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## PLANNING, ANALYSING AND DESIGNING OF A INTZE TYPE OVERHEAD TANK FOR DAMU NAGAR

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**Abstract :** Now-a-days, along with the growth of population there is an increased demand for water. The rapid increased demands for irrigation, human and industrial consumption, the available water resources are getting depleted in many parts of the world. The traditional methods of water quality control involve the manual collection of water at various locations. In this project the tank is designed using the working stress method and the columns, beams and footings are designed using limit state method. For the design of tank IS3370 part 4 codes and for the design of columns and footings IS456 code were referred. All drafting and detailed drawing is carried out in AutoCAD software and analysis is carried out using STAAD pro V8i software. The materials used are M20 and Fe415 for reinforced concrete members. **Key words:** Water resource, Overhead tank, intze tank, Population forecasting, Dome shape. **Keywords—**component, formatting, style, styling, insert (key words).

### 1 INTRODUCTION

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### 1.2 OBJECTIVES

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.

### 1.3 SOURCES 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
4. Oceans, generally not used for water supplies, at present. Sub-surface sources or underground sources, such as
  1. Springs
  2. Infiltration wells
  3. Wells and Tube-wells.

## 1.4 CLASSIFICATIONS OF WATER TANKS

Classification based on under three heads:

1. Tanks resting on ground
2. Elevated tanks supported on staggings
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

## 2 STUDY OF TANK DESIGN

### 2.1 WATER DEMAND

#### 2.1.1 Water 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 liters per day per head Population to be served. Quantity= Per demand x Population

#### 2.1.2 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 demand, which a city may have, may be broken into following class

#### 2.1.3 Water Consumption for Various Purposes

TABLE 2.1 – Water Consumption for Various Purpose

	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

#### 2.1.4 Factors affecting per capita demand

1. Size of the city
2. Presence of industries.
3. Climatic conditions.
4. Habits of economic status.
5. Quality of water
6. Cost of water.
7. Policy of metering and charging method

## 2.2 POPULATION FORECAST

### 2.2.1 Design 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:

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

### 2.2.2 Population Forecasting Methods

The various methods adopted for estimating future populations are given below.

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

### 2.3 DESIGN REQUIREMENT OF CONCRETE (I. S. I)

In water retaining structure a dense impermeable concrete is required therefore, proportion of fine and coarse aggregates to cement should be such as to give high quality concrete. Concrete mix lesser than M20 is not used. The minimum quantity of cement in the concrete mix shall be not less than 30 kN/m<sup>3</sup>. The design of the concrete mix shall be such that the resultant concrete 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 with impermissible limits. A reinforced concrete member of liquid retaining structure is designed on the usual principles ignoring tensile resistance of concrete in bending. 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 of cracking 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. 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  $11 \times 10^{-6} / ^\circ \text{C}$  and coefficient of shrinkage may be taken as  $450 \times 10^{-6}$  for initial shrinkage and  $200 \times 10^{-6}$  for drying shrinkage.

### 2.4 GENERAL DESIGN REQUIREMENTS (I.S.I)

#### 2.4.1 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.

#### 2.4.2 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 conform to the values specified. The permissible tensile stresses due to bending apply to the face of the member in contact with the liquid. In members less than 225mm  $\phi$  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: 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.

#### 2.4.3 Permissible 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:
  - a) Tensile stress in member in direct tension 1000 kg/cm<sup>2</sup>.
  - b) 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/cm<sup>2</sup>.
  - c) On face away from liquid for members 225mm or more in thickness 1250 kg/cm<sup>2</sup>.
  - d) Tensile stress in shear reinforcement for members less than 225mm thickness 1000 kg/cm<sup>2</sup> for members 225mm or more in thickness 1250 kg/cm<sup>2</sup>.
  - e) Compressive stress in columns subjected to direct load 1250 kg/cm<sup>2</sup>.

#### 2.4.4 Stresses due to drying Shrinkage or Temperature Change

- (i) Stresses due to drying shrinkage or temperature change may be ignored provided that
- (b) Adequate precautions are taken to avoid cracking of concrete during the construction period and until the reservoir is put into use.
- (c) 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  $300 \times 10^{-6}$  may be assumed.
- (iii) When the shrinkage stresses are allowed, the permissible stresses, tensile stresses to concrete (direct and bending) may be increased by 33.33 per cent.

