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Design of Circular Overhead Water Tank

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Abstract— **The water is the most essential element to a life on the earth. It is a liquid which covers about 71.4% of the earth. It is the most ubiquitous substance in the human body. The approximate consumption of water in a population of around 20,000 is 200 litres/head/day. The water is also important in the agricultural and industrial sectors. Water demand is one of the key issues in water supply planning. To overcome this issue, the present water tank designs have to be modified. Overhead water tank is the most effective storing facility used for domestic or even industrial purpose. The design and construction methods in reinforced concrete are influenced by the prevailing construction practices, the physical property of the material and the climatic conditions, linings, the ground conditions i.e. type of soil, soil bearing capacity etc. This paper gives an overall designing procedure of an Overhead Circular Intze tank using LIMIT STATE METHOD from IS-3370:2009. In IS-3370:2009, limit state method considering two aspects mainly limits the stress in steel and limits the cracking.**

Index Terms—**Economical Design, Intze tank, IS-3370:2009, Limit State Method**

I. INTRODUCTION

A water tank is container for storing water and any other liquid. The main objectives in any design of water tank are to provide safe drinkable water after storing for long time, optimizing cost, strength, service life and performance during special situations like earthquakes. The other objectives are to maintain pH of water and to prevent the growth of microorganism. Water is susceptible to a number of ambient negative influences, including bacteria, viruses, algae, changes in pH, and accumulation of minerals, accumulated gas. A design of water tank or container should do no harm to the water.

II. AIMS AND OBJECTIVES

- To study the various forces acting on a water tank. Understanding the most important factors that plays role in designing of a water tank.
- To study the guidelines of design of water tank according to IS code and checking the design.
- To know about the design philosophies of water tank design.
- Preparing a water tank design which is economical and safe, providing proper steel reinforcement in concrete and studying its safety according to various code.

III. INTZE TANK

A water tower built in accordance with the Intze Principle has a brick shaft on which the water tank sits. The base of the tank is fixed with a ring anchor made of iron or steel, so that only vertical, not horizontal, forces are transmitted to the tower. Due to the lack of horizontal forces the tower shaft does not need to be quite as solidly built.
The main advantages of such tank are that the outward thrust from top of conical part is resisted by ring beam B3.

IV. METHOD OF ANALYSIS

A. ANALYSIS OF SHELL STRUCTURES

- The first step is to make imaginary cut at the junction and assume the imaginary supports condition consistence with the membrane analogy. This assumption permits the determination of membrane forces and deformation due to different loading condition.
- The second step is to apply restraining forces at the edges consistent with the actual support condition to make the deformation compatible at the junction.

B. ANALYSIS OF ROOF WALL JOINT

- The roof may be designed as a spherical or conical dome.

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C. ANALYSIS OF THE SPHERICAL BOTTOM CONICAL

WALL JOINT

- The joint may either be supported on columns or on a circular shaft.
- If the tank is supported on columns, the two shells are connected through a ring beam to the columns and, if the tank is supported on a circular shaft, the threw shells can be jointed together without a ring beam.

D. MEMBRANE ANALYSIS

- In the membrane analysis the member are assumed to act independent of the others. Hence individually all components of the structure are designed.
- The member are therefore subjected to only direct stresses and as the joints are not considered rigid i.e. as all members are acting individual bending moment is not introduced.

VARIOUS STRUCTURAL ELEMENTS OF INTZE TANK ARE:

- Top Spherical dome
- Top ring beam B1
- Side wall (circular)
- Bottom ring beam B2
- Bottom Spherical dome
- Bottom ring beam B3

V. DESIGN OF INTZE TANK

A. POPULATION FORCAST

POPULATION FORECAST FOR A VILLAGE

Table II - POPULATION FORECAST

Year	Population	X increase	Y increase	% increase	% Decrease
1970	1200	-	-	-	-
1980	2000	800	-	66.67	-
1990	2800	800	0	40.00	26.67
2000	4000	1200	400	42.87	-2.87
Sum	-	2800	400	149.54	23.8
Avg.	-	933.33	200	49.84	11.9

Arithmetic Progression Method

Population (P) = P_o + nx= 4000 + 1 × 933.33= 4934

2. Geometric Progression Method

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

$r = \sqrt[3]{0.6667 \times 0.4 \times 0.4287} = 0.48533 = 48.53$ %

$P_{2010} = 4000 \times (1 + \frac{48.53}{100})^1 = 5942$

3. Incremental Increase Method

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

When n = 1;

$P_{2010} = 4600$

Assuming changing increase rate method

$P_{2010} = (42.87-11.9) \times \frac{4000}{1000} = 6118.8$

Considering geometric increase method

P = 5942= 6000

Therefore, design population of 6000

Assuming per capita demand 165 lpcd

Capacity required = 165 × 6000 lpcd

= 990000 lpcd

In one day = 990000 lpcd

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

B. DESIGN WITH MEMBRANE ANALYSIS

1. MEMBRANE ANALYSIS

In the membrane analysis the member are assumed to act independent of the others.

Hence individually all components of the structure are designed.

The design of membrane analysis is carried as follows,

Consider,

M30 concrete

HYSD Fe 415 bars

Intensity of wind pressure = 1200N/m^2

Thickness = 100mm

Bearing capacity = 180 KN/m^2

Let diameter of ring beam = $B_2 = D_o$ = 10 m

Let the diameter of cylindrical portion D = 15 m

R = 7.5 m

h = height of cylindrical

Rise h_1 = 1.8 m

Rise h_2 = 1.6 m

Radius R_2 of bottom dome is given;

