$

$$\therefore \quad R = 7.5\text{ m}$$

$$\cos\theta = \frac{R-h}{R} = \frac{7.5-2}{7.5}$$

$$\Rightarrow \theta = 42.83^\circ < 51.8^\circ$$

∴ Hoop stress will be Compressive throughout.

Step-II:- loading:-

Assuming the thickness of the spherical dome as t = 100mm

The U.DL per sq.m of surface area = 0.1×25 = 2.5 KN/m^2

Total load = 2.6 + 2.5 = 5.1 KN/m^2

(live load = 2.6 KN/m^2)

step - 3 :- calculation of stresses :-

Meridonial thrust per metre length $T = \dfrac{wR}{1+\cos\theta}$

$$= \frac{5100 \times 7.5}{1+\cos\theta} = \frac{38250}{1+\cos\theta} \rightarrow ①$$

Hoop stress, $f = \dfrac{wR\,(\cos^2\theta + \cos\theta - 1)}{(1+\cos\theta)\,t}$

$$= \frac{5100 \times 7.5\,(\cos^2\theta + \cos\theta - 1)}{(1+\cos\theta) \times 0.1}$$

$$= \frac{382500\,(\cos^2\theta + \cos\theta - 1)}{1+\cos\theta} \rightarrow ②$$

The values of meridonial thrust and hoop stress at four differen sections and for different values of 'θ' are calculated and tabulated below.

Angle 'θ'	Meridonial stress $= T/t$ (N/mm²)	Hoop stress (N/mm²)
θ = 0	191250	191250
θ = 15°	194564.81	174901.81
θ = 30°	204981.13	126273.58
θ = 42.83	220667.89	60029.06

$M.S = \dfrac{38250}{t(1+\cos\theta)}$

Maximum value of 'T' occurs when θ = 42.83° and its magnitude, $T_{max} = 220667.89$ N/m² = 0.22 N/mm²

Maximum hoop stress occurs at the crown i.e at $\theta = 0$ and its magnitude is given by $f_{max} = 191250 \ N/m^2 = 0.191 \ N/mm^2$

For M_{15} grade concrete, ~~fck~~ • $\sigma_{cbc} = 5 \ N/mm^2$

$\therefore$ T_{max} and f_{max} is less than $5 \ N/mm^2$

Hence it is safe.

Provide a minimum steel of 0.15% of gross c/s area, because no tension at any where in the cross section.

$$= \frac{0.15}{100} \times 1000 \times 100 = 150 \, mm^2$$

Assuming 8 mm dia bars, spacing $S = \frac{\pi/4 \times 8^2}{150} \times 1000 = 335 \, mm$

$\therefore$ provide 8 mm dia bars @ 300 mm c/c.

Provision for reinforcement :-

Maximum compressive stress = $0.22 \ N/mm^2$

Maximum hoop stress = $0.19 \ N/mm^2$

If hoop stress is tensile, maximum hoop tension per metre length of meridian, hoop stress $\times b \times t$ = $0.19 \times 1000 \times 100 = 19000 \ N$

$$A_{st} = \frac{x}{\sigma_{st}} = \frac{19000}{115} = \overset{165.21}{\cancel{135.71}} \ mm^2$$

Reinforcement for temperature $= \frac{0.15}{100} \times 1000 \times 100$ ($\because$ 0.15% gross c/s area)

$= 150 \, mm^2$

Total $A_{st} = \overset{165.21}{\cancel{135.71}} + 150 = \overset{315.21}{\cancel{285.71}} \ mm^2$

Using 8mm diameter, spacing $= \frac{\pi/4 \times 8^2}{\underset{315.21}{\cancel{285.71}}} \times 1000 = \overset{\cancel{175.95}}{\cancel{159.48}} \ mm$ 159.48

Provide 8mm dia bars at a spacing of 150 mm c/c.

Step-4:- Design of lower ring beam:-

The horizantal component of meridonial thrust 'T' will cause an outward force on the support causing hoop tension.

Horizantal Component of 'T' per metere length, $w = T \cos\theta \times t$

$$w = 220667.89 \cos 42.83 \times 0.1 = 16187.7 \text{ N/m}$$

Total hoop tension $= w \times D/2 = 16187.7 \times \frac{10.2}{2} = 82.55$ KN

Area of steel $= \frac{82.55 \times 10^3}{140} = 589.64 \text{ mm}^2$

(Assuming 16 mm dia bars, spacing $= \frac{\pi/4 \times 16^2}{589.64} \times 1000 = 341$ mm)

No. of bars $= \frac{589.64}{\pi/4 \times 16^2} = 2.93$ say 4

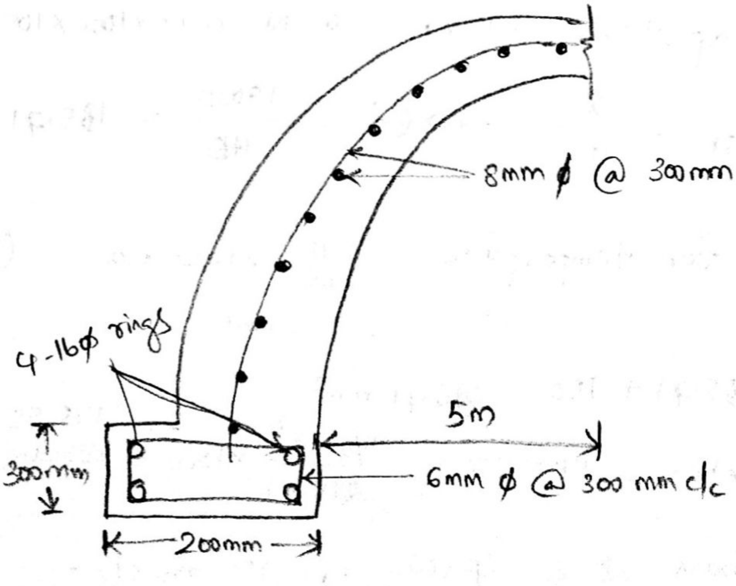
Provide 4 bars of 16 mm dia for symmetry.

$$\sigma_{cb} = \frac{T}{A_g + (m-1) A_{st}} \Rightarrow 1.2 = \frac{82.55 \times 10^3}{A_g + (18.67-1) \times \pi/4 \times 16^2 \times 4}$$

$$\Rightarrow A_g = 54596.98 \text{ mm}^2 \qquad \left(m = \frac{280}{3\sigma_{cbc}} = \frac{280}{3 \times 5} = 18.67\right)$$

Provide a ring beam of size 200×300 mm

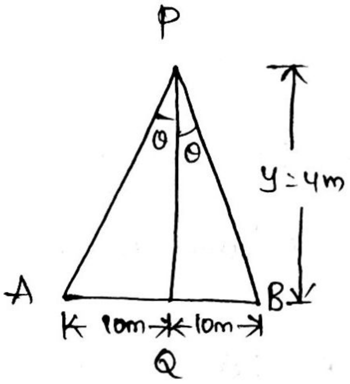
Provide 6mm dia stirrups @ 300mm c/c to tie the rings to the ring beam



Reinforcement Details

1) Design a Conical roof for a hall having a diameter of 20m. The rise of the dome has to be 4m. Assume the live load and other loads as 1500 N/m^2. Use M_{15} and mild steel bars.

sol:- step-1:- Geometry:-



$\tan\theta = 10/4 \Rightarrow \theta = 68.19°$

Diameter AB = 20m

Height of the dome, y = 4m.

$\sin\theta = 0.9285$ and $\cos\theta = 0.3715$

step-2:- loading:-

Let the thickness of shell be 100mm = t

Self weight of dome = 0.1 × 25 = 2.5 kN/m^2 = 2500 N/m^2

live load = 1500 N/m^2

Total load = 4000 N/m^2

step-3:- Calculation of stresses:—

Meridonial thrust, $T_{max} = \frac{w}{2} \times \frac{y}{\cos^2\theta}$

$= \frac{4000}{2} \times \frac{4}{0.3715^2} = 57965.86\ N$

$= 58\ kN$

Meridonial stress $= \frac{T_{max}}{bt} = \frac{58 \times 10^3}{1000 \times 100} = 0.58\ N/mm^2 < 5\ N/mm^2$

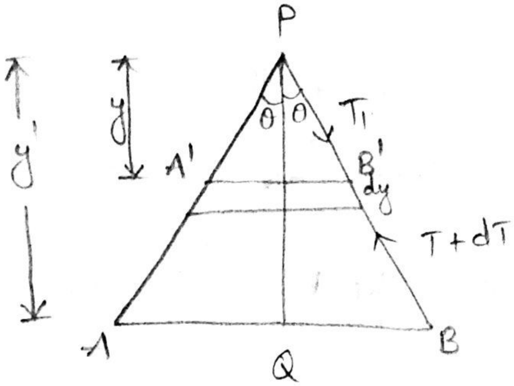
Hoop thrust, $H_{max} = wy\tan^2\theta = 4000 \times 4 \times \tan 68.2^2$

$= 100\ kN$

Hoop stress: $\frac{H_{max}}{bt} = \frac{100 \times 10^3}{1000 \times 100} = 1\ N/mm^2 < 5\ N/mm^2$

Hence Safe.

Conical dome :-



- * The Conical dome is formed by rotating the right angle triangle about . vertical axis.
- * Let 'w' be the intensity of UDL inclusive of its own weight on an unit area of dome.
- * Let 'y' be the distance of horizantal plane from apex 'p'

Diameter $A'B' = 2y \tan\theta$

length $A'P = \frac{y}{\cos\theta}$

Meridonial thrust, $T = \frac{w}{2} \times \frac{y}{\cos^2\theta} \rightarrow ①$

Intensity of meridonial stress, $= \frac{w}{2tb} \times \frac{y}{\cos^2\theta} \rightarrow ②$

Hoop thrust , $H = wy \tan^2\theta \rightarrow ③$

Intensity of hoop stress $= \frac{wy}{tb} \tan^2\theta \rightarrow ④$

The hoop stress will be Compressive through out.

