DESIGN OF R.C.C. OVER HEAD TANK

MAIN PROJECT REPORT

Submitted in partial fulfilment of the
Requirements for the award of the degree of

Bachelor of Technology
In
Civil Engineering

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2012

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ABSTRACT

Storage reservoirs and overhead tank are used to store water, liquid petroleum, petroleum products and similar liquids. The force analysis of the reservoirs or tanks is about the same irrespective of the chemical nature of the product. All tanks are designed as crack free structures to eliminate any leakage.

This project gives in brief, the theory behind the design of liquid retaining structure (Elevated circular water tank with domed roof and conical base) using working stress method. Elements are design in limit state method.

ACKNOWLEDGEMENTS

I would like to express my gratitude to all the people behind the screen who helped me to transform an idea into a real application.

I would like to express my heart-felt gratitude to my parents without whom I would not have been privileged to achieve and fulfill my dreams. I am grateful to our principal,
Mr. Jandyala N.Murthi who most ably run the institution and has had the major hand in enabling me to do my project.

I profoundly thank **Mr. G.Venkataramana**, Head of the Department of CIVIL Engineering who has been an excellent guide and also a great source of inspiration to my work.

I would like to thank my internal guide **Mr. V.Mallikarjun reddy** Asst.Professor for his technical guidance, constant encouragement and support in carrying out my project at college.

The satisfaction and euphoria that accompany the successful completion of the task would be great but incomplete without the mention of the people who made it possible with their constant guidance and encouragement crowns all the efforts with success. In this context, I would like thank all the other staff members, both teaching and non-teaching, who have extended their timely help and eased my task.

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SYMBOLS

A = Total area of section

A_b = Equivalent area of helical reinforcement.

A_c = Equivalent area of section

A_h = Area of concrete core.

A_m = Area of steel or iron core.

A_{sc} = Area of longitudinal reinforcement (comp.)

A_{st} = Area of steel (tensile.)

A_l = Area of longitudinal torsional reinforcement.

A_{sv}= Total cross-sectional are of stirrup legs or bent up bars within distance S_v

A_w =Area of web reinforcement.

A_Φ= Area of cross –section of one bars.

a = lever arm.

a_c = Area of concrete.

B =flange width of T-beam.

b = width.

b_r =width of rib.

C =compressive force.

c = compressive stress in concrete.

c'= stress in concrete surrounding compressive steel.

D = depth

d = effective depth

d_c = cover to compressive steel

d_s= depth of slab

d_t= cover to tensile steel

e = eccentricity.

= compressive steel depth factor ($=d_c/d$).

F =shear force characteristic load.

F_d= design load

F_r = radial shear force.

f= stress (in general)

f_{ck}= characteristic compressive stress.

F_y= characteristic strength of steel.

H = height.

I = moment of inertia.

I_e=equivalent moment of intertia of stress.

j= lever arm factor.

K_a=coefficient of active earth pressure.

K_p=coefficient of passive earth pressure.

k = neutral axis depth factor (n/d).

L=length.

L_d=devolopment length.

l = effective length of column; length; bond length.

M = bending moment; moment.

M_r=moment of resistance; radial bending moment.

M_t=torsional moment.

M_u= bending moment (limit state design)

M_θ=circumferential bending moment

m = modular ratio.

n = depth of neutral axis.

n_c=depth of critical neutral axis.

P_a=active earth pressure.

P_p= passive earth pressure.

P_u= axial load on the member(limit state design).

P = percentage steel.

P'= reinforcement ratio.

P_a=active earth pressure indencity.

P_e=net upward soil pressure.

Q= shear resistance.

q = shear stress due to bending.

q'=shear stress due to torsioN

R= radius.

s= spacing of bars.

s_a= average bond stress.

s_b= local bond stress.

T=tensile force.

T_u= torsional moment.

t= tensile stress in steel.

t_c= compressive stress in compressive steel.

V_u= shear force due to design load.

V_{us}=strength of shear reinforcement.

W= point load.

X= coordinate.

x_u= depth of neutral axis.

Z= distance.

α = inclination.

β = surcharge angle.

γ = unit weight of soil

γ_f=partial safety factor appropriate to the loading.

γ_m= partial safety factor appropriate to the material.

σ_{cc} = permissible stress in concrete.

σ_{cbc}= permissible compressive stress in concrete due to bending.

σ_{sc}= permissible compressive stress in bars.

σ_{st}= permissible stress in steel in tension.

σ_{st}= permissible tensile strss in shear reinforcement.

σ_{sy}= yield point compressive stress in steel.

μ = co efficient of friction.

INTRODUCTION

A water tank is used to store water to tide over the daily requirement. In the construction of concrete structure for the storage of water and other liquids the imperviousness of concrete is most essential .The permeability of any uniform and thoroughly compacted concrete of given mix proportions is mainly dependent on water cement ratio .The increase in water cement ratio results in 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. Design of liquid retaining structure has to be based on the avoidance of cracking in the concrete having regard to its tensile strength. Cracks can be prevented by avoiding the use of thick timber shuttering which prevent the easy escape of heat of hydration from the concrete mass .the risk of cracking can also be minimized by reducing the restraints on free expansion or contraction of the structure.

1.1 OBJECTIVE

1. To make a study about the analysis and design of water tanks.

2. To make a study about the guidelines for the design of liquid retaining structure according to IS Code.

3. To know about the design philosophy for the safe and economical design of water tank.

4. To develop programs for the design of water tank of flexible base and rigid base and the underground tank to avoid the tedious calculations.

5. In the end, the programs are validated with the results of manual calculation given in . Concrete Structure.

2.1SOURCES OF WATER SUPPLY :

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

Surface sources, such as

1. Ponds and lakes;
2. Streams and rivers;
3. Storage reservoirs; and
4. Oceans, generally not used for water supplies, at present.

Sub-surface sources or underground sources, such as

1. Springs;
2. Infiltration wells ; and
3. Wells and Tube-wells.

3.1Water Quantity Estimation

The quantity of water required for municipal uses for which the water supply scheme has to be designed requires following data:

Water consumption rate (Per Capita Demand in litres per day per head)

Population to be served.

Quantity= Per demand x Population

3.2 Water Consumption Rate

It is very difficult to precisely assess the quantity of water demanded by the public, since there are many variable factors affecting water consumption. The various types of water demands, which a city may have, may be broken into following class

Water Consumption for Various Purposes:

	Types of Consumption	Normal Range (lit/capita/day)	Average	%
1	Domestic Consumption	65-300	160	35
2	Industrial and Commercial Demand	45-450	135	30
3	Public including Fire Demand Uses	20-90	45	10
4	Losses and Waste	45-150	62	25

3.3 Fire Fighting Demand:

The per capita fire demand is very less on an average basis but the rate at which the water is required is very large. The rate of fire demand is sometimes treated as a function of population and is worked out from following empirical formulae:

	Authority	Formulae (P in thousand)	Q for 1 lakh Population)
1	American Insurance Association	Q (L/min)=4637 ÖP (1-0.01 ÖP)	41760
2	Kuchling's Formula	Q (L/min)=3182 ÖP	31800
3	Freeman's Formula	Q (L/min)= 1136.5(P/5+10)	35050
4	Ministry of Urban Development Manual Formula	Q (kilo liters/d)=100 ÖP for P>50000	31623

3.4 Factors affecting per capita demand:

- **Size of the city**: Per capita demand for big cities is generally large as compared to that for smaller towns as big cities have sewered houses.
- **Presence of industries.**
- **Climatic conditions.**
- **Habits of economic status.**
- **Quality of water:** If water is aesthetically $ people and their
- medically safe, the consumption will increase as people will not resort to private wells, etc.
- **Pressure in the distribution system.**
- **Efficiency of water works administration:** Leaks in water mains and services; and un authorised use of water can be kept to a minimum by surveys.
- **Cost of water.**
- **Policy of metering and charging method:** Water tax is charged in two different ways: on the basis of meter reading and on the basis of certain fixed monthly rate.

3.5 Fluctuations in Rate of Demand:

Average Daily Per Capita Demand

= Quantity Required in 12 Months/ (365 x Population)

If this average demand is supplied at all the times, it will not be sufficient to meet the fluctuations.

- **Seasonal variation**: The demand peaks during summer. Firebreak outs are generally more in summer, increasing demand. So, there is seasonal variation .
- **Daily variation** depends on the activity. People draw out more water on Sundays and Festival days, thus increasing demand on these days.
- **Hourly variations** are very important as they have a wide range. During active household working hours i.e. from six to ten in the morning and four to eight in the evening, the bulk of the daily requirement is taken. During other hours the requirement is negligible. Moreover, if a fire breaks out, a huge quantity of water is required to be supplied during short duration, necessitating the need for a maximum rate of hourly supply.

So, an adequate quantity of water must be available to meet the peak demand. To meet all the fluctuations, the supply pipes, service reservoirs and distribution pipes must be properly proportioned. The water is supplied by pumping directly and the pumps and distribution system must be designed to meet the peak demand. The effect of monthly variation influences the design of storage reservoirs and the hourly variations influences the design of pumps and service reservoirs. As the population decreases, the fluctuation rate increases.

<u>Maximum daily demand</u> = 1.8 x average daily demand

<u>Maximum hourly demand of maximum day i.e. Peak demand</u>

= 1.5 x average hourly demand

= 1.5 x Maximum daily demand/24

= 1.5 x (1.8 x average daily demand)/24

= 2.7 x average daily demand/24

= 2.7 x annual average hourly demand

4.1Design Periods & Population Forecast

This quantity should be worked out with due provision for the estimated requirements of the future. The future period for which a provision is made in the water supply scheme is known as the design period.

Design period is estimated based on the following:

- Useful life of the component , considering obsolescence, wear, tear, etc.
- Expandability aspect.
- Anticipated rate of growth of population, including industrial, commercial developments & migration-immigration.
- Available resources.
- Performance of the system during initial period.

4.2Population Forecasting Methods

The various methods adopted for estimating future populations are given below. The particular method to be adopted for a particular case or for a particular city depends largely on the factors discussed in the methods, and the selection is left to the discrection and intelligence of the designer.

1. Incremental Increase Method
2. Decreasing Rate of Growth Method
3. Simple Graphical Method
4. Comparative Graphical Method
5. Ratio Method
6. Logistic Curve Method
7. Arithmetic Increase Method
8. Geometric Increase Method.

WATER TANKS

5.1 CLASSIFICATIONS

Classification based on under three heads:

1. Tanks resting on ground
2. Elevated tanks supported on stagging
3. Underground tanks.

Classification based on shapes

1. Circular tanks
2. Rectangular tanks
3. Spherical tanks
4. Intze tanks
5. Circular tanks with conical bottom

6.1 DESIGN REQUIREMENT OF CONCRETE (I. S. I)

In water retaining structure a dense impermeable concrete is required therefore,proportion of fine and course aggregates to cement should besuch as to give high quality concrete. Concrete mix weaker than M20 is not used. The minimum quantity of cement in the concrete mix shall be not less than 30 kN/m3.The design of the concrete mix shall be such that the resultant concrete issu efficiently impervious. Efficient compaction preferably by vibration is essential. The permeability of the thoroughly compacted concrete is dependent on water cement ratio. Increase in water cement ratio increases permeability, while concrete with low water cement ratio is difficult to compact. Other causes of leakage in concrete are defects such as segregation and honey combing. All joints should be made water-tight as these are potential sources of leakage. Design of liquid retaining structure is different from ordinary R.C.C,structures as it requires that concrete should not crack and hence tensile stresses in concrete should be within permissible limits. A reinforced concrete member of liquid retaining structure is designed on the usual principles ignoring tensile resistance of concrete in bending. Additionally it should be ensured that tensile stress on the liquid retaining ace of the equivalent concrete section does not exceed the permissible tensile strength of concrete as given in table 1. For calculation purposes the cover is also taken into concrete area. Cracking may be caused due to restraint to shrinkage, expansion and contraction of concrete due to temperature or shrinkage and swelling due to moisture effects. Such restraint may be caused by .

(i) The interaction between reinforcement and concrete during shrinkage due to drying.

(ii) The boundary conditions.

(iii) The differential conditions prevailing through the large thickness of massive concrete

Use of small size bars placed properly, leads to closer cracks but of smaller width. The risk of cracking due to temperature and shrinkage effects may be minimized by limiting the changes in moisture content and temperature to which the structure as a whole is subjected. The risk ofcracking can also be minimized by reducing the restraint on the free expansion of the structure with long walls or slab founded at or below ground level, restraint can be minimized by the provision of a sliding layer. This can be provided by founding the structure on a flat layer of concrete with interposition of some material to break the bond and facilitate movement.In case length of structure is large it should be subdivided into suitable lengths separated by movement joints, especially where sections are changed the movement joints should be provided.Where structures have to store hot liquids, stresses caused by difference in temperature between inside and outside of the reservoir should be taken into account.

The coefficient of expansion due to temperature change is taken as 11 x 10-6 /° C and coefficient of shrinkage may be taken as 450 x 10-6 for initial shrinkage and 200 x 10-6 for drying shrinkage.

6.JOINTS IN LIQUID RETAINING STRUCTURES

6.1 MOVEMENT JOINTS. There are three types of movement joints.

(i)*Contraction Joint.* It is a movement joint with deliberate discontinuity

without initial gap between the concrete on either side of the joint. The

purpose of this joint is to accommodate contraction of the concrete.

The joint is shown in Fig. (a)

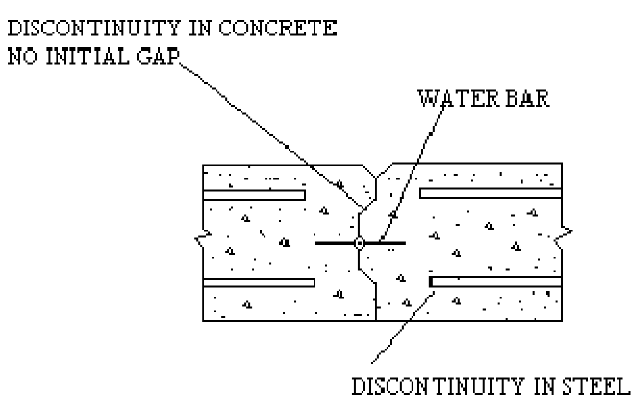


Fig (a)

A contraction joint may be either complete contraction joint or partial contraction joint. A complete contraction joint is one in which both steel and concrete are interrupted and a partial contraction joint is one in which only the concrete is interrupted, the reinforcing steel running through as shown in Fig.(b). Figure

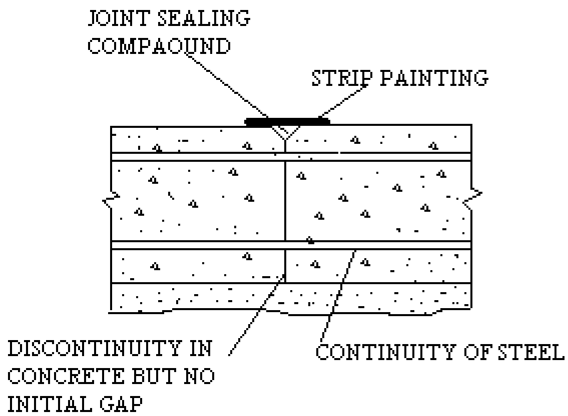
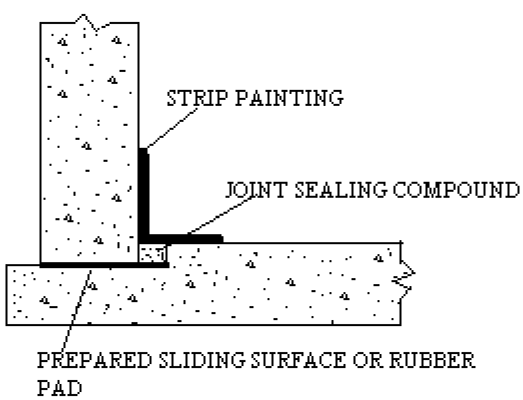


Fig (b)

(ii)***Expansion Joint.*** It is a joint with complete discontinuity in both reinforcing steel and concrete and it is to accommodate either expansion or contraction of the structure. A typical expansion joint is shown in Fig.(c)



Fig©

This type of joint is provided between wall and floor in some cylindrical tank designs.

6.2 CONTRACTION JOINTS

This type of joint is provided for convenience in construction. This type of joint requires the provision of an initial gap between theadjoining parts of a structure which by closing or opening accommodates the expansion or contraction of the structure.

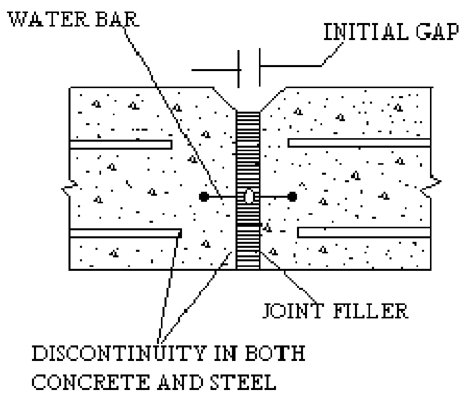


Fig (d)

(iii) ***Sliding Joint***. It is a joint with complete discontinuity in both reinforcement and concrete and with special provision to facilitatemovement in plane of the joint. A typical joint is shown in Fig. This type of joint is provided between wall and floor in some cylindrical tank designs.