$h \times (2R_2-h_2) = 5^2$

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

R_2 = 8.61 m

In general the volume of water stored is given by;
$V = \frac{\pi}{4}D^2h + \frac{\pi}{12}h_o(D^2 + D_0^2 + Dh_o) - \frac{\pi}{3}h_2^2(3R_2 - h_2)$

$$\sin\varphi_2 = 0.5807$$
$$\varphi = 35.5$$

Required volume = 1000 m^3
∴ h = 4.619 m
Allowing for free board; h = 5 m
For top dome, the radius R_1;
By property of circle,
$h_1 \times (2R - h_1) = 7.5^2$
$R_1 = 16.525$

$$\sin\varphi_1 = 0.453$$
$$\varphi = 26.9914$$

2. DESIGN OF TOP DOME

$R_1 = 16.525$ m $\sin\varphi_1 = 0.453$ $\cos\varphi_1 = 0.8910$
Let thickness $t_1 = 100$ mm = 0.1 m
Taking Live load = 1.5 KN/m^2
Pressure on top of dome p = 0.1 × 25000 + 1500
= 4000N/m^2
Meridional thrust at edge
$T_1 = \frac{pR_1}{1+\cos\varphi_1}$
$T_1 = 34599.857 \,{}^{N}/_{mm^2}$

Maximum hoop stress occurs at the center
$\frac{pR_1}{t_1} = \frac{4000\times16.525}{0.1} \times \frac{1}{2} = 330500 \,{}^{N}/_{m^2}$

$= 0.33 \,{}^{N}/_{mm^2}$ (safe)

Since stresses are within safe limit, provide nominal reinforcement @ 0.35%
$A_s = \frac{0.35}{100} \times 100 \times 1000 = 350$ mm^2
(A_{st} = 100mm) using 8mm Ø bars $A_{\emptyset} = 50mm^2$
Spacing = $\frac{1000\times50}{350}$ = 142.85 mm≅ 140 mm

Hence, Provide # 8mm Ø @ 140 mm ${}^{c}/_{c}$

3. DESIGN OF TOP RING BEAM B1

Horizontal component of T is
$P_1 = T_1\cos\emptyset_1 = 300831.02763 \,{}^{N}/_{m}$

Total Tension tending to rupture the beam
$= \frac{P_1\times D}{2} = 231232.71 \,{}^{N}/_{m}$
Permissible Stress in HYSD = 130 ${}^{N}/_{mm^2}$
Ash = $\frac{231232.7}{130}$ = 1778.71 mm^2

No. of 20 mm Ø bars = $\frac{1541.5}{380.13}$ = 6
Actual A_{sh} provided = 341.16 × 6 = 1884.95 mm^2
$\frac{\text{Tension causing rupture}}{[A + (9.333-1)\times Ash]} \leq 1.3 \,{}^{N}/_{mm^2}$
$$\frac{231232.7072}{[A + 8.333 \times 1884.95]} = 1.3$$
A = 162163.45 mm^2
Provide ring beam of 400 mm depth and 420 mm width. Tie the 20 mm Ø ring by 60 mm diameter nominal stirrups @ 200 mm ${}^{c}/_{c}$
A_{prov}= 400 × 420 = 168000 mm^2 ∴ OK

4. DESIGN OF CYLINDRICAL WALL

In the membrane analysis, the tank wall is assumed to be free at top and bottom.
Maximum hoop tension occurs at bottom of the wall
$\rho = p\frac{D}{2} = \frac{whD}{2}$ = 367500 N/m height
Provide rings on both the faces
On each face = 1413.461mm^2
Spacing of 14mmØ rings = $\frac{1000\times14^2\times\frac{\pi}{4}}{1413}$ = 108.9mm
Provide 14mmØ rings at 100mm spacing ${}^{c}/_{c}$ at bottom
This spacing can increase at the top
Actual Ash provide = $\frac{1000\times14^2\times\frac{\pi}{4}}{100}$ = 1539.3mm^2
On each face permitting 1.2N/mm^2 stress on composite section
$\frac{367500}{1000t+[9.33-1]\times1539.3\times2}$ = 1.2
t = 254.94mm = 25.4cm
Minimum thickness = (3H + 5) cm (H = 5m)
3×5+5=20cm
However provide t = 300mm at bottom and taper it to 200mm at top
Average t = $\frac{300+200}{2}$ = 250mm
Percent distribution steel
= 0.24% of surface zone of wall
As tank dimensions are not more than 15m
∴ 0.24% of surface zone
Therefore, A_{sh} = 325 mm^2
Let area of steel on each face be 325mm^2
A_{sh} = 650mm^2
Spacing of 8mm Ø bars = $\frac{1000\times50.3}{325}$ = 155mm
Provide8mm Ø bars at 150mm ${}^{c}/_{c}$ on faces keep a clear cover of25mm
To resist hoop tension at 2m below top
$A_{sh} = \frac{2}{5} \times 2826.93 = 1130.7692mm^2$
Therefore, spacing of 14mm Ø rings

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$= \frac{1000 \times \frac{\pi}{4} \times 14^2}{\frac{1130.7692}{2}}$

= 136.1357mm×2= 272.27mm

Hence provide the rings at 270mm $^{c}/_{c}$ in top 2m height (1-2m)

At 3m below top

A_{sh}= 1696.1538mm^2

Spacing of 12mmØ ring $= \frac{1000 \times \frac{\pi}{4} \times 14^2}{\frac{1696.1538}{2}}$ = 181.81mm

= 180mm

Hence provide the rings at 180mm$^{c}/_{c}$ in next 1m height (2-3m)

At 4m below the top

$A_{sh} = \frac{4}{5} \times 2826.923 = 2261.93$mm

$S_v = \frac{1000 \times \frac{\pi}{4} \times 14^2}{\frac{2261.93}{2}}$ = 136.1357mm

Provide 14mm Ø ring at 130mm at next 1m height (3-4m)

In last 1m height provide rings of 100mm $^{c}/_{c}$as found earlier (4-5m)

5. DESIGN OF RING BEAM B3

Ring beam B3 connects the tank wall conical dome. The vertical load at the junction of the wall with conical dome is transferred to ring beam B3 by meriodional thrust in conical dome. The horizontal component of thrust cause hoop tension at the junction. Ring beam takes up this hoop tension.