### 2.5 WALLS

#### 2.5.1 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.

#### 2.5.2 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.

#### 2.5.3 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.

### 2.6 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 conical 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 (conical dome). Both types may or may not have asymmetrical 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, planetarium 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.

#### 2.6.1 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. 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 1.1N/mm<sup>2</sup> to 1.70 N/mm<sup>2</sup> for direct tension and 1.5 N/mm<sup>2</sup> to 2.40 N/mm<sup>2</sup> 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/mm<sup>2</sup> (or 150N/mm<sup>2</sup>) in case of liquid retaining structures and 140 N/mm<sup>2</sup> (or190 N/ mm<sup>2</sup>) in other cases. The allowable stress specified in IS 3370 for such tanks are as follows: 11 Type of stresses: Permissible stress in N/mm<sup>2</sup> High yield strength Plain bars conforming to deformed bars as per Grade-I of IS 432-1966. IS 1786-1966 or is 1139-1966. Tensile stress in members under no table of contents entries found direct load. Direct tensile stress in concrete may be taken as 1.1 N/mm<sup>2</sup>, 1.2. N/mm<sup>2</sup> 1.32 N/mm<sup>2</sup>, 1.5 N/mm<sup>2</sup>, 1.6N/mm<sup>2</sup> and 1.7 N/mm<sup>2</sup> for M15, M20, MM25, M30, M35and M40 respectively, the value in tension due to bending i.e., being 1.5N/mm<sup>2</sup> ,1.7N/mm<sup>2</sup> ,1.82N/mm<sup>2</sup> ,2.0 N/mm<sup>2</sup> ,2.2 N/mm<sup>2</sup> and 2.4 N/mm<sup>2</sup> .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. Minimum reinforcement of each of two directions at right angles shall have an are 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.

### 3 STUDY OF COMPONENTS OF TANK

#### 3.1 INTZE WATER TANK

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 a result of which the ring girder supported on the columns will be relieved from the horizontal thrusts as the horizontal 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.

#### 3.2 COMPONENTS OF TANK

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 1/5th 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 intervals and have to be designed for wind pressure or seismic loads whichever govern.
- (9) Foundations: A combined footing is usual 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.

### 4 DESIGN

#### 4.1 ESTIMATION OF POPULATION

YEAR	POPULATION	INCREASE IN POPULATION
1991	500	-
2001	700	200
2010	1000	300

#### 4.1.1 ARITHMETIC INCREASE METHOD

TABLE 5.1 Population Forecasting-Arithmetic Increase Method

$X = 250$ ,  
 $n = 3$   
 $P_n = 1750$

#### 4.1.2 GEOMETRICAL INCREASE METHOD

YEAR	POPULATION	INCREASE IN POPULATION	PERCENTAGE INCREASE
1991	500	-	-
2001	700	200	40%
2010	1000	300	42.86%

TABLE 5.2 Population Forecasting- Geometrical Increase Method

$P_n = 1330$

#### 4.1.3 INCREMENTAL INCREASE METHOD

YEAR	POPULATION	INCREASE IN POPULATION	INCREMENTAL INCREASE
1991	500	-	-
2001	700	200	-
2010	1000	300	100

TABLE 5.3 population Forecasting-Incremental Increase Method

$X=250$   $Y=100$

$P_n = 2350$

App,  $P = 2000$

The water demand for domestic usage is 160 lpcd  
 WATER DEMAND  $\approx 400000$  LITRES

#### 4.2 DESIGN PARAMETERS

Design of an intze tank for a capacity of 400000 liters.

Assuming height of tank floor above the ground level is 11.9m.

Safe bearing capacity of soil 200kN/m<sup>2</sup>

Wind pressure as per IS875 1200N/m<sup>2</sup>

Assuming M20 concrete

For which  $\sigma_{cbe} = 7\text{N/mm}^2$ ,  $\sigma_{cc} = 5\text{N/mm}^2$

Direct tension  $\sigma_t = 5\text{N/mm}^2$

Tension in bending = 1.70 N/mm<sup>2</sup>

Modular ratio  $m = 13$

For Steel stress,

Tensile stress in direct tension = 115 N/mm<sup>2</sup>

Tensile stress in bending on liquid face = 115 N/mm<sup>2</sup> for  $t < 225$  mm and 125 N/mm<sup>2</sup> for  $> 225$  mm

