DESIGN OF CIRCULAR OVERHEAD WATER TANK BY WSM & LSM METHOD

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Abstract - *Water tanks are widely used for storing water. Water storage is very important as it plays a vital role in everyday life. Storage reservoirs and water tanks are used to store water, liquid petroleum, petroleum products and similar liquids. This project gives in brief, The theory behind the design of liquid retaining structure (Overhead Circular Water Tank) using Limit state method and Working stress method.*

***Key Words*: Circular overhead water tank with top and bottom dome, population forecasting method, working stress method, limit state method, IS code, etc.**

1. INTRODUCTION

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 can be classified into two types:

❖ Based on location –
- Tanks resting on ground
- Tanks under ground
- Elevated tanks

❖ Based on shapes –
- Circular tanks
- Rectangular tanks
- Square tanks
- Spherical tanks
- Intze tanks

The elevated water tanks must remain functional even after the earthquakes as water tanks are required to provide water for drinking and firefighting purpose. These structures has large mass concentrated at the top of slender supporting structure hence these structure are especially vulnerable to horizontal forces due to earthquakes. All over the world, the elevated water tanks were collapsed or heavily damaged during the earthquakes because of unsuitable design of supporting system or wrong selection of supporting system and underestimated demand or overestimated strength.

1.1 Proposed Site

The proposed site for our project is located at Ghumri village of Karjat taluka at Ahmednagar district. Our site situated at the place where all the natural condition are suitable for the construction of elevated overhead water tank. This location is one of the developing areas, where there is steady increase in population in recent years. The population of the area according to recent survey is around 1816. Thus this location requires a periodic water supply system at least twice a week. This location consist nearly 50% agricultural land. Around 450+ houses are there and so it requires more than 100 m^3 capacity water tank. From the three major types of water tank, we had adopted elevated overhead circular water tank because the location needs pressurized water supply.

1.2 Sources of Water Supply

The various sources of water can be classified into two categories:

❖ Surface sources –
- Ponds and lakes
- Streams and rivers
- Storage reservoir
- Oceans

❖ Sub surface sources –
- Springs
- Infiltration wells
- Wells and tube wells

2. OBJECTIVES

- To make a study about the design of water tanks.
- Design of circular overhead water tank by LSM method.
- Design of circular overhead water tank by WSM method.
- Comparison between WSM and LSM method.
- Preparing a water tanks design which is economical and safe, providing proper steel reinforcement in concrete and studying its safety according to various codes.

Table -1: Detail of data collection

Capacity of tank	450 m^3
Soil bearing capacity (SBC)	20 MT/m^2
Height of tank from ground	20.35 m
Grade of concrete for all members	M30
Ground water level	2 m
Type of staircase	ladder
Use of water	domestic purpose only
Freeboard	0.38 m only
Earthquake zone	IV
Thickness of wall	100 mm
No. of columns	6
Excavation	2 m
Water provided in area	Ghumri
Current population in 2011	1816
Population forecasting 2021	2825
Average daily consumption	135 lit/person/day

3. METHODOLOGY

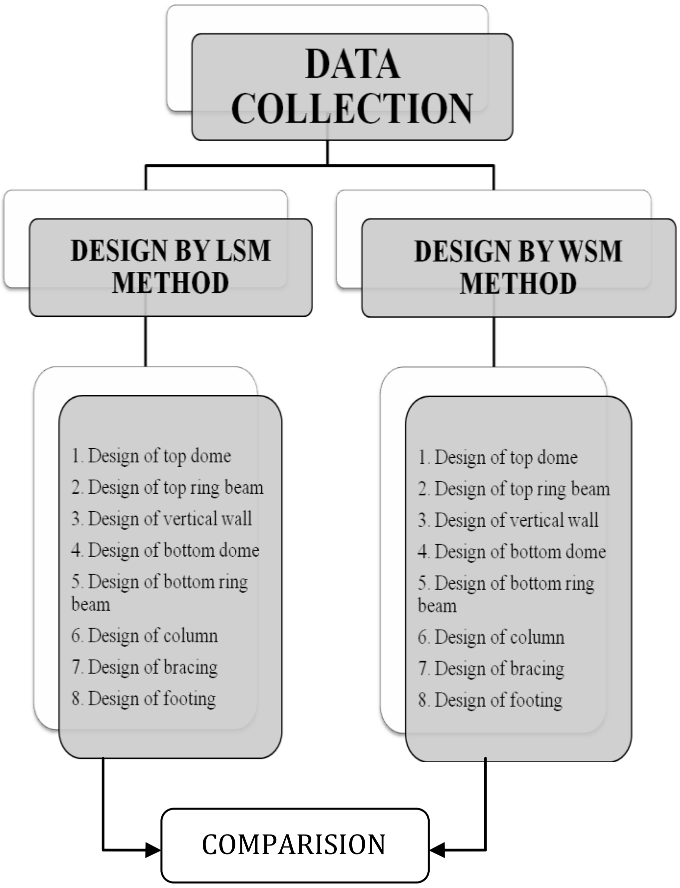

Fig -1: METHODOLOGY

4. DESIGN OF CIRCULAR OVERHEAD WATER TANK

I. POPULATION FORECAST

a. Arithmetic Progression Method:
This method is suitable for large and old city with considerable development. If it is used for small, average or comparatively new cities, it will give lower population estimate than actual value. In this method the average increase in population per decade is calculated from the past census reports. This increase is added to the present population to find out the population of the next decade. Hence, dp/ dt = C i.e., rate of change of population with respect to time is constant. Therefore, Population after nth decade will be Where, Pn = P+ n. c is the population after 'n' decades and 'P' is present population.

Year	Population	X - increase
1991	690	-
2001	1476	786
2011	1816	340
total		563

$Po + n_x = 1816 + (1 X 563)$
=2379
Population 2021= 2379

b. Geometric Progression Method:
This method gives higher values and hence should be applied for a new industrial town at the beginning of development for only few decades.

Year	Population	Increase	Geometric increase rate of growth
1991	690	-	
2001	1476	786	1.139
2011	1816	340	0.23

$IG = Pn = Po\left(1 + \frac{r}{100}\right)^n$
$r = \sqrt{1.139 \times 0.23}$
r = 0.51
$Pn = 1816 \left(1 + \frac{51.0}{100}\right)^1$
Pn = 2742.12

c. Incremental Increase Method:
The incremental increase is determined for each decade from the past population and the average value is added to the present population along with the average rate of increase.