Concentrated load at Apex (or) Vertex :-

Meridonial thrust on the ring, $T = \frac{W}{2\pi y \sin\theta}$

Hoop force $H = 0$

Steel reinforcement :-

The stresses worked out to be safe, Hence only nominal reinforcement has to be provided at 0.15% of area of concrete.

$$A_{st} = \frac{0.15}{100} \times 1000 \times 100 = 150 \text{ mm}^2$$

Using 8 mm dia bars, spacing $= \frac{\pi/4 \times 8^2}{150} \times 1000 = 335$ mm

Provide 8 mm dia bars @ 300mm c/c both the ways.

The meridonial bars may be discontinued near the apex and a wire mesh may be provided to avoided conjunction of steel.

Design of ring beam :-

The horizantal component of meridonial thrust 'T' will cause an outward force on the support causing hoop tension. Hence a ring beam is necessary.

Hoop tension in the ring beam, $P = T \sin\theta$

$$P = 58 \times 10^3 \times 0.9285 = 53853 \text{ N/m}$$

Hoop tension tried to rupture the beam $= P \times D/2$

$$= 53853 \times 20/2 = 538530 \text{ N}$$

Area of steel required, $A_{st} = \frac{538530}{\underset{115}{\boxed{140}}} = \overset{4682.87}{\boxed{3846.6}} \text{ mm}^2$

Using $\overset{30}{\boxed{25}}$ mm dia bars, No. of bars $= \underset{6.62}{\boxed{7.83}} \simeq 8$ no's.

A_{st} Provided $= 8 \times \pi/4 \times \underset{30}{\cancel{25}}^2 = \overset{5654.87}{\cancel{3927}} \text{ mm}^2$

Tie these bars by 8mm dia two legged stirreps @ 300mm c/c.

Let A_g be the gross area of ring beam.

Equivalent area of cross section, $A_e = A_g + (m-1) A_{st}$

$$m = \frac{280}{3\sigma_{cbc}} = \frac{280}{3 \times 5} = 18.67$$

$$A_e = A_g + (18-1) \times \overset{5654.87}{\cancel{3927}}$$

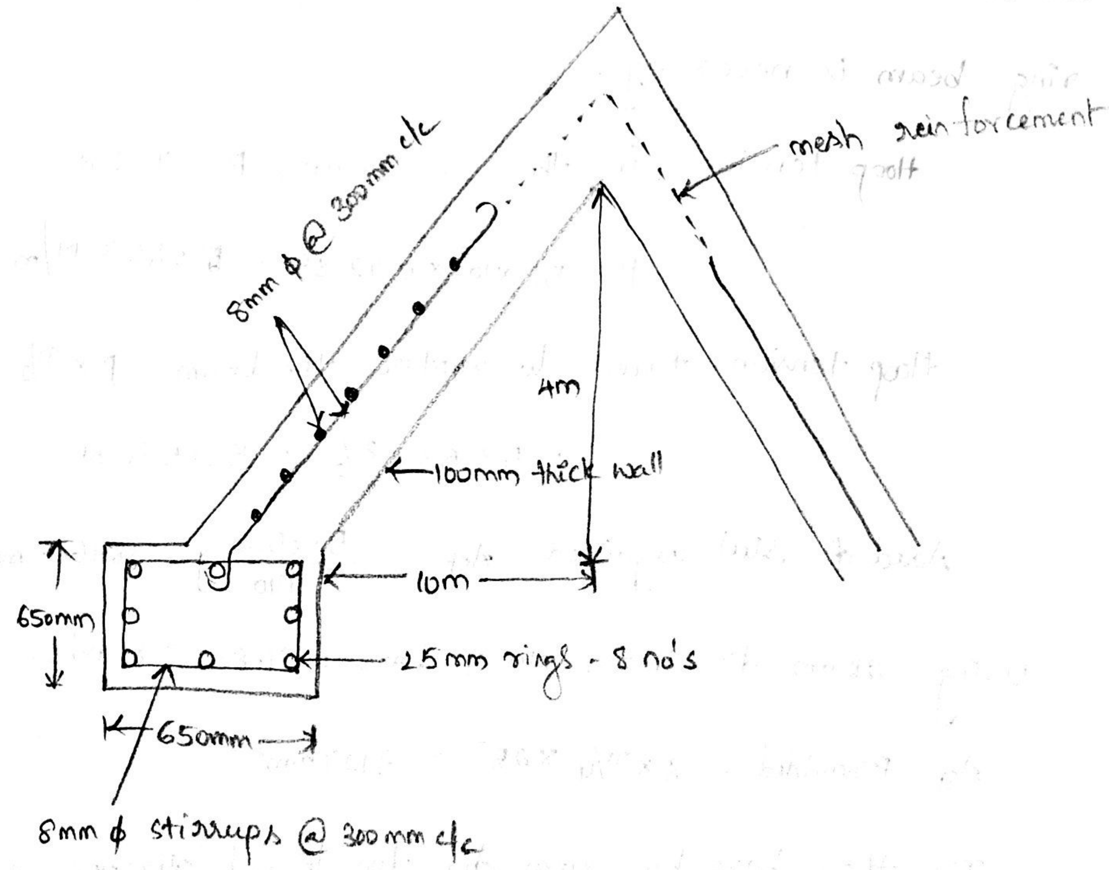
$$= A_g + \overset{99921.5}{\cancel{69390.09}}$$

Assuming the allowable tensile stress in Composite section be 1.2 N/mm².

$$\sigma_{cb} = \frac{T}{A_g + (m-1) A_{st}} \Rightarrow \frac{538530}{A_g + \underset{99921.5}{\cancel{69390.09}}} = 1.2$$

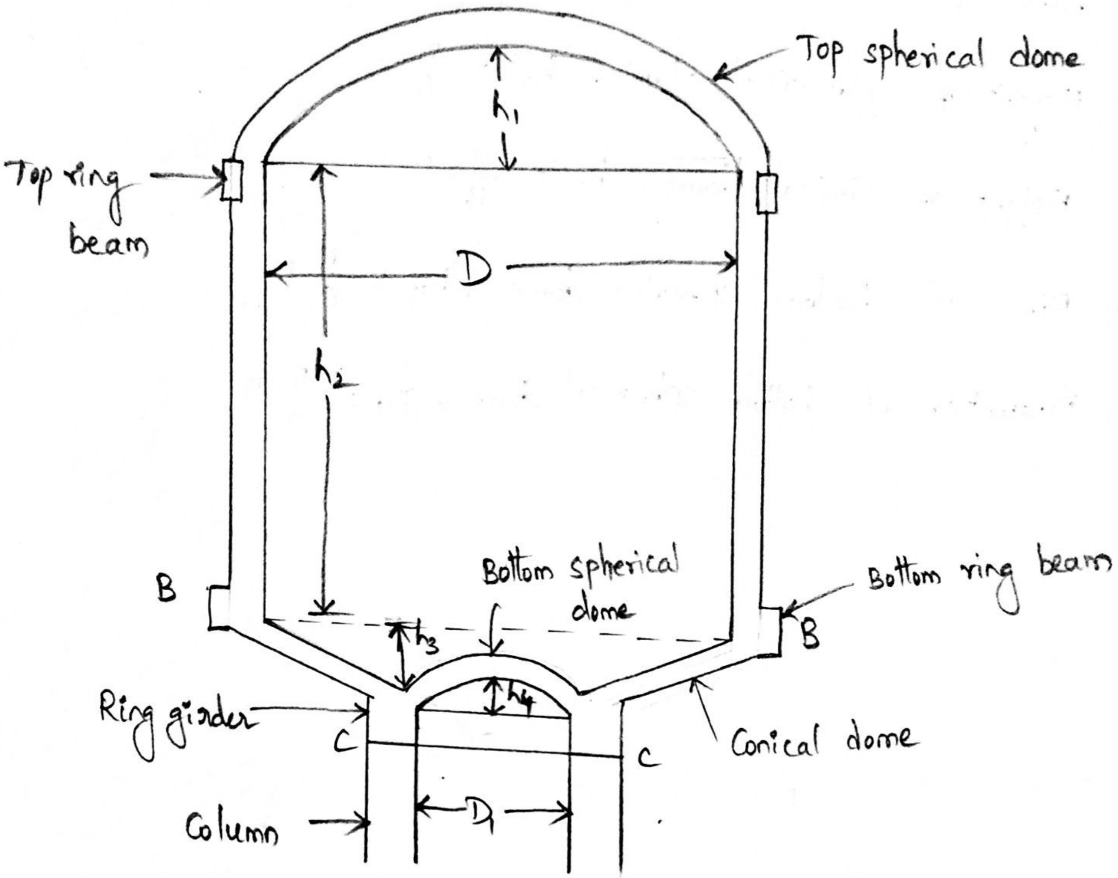
$$\Rightarrow A_g = \overset{348853.5}{\cancel{379384.}}91 \text{ mm}^2$$

Provide a ring beam of size 650 x 650mm.



Reinforcement Details

Intze tank :-



Typical Intze Tank

* For larger capacities of over head tanks, flat bottom circular tanks become uneconomical since the thickness of slab required increase considerably.

* In such cases, intze tanks are more economical.

