Design and Analysis of Overhead Water Tank at Phule Nagar, Ambernath

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Abstract - *In India more than 68% of its total population lives in rural area. Domestic water is major problem in this area, So as to solve this problem innovative design and solutions to existing problem is essential hence for that study of Elevated Storage Reservoir (ESR) is undertaking. There are so many case studies and report on failure during and post construction of ESR. The purpose of study of the ESR is to design and analysis safe ESR, Where in the damage to the structure and it's structural components even by natural hazard such as earthquake can be minimized. Indian standard for the design of liquid retaining structures have been revised in 2009. This revised edition Incorporated limits state design method. Limit state design method for water retaining structure was not adopted so far as liquid retaining structure should be crack free. However, This edition of Indian standard adopts limit state method mainly considering two aspects. Firstly it limits the stresses in steel so that concrete is not over stressed and in second aspect it limits the cracking width. This project gives in brief, The theory behind the design of liquid retaining structure (Elevated Circular Water Tank) using Limit state method with reference to IS 3370(2009)and Is 456:2000*

Keywords- *Population, Elevated service reservoir, Natural hazard, limit state method, IS code*

1. INTRODUCTION

Water tanks are liquid storage containers. These containers are usually storing water for human consumption, irrigation, fire, agricultural farming chemical manufacturing, food preparation, rainwater harvesting as well as many other possible solutions. Water plays a predominant role in day to day life so water storage is necessary to store the water.

The main objectives in design of water tanks are to provide safe drinkable water after storing for a long time, optimizing cost strength, service life, and performance during a special situation like earthquakes. The other objectives are to maintain pH of the water and to prevent the growth of the microorganism. Water is susceptible to a number of ambient negative influences, including bacteria, viruses, algae, change in pH and accumulation of minerals accumulated gas. A design of water tanks or container should do not harm to the water.

Water tanks parameters include the general design of the tank and choice of construction materials, linings. Reinforced concrete water tank design is based on IS code. The design depends on the location of tank i.e, overhead, on the ground or underground water tanks. Tanks can be made of RCC or even of steel. The overhead tanks are usually elevated from the ground level using a number of column and beams. On the other hand, the underground tanks rest below the ground level.

Water tanks are classified into two types based on position and shape of tanks: -Based on Location the water tanks are classified into three ways: -

- Underground water tanks
- Tanks are resting on the ground
- Elevated or overhead water tanks

Also, the water tanks are classified based on the shapes: -

- Circular tanks
- Rectangular tanks
- Intze tanks
- Circular tanks with conical bottom
- Square tanks

These structures plays a crucial role in storing water which can be used in various day to day activities, mostly in the urban region especially in Residential apartments which happen to be this project.

The common materials used for the construction of water tanks are concrete steel and masonry. RCC is commonly used in construction because it is supposed to be a durable material giving long maintenance free service.

The permeability of any uniform and thoroughly compacted concrete of given mix proportions is mainly dependent on the water-cement ratio. The increase in water-cement ratio results in an increase in the permeability. The decrease in water-cement ratio will, therefore, be desirable to decrease the permeability, but very much reduced water-cement ratio may cause compaction difficulties and prove to be harmful also.

2. Objectives

- To make a study about the design and analysis of water tanks.
- To make a study about the guidelines for the design of liquid retaining structure according to IS code.
- To know about the design Philosophy for the safe and economical design of water tanks.
- To study the various forces acting on a water tank. Understanding the most important factors that play role in designing of water tanks.
- Preparing a water tanks design which is economical and safe, providing proper steel reinforcement in concrete and studying its safety according to various code.

3. Data Collection

Table -1: Detail of Data Collection

1.	Capacity Of Tank	1000 cum
2.	Soil Bearing Capacity	20 T/sq.mt
3.	Height Of Tank From Ground	16 m
4.	Grade Of Concrete	• M30 (For All Members), • M25 (For Staging)
5.	Ground Water Level	3 m Below Existing Ground
7.	External Forces on Tank	Basic Wind Speed 44 m/s
8.	Free Board	0.3 m
9.	Width Of Gallery	1.2 m
10.	Earthquake Zone	IV
11.	Thickness Of Wall	230 mm
12.	Excavation	Up to 3.30m
15.	Types OF Staircase	Spiral Staircase
19.	Use Of Water	Domestic Purpose Only
20.	Water Provided In Area	Phulenagar
21.	Method Of Water Cost	Metering
22.	Current Population In Year 2011	4106
24.	Population Forecasting 2021	7400
25.	SPT Value [N]	30

Table -2: Soil Profile

Layer	Strata	Thickness In mm
Layer I	Soil With Murum	up To 0.10 m
Layer II	Yellowish/Brownish Completely Weathered Rock [Murum]	Below Layer I upto 3.60 m

4. Methodology

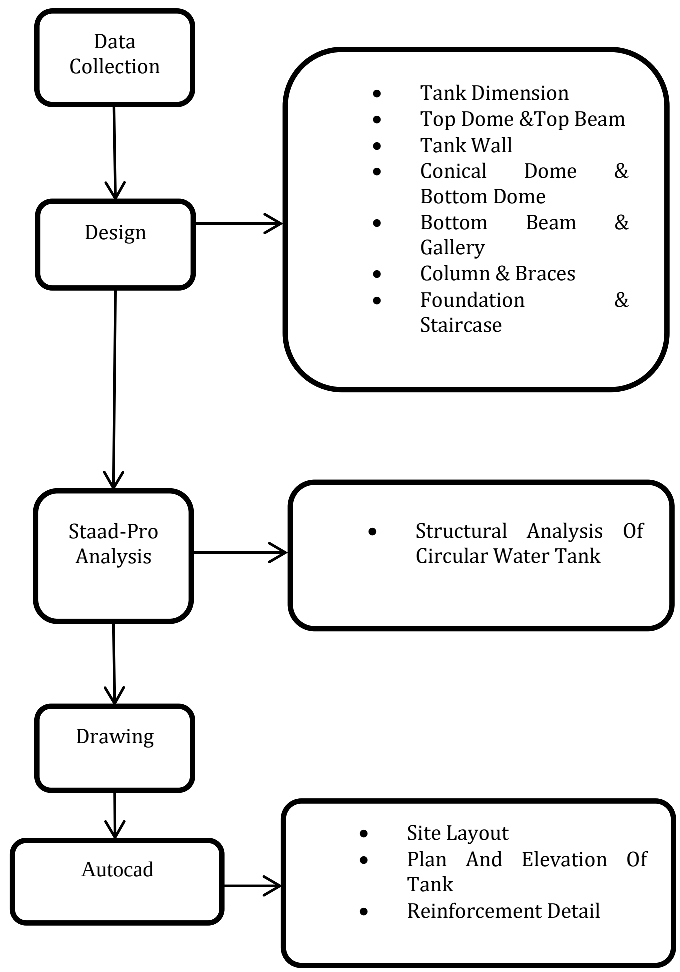

Chart – 1: Methodology

5. Design of intze tank

5.1 POPULATION FORECAST

Population forecast for a village

NOTE: The data of population given by the department is not as per the census of India it may vary.

Table -3: Population data

Year	Population	X Increase	Y Increase	% Increase	%Decrease
1981	690	-	-	-	-
1991	1470	780	-	113.043	-
2001	2685	1215	425	82.65	30.393
2011	4106	1421	206	52.9236	29.726
Sum	-	3416	641	248.6166	60.119
Avg.	-	1138.67	320.5	82.8722	30.0595

1. Arithmetic Progression Method

Population (P) = $P_o + nx = 4106 + 1 \times 1138.67 = 5244.67$

2. Geometric Progression Method

$P_n = P_o\left(1 + \frac{r}{100}\right)^n$

r = $\sqrt[n]{1.13043 \times 0.8265 \times 0.529236} = 0.79 = 79.08\ \%$

$P_{2021} = 4106 \times \left(1 + \frac{79.08}{100}\right)^1 = 7353.02$

3. Incremental increase method

$P_n = P_o + nx + \frac{n\times(n+1)}{2} \times y$

When n = 1

$P_{2021} = 4106 + 1 \times 1138.67 + \frac{1(1+1)}{2} \times 320.5 = 5565.17$

Assuming changing increase rate method

$P_{2021} = (52.9236 - 30.0595) \times \frac{4106}{1000} = 93.879$

Considering geometric increase method

P = 7353.02 = 7400

Therefore design population of 7400

Assuming per capita demand 135 lpcd

Capacity required = 135 × 7400 Ipcd = 999000 lpcd

In one day = 999000 lpcd

Design volume or capacity = 1×10^6ltr = 1000 m^3

5.2 Dimension of the tank

Let the diameter of cylindrical portion = D = 16m

Let the radius of cylindrical portion = R = 8m

Let the diameter of ring beam = B_2= 10 m

Height of conical dome = h_o= 3 m

Rise, h_1 = 1.8 m

Rise, h_2 = 1.6 m

The radius of bottom dome = $R^2 = (2R_2 - h_2) \times h_2$

$= \left(\frac{Do}{2}\right)^2 = (2R_2 - h_2) \times h_2$

$= 5^2 = (2R_2 - 1.6) \times 1.6$

$= R_2 = 8.61$ m

$\sin\theta_2 = \frac{5}{8.61} = 0.5807$

$\theta_2 = \sin^{-1}(0.5807)$

$\theta_2 = 35.50°$

$\cos\theta_2 = \cos(35.50) = 0.8141$

$\tan\theta_2 = \tan(35.50) = 0.7133$

$\cot\theta_2 = \cot(35.50) = 1.4019$

Let h be the height of cylindrical portion,

Capacity of tank,

$V = \frac{\pi}{4} \times D^2 \times h + \frac{\pi h_o}{12}(D^2 + Do^2 + D \times Do) - \frac{\pi {h_2}^2}{3}(3R_2\text{-}h_2)$