Year	Population	Increase	Incremental increase
1991	690	-	
2001	1476	786	
2011	1816	340	+ 446
Total		1126	446
Average		563	446

$Pn = Po + nx + \left(\frac{n(n+1)}{2}\right) \times y$
When n = 1
$Pn = 1816 + (563 \times 1) + \left(1 \times \frac{(1+1)}{2}\right) \times 446$
Pn = 2825
Therefore design population of 2825
Assuming per capita demand 135 lpcd
Capacity required = $\frac{2825 \times 135}{1000}$ = 381.375m
$\cong 450\ m^3$

II. DESIGN OF CIRCULAR OVERHEAD WATER TANK BY LSM METHOD

a. DIMENSION OF TANK:

- Diameter of cylindrical portion,

$D = \sqrt{\frac{4V}{\pi H}}$

Where,
D = Inner diameter
V = Volume of tank (capacity = 450 m3)
H = height of water (3.8 m)

$D = \sqrt{\frac{4 \times 400}{\pi \times 3.8}}$

D = 11.57m $\cong$ 12m

- radius of cylindrical portion,
R = 6m
- rise of top dome = $h_1 = 0.2 \times D = 0.2 \times 12 = 2.4$ m
- rise of bottom dome = $h_2 = 0.16 \times D = 0.16 \times 12$ = 2 m
- thickness of wall (t) = 100 mm
- diameter of cylindrical part (D) = 12 m
- arc equation of top beam = r_1

$r_1 = \frac{\left(\frac{D}{2}\right)^2 + (h_1)^2}{2 \times h_1} = \frac{\left(\frac{12}{2}\right)^2 + (2.4)^2}{2 \times 2.4} = 8.7\ m$

- arc equation of bottom beam = r_2 =

$r_2 = \frac{\left(\frac{D}{2}\right)^2 + (h_2)^2}{2 \times h_2} = \frac{\left(\frac{12}{2}\right)^2 + (2)^2}{2 \times 2} = 10\ m$

- height of vertical wall = h_3

Volume of cylindrical part = $\frac{\pi}{4} \times D^2 \times h_3$

Volume of bottom dome (sphere) =
$\pi \times (h_2)^2 \times \left(r_2 - \frac{h_2}{3}\right)$

450 = volume of cylinder − volume of bottom dome

$450 = \left(\frac{\pi}{4} \times D^2 \times h_3\right) - \pi (h_2)^2 \times \left(r_2 - \frac{h_2}{3}\right)$

$450 \left(\frac{\pi}{4} \times 12^2 \times h_3\right) - \pi (2)^2 \times \left(10 - \frac{2}{3}\right)$

$h_3 = 5.00$ m

b. DESIGN OF TOP DOME:

- dead load = 2.5 KN/m^2
- live load = 1.5 KN/m^2

- total load = dead load + live load = 2.5+1.5 = 4 KN/m^2

  $\sin\theta = \frac{D/2}{r_1} = \frac{12/2}{8.7} = 0.689$

  $\theta = 43.60^c$

  $\cos\theta = \cos 43.60^c = 0.724$
- maximum meridonal thrust = T_1

  $T_1 = \frac{w \times r_1}{1+\cos\theta} = \frac{4\times8.7}{1+0.724} = 20.185$ KN/m
- meridonal stress (radial) = $\frac{T_1}{area} = \frac{20.185\times10^3}{1000\times100}$

  = 0.2018 N/mm^2

  σcc = Permissible concrete stress in concrete (direct compression)

  For M30 =8 N/mm^2

  Minimum Ast = 0.35%

  Ast = $\frac{0.35}{100} \times 100 \times 1000$

  Ast = $350mm^2$

  Spacing = $\frac{1000\times(\frac{\pi}{4}\times8^2)}{350}$ = 143 mm

  So, provide 8mm Ø bars @ 140 c/c (radially and circumferencetially)

c. DESIGN OF TOP RING BEAM (B_1):

- hoop tension =

  $(T\cos\theta) \times \frac{D}{2} = (20.185 \times 0.724) \times \frac{12}{2}$

  = 87.67 KN
- steel required = $\frac{87.67\times10^3}{130}$ = 674.30 mm^2

  Permissible stress in steel (HYSD) = σst = 130 N/mm^2

  ∴Provide 6 NOS bars of 12 mm Ø
- direct tensile stress in concrete =

  Permissible direct tensile stress = σct = 1.5 N/mm^2
- Size of ring beam = 200 × 300

d. DESIGN OF VERTICAL WALL:

- Hydraulic pressure = P = 49050 N/m^2
- Hoop tension due to hydraulic pressure = P × $\frac{D}{2}$ =

  49050 × $\frac{12}{2}$ = 294300 N/m
- Ast = 2027.43 mm^2
- Spacing = 138.77 mm
- Provide 20 mm Øbar @ 130 mm c/c
- Tensile stress in concrete =

  $\frac{F}{Ac+(m-1)Ast} = \frac{294300}{(130\times1000)+(9.33-1)\times2827.43}$

  = 1.916 N/mm^2
- Total vertical load =

  Vertical component of thrust + dead load of wall + ring beam +top dome

  = (Tsin θ) + (25 ×0.10 ×5) + (25×0.2 ×0.3) +

  $\left(\frac{25\times0.10\times2\pi\times2.4\times8.7}{2\pi\times\frac{12}{2}}\right)$ = 36.619 KN/m

  Compressive stress = $\frac{36.61}{100}$ = 0.3661 N/mm^2

  Provide 8mm Ø bar @ 140 mm c/c each face.

e. DESIGN OF BOTTOM DOME:

- Dead load of dome = 2.5 KN/m^2
- Volume = volume of cylinder (v_1) – volume of bottom dome (v_2)

  Volume = 565.48 – 117.28 = 448.2 m^3

  Load intensity due to water =

  $\frac{wt.of water}{surface area of dome} = \frac{volume\times\rho\times g}{2\pi\times r_2\times h_2} =$

  $\frac{448.2\times1000\times9.81}{2\pi\times10\times2}$ =34.98 KN/m^2
- Total load = 2.5 + 34.98 = 37. 48 KN/m^2

  $\sin\theta = \frac{D/2}{r_2} = \frac{12/2}{10} = 0.6$

  $\theta = 36.86^c$
- Meridonal thrust = T_2

  $T_2 = \frac{w\times r_2}{1+\cos\theta} = \frac{37.48\times10}{1+\cos 36.86}$ = 208.21 KN/m

  Meridonal thrust compressive stress = $\frac{208.21}{0.1}$

  = 2.082 N/mm^2
- Provide 8mm Ø bar @ 140mm c/c

f. DESIGN OF BOTTOM RING BEAM (B_2):