* Intze tank consists of the following structural elements

(i) Top spherical dome
(ii) Top ring beam
(iii) cylindrical wall
(iv) Bottom ring beam
(v) Conical shell
(vi) Bottom spherical dome
(vii) Bottom ring girder
(viii) Columns
(ix) Foundations

Economical Proportions of structural elements:-

(i) Rise of top spherical dome $h_1 = \frac{1}{8} D$

(ii) Height of cylindrical tank, $h_2 = \frac{2}{8} D$

(iii) Height of Conical dome, $h_3 = \frac{3}{16} D$

(iv) Rise of bottom spherical dome, $h_4 = \frac{1}{8} D$

(v) Diameter of bottom spherical dome, $D_1 = \frac{5}{8} D$

1) Design an Intze tank of 900,000 litres capacity. The height of staging is 16 m upto the bottom of tank. The bearing capacity of soil may be assumed to be 150 KN/m^2. Assume the intensity of wind pressure as 1500 N/m^2. Use M_{20} Concrete and HYSD bars.

Sol: Step-1:- Dimensions of the tank :-

Let the diameter of circular portion, $D = 14m$

Diameter of bottom spherical dome, $D_1 = \frac{5}{8} D = 8.75 \simeq 10m$.

Height of Conical dome, $h_3 = \frac{3}{16} D = 2.625 \simeq 2m$

Rise $h_1 = h_4 = \frac{1}{8} D = 1.8m$, For practical appearance $h_4 = 1.6m$

The radius R_2 of the bottom dome is given by

$1.6 (2R_2 - 1.6) = 5^2 \Rightarrow R_2 = 8.61 m$

$\sin \phi_2 = \frac{5}{8.61} = 0.5807 \Rightarrow \phi_2 =$ ~~44.02°~~ 35.50°

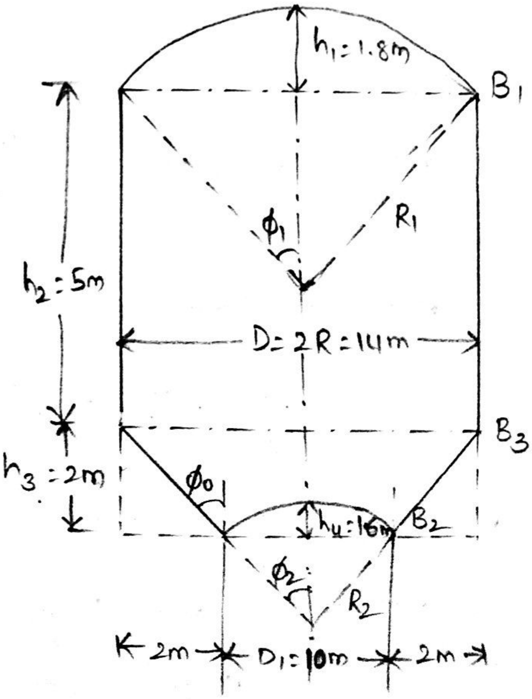
$\cos \phi_2 =$ ~~0.7702~~ 0.8141, $\tan \phi_2 =$ ~~0.8279~~ 0.7133, $\cot \phi_2 =$ ~~1.2078~~ 1.4019

Let h_2 be the height of cylindrical Portion.

Capacity of tank is given by

$$V = \frac{\pi}{4} D^2 h_2 + \frac{\pi h_3}{12} (D^2 + D_1^2 + DD_1) - \frac{\pi h_4^2}{3} (3R_2 - h_4)$$

Required volume = 900,000 litres = 900 m^3

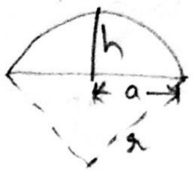


$$\therefore\ 900 = \frac{\pi}{4}(14^2)h_2 + \frac{\pi \times 2}{12}(14^2+10^2+14\times 10) - \frac{\pi \times 1.6^2}{3}(3\times 8.61 - 1.6)$$

$\Rightarrow h_2 = 4.78\,m$. Allowing for free board, $h_2 \simeq 5m$

For the top dome, the radius R_1 is given by

$$1.8(2R_1 - 1.8) = 7^2$$



$$r = \frac{a^2+h^2}{2h} = \frac{7^2+1.8^2}{2\times 1.8} = 14.51$$

$\Rightarrow R_1 = 14.51\,m$

$$\sin\phi_1 = \frac{7}{14.51} = 0.4824 \Rightarrow \phi_1 = 28.84^\circ$$

$\cos\phi_1 = 0.8760$

Step-2 :- Design of top dome :-

$R_1 = 14.51m$, $\cos\phi_1 = 0.8760$ and $\sin\phi_1 = 0.4824$

Let thickness $t_1 = 100mm = 0.1m$

Taking live load of 1500 N/m², Total load 'p' per sq m of dome

$= 0.1 \times 25 \times 1000 + 1500 = 4000\ N/m^2$

$$\text{Meridonial thrust at edges} = T_1 = \frac{pR_1}{1+\cos\phi_1} = \frac{4000\times 14.51}{1+0.8760}$$

$= 30938\ N/m$

$$\text{Meridonial Stress} = \frac{T_1}{t} = \frac{30938}{100\times 1000} = 0.31\ N/mm^2$$

Maximum hoop stress occurs at the centre and its magnitude

$$\text{is} = \frac{pR_1}{t_1}\times \frac{1}{2} = \frac{4000\times 14.51}{2\times 0.1} = 290200\ N/m^2 = 0.29\ N/mm^2$$

Permissible stress in direct compression (σ_{cc}) in M_{20} grade Concrete $= 5\ N/mm^2$

Meridonial and hoop stresses < 5 N/mm²

Hence safe.

Since the stresses are within the safe limits, provide nominal reinforcement @ 0.3% of gross c/s area

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

Using 8mm dia bars, spacing $= \frac{\pi/4 \times 8^2}{300} \times 1000 \simeq 160\,mm$

Hence provide 8mm dia bars @ 160mm c/c in both the directions.

Step-3:- Design of top ring beam B_1 :-

Horizantal Component of T_1 is given by

$$P_1 = T_1 \cos\phi = 30938 \times 0.8760 = 27102\ N/m$$

Total tension tending to rupture the beam $= P_1 \times D/2$

$$= 27102 \times 14/2 = 189712\ N$$

Permissible stress in HYSD bars $= 150\ N/mm^2$

$$A_{sh} = \frac{189712}{150} = 1265\,mm^2$$

Using 20 mm ϕ bars, No. of bars $= \frac{1265}{\pi/4 \times 20^2} \simeq 5$ bars.

Actual A_{sh} provided $= 5 \times \pi/4 \times 20^2 =$ ~~1257~~ $1571\,mm^2$

The area of cross section of ring beam is given by

$$\frac{189712}{A_g + (m-1)A_{st}} = \frac{189712}{A_g + \left(\frac{280}{3\times7} - 1\right) \times 1571} = 1.2$$

equivalent area of concrete

($\sigma_{cbc} = 7\ N/mm^2$ for M_{20} concrete)

(Allowing tensile stress of 1.2 N/mm² in equivalent concrete area)

$$\Rightarrow A_g = 138722.9\ mm^2$$

Provide a ring beam of 360 mm depth and 400 mm width. Tie the 20mm dia bars by 6 mm ϕ nominal stirrups @ 200mm c/c.

Step-4 :- Design of cylindrical shell :-

In membrane analysis, the tank wall is assumed to be free at top and bottom. Maximum hoop tension occurs at the base of the wall and its magnitude is given by

$$P = \overset{\text{unit weight of water}}{w}\, h\, D/2 = 9800 \times 5 \times 14/2 = 343000 \text{ N/m height}$$

Area of steel, $A_{sh} = \frac{P}{\sigma_{st}} = \frac{343000}{150} = 2286 \text{ mm}^2$ per metre height

Providing rings on both faces, A_{sh} on each face = 1143 mm²

Using 12mm dia rings, spacing = $\frac{\pi/4 \times 12^2}{1143} \times 1000 = 98.9$ mm.

∴ provide 12mm dia rings @ 95 mm c/c at bottom.

Actual A_{sh} provided = $\frac{\pi/4 \times 12^2}{95} \times 1000 = 1190 \text{ mm}^2$ on each face.

permitting 1.2 N/mm² stress on composite section

$$\frac{343000}{1000t + 12.33 \times 1190 \times 2} = 1.2 \qquad \left(\begin{aligned} A &= 1000 \times t \\ &= b \times t \end{aligned} \right)$$

$\Rightarrow t = 257.3$ mm

∴ Provide t = 300 mm at bottom and taper it to 200mm at to

~~Average $t = \frac{300+200}{2} = 250$mm~~ Average $t = \frac{300+200}{2} = 250$mm

~~% distribution of steel = $0.3\left(\frac{250-100}{450-100}\right) \times 0.1 = 0.24$~~

Distribution steel = $\frac{0.3}{100} \times 1000 \times 250 = 750 \text{mm}^2$

Area of steel on each face = $\frac{750}{2} = 375 \text{ mm}^2$

Using 8mm dia bars, spacing = $\frac{\pi/4 \times 8^2}{~~100~~\ 375} \times 1000 = 134.05$mm

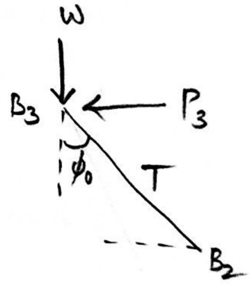
Hence provide 8 mm dia bars @ 130 mm c/c on both faces.

Keep a clear cover of 25mm. Extend the vertical bars to outer face into the dome to take care of continuity effects.

Step 5:- Design of Ring beam B_3 :-

This ring beam connects the tank wall with Conical dome.