#### 4.3 DIMENSIONS OF THE TANK

Consider

Diameter of the tank,  $D = 9.5$  m,

Height of the tank,  $H = 4.75$ m,

Height of the conical portion,  $h = 1.9$ m,

Diameter at the bottom of the tank,  $d = 5.7$ m,

assume  $h_1 = 1$ m,

Volume of the cylindrical portion,  $V_1 = 336.69$  m<sup>3</sup>

Volume of the conical portion,  $V_2 = 86.68$  m<sup>3</sup>

Volume of the sphere segment,  $V_3 = 13.19$ m<sup>3</sup>

Total volume,  $V = 410.2$ m<sup>3</sup>  $\approx 400$ m<sup>3</sup>.

Hence the assumed section gives the required capacity.

#### 4.4 DESIGN OF ROOF DOME

Considering a rise of 1.60 m,  
radius of the roof dome is given from  $= (4.75)^2$   
 $R = 7.85\text{m}$ .  
 $\sin \phi = 0.605$  and  
 $\phi = 37.24^\circ < 51.8^\circ$   
Hence no tension Assuming  $t = 150\text{mm}$ .

5.4.1 Hoop stress @ level of springing:

$$f = 0.0619 \text{ N/mm}^2$$

Hoop stress @ Crown:

$$\theta = 0^\circ$$

$$f = 0.129 \text{ N/mm}^2$$

Meridinal thrust @ level of spring:

$$T = 0.0216 \text{ N}$$

Compressive stress,  $C = 0.144 \text{ N/mm}^2$

Minimum Reinforcement

$$A_t = 0.2\% \text{ of area} = 300\text{mm}^2$$

Provide 8mm  $\phi$  bar @ 160mm c/c

#### 4.5 RING BEAM @ TOP

Horizontal component of  $H = 17224.2 \text{ N/m}^2$

Hoop stress in the ring beam = 81814.95 N

Area of steel required = 654.5 mm<sup>2</sup>

We have to provide 6 bars of 12 mm  $\phi$  @ 452.38 mm c/c

5.5.1 Size of the ring beam:

$$\sigma_{ct} = 1.2 \text{ N/mm}^2 \text{ AC}$$

$$A_c = 696 \text{ mm}^2$$

Provide 300mm  $\times$  300mm

#### 4.6 CYLINDRICAL WALL

Pressure intensity at the bottom of cylindrical wall = 46597.5 N/mm<sup>2</sup>

Consider bottom strip of the wall as 1 mm.

Hoop tension =  $P \times G @ = 221338.13 \text{ N}$

$$A_{st} = 885.35 \text{ mm}^2$$

Provide 8 bars of 12 mm diameter of 130 mm c/c

##### 4.6.1 Thickness of the wall

$$\sigma_{ct} = 221338.13$$

$$t_w = 173.6\text{mm}$$

provide  $t = 250\text{mm}$

Distribution steel = 750 mm<sup>2</sup>

Provide 10 mm diameter bars of spacing 105 mm c/c

##### 4.6.2 Check for compressive stress at the bottom of the cylindrical wall.

Vertical component  $V_1 = 13092.8 \text{ N}$ .

Weight of the wall = 28500 N.

Weight of ring beam = 2160 N.

Total load  $V_2 = 43752.8 \text{ N}$ .

Compressive stress = 0.18 N/mm<sup>2</sup>

Nominal vertical stress is equal to 0.24% of gross area. Vertical steel = 600 mm<sup>2</sup>

Provide 10 bars of 8 mm diameter of spacing 100 mm.

#### 4.7 RING BEAM AT B

Let  $T_2$  be the thrust /m run exerted by the conical wall at the junction B.

