ANALYSIS AND DESIGN OF ELEVATED R.C.C. WATER TANK

A
PROJECT REPORT

Submitted in partial fulfilment of the requirements for the award of the degree
Of

BACHELOR OF TECHNOLOGY
IN
CIVIL ENGINEERING

Under the supervision
of

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STUDENT'S DECLARATION

I hereby declare that the work presented in the Project report titled **“Analysis and Design of Elevated R.C.C. Water Tank”** submitted for partial fulfilment of the requirements for the degree of Bachelor of Technology in Civil Engineering at **Jaypee University of Information Technology, Waknaghat (H.P.)** is an authentic record of my work carried out under the supervision of **Mr. Anirban Dhulia**. This work has not been submitted elsewhere for the reward of any other degree/diploma. I am fully responsible for the contents of my project report.

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CERTIFICATE

This is to certify that the work which is being presented in the project report titled **"ANALYSIS AND DESIGN OF ELEVATED R.C.C. WATER TANK"** in partial fulfilment of the requirements for the award of the degree of Bachelor of Technology in Civil Engineering submitted to the Department of Civil Engineering, **Jaypee University of Information Technology, Waknaghat** is an authentic record of work carried out by **Vaibhav Singh (171615) and Manik Seth(171631)** during a period from August, 2020 to May, 2021 under the supervision of **Mr. Anirban Dhulia**, Department of Civil Engineering, Jaypee University of Information Technology, Waknaghat.

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SYMBOLS

A = Total area of section

A_c = Equivalent area of section

A_{sc} = Area of longitudinal reinforcement (compression)

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

As_v= Total cross-sectional area of stirrup legs or bent up bars within distance S_v

D = depth

d = effective depth

f_{ck} = characteristic compressive stress of concrete.

F_y = characteristic tensile strength of steel.

j= lever arm factor.

M = bending moment or moment.

M_r=moment of resistance or radial bending moment.

M_t=torsional moment.

M_u=ultimate bending moment

τ_v=nominal shear stress in concrete

τ_c= permissible shear stress in concrete

V_u=ultimate shear force due or design load.

V_{us}=shear carried by shear reinforcement.

α = inclination

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

σ_{cc}= permissible stress in concrete in direction compression

σ_{sc} = permissible compressive stress in steel

σ_{st} = permissible tensile stress in steel

σ_{sv} = permissible tensile stress in shear reinforcement

ABSTRACT

Elevated Water Tanks are one of the most important lifeline structures in the urban as well as rural areas. In major cities and also in rural areas, they form an integral part of water supply scheme. The elevated water tanks must remain functional even after the earthquakes as water tanks are required to provide water for drinking and firefighting purpose. These structures have large mass concentrated at the top of a slender supporting structure. Hence these structures are especially vulnerable to horizontal forces due to earthquakes. All over the world, the elevated water tanks were collapsed or heavily damaged during the earthquakes because of unsuitable design of supporting system, wrong selection of supporting system, underestimated demand or overestimated strength. So, it is very important to select proper supporting system and there is also a need to study the response of Elevated Water Tanks to dynamic forces to find out the design parameters for seismic analysis. It is also necessary to consider the sloshing effect on container roof slab. This sloshing of water considerably effects the parametric values used in design and economy of construction. The effect of hydrodynamic pressure must be considered in the seismic analysis of Elevated Water Tank. [*Keywords:* seismic analysis, sloshing of water, hydrodynamic pressure, slender supporting structure, dynamic forces]

CHAPTER 1

INTRODUCTION

1.1 General

Natural disasters such as earthquakes, droughts, floods, and cyclones are all common in the Indian subcontinent. The majority of states and territories are vulnerable to one or more disasters. Every year, these natural disasters result in a large number of deaths and property losses. Earthquakes are the most dangerous natural disaster. As a result, it is important to learn to cope with these occurrences. More than 60% of India is vulnerable to earthquakes, according to the seismic code IS: 1893(Part I): 2000. Property loss can be restored to some degree after an earthquake, but life loss cannot. The collapse of buildings is the leading cause of death. It is said that earthquakes do not destroy people; it is buildings that are poorly built that do. As a result, careful earthquake analysis of the system is important. Water supply is a lifeline that must stay operational in the event of a disaster. The majority of Indian municipalities have a water supply system that relies on elevated water tanks for storage. A large elevated water storage container designed to store a water supply at a height sufficient to pressurise a water delivery system is known as an elevated water tank. Because of the large total mass concentrated at the top of the slender supporting framework, these structures are particularly susceptible to horizontal forces such as earthquakes. As a result, it's critical to assess the severity of these powers in a given area. The main goal of this project is to investigate how an elevated water tank reacts to dynamic forces and to determine specific design parameters. For seismic analysis, the impact of hydrodynamic pressure on the sides and base slab of the container must be taken into account. In the seismic study of an elevated water tank, the effect of pressure due to wall inertia and the effect of vertical ground acceleration must also be considered.

1.2 Objective: The main objective of our project is:

1. Preparing a water tank design that is economical and safe according to the provisions of IS code and IITK-GSDMA guidelines.
2. Analysing the design of the water tank on structural analysis software STAAD Pro.

CHAPTER 2

REVIEW OF LITERATURE

Much of literature has been presented in the form of technical papers on dynamic analysis of water tanks. Some of them are listed below:

2.1.1 George W. Housner, 1963

The key plot of this paper revolves around the 1960 Chilean earthquake. He considered three scenarios: a completely loaded tank, an empty tank, and a partially filled tank. In the first two examples, the sloshing effect was ignored, but in the third case, it was taken into account. In conclusion, he claimed that the maximum force that a half-filled tank can withstand is less than the force that a fully-filled tank can withstand.

2.1.2 Dr. Suchita Hirde and Dr. Manoj Hedaoo, 2011

This paper examines the impact of water tank height, earthquake zones, and soil conditions on earthquake powers. The study is based on a R.C.C. circular water tank made of M-20 concrete and Fe-415 steel. The capacity of 50000 litres and 100000 litres is considered, with staging heights of 12, 16, 20, and 28 metres and a panel height of 4 metres. In this paper, the following observations were made:

1. Seismic forces and seismic zones are directly proportional.
2. The height of the supporting structure is inversely proportional to seismic powers.
3. As the tank's capability grows, so do the seismic powers.
4. Soft soils have higher seismic forces than hard soils..

2.1.3 R.Livaoglu and A.Dogangun, 2007

This paper discusses the response of water tower supporting systems. Supporting structures for elevated water tanks also included frame staging and concrete shafts. The conclusion of this paper is that where there is a high risk of seismic forces, a cylindrical shaft support system may be used because it has significant advantages over the widely used frame type system. The roof displacement response for frame support is also found to be higher than that of the concrete shaft support system.

2.1.4 Prasad S. Barve and Ruchi P. Barve, 2015

This paper discusses the response of the water tower's supporting systems. Supporting structures for elevated water tanks also included both frame staging and concrete shafts. The conclusion of this paper is that where the risk of seismic forces is strong, a cylindrical shaft support system should be used because it offers significant benefits over the widely used frame type system. The roof displacement response for frame support is also found to be higher than for concrete shaft support.

2.1.5 Mor Vyankatesh K. and More Varsha T., 2017

The main goal of this paper is to compare the research results of base shear and base moment with different capacities and to analyse the hydrodynamic effect on elevated water tanks with different supporting systems. The following conclusions have been reached:

1. Tanks supported by a concrete shaft have a higher base shear than tanks supported by frame staging.
2. Tanks with concrete shaft support have a higher base moment.
3. With increasing tank capacity, the deflection of staging is found to be decreasing.
4. The height of the slamming wave is roughly the same for tanks with different supporting systems, but it varies as tank capability increases.