The vertical load at the Junction of wall with Conical dome is transfered to ring beam B_3 by meridonial thrust in the Conical dome The horizantal component of thrust causes hoop tension at the Junction The ring beam is provided to take up the hoop tension. The load W transmitted through tank wall, at the top of Conical dome consists of the following.



(i) Load of top dome $= T_1 \sin\phi_1 = 30938 \times 0.4824$
$= 14924$ N/m

(ii) Load due to the ring beam B_1 $= 0.36 \times 0.4 \times 1 \times 25000 = 3600$ N/m

(iii) Load due to tank walls $= 5 \times \frac{0.2+0.3}{2} \times 1 \times 25000 = \overset{31250}{\cancel{10500}}$ N/m

(iv) Self weight of beam B_3 (1m × 0.6m say) $= 1 \times 0.6 \times 25000 = 15000$ N/m

Total W = 64774 N/m

Inclination of Conical dome with vertical $\phi_0 = 45^\circ$

$\sin\phi_0 = \cos\phi_0 = 0.7071 = 1/\sqrt{2}$, $\tan\phi_0 = 1$

$P_w = W \tan\phi_0 = 64774 \times 1 = 64774$ N/m

Horizantal force P_w caused due to water Pressure at top of Conical dome is given by

$$P_w = w \cdot h \cdot d_3 = 9800 \times 5 \times 0.6 = 29400 \text{ N/m}$$

Hence hoop tension in the ring beam is given by

$$P_3 = (P_w + P_w) D/2 = (64774 + 29400) \times 14/2 = 659218 \text{ N}$$

This is to be resisted entirely by steel hoops,

$$A_{sh} = \frac{659218}{150} = 4394.78 \text{ mm}^2$$

Using 30 mm dia bars, No. of bars $= \frac{4394.78}{\pi/4 \times 30^2} = 6.21 \simeq 7$ bars.

Provide 7 bars of 30 mm diameter.

$$A_{sh} \text{ provided} = 7 \times \pi/4 \times 30^2 = 4948.65 \text{ mm}^2$$

$$\text{Stress in equivalent section} = \frac{659218}{(1000 \times 600) + (12.33) \times 4948.65}$$

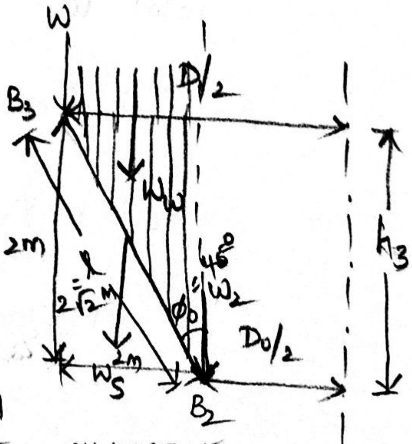
$$= 0.99 < 1.2 \text{ N/mm}^2$$

Hence Safe.

The 8 mm dia distribution bars (vertical bars) provided in the wall @ 150 mm c/c should be taken round the above rings to act as stirrups.

Step-6 :- Design of Conical dome :-

(a) Meridonial thrust :-



$$W_w = \pi/4 \, (14^2 - 10^2) \times 5 \times 9800 + \left[\frac{\pi \times 2 \times 9800}{12} \times 14^2 + 10^2 + 14 \times 10\right] - \pi/4 \times 10^2 \times 2 \times 9800 = 4392368 \text{ N}$$

3698282.87

1006994.78

1540957.1

3164326.55 - 3165897.25

Let the thickness of Conical slab be 400 mm.

∴ Total self weight W_s is given by

$$W_s = 25000\ \pi \left(\frac{14+10}{2}\right) \times 2\sqrt{2} \times 0.4 = 1066131\ N$$

weight W at B_3 = 64774 N/m

Hence vertical load W_2 per metre run is given by

$$W_2 = \frac{(\pi \times 14 \times 64774) + \overset{3164326.55}{\cancel{4392368}} + 1066131}{\pi \times 10} = \overset{225343.24}{\cancel{264433.04}}\ N/m$$

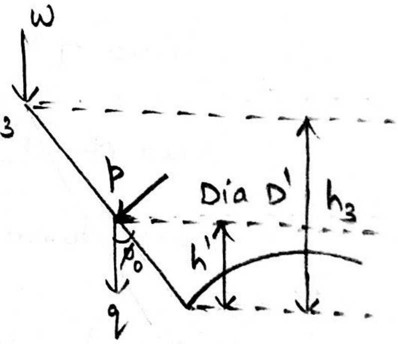
Meridonial thrust T_0 in the Conical dome is

$$T_0 = \frac{W_2}{\cos\phi_0} = \frac{\overset{225343.24}{\cancel{264433}}}{1/\sqrt{2}} = \overset{318683.47}{\cancel{373964.76}}\ N/m$$

$$\therefore \text{Meridonial stress} = \frac{\overset{318683.47}{\cancel{373964.76}}}{1000 \times 400} = \overset{0.796}{\cancel{0.939}} < 1.2\ N/mm^2$$

Hence Safe.

(b) Hoop tension :-



Diameter of Conical dome at any height h' above base is

$$D' = 10 + \left(\frac{14-10}{2}\right) h' = 10 + 2h'$$

Intensity of water pressure, $p = (5 + 2 - h') \times 9800 = (7-h')\, 9800\ N/m^2$

Self weight $q = 0.4 \times 1 \times 1 \times 25000 = 10000\ N/m^2$

Hence hoop tension p_0' is given by

$$p_0' = \left(\frac{p}{\cos\phi_0} + q \tan\phi_0\right)\frac{D'}{2} = (7-h')\, 9800\sqrt{2} + (10000 \times 1)\left(\frac{10+2h'}{2}\right)$$

$$= \left[13859(7-h') + 10000\right](5+h')$$

$P_o' = 535075 + 37720\,h' - 13859\,h'^2$

The values of P_o' at $h' = 0$, $h' = 1$ and $h' = 2$ are

h'	P_o' (Hoop tension) (N)
0	535075
1	558936
2	555079

For Maxima $\dfrac{dP_o'}{dh'} = 0 = 37720 - 2 \times 13859\,h'$

$\Rightarrow h' = 1.361\,m$

Maximum $P_o' = 535075 + 37720 \times 1.361 - 13859 \times 1.361^2$

$= 560739\ N$

(c) Design of walls:-

Maximum hoop tension = 560739 N

Area of steel, $A_{sh} = \dfrac{560739}{150} = 3738\ mm^2$

Area of steel on each face = $1869\ mm^2$

Using 16mm dia bars, spacing $= \dfrac{\pi/4 \times 16^2}{1869} \times 1000 = 107.5\ mm$

∴ Provide 16 mm dia bars @ 100mm c/c on each face.

Actual $A_{sh} = \dfrac{\pi/4 \times 16^2}{100} \times 1000 = 2010\ mm^2$

Maximum tensile stress in composite section

$= \dfrac{560739}{(400 \times 1000) + (12.33 \times 2010 \times 2)} = 1.39\ N/mm^2$

This is more than $1.2\ N/mm^2$. Hence increase the thickness to 420mm

This will reduce the tensile stress to $1.194\ N/mm^2$.

In the meridonial direction, Provide reinforcement

$$A_{sh} = \frac{0.3}{100} \times 420 \times 1000 = 1260 \text{ mm}^2$$

i.e 630 mm² steel on each face.

Using 10 mm dia bars, spacing $= \frac{\pi/4 \times 10^2}{630} \times 1000 = 124.68$ mm

Hence provide 10mm dia bars @ 120mm c/c on each face.

Provide a clear Cover of 25 mm.

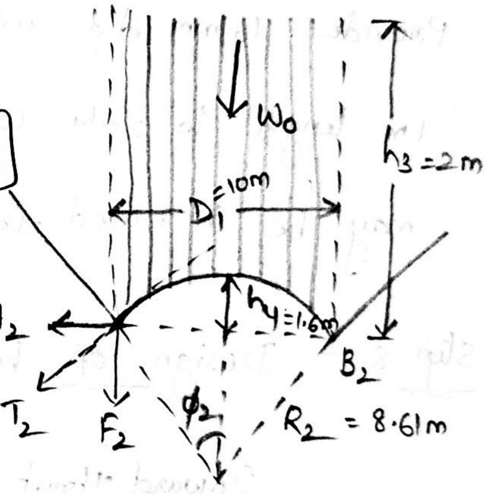
step-7:- Design of bottom dome:-

$R_2 = 8.61$ m, $\sin\phi_2 = 0.5807$, $\cos\phi_2 = 0.8141$

Weight of water W_0 on dome is given by

$$W_0 = \left[\frac{\pi}{4} \times 10^2 \times 7 - \frac{\pi \times 1.6^2}{3} \times (3 \times 8.61 - 1.6)\right] \times 9800$$

$= 4751259$ N



Let the thickness of bottom dome be 250mm.

Self weight $= 2\pi R_2 h_4 t \times 25000$

$= 2\pi \times 8.61 \times 1.6 \times 0.25 \times 25000 = 540982$ N

Total weight $W_T = 4751259 + 540982 = 5292241$ N

Meridonial thrust $= T_2 = \frac{5292241}{\pi \times 10 \times 0.5807} = 290093$ N/m

Meridonial stress $= \frac{290093}{250 \times 1000} = 1.16$ N/mm² < 1.2 N/mm²

Hence safe.

Intensity of load per unit area $= P_2 = \dfrac{5292241}{2\pi \times 8.61 \times 1.6} = 61142\ N/m^2$

Maximum hoop stress at Centre of dome

$$= \frac{P_2 R_2}{2t} = \frac{61142 \times 8.61}{2 \times 0.25} = 1052860\ N/m^2$$

$$= 1.05\ N/mm^2 < 1.2\ N/mm^2$$

Hence safe.