$1000 = \frac{\pi}{4} \times 16^2 \times h + \frac{\pi\times3}{12}(16^2+10^2+16\times10) - \frac{\pi\times1.6^2}{3}(3\times8.61\text{-}1.6)$

h = 3.8 m ≈ 4 m

Allowing for free board, keep h = 4 m

For top dome, the radius R_1 is given by

$R^2 = h_1(2R_1\text{-}h_1)$

$8^2 = 1.8\,(2R_1\text{-}1.8)$

R_1 = 18.7 m

$\sin\theta_1 = \cos(25.32°) = 0.9039$

5.3 Design of top dome

R_1 = 18.7 m; $\sin\theta_1 = 0.4278$; $\cos\theta_1 = 0.9039$

Let thickness t_1 = 100 mm = 0.1 m

Taking a live load of 1500 N/m² = 1.5 KN/m²

Total pressure per m² on dome =0.1 × γ_c +L.L

= 0.1×25000+1500

= 4000 N/m²

Meridional thrust at edges

$T_1 = \frac{P\times R_1}{1+Cos\emptyset_1} = \frac{4000\times 18.7}{1+0.9039}$ = 39288 N/m

Meridional stress = $\frac{T_1}{1000\times Thickness\ of\ dome} = \frac{39288}{1000\times 100}$ = 0.39 N/mm²

In IS: 3370 (Part-2), Table-2, For M-30 Concrete

Permissible stress in concrete = 8 N/mm²

Therefore, 0.39 N/mm² < 8 N/mm².......... Safe

Maximum hoop stress occurs at the centre and its magnitude = $\frac{pR_1}{t_1} \times \frac{1}{2}$

= $\frac{4000\times 18.7}{0.1\times 2}$

= 374000 N/mm²

= 0.374 N/mm² < 8 N/mm² ....Safe

Provide nominal reinforcement @ 0.3 %

$A_s = \frac{0.3}{100} \times 100 \times 1000$ = 300 mm²

Using 8mm Ø bar @ 160 mm $^c/_c$ in both the direction.

5.4 Design of top ring beam (B_1)

Horizontal Components of T_1 is given by,

$P_1 = T_1\ Cos\theta_1$

= 39288 × 0.9039

= 35512 N/m

Total tension tending to rupture the beam,

$T = P_1 \times \frac{D}{2}$

= 35512 × $\frac{16}{2}$

= 284096 N

Permissible stress in high yield strength deformed bars (HYSD) = 150 N/mm²

Ash = $\frac{284096}{150}$ = 1894 mm²

Therefore, No. of 20 mm Ø bars = $\frac{1894}{\frac{\pi}{4}\times 20^2}$ = 6 No.

Actual Ash provided = $\frac{\pi}{4} \times 20^2 \times 6$ = 1885 mm²

The area of cross – section of ring beam is given by = $\frac{T}{A+(m-1)Ash_{Actual}}$

= $\frac{284096}{A+8.333\times 1885}$ = 1.3

A = 202827.68 mm²

Provide ring beam of 410 mm depth and 500 mm width.

Therefore, Provide 8 Ø - 2 legged stirrups.

Asv = 2 × $\frac{\pi}{4}$ × 8² = 100 mm²

Sv = $\frac{0.87\times fy\times Asv}{0.4\times b} = \frac{0.87\times 415\times 100}{0.4\times 500}$ = 180.525 mm ≈ 180 mm

IS 456:2000, page-48

i. 0.75 × D = 0.75 × 410 = 307 mm
ii. 300 mm

Provide 8 Ø - 2 legged vertical stirrups @ 180 mm $^c/_c$

A_{pro} = 410 × 500 = 205000 mm² .... OK

5.5 Design of cylindrical wall

The tank load is assumed to be free at top and bottom maximum hoop tension occurs at the base of the wall, its magnitude being given by,

P = W × h × $\frac{D}{2}$ = 9800 × 4 × $\frac{16}{2}$ = 313600 N/m height

Area of steel

Ash = $\frac{313600}{150}$ = 2247.47 mm² ≈ 2093 mm² per meter height.

Provide ring on both the faces,

Ash on each face = $\frac{2093}{2}$ = 1047 mm²

Spacing of 12 mm Ø ring @ 100 mm $^c/_c$ at bottom.

This spacing can be increased at the top.

Actual $Ash_{provided} = \frac{1000\times113}{100} = 1130$ mm² on each face.

Permitting 1.2 N/mm² stress on composite section, $\frac{313600}{1000\times t+(9.33-1)\times1130\times2} = 1.2$

t = 242.51 mm

Minimum thickness = 3H + 5 = (3×4) +5 = 17 cm

However provide t= 300 mm at bottom and taper it to 200 mm at top.

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

Percent distribution steel = 0.24 % of surface zone of wall

Therefore, Ash = $\frac{0.24\times250\times1000}{100}$ = 600 mm²

Area of steel on each face = 300 mm²

Spacing of 8 mm Ø bars = $\frac{1000\times\frac{\pi}{4}\times8^2}{300}$=167.7 mm ≈ 160mm

Hence provide 8 mm Ø bars @ 160 mm $^c/_c$ on both face. Keep a clear cover of 25 mm. Extend the vertical bars of outer face into the dome to take care of the continuity effects.

To resist the hoop tension at 2m below top.

Ash = $\frac{2}{4}\times2093$ = 1047 mm²

Spacing of 12 mm Ø ring = $\frac{1000\times113}{\frac{1047}{2}}$ = 215 mm= 210 mm

Hence, provide the rings @ 210 mm $^c/_c$ in top 2 m height.

At 3 m below the top, Ash = $\frac{3}{4}\times2093 = 1570$ mm²

Spacing of 12 mm Ø rings = $\frac{1000\times113}{\frac{1570}{2}}$ = 144 mm ≈ 140 mm

Hence, provide the rings @ 140 mm $^c/_c$ in the next 1 m height.

In the last 1 m height (3 m to 4 m) provide rings 100 mm $^c/_c$ as found earlier.

5.6 Design of ring beam B_3

The ring beam connects the tank wall with conical dome. The vertical load at the junction of the wall with conical dome. The horizontal components of the thrust causes hoop tension at the junction. The ring beam is provided to take up this hoop tension.

The load W transmitted through tank wall at the top of conical dome consists of the following;

1. Load of top dome = T_1 Sin $Ø_1$ = 39288 × 0.4278 = 16807 N/m

2. Load due to the ring beam B_1 = 0.41 × (0.5-0.2) × 1 × 25000 = 3075 N/m

3. Load due to tank wall = $4\left[\frac{0.2+0.3}{2}\right]\times1\times2500$ = 25000 N/m

4. Self load of beam B_3 (1 m × 0.6 m, say) = (1 - 0.3) × 0.6 × 25000 = 10500 N/m

Total load, W = 55382 N/m

Inclination of conical dome wall with vertical = $Ø_0$ = 45°

Sin $Ø_0$ = Cos $Ø_0$ = 0.7071 = $\frac{1}{\sqrt{2}}$; tan $Ø_0$ = 1

P_W = W × tan $Ø_0$ = 55382 × 1 = 55382 N/m

P_W = W × h × d_3 = 9800 × 4 × 0.6 = 23520 N/m

Hence hoop tension in the ring beam is given by

$P_3 = (W + P_W)\times\frac{D}{2} = (55382 + 23520)\times\frac{16}{2}$ = 631216 N

This to be resisted entirely by steel hoops, the area of which is

Ash = $\frac{631216}{150}$ = 4208 mm²

No of 30 mm Øbars = $\frac{4208}{\frac{\pi}{4}\times30^2}$ = 5.95 ≈ 6 No

Hence, provide 6 rings of 30 mm Ø bars

Actual Ash = 4241 mm²

Stress in equivalent section = $\frac{631216}{(1000\times600)+8.33\times4241}$ = 0.99 N/mm² < 1.2 N/mm²...... Safe

The 8 mm Ødistribution bars (vertical bars) provided in the wall @ 150 mm $^c/_c$ should be taken round the above ring to act as stirrups.