Resolving vertically at B

$$T_2 = 61875.8 \text{ N.}$$

Resolving horizontally at B

$$H_2 = 43752.8 \text{ N}$$

This horizontal load  $H_2$  will produce a hoop tension in ring beam B

$$\text{Hoop tension due to } H_2 = 207825.8 \text{ N}$$

Let the ring beam be 500mm deep

$$\text{Water pressure on the ring beam} = 23298.75 \text{ N/m}^2$$

$$\text{Hoop tension due to water} = 110669 \text{ N}$$

$$\text{Total hoop tension} = 318494.86 \text{ N}$$

$$A_{st} = 1254.96 \text{ mm}^2 .$$

Provide 10 numbers of 18mm diameter bar @ 100mm c/c

$$A_{st \text{ prov}} = 2544.6 \text{ mm}^2$$

#### 5.7.1 Size of the ring beam

$$\sigma_{ct} = 318494.86$$

$$A_c = 234877.18 \text{ mm}^2$$

Provide 500mm×500mm size

### 4.8 DESIGN OF CONICAL SLAB

Conical slab should be designed for

a) Hoop tension

b) Bending as it spans on a sloping slab from the ring beam @ B at the ring girder

$$\omega_s = 2305080.5 \text{ N}$$

$$\text{Length of conical slab} = 2.36 \text{ m.}$$

Take thickness of the slab as 200 mm.

$$\text{Weight of the conical slab } T_s = 252674.52 \text{ N.}$$

$$\text{Hoop tension} = 773944.3 \text{ N.}$$

$$\text{Hoop steel on the entire section} = 3738.96 \text{ mm}^2 .$$

Provide 20 bars of 16 mm  $\phi$

$$A_{st \text{ prov}} = 2814.86 \text{ mm}^2 .$$

#### 4.8.1 Design for bending moment

$$\text{Load per meter width of the conical slab} = 53563.07 \text{ N.}$$

$$\text{Maximum bending moment} = 12721.23 \text{ Nm.}$$

$$\text{Axial compression } e_2 = T_2 \sin \alpha \quad T_2 = f @ NQRS = 1E63 @ .8 \quad 4.646 = 61875.80 \text{ N.}$$

Providing 16 mm diameter bar at clear covers of spacing 25 mm.

$$\text{Effective depth} = 167 \text{ mm.}$$

$$\text{Distance between centre of section and centre of steel } x = 67 \text{ mm}$$

$$\text{Resultant bending moment} = 16866908.6 \text{ Nmm.}$$

$$A_{st} = 487.92 \text{ mm}^2$$

Provide 3 bars of 16 mm diameter bars 420mm c/c

### 4.9 THE BOTTOM DOME

Let R be the radius of the dome = 4.06 m.

Let  $\theta$  be the angle subtended by the dome = 0.712

Thickness of dome = 200 mm

#### 4.9.1 Loads

$$\text{Dead load} = 4800 \text{ N/mm}^2$$

$$\text{Weight of water resting on the dome} = 1696900 \text{ N}$$

$$\text{Area of dome surface} = 25.5 \text{ m}^2 .$$

$$\text{Load intensity due to weight of water} = 66545.09 \text{ N/m}^2 . \text{ Total load intensity} = 71345.09 \text{ N/m}^2 .$$

$$\text{Meridinal thrust} = 169194.55 \text{ N/m.}$$

$$V_3 = 69E13.42 ] \times 3.6 = 3984.78 \text{ N } 39$$

$$\text{Meridinal compressive stress} = 0.85 \text{ N/mm}^2$$

$$\text{Hoop stress} = 0.185 \text{ N/mm}^2 .$$

Hoop stress at the crown  $\theta = 0^\circ$ .

$$\text{Maximum hoop stress} = 0.724 \text{ N/mm}^2$$

These stresses are low and hence provide nominal 0.3% steel.

$$A_t = 600 \text{ mm}^2$$

Provide 8 mm  $\phi$  bars @ 90 mm spacing



#### 4.9.2 Circular girder

The total load on the circular girder consists of the following

Total weight of water  $W_1 = 4001980.5\text{N}$

Weight of dome + cylindrical wall + ring beam at A  $W_2 = 1305808.018\text{ N}$ .

Weight of ring beam at B  $W_3 = 169646\text{ N}$ .

Weight of conical wall  $W_4 = 252674.52\text{N}$

Weight of lower dome  $W_5 = 122400\text{ N}$ .

Weight of girder =  $125097.37\text{ N}$ .

Torsion =  $5852696\text{ N}$ .

Angular distance for maximum torsion =  $12.44^\circ$  Let us provide 8 columns.

Bending moment at the support =  $138445.5\text{ Nm}$ .

Bending moment at centre =  $69389.56\text{ Nm}$ .

Torsion =  $10008.1\text{ N}$ .