Area of minimum steel $= \dfrac{0.3}{100} \times 250 \times 1000 = 750\ mm^2$ in each direction

Using 10mm dia bars, spacing $= \dfrac{\pi/4 \times 10^2}{750} \times 1000 = 104.73\ mm$

Hence provide 10mm dia bars @ 100 mm c/c in both directions. Also Provide 16 mm dia meridonial bars @ 100 mm c/c near water face for 1m length to take care of Continuity effect. The thickness of the dome may be increased from 250 mm to 280 mm gradually in 1m length.

Step 8 :- Design of bottom ring beam B_2 :-

from step 6(a)

Inward thrust from Conical dome, $= T_0 \sin \phi_0$

W_2 318683.47

= ~~373964.76~~ $\times 1/\sqrt{2}$

225343.24

= ~~264433.01~~ N/m

outward thrust from bottom dome $= T_2 \cos \phi_2$

$= 290093 \times 0.8141$

$= 236165\ N/m$

225343.24

Net in ward thrust = ~~264433.01~~ - 236165

= ~~28268.01~~ N/m

-10821.75

Hoop Compression in beam = ~~28268.01~~) $\times \frac{10}{2}$ = ~~141340~~ N

10821.75 54108.78

Assuming the size of beam be 600 mm × 1200 mm

$$\text{Hoop stress} = \frac{\overset{54108.78}{\cancel{141840}}}{600 \times 1200} = \overset{0.075}{\cancel{0.196}}\ \text{N/mm}^2 < 1.2\ \text{N/mm}^2$$

Hence Safe.

Vertical load on beam, per metre run $= T_0 \cos\phi_0 + T_2 \sin\phi_2$

$$= \overset{318683.47}{\cancel{373964.76}} \times 1/\sqrt{2} + 290093 \times 0.5807 \approx \cancel{[illegible]}$$

$$= \overset{393800}{\cancel{432890}}\ \text{N/m}$$

(Alternatively, vertical load $= w_2 + \dfrac{W_T}{\pi D_1}$ ← From step-1

$$= \overset{225343.24}{\cancel{264433.01}} + \frac{5292241}{\pi \times 10} \approx$$

$$= \overset{393800}{\cancel{432890.26}}\ \text{N/m}$$)

Self weight $= 0.6 \times 1.2 \times 1 \times 25000 = 18000$ N/m

The load on beam $w = \overset{393800}{\cancel{432890}} + 18000 = \overset{411800}{\cancel{450890}}$ N/m

Let us support the beam on 8 equally spaced columns at a mean diameter of 10m. Mean radius of curved beam is $R = 5$m

$$2\theta = 45^\circ = \pi/4 \overset{=22.5^\circ}{\Rightarrow} \theta = \pi/8$$

For 8 columns, $C_1 = \overset{0.066}{\cancel{0.0083}}$, $C_2 = \overset{0.030}{\cancel{0.00416}}$ and $C_3 = \overset{0.005}{\cancel{0.0006}}$, $\phi_m = 9\tfrac{1}{2}^{\circ}$

$$wR^2 2\theta = \overset{411800}{\cancel{450890}} \times 5^2 \times \pi/4 = \overset{8085683.98}{\cancel{8853204.44}}\ \text{Nm}$$

Maximum −ve B.M at Support, $M_0 = C_1 wR^2 2\theta = \overset{533655.14}{\cancel{584311.49}}$ Nm

Maximum +ve B.M at ~~Support~~ midspan, $M_c = C_2 wR^2 2\theta = \overset{242570.51}{\cancel{265596.13}}$ Nm

Maximum Torsional moment, $M_m = C_3 wR^2 2\theta = \overset{40428.42}{\cancel{44266.02}}$ Nm

For M_{20} Concrete, $\sigma_{cbc} = 7\ N/mm^2$ and HYSD bars, $\sigma_{st} = 150\ N/mm^2$

we have $K = 0.378$

$J = 0.874$

$Q = 1.156$

Required effective depth, $d = \sqrt{\dfrac{M}{Qb}}$ $\quad M = Qbd^2$

$$= \sqrt{\frac{584311.49 \times 1000}{1.156 \times 600}} = 917.84\ mm$$

However, Keep total depth = 1200 mm from shear point of view. Let $d = 1140\,mm$.

Max. shear force at supports, $F_0 = wR\theta = \overset{411800}{\cancel{450890}} \times 5 \times \pi/8$

$$F_0 = \overset{808567.41}{\cancel{885320.44}}\ N$$

shear force at any point is given by $F = wR(\theta - \phi)$

At $\phi = \phi_m$, $S = \overset{411800}{\cancel{450890}} \times 5\,(22.5^\circ - 9.5^\circ) \times \dfrac{\pi}{180} = \overset{467172.28}{\cancel{514518.47}}\ N$

Equivalent shear force, $S_e = S + 1.6\,\dfrac{T}{b}$

~~shear force = S~~

$$= \overset{467172.28}{\cancel{514518.47}} + 1.6 \times \frac{\overset{40428.42}{\cancel{44266.02}}}{\underset{0.6}{\cancel{1.2}}}$$

$$= \overset{521076.84}{\cancel{570539.83}}\ N$$

Main and longitudinal reinforcement

Equivalent B.M, $M_{e1} = M + M_t$

$$M_t = T\,\frac{(1 + D/b)}{1.7} = \frac{\overset{40428.42}{\cancel{44266.02}}\left(1 + \frac{1.2}{0.6}\right)}{1.7} = \overset{71344.27\ Nm}{\cancel{78116.5\ Nm}}$$

$$M_{e1} = \overset{533655.14}{\cancel{584311.49}} + \overset{71344.27}{\cancel{78116.5}} = \overset{604999.41}{\cancel{662427.99}}\ Nm$$

$$A_{st} = \frac{M_{e1}}{\sigma_{st}\,J\,d} = \frac{\overset{604999.41}{\cancel{662427.99}} \times 1000}{150 \times 0.874 \times 1160} = \overset{3978.27}{\cancel{4355.9}}\ mm^2$$

Using 25 mm dia bars, No. of bars $= \dfrac{\overset{5478.27}{\cancel{4355.9}}}{\pi/4 \times 25^2} = \overset{8.10}{\cancel{8.87}} \simeq 9$ bars ⑸⑼

Hence provide 9 bars of 25 mm diameter.

Transverse reinforcement

$S_e = V_e = \overset{521076.84}{\cancel{570539.83}}$ N

Equivalent shear stress, $\tau_{ve} = \dfrac{V_e}{bd} = \dfrac{\overset{521076.84}{\cancel{570539.83}}}{600 \times 11\underset{4}{\cancel{6}}0} = \overset{0.748}{\cancel{0.819}}$ N/mm²

for M_{20} Concrete $\tau_{c\,max} = 1.8$ N/mm²

$$\frac{100\,A_{st}}{bd} = \frac{100 \times 9 \times \pi/4 \times 25^2}{600 \times 1160} = 0.63$$

$\tau_c = 0.52$ N/mm²

Since $\tau_{ve} > \tau_c$. shear reinforcement is necessary.

The area of c/s, A_{sv} of the stirrups is given by

$$A_{sv} = \frac{T\,S_v}{b_1 d_1 \sigma_{sv}} + \frac{V\,S_v}{2.5\,d_1\,\sigma_{sv}}$$

where $b_1 = (600 - \overset{\swarrow \text{effective cover}}{40} \times 2) \cancel{-25} = 495$mm

$d_1 = 1200 - (40 \times 2) \cancel{-25} = 1095$ mm

$$\frac{A_{sv}}{S_v} = \frac{\overset{40428.42}{\cancel{44266.02}}}{495 \times 1095 \times 150} + \frac{\overset{521076.84}{\cancel{570539.83}}}{2.5 \times 1095 \times 150}$$

$= \overset{1.269}{\cancel{1.39}}$

Minimum transverse reinforcement is governed by

$$\frac{A_{sv}}{s_v} \geq \left(\frac{\tau_{ve} - \tau_c}{\sigma_{sv}}\right) b = \left(\frac{\overset{0.748}{\cancel{0.819}} - 0.63}{150}\right) \times 600 = \overset{0.472}{\cancel{0.756}}$$

Using 12mm dia 4 legged stirrups, $S_v = \dfrac{4 \times \pi/4 \times 12^2}{1.39} = 325.5$mm

However spacing should be the least of

x_1 = short dimension of stirrup = 495+25 +12 = 532 mm

y_1 = long " " " = 1095+25+12 = $\boxed{1032}$ mm

$$\frac{x_1+y_1}{4} = \frac{532+1032}{4} = 391 \text{ mm}$$

Hence provide 12 mm ϕ 4 legged stirrups @ 300 mm c/c.

Side face reinforcement :-

Since the depth is more than 450mm, provide side face reinforcement @ 0.1 %.

$$A_t = \frac{0.1}{100} \times 600 \times 1200 = 720 \text{mm}^2$$

Provide 3-16mm ϕ bars on each face.

$$A_{t\ provided} = 3 \times \pi/4 \times 16^2 = 1206 \text{ mm}^2$$

Step-9:- Design of Columns :-

The tank is supported on 8 columns, symmetrically placed on a circle of 10m mean diameter. Height of staging above ground level is 16m. Let us divide this height into four panels, each of 4m height. Let the column be connected to raft foundation by means of a ring beam, the top of which is provided at 1m below the ground level, so that the actual height of bottom panel is 5m.