In our design w consist of per running meter

(i) Load of top dome = $T_1 \sin \emptyset_1$ = 15703.41152 N

(ii) Load due to ring beam B_1

= 0.4×(0.42 − 0.2) × 1 ×25000

Depth = 0.4m

Breadth = 2400N/m

(iii) Load due to tank wall = $5 \times \left(\frac{0.2+0.5}{2}\right) \times 1 \times 25000 = 31250\ ^{N}/_{m}$

(iv) Self weight of beam B_3(1m × 0.6m)

((1-0.3) ×0.6) × 25000 = 10500 $^{N}/_{m}$

Therefore,

Total W = 59853.41 $^{N}/_{m}$

Inclination of conical dome wall with vertical = $\emptyset_0 = 45^0$

$\sin \emptyset_0 = \cos \emptyset_0 = \frac{1}{\sqrt{2}}$; $\tan \emptyset$ = 1s

$P_T = \tan \emptyset$ =59853.41152× tan 45=59853.41$^{N}/_{m}$

P_w= (water pressure) × area = w.h × (d_3 × 1)

= 29400N/m

$$P_3 = (P_T + P_w)\frac{D}{2}$$

= 669400.58 N

Hoop stress developed (tensile) resisted entirely in steel hoops. The area of which is

A_{sh} = 5149.235 mm^2

No. of 30 mm Ø bars = $\frac{5073.08}{\frac{\pi}{4} \times 30^2} = 7.28 = 8$ bars

Hence provide 8 rings of 30 mm Ø bars.

Actual A_{sh} = 5654.87 mm^2

Stress in equivalent section = $\frac{\text{force}}{\text{area of equivalent section of concrete}}$

Area of equivalent section of concrete = $A_c + m.A_{sh} - A_{sh}$ =647122.032 mm^2

Stress in equivalent section $= \frac{669400.58}{1000 \times 600 + 8.333 \times 5654.87}$ = 1.0344 $^{N}/_{mm^2}$< 1.3Hence safe.

The 8 mm Ø distribution bars (verticals bars) provided in the wall at 150 mm c/c should be taken around the above rings at act as stirrups.

6. DESIGN OF BOTTOM DOME

Bottom dome develops compressive stresses both meridionally as well as hoops due to weight of water supported by it and also due to its own weight.

R2 = 8.61 m $\sin\emptyset_2$ = 0.5807cosØ = 0.8141

Let, H0 be the total depth of water above the edges of dome

The weight of water above the surface of dome

$W0 = [\frac{\pi}{4} D_0^2 H_0 - \frac{\pi h_2^2}{3}(3R_2 - h_2)] \times 20$

Total surface area of dome = $2\pi R_2 h_2$

Self-weight of dome = $2\pi R_2 h_2 t_2 \times r_c$

t_2 → Thickness of bottom dome

Total load WT = W0 + $2\pi R_2 h_2 t_2 \times r_c$

($\pi D \times T_2$) $\sin\emptyset_2$ = Total load (WT)

T2 → Thrust per meter

T2 × πD → Total thrust force

$(\sin\emptyset)T_2 = \frac{W_T}{\pi D}$

$T2 = \frac{W_T}{\pi D \sin\emptyset_2}$

Intensity of load (p2) = $\frac{W_T}{\text{Surface area of dome}}$

$P2 = \frac{W_T}{2\pi R_2 h_2}$

We know,

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In spherical portions, max hoop stress (derived earlier) $\frac{\text{pressure} \times \text{radius}}{2 \times \text{thicness}} = \frac{P \times R}{2t}$

Hence, pressure p = $\frac{p_2 \times R_2}{2t_2}$

Weight of water W0 on the dome is

W0 = $\left(\frac{\pi}{4} \times 10^2 \times 7 - \frac{\pi}{3} \times 1.6^2 (3 \times 8.61 - 1.6)\right) \times 9800$= 4751259 N

Let the Thickness of bottom dome be 250 mm

Self-weight = $2\pi R_2 h_2 t_2 \times 2500$

= 54098.225 N

Total weight = 529224100 N

(T2) Meridional Thrust = $\frac{W_T}{\pi D_0 \sin\emptyset_0}$

= 290093 $N/_m$

Intensity of load per unit area = $\frac{5292241}{2\pi \times 8.61 \times 1.6}$

= 61142 $N/_m$

Meriodional Stress = $\frac{290093}{250 \times 100}$= 1.16 N/mm2 (safe)

Intensity of Pressure (load per unit area) = $\frac{W_T}{\text{surface area}} = \frac{W_T}{2\pi R_2 h_2}$

=61142N/m2

Max Hoop Stress → $\frac{p_2 \times R_2}{2t_2}$ = 1052860 $N/_{m^2}$

= 1.0528 $N/_{mm^2} < 2.0\ (forM30)$ (safe)

Area of Steel → Bottom dome provided 0.24% (min for HYSD) of steel in both the faces (As per IS 3370-PART-2)

As in each face (thickness = $\frac{0.25}{2}$ = 0.125m)

As = $\frac{0.24}{100} \times \frac{250}{2} \times 100$

= 300 mm2 in each face

Therefore, Total As = 2 × 300 = 600 mm2

(600 mm2 in each direction and 300 mm2 in each face)

Spacing of # 10 mm Ø bars = $\frac{1000 \times \frac{\pi}{4} \times 10^2}{600}$

= 130.89 mm

≅ 130 mm

Provide # 10mm @ 130 mm $c/_c$ in both directions. Also provide 16 mm ∅ meriodinal bar @ 100 mm c/c near water face for 1 m length to take of continuity effect. The thickness of dome maybe increased from 250 mm o 280 mm gradually in 1 m length.