  Assume 250×300 mm beam

- Dead load =

  Dead load of vertical wall + top dome + bottom dome + top ring beam + bottom ring beam

  = 32.905 KN/m
- Hydraulic pressure =

  $\frac{volume\times\rho\times g}{2\pi\times r} = \frac{(565.48-117.28)\times1000\times9.81}{2\pi\times6}$

  = 116.629 KN/m
- Total load = 32.905 + 116.629 = 149.53 KN/m

  This ring beam is design as circular beam supported by six columns of 300mm diameter.
- Coefficient for maximum moment =

No. of supports	2α	λ	λ'	λ''	βo
4	90^c	0.07	0.137	0.021	19.25^c
6	60^c	0.045	0.089	0.009	22.75^c
8	45^c	0.033	0.066	0.005	9.5^c
10	36^c	0.027	0.054	0.003	7.5^c
12	30^c	0.023	0.043	0.002	6.25^c

- Moment equivalent =

  Sagging moment at mid span = M+ = 2 w r^2 $\alpha\lambda$

  = 528.70 KN. m

Hogging moment at support = M - = -2 w r^2 $\alpha\lambda'$ = -1045.65 KN. m

Maximum torsional moment = T = 2 w r^2 $\alpha\lambda''$ = 105.74 KN. m

0.138 × fck × bd^2 = M

0.138 × fck × bd^2 = 1045. 65× 10^6 × 1.5

d = 870 mm

b = 500 mm

Depth of beam= D'= 900 mm

M equivalent = M + $\frac{T\times\left(1+\frac{D}{b}\right)}{1.7}$

= $1045.65 + \frac{105.74\times\left(1+\frac{0.9}{0.50}\right)}{1.7}$ = 1219.81 KN. m

- Longitudinal reinforcement =

1.5 × M eq = 0.87 × fy× Ast $\left(d - \frac{0.87\times fy\times Ast}{0.36\ fck\times b}\right)$

1.5 × 1219.81× 10^6 = 0.87 × 415 × Ast $\left(870 - \frac{0.87\times 415\times Ast}{0.36\times 30\times b}\right)$

Provide 6 NOS bar of 22 mm Ø bars

As depth exceeds 500 mm provide 0.1% steel along vertical sides

= $\frac{0.1}{100}$ × 500 × 870 = 470 mm^2

Provide 4 NOS bar of 12 mm Ø bars

- Transverse steel =

Ast = 0.52%

τc = 0.31 N/ mm^2

Shear force (v) = $\frac{load\times span}{2\times no.column}$ = $\frac{149.53\times 2\pi\times 6}{2\times 6}$ = 469.76 KN

$\tau v = \frac{V+1.6\times\frac{T}{B}}{b\times d} = \frac{469.76+1.6\times\frac{105.74}{500}}{500\times 870}$

= 1.080 N/ mm^2

30mm cover top and bottom

b_1 = 500 60 = 440 mm

d_1 = 900 60 = 840 mm

Asv = Area of transverse steel

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

Providing 2- legged 10mm dia bars

Spacing = 260mm

g. DESIGN OF COLUMN:

- 6 columns equally spaced on 12 m diameter circle. Distance between columns centre to centre 10.3m
- Height of column = 12.37 m
- Diameter of column = 300 mm
- Total load on ring beam = 149.53 KN
- Total design load on ring beam =

W = $\pi\times D\times w$ = $\pi\times 12\times 149.53$

= 5637.14 KN

- Vertical load on each column =

P = $\frac{5637.14}{6}$ = 939.52 KN

- Factored load =

Pu = 1.5 × P = 1.5 × 939.52 = 1444.73 KN

- Condition = column effectively held in position and restrained against rotation in both ends.
- L effective = 0.5 L = 0.5 × 12.37 = 6.185 m
- Slenderness ratio =

$\frac{L\ effective}{D} = \frac{6.185\times 10^3}{300}$ = 20.61 > 12 mm

- Minimum eccentricity =

$e_{min} = \frac{L}{500} + \frac{D}{30} = \frac{12370}{500} + \frac{300}{30}$ = 34.74 > 20mm

$\frac{e_{min}}{D}$ < 0.05 m

$\frac{34.74}{300}$ = 0.115 > 0.05

Member is subjected to axial force or uniaxial bending (Assumed uniaxial bending)

- Area of reinforcement = Asc

Ag = $\frac{\pi}{4}\times 300^2$ = 70685.83 mm^2

Ac = Ag − Asc = 70685.83 − Asc

Pu_z = 0.45 fck Ac + 0.75 fy Asc

1444.73 $\times 10^3 = 0.45\times 30\times (70685.83 - Asc) + 0.75\times 415\times Asc$

Asc = 2242.09 mm^2 = 2245 mm^2

Assume 5 % of steel.

Adopt 20 mm Ø bar

AØ = 314.159 mm^2

NOS = $\frac{Asc}{AØ} = \frac{2245}{314.59}$ = 7.146 ≅ 8 bars

Ast provided = 8×314.159 = 2513.272 mm^2

Asc < Ast provided .... Hence ok.

- Helical reinforcement (spiral ties) =

Assume cover 40 mm

Core diameter (dc) = D $-2\times cover$ = 300 -2×40 = 220 mm

Area of core (Ac) = $\frac{\pi}{4}\times dc^2 = \frac{\pi}{4}\times 220^2$ = 38013.27 mm^2

P = Pitch of spiral ties

Vc = Volume of core

Vc = $38013.27\times P$

Using 10 mm Ø spirals (helical reinforcement)

Volume of helical reinforcement =

Vhs = $\frac{\pi}{4}\times 10^2\ \times\pi\times(20-10)$ = 51815.423 mm^3

$\frac{volume\ of\ helical\ reinforcement}{volume\ core}$

=0.36× $\left(\frac{Ag}{Ac}-1\right)\times\frac{fck}{fy}$

$\frac{51815.423}{38013.27 \times P} = 0.36 \times \left(\frac{70685.83}{38013.27} - 1\right) \times \frac{30}{415}$

P = 60.939 mm

- Maximum pitch =

i. 75 mm

ii. $\frac{1}{6} \times$ core diameter = $\frac{1}{6} \times 220$ = 36.66 mm

- Minimum pitch =

i. 25 mm

ii. 3 × diameter of helical steel = 3 × 10 = 30 mm
Pitch = 36.66 mm > 30 mm
Provide pitch of 30 mm.