2.1.6 Dona Rose K J et. al., 2015

The dynamic response of an elevated circular style water tank is investigated. Tanks of varying capacities and staging heights were modelled. It is decided to use R.C.C. frame staging. The El Centro Earthquake acceleration records were used to do a time history study of the water tank. The study provided the peak displacements and base shear. The observations that follow were made:

1. With increasing height, the peak displacement increases.
2. The displacement of half-filled tanks is less than the displacement of full-filled tanks.
3. As the staging height increased, the base shear values increased as well.
4. Under the same staging conditions, the base shear for half-capacity tanks is lower than that for full-capacity tanks.

2.1.7 Urmila Ronad et. al.,2016

Dynamic response spectrum analysis, as per IS 1893:2002, is used to show the seismic activity of cylindrical liquid storage tanks. For an elevated circular R.C. tank, analyses were conducted for empty and full tank conditions in various soil conditions and zones. According to the findings, if the water tank is placed in a higher seismic environment, the resulting base shear and base moment would increase as well. The base shear and base moments are observed to change with soil conditions.

2.1.8 Nandagopan.M, Shinu Shajee, 2017

This paper is based on a manual dynamic study of various types of R.C.C. water tanks with varying water levels in order to determine base reactions at any 10% of maximum water level. The research is based on G.W. Housner's two-mass model for elevated water tanks, which he suggested in 1963. The water mass in the tank is divided into two types in this model: impulsive mass and convective mass. The observations that follow were made:

1. As the water level rises, the base shear and base moment increase.

2. Elevated water tanks have higher base reactions than tanks that are supported on the ground. As a result, the base reaction increases as the staging height rises.

3. The base shear and base moment of an elevated circular water tank are 1.37 percent and 3.69 percent higher than the base shear and base moment of an elevated rectangular water tank, respectively.

2.1.9 Keerthi Gowda B.S. et.al., 2014

A circular elevated reinforced concrete water tank with shaft staging support was investigated in this paper. The earthquake intensity of these tanks was determined based on Indian conditions. Two separate methods were used to conduct seismic analysis of these tanks (while taking into account the impact of sloshing:

1. Used the Indian standard code 1893-Part 1 to build a lumped mass model (2002)

2. Using two mass models based on the draught Indian standard code Part 2 of 1893. (2005)

considering a fixed foundation for various soil conditions As compared to the lumped mass model, the base shear and overturning moments in the Two Mass model are significantly lower. As a result, idealising the tank based on a single degree of freedom is unacceptable and may result in a 19 percent overestimate study that is economically inapplicable.

1.1.10 Dr. Abdulamir Atalla et.al.

An empty elevated concrete cylindrical liquid storage tank supported on a four-story frame was subjected to free vibration analysis in this study. Four different heights of the frame are investigated: 12,16,20, and 24 metres, as well as four different angles of inclination: 0,2.38,4.76,7.125 degrees measured from vertical. Due to the supporting frame's stiffness decrement, the natural frequency decreases with rising frame height and angle of frame inclination. On an elevated tank, a forced vibration analysis is performed under three filling conditions: 0%, 50%, and 100%. The seismically excited tank stands 15.9 metres tall and was subjected to the 1999 Kocaeli earthquake in Turkey. The dynamic response is calculated, including maximum stresses and displacements.

The main observations obtained from the study are:

1. The most tension is concentrated at the points of interaction between the frame and the container's body.
2. As frame height increases, natural frequency decreases. As the frame height is doubled, there is a 68 percent increase in natural frequency..

3. As the frame tendency increased, the stiffness of the columns and beams decreased and their masses increased, resulting in a decrease in frequency.
4. Due to column buckling, the lateral displacement in the x-direction caused by the earthquake reaches its highest value near the centre of the frame height.

2.2. Conclusion based upon literature survey

The value of earthquake analysis and construction of elevated water tanks cannot be overstated. And after an earthquake, these systems must continue to work. Elevated water tanks are especially vulnerable to earthquake damage because they usually consist of a large mass supported on top of a slender staging. As a result, the study and design of such structures to withstand earthquakes is critical.
Following points should be considered during seismic analysis of an elevated water tank after a thorough review of all documents:

1. When analysing elevated water tanks, three scenarios are typically considered: empty condition, partially filled condition, and completely filled condition. The tank would act as a one mass structure in the first two cases, and as a two mass structure in the third case.
2. The majority of the elevated water tanks are never fully stocked. As a result, a two-mass idealisation is preferable to a single-mass idealisation.
3. Seismic forces are inversely proportional to the height of the supporting structure and are directly proportional to seismic zones.
4. In addition to the hydrostatic pressure, water in the tank vibrates and exerts impulsive and convective hydrodynamic pressure on the tank wall and tank base during earthquakes. The impact of these forces should be taken into account when analysing tanks.
5. In the study, the impact of water sloshing must be taken into account. The amount of free board in the tank must be determined by the sloshing wave height..

6. There is only one IS code in India, namely IS 1893: 1984, which contains provisions for seismic construction of elevated water tanks. IS 1893(Part-1): 2002 is the fifth revision of the standard, which is still being updated. As a result, the above IS code does not provide detailed requirements for seismic analysis of elevated water tanks. As a result, all of the above author's advice and suggestions must be taken into account during the review. The IITK-GSDMA has given some seismic design guidelines for elevated water tanks that should be considered during the study.

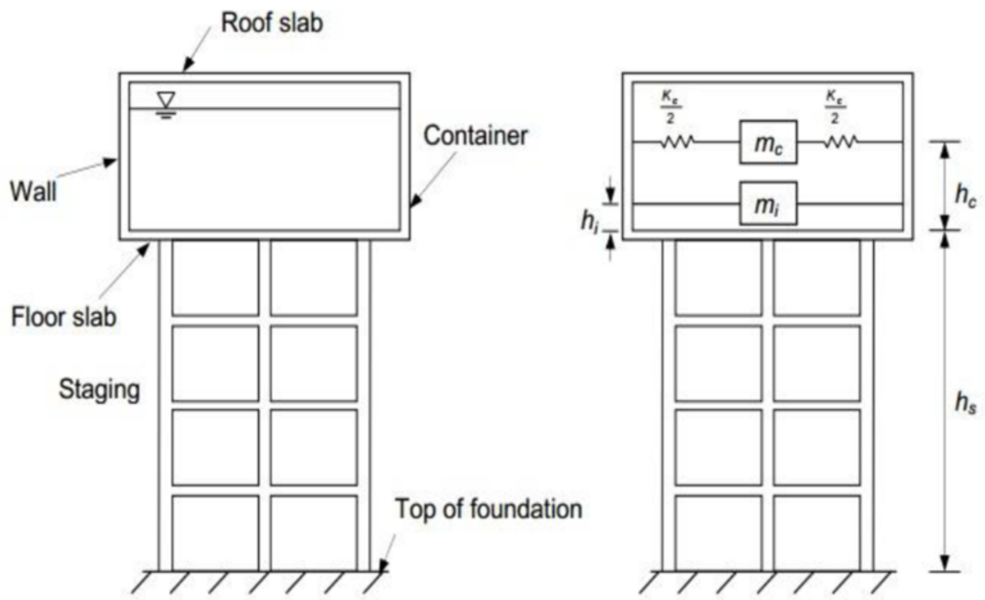

Fig 2.1 Elevated tank and the Spring mass model of elevated tank

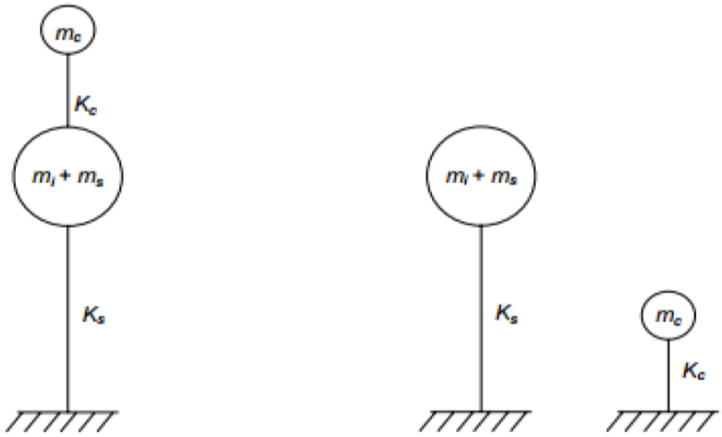

Fig 2.2 Two mass idealization of water tank and its equivalent uncoupled system

Image source: More Varsha T. et. al., *IOSR Journal of Mechanical and Civil Engineering (IOSR-JMCE), Volume 14, Issue 1 Ver. I (Jan. - Feb. 2017), PP 38-46*