Angular distance for maximum torsion =  $3^\circ 10'$

Load at each support =  $8 = 731587.01\text{N}$ .

Shear force at the support  $V = 2926348.07\text{N}$

#### 4.9.3 Ring Beam

Total vertical load =  $47736.98\text{N}$

Horizontal load =  $39768.6\text{N}$

##### 4.9.3.1 Trial section

Assume  $500\text{mm} \times 1000\text{mm}$

Self weight =  $12000\text{N}$  Vertical load per m =  $59736.98\text{N}$

#### 4.10 COLUMNS

Columns should be designed for direct loads coming upon them and for the bending moments caused by wind load.

Vertical load on one column at top =  $731587.02\text{ N}$ .

length of column =  $10\text{ m}$ .

Providing  $600\text{ mm} \times 600\text{ mm}$  column.

Let number of columns =  $8$

Weight Of column =  $69120\text{ N}$

Total vertical load =  $800707.02\text{ N}$

Corresponding axial load =  $804730.67\text{ N}$

Weight of water in tank =  $500247.56\text{ N}$

on each column Vertical load on each column when tank is empty =  $300459.46\text{ N}$

Corresponding axial load =  $238553.805\text{ N}$

Ignoring wind load effect if the steel requirement is  $A_{sc}$  Then  $cA_c + tA_{sc} = 858730.67$

$A_{sc} = 3260.57\text{mm}^2$ .

Min. Requirement of steel =  $2000\text{ mm}^2$

Provide 7 bars of  $20\text{mm}$  diameter

$A_{st} = 2199.11\text{mm}^2$

More steel has been provide since column is subjected to B.M caused by wind load.

##### 4.10.1 Analysis due to wind pressure

Wind pr. =  $1200\text{ mm}^2$ .

Wind force on the top dome & cylindrical walls =  $14.745\text{m}$

Wind force on the circular wall =  $94. @ > .1 @ \times 1.9 \times 1200 = 18924\text{ N}$

Wind force on circular girder =  $0.6 \times 6.4 \times 1200 = 4608\text{ N}$

Wind force on column & braces =  $41850\text{ N}$

Total moment of wind pr. About the base =  $5 = 1478114.28\text{Nm}$ .

Vertical load on any column due to wind load =  $96\text{m}^2$

Max. Wind load force in the most leeward side & the most windward side =  $61588.095\text{ N}$

$4\text{ columns} = 43549.35\text{N}$

Maximum axial load due to wind load & dead loads =  $362047.55\text{N}$

Corresponding axial load =  $363866.89\text{ N}$

Horizontal component of the axial forces caused by wind action =  $133314\text{ N}$

Actual horizontal force @ base =  $89764.64\text{N}$

Horizontal shear column =  $11220.58\text{ N}$ .

Maximum bending moment for the column = 19636.015N.

#### 4.10.2 Analysis of column section

Direct load = 363866.89N.

Bending moment = 19636.015 Nm.

Provide 600×600 column.

Provide 8bars of 20 mm diameter at effective cover of 50 mm.

$A_{st} = 2513.27 \text{ mm}^2$  ,

Equivalent concrete area = 390159.29 N

Polar moment of inertia of the equivalent concrete section =  $1.885 \times 10^9 \text{ mm}^4$  .

on each column Vertical load on each column when tank is empty = 300459.46 N

Corresponding axial load= 238553.805 N

Ignoring wind load effect if the steel requirement is  $A_{sc}$  Then  $cA_c + tA_{sc} = 858730.67$

$A_{sc} = 3260.57 \text{ mm}^2$  .

Min. Requirement of steel = 2000  $\text{mm}^2$

Provide 7 bars of 20mm diameter

$A_{st} = 2199.11 \text{ mm}^2$

More steel has been provide since column is subjected to B.M caused by wind load.

#### 4.10.3 Analysis due to wind pressure

Wind pr. = 1200  $\text{mm}^2$  .

Wind force on the top dome & cylindrical walls = 14.745m

Wind force on the circular wall =  $94. @ > .1 @ \times 1.9 \times 1200 = 18924 \text{ N}$

Wind force on circular girder =  $0.6 \times 6.4 \times 1200 = 4608 \text{ N}$

Wind force on column & braces = 41850 N

Total moment of wind pr. About the base =  $5 = 1478114.28 \text{ Nm}$ .