5.7 Design of conical dome

a. Meridional thrust

The weight of water (W_w);

$W_w = \frac{\pi}{4}(16^2 - 10^2) \times 4 \times 9800 + \left\{\left(\frac{\pi \times 3 \times 9800}{12}\right) \times (16^2 + 10^2 + 16 \times 10)\right\} - \frac{\pi}{4} \times 10^2 \times 3 \times 9800$

W_w = 6465398 N

Let the thickness of conical slab be 500 mm.

Total self weight (W_s);

$W_s = 25000 \times \pi \times \left[\frac{16+10}{2}\right] \times 4.24 \times 0.45 = 2164557\ N$

Weight W at B_3 = 55382 N/m

Hence vertical load W_2 per meter run,

$W_2 = \frac{\pi \times D \times W + W_w + W_S}{\pi \times D_O} = \frac{(\pi \times 16 \times 55382) + 6465398 + 2164557}{\pi \times 10} = 363311$ N/m

Meridional thrust T_o in the conical dome is

$T_o = \frac{W_2}{Cos\theta_O} = \frac{363311}{Cos(45)} = 513799\ N/m$

Meridional stress = $\frac{513799}{1000 \times 400}$ = 1.02 N/mm² < 8 N/mm² ……….. Safe

b. Hoop tension

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

$D' = 10 + \left[\frac{16-10}{2}\right] h' = 10 + 3h'$

Intensity of water pressure P = (4 + 3 - h') × 9800

= (7 - h') × 9800 N/m²

Self weight q = 0.5 × 1 × 1× 25000 = 12500 N/m²

Hence, Hoop tension P_o'

$P_o' = \left[\frac{P}{Cos\theta_O} + q \times tan\theta_O\right] \times \frac{D'}{2}$

$= \left[\frac{(7-h') \times 9800}{Cos(45)} + 12500 \times \tan(45)\right]\left[\frac{10+3h'}{2}\right]$

$= [13859(7 - h') + 12500][5 + 1.5h']$

$= [97013 - 13859\,h' + 12500][5 + 1.5h']$

$= [109513 - 13859h'][5 + 1.5h']$

$= \left[547505 - 69295h' + 164270h' - 20789h'^2\right]$

$= 547505 + 94975h' - 20789h'^2$

The values of P_o' at h'= 0, h'=1, h'=2, h'=3 are tabulated below;

Table -4: Hoop tension

h'	Hoop tension
0	547505
1	621691
2	654299
3	645329

For maxima $\frac{dPo'}{dh'}$ = 0 = 94975 – 2 × 20789h'

From which h' = 2.28 m

Max P_o' = 535065 + 91225 × 2.28 – 20789 × 2.28²

= 655978 N

c. Design of walls

Meridional stress = 1.02 N/mm²

Max. Hoop stress = 655978 N

Whole of which is to be resisted by steel,

As = $\frac{655978}{150}$ = 4378 mm²

Area of each face = 2189 mm²

Spacing of 16 mm Ø bars = $\frac{100 \times 201}{2189}$ =91.82 mm ≈ 90 mm

Hence provide 16 mm Ø bars hoops @ 90 mm $^c/_c$ on each face.

Actual As = $\frac{1000 \times 201}{90}$ = 2233 mm²

Max. Tension stress in composite section = $\frac{655978}{500 \times 1000 + 8.33 \times 2233 \times 2}$

= 1.2 N/mm²

This is equal to permissible value of 1.2 N/mm²

In the Meridional direction, provide reinforcement @

$\left\{0.24 - \left[\frac{500-100}{500-100}\right] 0.1\right\}\% = 0.24\ \%$

As_d = 0.24 × 4466 =1072 mm² or 536 mm² on each face.

Spacing of 10 mm Ø bars = $\frac{1000\times78.5}{536}$ = 147 mm

Hence, provide 10 mm bars @ 140 mm $^c/_c$ on each face.

Provide clear cover of 25 mm.

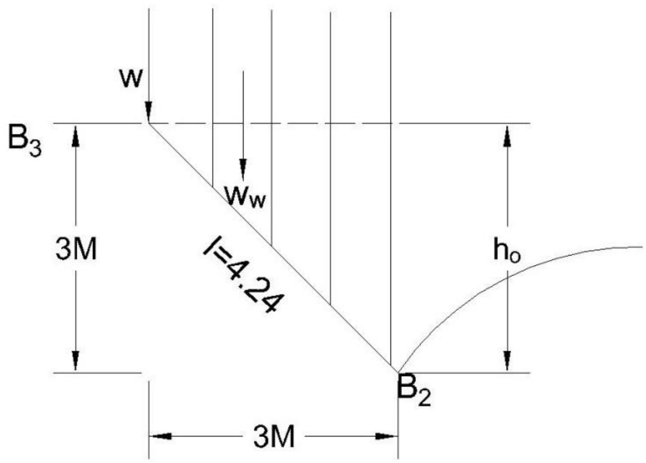

Fig -1: Load on conical dome

5.8 Design of bottom dome.

R_2 = 8.61 m ; Sin θ_2 = 0.5807 ; Cos θ_2 = 0.8141

Weight of water W_o on the dome

$$W_o = \left[\frac{\pi}{4} \times D_o{}^2 \times H_o - \frac{\pi\times h_2{}^2}{3}(3R_2 - h_2)\right] \times w$$

$$W_o = \left[\frac{\pi}{4} \times 10^2 \times 7 - \frac{\pi\times1.6^2}{3}(3 \times 8.61 - 1.6)\right] \times 9800$$

W_o= 4751259 N

Let the thickness of bottom dome be 250 mm.

Self weight = 2 × π × 8.61 × 1.62 ×25000 × 0.25 = 540982 N

Total weight W_T = 4751259 + 540982

= 5292241 N

Meridinal thrust T_2 = $\frac{W_T}{\pi\times D_o\times Sin\theta_2}$ = $\frac{5292241}{\pi\times10\times0.5807}$ = 290093 N/m

Meridional stress = $\frac{290093}{250\times1000}$ = 1.16 N/mm² < 8 N/mm² .... Safe

Intensity of load per unit area, P_2 = $\frac{WT}{2\pi\times R_2\times h_2}$ = $\frac{5292241}{2\pi\times8.61\times1.6}$ = 61142 N/m²

Max. Hoop stress at centre of dome =$\frac{P_2\times R_2}{2\times t_2}$ =$\frac{61142\times8.61}{2\times0.25}$ = 1.05 N/mm² <2 .... Safe

Area Of minimum steel= 0.24 – $\left[\frac{250-100}{500-100}\right]$ × 0.1 = 0.2 %

As = 0.2 × 2233 = 447 mm² in each direction.

Spacing of 10 mm Ø @170 mm $^c/_c$ on both the direction. Also provide 16 mm Ø meridional bar @100 mm $^c/_c$ near water face, for 1 m length to take care of continuity effect. The thickness of dome may be increased from 250 mm to 280 mm gradually in 1 m length.

5.9 Design of bottom circular beam B2

Inward thrust from conical dome = T_o Sin θ_o= 513799 × 0.7071 = 363307 N/m

Outward thrust from bottom dome = T_2 Cos θ_2= 290093 × 0.8141 = 236165 N/m

Net inward thrust = 363307 – 236165 = 127142 N/m

Hoop compression in beam = 127142 × $\frac{10}{2}$ = 635710 N

Assuming the size of beam to be 600 × 1200 mm

Hoop stress = $\frac{635710}{600\times1200}$ = 0.883 N/mm²

Vertical load on beam, per meter run = T_o Cos θ_o + T_2 Sin θ_2

= 513799 × 0.7071 + 2900093 × 0.5807

= 531764 N/m

$$\left[Alternatively\ vertical\ load = W_2 + \frac{Wt}{\pi \times D_o} \right.$$
$$\left. = 363311 + \frac{5292241}{\pi \times 10} = 531798\ N/m \right]$$

Self weight = 0.6 × 1.20 × 1 × 25000 = 18000 N/m

The load on beam = W = 531768 + 18000 = 547968 N/m

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

2θ = 45° =$\frac{\pi}{4}$

θ = 22.5° = $\frac{\pi}{8}$ radius

C_1 = 0.066, C_2 = 0.030, C_3 = 0.005

θ_m = 9.$\frac{1°}{2}$

$WR^2\,(2\theta) = 549768 \times 5^2 \times \frac{\pi}{4} = 10794669$ N.m

Maximum negative B.M at support = $M_o = C_1 \times WR^2\,(2\theta)$

$= 0.066 \times 10794669 = 712448$ N.m

Maximum positive B.M at support = $C_2 \times WR^2\,(2\theta)$

$= 0.030 \times 10794669 = 323840$ N.m

Maximum torsional moment = $M'_m = C_3 \times WR^2 \times 2\theta$

$= 0.005 \times 10794669 = 53973$ N.m

For M30 concrete σcbc = 10N/mm²

For HYSD bars σst = 150 N/mm²

K = 0.378, j = 0.874, R = 1.156

Required effective depth = $\sqrt{\frac{712448\times1000}{600\times1.156}}$ = 1013 mm

However, keep total depth = 1200 mm from shear point of view.