CHAPTER 3

WATER DEMAND CALCULATION

3.1 Water Quantity Estimation

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

1. Water consumption rate (Per Capita Demand in liters per day per head)
2. Population to be served.

Quantity= Per Capita 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 demand, which a city may have, may be broken into following types:

Table 3.1 Water Consumption for Various Purposes:(IS-1172 -1992)

	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	Transmission Losses and Waste	45-150	62	15

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:

Table 3.2 Formulas for fire-fighting demand

	Authority	Formulae (P in thousand)	Q (for 1 lakh Population)
1	American Insurance Association	Q (L/min) =4637 P (1-0.01 P)	41760
2	Kuchling's Formula	Q (L/min) =3182 P	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 P for P>50000	31623

3.3 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 houses with sewers.
- Presence of industries.
- Climatic conditions.
- Habits or economic status.
- **Quality of water:** If water is aesthetically and 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 unauthorized 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.

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.
- **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. Maximum daily demand = 1.8 x average daily demand

Maximum hourly demand of maximum day i.e., Peak demand

= 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

3.4 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:

- 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.

Population 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 discretion 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

CHAPTER 4

DESIGN CONSIDERATIONS FOR WATER TANKS

4.1 DESIGN REQUIREMENT OF CONCRETE (IS-3370)

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 weaker than M20 is not used. The minimum quantity of cement in the concrete mix shall be not less than 330 kN/m^2. The design of the concrete mix shall be such that the resultant concrete is sufficiently 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 face of the equivalent concrete section does not exceed the permissible tensile strength of concrete as given in table below. 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.

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. 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.

4.2 JOINTS IN LIQUID RETAINING STRUCTURES:

4.2.1 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 4.1(a).

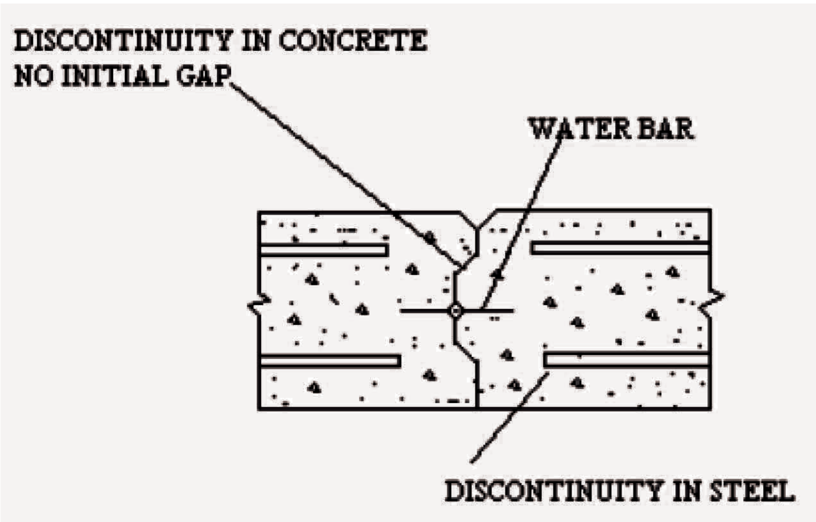

Fig 4.1(a) Complete contraction joint

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.4.1 (b)

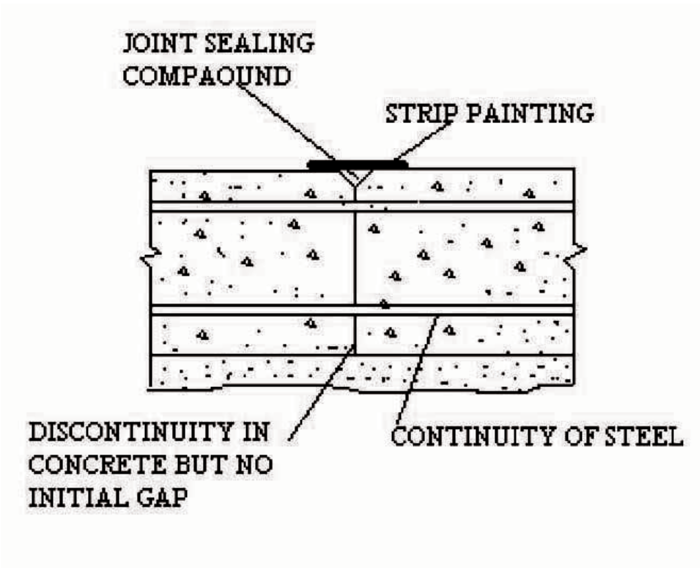

Fig 4.1(b) Partial contraction joint

(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 4.1 (c)

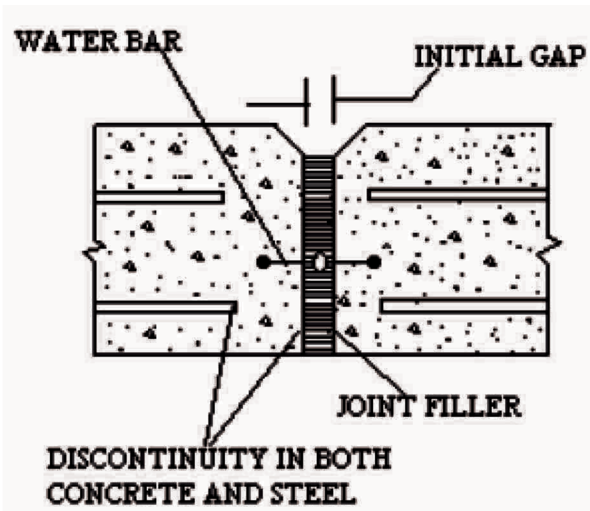

Fig.4.1(c)Typical Expansion joint

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

4.2.2CONTRACTION JOINTS:

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

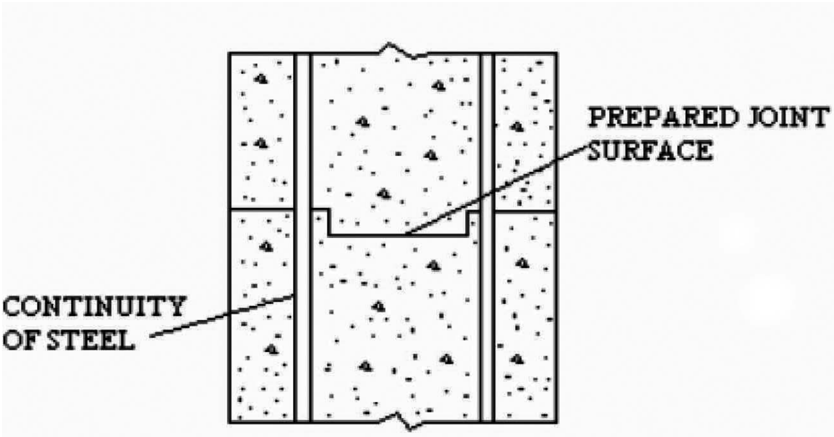

Fig 4.2(a) Typical contraction joint

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

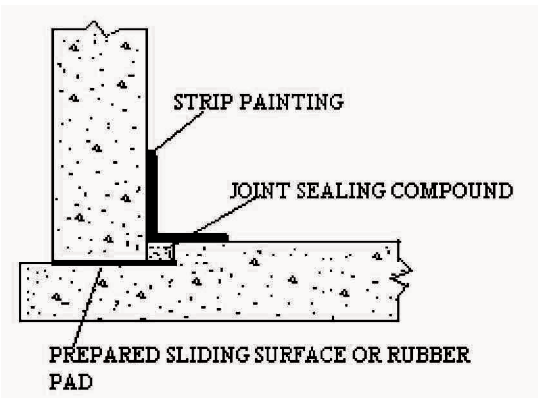

Fig. 4.2(b) Typical sliding joint

4.2.3TEMPORARY 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 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 4.2.3