7. DESIGN OF BOTTOM CIRCULAR BEAM (B2)

The ring beam B_2 receives an inward inclined thrust T_0 from conical dome and an outward thrust T_2 from bottom dome. The horizontal components are

$T_0 \sin\emptyset_0$ and $T_2 \cos\emptyset_2$

They are acting in opposite direction, Therefore, net horizontal force on B_2

P = $T_0 \sin\emptyset_0$ - $T_2 \cos\emptyset_2$

$T_0 \sin\emptyset_0 < T_2 \cos\emptyset_2$

The dimensions of tank should be so adjusted that either P is zero or P is compression. The hoop force is given by

$P_H = P \times \frac{P_o}{2}$

If b_2 is width and d_2 is depth of ring beam, the stress is given by;

$P_H = P \times \frac{P_o}{2} \times \frac{1}{bd}$

The vertical load per unit length is given by;

$P_v = T_0 \cos\emptyset_0 + T_2 \sin\emptyset_2$

Inward thrust from conical dome = $T_0 \sin\emptyset_0$

= 401774.5443$N/_m$

Outward thrust from bottom dome = $T_2 \cos\emptyset_2$

= 236165 $N/_m$

Net inward thrust = 401774.5443 – 236165 = 165609.5443 $N/_m$

Hoop compression in beam = 165609.5443 × $\frac{10}{2}$

= 828047.72

Assuming the size of the beam to be 600 mm ×1200 mm

Hoop stress =$\frac{828047.72}{600 \times 1200}$ = 1.15 $N/_{mm^2}$ (safe)

Vertical load on beam, per meter run = $T_0 \cos\emptyset_0 + T_2 \sin\emptyset_2$ = 489970.8576 N

Alternately vertical load = $w_2 + \frac{W_T}{\pi D_o}$= 495188.43 N

Self wt = 0.6 ×1.2×1×25000=18000 $N/_m$

Therefore, the load on beam = w = 507970 $N/_m$

Let us provide the beam on 8 equally spaced column at a mean diameter of 10 m.

Mean radius of curved beams R = 5 m

$2\theta = 45 = \frac{\pi}{4}$

$$\theta = 22.5 = \frac{\pi}{8}$$

From table;

C_1 = 0.066

C_2 = 0.030

C_3 = 0.005

$$\emptyset_m = 9.5^\circ$$

$$w\,R^2(2\theta) = 5077970 \times 5^2 \times \frac{\pi}{4}$$

= 9973967.627 Nm

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Maximum –ve B.M at support = $M_o = C_1 w R^2(2\theta)$ = 658281.8654 Nm

Maximum +ve B.M at support = $M_2 = C_2 w R^2(2\theta)$ = 299219.03 Nm

Maximum torsional moment $M'_m = C_3 w R^2(2\theta)$ = 49869.8 Nm

For M30 concrete $\sigma_{cbc} = 10\ N/mm^2$

For HYSD bars $\sigma_{st} = 130\ N/mm^2$

k = 0.41791 j = 0.86069 R = 1.79845

Required effective depth = $\sqrt{\frac{65824.86 \times 1000}{600}}$=781.05 mm

Howeverkeep total depth = 1200 mm from shear point of new

Let d = 1140 mm

Maximum shear force at support $F_o = w R \theta$ =997396.7627 N

S.F at any point is given by $F = w R (\theta - \emptyset)$

At $\emptyset = \emptyset_m$;F = 576273.685 N

BM at point of maximum torsional moment ($\emptyset = \emptyset_m = 9.5^\circ$) is given by;

$M_o = w R^2 (\theta \sin\emptyset + \theta \cot\theta \cos\emptyset - 1)$

= 1632.83 N-m (sagging)

= 1632.83 N-m (hogging)

The torsional moment at any point is given by

$M_q^t = w R^2 (\theta \cos\emptyset - \theta \cot\theta \sin\emptyset - (\theta - \emptyset))$

At support $\emptyset = 0$;

$M_o^t = w R^2(\theta - \emptyset)$

At midspan$\theta = \emptyset = 22.5 = \frac{\pi}{8}$

$M_e^t = w R^2 \left(\theta \cos\emptyset - \theta \frac{\sin\emptyset}{\sin\emptyset} \cos\emptyset\right)$

We have following combination of BM and torsional moment;

(a) At the support

M_o = 1632.83 N-m (hogging); $M_o^t = 0$

(b) At mid-span

M_e =299221.03 Nm (sagging); $M_o^t = 0$

(c) At point of maximum torsion ($\emptyset = \emptyset_m = 9.5^\circ$)

M_o = 1632.83 Nm (hogging); M_m^t = 49869.8 N-m

Main and longitudinal reinforcement

(d) Section at point of maximum torsion

T= M_{max}^t = 49869.8 Nm; $M_Q = M$ = 1632.8 N-m

As per IS 456-2000

$$M_{e1} = M + M_T$$
$$M_T = 88005.26 \text{ Nm}$$

M_{e1} = 1632.8 + 88005.26 = 89638.06 Nm

$A_{st1} = \frac{M_{eq}}{\sigma_{st} \times j \times d} = 690.63 mm^2$

No of 30mm $\emptyset$ bars = $\frac{589.474}{\frac{\pi}{4} \times 30^2} \approx 1$

Provide a minimum of 2 bars

Since $M < M_T$

$$M_{e1} = M_T - M$$

= 88005.26 − 1632.8

= 86372.46 Nm

$A_{st2} = \frac{86372.46}{130 \times 0.874 \times 1160} = 665.76\ mm^2$

No of 25mm $\emptyset$ bars =$\frac{665.76}{\frac{\pi}{4} \times 25^2}$ = 1.36 ≈ 2

Provide a minimum of 2 bars

Thus at point of maximum torsion.

Provide 2-15 mm $\emptyset$ bars each at top and bottom

(b) Section at maximum hogging BM

$M_o = \frac{65828.863 \times 1000}{130 \times 0.874 \times 1160}$ = 5071.869 N-m

No of 30mm $\emptyset$ bars =$\frac{5071}{\frac{\pi}{4} \times 30^2} \approx 8$ bars

Hence provide 5 nos 30mm $\emptyset$ bars in one layer and 3 nos 30mm $\emptyset$ bars in the second layer.

They will be provided at top of the section, near support.

(c) Section at maximum sagging BM at mid-span

$M_o = \frac{299219 \times 1000}{130 \times 0.874 \times 1160}$ =2305.372 N-m

No of 30mm $\emptyset$ bars =$\frac{2305}{\frac{\pi}{4} \times 30^2} \approx 4$ bars

Hence the scheme of reinforcement will be as follows

At the support provide 5 nos 30mm $\emptyset$ bars in one layer and 3 nos 30mm $\emptyset$ bars in the second layer.