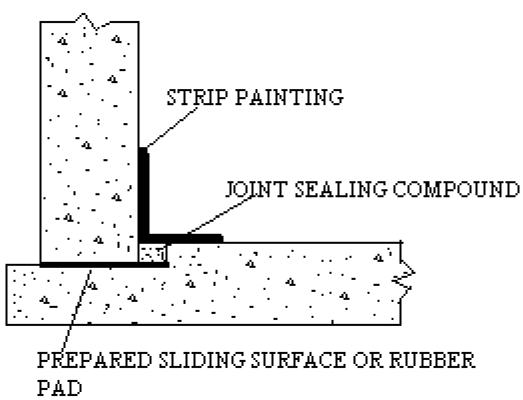


Fig (e)

6.3CONTRACTION JOINTS

This type of joint is provided for convenience in construction. Arrangement is made to achieve subsequent continuity without relative movement. One application of these joints is between successive lifts in a reservoir wall. A typical joint is shown in Fig(f)

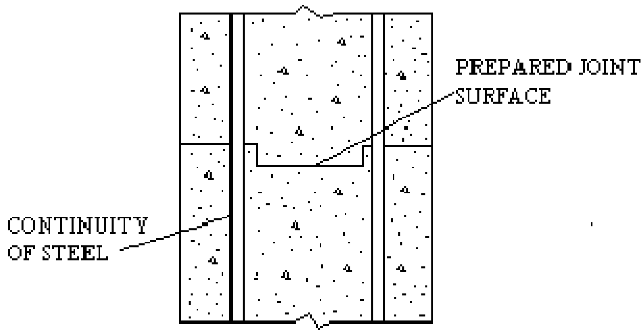


Fig (f)

6.4 TEMPORARY JOINTS

A gap is sometimes left temporarily between the concrete of adjoining parts of a structure which after a suitable interval and before the structure is put to use, is filled with mortar or concrete completely as in Fig.3.5(a) or as shown in Fig.3.5 (b) and (c) with suitable jointing materials. In the first case width of the gap should be sufficient to allow the sides to be prepared before filling.Figure (g)

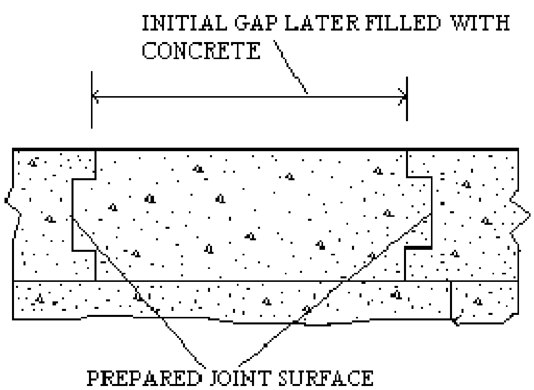


Fig (g)

7 GENERAL DESIGN REQUIREMENTS (I.S.I)

7.1 Plain Concrete Structures. Plain concrete member of reinforced concrete liquid retaining structure may be designed against structural failure by allowing tension in plain concrete as per the permissible limits for tension in bending. This will automatically take care of failure due to cracking. However, nominal reinforcement shall be provided, for plain concrete structural members.

7.2. Permissible Stresses in Concrete.

(a) For resistance to cracking. For calculations relating to the resistance of members to cracking, the permissible stresses in tension (direct and due to bending) and shear shall confirm to the values specified in Table 1.The permissible tensile stresses due to bending apply to the face of the member in contact with the liquid. In members less than 225mm. thick and in contact with liquid on one side these permissible stresses in bending apply also to the face remote from the liquid.

(b) For strength calculations. In strength calculations the permissible concrete stresses shall be in accordance with Table 1. Where the calculated shear stress in concrete alone exceeds the permissible value, reinforcement acting in conjunction with diagonal compression in the concrete shall be provided to take the whole of the shear.

7.3Permissible Stresses in Steel

(a) For resistance to cracking. When steel and concrete are assumed to act together for checking the tensile stress in concrete for avoidance of crack, the tensile stress in steel will be limited by the requirement that the permissible tensile stress in the concrete is not exceeded so the tensile stress in steel shall be equal to the product of modular ratio of steel and concrete, and the corresponding allowable tensile stress in concrete.

(b) For strength calculations.

In strength calculations the permissible stress shall be as follows:

(i) Tensile stress in member in direct tension 1000 kg/cm2

(ii) Tensile stress in member in bending on

liquid retaining face of members or face away from

liquid for members less than 225mm thick 1000 kg/cm2

(iii)On face away from liquid for members 225mm or more

in thickness 1250 kg/cm2

(iv)Tensile stress in shear reinforcement,

For members less than 225mm thickness 1000 kg/cm2

For members 225mm or more in thickness 1250 kg/cm2

(v)Compressive stress in columns subjected to direct load 1250 kg/cm2

7.4 Stresses due to drying Shrinkage or Temperature Change.

(i)Stresses due to drying shrinkage or temperature change may be ignored provided that .

(a) The permissible stresses specified above in (ii) and (iii) are not otherwise exceeded.

(b) Adequate precautions are taken to avoid cracking of concrete during the construction period and until the reservoir is put into use.

(c) Recommendation regarding joints given in article 8.3 and for suitable sliding layer beneath the reservoir are complied with, or the reservoir is to be used only for the storage of water or aqueous liquids at or near ambient temperature and the circumstances are such that the concrete will never dry out.

(ii)Shrinkage stresses may however be required to be calculated in special cases, when a shrinkage co-efficient of 300 x 10-6may be assumed.

(iii) When the shrinkage stresses are allowed, the permissible stresses,tensile stresses to concrete (direct and bending) as given in Table 1 may be increased by 33.33 per cent.

7.5 Floors

(i)Provision of movement joints.

Movement joints should be provided as discussed in article 3.

(ii) Floors of tanks resting on ground.

If the tank is resting directly over ground, floor may be constructed of concrete with nominal percentage of reinforcement provided that it is certain that the ground will carry the load without appreciable subsidence in any part and that the concrete floor is cast in panels with sides not more than 4.5m. with contraction or expansion joints between. In such cases a screed or concrete layer less than 75mm thick shall first be placed on the ground and covered with a sliding layer of bitumen paper or other suitable material to destroy the bond between the screed and floor concrete. In normal circumstances the screed layer shall be of grade not weaker than M 10,where injurious soils or aggressive water are expected, the screed layer shall be of grade not weaker than M 15 and if necessary a sulphate resisting or other special cement should be used.

(iii) Floor of tanks resting on supports

(a) If the tank is supported on walls or other similar supports the floor slab shall be designed as floor in buildings for bending moments due to water load and self weight.

(b)When the floor is rigidly connected to the walls (as is generally the case) the bending moments at the junction between the walls and floors shall be taken into account in the design of floor together with any direct forces transferred to the floor from the walls or from the floor to the wall due to suspension of the floor from the wall.If the walls are non-monolithic with the floor slab, such as in cases, where movement joints have been provided between the floor slabs and walls, the floor shall be designed only for the vertical loads on the floor.

(c) In continuous T-beams and L-beams with ribs on the side remote from the liquid, the tension in concrete on the liquid side at the face of the supports shall not exceed the permissible stresses for controlling cracks in concrete. The width of the slab shall be determined in usual manner for calculation of the resistance to cracking of T-beam, L-beam sections at supports.

(d)The floor slab may be suitably tied to the walls by rods properly embedded in both the slab and the walls. In such cases no separate beam (curved or straight) is necessary under the wall, provided the wall of the tank itself is designed to act as a beam over the supports under it.

(e)Sometimes it may be economical to provide the floors of circular tanks, in the shape of dome. In such cases the dome shall be designed for the vertical loads of the liquid over it and the ratio of its rise to its diameter shall be so adjusted that the stresses in the dome are, as far as possible, wholly compressive. The dome shall be supported at its bottom on the ring beam which shall be designed for resultant circumferential tension in addition to vertical loads.

7.6 Walls

(i)Provision of joints

(a)Where it is desired to allow the walls to expand or contract separately from the floor, or to prevent moments at the base of the wall owing to fixity to the floor, sliding joints may be employed.

(b)The spacing of vertical movement joints should be as discussed in article 3.3 while the majority of these joints may be of the partial or complete contraction type, sufficient joints of the expansion type should be provided to satisfy the requirements given in article

(ii)Pressure on Walls.

(a) In liquid retaining structures with fixed or floating covers the gas pressure developed above liquid surface shall be added to the liquid pressure.

(b)When the wall of liquid retaining structure is built in ground, or has earth embanked against it, the effect of earth pressure shall be taken into account.

(iii) Walls or Tanks Rectangular or Polygonal in Plan.

While designing the walls of rectangular or polygonal concrete tanks, the following points should be borne in mind.

(a) In plane walls, the liquid pressure is resisted by both vertical and horizontal bending moments. An estimate should be made of the proportion of the pressure resisted by bending moments in the vertical and horizontal planes. The direct horizontal tension caused by the direct pull due to water pressure on the end walls, should be added to that resulting from horizontal bending moments. On liquid retaining faces, the tensile stresses due to the combination of direct horizontal tension and bending action shall satisfy the following condition:

$(t. / t) + (óct. / óct) \leq 1$

t. = calculated direct tensile stress in concrete
t = permissible direct tensile stress in concrete (Table 1)
óc t = calculated tensile stress due to bending in concrete.
ó*c t* = permissible tensile stress due to bending in concrete.

(d)At the vertical edges where the walls of a reservoir are rigidly joined, horizontal reinforcement and haunch bars should be provided to resist the horizontal bending moments even if the walls are designed to withstand the whole load as vertical beams or cantilever without lateral supports.

(c) In the case of rectangular or polygonal tanks, the side walls act as twoway slabs, whereby the wall is continued or restrained in the horizontal direction, fixed or hinged at the bottom and hinged or free at the top. The walls thus act as thin plates subjected triangular loading and with boundary conditions varying between full restraint and free edge. The analysis of moment and forces may be made on the basis of any recognized method.

(iv) Walls of Cylindrical Tanks.

While designing walls of cylindrical tanks the following points should be borne in mind:

(a)Walls of cylindrical tanks are either cast monolithically with the base or are set in grooves and key ways (movement joints). In either case deformation of wall under influence of liquid pressure is restricted at and above the base. Consequently, only part of the triangular hydrostatic load will be carried by ring tension and part of the load at bottom will be supported by cantilever action.

(b)It is difficult to restrict rotation or settlement of the base slab and it is advisable to provide vertical reinforcement as if the walls were fully fixed at the base, in addition to the reinforcement required to resist horizontal ring tension for hinged at base, conditions of walls, unless the appropriate amount of fixity at the base is established by analysis with due consideration to the dimensions of the base slab the type of joint between the wall and slab, and , where applicable, the type of soil supporting the base slab.

7.7 Roofs

(i) Provision of Movement joints.

To avoid the possibility of sympathetic cracking it is important to ensure that movement joints in the roof correspond with those in the walls, if roof and walls are monolithic. It, however, provision is made by means of a sliding joint for movement between the roof and the wall correspondence of joints is not so important.

(ii)Loading

. Field covers of liquid retaining structures should be designed for gravity loads, such as the weight of roof slab, earth cover if any, live loads and mechanical equipment. They should also be designed for upward load if the liquid retaining structure is subjected to internal gas pressure. A superficial load sufficient to ensure safety with the unequal intensity of loading which occurs during the placing of the earth cover should be allowed for in designing roofs. The engineer should specify a loading under these temporary conditions which should not be exceeded. In designing the roof, allowance should be made for the temporary condition of some spans loaded and other spans unloaded, even though in the final state the load may be small and evenly distributed.

(iii)Water tightness. In case of tanks intended for the storage of water for domestic purpose, the roof must be made water-tight. This may be achieved by limiting the stresses as for the rest of the tank, or by the use of the covering of the waterproof membrane or by providing slopes to ensure adequate drainage.

(iv) Protection against corrosion. Protection measure shall be provided to the underside of the roof to prevent it from corrosion due to condensation.

7.8 Minimum Reinforcement

(a)The minimum reinforcement in walls, floors and roofs in each of two directions at right angles shall have an area of 0.3 per cent of the concrete section in that direction for sections up to 100mm, thickness. For sections of thickness greater than 100mm, and less than 450mm the minimum reinforcement in each of the two directions shall be linearly reduced from 0.3 percent for 100mm thick section to 0.2 percent for 450mm, thick sections. For sections of thickness greater than 450mm, minimum reinforcement in each of the two directions shall be kept at 0.2 per cent. In concrete sections of thickness 225mm or greater, two layers of reinforcement steel shall be placed one near each face of the section to make up the minimum reinforcement.

(b)In special circumstances floor slabs may be constructed with percentage of reinforcement less than specified above. In no case the percentage of reinforcement in any member be less than 0.15% of gross sectional area of the member.

7.9 Minimum Cover to Reinforcement.

(a)For liquid faces of parts of members either in contact with the liquid (such as inner faces or roof slab) the minimum cover to all reinforcement should be 25mm or the diameter of the main bar whichever is grater. In the presence of the sea water and soils and water of corrosive characters the cover should be increased by 12mm but this additional cover shall not be taken into account for design calculations.

(b)For faces away from liquid and for parts of the structure neither in contact with the liquid on any face, nor enclosing the space above the liquid, the cover shall be as for ordinary concrete member.

8.DOMES:

A dome may be defined as a thin shell generated by the revolution of a regular curve about one of its axes. The shape of the dome depends on the type of the curve and the direction of the axis of revolution. In spherical and conoidal domes, surface is described by revolving an arc of a circle. The centre of the circle may be on the axis of rotation (spherical dome) or outside the axis (conoidal dome). Both types may or may not have a symmetrical lantern opening through the top. The edge of the shell around its base is usually provided with edge member cast integrally with the shell.

Domes are used in variety of structures, as in the roof of circular areas, in circular tanks, in hangers, exhibition halls, auditoriums, planetorium and bottom of tanks, bins and bunkers. Domes may be constructed of masonry, steel, timber and reinforced concrete. However, reinforced domes are more common nowadays since they can be constructed over large spans

Membrane theory for analysis of shells of revolution can be developed neglecting effect of bending moment, twisting moment and shear and assuming that the loads are carried wholly by axial stresses. This however applies at points of shell which are removed some distance away from the discontinuous edge. At the edges, the results thus obtained may be indicated but are not accurate.

The edge member and the adjacent hoop of the shells must have very nearly the same strain when they are cast integrally. The significance of this fact is usually ignored and the forces thus computed are, therefore, subject to certain modifications.

Stresses in shells are usually kept fairly low, as effect of the edge disturbance, as mentioned above is usually neglected. The shell must be thick enough to allow space and protection for two layers of reinforcement. From this point of view 80 mm is considered as the minimum thickness of shell.

9.Membrane Theory of Shells of Revolution

Fig shows a typical shell of revolution, on which equilibrium of an element, obtained by intersection of meridian and latitude, is indicated. Forces along the circumference are denoted by Nφ and are called meridian stresses and forces at right angles to the meridian plane and along the latitude are horizontal and called the hoop stresses, denoted by Nθ . Neglecting variations in the magnitudes of Nφand Nθ, since they are very small.the state of stress in the element is shown in fig (b).