- Transverse steel =
Diameter of circular ties =
$\emptyset r = \frac{1}{4} \times \emptyset L$ = 5 mm or 8 mm ∴ $\emptyset r$ = 8 mm

- Spacing of circular ties =

i. D = 300 mm

ii. 16× ØL = 16 × 20= 320 mm

iii. 300 mm
Take whichever is less
Spacing = 300 mm c/c

- Lap length =
Ld = development length of bars
$Ld = \frac{\emptyset \times \sigma_s}{4 \times \tau_{bd}}$
Ø = nominal diameter
σ_s = stress in bar at the section considered at design load
τ_{bd} = design bond stress (M30)
$\sigma_s = 0.87 \times fy = 0.87 \times 415 = 361.05$
$Ld = \frac{20 \times 361.05}{4 \times 1.5}$ = 1203.5 mm
Lap length = 30 × Ø = 30 × 20 = 600 mm
Ld =1203.5 mm

- Lap length = 600 mm
Take whichever is greater. ∴ Provide 1203 mm lap length.

h. DESIGN OF BRACE BEAM :

- Square beam = 300 × 300 mm
- Length between bracing = 3.0925 m
- Self weight of slab = 25 × 0.1 = 2.5 KN
- Self weight of beam = 0.3 × 0.3 × 25 = 2.25 KN
- Live load = 2.5 KN
- Total load = 2.5 +2.5+2.25 = 7.25 KN/m
- Effective depth = 300 – 50 = 250 mm
- Load calculation =
Design load = 7.25 × 1.5 = 10.875 KN/m
Moment calculation =
$Mu = \frac{W \times L^2}{8} = \frac{10.875 \times (3.0925)^2}{8}$ = 13.00 KN. m

Mub = 0.138 × fck × bd² = 0.138 × 30 × 300 × 300² = 111.78 × 10^6 KN. m

- Reinforcement details =
Mub = 0.87 fck Ast$\left(d - \frac{0.87\, fy\, Ast}{0.36\, fck\, b}\right)$
111.78 × 10^6 = 0.87× 415 × Ast
$\left(d - \frac{087 \times 415 \times Ast}{0.36 \times 30 \times 300}\right)$Ast = 1350. 62 mm^2
Assume 4 – 25 mm Ø bar

- Shear reinforcement =
$Vu = \frac{W \times L^2}{2} = \frac{10.875 \times (3.092)^2}{2}$ = 51.985 KN
$\tau_v = \frac{Vu}{bd} = \frac{51.985 \times 10^3}{300 \times 300}$ = 0.577 N/mm^2
Pt % = 100 $\frac{Ast}{bd}$ = 100 × $\frac{1350.62}{300 \times 300}$ = 1.500 %
From table 19 , IS 426- 2000
τ_c = 0.76 N/mm
Hence $\tau_c > \tau_v$ the section is safe in shear yet minimum shear reinforcement is provided for beam.
Assuming 8 mm Ø bar 2 legged
$Sv = \frac{0.87\, fy\, Asv}{0.4\, b} = \frac{0.87 \times 415 \times 100.53}{0.4 \times 300}$ = 302.472 mm
≅ 300 mm
Provide stirrups 8 mm @ 300 mm c/c

i. DESIGN OF FOOTING:

- Area of footing =
As per IS code guideline self weight of footing is taken 10% of column load.
W= load carried by column + self weight of footing
W = 1444.73 + $\frac{10}{100}$ × 1444.73 = 1589.203 KN
Load on foundation soil.
Note: as per IS recommendation for the purpose of design of circular column of size 0.7170 in diameter given circle is taken.
Design of square footing is done exactly in the same manner as it was for square column.

- Side of square column =
b = 0.717 D = 0.717 × 300 = 215.1 ≅ 220 mm
area of footing = $\frac{load}{SBC} = \frac{1589.203}{200}$ = 7.946 m^2
side of square footing =
B = $\sqrt{7.946}$ = 2.81 ≅ 3 m
Therefore, size of square footing for circular column
B × B = 3 × 3 m

- Factored soil pressure on footing =

$q_u = \frac{factored\ load}{actual\ area} = \frac{1444.73}{3 \times 3}$ = 160.525 KN/m^2

- Depth of footing by bending moment criteria = Critical section for BM is taken as face of column.

  Mu = $q_u \times B \times \left(\frac{B-b}{2}\right) \times \left(\frac{\frac{B-b}{2}}{2}\right) = q_u \times B \times \left(\frac{(B-b)^2}{8}\right) = 481.575 \times \left(\frac{(3-0.22)^2}{8}\right)$

  = 465.22 KN. m

  B M at critical section.

  Note: in equilibrium condition, Mu = Mu lim

  Mu lim = 0.138 $\times$ fck $\times$ Bd^2

  465.22 $\times 10^6$ = 0.138 $\times$ 30 $\times$ 3000 $\times$ d^2

  d = 193.538 mm $\cong$ 194 mm

  Increase d 1.75 to 2.25 times to make depth of footing safe in shear action.

  d = 2 $\times$ 194 = 388 mm $\cong$ 390 mm

- Check depth of footing against one way shear action, the critical section for one way shear is at a distance d from force of column.

  Vu = $q_u \times B \times \left(\frac{B-b}{2} - d\right)$

  = 481.575 $\times \left(\frac{3-0.22}{2} - 0.390\right)$ = 481.575 KN

- Factored S.F at critical section =

  Shear stress developed at critical section =

  $\tau_v = \frac{Vu}{bd} = \frac{481.57 \times 10^3}{3000 \times 390}$ = 0.41 N/mm^2

  Shear strength of concrete =

  It depends upon grade of concrete and percentage of steel. Assume Pt% = 0.5 %

  Value of τ_c from table 19, IS 426- 2000

  τ_c = 0.50 N/mm^2

  As, $\tau_c > \tau_v$ hence depth of footing in safe against one way shear.