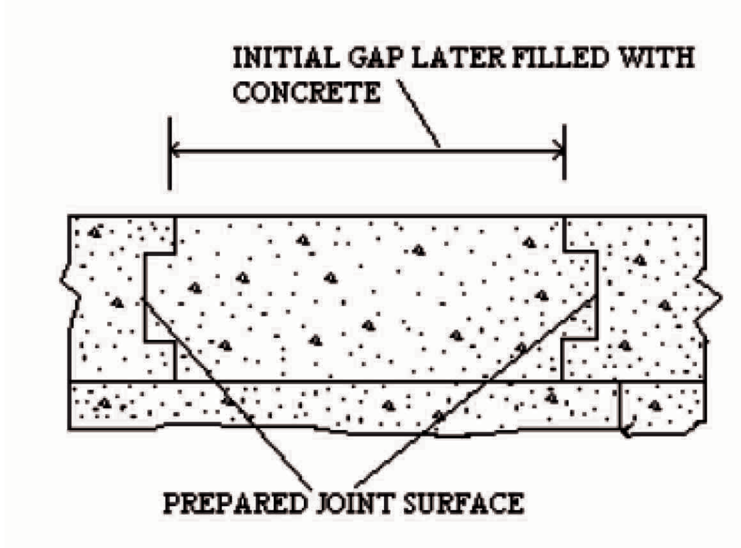

Fig 4.3 Typical temporary joint

4.3 GENERAL DESIGN REQUIREMENTS (IS-3370)

4.3.1*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 4.1. 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 4.1. 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.

Table 4.1. Permissible concrete stresses in calculations relating to resistance to cracking

Grade of Concrete	Permissible tensile stress in kN/m^2		Shear (kN/m^2)
	Direct	Bending	
M15	1.1	1.5	1.5
M20	1.2	1.7	1.7
M25	1.3	1.8	1.9
M30	1.5	2.0	2.2
M35	1.6	2.2	2.5
M40	1.7	2.4	2.7

4.3.2Permissible 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 in Table 4.2

Table4.2-Permissible stresses in steel

S.No.	Types of stresses in steel reinforcement	Permissible stresses in N/mm^2	
1.	Tensile stress in members under direct tension	Plain Mild Steel Bars	HYSD Bars
		115	150

2.	Tensile stress in member in bending on liquid retaining face of members	115	150
	On face away from liquid for members less than 225mm thick	115	150
3.	On face away from liquid for members 225mm or more in thickness	125	150
4.	Tensile stress in shear reinforcement, for members less than 225mm thickness	115	150
	For members 225mm or more in thickness	125	175
5.	Compressive stress in columns subjected to direct load	125	175

4.3.3 Floors:

(i) *Provision of movement joint* **:** Movement joints should be provided as discussed in article 4.2.1.

(ii) *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.

(c) 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.

(d) 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.

4.3.4Walls

Walls of Tanks Rectangular or Polygonal in Plan:

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

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:

$$(f_{ct}/\sigma_{ct}) + (f_{cbt}/\sigma_{cbt}) \leq 1$$

f_{ct}= calculated direct tensile stress in concrete

σ_{ct} = permissible direct tensile stress in concrete (Table 4.1)

f_{cbt} = calculated tensile stress due to bending in concrete.

σ_{cbt}= permissible tensile stress due to bending in concrete.

4.3.5Minimum 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 10 mm, 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 from0.3% for 100mm thick section to 0.2% 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%. 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.

4.3.6 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 greater. 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.

CHAPTER 5
DESIGN OF INTZE TANK

5.1 INTRODUCTION

Overhead water tanks of various shapes can be used as water storage structures in various water supply schemes. Reinforced concrete water tank 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 tank consists of:

1. Top dome
2. Top ring beam
3. Cylindrical wall
4. Bottom ring beam
5. Conical bottom
6. Bottom spherical dome
7. Bottom circular girder
8. Columns of supporting tower
9. Bracings
10. Foundation

Types of water Tanks may be:

(a) Rectangular;

(b) Circular, flat or domed at bottom

Among these, the circular types are proposed for large capacities. Such 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 tanks, 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. The design of the tank will involve the following:

(1) **Top 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) **Top Ring beam**: 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) **Bottom Ring beam**: 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 and meridional thrust. 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) **Ring girder**: This will be designed to transfer the load of the tank to the supporting columns. 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 transferring the load of the water retaining portion to the foundation. The columns will be braced at intervals and have to be designed for wind pressure or seismic loads whichever govern.

(9) **Foundations**: A raft footing is provided with a raft slab at the base and a circular raft girder.

5.2 WATER DEMAND CALCULATION (IS:1172-1992)

Total number of students (estimated): 2000

Average daily consumption: 120 lpcd

Considerations due to losses: 15% of average daily consumption

Total water consumption in one day: 2000*120*1.15 = 276000 liters/day

We design the tank for 2.5 days capacity

Volume of the tank: 276000*2.5 = 6,90,000 liters = 690 cu.m.

5.3DESIGN OF INTZE WATER TANK

5.3.1 Data

Design of Intze tank for a capacity of 6,90,000 litres

Assuming height of tank above GL as 16 m.

Number of columns in staging= 8

Depth of foundation below ground level= 1m.

5.3.2 Permissible stresses

M25 Grade Concrete: σ_{ct}=1.3 N/mm^2, σ_{cb}=1.8 N/mm^2, σ_{cc}=6 N/mm^2, σ_{cbc}=8.5 N/mm^2