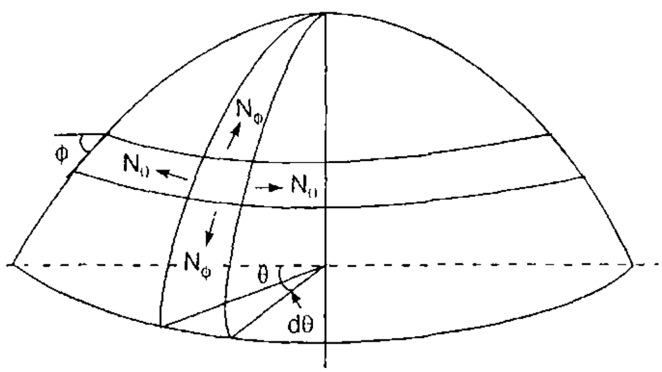


Fig (a)

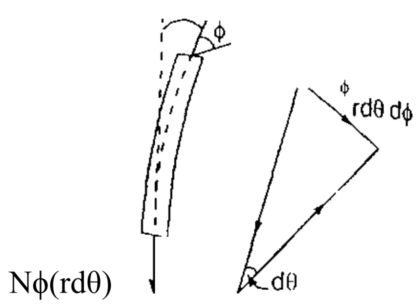
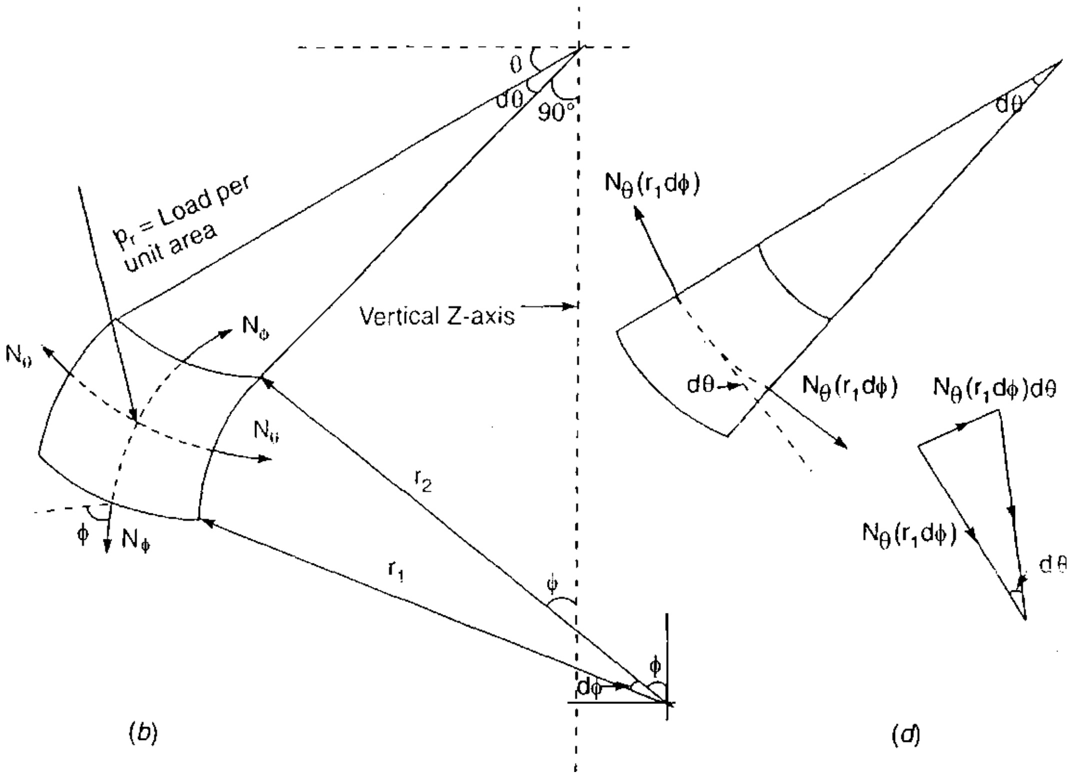


Fig (c)



Shell of Revolution.

two forces $N\varphi(rd\ \theta)$ have the resultant $N\varphi(rd\theta)d\varphi$ as shown in Fig.(c) and the resultant acts normal to the surface pointed towards the innerside. Forces $N\theta(r1d\varphi)$ again have horizontal resultant of magnitude $N\varphi(r1\ d\varphi)\ d\theta$ as shown in Fig (d). It has a component $N\varphi(r1d\varphi)d\theta\sin\varphi$ directed normally to the shell and pointing towards the inner side. These two forces and the external force normal to the surface and a magnitude$Pr(rd\theta)$ must be in equilibrium.

Thus,$N\varphi(rd\theta)d\varphi+N\theta(r1d\varphi)d\theta\sin\varphi+Pr(rd\theta)(r1d\theta)= 0.$

Combining and as $r = r2\sin\varphi$ from Fig. (a)

$N\varphi/r1 + N\theta/r2 = -Pr =$ pressure normal to the surface

In this equation pr is considered positive when acting towards the inner side and negative when acting towards the outer side of the shell. Values and $N\varphi$ and $N\theta$ will be positive when tensile and negative compressive.

The equation is valid not only for shells in the form of a surface of revolution, but may be applied to all shells, when the coordinate lines for ϕ = constant and θ = constant, are the lines of curvature of the surface

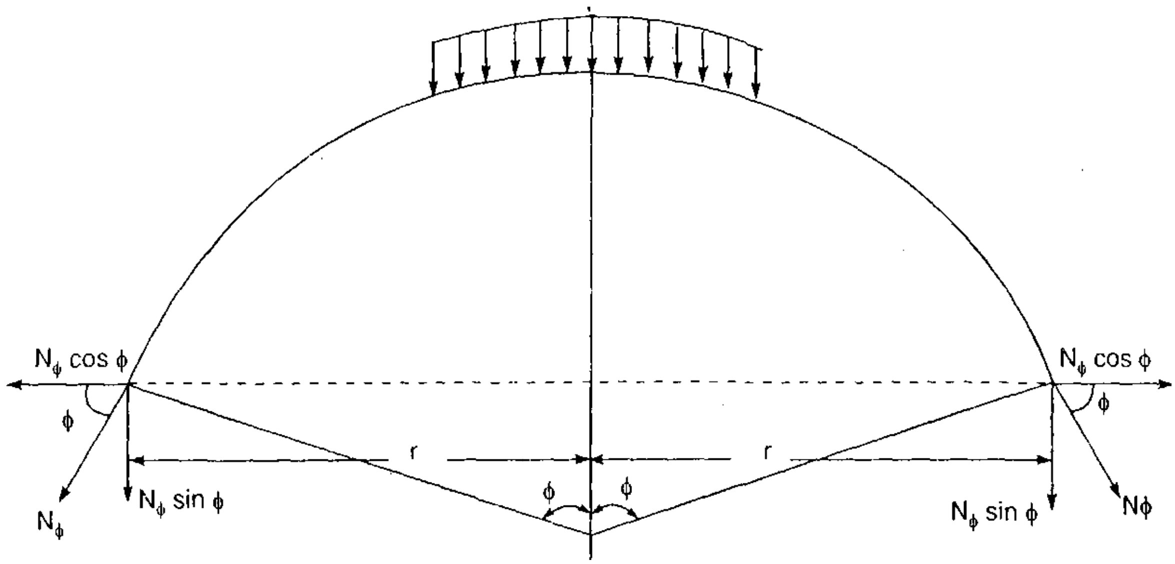


Fig. Forces in shell

Force Nφ act tangentially to the surface all around the circumference. Considering the equilibrium of a segment of shell cut along the parallel to latitude defined by the angle ϕ as shown in Fig

$2\pi r\ N\varphi \sin\varphi + W = 0$,

Where W= total load in the vertical direction on the surface of the shell above the cut.

This gives, $N\varphi = -W/2\pi r \sin\varphi$

Eq. is readily solved for Nφ and $N\theta$ may then be determined by Eq. This theory is applicable to a shell of any material as only the conditions of equilibrium have been applied and no compatibility relationships in terms of deformation have been introduced. It is, therefore, immaterial whether Hooke's law is applicable or not.

10.Water tank with spherical bottom :

Referring to the tank in Fig.(a),supported along the circumference as shown, the magnitudeof Na may be obtained from consideration of equilibrium. If it is required to obtain Na at section 1 - 1 from calculation of the total downward load, there are two possibilities. The downward load may be taken to be the weight of water and tank of the annular part i.e. W1 shown in Fig.(b)

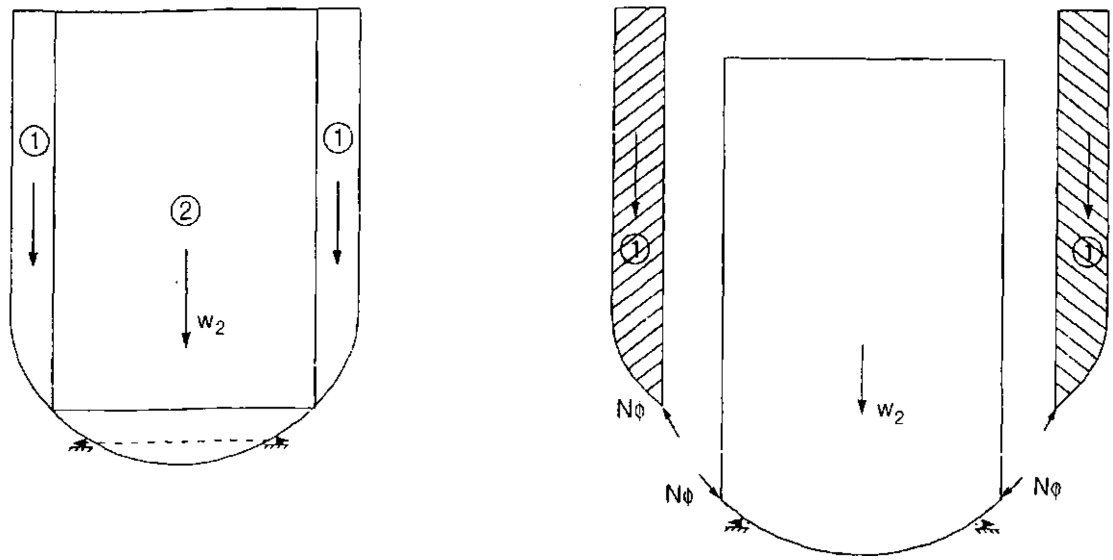


Fig. Water tank with spherical bottom.

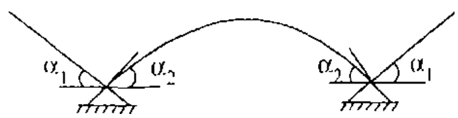


Fig intze reservoir

Alternatively, the downward load may be calculated from the weight of water and tank bottom of the part i.e W2 less upward reaction of the support as shown in Fig.

For section which cuts the tank bottom inside the support, the reaction has to be considered with the weight of water and tank of the annular part.

Similar is the case with Intze reservoir as in Fig. (a), which combines a truncated dome with a spherical segment. Pattern of the two forces $N\varphi1$and $N\varphi2$ at point A are shown in Fig(b). To eliminate horizontal forces on the supporting ring girder, it is necessary that $N \varphi1\cos \alpha1 = N\varphi2\cos \alpha2$.

11.Design of Reinforced Concrete Domes:

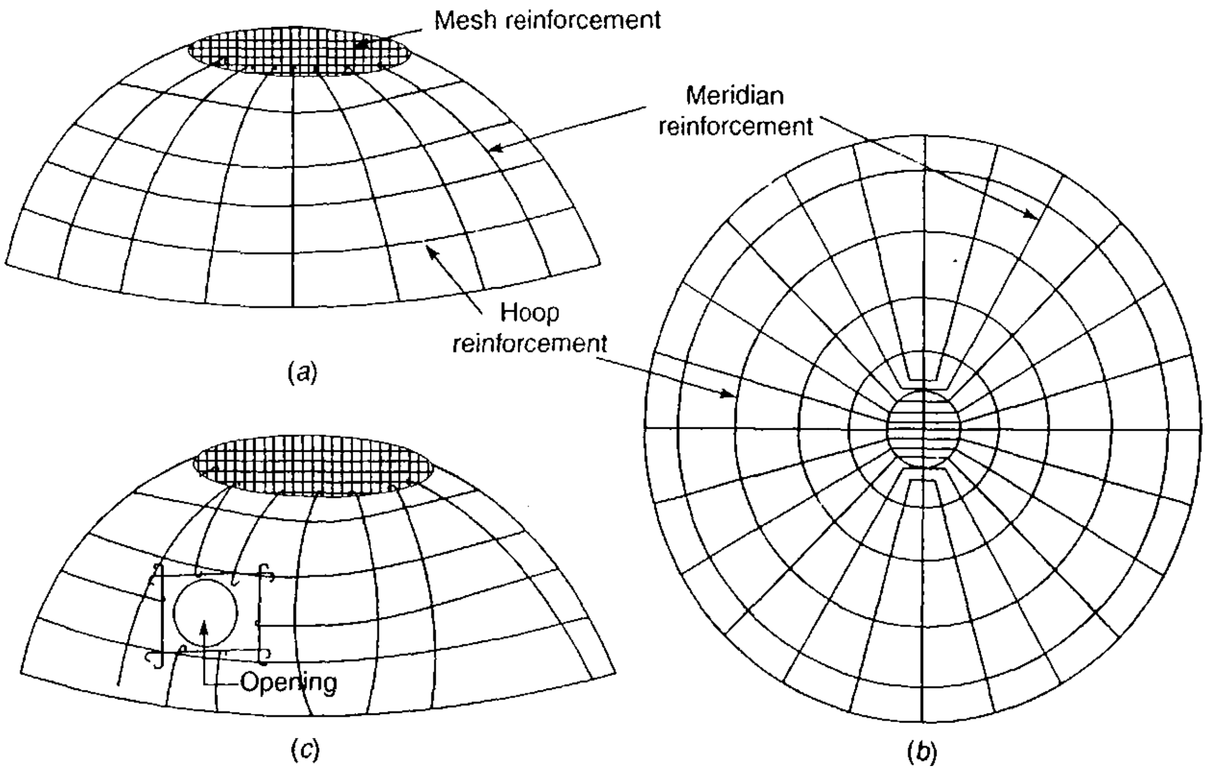
The requirements of thickness of dome and reinforcement from the point of view of induced stresses are usually very small. However, a minimum of 80 mm is provided so as to accommodate two layers of steel with adequate cover. Similarly a minimum of steel provided is 0.15% of the sectional area in each direction along the meridians as well as along the latitudes. This reinforcement will be in addition to the requirements for hoop tensile stresses.

The reinforcement is provided in the middle of the thickness of the dome shell Near the edges usually some ring beam is provided for taking the horizontal component of the meridian stress. Some bending moment develops in the shell near the edges. As shown in Fig. it is normal to thicken the shell near the edges and provide increased curvature. Reinforcements near the top as well as near the bottom face of the shell are also provided. The size of the ring beam is obtained on basis of the hoop tension developed in the ring due to the horizontal component of the meridian stress. The concrete area is obtained so that the resulting tensile stress when concrete alone is considered does not exceed 1.1 N/mm2 to 1.70 N/mm2 for direct tension and 1.5 N/mm2 to 2.40 N/mm2 for tension due to bending in liquid resisting structure depending on the grade of concrete.

Reinforcement for the hoop stress is also provided with the allowable stress in steel as 115 N/mm2 (or 150 N/mm2) in case of liquid retaining structures and 140 N/mm2 (or 190 N/ mm2) in other cases. The ring should be provided so that the central line of the shell passes through the centroid of the ring beam. Renforcement has to be provided in both the directions. If the reinforcement along the meridians is continued upto the crown, there will be congestion of steel there. Hence, from practical considerations, the reinforcement along the meridian is stopped below the crown and a separate mesh, as shown in Fig(a), is provided. Alternatively, the arrangement of the bars may be made as shown in plan in Fig.(b)

In case of domes with lantern opening with concentrated load acting there, ring beam has to be provided at the periphery of the opening. The edge beam there will, however, be subjected to hoop compression in place of hoop tension.

Openings may be provided in the dome as required from other functional or architectural requirements. However, reinforcement has to be provided all around the opening as shown in Fig. (c). The meridian and hoop reinforcement reaching the opening should be well anchored to such reinforcement.



The allowable stresss specified in IS 3370 for such tanks are as follows:

Type of stresses: Permissible stress in N/mm².	*Plain bars confirming to Grade-I of IS 432-1966.*	*High yield strength deformed bars as per IS 1786-1966 or is 1139-1966.*
(/') Tensile stress in members under direct load.		

Direct tensile stress in concrete a may be taken as 1.1 N/mm^2,1.2. N/mm^2,1.3 N/mm^2, 1.5 N/mm^2, 1.6 N/mm^2 and 1.7 N/mm^2 for M15, M20, M25, M30, M35 and M40 respectively, the value in tension due to bending *i.e., o* being 1.5 N/mm^2,1.7 N/mm^2,1.8 N/mm^2,2.0 N/mm^2,2.2 N/mm^2 and 2.4 N/mm^2.

When steel and concrete are assumed to act together for checking the tensile stress in concrete for avoidance of cracks, the tensile stress in the steel will be limited by the requirements that the stress as mentioned above should not be exceeded. The tensile stress in steel will be modular ratio multiplied by the corresponding allowable tensile stress in concrete.

Stresses due to shrinkage or temperature change may be ignored if these permissible stresses in concrete and steel are not exceeded and adequate precautions are taken to avoid cracking of concrete during construction period, until the reservoir is put into use and if it is assured that the concrete will never dry out. If it is required to calculate shrinkage stresses, a shrinkage strain of 300 $(10)^{-6}$ may be assumed.

When shrinkage stresses are considered, the permissible stresses may be increased by 33-j% .

When shrinkage stresses are considered it is necessary to check the thickness for no crack.

Minimum reinforcement of each of two directions at right angles shall have an area of 0.3% for 100 mm thick concrete to 0.2% for 450 mm thick concrete wall. In floor slabs, minimum reinforcement to be provided is 0.15%. The minimum reinforcement as specified above may be decreased by 20%), if high strength deformed bars are used.

Minimum cover to reinforcement on the liquid face is 25 mm or diameter of the bar, whichever is larger and should be increased by 12 mm for tanks for sea water or liquid of corrosive character.