- Check depth of footing for bending shear action, The critical section for punching shear is at 'a' distance d/2 from face of the column,

  a = $\frac{d}{2} + \frac{d}{2}$ + b = $\frac{390}{2} + \frac{390}{2}$ + 220 = 610 mm

  Vu' = $q_u \times$ (B^2- a^2) = 160.52 $\times$ (3^2- 0.61^2) = 1384.99 KN

  Shear stress developed by punching shear =

  b_0 = perimeter of critical section = 4 $\times$ 610 = 2440 mm

  $\tau_v{}' = \frac{Vu'}{b_0 d} = \frac{1384.99 \times 10^3}{2440 \times 220}$ = 2.58 N/mm^2

- Shear strength of concrete against punching =

  $\tau_c{}'$ = K $\times$ 0.2 $\times \sqrt{fck}$ = 0.50 N/mm^2

  Where,

  K = depends upon depth of footing slab and for d> 300 mm = 1

  $\tau_c{}'$ = 0.2 $\times \sqrt{30}$ = 1.095 N/mm^2

- Area of steel =

  Mub = 0.87 fck Ast $\left(d - \frac{fy\,Ast}{fck\,b}\right)$

  456.22 $\times 10^6$ = 0.87 $\times$ 415 $\times$ Ast $\left(390 - \frac{415 \times Ast}{30 \times 3000}\right)$

  Ast = 3374.62 mm^2

  Now using 18 mm Ø bar

  Spacing = B $\times \frac{ast}{Ast}$ = 3000 $\times \frac{254.46}{3374.62}$ = 226.22 $\cong$ 220 mm

j. DESIGN OF STAIRCASE (SPIRAL) :

- Staging height = 12.37 m
- Total height = 20.35 m (up to top of dome)

  Assume riser = 250 mm

- No. of steps = $\frac{20.35}{0.25}$ = 81.43 $\cong$ 81
- Considering weight of each precast steps = 0.1 $\times$ T
- Live load = 0.05 $\times$ T
- Total = 0.15 $\times$ T = 0.15 $\times$ 81 T = 12.15 T
- Self wt. (DL) = 25 $\times \frac{\pi}{4} \times$ d^2 $\times$ 2.55 = 25 $\times \frac{\pi}{4} \times 0.3^2 \times 2.55$ = 4.506 T

  Total = 12.15 T + 4.506 T = 16.65 T $\cong$ 20

  Providing diameter 300 mm column with 6 – 12 Tor load carrying capacity of concrete alone in M-30

  = $\left(\frac{\pi}{4} \times 300^2 - 6 \times \frac{\pi}{4} \times 12^2\right) \times \frac{4}{9810}$ = 70685.83 – 678.584 $\times \frac{4}{9810}$ = 28.54 T $\cong$ 20 T

III. DESIGN OF CIRCULAR OVERHEAD WATER TANK BY WSM METHOD

a. DESIGN OF TOP DOME :

- Thickness of dome = 100 mm
- Meridonal force (T1) =
- Hoop tension (T2) =

  T1 = $\frac{W \times R}{1 + cos\theta}$

  W = load of dome

  L.L = 1.5 KN/ m^2

  Self weight = thickness $\times$ density = 0.10 $\times$ 25 = 2.5 KN/ m^2

  Total load = 1.5 + 2.5 = 4 KN/ m^2

- Radius of curvature of dome =

h = rise of dome

h = 0.2 $\times$ D = 2.4 m

R = 8.7 m

Sin $\theta = \frac{\frac{D}{2}}{R} = \frac{6}{8.7} = \theta = 43.60^{o}$

Cos θ = Cos (43.60°) = 0.724

T1 = $\frac{W \times R}{1+cos\theta} = \frac{4 \times 8.7}{1+0.724}$ = 20.18 KN/ m

Meridonal stress = $\frac{force}{area} = \frac{20.18 \times 10^3}{1000 \times 10}$

= 0.202 N/ mm^2

- Direct tension stress = σ_{ct}
  For M30 concrete = 15 Kg/ cm^2
- Permissible stress in concrete = 8 N/ mm^2
  0.202 < 8 N/ mm^2 … safe
- Area of reinforcement =
  Provide 0.24 % minimum reinforcement
  Ast = $\frac{0.24}{100} \times 1000 \times 100$ = 240 mm^2
  Provide 8 mm Ø bar @ 200 mm c/c (Ast = 251 mm^2)
  For hoop force (T2) =
  T2 = W $\times$ R $\times$ $\left(cos\theta - \frac{1}{1+cos\theta}\right)$ = 5 KN/ m
  Hoop stress = $\frac{5 \times 10^3}{1000 \times 100}$ = 0.05 < 8 N/ mm^2 … safe
  Provide 0.24 % minimum reinforcement.

b. DESIGN OF TOP RING BEAM :

- It is designed for hoop tension
  W = T1 $cos\theta$ = 20.18 $\times$ Cos (43.60)
  = 14.61 KN/ m
  Total hoop tension in beam =
  W $\times \frac{D}{2}$ = 14.61 $\times \frac{12}{2}$ = 87.66 KN
- Ast for hoop tension =
  $\frac{T}{\sigma_{st}} = \frac{87.66 \times 10^3}{30}$ = 674.30 mm^2
  Provide 12 mm Ø bar @ 150 mm c/c (Ast = 753 mm^2)
- To find out dimension of R.B =
  $\sigma_{ct} = \frac{T}{Ag+(m-1)Ast} = \frac{87.66 \times 10^3}{250 \times D+(9.33-1) \times 753}$ < 1.5
  Ag = b $\times$ D
  m = $\frac{280}{3\ \sigma_{cbc}} = \frac{280}{3 \times 10}$ = 9.33
  Assume b = 250 mm
  σ_{ct} = 87.66 $\times$ 10^3 < 375 D + 9408.73
  = 208.67 < D
  Consider D = 300 mm
  Size of beam = 250 $\times$ 300 mm
  Provide minimum shear reinforcement
  8 mm Ø bar – 2 legged vertical stirrups

Sv = $\frac{0.87 \times fy \times Asv}{0.4 \times b}$ = 362.96 mm

Asv = $\left(\frac{\pi}{4} \times d^2\right) \times 2$ = 100.53 mm^2

- Spacing limit =

i. 0.75 $\times$ D = 0.75 $\times$ 300 = 225 mm

ii. 300 mm

Provide 8 mm Ø bar – 2 legged vertical stirrups @ 225 c/c

c. DESIGN OF TANK WALL :

- Maximum hoop tension at base =
  T = $\frac{r_w \times H \times D}{2} = \frac{10 \times H \times 12}{2}$ = 60 H KN/ m
  Ast = $\frac{T}{\sigma_{st}} = \frac{60 \times H}{130} \times 10^3$ = 461.54 H mm^2

Depth from top	Area required Ast (mm^2) 461.54 H	Area on each face mm^2 (230.76H)	Reinforcement provided on each face (horizontal)
1	461.54	230.76	8 mm Ø bar @ 210 mm c/c (Ast = 239 mm^2)
2	923	461.54	10 mm Ø bar @ 170 mm c/c (Ast = 462 mm^2)
3	1384.62	692.31	10 mm Øbar @ 110 mm c/c (Ast = 239 mm^2)
4	1846.16	923	12 mm Øbar @ 120 mm c/c (Ast = 942 mm^2)
5	2307.7	1153.85	12 mm Øbar @ 120 mm c/c (Ast = 935 mm^2)