Continue upto section of maximum torsion i.e. at $\emptyset_m = 9.5^\circ = 0.116$ rad

at distance = $R\emptyset_m$ = 5× 0.116 = 0.83m

L_d= 52 $\emptyset$ = 1560 mm from support

At this point discontinuous 4 bars while continue remaining 4 bars

Similarly provide 4 bars of 25mm $\emptyset$ at the bottom throughout the length.

8. TRANSVERSE REINFORCEMENT

(a) At the point of maximum torsional moment

At the point of maximum torsion V= 576273.66 N

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

=576273.66 +$\frac{1.6 \times 49869.5}{0.6}$ = 709259.78 N

$\tau_{ve} = \frac{Ve}{bd} = 1.019\ N/mm^2$

This is less than t_{cmax}

$\frac{100\ A}{bd}$ = = 0.406

τ_c= 0.35 N/mm^2

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Since $\tau_c < \tau_{ve}$ shear Reinforcement is necessary

The area of cross section A_{sv} of the stirrups is given by

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

$d_1 = 1200 - 40 \times 2 - 25 = 1095$

$b_1 = 600 - 40 \times 2 - 25 = 495$

$$\frac{A_{sv}}{s_v} = 1.404$$

$$\frac{A_{sv}}{s_v} \geq \frac{\tau ve - \tau c}{\sigma_{sv}}$$

=2.676

Hence $\frac{A_{sv}}{s_v} = 2.676$

Using 12 mm diameter 4legged stirrups

$$A_{sv} = 4 \times 113 = 452\text{mm}^2$$

Or

$$A_{sv} = \frac{452}{2.676}$$
$$= 168.\text{mm}^2$$

However the Spacing should not exceed least of Spacing should not exceed x_1, 300, $\frac{x_1+y_1}{4}$

$x_1 = 495 + 25 + 12 = 532$ mm

$$y_1 = 1095 + 25 + 12 = 1032\text{ mm}$$

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

Hence provide 12mm diameter stirrups @ 160 mm c/c

(b) At the point of maximum shear (supports)

At supports F_o= 997396.7629N

τ_c= 0.38

$\tau_c < \tau_v$

$V_c = 0.38 \times 600 \times 1600 = 264480$N

$V_s = F_o - V_c$=732916.762N

The Spacing of 10mm diameter 4 legged Stirrups having A_{sv}= 314mm^2

$$\frac{A_{sv}\ \sigma_{sv} d_1}{V_s} = s_v$$

= 74.546 mm…….is to small

Hence use 12mm diameter 4 legged Stirrups having Asv = 452.39mm^2

$S_v = \frac{150 \times 452.39 \times 1160}{732916.762}$= 107 mm

Provide spacing 100 mm

(c)At mid-span SF is zero hence provide nominal shear reinforcement given by

$$\frac{A_{sv}}{b s_v} \geq \frac{0.4}{f_y}$$

$$\frac{A_{sv}}{s_v} = 0.578$$

Choosing 10mm diameter 4 legged stirrups $A_{sv} = 314\text{ mm}^2$

$$s_v = \frac{314}{0.578} = 543\text{mm}$$

Maximum permissible spacing = 0.75d = 0.75× 1160
= 870 mm
Or 300 mm

Hence provide 10mm 4 legged stirrups @ 300 mm c/c

Side face reinforcement

Since Depth>450mm &Torsional moment present

Provide side face reinforcement of 0.1%

$A_{sl} = \frac{0.1}{100} \times (600 \times 1200) = 720\text{mm}^2$

Provide 3-16mm diameter bars on each face having $A_{sl} = 6 \times 201 = 1206\text{ mm}^2$

9. DESIGN OF COLUMN

(a) Vertical loads on column

1. Weight of water = $W_w + W_0 = 10459794.66$N
2. Weight of tank = (Weight of top dome + cylindrical walls) + (Weight of conical dome) + (bottom dome) + (Bottom ring beam)

Weight of top dome + cylindrical wall=2820525.5N

Weight of conical dome = 1571287.201N

Weight of bottom dome = 540982N

Weight of bottom ring beam = 18000×π×10=565487N

Total weight on the tank = 5498281N

Total superimposed load = 5498281+10459794.66=15958075N

Load per column = $\frac{15958075}{8}$ = 1 994759.375N

Let the column be 400mm diameter = $\frac{\pi}{4} \times 0.7^2 \times 4200 = 9620$ N

Let the brace be of 300 mm × 600 mm

Length of each brace (L)

L = R,

$R \frac{\sin=\frac{\pi}{n}}{\cos=\frac{\pi}{n}}$ = 3.83 m

Clear length of each brace = 3.83 - 0.7 = 3.13 m

Weight of each brace = 0.3 × 0.6 × 3.13 × 2800 = 14085 N

Total height of structure = 6 + 1.2 + 2 + 5 +1.9
=26.1 m

Terrain category 2

Location: near Chennai

V_b= 50 m/s

K = 0.9 (Table 1) IS 875 part 3

Mean probable design life = 25 years

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Basic Wind speed(V_b) = 50 m/s

$k_1 = 0.9$

Table No 2.k_2

Terrain category 2

Height = 26.1m

20 1.07

26.1 k_2

30 1.12

$K_2 = 1.101$

$k_3 = 1$ (Plain topography)

$V_z = k_1 k_2 k_3 V_b^2$

$V_2 \approx 50$

$P_2 = 0.6V_z^2 = 1500 N/m^2$

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

Wind load on tank dome and ring beam

Wind load = $(5 \times 15.4) + \left(15.2 \times \frac{2}{3} \times 1.9\right) + (2\times 13.2) + (10.6\times1.23) + (1500\times0.7)$= 142142N.