12.Overhead Water Tanks And Towers

Overhead water tanks of various shapes can be used as service reservoirs, as a balancing tank in water supply schemes and for replenishing the tanks for various purposes. Reinforced concrete water towers have distinct advantages as they are not affected by climatic changes, are leak proof, provide greater rigidity and are adoptable for all shapes.

Components of a water tower consists of-

(a) Tank portion with -

(1) Roof and roof beams (if any) (2) sidewalls

(3) Floor or bottom slab (4) floor beams,includingcircular girder

(b) Staging portion, consisting of-

(5) Columns (6)Bracings and

(7)Foundations

Types of water Tanks may be –

(a) Square-open or with cover at top (b)Rectangular-open or with cover at top

(c) Circular-open or with cover at which may be flat or domed.

Among these the circular types are proposed for large capacities. Such circular tanks may have flat floors or domical floors and these are supported on circular girder.

The most common type of circular tank is the one which is called an Intze Tank. In such cases, a domed cover is provided at top with a cylindrical and conical wall at bottom. A ring beam will be required to support the domed roof. A ring beam is also provided at the junction of the cylindrical and conical walls. The conical wall and the tank floor are supported on a ring girder which is supported on a number of columns.

Usually a domed floor is shown in fig a result of which the ring girder supported on the columns will be relieved from the horizontal thrusts as the horizonal thrusts of the conical wall and the domed floor act in opposite direction.

Sometimes, a vertical hollow shaft may be provided which may be supported on the domed floor.

The design of the tank will involve the following

. (1) The dome at top usually 100 mm to 150 mm thick with reinforcement along the meridians and latitudes. The rise is usually l/5th of the span.

(2) **Ring beam supporting the dome**. The ring beam is necessary to resist the horizontal component of the thrust of the dome. The ring beam will be designed for the hoop tension induced.

(3) **Cylindrical walls :** This has to be designed for hoop tension caused due to horizontal water pressure.

(4) **Ring beam at the junction of the cylindrical walls and the conical wall.** This ring beam is provided to resist the horizontal component of the reaction of the conical wall on the cylindrical wall. The ring beam will be designed for the induced hoop tension.

(5) **Conical slab**, This will be designed for hoop tension due to water pressure. The slab will also be designed as a slab spanning between the ring beam at top and the ring girder at bottom.

(6) **Floor of the tank.** The floor may be circular or domed. This slab is supported on the ring girder.

(7) **The ring girder:** This will be designed to support the tank and its contents.The girder will be supported on columns and should be designed for resulting bending moment and Torsion.

(8) **Columns:** These are to be designed for the total load transferred to them. The columns will be braced at intervels and have to be designed for wind pressure or seismic loads whichever govern.

(9) **Foundations :** A combined footing is usuals provided for all supporting columns. When this is done it is usual to make the foundation consisting of a ring girder and a circular slab.

Suitable proportions for the Intze.

for case(1) suggested by Reynolds. Total volume $\sim 0.585D^3$

for case (2), the proportion was suggested by Grey and Total Volume is given by

V1 = π(D)^2/4 * (H) = 0.3927 D^3 for H =D/2

V2 = π.h/12(D^2+d^2+d) =0.10263 D^3

With h = D/5 and d =3/5D

V3 =(π h1)/6 ((3r)^2+(h1)^2) = 0.0179 D^3

With h1 = 3/25D and r =3/10D

Volume V =o.4693 D^3

With h1 = D/6 and r =3/10D

Volume V= $0.493\ D^3$

DESIGN

14.DETAILS OF DESIGN:

Population calculation:

Year	Population	Increase per decade.	Incremental increase.	Percentage increase.	Decrease in percentage increase.

1970	300	------	------	------	------
1980	500	200	------	66.67	------
1990	700	200	0	40.00	26.67
2000	1000	300	100	42.87	-2.87
TOTAL		**700**	**100**	**149.54**	**23.8**
AVERAGE		**233**	**50**	**49.84**	**11.9**

Population in 2010.

i. Arithmetical progression method

= 1000+233 = 1233.

ii. Geometrical progression method
= 1000 + (49.84/100) (1000)
= 1498.4

iii. Incremental increase method
= 1000 + 233 + 50
= 1283

iv. Changing increase rate method
= 1000 + (42.87 – 11.9) (1000) / 100
= 1529.7

Considering geometrical progression method = 1498.4 ≈ 1500 population.

PER CAPITA DEMAND:

=1500 X 165 =2,50,000 liters.

Design of tank:

Design of an intez tank for a capacity of 250,000lts

Assuming height of tank floor above G.L 12m

Safe bearing capacity of soil 100kn/m^2

Wind pressure as per IS875 1200N/m^2

Assuming M20 concrete

For which σcbe = 7N/mm^2 , σcc = 5N/mm^2

Direct tension σt = 5N/mm^2

Tension in bending = 1.70 N/mm^2

Modular ratio m = 13

For Steel stress,

Tensile stress in direct tension =115 N/mm^2

Tensile stress in bending on liquid face =115 N/mm^2 for t < 225 mm

and 125 N/mm^2 for / > 225 mm.

Solution : Taking the volume as 0.585 D^3 for proportion given in Fig.

D = 7.50 m. The dimension of the Tank is shown in fig.

Design of Roof Dome:

Considering a rise of 1.50 m, radius of the roof dome is given from

$$1.50(2R-1.50) = (3.75)^2$$

$$R = 5.4375\text{m}.$$

$$\cos\varphi = (5.4375 - 1.50)/5.4375$$

$$= 0.7241$$

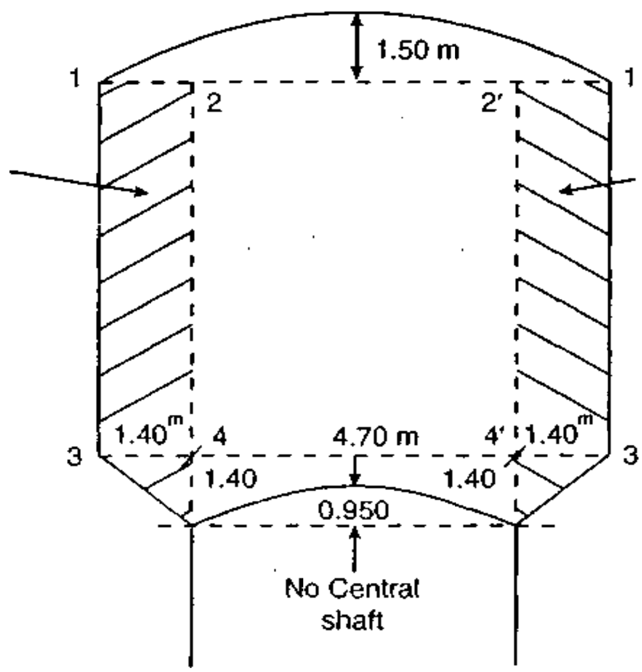
and $\varphi = 43.602 < 51.8^\circ$

Hence no tension

Assuming t = 100mm.

Self wt.=2400N/m^2

.



Fig

Equivalent of wind load, accidental loading and live load =2600 N/m^2/5000N/m^2

Meridian stress at of edge of dome

$$N\varphi = -wR/(1+\cos\varphi)$$
$$= -5000(5.4375)/(1+0.7241)$$
$$=15769.10 \text{ N}$$

And meridian stress = 15769.10/1000(100) = -0.1577N/mm^2(comp)

Maximum hoop stress at crown = -wR/2t = -5000(5.4375)/2(100)(1000)

= -0.136 N/mm^2(comp)

Use nominal reinforcement 0.3% =300mm^2

Use 8 mm bars @ 160 mm c/c. both ways.

Design of ring beam at top:

Horizontal component of Nφ = Nφcosφ

=15769.10(0.7241)

=11418.41N

Hoop tension in ring = 11418.41(7.5/2) = 42819.10N

Steel required for hoop tension = 42819.10/115

= 372.33mm^2

Use 4 Nos.12 mm bars at corners.

Area of cross section of ring beam considering concrete only

= 42819.10/1.20

= 35682.58mm^2

Use a ring beam 225mmX160mm

Area provided = 36000mm^2 > 35682.58mm^2

Use 6mm dia nominal stirrups @ 100mm c/c.

Shear stess along the edge = $N\phi \sin\phi$ =15769.10 (0.690) =10880.68N

Shear stress=10880.68/100(1000) = 0.1088N/mm^2 –very low.

Design of cylindrical wall:

Height of the wall = 5.0 m

P_r =10000(5) =50000N/m^2

$N\phi$ = 50000(7.50/2) =187500N

Area of steel required =187500/115 =1630.43mm^2

Use 12 mm bars @ 135mm c/c onboth faces.

With As = 1675.52m^2

Thickness of concrete required $\frac{187500+60\ (1675.52}{1.2}$ =240026.

Considering A = A_c+(m-1) A_s = A_c+(12) (1675.52) = A_c + 20106.24 =240026

A_c = 219919.76mm^2

Use t = 230mm at bottom, tapered to 200mm at top

Minmum A_{st} = 0.30-$\frac{0.10}{350}$ (130) = 0.2629%

Maximum A_{st} = 604.57mm^2

Hence 12mm φ @135 c/c. at both faces adequate.

Cantilever beam :

From table 1, for H^2/Dt = 14.493

Cantilever B.M. 0.0090 (wH^3) = 0.0090 (10000) $(5)^3$ = 11250Nm/m

For no crack t = 230mm adequate.

Use vertical steelAnd A_{st} =<u>11250 (1000)</u> =573.42mm^2

115(0.853)(200)

Use 12 mm ϕ vertically @150 mm c/c. as both faces.

Roof load = $(2\pi R)\ h1\ (w_{d+l}) = 2\pi R^2\ (1+\cos\phi)\ w_{d+l}$

$= 2\pi\ (5.4375)^2\ (1-0.7241)\ (5000) = 256271.38$N

Wt. of ring beams = (0.225) (0.160) (π) (7.725) (24000)

= 20968.25N

Wt. of side wall = (7.715) (0.215) (5) (25000) =625324.59N

902564.22N

Direct compressive stress = $\dfrac{902564.22}{\pi(7.715)\ (1000)\ (230)}$ = 0.162 N/mm^2

Hence ok

Design of ring beam at bottom of cylindrical wall and top of conical slab:

From top ΣW =902564.22N

Self Wt. ofring beam = 80000.00N

982564.22N

Inclined thrust at topof conical portion = $\dfrac{982564.22}{\pi(7.725)\sin\theta}$ =57256.10N/m

where θ =45˚

Horizontal component = 57256.10(cos45˚) = 40486.18N/m

Hoop tension in ring beam = 40486.18 (7.50/2) =151823.16N

Pressure of water at ring level =10000 (5) = 50000N/m

Hoop tension due to this on the ring beam

50000(7.5/2) (width of the ring beam) = 56250N

Taking width as 300mm

Total hoop tension = 151823.16+56250 =208073.16

Steel required = 208073.16/115 = 1809.33mm^2

Use 6 nos. 20 mm Øbars

$A_c + (m-1)A_{st} = A_c + 12(1884.96)$

Hence A_c = 208073.16-12(1884.33) = 154551mm^2

1.20

Use ring size 520mmX300mm

Use 12 mm 2 legged stirrups in ring beam @ 150c/c.

Design of conical slab:

Wt of side wall and dome =902564.22N

Wt of conical bottom assuming

250 mm thick $=\frac{\pi 97.50+4.70)}{2}(1.40\sqrt{2})$

250/1000(24000) = 227653.345N

Wt. of water on dashed part in fig

$\pi/4((7.50)^2-(4.70)^2)(5.00)(10000)+\pi(1.40)/12$

$=((7.50)^2+(4.70)^2+(7.50)(4.70))(10000)-\pi/4(4.70)^2(1.40)(10000)$

$=\underline{1514896.92N}$

2645114.48N

Meridian force $N\phi = \frac{2645114.48\sqrt{2}}{\pi(4.70)} = 253344.65N/m$

compressive stress $= \frac{253344.65}{(1000)(250)} = 1.013\ N/mm^2$

Hoop tension $N\theta$

Diameter of conical dome at ht 'h'

Above base $= 4.70 + \frac{7.50-4.70}{1.40}(h) = 4.70+2h$

Intensity of water pressure there $= (5+1.40-h)(10000) = 64000-10000h$

Self Wt. $=250/1000 * 24000 = 6000N/m^2$

$N\theta = ((6.40-h)10000 + 6000/\sqrt{2})\left(\frac{4.70+2h}{2}\right)\sqrt{2}$

$=(10000\sqrt{2}(6.40-h)+6000)\left(\frac{4.70+2h}{2}\right)$

$N\theta$ from $N\varphi/r1 + N\theta/r2 = -P_r$

r1 =infinity $r2 = \left(\frac{4.70+2h}{2}\right)$

At h =0 ,$N\theta$ = 2226797.72N

h = 0.70 $N\theta$ =264161.028N

$h = 1.40\ N\theta = 287665.05N$

maximum $A_{st} = \frac{287665.05}{115} = 2501.44 mm^2$

Use 16 mm Øbars on each face @ 160 mm c/c.

Checking thickness,

$$A_{eq} = A_c + (m-1)A_{st} = A_c + 12(2513.27)$$

$$\text{Thickness reqd.} = 1/1000\left[\frac{287665.05}{1.20} - 12(2513.27)\right] \sim 210mm$$

Hence t=250mm in adequate

Design as inclined slab of conical Dome:

Total *W* on top of Inclined slab = 2645114.48 *N*- Wt. of side wall and done.

= 2645114.48-902564.22 = 1742550.26N

Vertical load on slab /m = 174255026/ (π(7.50+4.70)/2) = 90929.67 N/m

B.M. = 90929.67(1.40)/8 = 15912.70 Nm

with partial fixity B.M. = 12730.16 Nm

Axial compression = 902564.22 N = 37238.50 N/m.

ResultingB.M. = 15912.70 +37238.50$\sqrt{2}$ [(0.210-0.125)] =20389.07Nm

A_{st} = 20389.07(1000)/ 115 (0.853) (210) = 989.76 mm^2

Min^m Steel = 0.30- (0.10/350)(150) = 0.257% *i.e.,* = 642.86 mm^2

Use 12 mm Ø @ 110 mm c/c on both faces.

Design of Bottom dome:

Span of the dome = 4.70 m.

Rise of the dome = 0.950 m.

Radius of the dome from $0.950\ (2R - 0.950) = (4.70/2)^2$

Hence $R = 3.3816m$

Angle subtended by the dome $= 2\theta$

$$\sin\theta = (4.70/2)/3.3816 = 0.695$$

and $\theta = 44.02°$; $\cos\theta = 0.71$

Take thickness of dome as 200 mm

Loading

D.L. of dome = 0.200 (24000) = 4800 N/m^2

Wt. of water on dome $= 10{,}000\ [\pi/4(4.70)^2\ (6.40) - \pi/6(0.950)\ (3 \times 2.35^2 + 0.950^2)]$

$$= 1023465.44\ N$$

Area of dome surface $= 2\ n\ (3.3816)\ (0.950) = 20.185\ m^2$

$$= 2\ \pi\ (3.3816)\ (0.950) = 20.185\ m^2$$

Load intensity $= (1023465.44/20185) + 4800 = 55504.26 N/m^2$

Meridian thrust at springing level $= wR/(1+\cos\theta) = \frac{55504.26(3.3816}{1.719} = 109187.43 N/m$

Meridian compressive stress $= \frac{109187.43}{1000(200)} = 0.546 N/m^2$

Hoop stress $= wR/t[\cos\theta - (1/1+\cos\theta)$

Maximum at $\theta = 0$, where

Max hoop stress $= 0.469 N/mm^2$

Stresses are low and provide 0.30 % steel.

Use 8 mm Ø bars @ 80 mm c/c.

Design of Circular Girder:

Assume size of girder as 400 x 600 (deep) Center to center of Girder = 4.70 + 0.400 = 5.10 m.
Check for Hoop Stress on Ring Beam

From inclined conical slab with $\varphi_0 = 45°$

$N\varphi_0 = 253344.65N$

Horizontal component=253344.65 $\cos\varphi_0$ = 179141.72N

From bottom of Dome $N\varphi_1$ =109187.43 N/m

with φ_1=44.02°

Horizontal component = 78505.76 N

Hoop stress as Ring Beam = (179141.72 -78505.76)(5.10/2) =256621.70N

Hoop stress (compression) =256621.70/(1000(400))

=0.642N/mm^2

Total loads on the circular girder are

Weight of water = 1514896.92+ 1023465.06 N = 2538362.06 N

Wt. of Top dome and side slab = 902564.22 N

Wt. of Conical wall = 227653.34 N

Wt. of Lower dome = (4800) (20.185) = 96888.0 N

Wt. of Circular girder

with a size of 400 x 600 = 78200.0 N

Total Wt. 3843667.62 N

With D = 4.70 + 0.400 = 5.10,

W = 239897.53 N/m

Provide Six Columns.