- Thickness of wall =
  T = 60 $\times$ 5 = 300 KN
  σ_{ct} = 1.5
  Ag = 1000 $\times$ t
  (m -1) = 9.33 – 1 = 8.33
  $\sigma_{ct} = \frac{T}{Ag+(m-1)Ast} = \frac{87.66 \times 10^3}{1000 \times t+8.33 \times (2 \times 935)}$
  300 $\times$ 10^3 < 150.0 $\times$ t + 23365.65

t > 184.42
Provide (t = 250 mm) at base and 200 mm at top
Avg = thickness of wall = $\frac{200+250}{2}$ = 225 mm

- Distribution steel =
Base = $\frac{H}{3} = \frac{5}{3}$ = 1.67 m
Cantilever moment (m) = $\frac{\gamma_w \times H \times \frac{H}{3}}{6} = \frac{10 \times 5 \times 2.18}{6}$ = 23.16 KN. m
Ast for moment = $\frac{M}{\sigma_{ct} \times j \times d} = \frac{23.16 \times 10^3}{130 \times 0.867 \times 175}$ = 1174.18 mm^2
Provide 20 mm Ø bar @ 260 mm c/c (Ast = 1200 mm^2)

d. DESIGN OF BOTTOM DOME:

- Thickness of dome slab assumed = 100 mm
- Diameter of tank = 12 m
- Central rise = h_2 = 2 m
- Radius of dome = R_2 = 10 m
- Self weight of dome slab = 2 $\pi \times h_2 \times R_2 \times$ 0.1 $\times$ 25 = 314.159 KN
- Volume = V_1 - V_2 =565.48 – 117.28 = 448.2 m^3
- Weight of water = 448.2 $\times$ 10 = 4482 KN
- Total load on dome = 314.159 + 4482 = 4796.159 KN
- Load / unit area =
w = $\frac{4796.159}{\frac{\pi}{4} \times 12^2}$ = 42.407 KN/ m^2
- Meridonal thrust =
$T_1 = \frac{w \times R_2}{1+\cos\theta} = \frac{42.207 \times 10}{1+\cos 36.86}$ = 235.58 KN/ m
θ = 36.86 °
- Meridonal stress = $\frac{235.58 \times 10^3}{100 \times 1000}$ = 2.3558 N/ mm^2
- Circumferential force =
w $\times$ R $\left(\cos\theta - \frac{1}{1+\cos\theta}\right)$ = 42.407 $\times$ 10 $\left(\cos\theta - \frac{1}{1+\cos\theta}\right)$ = 103.71 KN/m
- Hoop stress = $\frac{103.71 \times 10^3}{100 \times 1000}$ = 1.037 N/ mm^2
Provide nominal reinforcement of 0.3 %
Ast = $\frac{0.3 \times 100 \times 1000}{100}$ = 300 mm^2
Assume 8 mm diameter of bar
Spacing = $\frac{100}{\frac{300}{\frac{\pi}{4} \times 8^2}}$ = 167.55 mm
Provide 8 mm Ø bar @ 160 mm c/c circumferentially and along the meridians.

e. DESIGN OF BOTTOM RING BEAM:

- Loads on ring beam =

a) Load due to top dome = Meridonal thrust $\times$ Sin θ
= 235.58 $\times$ Sin 36.86 = 141.31 KN/ m
b) Load due to top ring beam = 0.3 $\times$ 0.25 $\times$ 25
= 1.875 KN/ m
c) Load due to cylindrical wall = 5 $\times$ 0.1 $\times$ 25
= 12.5 KN/m
d) Self wt. of ring beam (assuming 250 $\times$ 300 mm beam)= 0.25 $\times$ 0.3 $\times$ 25 = 1.875 KN/m

Total vertical load = V_1 = 143.31 + 1.875 + 12.5 + 1.875 = 159.56 KN/m
Horizontal force = H = V_1 Cos 45 = 159.56 Cot 45 = 159.56 KN

- Hoop tension due to vertical loads =
Hg = $\frac{H \times D}{2} = \frac{159.56 \times 12}{2}$ = 957.36 KN
- Hoop tension due to water pressure =
Hw = $\frac{w \times d \times D \times h_s}{2} = \frac{10 \times 5 \times 0.3 \times 12}{2}$ = 90 KN
Total hoop tension = Hg + Hw = 957.36 + 90 = 1047.36 KN
Ast = $\frac{1047.36 \times 10^3}{150}$ = 6982.4 mm^2
Assume 18 - 22 mm Ø bar (Ast = 6842.38 mm^2)
- Maximum tensile stress =
$\frac{1047.36 \times 10^3}{(500 \times 800)+(18 \times 6911.50)}$ = 1.997 N/ mm^2
Provide a ring beam of 500 $\times$ 800 mm with 18 bars of 22 mm Ø and distribution bars of 10 mm Ø from cylindrical wall taken round the main bars as stirrups at 180 mm c/c.

f. DESIGN OF COLUMN:
The water tank having 6 columns equally spaced on a circle of 12 m diameter.

- Total load on ring beam = 159.56 KN
- Total design load on ring beam =
W = $\pi \times D \times w = \pi \times 12 \times 159.56$ = 6015.27 KN

i. Vertical load on each column
$= \frac{6015.27}{6}$ = 1002.54 KN
ii. Self wt. of column of height 12.37 m and 300 mm diameter
$= \frac{\pi}{4} \times (0.3)^2 \times 12.37 \times 25$
= 21.859 KN
iii. Self wt. of bracing (3 numbers of 3.0925 m intervals, size of bracing is 300 $\times$ 250 mm)

$= \frac{\pi}{8} \times 0.3 \times 3 \times 0.25 \times 25$

= 2.2089 KN

- Total vertical load on each column = 1002.54 + 21.85 + 2.208 = 1026.598 KN
- Wind forces on column =
  Intensity of wind pressure = 1.5 KN/ m^2
  Reduction coefficient for circular shapes = 0.7

i. Wind force on top dome and cylindrical wall
$= \left(6 + \frac{2}{2}\right) \times 0.7 \times 1.5 \times 12$ = 88.2 KN

ii. Wind force on bottom ring beam
$= 6 \times 0.7 \times 1.5 \times 0.5$ = 3.15 KN