Let d = 1140 mm

Max. Shear force at supports,

$F_0 = WR\,\theta = 549768 \times 5\times\frac{\pi}{8} = 10789467$ N

SF at any point is given by,

$F = WR\,(\theta - \emptyset)$

At $\emptyset = \emptyset_m$, F = 549768 × 5 (22.5°- 9.5°)$\frac{\pi}{180}$ = 623692 N

B.M at the point of maximum torsional moment $(\theta = \theta_m = 9.\frac{1^\circ}{2})$

$M_\theta = WR^2\,(\theta\, sin\emptyset + \theta\, Cot\,\theta\ \times Cos\emptyset - 1)$ (sagging)

$M_\theta = 549768 \times 5^2\,(\frac{\pi}{8}\ Sin\ 9.5 + \frac{\pi}{8}\ Cot\ 22.5^\circ \times Cos\ 9.5^\circ - 1)$

M_θ = - 1767 N.m (sagging)

M_θ = 1767 N.m (Hogging)

The torsional moment at any point,

$M_o{}^t = WR^2\,(\theta Cos\emptyset - \theta Cot\theta \times sin\emptyset - (\theta - \emptyset))$

At the supports, $\emptyset = 0$,

$M_o{}^t = WR^2\,(\theta - \theta)$ = Zero

At the mid span, $\emptyset = \theta = 22.5^\circ = \frac{\pi}{8}\ rad$

$M_o{}^t = WR^2\left[\theta\ Cos\theta - \theta\ \frac{Cos\emptyset}{Sin\emptyset}\ Sin\emptyset\right]$ = Zero

Hence, we have following combinations of B.M at torsional moment.

a. At the supports,

M_0 = 712448 N.m (hogging or negative)

$M_o{}^t$ = Zero

b. At mid span,

M_c = 323840 N.m (sagging or positive)

$M_o{}^t$ = Zero

C. At the point of max. Torsion $(\theta = \theta_m = 9.\frac{1^\circ}{2})$

$M_\emptyset$= 1767 N.m (hogging or negative)

$M_m{}^t$ =53973 N.m

- Main and longitudinal reinforcement

a. Sectional at point of maximum torsion

$T = M_{max}{}^t$ = 53973 N.m

$M_\emptyset = M = 1797$; $M_{e1} = M + M_T$

Where, $M_T = T\left[\frac{1+\frac{D}{b}}{1.7}\right] = 53973\left[\frac{1+\frac{1.2}{0.6}}{1.7}\right]$ = 95247 N.m

M_{e1} = 1767 + 95247 = 97014 N.m

$Ast_1 = \frac{M_{e1}}{\sigma st\times j\times d} = \frac{97014\times1000}{150\times0.874\times1160}$ = 638 mm²

No. of 25 mm Ø bars = $\frac{638}{491} = 1.29$

Let us provide minimum of 2 bars.

Since $M_T > M$,

$M_{e2} = M_T - M$ = 95247 -1767 = 93480 N.m

$Ast_2 = \frac{93480\times1000}{150\times0.874\times1160}$ = 615 mm²

No. of 25 mm Ø bars = $\frac{615}{491} = 1.25 \approx 2$ No

Provide a minimum of 2 bars. Thus at the point of maximum torsion, provide 2-25 mm Ø bars each at top at bottom .

b. Section at Max. Hogging B.M (Support)

M_0 = 712448 N.m = M_{Max} ; $M_o^t = 0$

Ast = $\frac{712448\times1000}{150\times0.874\times1160}$ = 4685 mm^2

No. of 25 mm Ø bars = $\frac{4685}{491}$ = 9.5 = 10 Nos.

Hence provide 8 Nos of 25 mm Ø bars in one layer and 2 bars in the second layer. These will be provided at the top of the section, near supports.

C. Section at Max. Sagging B.M (Mid span)

M_c = 323840 N.m ;$M_c' = 0$

Therefore, For positive B.M steel will be to the other face where stress in steel (σst) can be taken as 190 N/mm^2. The constants for M30 concrete having C = 10 N/mm^2 and M = 9.33 will be

K = 0.324 ; j = 0.892, R = 1.011

Ast = $\frac{323840\times1000}{190\times0.892\times1160}$ = 1647 mm^2

No. of 25 mm Ø bars = $\frac{1647}{491}$ = 3.35 Nos

Hence the scheme of reinforcement will be as follows ;

At the supports, provide 8 -25 mm Ø bar at top layer and 2-25 mm Ø bars in the second layer. Continue these upto the section of maximum torsion (i.e. at $\emptyset_m$ = 9.5° = 0.166 rad) at a distance = 5 × 0.166 = 0.83 m or equal to L_d = 52 Ø =1300 mm from supports.

At the point, discontinue four bars while continue the remaining four bars. Similary provide 4 bars of 25 mm Ø at the bottom, throughout the length. These bars will take care of both the max. Positive B.M as well as maximum torsional moment.

- Transverse reinforcement

a. At point of max. Torsional moment ;

At the point of max. Torsion, v = 633692 N

$V_e = V + 1.6\,\frac{T}{b}$

Where, T = M_m^t = 53973 N.m ; b = 600 mm = 0.6

V_e = 633692 + 1.6 × $\frac{53973}{0.6}$ = 777620 N

$\tau_{ve} = \frac{777620}{600\times1160}$ =1.117 N/mm^2

This is less than τc_{max} , Hence OK

$\frac{100\,As}{bd} = \frac{100(4\times491)}{600\times1160}$ = 0.282

Hence, τc = 0.23 N/mm^2

Since $\tau_{ve} > \tau c$, shear reinforcement is necessary. The area of cross – section Asv of the stirrups is given by

$$Asv = \frac{T\times Sv}{b_1 d_1 \sigma sv} + \frac{V\times sv}{2.5\times d_1\times\sigma sv}$$

Where,

b_1 = 600 – (40 × 2) – 25 = 495 mm

d_1 = 1200 – (40 × 2) – 25 = 1095 mm

$$\frac{Asv}{sv} = \frac{53973\times1000}{495\times1095\times150} + \frac{633692}{2.5\times1095\times150} = 2.207$$

Minimum transverse reinforcement is governed by

$$\frac{Asv}{sv} \geq \left[\frac{\tau_{ve}-\tau c}{\sigma sv}\right] b$$

$$\frac{Asv}{sv} = \frac{1.117-0.23}{150} \times 600 = 3.548$$

Hence depth $\frac{Asv}{sv}$= 3.548

Using 12 mm Ø 4 lgd stirrups, Asv = 4 × 113 = 452 mm^2

Or, Sv = $\frac{452}{3.548}$= 127.39 ≈ 128 mm

However the spacing should not exceed the last of X_1, $\frac{X_1+Y_1}{4}$ and 300 mm where

X_1 = Short dimension of stirrups = 495 + 25 +12 = 532 mm

Y_1 = long dimension of stirrups = 1095 + 25 +12 = 1032 mm

$$\frac{X_1+Y_1}{4} = \frac{532+1032}{4} = 391\ mm$$

Hence provide 12 mm Ø 4 lgd stirrups @ 120 mm $^c/_c$

b. At the point of max. Shear (supports)

At supports, F_o = 1079467 N

$$\tau_v = \frac{1079467}{600\times1160} = 1.55\ N/mm^2$$

At supports, $\frac{100\,As}{bd} = \frac{100(8\times491)}{600\times1160} = 0.564$

τ_c = 0.31 N/mm^2

Hence shear reinforcement is necessary

$V_c = 0.31 \times 600 \times 1160 = 215760$ N

Therefore, $V_s = F_o - V_c = 1079467 - 215760 = 863707$ N

The spacing of 10 mm Ø 4 lgd stirrups having Asv = 314 mm² is given by

$$Sv = \frac{\sigma sv \times Asv \times d}{Vs} = \frac{150 \times 314 \times 1160}{863707} = 63.25\ mm$$

This is small, hence use 12 mm Ø 4 lgd stirrups having ;

$Asv = 4 \times \frac{\pi}{4} \times 12^2 = 452.39$ mm²

At spacing, $Sv = \frac{150 \times 452.39 \times 1160}{863707} = 90$ mm

C. At the mid – span S.F is Zero. Hence provide Minimum / nominal shear reinforcement, given by

$$\frac{Asv}{b.Sv} \geq \frac{0.4}{fy}$$

Or $\frac{Asv}{Sv} = \frac{0.4 \times b}{fy}$ For HYSD bars, fy = 415 N/mm²

$$\frac{Asv}{Sv} = \frac{0.4 \times 600}{415} = 0.578$$

Choosing 10 mm Ø 4 lgd stirrups, Asv = 314 mm²

$Sv = \frac{314}{0.578} = 543$ mm

Max. Permissible spacing = 0.75 × d = 0.75 × 1160 = 870 or 300 mm

Whichever is less, hence provide 10 mm Ø 4 lgd stirrups @ 300 mm $^c/_c$

- Side face reinforcement

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

$A_l = \frac{0.1}{100}(600 \times 1200) = 720$ mm²

Provide 3-16 mm Ø bars on each face, having total $A_l = 6 \times 201 = 1206$ mm²