At location of Maxm+ V_eBM. at Center of the Girder *i.e.,* at $\varphi = 0$, Torsional moment = 0

From $$M_\varphi = \left(\frac{\theta}{\sin\theta}\cos\varphi - 1\right)WR$$

$$T_\varphi = [(\theta/\sin\theta)\sin\varphi - \theta]WR$$

$$V_\varphi = WR\varphi$$

For the case $2\theta = 60°$; With 6 Columns
Considering column of diameter 600 mm (531.74 mm eq. square)

$\varphi = 24°$ on face of column from center of span

$\varphi = 0°$ at center.

$\varphi = 17.27°$ from center for T_{max}

Design negative BM on face of column = $[(\pi/6)/0.50)\cos24°](239897.53)(2.55)^2$

$= 67603.52$Nm

Maximum +Ve BM at $\varphi = 0$,

$$B.M. =[(\pi/6)/0.50)\cos0°](239897.53)(2.55)^2$$

$$=73625.05\text{Nm}$$

Maxm. Torsion Moment at $\varphi = 17.27°$ from center

$$T_{max} = \left[\frac{(\pi/6)}{0.05}\sin17.27° - \frac{(17.27)(\pi)}{180}\right](239897.53)(2.55)^2$$

$$= 14769.97\text{Nm.}$$

At location of- Ve B.M. on face of columns.

$$T_{\varphi} = \left[\frac{(\pi/6)}{0.05}\sin24° - \frac{(24)(\pi)}{180}\right](239897.53)(2.55)^2$$

$$= 11004.70 \text{ Nm}$$

S.F. at distance d = 560 mm from face of support
Where $\varphi = 11.44°$

$$V = wR\varphi = (239897.53)(2.55)\frac{(\pi)(11.44)}{180} = 122143.22\text{N}$$

S.F on face of column $V_e = V + 1.6(TR/b) = 122143.22 + 1.6\,(11004.70/0.400)$
$= 166162$N

$$\tau_{ve} = \frac{166162}{(400)(560)} = 0.742 < \tau_{max}\text{ , i,e., } 1.80 \text{ N/mm}^2$$

Shear reinforcement necessary

Design moment at face of column = $M+M_t$

$= 67670.52+(11004.70/1.70)(1+600/400)$

$= 83853.90M$

Hence section is under reinforced

$$A_{st} = \frac{83583.90(1000)}{115(0.853)(550)} = 1526.47 \text{ mm}^2$$

Use 5 Nos. 20 mm *§* at top in one layer

As $M_t < M$, longitudinal reinforcement on flexural compression face not necessary.

For positive B.M.

$$A_{st} = \frac{73625.05(1000)}{115(0.853)(560)} = 1526.47 \text{ mm}^2$$

Use 5 Nos. 20 mm at bottom.

SF at a distance d = 56mm from face of column where $\phi = 11.44°$

and SF there = $166162.42=V_e$

τ_{ve} there = $0.742 \text{ N/mm}^2 < 1.80 \text{ N/mm}^2$

Shear reinforcement is necessary.

At location of Tmax.

$$V_e = 184389.28 + 1.6+(14769.97/0.40)$$

$$\tau_{ve} = 1.087 \text{ N/mm}^2 < \tau_{max}$$

$100(A_{st}/bd) = 0.70$ and $\tau_c = 0.340 \text{N/mm}^2$

Transverse reinforcement to be not less than

$$A_{sv} = (\underline{\tau_{ve}} - \tau_c)b.S_v/S_v = [(1.087-0.340)\,400(S_v)]/125$$

For two legged 12 mm ϕ stirrups. S_v @ 95 mm c/c.

Also $\quad A_{sv} = (T\,S_v/b_1 d_1 \sigma_{sv}) + V\,S_v/2.5\,d_1\,\sigma_{sv}$

b_1 =400-2(25+ 12+10) = 306mm

d_1= 600 -2(25 +12 +10)=506mm

A_{sv}= [(14769.97 (1000)S_v)/ (306) (506) (125)]+[(184389.28 S_v/2.5 (506) (125)]

=0.76313 S_v+1.16610 S_v =1.92923 S_v

With 2 legged 12 mm ϕ stirrups

S_v = 117 mm c/c. Use S_v = 95 c/c.

Design of Columns – Six columns of 600 mm diameter to be symmetrically place at 60° c/c. Length of column = 12 + 1 = 13 m. Consider column to have a batter of 1 in 20,

α = 2.862° and cos α = 0.999; sin α = 0.050

Total load:

Load from Top	= 3843667.62 N
Self wt. of 6 columns	= 529300.00 N
Wt. of Bracing	= 60000.00N
Total	= 4432967.62 N

Load on each column due to W = 738828 N

Thrust on each column = 739567.50 N

Diameter at base = 5.10+(12/10) =6.30m

Wind Loading Considering V_b - 50 m/sec.

$V_z = V_b k_1 k_2 k_3 = 0.90 V_b k_2 k_3$ taking k_1 as 0.90 for 25 yrs. life.

Taking k_2 and k_3 both as unity.

$$p_z = 0.60\, v^2 = 0.60\, (45)^2 = 1215\ \text{N/m}^2$$

P1 = [7.5+0.450)(5.0(7.69)(2/3)(1.60)+(7.69+5.10)(1.60)/2]

(1215)(0.70) = 49915N

taking shape factor as 0.70

This acts at a height $= 12 + 1/2(1.60 + 5 + 1.6)$

P_2 = Due to column, Bracing and circular Girder

$= [(5.50)(0.600) + Vi(0.60)(4.0)(6.0)](1215)(0.70)$

$+(5.50)(0.300)(1215)(0.70)$

= 10333.58N at l2m above G.L.

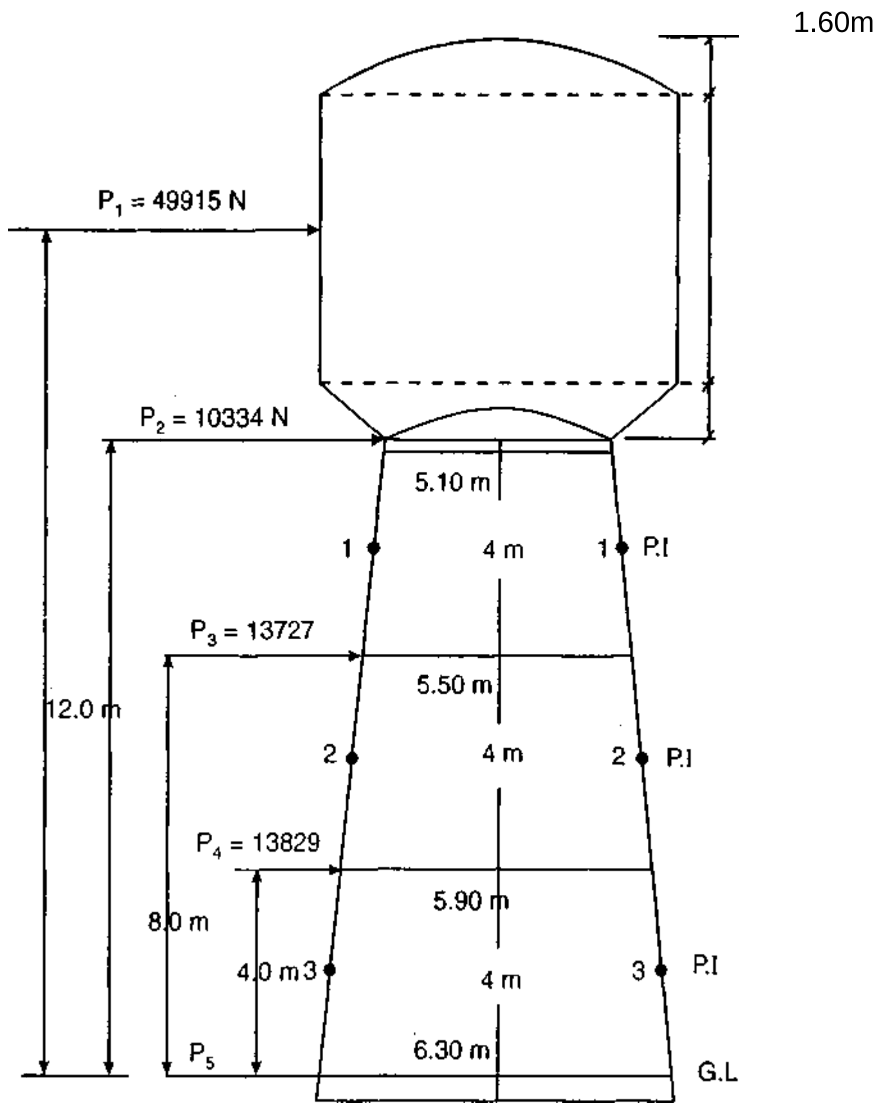
P_3 = On column and Bracing

$= [6(0.600)(4) + (5.80)(0.300)](1215)(0.70)$

= 13727 Nat 4 m. above G.L

$P_4 = [6(0.600)(4) + (6.20)(0.300)](1215)(0.70)$

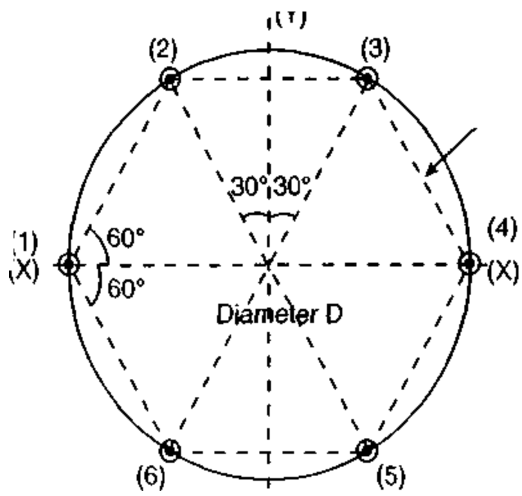
= 13829 *N at* 4 m above G.L.



=16.10maboveG.L.

Consider column fixed at base
Considering P.I. at mid height of column

Levels	Shear (N)	M(Nm)
1-1	49915+10334=60249	49915 (4.10+2)+10334(2) =325149.50
2-2	60249+13727 = 73976	49915 (6.10+4)10334(6)+13727(2) =667575.50
3-3	73976+13829=87805	49915(14.10)+13727(6)+10334(10) +13829(2) = 917161.50



Referring to Fig. For XX axis

$\sum y2 = 4(D*\surd 3)\text{^}2/8 = 3/4d\text{^}2$

For YY axis $\sum x2 = 2(D/2)\text{^}2 + 4(D/4)\text{^}2 = 3/4D\text{^}2$

For bending about *XX*axis, due to wind loading Referring to line 3-3.

V1 = V4 =0

And V2 =V3 =V5 = V6 = ±917161.50/(3/4(6.10)^2 (√3/4(6.10)

= ± 86807.12M

For bending about YYaxis,

$$V1 = V4 = \frac{917161.50*6.10}{\frac{3}{4}(6.10)\text{^}2}$$

$V_2 = V_4 = V_5 = V_6 = \pm\ 50118.1$ N

Maxm. Thrust in column = 739567.50 + 100236.23 /0.999 =83994 N

At point 3-3,

On each column H= 878.5/6 =14634.17 N

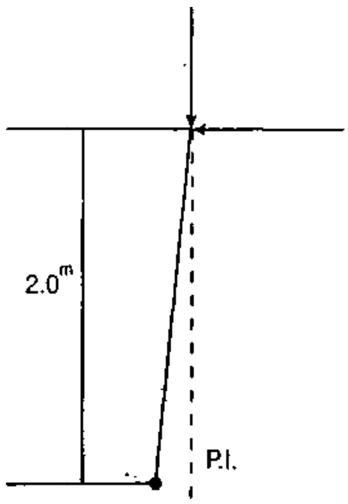
Vertical load at (3) due to W.L. = 100236.23 N.

B.M on column = 14634.17(2)-100236.23N

= 19244.72 Nm.

(Refer Fig.) design of column can be done by limit state method

M - 20 Concrete F_e 415 steel. Self wt. of a column = 81530 N



Axial force = 839904 N + 81530N

M_{yy} =19244.72 Nm. This case 1

Axial force = 839904 N +81530N

M_{xx}=2058.64Nm. this is case 2

Safety factor to be used is 1.20

Case-I:P_u1105720.80 N

M_{Uy}=23093.664 Nm.

CaseII : Pu = 1089589.38N

Muxx =24705.17 N

With 600φ column

Pu/(fck d^2)=0.154 for Case -1 and 0.151 for Case – II

Mu/(fck D^3) = 0,0053 for Case -1 and 0.0057 for Case – II

d/D = (40+8)/0.08

use d/D = 0.10

Min Steel = 0.80 % *i.e.,* p/fck =0.04

Muxx ~ Muyy = 0.06(20) (600)^3

Hence column is adequate.

Use 8 Nos. 20 mm § bars as longitudinal bars; Ties 10 mm @ 300 c/c.

Design of Bracing:

∑M at Joint above (3) = 20587.64 + 9167(2) = 389.21.64 Nm

Moment in bracing = 38931.64 √2

Providing 300 mm x 300 mm section and design as a doubly reinforced beam with equal steel at >p and bottom.

Ast = Asc = [(38921.64(1.2 (1000)]/(0.87(415)(260)

=497.54 mm^2

Use 4 Nos. 16 mm both at top and bottom

Use 2 legged 12 mm stirrups @ 200 c/c

Check for Seismic effect:

Refer IS 1893 - Consider zone III

Wt. of empty Tank = 1305305.56 N

Wt. of full Tank = 3843667.62 N

For Empty condition Total Wt. = 1305305.56+1/3(600.000)= 1505305.56 N

For Tank full condition

Total Wt.= 4043667.62

Stiffness of a column in a bay

K_c= 12EI/L^3

$E = 5000\sqrt{fck} = 22360.68 N/mm^2$

$I = \pi/4(D)^4 = 6.362\ (10)^9\ mm^4$

L = 4m =4000 mm.

$k_c = 12(22360.68)(6.362)10^9/4000^3$

= 26673.56 N/mm

Stiffness of six column,

$\sum k_c = 6(26673.50) = 160040.98$ N/mm

Neglecting effect of bracing ot stiffness.

$$1/K = \sum 1/k,$$

When $K = \frac{160040.98}{3} = 53347 N/mm$

Fundamental period $T = 2\pi\sqrt{W/gK} = 2\pi\sqrt{4043667.62/9810(53397)}$

$= 0.552 sec.$

From IS:1893, $A_h = ZI\ S_a/2Rg$

For zone III $Z = 0.16;\ I = 1:\ R = 2.50$

Considering hard soil site

$$S_a/g = 1.00(1)/T(0.552) = 1.8116$$

Taking it as 1.887 > 1.8116

and $Ah = 0.16(1.887)/\ 2(2.50) = 0.0604$

Hence $V = 0.0604\ (4043667.62) = 244237.52\ N$

acting horizontal at the C.G. of the tank. Seismic effect $V > \sum H$ forces due to wind. Hence seismic effect controls.

Design of column has to be done considering seismic effects

$V = 244237.52$ N acts at 14.10m above hinge along 3-3

$M = 244237.52\ (14.10)$ Nm

Maxm. Vertical force on column $= [244237.52(14.10)(6.10)]/\frac{3}{4}(6.10)^2(2)$

$= 376366.02N$

Maxm. Thrust on the column $= 739567\ 50 + 37666.02/0.999$

$= 1116310.26N$

Horizontal force on column $= 232157.52/6 = 38692.92N$

As in case of wind forces.

BM on column $= 38692.92\ (2) - 376366\ (0.10)$

$= 39749.24$ Nm

Using Limit state Method

$$P_u = 1.20\ (1116310.26 + 81530) = 1437408.33\ N$$

$$P_u/f_{ck}D^2 = 0.200$$

$$M_u = 1.20\ (41610.74) = 49932.89$$

Refer to fig. A.9

$$d'/D = 0.08\sim0.10$$

with minimum steel 0.08%, $p/f_{ck} = 0.04$

$$M_u = 0.056\,(20)\,(600)3 = 241920\text{Nm.}$$

Hence adequate

Use bracing

Empty condition

A_h taken 0.08 the maximum value

$$V = 0.08\,(1505305.56) = 120424.45\text{ N} < 232157.52\text{ N}$$

Hence empty condition does not control.

The column size can be reduced to 450 mm Ø

Design of the ring beam at top will practically remain the same as before except that the Maximum BM and Torsion at the face of the column (eq square = 400 mm) to be calculated and checked.