iii. Wind force on bracing
$= 6 \times 3 \times 1.5 \times 0.3$ = 8.1 KN

Total horizontal wind force = 88.2 + 3.15 + 8.1
= 99.45 KN

- Moment at base of column is computed as =
  M = $\frac{99.45 \times 3.0925}{2}$ = 153.77 KN. m
  If M_1 = moment at the base of the column due to wind loads = (88.2$\times$ 25) + (3.15 $\times$12.37) + (6 $\times$12) + (6 $\times$6) + (6$\times$3) = 2369.965 KN. m
- Moment in each column at base = $\frac{153.77}{6}$ = 25.628 KN. m
- Design ultimate moment in each column
  Mu = (1.5$\times$ 25.628) = 38.442 KN. m
- Design ultimate axial load =
  Pu = (1.5$\times$ 1026.598) = 1539.897 KN
  Compute the parameters = $\left(\frac{Pu}{fck \times D^2}\right)$ =
  $\frac{1539.897 \times 10^6}{30 \times 300^2}$ = 0.5703
  $\left(\frac{Mu}{fck \times D^3}\right) = \frac{38.442 \times 10^6}{30 \times 300^3}$ = 0.0474
- From chart – 56 (SP-16) for circular columns with compression and bending and 8 longitudinal bars and the ratio (d'/D) = 0.10. the corresponding percentage reinforcement is read as,
  $\left(\frac{Pu}{fck}\right)$ = 0.05
  P = 0.05$\times$30 = 1.5 %
  But minimum reinforcement in column = 0.8 % providing 1.5 % reinforcement, we have
  Asv = $\left(\frac{P \times \pi \times D^2}{400}\right) = \left(\frac{1.5 \times \pi \times 300^2}{400}\right)$ = 1060.28 mm^2
  Provide 8 bars of 16 mm Ø (Asv = 1608.49 mm^2)
- Diameter of lateral ties not less than
  $= \left(\frac{1}{4} \times 16\right)$ = 4 mm
  Adopt 8 mm diameter lateral ties.
  Pitch of lateral ties shall be least of

i. Least lateral dimension = 300 mm
ii. 16 $\times$ 16 = 256 mm
iii. 300 mm

Hence adopt 8 mm Ø lateral ties at 300 mm c/c

g. DESIGN OF BRACE BEAM :

- Service moment in brace =
  M = 2 $\times$ moment in column $\times \sqrt{2}$ =
  2 $\times$ 25.628 $\times \sqrt{2}$ = 72.486 KN. m
- Design of ultimate moment =
  Mu = 1.5 $\times$ 72.486 = 108.73 KN. m
- Section of brace = 300 $\times$ 250 mm
  b = 300 mm, d =250 mm
- Limiting moment of resistance of the section is computed as =Mu lim = 0.138 $\times$ fck $\times$ bd^2 =
  $(0.138 \times 30 \times 300 \times 250^2) \times 10^{-6}$
  = 77.625 KN.
  m < Mu Hence section is under reinforced.
- Compute the parameters =
  $\left(\frac{Mu}{bd^2}\right) = \left(\frac{108.73 \times 10^6}{300 \times 250^2}\right)$ = 5.798
  Ast = $\left(\frac{1.414 \times 300 \times 250}{100}\right)$ = 1060.5 mm^2
  Provide 4 bars of 20 mm Ø (Ast = 1256.63 mm^2) both at top and bottom since wind direction is reversible.
- Length of brace = L = 2 $\times$ 4 $\times$ Sin 18.43 = 2.529 m
- Maximum service load shear force in brace is computed as =
  V = $\frac{moment\ in\ brace}{half\ length\ of\ brace} = \frac{72.486}{0.5 \times 2.529}$ = 57.32 KN
- Design ultimate shear force =
  Vu = 1.5 $\times$ 57.32 = 85.985 KN
- $\tau_v = \frac{Vu}{b \times d} = \frac{85.985 \times 10^3}{300 \times 250}$ = 1.146 N/ mm^2
  $\left(\frac{100 \times Ast}{b\ d}\right) = \left(\frac{100 \times 1256.63}{300 \times 250}\right)$ = 1.675
  τ_c = 0.788 N/ mm^2, since $\tau_v > \tau_c$ shear reinforcement are required.
- Shear force carried by concrete
  $= \tau_c$ b d = $(0.788 \times 300 \times 250) \times 10^{-3}$
  = 59.1 KN
  Balance shear force = 85.985 – 59.1 = 26.885 KN

Using 10 mm Ø 2- legged stirrups,

- Spacing =

Sv = $\frac{0.87 \times 415 \times 2 \times 79 \times 250}{26.885 \times 10^3}$ = 530.462 mm

But Sv not greater than 0.75d or 300 mm whichever is less

Hence, provide 10 mm Ø 2- legged stirrups at 300 mm c/c

h. DESIGN OF FOOTING:

- Total column load = 1002.54 KN
- Approximate weight of footing = 140 KN

  Total = 1002.54 + 140 = 1142.54 KN
- Safe bearing capacity of soil = 200 KN/ m^2
- Area of footing required = $\frac{1142.54}{200}$ = 5.7127 m^2

  Let the diameter of the footing be x meter,

  $\frac{\pi x^2}{4}$ = 5.712

  x = 2.696 m

  Provide a diameter of footing equal to 2.70 m

  Radius of footing = 1.35 m
- Net upward pressure intensity on the footing =

  $$P = \frac{1002.54 \times 10^3}{\pi \times 1.35^2} = 175099.2 \text{ N/ m}^2$$
- Depth of footing =

  BM consideration, consider the shaded area of the plan of the footing

  Distance of the centroid of the shade area from the axis of the column

  $= 0.6 \times \left(\frac{R^2 + r^2 + Rr}{R + r}\right)$

  $= 0.6 \times \left(\frac{1.35^2 + 0.15^2 + 1.35 \times 0.15}{1.35 + 0.15}\right)$ = 0.819 m
- Area shaded = $\frac{\pi}{4} \times (R^2 - r^2)$

  $= \frac{\pi}{4} \times (1.35^2 - 0.15^2)$ = 1.4137 m
- Load on the shaded area = 175099.2 × 1.4137

  = 247537.7 N
- Maximum bending moment = M

  = 247537.7 × 0.819 = 202733.4 N. m
- Breadth of shaded part at column face = $\frac{\pi \times 300}{4}$

  = 235.61 mm

  Adopting c = 10 N/ mm^2, t = 230 N/ mm^2 and equating the moment of resistance to the bending moment

  = 1.213 bd^2

  202733.4 × 10^3 = 1.213 × 235.61 × d^2

  d = 842.23 ≅ 845 mm

  Providing a clear cover of 60 mm to the lower layer of bars and providing 16 mm diameter bars.

  Effective cover to the centre of the upper layer of bars = 60+16+8 = 84 mm

  Overall depth required = 845 +84 = 929 mm

  Provide an overall depth 950 mm

  Actual effective depth = 950 – 84 = 866 mm
- Punching shear consideration =

  Safe punching shear stress = 1.2 N/ mm^2

  Punching resistance = Punching load

  $\pi \times 300 \text{ D} \times 1.2 = 175099.2$

  $\left(\frac{\pi}{4}(2.70^2 - 0.3^2)\right)$

  D = 875.49 mm

  Hence depth of the footing is governed by the bending moment consideration.