5.10 Design of gallery

Consider D = 120 mm to 100 mm thick

Outer diameter = 16.4 m of container

Outer diameter = 16.4 + 1.2 + 1.2 m gallery = 18.8 m of gallery

Load on gallery,

W = 0.3 L.L + 0.11 × 2.55 D.L + 0.1 F.F / railing

W = 0.3 × 1 + 0.11 × 2.55 × 1 + 0.1 × 1

W = 0.6805 T/m²

Moment = $0.6805 \times 1.2 \times \frac{1.2}{2} \times \frac{17.6}{16.4} = 0.5258$

Consider d = 120 – 30 – 5 = 85 = 0.085 m

Total $Ast_{required} = \frac{\pi \times 16.4 \times 0.5258}{2.3 \times 0.903 \times 0.085} = 154.096$ cm² with live load

Spacing = $1000 \times \frac{\frac{\pi}{4}}{159.316} = 492.981$ mm

Provide 490 mm spacing 10 Nos TOR bars

$Ast_{provide} = 160.285$ cm²

Spacing at critical section = $3.1416 \times \frac{16.4}{490} = 0.105147 = 105$ mm

3d = 3 × 85 = 255 mm (Whichever is minimum)

5.11 Design of columns

a. Vertical loads on columns

1. Weight of water = $W_w + W_o = 6465398 + 4751259 = 11216657$ N

2. Weight of tank;

   i. Weight of top dome + cylindrical walls etc (W) = 55382 × π × 16
      W = 2783803 N
   ii. Weight of conical dome = W_s= 2164557 N
   iii. Weight of bottom dome = 540982 N
   iv. Weight of bottom ring beam = 18000 × π × 10 = 565487 N
   v. Total weight of tank = 6054829 N

Total superimposed load = 6054829 + 11216657 = 17271486 N

Check;

Total load = Load on bottom beam per meter × π × 10

Total load = 549768 × π × 10

Total load = 17271486 N

Therefore, Load per column = $\frac{17271486}{8}$ = 2158936 N

Let the column be of 700 mm diameter

Weight of column per meter height = $\frac{\pi}{4} \times (0.7)^2 \times 1 \times 25000$ = 9620 N

Let the brace be of 300 × 600 mm size

Length of each brace = L = $R\frac{Sin\frac{2\pi}{n}}{Cos\frac{\pi}{n}} = 5 \times \frac{Sin\frac{\pi}{4}}{Cos\frac{\pi}{8}} = 3.83$ m

$$\left[Alternatively, L = \frac{\pi \times 10}{8} = 3.93\ m\right]$$

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

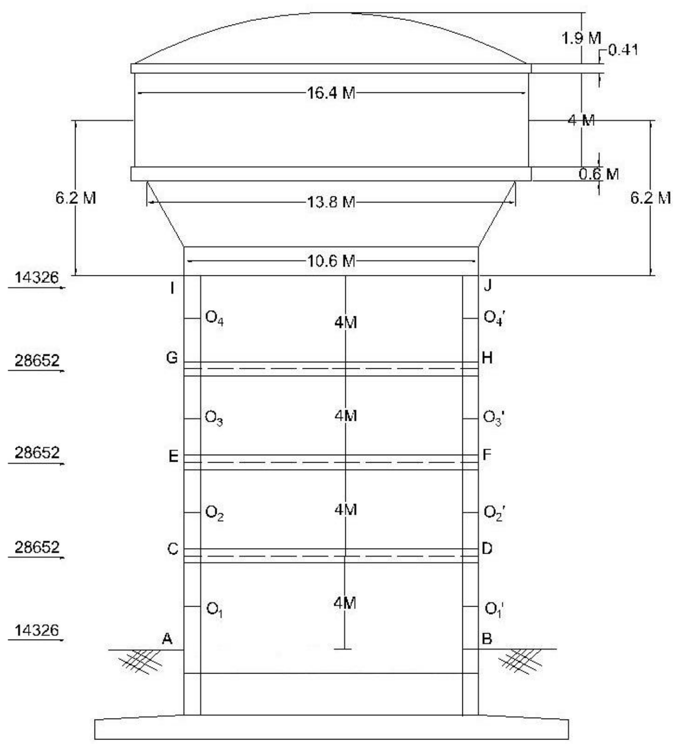

Fig -2: Wind load on tank

Hence total weight of column just above each brace is tabulated below

- Brace GH ; W = 2158936 + 4 × 9620 = 2197416 N
- Brace EF ; W = 2158936 + 8 × 9620 = 2235896 N
- Brace CD ; W = 2158936 + 12 × 9620 = 2274376 N
- Bottom of column ;

W = 2158936 + 17 × 9620 = 2322476 N

b. Wind loads

Total height of structure = 16 + 1.2 + 3 + 4 + 1.9 = 26.1 m

Refer IS 875 part-3

Terrain category 3, class B

Location – Near Mumbai

V_b = 44 m/s .... Design wind speed

Risk co-efficient = K_1 = 1

Table no – 2 K_2, category 3

Total height = 26.1 m

Table -5: Interpolation of k_2 factor

20	1.01
26.1	K_2
30	1.06

K_2 = 1.04

K_3 = 1

Design wind speed = $0.6\ V_z^2$

= $0.6 \times (K_1 \times K_2 \times K_3 \times V_b)^2$

= $0.6 \times (1 \times 1.04 \times 1 \times 44)^2$

= 1256.38656 N/m² ≈ 1300N/m²

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

Wind load on tank, dome & ring beam = $\left[(4 \times 16.4) + \left(16.2 \times \frac{2}{3} \times 1.9\right) + (3 \times 13.8) + (10.6 \times 1.2)\right] \times 1300 \times 0.7$ = 127618 N

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

Wind load on each panel of 4 m height of columns = (4 × 0.7 × 8) × 1300 × 0.7 + (0.6 × 10.6) × 1300 = 28652 N

Wind load at the top end of top panel = $\frac{1}{2} \times 28652 = 14326\ N$

Wind load are shown in diagram. The points of contraflexure O_1, O_2, O_3 & O_4 are assumed to be at the mid height of each panel. The shear forces Qw and moments Mw due to wind at these planes are given below.

Table -6: Shear force & bending moment due to wind load

Level	Qw (N)	Mw (N.m)
O_4	127618 + 14326 = 141944	127618 × 8.2 + 14326 × 2 1075119.6
O_3	127618 + 14326 + 28652 = 170596	127618 × 12.2 + 14326 × 6 28652 × 2 = 1700199.6
O_2	127618 + 14326 + 28652 28652 =199248	127618 × 16.2 + 14326 × 10 28652 × 2 = 2439887.6
O_1	127618 + 14326 + 28652 28652 + 28652 =227900	127618 × 20.2 + 14326 × 14 28652 × 10 + 28652 × 6 + 2865 2 = 3294183.6

The axial thrust $V_{max} = \frac{4 \times Mw}{n \times D_o} = \frac{4 \times Mw}{8 \times 10} = 0.05\ Mw$

In the farthest leeward column, the shear force

$S_{max} = \frac{2 \times Qw}{n} = 0.25\ Qw$ in the column on the bending moment M $= S_{max} \times \frac{h}{2}$ in the columns are tabulated below:

Table -7: Max. shear force & bending moment

Level	V_{max}	S_{max}(N)	M (N.m)
O_4	53755.98	35486	70972
O_3	85009.98	42649	85298
O_2	121994.38	49812	99624
O_1	164709.18	65975	113950

The farthest leeward column will be subjected to superimposed axial load plus V_{max} 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 combination for various panels of these columns are tabulated below.

Table -8: Axial load & bending moment

Panel	Earthest leeward column		Column on bending axis	
	Axial load (N)	V_{max}	Axial load (N)	M (N.m)
$O_4\ O_4^1$	2197416	53755.98	2197416	70972
$O_3\ O_3^1$	2235896	85009.98	2235896	85298
$O_2\ O_2^2$	2274376	121994.38	2274376	99624
$O_1\ O_1^1$	2322476	164709.18	2322476	113950

According to IS, When effect of wind load is to be considered. The permissible stresses in the materials may be increased by $33\frac{1}{3}$ % for the farthest leeward column the axial thrust V_{max} due to wind load is less than even 10 % of the super imposed axial load hence the effect of maximum B.M of 113950 N.m due to wind along with the super imposed axial load of 2322476 N at the lowest panel. Use M30 concrete for which &σcbc = 10 N/mm² and σcc = 8 N/mm². For steel σst = 230 N/mm². 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.