For the case $\varphi = 25.50°$

Negative BM on face of column = -85507.61 Nm

$$T_\varphi \text{ there } = 9002.70\text{ Nm}$$

$$M_e = 85007.61 + \frac{9002.70}{1.70}(1+600/400) = 98746.88\text{Nm}$$

$$A_{st} = 98746.88(1000)/115(0.853)(560)$$

$$= 1797.58\text{mm}$$

Use 6 Nos. 20 mm at top

Other details will be the same as before

Check for Seismic effect

for empty Tank = 1505305.56 N

for Tank full=4043667.60 N

for the column $I = \pi/64(450)^4 = 2.012889590(10)^9\text{mm}^4$

$$L = 4000\text{ mm}$$

$$k_c = 8439.30\text{ N/m}$$

As before $K = 6(8439.30)/3 = 16878.60\text{ N/m}$

$$T = 2\pi\sqrt{4043667.62/9810(16878.60)} = 0.982\text{sec}$$

Considering for safety soft soil (in place of hard soil considered earlier)

$S_a/g = 1.67/0.982 = 1.70$

$A_h = 0.16/2(250) =(1.70) = 0.0544$

$V= 0.0544\ (4043667.62) = 219975.52$ N

acting at 14.10 m. above 3-3

$M = 219975.52(14.10)$ Nm

Maxm. vertical force on column = $219975(14.10)(6.10)/¾(6.10)^2(2)$

$= 338978.67$N

Maxm. Thrust on column = 739567 50 + (338978.67/0.999)

= 1078885.42 N

Horizontal force on column = 219975.52/6 = 36662.59 N

As before

BM on column = 36662.59 (2) - 338978.6 (0.10) = 39427.32 Nm

Using Limit State Method

$Pu = 1.20\ (1078885.42+45804.42) = 1349627.81$ N

$P_u/f_{ck}D^2 = 0.332$

$M_u = 1.20(39427.32) = 47312.78$ Nm

With 0.80% steel $P/f_{ck} = 0.04$

$d'/D = 50/450 = 0.11 \sim 0.10$

from fig A-9,

$M_{u1} = 0.035(20)(450)^3 = 63787.50\text{Nm} > 47312.78\text{Nm}$

Hence 450 φ column adequate.

Use 0.80% steel i.e., Use 8 Nos 16 mm φ longitudinal

Ties For bracing Use 10mmφ @ 250 c/c.

Total M. from (2) (2) and (3) (3) = 2 (39427.32) Nm

∑M on column = 2 (39427.32) Nm

Part of this moment taken by two bracings equally.

partly by Moment and partly by Torsion.

Angle=60°

M= 2(23427.32sin30°)/2 = 19713.67Nu.M

T = 39427.32 cos 30° = 34145.06 Nm

V = 6160N

$M_e = 1.20(19713.67+\frac{34145.06}{1.70}(1+350/300)$ =75878.28N

d= 300mm,D = 350 OK;d=350-(25+10) = 315mm

Use A_{st}= 811.92 mm^2 i.e. Use 5 Nos. 16 mm φ at top and bottom.

$V_e = [6160+1.6(\frac{34145.06}{0.300})](1.20)$ =225920.384N

τ_{ve} =2.39<2.80 N/m

$100(A_{st}/bd) = 5(\pi/4)(16)^2(100)/300*315 = 1.069$

$\tau_c = 0.633$

$A_{sv} = \tau_{ve} - \tau_c (bS_v)/0.87f_y = (2.39-0.633)(300)S_v/0.87(415)$

$= 1.46\ S_v$

Also $A_{sv} = \frac{\tau_u\ S_v}{b_1.d_1(0.87)(415)} + \frac{V_u\ S_v}{2.5d_1(0.87)(415)} = 2.03\ S_v$

as b_1= 300-(25+12+8) =210; d_1 =270; S_v =111

Use 12mm@ 100 c/c.

Design of Foundation:

Consider bottom of foundation at 2 m below G.L.

Vertical load:

Tank full = 3843667.62 + 274826.52 + 75000 =4193494.15 N

Tank Empty = 1305305.56 + 274826.52 + 75000 = 1655132.08 N

Moment at bottom of Foundation :

For Tank fulI = 219975.52(18.10) = 3981556.91Nm

For Tank Empty 113753.152(18.10) = 2058932.05Nm,

with A_h = 0.08 (for safety maximum value of A_h taken);

Diameter at base c/c. of columns = 6.40 m.

Consider an outside diameter of Foundation as 9.60 m and 3.20 m. inside diameter.

$$\text{Area} = \pi(4.80^4 - 1.60^2) = 64.33982 \text{ m}^2$$

$$I = \pi/4\,(4.80^4 - 1.60^4) = 411.77483 \text{ m}^4$$

Considering 10% as the approxiamte Wt. of

Foundation $P = \dfrac{1.10(4193494.15)}{64.33982} = 71.695 \text{ KN/m}^2 < 100 \text{ kN/m}^2$

For Tank full:

$p_{max} = 118.1074392 \text{ kN/m}^2$

$P_{min} = 25.2825661$ kN/m2< 1.25 (100) *i.e.,* 125 kN/m^2

For Tank Empty:

$p_{max} = 56.24325 \text{ kN/m}^2$

$P_{min} = 8.241 \text{ kN/m}^2$

No Tension and Base is adequate.

Assume a Ring girder of Size 450 x 700.

Design on basis of $p_{ne(} = 65.177 \text{ kN/m}^2$

Supported on six column of 450 mm diameter

eq. square = 400 mm and 0 = 30°

On face of column $\varphi = 25.50°$

Location of Maxm. T, $\varphi = 17.27°$

For S.F at distance d =620 mm from

Location of maximum. T, $\varphi = 15.32°$

Load on ring beam =65.177(3.20) = 208.567 kN/m

As self wt. acts toward.

Maximum -V_e BM on face of column = 117.0696 kN/m

T_φ at the location =12.3257kN/m

Maximum + V_e BM at center = 100.801kNm

T_{max} at $\varphi = 17.27° = 20.22176$kNm

SF at $\varphi = 17.27°$ =201.171 kN

Design by limit state method.

As only vetical (D.L+L.L) considered

load factor =1.50

with M-20 concrete and Fe 415 steel

Mu, limit = 0.138(20)(450)(620)2 = 477.425kNm

Section is under reinceforced

At face of column $Mu= 1.50 \left[117.0696+\frac{12.3257(1+(700/450)}{1.70}\right] =203.3976\text{kNm}$

Lever arm j = 0.927 and A_{st} =980.18mm^2

Use 5nos. 16mm at bottom ; minimum A_{st} =572 mm^2

For +V_e BM =100.801 kNm

No torsion occurs there,

$$Mu = 1.50(100.801) =151.2015\text{kNm}$$

$$J = 0.947 \text{ and } A_{st} =713.25 \text{ mm}^2$$

Use 4Nos.16mm φ at top.

At of T_{max}

SF V_e =201.171+1.6(20.22176/0.450) =273.071kN

V_{ue}= 1.50(273.071) = 409.606kN

$\tau_{ve} = 409.606(10)^3/450(620) = 1.468 < \tau_{cmax}$.

$100(A_{st}/bd) =[\ 100(4)\pi/4\ (16)^2\ ] / (450)(620) =0.288\%$

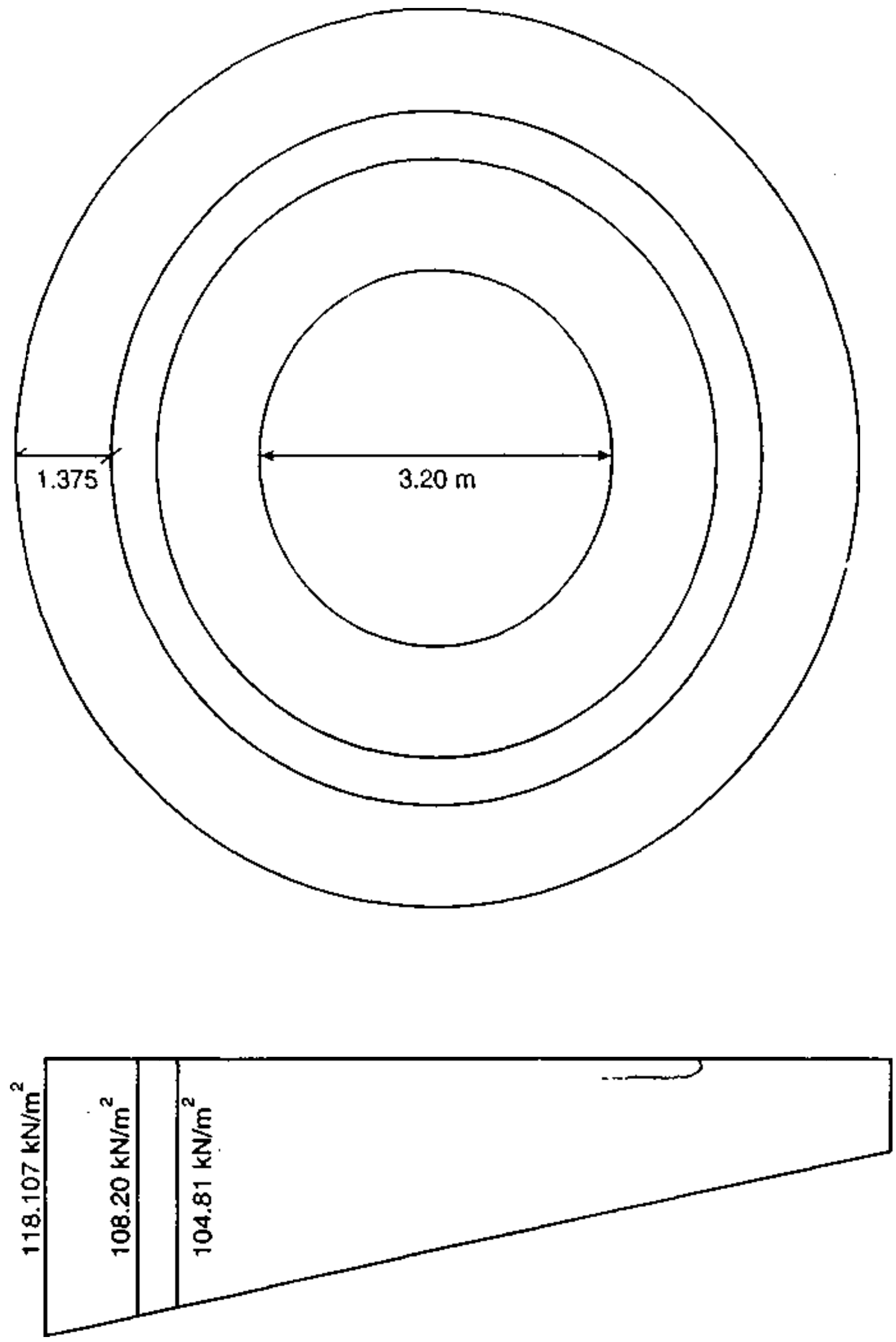


Fig3

$\tau_c = 0.378$ N/mm^2

minimum transverse reinforcement

$$A_{sv} = (\tau_{ve} - \tau_c) b S_v / 0.87 f_y = 1.35854 S_v$$

With 2 legged 10 mm O @ 115 c/c. or 12 mm 0@ 160 c/c.

Also $$A_{sv} = \frac{T_u S_v}{b_1 d_1 (0.87 f_y)} + \frac{V_u S_v}{2.5(d_1)(0.87) f_y}$$

b_1 =450-2(40+20+8) = 330 mm

d_1 =700-2(40+12+8) =580 mm

and A_{sv}=1.0153 S_v –this gives smaller spacing

Use 12 mm φ @ 150 c/c.

For side face reinforcement use 0.10% of web area

= 0.10/100(500) (450) = 225 mm^2

Half on each face i.e. Use 12 mm 0 bar longitudinally.

Design of bottom slab — Use 400 mm thick slab

Projection =1.60(0.450/2)= 1.375 m

Designed for variation of bearing pressure considering effect of Moment Downward load from top due to slab and soil = 40 kN/m^2 Referring to Fig.

MaximumBM=[(104.81-40)$(1.375)^2$/2]+13.297[$(1.375)^2$ /3] =69.4556kNm

Maxm. SF at distance d = 3 50 from face of beam

=[½(108.20 +118.107) -40] (1.375 -0.350)

= 74.982 KN

τ_v= 1.5(74.962)$(10)^3$/1000(350) = 0.321 N/mm^2

100A_{st}/be required = 0.202%

For BM. M_u=1.50(69.4556) = 104.1834kNm

J = 0.948 and A_{st} =869.67mm^2

Use 12mmφ @125 c/c

100(A_{st}/bd) = 0.258% hence it is adequate

Check for stability:

Sliding - Due to seismic loading

V= 244237.52 N

W=4193494.15 + Wt. of base + Circular Bear

=4193494.15 + 617662.25 + 65144.06

=4876300.56 N

FS against sliding μW/244237.52 = 9.98>2 OK

FS against overturning = 4876300.56(4.80)/244237.52(18.10)

=5.295>2 OK

For empty condition = W = 1655132.08 + 617662.25+65144.06

= 2337938.39

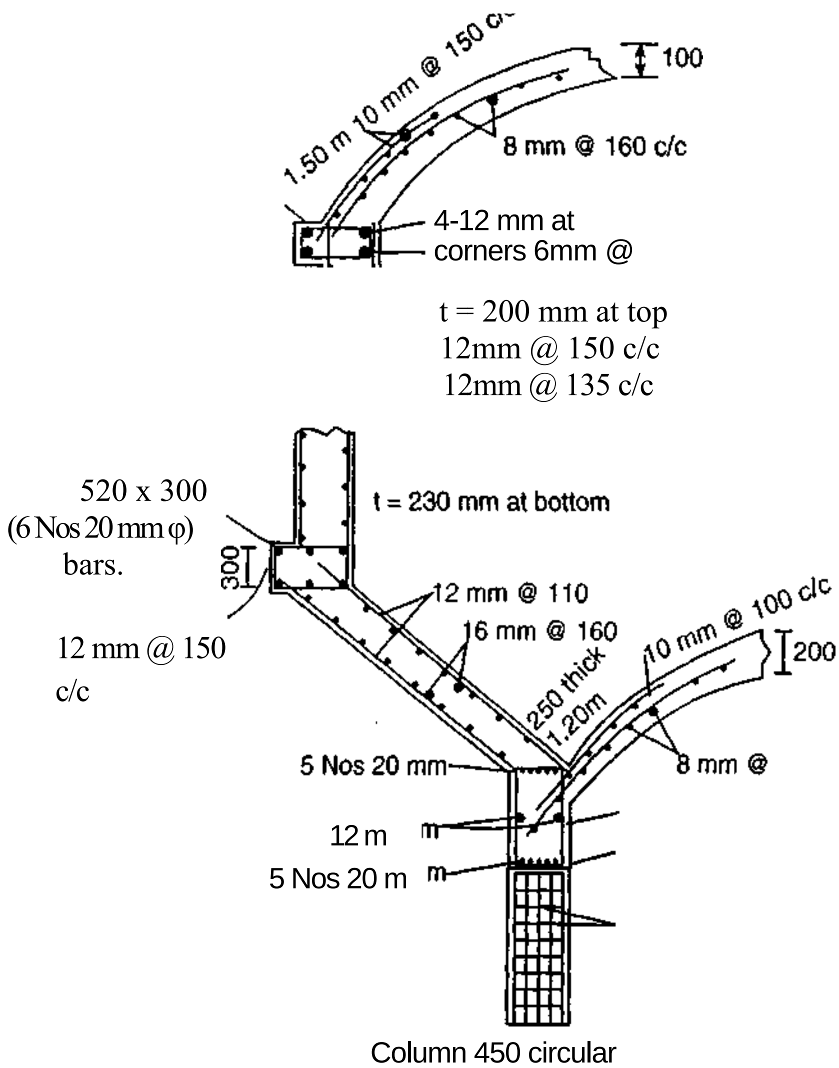
FS against sliding = $\mu W/V$

V = 120424.45N =9.707> 2.0

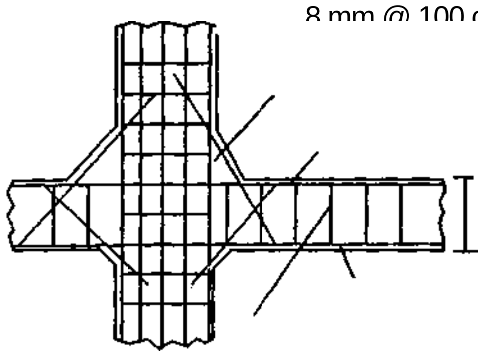
F.S against over turning =2337938.39(4.80)/ 120424.45(18.10)

= 5.15>2.0

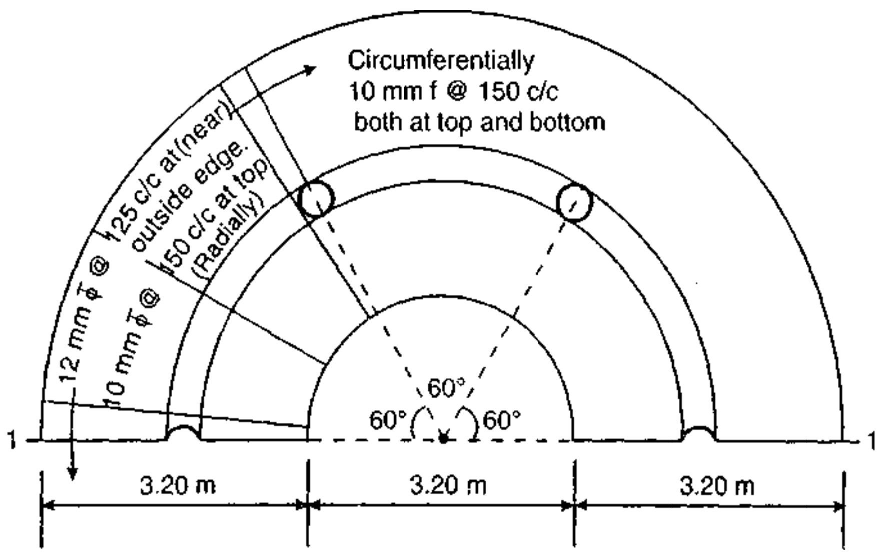
Fig 4 shows the details of the tank.