  Ast = $\frac{2027733.4 \times 10^3}{230 \times 0.87 \times 866}$ = 1169.931 mm^2

  Provide 8 mm bars of 16 mm Ø (Ast = 1608.48 mm^2)

  The reinforcement to the above extent should be provided in two principle directions and in a width equal to the side of square in scribed in the plan of the footing length of the side of the inscribed square.

  = R $\sqrt{2}$ = 1.35 $\sqrt{2}$ × 1000 = 1909.1 mm ≅ 2000 mm
- Check for shear =

  The critical section for shear is considered at a distance equal to the effective depth from the face of the column. distance of 866 mm from the face of the column.

  Let the depth of the footing be reduced to 300 mm at the ends.

  Overall depth at critical section

  $= 950 - \left(\frac{950 - 300}{1375}\right) \times 866$ = 541 mm

  Effective depth at the critical section = 541 − 84 = 457 mm

  Radius of the critical circle = 150 + 866 = 1016 mm
- Nominal shear stress at the critical section

  $= \frac{175099.2\ \pi\ (1.35^2 - 1.016^2)}{2\ \pi \times 1016 \times 457}$ = 0.149 N/ mm^2
- Percentage of steel provided at the critical section

  $= \frac{8 \times 201}{\pi \times \frac{1016}{2} \times 457} \times 100$ = 0.220 %

For 0.220 % of steel

τ_c = 0.37 N/ mm^2

$\tau_v < \tau_c$

5. RESULTS

A. Results of LSM method

- Number of columns = 6
- Type of foundation = square footing
- Diameter of tank = 12 m
- Load on top dome = 4 KN/ m^2
- Load due to ring beam B_1 = 1.5 KN/m^2
- Load due to tank wall = 36.619 KN/m^2
- Load of bottom ring beam B_2 = 149.53 KN/m^2
- Load of bottom dome = 37.48 KN/m^2
- Load on each column = 939.32 KN
- Diameter of column = 300 mm
- Total height of structure = 20.35 m
- Height of staircase = 20.35 m (up to top dome)
- Number of steps in staircase = 81

B. Results of WSM method

- Number of columns = 6
- Type of foundation = circular footing
- Diameter of tank = 12 m
- Load on top dome = 4 KN/ m^2
- Load due to ring beam B_1 = 1.875 KN/m^2
- Load due to tank wall = 12.5 KN/m^2
- Load of bottom ring beam B_2 = 159.56 KN/m^2
- Load of bottom dome = 4796.159 KN/m^2
- Load on each column = 1026.598 KN
- Diameter of column = 300 mm
- Total height of structure = 20.35 m

6. COMPARISION

SR. NO	PARTICULAR	LSM METHOD	WSM METHOD	REMARK
1.	TOP DOME	Ast = 0.35 % = 350 mm^2 8 mm Ø bar @ 200 mm c/c	Ast = 0.24 % = 240 mm^2 8 mm Ø bar @ 200 mm c/c	Ast Provided in LSM is more than WSM.
2.	TOP RING BEAM	Size = 200 X 300 mm 6 bars – 12 mm Ø Stirrups = 8 mm Ø bar – 2 legged @ 200 mm c/c	Size = 250 X 300 mm 6 bars – 12 mm Ø Stirrups = 8 mm Ø bar – 2 legged @ 225 mm c/c	Area of WSM is more than LSM. WSM required more spacing than LSM.
3.	VERTICAL WALL	Thickness = 100 mm Ast = 0.35 % = 350 mm^2 Hoop bars = 20 mm Ø bar @ 130 mm c/c Vertical bars = 8 mm Ø bar @ 140 mm c/c	Thickness = 175 mm At 1 m depth from top = Ast = 239 mm^2 8 mm Ø bar @ 210 mm c/c At 2 m depth from top = Ast = 462 mm^2 10 mm Ø bar @ 170 mm c/c At 3 m depth from top = Ast = 714 mm^2 10 mm Ø bar @ 110 mm c/c At 4 m depth from top = Ast = 942 mm^2 12 mm Ø bar @ 120 mm c/c At 5 m depth from top =	Thickness of wall is more in WSM than LSM. Ast provided in WSM is more than LSM. Bars in WSM is more than LSM

			Ast = 935 mm^2 12 mm Ø bar @ 120 mm c/c	
4.	BOTTOM DOME 10 mm thickness	Ast = 0.35 % = 350 mm^2 8 mm Ø bar @ 140 mm c/c Circumferentially and along the meridians.	Ast = 0.3 % = 300 mm^2 8 mm Ø bar @ 160 mm c/c Circumferentially and along the meridians.	WSM required more spacing than LSM.
5.	BOTTOM RING BEAM (500 X 800 mm)	Ast = 6490 mm^2 Longitudinal reinforcement 6 bars – 22 mm Ø And at vertical sides 4 bars – 12 mm Ø bar And To hold stirrups 2 bars – 12 mm Ø bar Stirrups = 10 mm Ø bar @ 260 mm c/c	Ast = 6842.38 mm^2 Distribution bars = 18 bars – 22 mm Ø Stirrups = 10 mm Ø bar @ 180 mm c/c	Ast provided in WSM is more than LSM. Bars in WSM are more than LSM.
6.	COLUMN6 columns of 300 mm diameter	Ast = 2513.275 mm^2 Longitudinal steel = 8 bars – 20 mm Ø bar Lateral ties = 10 mm Ø bar @ 300 mm c/c	Ast = 1608.49 mm^2 Longitudinal steel = 8 bars – 16 mm Ø bar Lateral ties = 8 mm Ø bar @ 300 mm c/c	Ast provided in LSM is more than WSM. Bars in LSM are more than WSM.
7.	BRACE BEAM	Size = 300 X 300 mm Ast = 1350.62 mm^2 Longitudinal steel = 4 bars – 25 mm Ø bar Stirrups = 8 mm Ø bar – 2 legged @ 300 mm c/c	Size = 300 X 250 mm Ast = 1060.50 mm^2 Longitudinal steel = 4 bars – 20 mm Ø bar Stirrups = 10 mm Ø bar – 2 legged @ 300 mm c/c	Area of LSM is more than WSM. Bars in LSM are more than WSM.

7. CONCLUSIONS

- Population forecasting has been calculated which helped us, to know about the population in village area and further helped in design the tank.
- 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.

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