Asc = $\frac{\pi}{4} \times 30^2 \times 12 = 8482\ mm^2$

Equivalent area of column = $\frac{\pi}{4} \times 700^2 + (9.33-1) \times 8482 = 455500\ mm^2$

Equivalent moment of inertia = $\frac{\pi}{64} \times d^4 + (m-1)\ \frac{Asc \times d'^2}{8}$

Where, d=100 mm ; d'= 700-2×40 = 620 mm

$I_c = \frac{\pi}{64}(700)^4 + (9.33-1) \times \frac{8482 \times 620^2}{8} = 1.518085 \times 10^{10}\ mm^4$

Direct stress in column = $\sigma cc' = \frac{2322476}{455500} = 5.09\ N/mm^2$

Bending stress in column = $\sigma cbc' = \frac{113950 \times 1000}{1.5180 \times 10^{10}} \times 350 = 2.62$ N/mm²

For the safety of the column, we have the condition

$$\frac{\sigma cc'}{\sigma cc} + \frac{\sigma cbc'}{\sigma cbc} \geq 1$$

$$\frac{5.09}{1.33 \times 8} + \frac{2.62}{1.33 \times 10} < 1$$

0.675 < 1 .... Hence safe

Use 10 mm Ø wire rings of 250 mm $^c/_c$ to tie uo the main reinforcement. Since the columns are of 700 mm diameter, increase the width of curved beam B_2 from 600 mm to 700 mm.

- **Check for seismic effect**

For empty tank = 6054829 N

For tank full = 17271486 N

For column I

According to revised classification of earthquake zone, Mumbai comes under zone III

Therefore zone III IS 1893 – 2002 Stiffness of column in a bay

Kcc = $\frac{12\ EI}{L^3}$

E = 5000 × $\sqrt{fck}$ = 5000 × $\sqrt{30}$ = 27386.128 N/mm²

Ic = 1.518085 × 10^{10} mm⁴ (from column design)

L = 4 (i.e the distance between two braces and a panel)

Kc = $\frac{12\times27386.128\times1.518085\times10^{10}}{4000^3}$ = 77952.131 N/mm

Stiffness of 8 column

$\sum$Kc = 8 × 77952.131

$\sum$Kc = 623617.0519

Neglecting effect of bracing on stiffness $\frac{1}{k} = \sum \times \frac{1}{K}$

When K = 1, Fundamental = $2\pi\sqrt{\frac{W}{g\times K}} = 2\pi \times \sqrt{\frac{17271486}{9810}} \times 1$= 4.39 sec

By interpolation, $\frac{Sa}{g}$ = 0.2 From Fig-2, IS 1893-1980 Page No, 18

From IS 1893

An = $\frac{Z\times I\times S\times a}{2\times R\times g}$ from zone III

Z = 0.16 (Zone III)

I = 1.0 (Important factor) Table No: 6

R = 2.5 (Responser education factor) Table No.7 Is Code

An = $\frac{0.16\times1.0}{2\times2.50} \times 0.2 = 6.4\times10^{-3}$

Force due to earthquake Feh

Feh_1 = Mass × Acceleration = 17271486 × 6.4×10^{-3} = 110539.4304 N

$\sum$m = Due to wind = 227900 N > Feh

Therefore no need to consider earthquake in a design of column.

5.12 Design of braces

The bending moment m_1 in a brace is given by its maximum value being governed by

$\tan\left(\theta + \frac{\pi}{8}\right) = \frac{1}{2}Cos\theta\theta = 24.8°$

$(M_1)_{max} = \frac{Qw_1\times h_1 + Qw_2\times h_2}{n\times Sin\frac{2\pi}{n}} \times Cos^2\theta \times Sin\left[\theta + \frac{\pi}{h}\right]$

For the lowest junction C

h_1 = 5 m & h_2 = 4 m

$(M_1)_{max}= \frac{(227900\times5)+(199248\times4)}{8\times Sin\times\frac{2\pi}{8}} Cos^2(24.8°) \times Sin\left[24.8 + \frac{\pi}{8}\right]$ = 207318 N.m

The maximum shear force $(Sb)_{max}$ in a brace, For $\theta = \frac{\pi}{8}$

$(Sb)_{max}= \frac{(227900\times5)+(199248\times4)}{3.93\times8\ \sin\frac{2\pi}{8}}\left[2Cos^2\frac{\pi}{8}\times Sin\frac{2\pi}{8}\right]$ = 105146 N

For $\theta=\frac{\pi}{8}$, the value of M_1

$[(M)]_{\theta=\frac{\pi}{8}} = \frac{(227900\times5)+(199248\times4)}{8\ Sin\left[\frac{2\times\pi}{8}\right]} \times \left[Cos\left(\frac{\pi}{8}\right)^2\right] \times Sin\left[\frac{\pi}{8}+\frac{\pi}{8}\right]$= 206612 N.m

Twisting moment at $\theta = \frac{\pi}{8}$ is M^t = 0.05 m_1 = 0.05 × 206612 = 10331 N.m

Thus the brace will be subjected to a critical combination of max. Shear force $(Sb)_{max}$ and a twisting moment (M^t) when the wind blows parallel to it (i.e. when $\theta = \frac{\pi}{8}$).

The brace is reinforced equally at top and bottom since the sign of moment (M_1) will depend upon the direction of wind.

For M30 Concrete,

C = σcbc = 10 N/mm²

σst = t = 230 N/mm²

M = 9.33

K = 0.28865

J = 0.9038

R = $\frac{1}{2}$ × 0.9038 × 0.28865 × 10 = 1.30441

Depth of NA = 0.28865

Equating the moment of equivalent area about N.A

$\frac{1}{2}$ × b × (0.288d)² + (9.33-1) × pbd (0.288d-0.1d)

P = 8.168 ×10^{-3}

% p = 0.8168 % = 0.008168

Since the brace is subjected to both the B.M as well as twisting moment, we have

$Me_1 = M + Mr$

Where M = B.M = $(M_1)_{max}$ = 207318

$MT = T\left[\frac{1+\frac{D}{b}}{1.7}\right]$, where T = M^t = 10331 N.m

Let D = 700 mm

$MT = 10331 \times \left[\frac{1+\frac{700}{300}}{1.7}\right] = 20257$ N.m

Me_1 = 207318 + 20257 = 227575

In order to find the depth of the section, equate the moment of resistance of the section to the external moments.

$b \times n \times \frac{C}{2} \times \left[d - \frac{n}{3}\right] + (m_c - 1)\, Asc \times C'\, (d - d_c) = Me_1$

Here

C = 1.33 × 10 = 13.33

$M_c = 1.5 \times m = 1.5 \times 9.33 = 13.99 \approx 14$

d = compression at steel level = $13.33 \times \frac{(0.288-0.1)d}{0.288d} = 8.7035$ N/mm² Hence

$300 \times 0.288 \times \frac{13.33}{2} \times \left[1 - \frac{0.288}{3}\right] \times d + (14-1) \times (0.008168 \times 300d) \times 8.7035\, (1-0.1)d$

$= 227575 \times 10^3$

$520.57 \times d^2 + 249.52 \times d^2 = 227575 \times 10^3$

d = 543.61

Adopt D = 700 mm so that d = 700 – 25 – 10 = 665 mm

Asc = Ast = pbd = 0.008168 × 300 × 700 = 1715.28 mm²

No. of 25 mm Ø bars = $\frac{1715}{\frac{\pi}{4}\times 25^2} = \frac{1715}{491} = 3.49 = 4$ nos

Provide 4 Nos of 25 mm Ø bars each at top and bottom

$\frac{100\, As}{b \times d} = \frac{100\times4\times491}{300\times700} = 0.94\ \%$

Maximum shear = 105146 N

$Ve = V + \frac{1.6\, T}{D} = 105146 + 1.6 \times \frac{10331}{0.3}$

Ve = 160245 N

$\tau_{ve} = \frac{160245}{300\times700} = 0.763\ N/mm^2$

This is less than τc = 0.37 N/mm².

Hence transverse reinforcement is necessary.

$Asv = \frac{T.sv}{b_1 \times d_1 \times \sigma sv} + \frac{V \times sv}{2.5 \times d_1 \times \sigma sv}$

Where , b_1 = 300 – (25×2) – 25 = 225 mm

d1 = 700 – (25×2) – 25 = 625 mm

Using 12 mm Ø 2 legd stirrups

$Asv = 2 \times \frac{\pi}{4} \times 12^2 = 226$ mm²

$\frac{Asv}{sv} = \left[\frac{10331\times1000}{225\times625\times230} + \frac{105146}{2.5\times625\times230}\right] = 0.31941 + 0.2925 = 0.6119$

Minimum transverse reinforcement is given by

$\frac{Asv}{sv} \geq \left(\frac{\tau_{ve}-\tau_c}{\sigma sv}\right) b$

$\frac{Asv}{sv} = \frac{0.763-0.37}{230} \times 300 = 0.5126$

$Sv = \frac{Asv}{0.6119} = \frac{226}{0.6119} = 369.34$ mm

Sv = 360 mm

This spacing should not exceed least of X_1, $\frac{X_1+Y_1}{4}$ and 300 mm

Where X_1 = short dimension of stirrup = 225 + 25 + 12 = 262 mm

Y_1 = Long dimension of stirrup = 625 + 25 + 12 = 662 mm

$\frac{X_1+Y_1}{4} = \frac{262+662}{4} = 231\ mm$

Hence provide 12 mm Ø 2 leg stirrups at 230 mm $^c/_c$ throughout. Since the depth of section exceeds 450 mm provide side face reinforcement @ 0.1 %

$A_1 = \frac{0.1}{100} \times (300 \times 700) = 210\ mm^2$

Provide 2 – 10 mm Ø bars at each face, giving total = 4 × 78.5 = 314 mm²

Provide 300 × 300 mm haunches at the junction of braces with columns and reinforce it with 10 mm Ø bars.