Fe 415 Grade Steel: $\sigma_{st=}$ 150 N/mm^2

5.3.3Dimensions of tank

Using Reynold's formula, volume=0.585D^3

Diameter D=10.5 m

Height of cylindrical portion= 6.5 m

Depth of conical dome= 1.75 m

Spacing of bracings= 4m

Diameter of supporting tower= 7.5 m

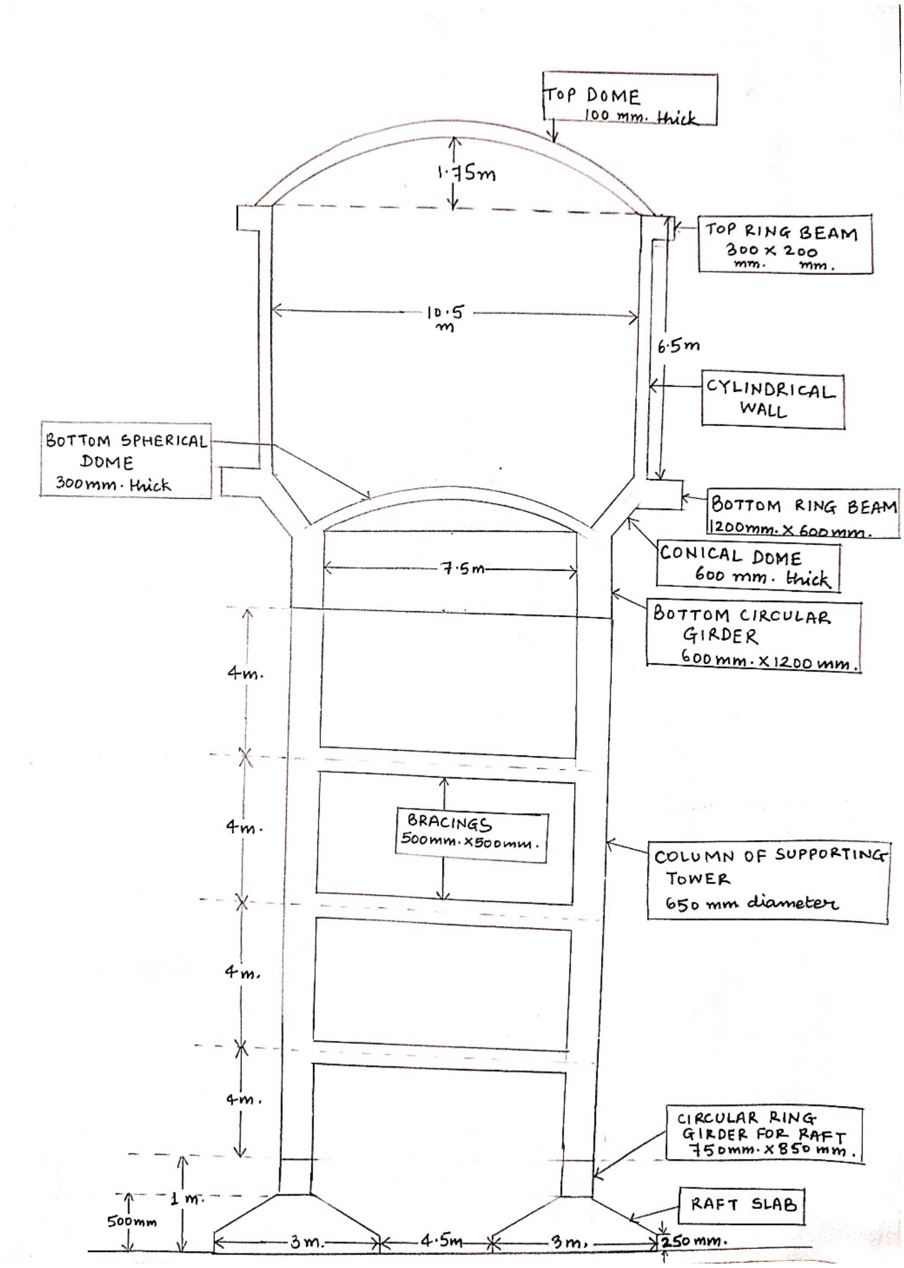

Fig5.1 Design details of Intze water tank

5.3.4 Design of top dome

Assuming thickness of dome slab= 100 mm

Self weight of dome = (0.1× 24)= 2.4 kN/m^2

Live load= 1.5 kN/m^2

Finishes= 0.1 kN/m^2

Total load, w= 4.0 kN/m^2

If R = radius of the dome

D = diameter at base = 10.5 m

r = central rise[(1/6) × 10.5] = 1.75 m

R = $\left(\frac{(D/2)^2 + r^2}{2r}\right) = \left(\frac{5.25^2+1.75^2}{2\times1.75}\right)$ = 8.75 m

Semi-central angle cos θ= cos(36.86)=0.8

Meridional thrust, T_1 = $\left(\frac{wR}{1+cos\theta}\right)$

= $\left(\frac{4\times8.75}{1+0.8}\right)$ =19.44 kN/m

Circumferential force = wR $\left(cos\theta - \frac{1}{1+cos\theta}\right)$

= $(4 \times 8.75)\left(0.8 - \frac{1}{1.8}\right)$

= 8.55 kN/m

Meridional stress= $\left(\frac{22.22\times10^3}{1000\times100}\right)$

= 0.22 N/mm^2 < 5 N/mm^2

Hoop stress = 0.085 N/mm^2 < 8 N/mm^2

The stresses are within safe limits.

providing nominal reinforcements of 0.3%

$A_{st} = \left(\frac{0.3\times100\times1000}{100}\right)$ = 300 mm^2

Provide 6 -8 mm diameter bars at 160 mm c/c both circumferentially and meridionally.

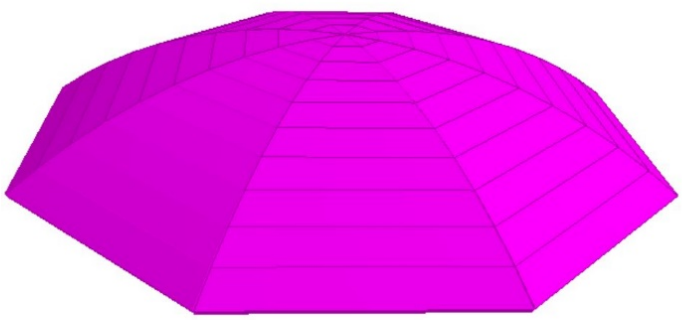

Fig 5.2 Top Dome modelled on STAAD Pro

5.3.5 Design of top ring beam

Hoop tension $F_t = \left(\frac{T_1 cos\theta D}{2}\right)$

$= \left(\frac{19.44 \times 0.8 \times 10.5}{2}\right) = 81.65$ kN

$A_{st} = \left(\frac{81.65 \times 10^3}{150}\right) = 544$ mm^2

Provide 6 bars of 12 mm diameter (A_{st} = 680 mm^2)

If A_c = cross-sectional area of ring beam

$\left(\frac{81.65 \times 10^3}{A_c + 11 \times 680}\right) = 1.3$

Therefore A_c = 55327.7 mm^2

Provide 300 mm × 200 mm top ring beam, with 6 bars of 12 mm diameter as main reinforcement and 6 mm diameter stirrups at 150 mm c/c.

5.3.6 Design of cylindrical tank wall:

Maximum hoop tension at the base of the wall

Ft = (whD/2)

Where w= unit weight of water= 10kN/m3

h= depth of water

Therefore Ft = ((10x8x12)/2)

=341.25 kN/m

Tension reinforcement per meter height

Ast = ((341.25x10^3)/150)

=2275 mm^2/m height

Provide 8-20mm diameter bars at 180mm c/c on each face (Ast=2500mm^2)

Tension reinforcement required at 2m below the top is

Ast= (3200x2/8)

=800mm^2

If t=thickness of side wall at bottom,

((341.25x10^3)/1000t+(11x2275)) =1.3

Therefore t= 237.475mm.

Adopt 250 mm thick walls at the bottom, gradually reducing the thickness to 200mm at the top.

Distribution steel: At bottom, A_{st}=0.2% of cross sectional area=500mm^2(Provide 6-10 mm.

diameter bars at 200 mm. c/c)

At top, 0.3% of cross-sectional area=600mm^2(Provide 10mm. diameter bars at 250 mm. c/c)

The details of reinforcements provide in the cylindrical tank walls at different heights are shown in Table 5.1

Table 5.1: Details of reinforcements in water tank walls

Distance From top (m)	Main hoop steel each face (mm. c/c)	Vertical distribution steel, each face (mm. c/c)
0-2	10-180	10-300
2-4	16-200	10-250
4-8	20-180	10-180

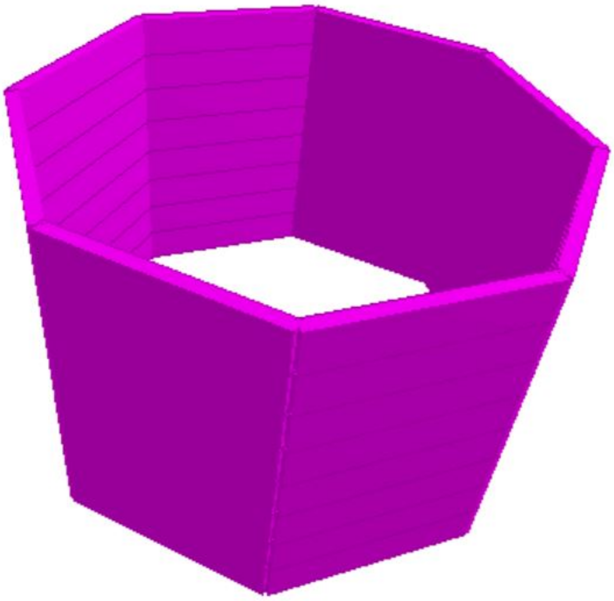

Fig 5.3 Cylindrical Tank wall modelled on STAAD Pro

5.3.7 Design of bottom ring beam:

Loads on ring beam :

Load due to top dome = (meridional thrust x $\sin\phi$) = 11.66 kN/m

Load due to top ring beam = (0.3 x 0.2 x 24) = 1.44 kN/m

Load due to cylindrical wall = 35.1 kN/m

Self weight of ring beam (assuming a section 1.2 m x 0.6 m) = (1.2 x 1.6x2.4)= 17.28 kN/m

Therefore, total vertical load V = 65.28 kN/m

Hoop tension due to vertical loads, H_v=VD/2=343.77 kN

Hoop tension due to water pressure, H_w=whdD/2=204.75 kN

Therefore, hoop tension = (H_v + H_w)=548.52 kN

A_{st}=3656.8 mm^2

Provide 12 bars of 20mm diameter (Ast = 3770 mm^2).