(a) Vertical load on Columns:-

step 6 step-1

1. weight of water = $W_\omega + W_0$ = ~~4392368~~ 3164326.55 + 4751259

= ~~9143627~~ 7915585.55 N

2. weight of tank :

(i) weight of top dome + cylindrical walls = W

step-5

= 64774 × π × 14 = 2847589.84 N

step 6

(ii) weight of Conical dome = W_s = 1066131 N

step-1

(iii) weight of bottom dome = 540982 N

step 8

(iv) weight of bottom ring beam = 18000 × π × 10 = 565487 N

Total weight of tank = 5020189.89 N

Total super imposed load = 1+2 = ~~14163816.84~~ 12935775.44 N

load per column = $\frac{\cancel{14163816.84}\ 12935775.44}{8}$ = ~~1770477.1~~ 1616971.92 N

Let the column be of 700 mm diameter

Weight of column per metre height = $\pi/4 \times 0.7^2 \times 1 \times 25000$

= 9620 N

Let the brace be of 300 mm × 600 mm Size

length of each brace = L = $R \dfrac{\sin \frac{2\pi}{n}}{\cos \frac{\pi}{n}} = 5 \times \dfrac{\sin \pi/4}{\cos \pi/8}$ = 3.83 m

(Alternatively L = $\frac{\pi \times 10}{8}$ = 3.93 m.)

clear length of each brace = 3.83 − 0.7 = 3.13 m

weight of each brace = 0.3 × 0.6 × 3.13 × 25000 = 14085 N

Hence total weight of column Just above brace is tabulated below.

Brace GH :

$W = $ ~~1770477~~ 1616971.93 $+ 4 \times 9620$

$= $ ~~1808957~~ 1655451.93 N

Brace EF :

$W = $ ~~1770477~~ 1616971.93 $+ 8 \times 9620 + 14085$

$= $ ~~1861522~~ 1708016.93 N

Brace CD :

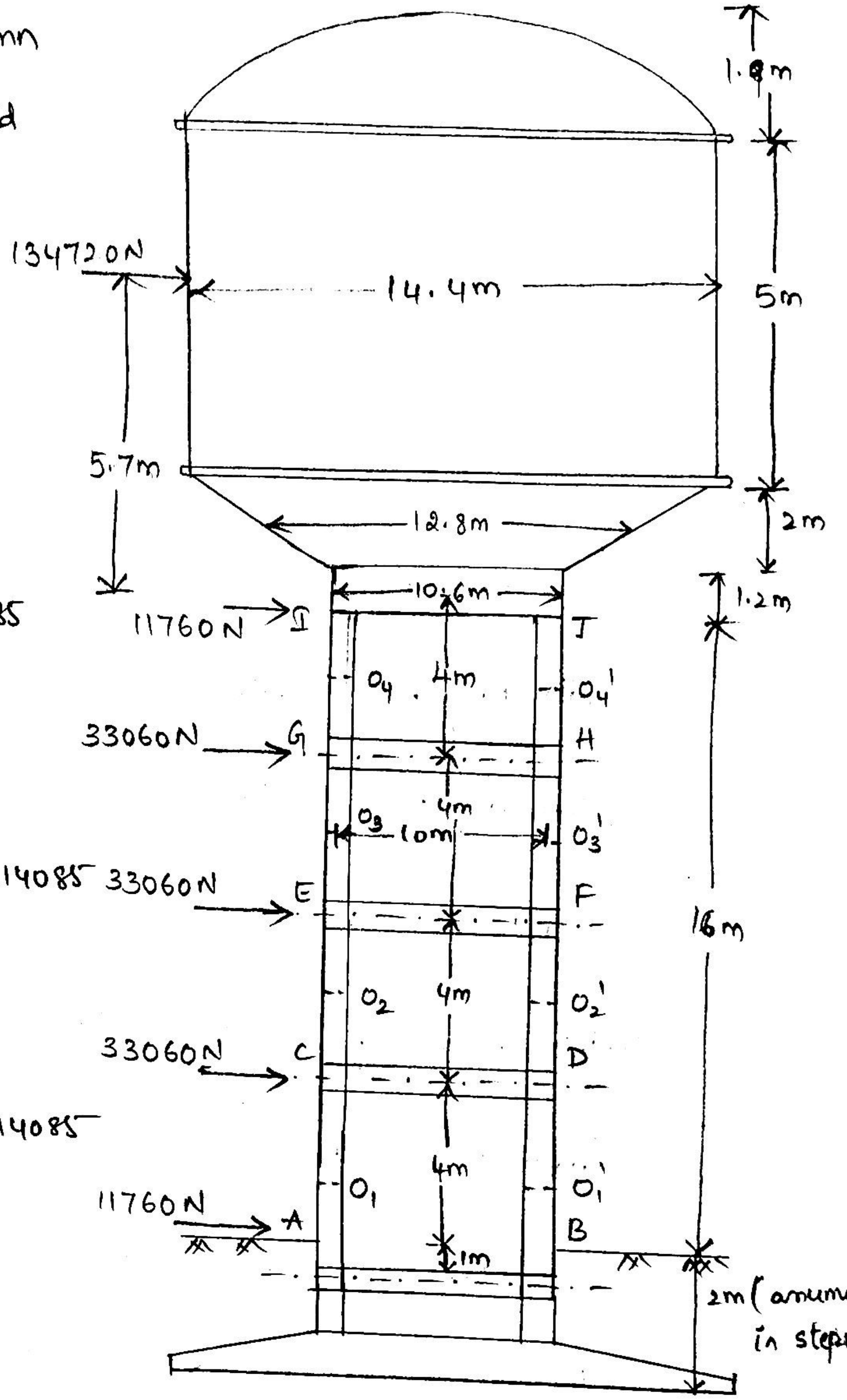
$W = $ ~~1770477~~ 1616971.93 $+ 12 \times 9620 + 2 \times 14085$

$= $ ~~1914087~~ 1760581.93 N

Bottom of column

$W = $ ~~1770477~~ 1616971.93 $+ 17 \times 9620 + 3 \times 14085$

$= $ ~~1976272~~ 1822766.93 N



(b) wind loads :-

Intensity of wind pressure = 1500 N/m²

Let us take a shape factor of 0.7 for Sections circular in plan.

wind load on tank, domes and ring beam

$$= \left[(5 \times 14.4) + \left(14.2 \times \frac{2}{3} \times 1.9\right) + (2 \times 12.8) + (10.6 \times 1.2)\right] \times 1500 \times 0.7$$

$= 134720$ N

This may be assumed to act at about 5.7m above the bottom of ring beam.

Wind load on each Panel of 4m height of Columns

$= (4 \times 0.7 \times 8) \times 1500 \times 0.7 + (0.6 \times 10.6) \times 1500 = 23520 + 9540$

$= 33060$ N

Wind load at the top end of top Panel $= \frac{1}{2} \times 23520$

$= 11760$ N

The point of Contraflexure O_1, O_2, O_3 and O_4 are assumed to be at the mid height of each panel.

The shear forces Q_w and moments M_w due to wind at these Panels are given below.

Level	Q_w (N)	M_w (Nm)
O_4	$134720 + 11760 = 146480$ N	$134720 \times 7.7 + 11760 \times 2 = 1060860$
O_3	$134720 + 11760 + 33060 = 179540$ N	$134720 \times 11.7 + 11760 \times 6 + 33060 \times 2 = 1712900$
O_2	$134720 + 11760 + 2 \times 33060 = 212600$	$134720 \times 15.7 + 11760 \times 10 + 33060(6+2) = 2497180$
O_1	$134720 + 11760 + 3 \times 33060 = 245660$	$134720 \times 20.2 + 11760 \times 14.5 + 33060 (10.5 + 6.5 + 2.5) = 3418930$

The axial thrust $V_{max} = \dfrac{4 M_w}{n D_o} = \dfrac{4 M_w}{8 \times 10} = 0.05 M_w$ in the farthest lee ward Column, the Shear force $S_{max} = \dfrac{2 Q_w}{n} = 0.25Q$ in the Column on the bending axis at each of the above levels an the bending moment $M = S_{max} \times h/2$ in the Columns are tabulated below

Level	Vmax	Smax (N)	M (Nm)	
O_4	$5304\overset{3}{\cancel{0}}$	36620	73240	(h = 4)
O_3	$856\overset{45}{\cancel{50}}$	$448\overset{85}{\cancel{90}}$	$897\overset{7}{\cancel{8}}0$	(h : 4)
O_2	$1248\overset{59}{\cancel{60}}$	53150	106300	(h : 4)
O_1	$1709\overset{46.5}{\cancel{50}}$	$614\overset{15}{\cancel{20}}$	$1535\overset{37.5}{\cancel{50}}$	(h : 5)

The farthest leeward column will be subjected to the superimposed axial load + Vmax given above.

The column on the bending axis, on the other hand will be subjected to super imposed axial load plus a bending moment 'M' given above.

These critical combinations for various panels of these columns are tabulated below.

Panel	Farthest leeward column		Column on bending axis	
	Axial load (N)	Vmax (N)	Axial load (N)	M (Nm)
$O_4 O_4'$	1655451.93 ~~1808957~~	$5304\overset{3}{\cancel{0}}$	1655451.93 ~~1808957~~	73240
$O_3 O_3'$	1708016.93 ~~1861522~~	$856\overset{45}{\cancel{50}}$	1708016.93 ~~1861522~~	$897\overset{7}{\cancel{8}}0$
$O_2 O_2'$	1760581.93 ~~1911087~~	$1248\overset{59}{\cancel{60}}$	1760581.93 ~~1911087~~	106300
$O_1 O_1'$	1822766.93 ~~1976272~~	$1709\overset{46.5}{\cancel{50}}$	1822766.93 ~~1976272~~	$1535\overset{37.5}{\cancel{50}}$

According to IS Code, when effect of wind load is to be considered, the permissible stresses in the materials may be increased by 33⅓ %.