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

It acts at C.G of projected area. In this case it is about 5.7m from bottom of ring beam B_2

Wind load on each panel of 4m height = (4×0.7×8) ×1500×0.7+ (0.6×10.6) ×1500 = 33060 N

Wind load at top panel = $\frac{1}{2} \times 23520 = 11760$N

The points of contraflexure

o_1, o_2, o_3 and o_4 are assumed to be at mid height of each panel

The shear force Q_w and moments M_w are due to wind at these planes are given below

Level	Q_w(N)	M_w(N/m)
o_4	142142+11760=153902	142142×7.7+11760×2=1118013.4
o_3	142142+11760+33060=186962	142142×11.7+11760×6+33060×2=1799741.4
o_2	142142+11760+33060+33060=220022	142142×15.7+11760×10+33060×8=2613709.4
o_1	142142+11760+33060+33060+33060=253082	142142×20.2+11760×14.5+33060×(10.5-6.5+2.5)=3256678.4

The axial thrust $V_{max} = \frac{4M_w}{nV_0}$

(n=8 columns)

$= 0.05\ M_w$ in the farther leeward column, the shear force

$S_{max} = \frac{2Q_w}{n}$ =0.25M_w in the farthest column, leeward shear force(S_{max})

In column on bending axis at crown of the above levels and bending moment $M = S_{max} \times \frac{h}{2}$ in column is tabulated

Table Vi - Maximum Shear And Moment Stress

Level	V_{max}	S_{max}	M(Nm)
o_4	55900	38475.5	76957
o_3	89987.07	46740.5	93481
o_2	130685.5	55005.5	110011
o_1	162833.92	63270.5	126541

The critical combination for various panel of the column are tabulated below

Table Vii Forces And Moments Calculations

Panel	Farthest leeward column		Column on bending axis	
	Axial load (N)	V_{max}	Axial load (N)	M(N/m)
o_4	2033240	55900	2033240	76957
o_3	2085804.375	89987.07	2085804.375	93481
o_2	21383.69	13065.5	2138369	11011
o_1	2200554.311	162833.92	2200554.325	1265001

Use M30 concrete for which

$\sigma_{cbc} = 10\ N/mm^2$

$\sigma_{cc} = 8\ N/mm^2$

For steel,

$\sigma_{st} = 230\ N/mm^2$

All the three values can be increased by 33.33% when taking wind into account.

Diameter of column = 700 mm

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Use 12 bars of 30mm Ø at an effective cover of 40mm

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

Equivalent area of column = A_c+ (m-1)

A_{sc}= 455525.607

M = $\frac{280}{3\times10}$ = 9.333

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

D = 700mm d' = 700 - 2 × 40 = 620

I_c= 1518086× $10^{10}$$mm^4$

Actual direct stress in column = σ'_{cc} = $\frac{126541\times10^3}{1.578086\times10^{10}} \times 350 = 2.80\ N/mm^2$

For safety of column, we have the condition

$$\frac{\sigma_{cc}'}{\sigma_{cc}} + \frac{\sigma_{cbc}'}{\sigma_{cbc}} \leq 1$$

$$= 0.66289 \leq 1$$

Hence safe

Use 10mmØ wire rings of 250mm c/c to tie up the main reinforcement.

Since column are 700mmØ increase the width of B_2beam 600 mm to 700 mm

Check for seismic effect

For empty tank = 5498281 N

For tank full = 15958075 N

For column 1

According to revised classification of earthquake zone, Madras comes under zone III (earlier to 2002 it was zone II (zone II and zone I are merged) after 2002)

Therefore, Zone III IS 1893-2002

Stiffness of column in a bay

$$I_{cc} = \frac{12EI}{L^3}$$

As it is the case of circular group of column

Young's modulus

E = 5000$\sqrt{f_{ck}}$ = 27386.128N/mm^2

$$I_c = 1.578080 \times 10^{10}\ mm^4$$

L= 4

(i.e; the distance between two braces and a panel)

$$k_c = \frac{12 \times 27386.125 \times 1.518086 \times 10^{10}}{4000^3}$$
$$= 81033.12181\ N/mm$$

Stiffness of 8 column

$\sum k_c$= 8×81033.12182

$\sum k_c$= 648264.98

Neglecting effect of bracing on stiffness

$$\frac{1}{k} = \sum\frac{1}{k}$$

When k=1,Fundamental=$2\pi\sqrt{\frac{w}{g\times k}} = 0.3147sec$

$$\frac{S_a}{g} = 0.2 \text{ from fig. 2, IS 1893} - 1980 \text{ pg. } 18$$

From IS 1893

$$A_n = \frac{zIS_a}{2Rg} \text{ for zone III } z = 0.16, I = 1, R = 2.50$$

$$A_n = 6.4 \times 10^{-3}$$

Force due to earthquake F_{eh}

F_{eh1} = mass × acceleration = 102131.68N

$\sum$M = Due to wind = 253082> F_{eh}

Therefore no need to consider earthquake in design of columns.

10. DESIGN OF BRACINGS

$$\frac{m_1}{\sin\left(\theta + \frac{\pi}{n}\right)} = \frac{m_2}{\sin\left(\theta - \frac{\pi}{n}\right)} = \frac{M}{\sin\left(\frac{2\pi}{n}\right)}$$

Hence, $m_1 = \frac{M}{\sin\left(\frac{2\pi}{n}\right)}\sin\left(\theta + \frac{\pi}{n}\right)$

And, $m_2 = \frac{M}{\sin\left(\frac{2\pi}{n}\right)}\sin\left(\theta - \frac{\pi}{n}\right)$

But,M = $S_1 \times \frac{h_1}{2} + S_2 \times \frac{h_2}{2}$,

$$\therefore M = \frac{Q_{w1}.h_1 + Q_{w2}.h_2}{n}\cos^2\theta$$

Where, Q_{w1} and Q_{w2} are the shear at the equivalent cylinder, at the point of contraflexure. Substituting the value of M in m_1 and m_2, we get

$$m_1 = \frac{Q_{w1}.h_1 + Q_{w2}.h_2}{n\sin\frac{2\pi}{n}}\cos^2\theta.\sin\left(\theta + \frac{\pi}{n}\right)$$

For m_1 to be maximum, differentiate it with respect to θ and equal it to zero.

$$\therefore \frac{d}{dx}\left[\cos^2\theta.\sin\left(\theta + \frac{\pi}{n}\right)\right] = 0$$

or

$\tan\left(\theta + \frac{\pi}{n}\right) = \frac{1}{2}\cos\theta$ Eqn 5.12.1

Solving the above, θ can be found.