Vertical load on any column due to wind load = 96m<sup>2</sup>

Max. Wind load force in the most leeward side & the most windward side = 61588.095 N

4columns = 43549.35N

Maximum axial load due to wind load & dead loads = 362047.55N

Corresponding axial load = 363866.89 N

Horizontal component of the axial forces caused by wind action = 133314 N

Actual horizontal force @ base = 89764.64N

Horizontal shear column = 11220.58 N.

Maximum bending moment for the column = 19636.015N.

#### 4.10.4 Analysis of column section

Direct load = 363866.89N.

Bending moment = 19636.015 Nm.

Provide 600×600 column.

Provide 8bars of 20 mm diameter at effective cover of 50 mm.

$A_{st} = 2513.27 \text{ mm}^2$  ,

Equivalent concrete area = 390159.29 N

Polar moment of inertia of the equivalent concrete section =  $1.885 \times 10^9 \text{ mm}^4$  .

Equivalent moment of inertia about full section =  $942.62 \times 10^6 \text{ mm}^4$  .

Direct stress in concrete =  $0.93 \text{ N/mm}^2$  .

Bending stress in concrete =  $6.25 \text{ N/mm}^2$  .

Maximum stress =  $7.18 \text{ N/mm}^2$  .

#### 5.11.1 DESIGN OF BRACES

Moment in brace = 55530.65 Nm.

Provide 500×500 mm bar section and a doubly reinforced beam with equal steel at top and bottom.

$A_{st} = A_{sc} = 944.39 \text{ mm}^2$  .

Provide 4 bars of 18 mm diameter at top and equal amount at bottom.

Shear force for brace = 44925.24 N.

Nominal shear stress  $v_v = 0.195 \text{ N/mm}^2$  .

Provide nominal stirrups of 10 mm diameter stirrups at 200 mm clear cover.

#### 5.11.2 DESIGN OF FOUNDATION

Total load on the column = 6869845.36 N.

Approximate weight of foundation is 10% of column loads. = 686984.536 N.

Then total load is equal to 7556829.89 N.

Safe bearing capacity of 200 KN/m<sup>2</sup> ,

Area = 37.78m<sup>2</sup> .

Let us provide outer dia of 9 m and inner dia of 5 m.

Area = 43.98 m<sup>2</sup> .

Net intensity = 171.8 KN/m<sup>2</sup> < 200 KN/m<sup>2</sup> .

### 5.11.3 Design of circular girder

Maximum bending moment occurs at support = 162506.192 Nm.

Maximum torsion = 11747.44 Nm

Maximum shear force at support = 429365.335 N.

### 5.11.4 Design at support section

Moment of resistance = maximum bending moment at support.

$bd^2 = 177991448$

$d = 590$  mm

clear cover = 60 mm

$D = 650$  mm.

Equivalent shear stress  $V_v = 466957.143$  N

Equivalent nominal shear = 1.58 N/mm<sup>2</sup>

but  $y_c = 1.8$  N/mm<sup>2</sup>

Hence  $y_v < y_c$

#### 5.11.5.1 Longitudinal reinforcement

$M_e = M + M_t$

$M_t = 15893595.29$  N

$M_e = 178399.787 \times 1000$  N

$A_{st} = 1460.7$  mm<sup>2</sup>

Provide 9 bars of 16 mm diameter Hence area of steel required = 1809.55 mm<sup>2</sup> .

5.12.2.2 Transverse reinforcement:

providing 4 legged 10 mm diameter of stirrups.

$A_{sv} = 314$  mm<sup>2</sup>

$b_1 = 420$  mm ,

$d_1 = 530$  mm,

$314 = S_v[0.229 + 1.4]$  ,

$S_v = 192.76$  mm.

Let us provide 180 mm clear cover

5.12.3 Design of bottom slab:

Provide a cantilever projection beyond the face of the beam = 0.6 m.

Maximum bending moment for 1 m wide stirrup = 30928.36 Nm.

Equating moment of resistance to bending moment ,

$b = 1000$  mm.

Then  $d_2 = 33875.5$

$d = 184$  mm.

Let us provide 190 mm effective depth and 40 mm clear cover.

$D = 230$  mm

$A_{st} = 786.38$  mm<sup>2</sup> .