Use M_{20} Concrete, $\sigma_{cbc} = 7\ N/mm^2$, $\sigma_{cc} = 5\ N/mm^2$. For steel $\sigma_{st} = 230\ N/mm^2$. All the three can be increased by $33\frac{1}{3}\%$ when taking into account wind action.

Diameter of Column = 700 mm

Use 12 bars of 30 mm dia at an effective Cover of 40 mm.

$$A_{sc} = \pi/4 \times 30^2 \times 12 = 8482\ mm^2$$

$$\text{Equivalent area of Column} = \pi/4\,(700)^2 + (13.33-1)\,8482$$
$$= 489478.06\ mm^2$$

$$\text{Equivalent moment of Inertia} = \frac{\pi d^4}{64} + (m-1)\frac{A_{sc}\,d'^2}{8}.$$

where $d = 700\,mm$, $d' = 700 - 2\times40 = 620\ mm$

$$I_c = \frac{\pi}{64} \times 700^4 + (13.33-1) \times \frac{8482 \times 620^2}{8}$$

$$= 1.6811 \times 10^{10}\ mm^4$$

$$\text{Direct stress in column, } \sigma_{cc}' = \frac{\overset{1822766.93}{\cancel{1976272}}}{489478.06} = \overset{3.72}{\cancel{4.03}}\ N/mm^2$$

$$\text{Bending stress in column} = \sigma_{cbc}' = \frac{1535\overset{37.5}{\cancel{50}} \times 1000}{1.6811 \times 10^{10}} \times 350$$

$$= 3.19\ N/mm^2$$

For the safety of the column, we have

$$\frac{\sigma_{cc}'}{\sigma_c} + \frac{\sigma_{cbc}'}{\sigma_{cbc}} \geq 1 \Rightarrow \frac{\overset{3.72}{\cancel{4.03}}}{1.33\times5} + \frac{3.19}{1.33\times7} = \overset{0.9020}{\cancel{0.948}} < 1$$

Hence Safe.

Use 10mm dia wise rings of 250 mm c/c to tie up main reinforcement.

Since the Columns are of 700mm dia, increase the width of curved beam f 600 to 700 mm.

step 10 :- Design of braces :-

The braces are designed for a bending moment equal to sum of the moments in the Column Just above and below brace level.

B.M for brace = load on each Column $\times h/2$

$= \overset{33060}{\cancel{1770477.1}} \times 4/2 = \overset{66120}{\cancel{3540954.2}}$ Nm

Let the brace Section be $300 \times \overset{600}{\boxed{700}}$ mm

The Section will be designed as a doubly reinforced beam Section,

Let the effective Cover = 40mm.

Distance between centres of Compressive and tensile steel

$= \overset{600}{\boxed{700}} - 80 = \overset{520}{\boxed{620}}$ mm

$$A_{sc} = A_{st} = \frac{\overset{66120}{\cancel{3540954.2}} \times 1000}{\underset{150}{\boxed{230}} \times \underset{520}{\boxed{620}}} = 463.67 \text{ mm}^2$$

Provide 5 bars of 12mm dia at top and 5 similar bars at the bottom.

$$\text{S.F for the brace} = \frac{\text{B.M for brace}}{\text{half span of brace}}$$

$$= \frac{66120}{(10/2)} = 13224 \text{ N}$$

$$\text{Nominal shear stress, } \tau_v = \frac{V_u}{bd} = \frac{13224}{300 \times \underset{560}{\boxed{660}}} = 0.06 \text{ N/mm}^2$$

$$P_t = \frac{100 A_{st}}{bd} = \frac{100 \times 5 \times \pi/4 \times 12^2}{300 \times \boxed{660}\,560} = 0.285$$

For $P_t = 0.285\%$, $\tau_c = 0.37$ N/mm²

$\tau_v < \tau_c$ - Hence provide nominal shear reinforcement.

Provide 2 legged 8mm dia stirrups @ 220mm c/c.

Step 11:- Design of raft foundation :-

step-10 ↙

Vertical load from filled tank and columns $= \overset{1822766.93}{\cancel{1976272}} \times 8$

$= \overset{14582135.44}{\cancel{15810176}}$ N

weight of water $= \overset{7915585.55}{\cancel{9143627}}$ N ← step-9

Vertical load of empty tank and columns $= \overset{14582135.44 - 7915585.55}{\cancel{15810176 - 9143627}}$

↙ Vmax step-10 = 6666549 N

V_{max} due to wind load $= 1709\overset{46.5}{\cancel{50}} \times 8 = 1367\overset{572}{\cancel{600}}$ N which is less tha

33⅓ % of the super imposed load.

Assume Self weight = 10% $= \overset{1458213.54}{\cancel{1581017.6}}$ N.

∴ Total load $= \overset{14582135.44 + 1458213.54}{\cancel{15810176 + 1581017.6}} = \overset{16040348.98}{\cancel{17391193.6}}$ N.

∴ Area of foundation required $= \dfrac{\overset{16040348.98}{\cancel{17391193.6}}}{S.B.C}$

$= \dfrac{\overset{16040348.98}{\cancel{17391163.6}}}{150 \times 10^3} = \overset{106.93}{\cancel{115.94}}\ m^2$

Circumference of Column circle $= \pi D = \pi \times 10 = 31.42$ m

∴ width of foundation $= \dfrac{\overset{106.93}{\cancel{115.94}}}{31.42} = \overset{3.403}{\cancel{3.69}}$ m

Hence inner diameter $= 10 - \overset{3.403}{\cancel{3.69}} = \overset{6.596}{\cancel{6.31}}$ m

outer diameter $= 10 + \overset{3.403}{\cancel{3.69}} = 13.\overset{403}{\cancel{69}}$ m.

Area of annular raft $= \pi/4 \left(13.\underset{403}{\cancel{69}}^{2} - 6.\underset{59}{\cancel{31}}^{2}\right) = \overset{106.98}{\cancel{115.94}}\ m^2$

Moment of inertia of slab about a diametrical axis

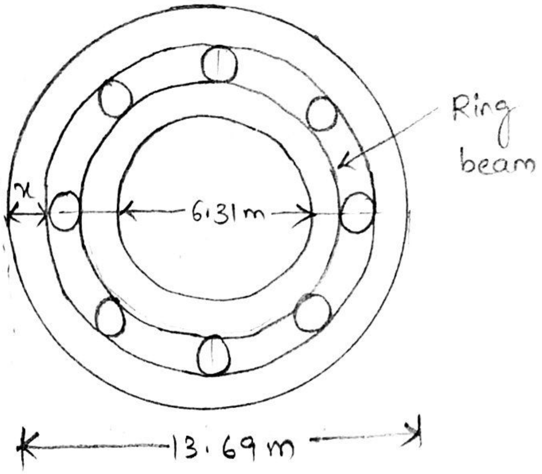
$= \dfrac{\pi}{64}\left(13.\underset{403}{\cancel{69}}^{4} - 6.\underset{59}{\cancel{31}}^{4}\right) = \overset{1491.5}{\cancel{1646.36}}\ m^4$

Total load, tank empty

$$= 6666549 + \overset{1458213.54}{\cancel{1581017.6}} = \overset{8124762.54}{\cancel{8247566.6}}\ \text{N}$$

∴ Stabilising moment

$$= \overset{8124762.54}{\cancel{8247566.6}} \times \frac{13.\overset{403}{\cancel{69}}}{2} = \overset{54448096.16\ \text{Nm}}{\cancel{56454593.38\ \text{Nm}}}$$



Let the base of the raft be 2m below ground level.

∴ M_w at base

$$= 134720 \times 23.7 + 11760 \times 18 + 33060\,(14+10+6) = 4396344\ \text{Nm}$$

Hence the soil pressures at the edges along a diameter are

(a) Tank full $= \dfrac{\overset{16040348.98}{\cancel{17391193.6}}}{\underset{\cancel{115.94}}{106.98}} \pm \dfrac{4396344}{1646.36} \times \dfrac{13.69}{2}\ \cancel{8}$

(Area of annular raft)

$$= \overset{149937.82}{\cancel{150001.66}} \pm 18278.48$$

$$= 1682\overset{16.3}{\cancel{80.15}}\ \text{N/m}^2 \quad (\text{or}) \quad 131\overset{659.34}{\cancel{723.18}}\ \text{N/m}^2$$

(b) Tank Empty $= \dfrac{\overset{8124762.54}{\cancel{8247566.6}}}{\underset{\cancel{115.94}}{106.98}} \pm \dfrac{4396344}{1646.36} \times \dfrac{13.69}{2}$

$$= \overset{75946.55}{\cancel{71136.56}} \pm 18278.48$$

$$= \overset{94225.03}{\cancel{89414.98}}\ \text{N/m}^2 \quad (\text{or}) \quad \overset{57668.07}{\cancel{52858.08}}\ \text{N/m}^2$$

Under the wind load, the allowable bearing capacity is increased to

150 × 1.33 = 199.95 ≃ 200 kN/m² which is greater than the maximum

Soil pressure 168.2~~8~~ kN/m²

Hence the foundation raft will be designed only for superimposed load.

The layout of foundation is shown in figure.