Maximum tensile stress= (548.52*1000)/(1200*600+11*3656)

= 0.75 $N/mm^2 < 1.3 N/mm^2$

Provide a ring beam 1200 mm wide and 600mm deep with 12 bars of 20mm diameter and distribution bars of 10 mm. diameter from cylindrical wall taken round the main bars as stirrups at 180 mm. c/c spacing.

5.3.8 Design of conical dome:

Average diameter of conical dome = (10.5+7.5)/2= 9 m.

Average depth of water = (6.5+1.75/2) = 7.375 m

Weight of water above conical dome= (π*9*7.375*1.75*8.250)= 3010kN

Assuming 600 mm thick slab,

Self weight of slab = (π*2.3*9*0.6*24)

=938.4 kN

Load from top dome, top ring beam, cylindrical wall and bottom ring beam

= (π*10.5*65.28) = 2153.37 kN

Therefore, load at base of conical slab= 6102 kN

Load/unit length V_2=(6102/π*6.5) = 300 kN/m

Meridional thrust= T= V_2 cosec θ= 300* cosec 45^0= 425kN

Meridional stress=(425*10^3)/(600*1000)= 0.708 N/mm^2

Hoop tension in the conical dome will remain maximum at the top of the conical dome slab since diameter D is maximum at this section.

Hoop tension H = (p cosec θ+qcot θ)(D/2)

Water pressure p = (10*6.5)= 65 kN/m^2

θ = 45^0

D = 10.5 m

Therefore H = (65 cosec 45^0 + 14.4 cot 45^0)(10.5/2) = 558.20 kN

Therefore A_{st}= (558.20*10^3)/150= 3721 mm^2

Provide 8-25 mm diameter bars at 180mm c/c (Ast = 3926mm^2) on both faces of slab

Distribution reinforcement =(558.20*10^3)/((600*1000)+(11*3926))

Provide 10mm diameter bars at 130mm c/c on both faces along the meridians.

Maximum tensile stress =

= 0.86 N/mm^2 < 1.2N/mm^2

Stress is within safe limits.

5.3.9 Design of bottom spherical dome:

Assume thickness of dome slab = 300 mm

Diameter at base D= 7.5m

Central rise r = (1/5x7.5)=1.5m

If r = Radius of the dome =(2R-r)r = $(D/2)^2$

=(2R-1.5)1.5 = 3.75^2

Therefore, R = 5.44 m

Self weight of dome slab=(2*π*5.44*1.5*0.3*24)= 370kN

Volume of water above the dome= 308.6 m^3

Weight of water = 3080 kN

Therefore, total load on dome = (3080+370)) = 3450 kN

Load/unit area w = (3450/(π*3.25^2))= 103.9 kN/m^2

Meridional thrust T1 = $\left(\frac{wR}{1+\cos\theta}\right)$

cos θ= (3.94/5.44)=0.724

Therefore $\theta = 44.5^0$

Therefore T_1=(104*5.44)/1.724=328.16 kN/m

Meridional stress = (328.16*10^3)/(300*1000)= 1.09

Stress is within safe limits.

Circumferential force = $wR = \left(\cos\theta \frac{1}{1+\cos\theta} \right)$

= 81.44kNm

Hoop stress = (81.44*10^3)/(300*1000)=0.27 N/mm^2

Stress is within safe limits.

Provide nominal reinforcement of 0.3 %.

A_{st}= (0.3*300*1000)/100= 900 mm^2

Provide 8-12 mm diameter bars at 120mm c/c circumferentially and meridionally.

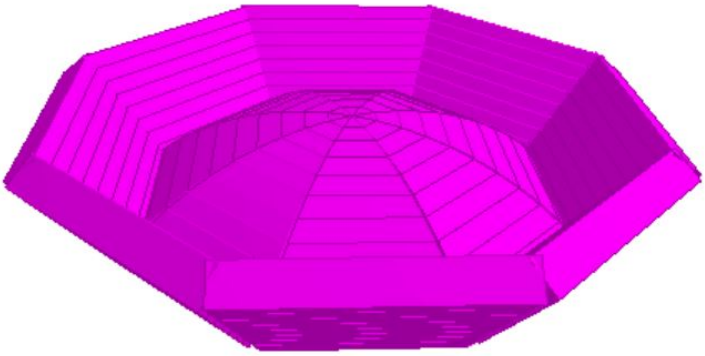

Fig 5.4 Bottom Spherical dome and conical dome modelled on STAAD Pro

5.3.10 Design of bottom circular girder

Thrust from conical dome, T_1=425 kN/m acting at an angle α = 45^0 to the horizontal

Net horizontal force on the ring beam

= $(T_1 \cos\alpha + T_2 \cos\beta)$

= [(425x0.707)-(328x0.713)] = 66.71 kN

Hoop compression in beam= (66.61*65)/2= 216.5 kN

Assuming the ring girder to be 600 mm wide and 1200mm deep.

Hoop stress = (216.5*10^3)/(600*1200) = 0.3N/mm^2

Vertical load on ring beam

=$[T_1 \sin\alpha + T_2 \sin\beta]$

[(425x0.707)-(328x0.70)]=530.07 kN/m

Self weight of beam= 0.6*1.2*24= 17.28 kN/m

Total load w = (530+17.28) = 547.3 kN/m

Total design load on the ring girder

W= πDw= π*7.5*547.3=12895.4 kN

The circular girder is supported on 8 columns using the moment coefficients.

The moment coefficients for different numbers of columns are compiled in Table 5.2

Table 5.2: Moment coefficients in circular girders supported on columns

Number of Columns π	Negative BM at support K_1	Positive BM at center of spans K_2	Maximum twisting moment or torque K_3	Angular distance for maximum torsion 0		Angle between the columns 0
4	0.0342	0.0176	0.0053	19	12	90
6	0.0142	0.0075	0.0015	12	44	60
8	0.0083	0.0041	0.0006	9	33	45
10	0.0054	0.0023	0.0003	7	30	36
12	0.0037	0.0014	0.0017	6	15	30

Using the moment coefficients given in Table 5.2

Maximum negative BM at support section

= (0.0083 wR) = (0.0083*125895.4*3.75) = 467 kN-m

Maximum positive BM at mid-span section

= (0.0041wR)= (0.0041 x12895.4*3.75) = 198.27 kN-m

Shear force at support section.

V=(wR*(π/4))/2= 805.96kN

Shear force at the section of maximum torsion (at an angle of 9.5^0 from column support)

V=465.66kN

(a)Design of support section

M = 401.4 kN-m

V = 805.96 kN

k_b=0.39

Q= 1.38

Therefore d = √((401.4*10^6)/(1.38*600))=696.3 mm.