5.13 Design of raft foundation

Vertical load from filled tank and columns = 2322476 × 8 = 18579808 N

Weight of water = 11216657 N

Vertical load of empty tank and columns = 7363151 N

V_{max} due of wind load = 164709.18 × 8 which is less than $33\frac{1^{\circ}}{2}$ % of the super imposed load.

164709.18 × 8 = 1367600

$\left[\frac{33.33}{100} \times 11216657\right] = 3738511.778\ N$

Assume self-weight etc. = 10 % = 1857980.8 N

Total load = 18579808 + 1857980.8 = 20437788.8 N

Area of foundation required = $\frac{20437788.8}{196133}$ = 104.20 mm²

Circumference of column circle = $\pi \times 10 = 31.42m$

(i.e. 10.6 – 2 × 0.3 = 10 m)

Width of foundation = $\frac{104.20}{31.42}$= 3.316 ≈ 3.32

Hence inner diameter = 10 – 3.32 = 6.68 m

Outer diameter = 10 + 3.32 = 13.32 m

Area of annular raft = $\frac{\pi}{4} \times (13.32^2 - 6.68^2)$ = 104.30 m²

Moment of inertia of slab about diametrical axis = $\frac{\pi}{64}\times(13.32^4 - 6.68^4)$ = 1447.5 M²

Total load, tank empty = 7363151 + 1857980.8 = 9221131.8 N

Stabilizing moment = 9221131.8 × $\frac{13.32}{2}$ = 61412737.79 N.m

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

Mw at base = (127618 × 24.2) + (14326 × 18) + 28652 (14+10+6) = 4205783.6 N.m

Hence the soil pressures at the edges along a diameter are

a. Tank full = $\frac{20437788.8}{104.30} \pm \frac{4205783.6}{1447.5} \times \frac{13.32}{2}$ = 215302.92 N/m² or 176600.99 N

b. Tank empty = $\frac{9221131.8}{104.30} \pm \frac{4205783.6}{1447.5} \times \frac{13.32}{2}$ = 107760.66N/m²or 69058.737 N/m²

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

196.133 × 1.33 = 260.856 kN/m²

Which is greater than the minimum soil pressure of 215.302 KN/m². Hence the foundation raft will be designed only for super-imposed Load.

The layout of the foundation is shown

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

P = $\frac{18579808}{104.30}$ = 178138.14 N/m²

The overhang X of raft slab = $\frac{1}{2}\left[\frac{1}{2}(13.32 - 6.68) - 0.7\right]$ = 1.31 m

B.M = 178138.14 × $\frac{1.31^2}{2}$ = 152851.431 N.m

S.F = 152851.431 × 1.31 = 200235.3746 N

d = $\sqrt{\frac{152851.431\times1000}{1000\times1.30441}}$ = 342.316 mm

Provide 400 mm thick slab with effective depth of 350 mm. Decrease the total depth of 250 mm at the edges .

Ast = $\frac{152851.431\times1000}{230\times0.9038\times350}$ = 2100.88 mm²

Spacing of 16 mm Ø radial bars @ 95 mm c/c at the bottom of slab

Area of distribution steel = $\frac{0.15}{100} \times 1000 \times 400$ = 600 mm²

Spacing of 10 mm bars = 1000 × $\frac{78.5}{600}$ = 130.83 mm

Hence Provide 10 mm Ø bars @ 130 mm c/c at the supports. Increase this spacing to 200 mm at the edge.

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

Design load = $\frac{18579808}{\pi\times10}$ = 591413.657

The circular beam B_2 was designed for w = 591413.657

The circular beam B_2 was designed for w = 549768

Hence the B.M etc will be increased in this ratio of $\frac{591413.657}{549768}$ = 1.07575

Max (-) B.M at support = M_o = 712448 × 1.07575 = 766415.936 N.m

Max (+) B.M at mid span = M_c = 323840 × 1.07575 = 348370.88 N.m

Max torsional moment $M_m{}^t$ = 53973 × 1.07575 = 58061.45475 N.m

B.M at the point of Max. Torsion = 1767 × 1.07575 = 1900.850 N.m

At $\theta = \theta_m = 9\frac{1^\circ}{2}$, F = 633692 × 1.07575 = 681694.169 N

Max. Shear force at supports = 1079467 × 1.07575 = 1161236.625 N

Use b = 700 mm = diameter of columns

Use M20 concrete

σst = 230 N/mm²

d = $\sqrt{\frac{766415.936\times1000}{700\times1.30441}}$ = 916.1702 mm

However keep total depth of 1200 mm from shear point of view, using an effective cover of 60 mm

d = 1140 mm

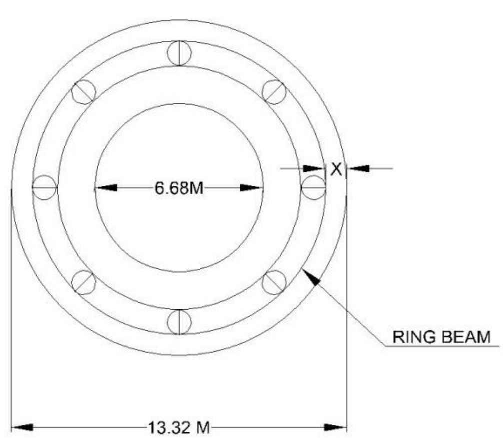

Fig -3: Raft foundation detail

- Main or longitudinal reinforcement

a. section at point of maximum torsion

T = m^t_{max} = 58061.45475 N.m

$M\theta$ = M = 1900.850 N.m

$Me_1 = M + M_T$

Where $M_T = T\left[1+\frac{\frac{D}{b}}{1.7}\right] = 58062\left[1+\frac{\frac{1200}{700}}{1.7}\right]$ = 116611.916 N/m

Me_1 = 1901 + 116612 = 118513 N/m

Ast = $\frac{Me_1}{\sigma st\times j\times d} = \frac{118513\times1000}{230\times0.9038\times1140}$ = 500 mm²

No. of 25 mm Ø bars = $\frac{500}{491}$ = 1.01

Since $M_T > M$,

$Me_2 = M_T - M$ = 116612 – 1901 = 114711

$Ast_2 = \frac{114711\times1000}{230\times0.9038\times1140}$ = 484 mm²

Therefore, No of 25 mm Ø bars = $\frac{484}{491} = 0.98 \cong 1$

However provide minimum of 2 bars each at top and bottom

b. Section at max. Hogging B.M (support)

M_o = 766416 N.m = M_{max}, $M_o{}^t = 0$

Ast = $\frac{766416\times1000}{230\times0.9038\times1140}$ = 3234 mm²

No. of 25 mm Ø bars = $\frac{3234}{491}$ = 6.58 ≈7

However provide 7 bars of 25 mm Ø at the bottom of the section, near supports

c. Section at max. Sagging B.M (Mid span)

Mc = 348371 N.m, $Me_t = 0$

Ast = $\frac{348371\times1000}{230\times0.9038\times1140}$ = 1470 mm²

No. of 25 mm Ø bars = $\frac{1470}{491}$ = 2.99

Hence the scheme of reinforcement along the span will be as follows;

At supports provide 6 – 25 mm Ø bars at bottom of section. Continue these upto the section of maximum torsion (i.e. at $\emptyset_m$ = 9.5° = 0.116 rad) at a distance = R θ_m = 5 × 0.166 = 0.83 or equal to $L_d = \frac{\emptyset\times\sigma st}{4\times\tau_{bd}} = \frac{\emptyset\times230}{4\times1.12}$ = 52Ø = 52 × 25 = 130 mm whichever is more

Beyond this discontinue 2 bars, while the remaining 4 bars may be continued throughout the length.

Similarly provide 4 – 25 mm Ø bars at top, throughout the length. These bars will take care of both the maximum positive B>M as well as Maximum torsional moment.

- Transverse reinforcement

a. At the point of maximum torsional moment

V = 681695 N.m

$V_e = V + 1.6\frac{T}{b} = 681695 + 1.6 \times \frac{58062}{0.7} = 814408$ N

$\tau ve = \frac{814408}{700 \times 1140} = 1.02$ N/mm²

This is less than τ_c = 0.22 N/mm² hence shear reinforcement is necessary.