Provide 4 bars of 18 mm diameter

$A_{st} = 1017.87$  mm<sup>2</sup> and spacing of the bars is 250 mm clear cover. distribution steel:

Provide 0.12 % steel

$A_{st} = 276$  mm<sup>2</sup>

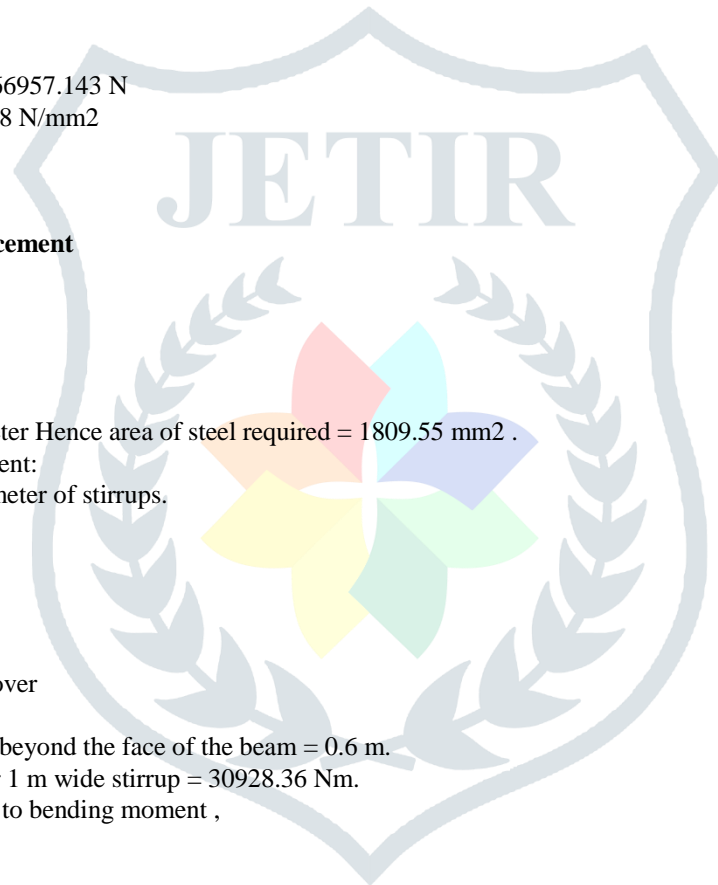
Provide 6 bars of 8 mm diameter bars and spacing 160 mm clear cover

### 5.12 CHECK FOR SLIDING

Total load on the foundation when tank is empty = 3554849.39 N

Horizontal force on the base = 89764.64 N

Let coefficient of friction = 19.8



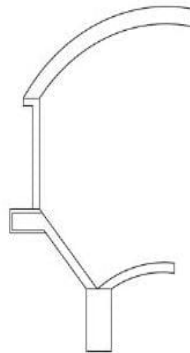
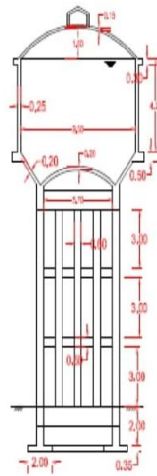
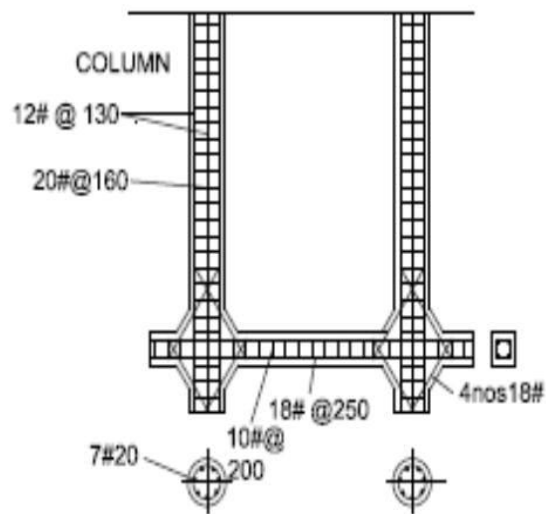


Figure 5.2 REINFORCEMENT DETAILS 1



Columns and bracings

FIGURE 5.3 REINFORCEMENT DETAILS 2

## 6 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.

Intze tank is constructed to minimize the project cost why because lower dome in this construction resists the horizontal thrust. Thus it is economically suitable.

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