$$m_1' = \frac{M'}{\sin\left(\frac{2\pi}{n}\right)}\sin\left(\theta - \frac{3\pi}{n}\right)$$

Where M' = Joint moment at joint A = $\frac{Q_{w1}.h_1 + Q_{w2}.h_2}{n}\cos^2\theta.\left(\theta - \frac{2\pi}{n}\right)$

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$$m_1' = \frac{Q_{w1}.h_1 + Q_{w2}.h_2}{n\sin\frac{2\pi}{n}}\cos^2\left(\theta - \frac{2\pi}{n}\right).\sin\left(\theta - \frac{3\pi}{n}\right)$$

If L is the horizontal length of brace AB, shear force in it is given by:

$S_b = \frac{m_1 - m_1'}{L} \times$

Or $S_b = \frac{1}{L}\frac{Q_{w1}.h_1 + Q_{w2}.h_2}{n\sin\frac{2\pi}{n}}\cos^2\theta.\sin\left(\theta + \frac{\pi}{n}\right)$ - $\cos^2\left(\theta - \frac{2\pi}{n}\right).\sin\left(\theta - \frac{3\pi}{n}\right)$

Differentiating the above for maximum value, we get $\theta = \frac{\pi}{n}$. The angle at B_1 (fig…) will then be = $\theta - \frac{\pi}{n} = \frac{\pi}{n} - \frac{\pi}{n} =$ zero

Hence, maximum shear force in a brace occurs when the wind blows parallel to it.

$\therefore\ (S_b)_{max} = \frac{Q_{w1}.h_1 + Q_{w2}.h_2}{L n\sin\frac{2\pi}{n}} \times \left[\cos^2\frac{\pi}{n}\sin\frac{2\pi}{n} + \cos^2\frac{\pi}{n}\sin\frac{2\pi}{n}\right]$

$= = \frac{Q_{w1}.h_1 + Q_{w2}.h_2}{L n\sin\frac{2\pi}{n}} \times \left(2\cos^2\frac{\pi}{n}\sin\frac{2\pi}{n}\right)$

The bending moment M_1 in brace is given by

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

Solving graphically we get $\theta = 24.8$,

$$M_{1max} = \frac{Q_{w1}h_1 + Q_{w2}h_2}{n\sin\frac{2\pi}{n}}\cos\theta^2 \times \sin(\theta + \frac{\pi}{n})$$

For the lowest junction C

$$h_1 = 5m, h_2 = 4m$$

$$M_{1max} = 255394.186Nm$$

The maximum shear force (S_b) in brace is $\frac{Q_{w1}h_1 + Q_{w2}h_2}{n\sin\frac{2\pi}{n}}\cos\theta^2 \times \sin(\theta + \frac{\pi}{n})$

$$S_{bmax} == 116199.0841N$$

Calculation of length of brace (L)

Each angle of polygon = (No. of sides - 2) × 180^0

8 columns forms octagon

n=8

Each angle = (8 - 2) × $180^0 = 135^0$

$$\tan 67.5 = \frac{L_1}{5.15 - L_1}$$

$$L_1 = 3.64163$$

$$5.15 - L_1 = 1.51$$

$$L = \sqrt{1.57^2 + +3.6416^2} = 3.94m$$

For $\theta = \frac{\pi}{8}$ the value of m_1

$(((m_1))_{\theta=\frac{\pi}{n}} = \frac{Q_{w1}h_1 + Q_{w2}h_2}{n\sin\frac{2\pi}{n}}\cos\frac{\pi^2}{n} \times \sin(\frac{\pi}{8} + \frac{\pi}{8})$

= 228912.1366

Twisting moment at $\theta = \frac{\pi}{8}$ is $M_t = 0.05m_1$

$0.05\times 228912.1366 = 11445.60683m$

Thus the brace will be subjected to the critical combination of maximum shear force and a twisting moment M_t when the wind blows parallel to it (i.e; $\theta = \frac{\pi}{8}$)

For M30 concrete $c = \sigma_{cbc} = 10\frac{N}{mm^2}$

$$\sigma_{st} = 230\frac{N}{mm^2}$$

$$k = 0.28865$$

$$k = 0.9038$$

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

Equating the moment of area at NA

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

From which $p = 8.168\times 10^{-3}$

% p = 0.8168 %

Since the brace is subjected to both BM and TM we have

$$M_{eq} = M_T + M$$

$$M = BM = (M_1)_{max} = 255394.19$$

$$M_T = \frac{T}{1.7}\left[1 + \frac{D}{B}\right] \text{where } T = M^t = 11445.607$$

$$M_{eq} = 277836.5523Nm$$

In order to find the depth of section equate M.R of section to external moment

$$b \times x_a \times \frac{c}{2}\left(d - \frac{x_a}{2}\right) + (m-1)A_{sc} \times c' \times (d - d_c') = M_{eq}$$

c = (increase by 33.33%) = 1.333 × 10 = 13.33

According to IS 456 modified modular ratio of steel in compression zone of doubly reinforced section is 1.5m

$$m' = 1.5m$$

$$= 1.5 \times 9.33$$

$$= 13.99$$

$$\cong 14$$

c' = Compression at steel level

$= 13.33\times\frac{(0.288-0.1)d}{0.288d}$

$= 8.7035\ {}^{N}/_{mm^2}$

Substituting the values in the above equation;

d = 600.2139 mm

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Adopt D = 700 mm so that c' = 700 – 25 - 125 =662.5 mm

A_{st} = $p \times b \times D$ = 1715.07 mm^2

No. of 25 mm bars $= \frac{1715}{\frac{\pi}{4}\times 25^2} = 3.49377$

$\cong$ 4 nos.