Adopt effective depth d= 800 mm, cover = 50 mm

A_{st}=(401.4*10^6)/(150*1.38*800)= 2423.9 mm^2

Provide 8 bars of 20 mm diameter (A_{st} = 2513 mm^2)

τ_v=(805.96*10^6)/(600*800)= 1.67N/mm^2

(100 A_{st}/bd)=0.523

From Table 1.3b, τ_c = 0.31 N/mm^2

Since $\tau_c < \tau_v$, shear reinforcement are required

Shear taken by concrete=(0.31*600*800/1000)=148.8kN

Balance shear = (805.9-148.8) = 657kN

Using 12 mm diameter 4 – legged stirrups, spacing

S_v=(150*4*113*800)/(657*10^3)= 82.55 mm

Adopt 12 mm diameter 4 legged stirrups at 80 mm c/c near supports

(b) Design of mid – span section

Maximum positive moment in the section=198.27 kNm

A_{st}=(198.27*10^6)/(150*0.9*750)= 1836 mm^2

Minimum area of steel in section=(0.24*600*800)/100=1152 mm^2

Provide 6 bars of 20 mm. diameter at mid-span section (A_{st}=1884 mm^2) and 4-legged stirrups of 10mm diameter at 250 mm c/c

(c) Design of section subjected to maximum torsion

T = 29 kNm

D = 800 mm

V = 465.66 kN

b= 600 mm

M=0

d=750mm

M_t=T(1+D/b)/1.7=40 kN-m

IS 456 – 2000, clause B-6.4.2

Therefore $M_{e_1} = (M + M_1) = (0 + 60) = 60kNm$

A_{st}=(40*10^6)/(150*0.9*750)=395 mm^2

But minimum area of tension reinforcement= 1152mm^2

IS: 456-2000, Clause B 6.3.1

Provide 4 bars of 20 mm. diameter (A_{st} = 1256 mm^2)

Equivalent shear, V_e = (V+1.6(T/d)

=528 kN

τ_v=V_e/bd=(528*10^3)/(600*750)=1.17N/mm^2

(100 A_{st}/bd)=(100*1256)/(600*750)=0.279N/mm^2

Since τ_v>τ_c, shear reinforcements are required.

Using 10mm diameter 4 legged stirrups with the side covers of 25 mm and top and bottom covers of 50 mm. spacing.

IS : 456-2000 Clause B6.4.3.

S_v=(4*78.5*150)/((1.17-0.28)*600)=88 mm.

Adopt 10 mm. diameter 4 legged stirrups at 80mm c/c.

5.3.11 Design of column of supporting tower

The supporting tower comprises 8 equally spaced columns on a circle of 7.5m. diameter.

Vertical load on each column = (12895.4/8)=1611 kN

Self weight of column of height 16m and diameter 650mm.

$$\left(\frac{\pi}{4} \times 0.65^2 \times 16 \times 24\right) = 127kN$$

Self weight of bracings (3 numbers at 4 m intervals,

(size 500 mm x 500 mm) = $\left(3 \times 0.5 \times 0.5 \times \frac{\pi \times 8}{8} \times 24\right) = 57kN$

Total vertical load on each column= 1795 kN

Wind forces: Intensity= 1.7 kN/m^2

Reduction coefficient=0.7

- Wind forces on top dome and cylindrical wall= (6.5+1.75)*0.7*1.7*10.5=103.08 kN

- Wind forces on conical dome= 1.7*0.7*9*2= 21.42 kN
- Wind force on bottom ring beam= 1.7*0.7*1.2*8= 11.424 kN
- Wind forces on 5 columns= 5*0.65*16*0.7*1.7=61.88 kN
- Wind forces on bracings=1.7*0.5*3*7=17.85 kN

Total horizontal wind force= 215.65 kN

Assuming contra flexure points at mid-height of columns and fixity at base due to raft foundations, the moment at the base of the columns is computed as

M=(215.65*4)/2 =431 kNm

M_1 = moment at the base of the column due to wind loads

=3555.63 kNm

And V = reaction developed at the base of exterior columns

$$M_1 = \sum M + \frac{V}{r_1}\sum r^2$$

Therefore V= 179 kN

Total load on leeward column at base= (1795+215) = 2010 kN

Moment in each column at base = (431/8) = 53.875kNm

Reinforcement in column:

Axial load, P = 2010kN

Bending moment, M = 53.875 kNm

Eccentricity, e = (M/P) = 26.8 mm.

Since eccentricity is small, direct stresses are predominant.

Using 8 bars of 32 mm diameter and lateral ties of 10mm diameter at 300mm c/c.

$$A_{sc=}\left(8\times 804\right) = 6432mm^2$$

Equivalent area of composite section

$$A_c = \left(\frac{\pi(650)^2}{4} + (1.5 \times 13 \times 6432)\right) = 0.45 \times 10^6 mm^2$$

Equivalent second moment of area of composite section

$$lc = \left(\frac{\pi \times 325^4}{4}\right) + (1.5 \times 13)\left[(2 \times 804 \times 275^2) + 4 \times 804\left(\frac{275}{\sqrt{2}}\right)^2\right]$$

$= 13.48 \times 10^9 mm^4$

Direct compressive stress

$$\sigma_{cc} = \left(\frac{2.010 \times 10^3}{0.45 \times 10^6}\right) = 4.467\text{N/nm}^2$$

Bending stress = $\sigma_{cb} = \left(\frac{53.875\times10^6\times325}{13.48\times10^9}\right) = 1.298N/mm^2$

Permissible stresses in concrete are increased 33.33% while considering the wind effect.

I.S. 456 – 2000, clause B – 41 , $\left(\frac{\sigma_{cc}^{'}}{\sigma_{cc}} + \frac{\sigma_{cb}^{'}}{\sigma_{cb}}\right) < 1$

Therefore

$$\left(\frac{4.467}{6} + \frac{1.2989}{8.5}\right) = 0.89 < 1$$

Stress is within safe limits.

5.3.12 Design of bracings

Moment in brace = (2xmoment in column x $\sqrt{2}$)

M = (2x53.875x $\sqrt{2}$)

=152.38 kN-m

Section of brace = 500 mm x 500mm

Therefore b = 500 mm, d = 450mm

Moment of resistance of section

M_1= (0.897x500x450^2) = 91 kN-m

Balance moment M_2 = (M-M_1)

= 61.38 kNm

$$A_{st1} = \left(\frac{91 \times 10^6}{230 \times 0.9 \times 450}\right) = 977\ mm^2$$

$$A_{st2} = \left(\frac{61.38 \times 10^6}{230 \times 0.9 \times 400}\right) = 741\ mm^2$$

$$A_{st} = (As_1 + As_2) = 1718mm^2$$

Provide 4 bars of 25 mm diameter (Ast = 1964 mm^2) at the top and bottom since wind direction is reversible.

Length of brace L = (2x3.75xsin 22.5^0) = 2.87 m

Maximum shear force in brace = $\left(\frac{moment\ in\ brace}{\frac{1}{2}*lengthofbrace}\right)$

=106.2 kN

$$\tau v = \left(\frac{102\times10^3}{500\times450}\right) = 0.45N/mm^2$$

$$\left(\frac{100Ast}{bd}\right) = 0.769$$

IS: 456-2000 Clause B.5.4 and Table 23

From Table 1.3b.

$$\tau_c = 0.38N/nm^2$$

Since $\tau_v > \tau_c$, shear reinforcement are required,

Shear carried by concrete

$$= \left(\frac{0.38\times500\times450}{1000}\right) = 85.5kN$$

Balance Shear = 20.7kN

Using 10mm diameter 2 – legged stirrups

Spacing, S_v = (230*2*79*450)/(20.6*10^6)

= 793 mm.