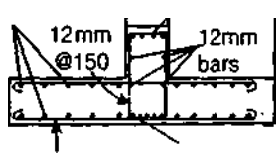


(a) Details of tank



(b) Bracing of Columns





(c) Details of foundation

ESTIMATION

Detailed estimation:

Detailed estimate is an accurate estimate and consists of working out the quantities of each item of works, and working the cost. The dimensions, length, breadth and height of each item are taken out correctly from drawing and quantities of each item are calculated, and abstracting and billing are done.

The detailed estimate is prepared in two stages:

Details of measurement and calculation of quantities.

The details of measurements of each item of work are taken out correctly from plan and drawing and quantities under each item are calculated in a tabular form named as details of measurement form.

Abstract of estimated cost:

The cost of each item of work is calculated in a tabular form the quantities already computed and total cost is worked out in abstract estimate form. The rates of different items of work are taken as per schedule of rates or current workable rates for finished item of work.

Detailed estimation

S.No.	DECRIPTION OF WORK	**NOS**	L m	B m	A m^2	D m	QTY m^3	**REMARKS**
1	EARTH WORK IN EXCUVATION	1			64.32	2	128.64	L =2πR = 2π*2.55 = 16.022m,R =5.1/2 = 2.55m
2	EARTH WORK IN FILLING	1					100.198	L =2πR = 2π*3.75 = 23.56m,R =7.5/2 = 3.75m
3	RCC WORK IN FOUNDATION (1:1.5:3)	1			64.32	0.4	25.728	L =2πR = 2π*3.75 = 23.56m,R =7.5/2 = 3.75m
4	RCC WORK IN COLOUMNS BELOW G.L (1:1.5:3)	6			0.282	1.6	2.714	Sa = 2πhRc =π (h2 + r2) =π (1.5^2+5.4375^2) =99.95m^2,h =1.5m,r = 5.4375
5	RCC WORK IN COLOUMNS ABOVE G.L UPTO 4M HT (1:1.5:3)	6			0.282	4	6.785	Davg =(7.5+5.1)/2 = 6.3m,R=6.3/2= 3.15m,Sa =πr(r+h) = π*3.15(3.15+1.6) = 47.006m^2
6	RCC WORK IN COLOUMNS FROM 4M TO 8M HT(1:1.5:3)	6			0.282	4	6.785	R = 3.3816m,Sa = 2πhRc =π (h2 + r2) =π (0.950^2+3.3816^2) =

								38.760m^2
7	RCC WORK IN COLOUMNS FROM 8M TO 12M HT (1:1.5:3)	6			0.282	4	6.785	D = (0.23+0.2) =.215m,Sa =2πR h= 2π*3.75*5 = 117.80m
8	TOTAL RCC WORK IN COLOUMNS (1:1.5:3)						23.069	QTY = 2*6*0.3*0.3*0.6 =0.648m^3
9	RCC WORK IN BRACING AT 4m HT (1:1.5:3)	1	18.535	0.3		0.3	1.668	QTY = 23.609 - 0.648 = 22.961m^3
10	RCC WORK IN BRACING AT 8m HT (1:1.5:3)	1	17.278	0.3		0.3	1.555	QTY =25.728+2.714+3*6.785+ 22.961+1.668+1.555+3.845+3.675+0.848+9.995+11.751+7.752+25.327=138.174m^3
11	RCC WORK IN CIRCULAR GIRDER (1:1.5:3)	1	16.022	0.4		0.6	3.845	,R=6.3/2= 3.15m,Sa =πr(r+h) = π*3.15(3.15+1.6) = 47.006m^2
12	RCC WORK IN RING BEAM AT BOTTOM OF THE CL WALL (1:1.5:3)	1	23.56	0.3		0.52	2.675	R=6.3/2+0.5= 3.65m,Sa =πr(r+h) = π*3.65(3.65+1.6) = 60.2m^2
13	RCC WORK IN RING BEAM AT TOP OF THE CL WALL (1:1.5:3)	1	23.56	0.16	99.95	0.225	0.848	R = 3.3816m,Sa = 2πhRc =π (h2 + r2) =π (0.950^2+3.3816^2) = 38.760m^2
14	RCC WORK IN DOMED ROOF(1:1.5:3)	1				0.1	9.995	Sa = 2πhRc =π (h2 + r2) =π (1.5^2+5.4375^2) =99.95m^2,h =1.5m,r = 5.4375
15	RCC WORK IN CONICAL SLAB (1:1.5:3)	1			47.06	0.25	11.751	Davg =(7.5+5.1)/2 = 6.3m,R=6.3/2= 3.15m,Sa =πr(r+h) = π*3.15(3.15+1.6) = 47.006m^2
16	RCC WORK IN CONICAL DOME (1:1.5:3)	1			38.76	0.2	7.752	R = 3.3816m,Sa = 2πhRc =π (h2 + r2) =π (0.950^2+3.3816^2) = 38.760m^2
17	RCC WORK IN CYLINDRICAL WALL (1:1.5:3)	1		0.215	117.8	5	126.35	D = (0.23+0.2) =.215m,Sa =2πR h= 2π*3.75*5 =

								117.80m
18	DEDUCTIONS IN RCC WORK IN BRACINGS IN COLOUMNS	2*6	0.3	0.3		0.6	0.648	QTY = 2*6*0.3*0.3*0.6 =0.648m^3
19	T0TAL RCC WORK IN COLOUMNS AFTER DEDUCTIONS						22.901	QTY = 23.609 - 0.648 = 22.961m^3
20	TOTAL RCC WORK (1:1.5:3)						138.174	QTY =25.728+2.714+3*6.785+ 22.961+1.668+1.555+3.8 45+3.675+0.848+9.995+1 1.751+7.752+25.327=138 .174m^3
21	PLASTERING IN C M (1:2) FOR INNER SURFACE OF CONIVAL SLAB (12MM)	1			47.06		47.006	,R=6.3/2= 3.15m,Sa =πr(r+h) = π*3.15(3.15+1.6) = 47.006m^2
22	PLASTERING IN C M (1:6) FOR OUTER SURFACE OF CONICAL SLAB (12MM)				60.2		60.2	R=6.3/2+0.5= 3.65m,Sa =πr(r+h) = π*3.65(3.65+1.6) = 60.2m^2
23	PLASTERING IN C M (1:2) FOR INNER SURFACE OF CONICAL DOME (12MM)	1			38.76		38.76	R = 3.3816m,Sa = 2πhRc =π (h2 + r2) =π (0.950^2+3.3816^2) = 38.760m^2
24	PLASTERING IN C M (1:6) FOR OUTER SURFACE OF CONICAL DOME (12MM)				43.13 5		43.135	R = 3.3816+0.2m = 3.5816,Sa = 2πhRc =π (h2 + r2) =π (0.950^2+3.3.5816^2) = 43.135m^2
25	PLASTERING IN C M (1:2) FOR INNER SURFACE OF CYLINDRICAL WALL (12MM)				117.8		117.8	D = (0.23+0.2) =.215m,Sa =2πR h= 2π*3.75*5 = 117.80m
26	PLASTERING IN C M (1:6) FOR OUTER SURFACE OF CYLINDRICAL WALL (12MM)				125.0 3		125.03	D = (0.23+0.2) =.215m,R=3.75+.23 =3.98m,Sa =2πR h= 2π*3.98*5 = 125.03m
27	PLASTERING IN C M (1:2) FOR INNER SURFACE OF DOMED ROOF (12MM)				96.5		96.556	Sa = 2πhRc =π (h2 + r2) =π (1.5^2+5.3375^2) =96.56m^2,h =1.5m,r = 5.3375
28	PLASTERING IN C M (1:6) FOR OUTER SURFACE OF DOMED ROOF (12MM)				99.95		99.95	Sa = 2πhRc =π (h2 + r2) =π (1.5^2+5.4375^2) =99.95m^2,h =1.5m,r =

								5.4375
29	PLASTERING IN C M (1:6) FOR COLUMNS (12MM)	6			45.23		271.433	P =2πRh =2π*.6*12 = 45.23m^2
30	PLASTERING IN C M (1:6) FOR CIRCULAR GIRDER (12MM)	1	16.022			0.6	91.732	L =2πR = 2π*2.55 = 16.022m,R =5.1/2 = 2.55m
31	PLASTERING IN C M (1:2) FOR RING BEAM AT TOP (12MM)		23.56	0.16			18.213	Sa =2*23.56*0.225+2*0.225 *0,16+2*0.16*23.56 = 18.213m^2
32	PLASTERING IN C M (1:2) FOR RING BEAM AT BOTTOM (12MM)		23.56	0.3		0.225	38.95	Sa =2*23.56*0.52+2*0.52*0 .3+2*0.3*23.56 = 38.950m^2
33	PLASTERING IN C M (1:6) FOR BRACING AT 4M HT (12MM)		18.535	0.3		0.52	22.422	Sa =2*18.535*0.3+2*0.3*0. 3+2*0.3*18.535 = 22.422m^2
34	PLASTERING IN C M (1:6) FOR BRACING AT 8M HT (12MM)		17.278	0.3		0.3	20.936	Sa =2*17.278*0.3+2*0.3*0. 3+2*0.3*17.278 = 20.936m^2
35	TOTAL PLASTERING IN CM (1:2) 12MM THICK					0.3	357.289	QTY = 47.006+38.76+117.8+96. 56+18.213+38.95 = 357.289m^2
36	TOTAL PLASTERING IN CM (1:6) 12MM						652.838	QTY = 60.2+43.135+125.03+99. 95+271.433+9.732+22.42 2+20.936 = 652.838m^2
37	THICK WATER PROOF CEMENT PAINTING FOR TANK PORTION						647.174	QTY =47.006+60.2+38.76+43. 135+117.8+125.03+96.56 +99.95+18.213+0.52=647 .174m^2
38	WHITE WASHING FOR COLUMNS	6			45.23		271.433	P =2πRh =2π*.6*12 = 45.23m^2
39	TOTAL WHITE WASHING						918.607	QTY =647.174+271.433 = 918.607m^2

ABSTRACT

S.NO	DESCRIPTION OF WORK	QTY OR NOS	RATE RS PS	COST RS PS
1	Earth work in excuvation	**28.30cumec**		
2	Beldars	**5nos**	**215.00**	**1075.00**
3	Mazdoors	**4nos**	**215.00**	**860.00**
4	Total			**1935.00**
5	Total earth work in Excavation for 128.64cumec	**128.64/28.30** **=4.6*1935** **=8901**		**8901.00**
6	Earth work in filling In foundation	**28.30**		
7	Beldar	**3**	**215.00**	**645.00**
8	Bhisthi	½	**260.00**	**130.00**
9	Total			**775.00**
10	Total earth work in Filling 100.198 cumec	**100.198/28.30** **=3.6*775** **=2790**		**2790.00**
11	Disposal of surplus earth in a lead 30m			
12	Mazdoor		**215.00**	**645.00**
13	Total			12336**.00**

DATA SHEET

RCC M- 20 Nominal mix (Cement:fine aggregate: coarse aggregate) corresponding to Table 9 of IS 456 using 20mm size graded machine crushed hard granite metal (coarse aggregate) from approved quarry including cost and conveyance of all materials like cement				
FOUNDATION				
A. MATERIALS:	UNIT	QTY	RATE RS	AMOUNT RS
20mm HBG graded metal	Cum	0.9	1405.04	1264.54
Sand	Cum	0.45	509.92	229.46
Cement	Kgs	400	5.42	2169.60
1st Class Mason	Day	0.133	285.00	37.91
2nd Class Mason	Day	0.267	260	69.42
Mazdoor (Both Men and Women)	Day	3.6	215	774.00
Concrete Mixer 10 / 7 cft (0.2 / 0.8 cum) capacity	Hour	1	248.40	248.40
Cost of Diesel for Miller	Liters	0.133	45	5.99
Cost of Petrol for Vibrator	Liters	0.667	68	45.36
Water (including for curing)	Kl	1.2	77.00	92.40
Add 20% in Labour (1st Floor)				176.27
Add MA 20%				211.52
Add TOT 4%				212.99
BASIC COST per 1 cum				**5538.00**

Description	**Unit**	**Quantity**	**Rate Rs.**	**Amount Rs.**
COLUMNS				
20mm HBG graded metal	Cum	0.9	1405.04	1264.54
Sand	Cum	0.45	509.92	229.46
Cement	Kgs	400	5.42	2169.60
1st Class Mason	Day	0.167	285.00	47.60
2nd Class Mason	Day	0.167	260	43.42
Mazdoor (Both Men and Women)	Day	4.7	215	1010.50
Labour for centering	Cum	1	971	971.00
Material hire charges for centering	Cum	1	89	89.00
Concrete Mixer 10 / 7 cft (0.2 / 0.8 cum) capacity	Hour	1	248.40	248.40
Water (including for curing)	Kl	1.2	77.00	92.40
Add 20% in Labour				432.304
Add MA 20%				500.96
Add TOT 4%				283.96
BASIC COST per 1 cum				**7383.144**

RCC RING BEAM AT TOP	UNIT	QTY	RATE RS	COST
20mm HBG graded metal	cum	0.9	1405.04	1264.54
Sand	cum	0.45	509.92	229.46
Cement	Kgs	400	5.42	2169.60
1st Class Mason	day	0.067	285.00	19.10
2nd Class Mason	day	0.133	260	34.58
Mazdoor (Both Men and Women)	day	2.5	215	537.50
Labour for centering	Cum	1	1002	1002.00
Material hire charges for centering	Cum	1	893	893.00
Concrete Mixer 10 / 7 cft (0.2 / 0.8 cum) capacity	hour	0.267	248.40	66.32
Water (including for curing)	kl	1.2	77.00	92.40
Add 20% in Labour				497.24
Add MA 20%				358.08
Add TOT 4%				286.56
BASIC COST per 1 cum				**7450.37**

RCC Domed roof 100 mm thick				
20mm HBG graded metal	cum	0.9	1405.04	1264.54
Sand	cum	0.45	509.92	229.46
Cement	Kgs	400	5.42	2169.60
1st Class Mason	day	0.067	285.00	19.10
2nd Class Mason	day	0.133	260	34.58
Mazdoor (Both Men and Women)	day	2.5	215	537.50
Labour for centering	Sqm	10	1843	18430
Material hire charges for centering	Sqm	10	1915	19150
Concrete Mixer 10 / 7 cft (0.2 / 0.8 cum) capacity	hour	0.267	248.40	66.32
Water (including for curing)	kl	1.2	77.00	92.40
Add 20% in Labour				7634.236
Add MA 20%				9161.09
Add TOT 4%				2351.56
BASIC COST per 1 cum	day			**61,141**

C0NE SHAPED DOMICALSLAB AND INCLIND SLAB 200 mm thick				
n HBG graded metal	cum	0.9	1405.04	1264.54
l	cum	0.45	509.92	229.46
ent	Kgs	400	5.42	2169.60
lass Mason	day	0.067	285.00	19.10
Class Mason	day	0.133	260	34.58
door (Both Men and Women)	day	2.5	215	537.50
ur for centering	Sqm	5	1498	7490
rial hire charges for centering	Sqm	5	1915	9575
rete Mixer 10 / 7 cft (0.2 / 0.8 cum) capacity	hour	0.267	248.40	66.32
r (including for curing)	kl	1.2	77.00	92.40
20% in Labour				1616.23
MA 20%				1939.48
TOT 4%				
IC COST per 1 cum	day			**25035**

Description	Unit	Quantity	Rate Rs.	Amount Rs.
RCC CYLINDRICAL WALL				
20mm HBG graded metal	Cum	0.9	1405.04	1264.54
Sand	Cum	0.45	509.92	229.46
Cement	Kgs	400	5.42	2169.60
1st Class Mason	Day	0.167	285.00	47.60
2nd Class Mason	Day	0.167	260	43.42

Mazdoor (Both Men and Women)	Day	4.7	215	1010.50
Labour for centering	Cum	1	896	896.00
Material hire charges for centering	Cum	1	89	89.00
Concrete Mixer 10 / 7 cft (0.2 / 0.8 cum) capacity	Hour	1	248.40	248.40
Water (including for curing)	Kl	1.2	77.00	92.40
Add 20% in Labour (1st Floor)				399.50
Add MA 20%				479.40
Add TOT 4%				278.79
BASIC COST per 1 cum				**7249.00**