Provide 4 Nos. of 20 mm Ø bars each at top and bottom

$100\frac{A_s}{bD}$ = 0.935%

Maximum shear = 116199.057N

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

= 177242.3N

τ_{ve}= 0.84450Mpa

$\tau_{ve} = \tau_{c,max}$

τ_c= 0.37 + (18.5 × 0.04/25)

= 0.4 %

Hence Transverse reinforcement is necessary

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

b_1 = 300-25× 2 − 25= 225

d_1 = 700 - 25× 2 − 25 = 625

using 12 mm diameter 2 legged stirrups

$$A_{sv} = 2 \times \frac{\pi}{4} \times 12^2$$

= 226mm^2

$\frac{A_{sv}}{s_v}$ = 0.847069

Minimum Reinforcement

$$\frac{A_{sv}}{bs_v} = \frac{\tau ve - \tau c}{\sigma_{sv}}$$

=0.57913

$$\frac{A_{sv}}{s_v} = 0.84707$$

$$s_v = 579.496 \text{ mm}$$

Spacing should not exceed x_1, 300,$\frac{x_1+y_1}{4}$

x_1 = 225 + 25 + 12 = 272 mm

$$y_1 = 625 + 25 + 12 = 672 \text{ mm}$$

$\frac{x_1+y_1}{4}$=236 mm

Hence provide 12mm diameter stirrups @ 230 mm c/c

D>450mm, hence provide side face reinforcement of 0.1%

A_{sl}= 210 mm^2

Provide 2-10mm diameter at each face giving total A_{sl}= 44 × 78.5 = 314 mm^2

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

11.DESIGN OF RAFT FOUNDATION

Vertical load from filled tank and column = 2200554.375 × 8 = 17604435 N

Weight of water = 10459794.66

Vertical load on empty tank and column = 17604435 – 1045979 =7144640.34 N

V_{max} due to wind load = 162833.9 × 8

= 1302671.2 N

which is less than 33.33% if the super imposed load $= \left(\frac{33.33}{100} \times 10454794.7\right)$ = 3486598.22 N

Assume Self weight etc. = 10% = 1760443.5 N

Total load = 1.1 × 17104435 = 19364878.5 N

Area of foundation equation = $\frac{17364878.5}{180000}$

= 107.58 m^2

Circumference of circular column = π × 10 = 31.42 m {i.e. (10.6 – 2 × 0.3 = 10 m)}

Width of foundation required = $\frac{107.57}{31.42}$

= 3.424 m

Take width = 3.64 m

Hence, inner diameter = 10 - 3.64

= 6.36 m

Outer diameter = 10 + 3.64 = 13.64 m

Area of annular raft = $\frac{\pi}{4}$ × (13.64^2- 6.36^2)

= 114.35 m^2

Moment of inertia of slab @ diameter $\frac{\pi}{64} \times (13.64^2 - 6.36^2)$ = 1618.8 m^4

Total load on tank empty = 7144640-34 +1760448.5

= 8905083.8450 N

Stabilizing moment

= 8905083.84 × $\frac{13.84}{2}$

= 60732671.8 N-m

Let the base of raft be 2m below ground level

Therefore M_w at base = 142142 ×11760 × 33060 × (14 + 10 + 6)

= 44572245.40 N-m

Hence the soil pressure @ edges along diameter are

(a) Tank full =188610.2864 N/m^2 or 150084.511 N/m^2

(b) Tank empty = 97138.54 N/m^2 or 58612.8211 N/m^2

Under the wind load the allowable bearing capacity is increased to 180 × 1.333 = 240 KN/m^2

Which is greater than the maximum soil pressure = 188.610 KN/m^2

Hence the foundation raft will be designed only for super imposed load

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A ring beam of 100 mm width may be provided. The foundation will be designed for an average pressure.

$p = \frac{17604435}{114.35} = 153952.2081 \ ^{N}/_{m^2}$

The overhang "x" of raft slab $= \frac{1}{2} \times \{(13.64 - 6.36) - 0.7\} = 1.47$ m

B.M = 166337.6633 N-m

S.F = 153952.2 × 1.47

= 226309.746 N

d = 357.98 mm

Provide 400 mm thick slab with effective depth = 340 mm

Provide total depth of 250 mm at the edge

$A_{st} = \frac{166337.66 \times 1000}{230 \times 0.9038 \times 360}$

$= 2141.69 \text{ mm}^2$

Spacing of 20 mm Ø bars $= \frac{1000 \times \frac{\pi}{4} \times 20^2}{2141.7}$

= 146.7 mm

Hence provide 20 mm Ø radial bars @ 140 mm $^{c}/_{c}$ at bottom of slab

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

$= 600 \text{ mm}^2$

Spacing of 10 mm Ø bars $= \frac{1000 \times 78.5}{600}$

= 130.5 mm

≅ 130 mm

Hence provide 10 mm Ø bars @ 130 mm $^{c}/_{c}$ at supports. Increase spacing as 200 mm @ edges.

VI – CONCLUSION

Elevated water tanks provide head for supply of water. When water has to be pumped into the distribution system at high heads without any pumps for supply however pumps are necessary for pumping only till tank is filled. Once it is stored in tank the gravity creates the pressure for free, unlike pumps. We need pressurized water to fledge and make taps eject water at an appropriate rate. Elevated tanks do not require continuous operation of pump, as it will not affect the distribution system since the pressure is maintained by gravity. Strategic location of tank can equalize water pressure in the distribution system

The pressure of water flowing out of an elevated tank depends upon the depth of the water in tank .A nearly empty tank probably will not provide enough pressure while a completely full tank may provide too much pressure the optimal pressure is achieved at only one depth .While elevated tank provide can provide the best pressure, they are far more expensive and generally, it is used where supply is high demand

Elevated circular water tanks with large capacity and flat bottom needs large reinforcement at the ring beams. 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.

VII – SUMMARY

An effort has been taken to provide a design of circular overhead water tank which is more economical, simple and having a better life span with the help of IS 3370-2009 in WORKING STATE METHOD.

Design of water tank manually is tedious job ,in this project circular INTZE WATER TANK is designed using membrane analysis separate continuity analysis is not done Calculations for continuity effect can be done by stiffness methods but it makes the process very complicated .continuity is taken in to account by introducing sufficient steel at joints.

FUTURE SCOPE

From the review of all the papers, it has been concluded that most of the authors have designed the circular water tank with the help of SAP2000, C++ & STAADPRO software. So, the attempt is to be made to design the overhead circular water tank with the help of ETABS software. A reinforced concrete member of liquid retaining structure is designed on the usual principles ignoring tensile resistance of concrete in bending

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