A ring beam of 700mm width may be provided. The foundation will be designed for an average pressure p

$$p = \frac{\overset{1458213.54}{\cancel{15810176}}}{\underset{106.93}{\cancel{115.94}}} = 136365.15 \ N/m^2$$

The overhang 'x' of raft slab = $\frac{1}{2}\left(\frac{1}{2}(13.69 - 6.31) - 0.7\right)$

$= 1.495 m$

Bending moment $= 136365.15 \times \frac{1.495^2}{2} = 152389.76$ Nm

Shear force $= 136365.15 \times 1.495 = 203865.89$ N.

Effective depth, $d = \sqrt{\frac{152389.76 \times 1000}{1000 \times 0.897}} = 412.17$ mm

Provide 460 mm thick slab with effective depth of 420 mm. Decrease the total depth of 250 mm at the edges.

$$A_{st} = \frac{152389.76 \times 1000}{230 \times 0.906 \times 420} = 1741.2 \ mm^2$$

$\leftarrow (\sigma_{st} \times J \times d)$

Using 16 mm dia bars, spacing $= \frac{\pi/4 \times 16^2}{1741.2} \times 1000 = 115.48$ mm

Hence provide 16mm ϕ radial bars @ 110mm c/c at the bottom of slab

Area of distribution steel $= \frac{0.15}{100} \times 460 \times 1000 = 690 \ mm^2$

Using 10mm dia bars, spacing $= \frac{\pi/4 \times 10^2}{690} \times 1000 = 113.84$ mm

Hence provide 10mm dia bars @ 110mm c/c at support. Increase this spacing to 200mm at the edge.

Design of circular beam of raft :-

The design of circular beam of raft will be practically similar to the circular beam B_2 provided at the top of the columns.

$$\text{Design load} = \frac{15810176}{\pi \times 10} = 503253.53 \ \text{N/m}$$

The circular beam B_2 was designed for $w = 450890$ N/m

Hence the bending moment will be increased in the ratio of

$$\frac{503253.53}{450890} = 1.116$$

Maximum -ve B.M at support $= M_0 = 1.116 \times 584311.49 \simeq 652091.62$ Nm

Maximum +ve B.M at mid span $= M_c = 1.116 \times 265596.13 \simeq 296405.28$ Nm

Max. Torsional moment $= M_m = 1.116 \times 44266.02 = 49400.87$ Nm.

At $\phi = \phi_m = 9\frac{1}{2}^\circ$, $S = F = 1.116 \times 511518.47 \simeq 570854.61$ N

$$\text{Effective depth, } d = \sqrt{\frac{652091.62 \times 1000}{0.897 \times 700}}$$ (Assume b = dia of column = 700 mm)

$= 1019.08$ mm

Keep total depth of 1200 mm. using effective cover 60mm,

Effective depth = 1140mm.

Main and longitudinal reinforcement :-

Equivalent B.M, $M_{e1} = M + M_t$

$$M_t = \frac{T \cdot (1 + D/b)}{1.7} = \frac{49400.87 \left(1 + \frac{1.2}{0.7}\right)}{1.7} = 78875.33 \ \text{Nm}$$

$$M_{e1} = 652091.62 + 78875.33 = 730966.95 \ \text{Nm}.$$

$$A_{st} = \frac{M_{e1}}{\sigma_{st}\, J d} = \frac{730966.95 \times 1000}{\cancel{230}\ 150 \times 0.874 \times 1160} = 4806.58\ mm^2$$

using 25 mm dia bars, No. of bars $= \frac{4806.58}{\pi/4 \times 25^2} = 9.79 \simeq 10$ bars

Hence provide 10 bars of 25 mm diameter.

Transverse reinforcement :-

$S = 570854.61\ N$

$$S_e = V_e = S + 1.6\frac{T}{b} = 570854.61 + 1.6 \times \frac{49400.87}{700}$$

$$= 570967.52\ N$$

Equivalent shear stress $= \frac{V_e}{bd} = \frac{570967.52}{700 \times 1160} = 0.703\ N/mm^2$

$$p_t = \frac{100\, A_{st}}{bd} = \frac{100 \times 10 \times \pi/4 \times 25^2}{700 \times 1160} = 0.604\,\%$$

$\tau_c = 0.512\ N/mm^2$ for $p_t = 0.604\,\%$

Since $\tau_{ve} > \tau_c$. shear reinforcement is necessary.

The area of c/s, A_{sv} of the stirrups is given by

$$A_{sv} = \frac{T\, S_v}{b_1 d_1 \sigma_{sv}} + \frac{V\, S_v}{2.5\, d_1\, \sigma_{sv}}$$

$$\frac{A_{sv}}{S_v} = \frac{49400.87}{595 \times 1095 \times 150} + \frac{570967.52}{2.5 \times 1095 \times 150} = 1.39$$

$b_1 = (700 - 40 \times 2) - 25 = 595\ mm$

$d_1 = 1200 - (40 \times 2) - 25 = 1095\ mm$

Minimum transverse reinforcement is ~~given~~ governed by

$$\frac{A_{sv}}{S_v} \geq \left(\frac{\tau_{ve} - \tau_c}{\sigma_{sv}}\right) b = \left(\frac{0.703 - 0.512}{150}\right) \times 700 = 0.891$$

Using 12 mm dia 4 legged stirrups, $S_v = \frac{4 \times \pi/4 \times 12^2}{1.39} = 325.5 mm$

However spacing should be the least of

$x_1 = 595 + 25 + 12 = 632 mm$

$y_1 = 1095 + 25 + 12 = 1032 mm$

$\frac{x_1 + y_1}{4} = \frac{632 + 1032}{4} = 416 mm$

Hence provide 12mm ϕ 4 legged stirrups @ 300 mm c/c.

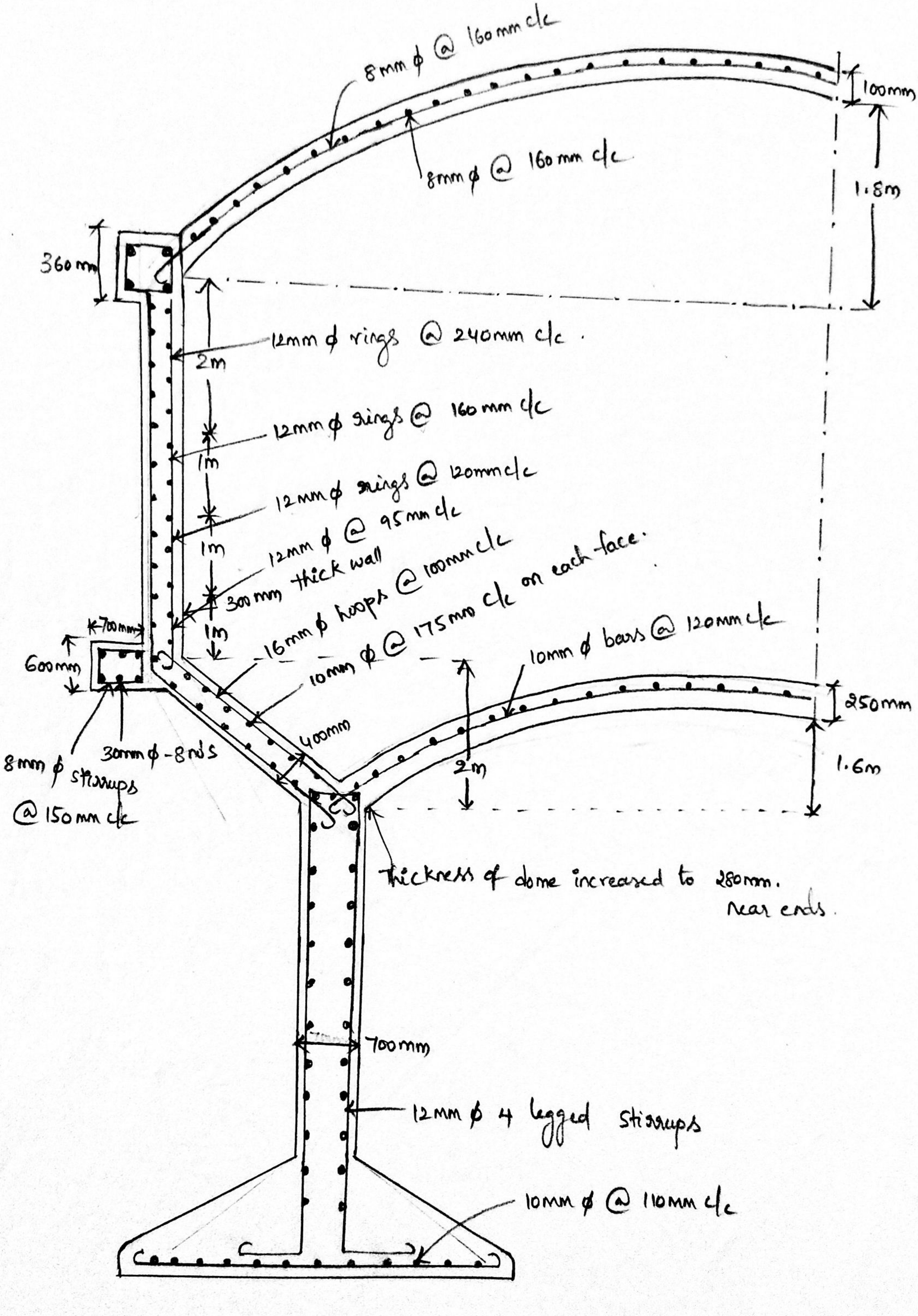
Side face reinforcement :-

Since the depth is more than 450mm, provide side face reinforcement @ 0.1 %.

$A_t = \frac{0.1}{100} \times 700 \times 1200 = 840 mm^2$

Provide 3-16mm ϕ on each face.

$A_{t \, provided} = 3 \times \pi/4 \times 16^2 = 1206 mm^2$



Reinforcement Details