RCC RING BEAM AT BOTTOM OF CYLINDRICAL WALL				
20mm HBG graded metal	cum	0.9	1405.04	1264.54
Sand	cum	0.45	509.92	229.46
Cement	Kgs	400	5.42	2169.60
1st Class Mason	day	0.067	285.00	19.10
2nd Class Mason	day	0.133	260	34.58
Mazdoor (Both Men and Women)	day	2.5	215	537.50
Labour for centering	Cum	1	1113	1113.00
Material hire charges for centering	Cum	1	1276	1276.00
Concrete Mixer 10 / 7 cft (0.2 / 0.8 cum) capacity	hour	0.267	248.40	66.32
Water (including for curing)	kl	1.2	77.00	92.40
Add 20% in Labour				341
Add MA 20%				409.036
Add TOT 4%				302.10
BASIC COST per 1 cum				**7854.636**

RCC CIRCULAR GIRDER				
20mm HBG graded metal	cum	0.9	1405.04	1264.54
Sand	cum	0.45	509.92	229.46
Cement	Kgs	400	5.42	2169.60
1st Class Mason	day	0.067	285.00	19.10
2nd Class Mason	day	0.133	260	34.58
Mazdoor (Both Men and Women)	day	2.5	215	537.50
Labour for centering	Cum	1	751	751.00
Material hire charges for centering	Cum	1	893	893.00
Concrete Mixer 10 / 7 cft (0.2 / 0.8 cum) capacity	hour	0.267	248.40	66.32
Water (including for curing)	kl	1.2	77.00	92.40
Add 20% in Labour (1st Floor)				268.44
Add MA 20%				322.12
Add TOT 4%				265.92
BASIC COST per 1 cum				**6914.00**

RCC BRACING AT 4M HEIGHT				
20mm HBG graded metal	cum	0.9	1405.04	1264.54
Sand	cum	0.45	509.92	229.46
Cement	Kgs	400	5.42	2169.60
1st Class Mason	day	0.067	285.00	19.10
2nd Class Mason	day	0.133	260	34.58
Mazdoor (Both Men and Women)	day	2.5	215	537.50
Labour for centering	Cum	1	875	875.00
Material hire charges for centering	Cum	1	1276	1276.00
Concrete Mixer 10 / 7 cft (0.2 / 0.8 cum) capacity	hour	0.267	248.40	66.32
Water (including for curing)	kl	1.2	77.00	92.40

Add 20% in Labour (1st Floor)				293.23
Add MA 20%				351.88
Add TOT 4%				288.35
BASIC COST per 1 cum				**7497.22**

RCC BRACING AT 8M HEIGHT				
20mm HBG graded metal	cum	0.9	1405.04	1264.54
Sand	cum	0.45	509.92	229.46
Cement	Kgs	400	5.42	2169.60
1st Class Mason	day	0.067	285.00	19.10
2nd Class Mason	day	0.133	260	34.58
Mazdoor (Both Men and Women)	day	2.5	215	537.50
Labour for centering	Cum	1	954	954.00
Material hire charges for centering	Cum	1	1276	1276.00
Concrete Mixer 10 / 7 cft (0.2 / 0.8 cum) capacity	hour	0.267	248.40	66.32
Water (including for curing)	kl	1.2	77.00	92.40
Add 20% in Labour (1st Floor)				309.036
Add MA 20%				370.84
Add TOT 4%				292.93
BASIC COST per 1 cum				**7616.30**

Plastering with CM (1:3), 12 mm thick - 10 Sqm				
Cement Mortor (1:3)	cum	0.15	3191.00	478.65
Mason 1st class	day	0.6	285.00	171.00
Mazdoor (unskilled)	day	0.96	215	206.40
Add MA 20%				75.48
Add TOT 4%				37.26
Grand Total				**969.00**

Plastering with CM (1:6), 12 mm thick - 10 Sqm				
Cement Mortor (1:6)	cum	0.15	1889.00	283.35
Mason 1st class	day	0.6	285.00	171.00
Mazdoor (unskilled)	day	0.96	215	206.40
Add MA 20%				75.48
Add TOT 4%				29.45
Grand Total				**766.00**

Painting to new walls of tank portion with 2 coats of water proof cement paint of apporved brand and shade over a base coat of approved cement primer grade I making making 3 coats in all to give an even shade after thouroughly brushing the surface to remove all dirt and remains of loose powdered materials, including cost and conveyance of all materials to work site and all operational, incidental, labour charges etc. complete for finished item of work as per SS 912 for walls				
Epoxy primer for Hibond floor & protective coatings : Procoat SNP2 or Zoriprime EFC 2	Pack	**1**	**725.00**	725
1st class painter	day	0.21	285.00	59.85
2nd class painter	day	0.49	260	127.40
cost of water proof cement paint	1.00 cum (35.28 cft)	3.50	35	122.50
1st class painter	day	0.15	285.00	42.75

2nd class painter	day	0.35	260	91.00
Mazdoor (unskilled)	day	1.50	215	322.50
Add MA 20%				128.70
Add TOT 4%				39.79
Total cost/ 10 sqm				**1660**

Painting to new columns with 2 coats of water proof cement paint of apporved brand and shade over a base coat of approved cement primer grade I making making 3 coats in all to give an even shade after thouroughly brushing the surface to remove all dirt and remains of loose powdered materials, including cost and conveyance of all materials to work site and all operational, incidental, labour charges etc. complete for finished item of work as per SS 912 for walls				
Cost of Cement Primer	kg	1.00	100	100.00
1st class painter	day	0.21	285.00	59.85
2nd class painter	day	0.49	260	127.40
cost of water proof cement paint	1.00 cum (35.28 cft)	3.50	35	122.50
1st class painter	day	0.15	285.00	42.75
2nd class painter	day	0.35	260	91.00
Mazdoor (unskilled)	day	1.50	215	322.50
Add MA 20%				128.70
Add TOT 4%				39.79
Total cost/ 10 sqm				**1035.00**

S.no	Quantity	Description	per	rate	Amount
1	25.728 Cum	V.R.C.C (1:1 1/2 :3) 20mm size HBG, machine crushed chips including cost, seignorage and conveyance of all materials and labour charges such as Machine mixing, vibrating, curing etc., -Foundation - SF	1 cum	5538.00	142,482
2	23.069 Cum	V.R.C.C (1:1 1/2 :3) 20mm size HBG, machine crushed chips including cost, seignorage and conveyance of all materials and labour charges such as Machine mixing, vibrating, curing etc., -columns - SF	1 cum	7383.144	170,322
3	0.848 cumec	V.R.C.C (1:1 1/2 :3) 20mm size HBG, machine crushed chips including cost, seignorage and conveyance of all materials and labour charges such as Machine mixing, vibrating, curing etc., -Ring beam at top - SF	1 cum	7450.37	6,318
4		V.R.C.C (1:1 1/2 :3) 20mm size HBG, machine crushed chips including cost, seignorage and conveyance of all materials and			

	9.995 Cum	labour charges such as Machine mixing, vibrating, curing etc., -domical roof - SF	1 cum	61,141	6,11,105
5	7.752 Cum	V.R.C.C (1:1 1/2 :3) 20mm size HBG, machine crushed chips including cost, seignorage and conveyance of all materials and labour charges such as Machine mixing, vibrating, curing etc., -conical dome base slab - SF	1 cum	25,035	1,94,072
6	126.635 cum	V.R.C.C (1:1 1/2 :3) 20mm size HBG, machine crushed chips including cost, seignorage and conveyance of all materials and labour charges such as Machine mixing, vibrating, curing etc., -cylindrical wall - SF	1 cum	7249	9,17,978
7	3.675 Cum	V.R.C.C (1:1 1/2 :3) 20mm size HBG, machine crushed chips including cost, seignorage and conveyance of all materials and labour charges such as Machine mixing, vibrating, curing etc., -ring beam at bottom of cylindrical wall - SF	1 cum	7,854.636	28,866
8	3.845 cum	V.R.C.C (1:1 1/2 :3) 20mm size HBG, machine crushed chips including cost, seignorage and conveyance of all materials and labour charges such as Machine mixing, vibrating, curing etc., -circular girder - SF	1 cum	6,914	26,585
9	11.751 cum	V.R.C.C (1:1 1/2 :3) 20mm size HBG, machine crushed chips including cost seignorage and conveyance of all materials and labour charges such as Machine mixing, vibrating, curing etc., - inclind cone shaped slab - SF	1 cum	25,035	29,4,187
10	1.668 cum	V.R.C.C (1:1 1/2 :3) 20mm size HBG, machine crushed chips including cost seignorage and conveyance of all materials and labour charges such as Machine mixing, vibrating, curing etc., - Bracing at 4m heigh - SFt	1 cum	7,498	12,507

S.no	Quantity	Description	per	rate	Amount
11	1.555 cum	V.R.C.C (1:1 1/2 :3) 20mm size HBG, machine crushed chips including cost seignorage and conveyance of all materials and labour charges such as Machine mixing, vibrating, curing etc., - Bracing at 8m heigh - SFt	1 cum	7617	11,845
12	41.45 mt	Supplying,placing and fitting of HYSD bars reinforcement, complete as per drawings and technical specifications for bars below 36 mm dia including over laps and wastage, wherethey are not welded-SF	1 mt	55419.00	22,97240
13	357.29 sqm	Plastering inside 12mm thick in single coat in cm (1:3) with finishing including of cost of conveyance of all materials and water to work site and all operational incidental labour charges such as scaffolding. Mixing mortar ,curing etc., complete for finished item of work.-SF	10 sqm	969	34,622
14	652.84 sqm	Plastering outside 12mm thick in single coat in cm (1:6) with finishing including of cost of conveyance of all materials and water to work site and all operational incidental labour charges such as scaffolding. Mixing mortar ,curing etc., complete for finished item of work.-SF	10 sqm	766	50,000
15	647.174 sqm	Painting to new outer walls with 2 coats of Epoxy primer for Hibond floor & protective coatings : Procoat SNP2 or Zoriprime EFC 2 approved brand and shade over primary coat, after thoroughly brushing thr surface yo removing of loose powdered materials and all operational incidental labour charges etc.,completed for finished item of work-SF	10 sqm	1660	10,431
		olumns with 2 coats of water proof cement paint of apporved brand and shade over a base coat of approved cement primer grade I making making 3 coats in all to give an even shade after thouroughly brushing the surface to remove all dirt and remains of loose powdered materials, including cost and conveyance of all materials to work site and all			

16	271.433 sqm	operational, incidental, labour charges etc. complete for finished item of work as per SS 912 for walls	10 sqm	1035	28,152

Total	47,33,576
Eath work	12,336
Over all cost	47,45,912
Add 10% contractors profit	4,74,591
Total cost	52,20,503

Total cost estimated by the reference of standard scheduled rates (SSR 2011-2012).

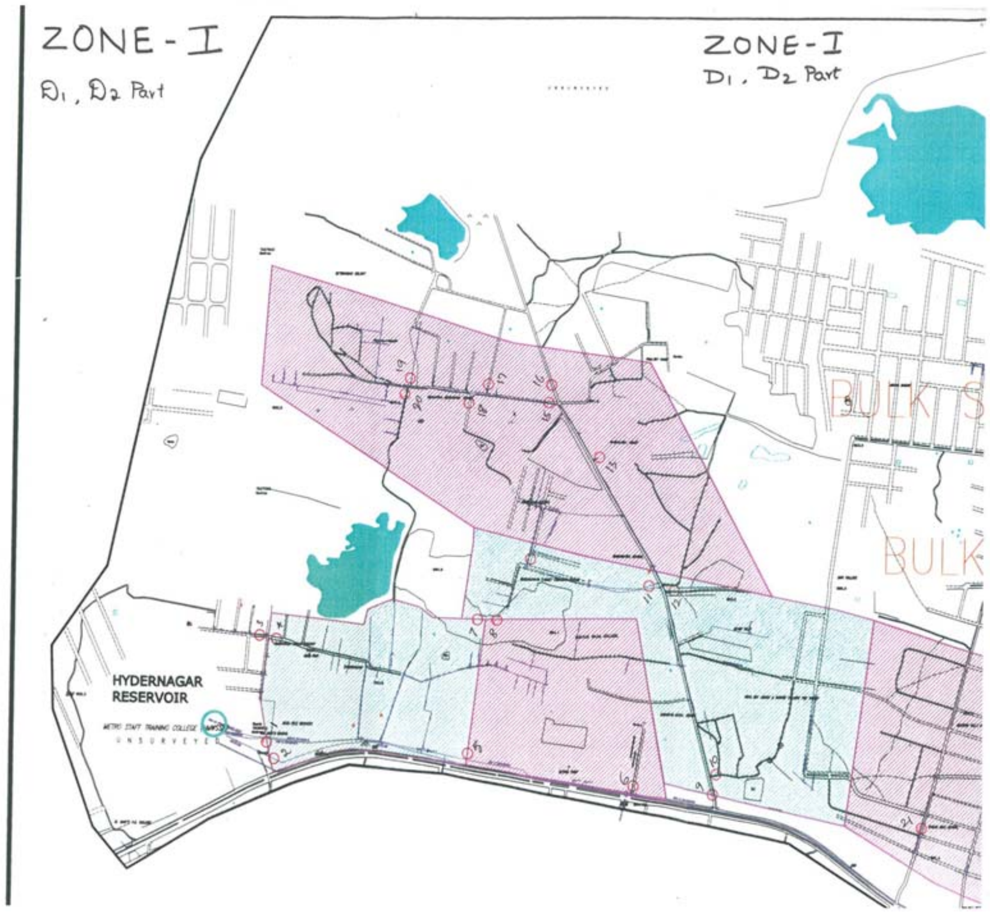
REFERANCES:

Table 16.2. Coefficients for moment in cylindrical wall fixed at base (Per IS3370)
Moment = Coefficient (wH^3) Nm/m

$\frac{H^2}{DT}$	Co efficient at points							
	0.1 H	*0.2 H*	*0.3 H*	*0.4 H*	*0.5 H*	*0.6 H*	*0.7 H*	*0.8H*
0.4	+ 0.0005	+ 0.0014	+ 0.0021	+ 0.0007	- 0.0042	-0.0150	-0.0302	-0.0529
0.8	+0.0011	+ 0.0037	+ 0.0063	+ 0.0080	+ 0.0070	+ 0.0023	+ 0.0068	-0.0024
1.2	+ 0.0012	+ 0.0042	+ 0.0077	+ 0.0103	+ 0 0 1 1 2	+ 0.0090	+ 0.0022	-0.0108
1.6	+ 0.0011	+ 0.0041	+ 0.0075	+ 0.0107	+ 0.0121	+ 0.0111	+ 0.0058	-0.0051
2.0	+ 0.0010	+ 0.0035	+ 0.0068	+ 0.0099	+ 0.0120	+0.0115	+ 0.0075	-0.0021
3.0	+ 0.0006	+ 0.0024	+ 0.0047	+ 0.0071	+ 0.0090	+ 0.0097	+ 0.0077	+0.0012
4.0	+ 0.0003	+ 0.0015	+ 0.0028	+ 0.0047	+ 0.0066	+ 0.0077	+ 0.0069	+0.0023
5.0	+ 0.0002	+ 0.0008	+ 0.0016	+ 0.0029	+ 0.0046	+ 0.0059	+ 0.0059	+0.0028
6.0	+ 0.0001	+ 0.0003	+ 0.0008	+ 0.0019	+ 0.0032	+ 0.0046	+ 0.0051	+0.0029
8.0	0.0000	+ 0.0001	+ 0.0002	+ 0.0008	+ 0.0016	+ 0.0028	+ 0.0038	+0.0029
10.0	0.0000	+ 0.0000	+ 0 0001	+ 0.0004	+ 0.0007	+ 0.0019	+ 0.0029	+0.0028
12.0	0.0000	+ 0.0000	+ 0.0001	+ 0.0002	+ 0.0003	+ 0.0013	+ 0.0023	+0.0026
14.0	0.0000	0.0000	0.0000	0.0000	+ 0.0001	+ 0.0008	+ 0.0019	+0.0023
16.0	0.0000	0.0000	-0.0001	- 0.0002	-0.0001	+ 0.0004	+ 0.0013	+0.0019

REFERENCE BOOKS:

- I.S 496:2000 for RCC.
- I.S 800:1984 for STEEL.
- I.S 872 Part I and Part II.
- I.S 3373 (Part IV-1967).
- Rein force concrete structures (Dr B.C PUNMIA).
- Element of environmental engineering (BIRIDI).



VINAYAK NAGAR COLONY

CONCLUSION

Storage of water in the form of tanks for drinking and washing purposes, swimming pools for exercise and enjoyment, and sewage sedimentation tanks are gaining increasing importance in the present day life. For small capacities we go for rectangular water tanks while for bigger capacities we provide circular water tanks.

Design of water tank is a very tedious method. With out power also we can consume water by gravitational force.