$Asv = \frac{T \times Sv}{b_1 \times d_1 \times \sigma sv} + \frac{V \times Sv}{2 \times Sd_1 \times S_v}$

Where b_1 = 700 – (40×2) -25 = 595 mm

d_1 = 1200 – (40 × 2) – 25 = 1095 mm

$\frac{Asv}{Sv} = \left[\frac{58062 \times 1000}{595 \times 1095 \times 230} + \frac{621694}{2.5 \times 1095 \times 230}\right] = 1.47$

Minimum transverse reinforcement is governed by

$$\frac{Asv}{Sv} \geq \left(\frac{\tau ve - \tau c}{\sigma sv}\right) b$$

$\frac{Asv}{Sv} = \frac{1.02-0.22}{230} \times 700 = 2.43$

Hence adopt $\frac{Asv}{Sv} = 2.43$

Using 12 mm Ø 4 leg stirrips

Asv = 4 × 113 = 452 mm²

$Sv = \frac{452}{2.43} = 186$ mm

However, spacing should not exceed of X_1, $\frac{X_1+Y_1}{4}$ and 300 mm where

X_1 = Short dimension of stirrup = 595 + 25 + 12 = 632 mm

Y_1 = Long dimension of stirrup = 1095 + 25 + 12 = 1132 mm

$\frac{X_1+Y_1}{4} = \frac{632+1132}{4} = 441$ mm

Hence Provide 12 mm Ø 4 lgd stirrups @ 186 mm$^c/_c$

b. At the point of max. Shear (supports)

At supports F_o = 1161237 N

$\tau v = \frac{1161237}{700 \times 1140} = 1.5$ N/mm²

At supports ,$\frac{100\,As}{bd} = \frac{100(6 \times 491)}{700 \times 1140} = 0.37$ %

Hence τc = 0.26 N/mm² . Hence shear reinforcement is necessary

Vc = 0.26 × 700 × 1140 = 2.7480 N

Vs = F_o – Vc = 1161237 – 207480 = 953757 N

The spacing of 12 mm Ø 4 – lgd stirrups having

$Asv = 4 \times \frac{\pi}{4} \times 12^2 = 452.4$ mm² is given by

$Sv = \frac{\sigma sv \times Asv \times d}{V} = \frac{230 \times 452.4 \times 1142}{9536757} = 124.37$ mm

Hence provide 12 mm Ø 4 lgd stirrups @ 124 mm $^c/_c$

C. At mid span ; At the mid span

S.F is zero hence Provide, minimum / nominal shear reinforcement given by

$$\frac{Asv}{b \times Sv} \geq \frac{0.4}{fy} \; or \; \frac{Asv}{Sv} = \frac{0.4 \times b}{fy} = \frac{0.4 \times 700}{415} = 0.075$$

Choosing 10 mm Ø 4 leg stirrups, Asv = 314 mm²

$Sv = \frac{314}{0.675} = 465$ mm

Max. Permissible spacing = 0.75 × d = 0.75 ×1140 = 855 or 300 mm , whichever is less.

Hence provide 10 mm Ø 4 lgd stirrups @ 300 mm

Side face reinforcement; Since depth is more than 450 mm, provide side face reinforcement @ 0.1 %

$A_2 = \frac{0.1}{100}(700 \times 1200) = 840$mm²

Provide 3 – 16 mm Ø bars on each face, having total A_2 = 6 × 201 = 1206 mm²

5.14 Design of staircase.

Staging height = 16 meter

Total height = 20.2 meter (Upto gallary)

Assume riser = 250 mm

Nos of steps required = $\frac{20.2}{0.25}$ = 80.8 ≈ 81

Considering weight of each precast step = 0.1 ×T

L.L = 0.05 × T

Total = 0.15 T

Total load = 0.15 × 81 = 12.15 T

Self (D.L) = 25 × $\frac{\pi}{4}$× d^2 × 2.55 (constant) = 0.25 × $\frac{\pi}{4}$ × 0.3^2 × 2.55 = 4.5 T

Total = 16.8 T say 20 T

Providing 300 mm diameter column with 6 – 12 TOR load carrying capacity of concrete alone in m-15

$= \left(\frac{\pi}{4} \times D^2 - 6 \times \frac{\pi}{4} \times 12^2\right) \times \frac{4}{9810}$

$=\left(\frac{\pi}{4} \times 300^2 - 6 \times 113\right) \times \frac{4}{9810}$

= 28.55 T ≫ 20 T

Footing design

S.B.C = 20T/sq.mt

Area of footing required = $\frac{20}{20} = 1$ sq.mt

Provide 1000 × 1000 mm size of footing

Depth = 350 mm to 200 mm

Moment, M = 30× 0.35 × 0.35 ×$\frac{1}{2}$ = 1.84 TM

$ASt_{req} = \frac{m}{\sigma st \times \tau \times d} = \frac{1.84}{2.3 \times 0.903 \times 0.275}$ = 3.22 sq.cm

Provide 6-10 mm TOR blw

6. Analysis in staad pro

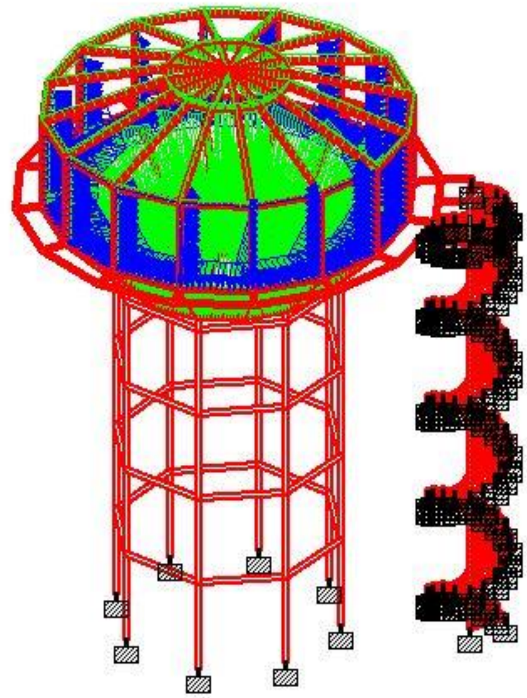

Fig –4: Loading diagram

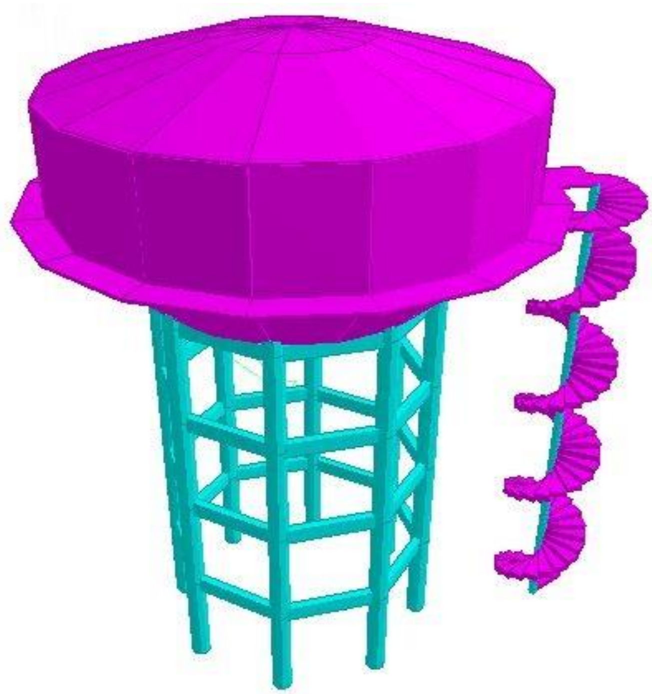

Fig -5 : 3D rendered view

7. Results

1. Total Volume of concrete = 174.2 Cu.meter
2. Total quantity of steel = 87948 Kg
3. Numbers of columns = 8 Nos.
4. Type of foundation = Raft foundation
5. Diameter of tank = 16 m
6. Total pressure per m^2 on the dome = 4000 N/m^2
7. Load on top dome = 16807 N/m
8. Load due to ring B_1 = 3075 N/m
9. Load due to Tank wall = 25000 N/m
10. Load of beam B_3 = 10500 N/m
11. Inclination of conical dome = 45°
12. Weight of water on dome = 4751259 N
13. Weight of gallery = 1.2 m
14. Total weight of tank = 6054829 N
15. Weight on each column = 2158936 N
16. Diameter of column = 700 mm
17. Total height of structure = 26.1 m
18. Height of staircase = 20.2 m (Up to gallery)
19. Numbers of steps in staircase = 81 steps

6. Conclusion

1. Elevated circular water tank with large capacity and flat bottom needs large reinforcement at the ring beam, to overcome this in intze tank, by providing a conical bottom and another spherical bottom reduces the stresses in ring beams. intze tank is more economical for high capacity reducing the steel requirement.

2. Per capita demand has been calculated which helped us, to know about the water consumption in residential area and further helped in design the tank.

3. Limit state method was found to be most economical for design of water tank as the quantity of steel and concrete needed is less as compare to working stress method.

4. After manual design and analysis in staad pro our structure is safe.

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