0.75d=337.5

Therefore, S_v=337.5mm

Adopt 10mm diameter 2 – legged stirrups at 330 mm c/c

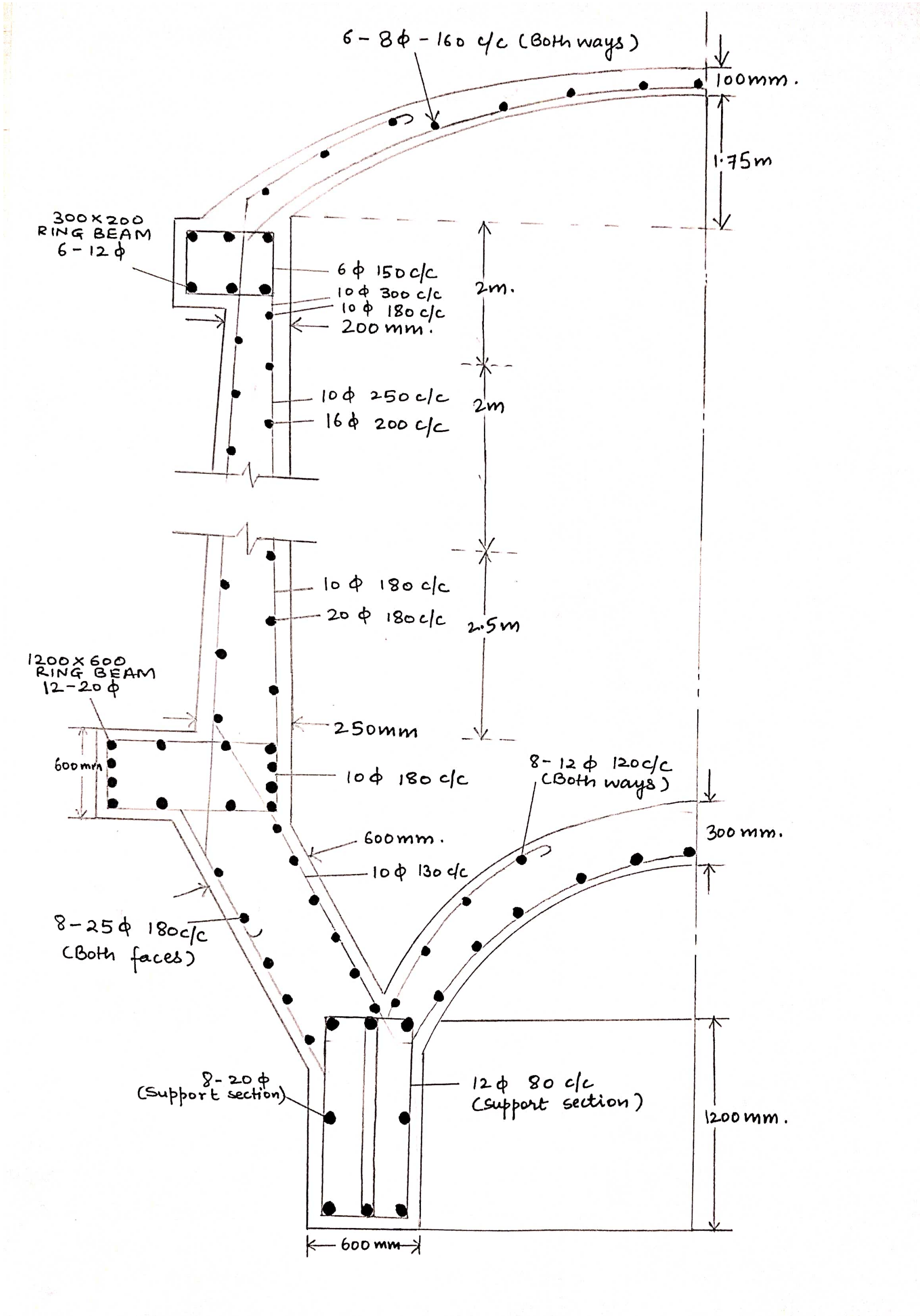

Fig 5.5 Reinforcement details of various components of water tank

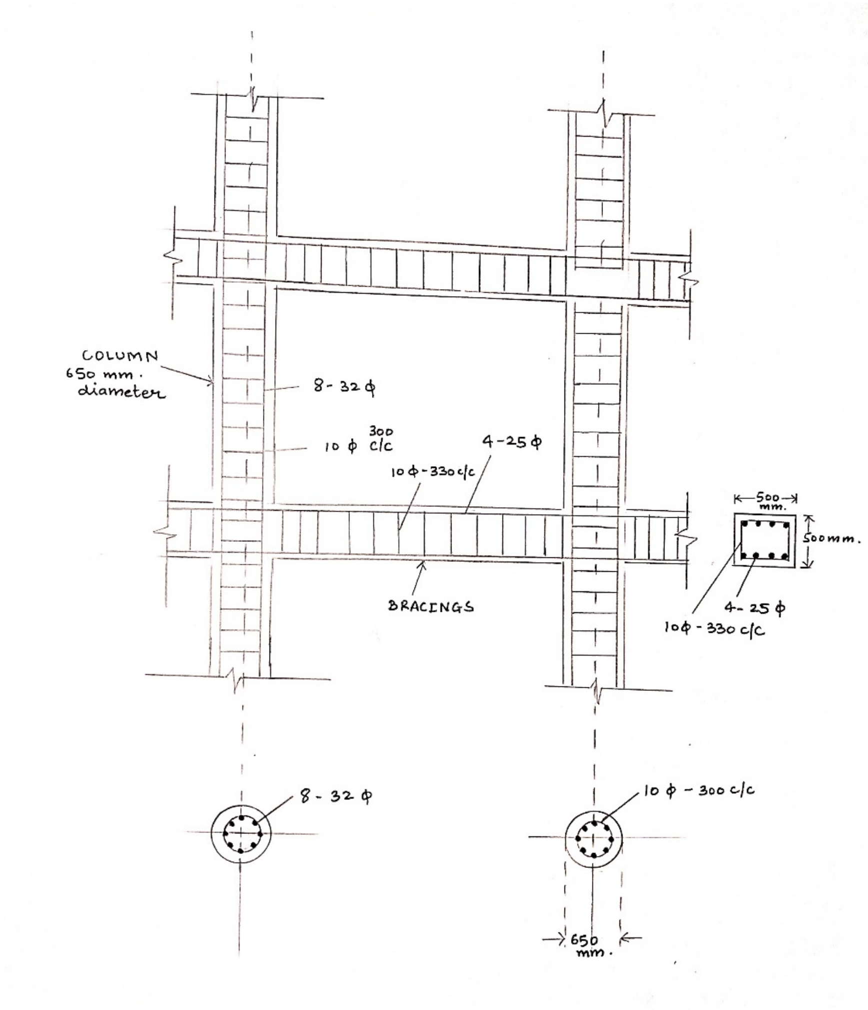

Fig 5.6 Reinforcement details in the staging of elevated water tank

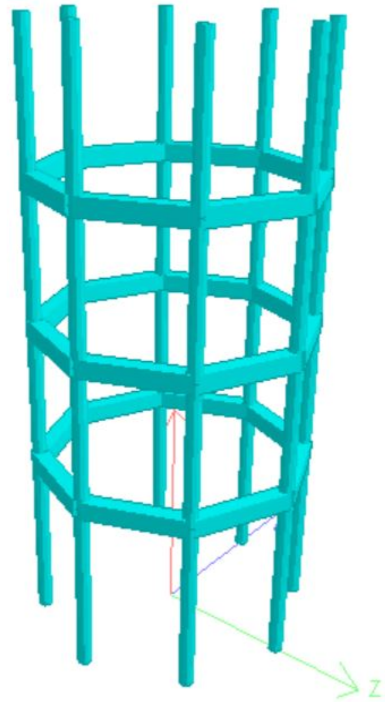

Fig 5.7 Supporting structure of the elevated water tank modelled on STAAD Pro

5.3.13 Design of Foundation

Total vertical load on foundation= (1795 x 8) = 14360 kN

Self weight of foundation at 10%= 1436 kN

Total load= 15796 kN

Safe bearing capacity of soil at site= 250 kN/m^2

Therefore, area of foundation= (15796/250) = 63.2 m^2

Π x 7.5 x b= 63.2

b = 2.68 m = 3 m approx.

inner diameter = 4.5 m

outer diameter = 11 m

Design of Circular Girder of Raft slab

Total load on circular girder= w = 14360 kN

Load per meter run on girder= (14360/8π) = 571.4 kN/m

Maximum negative moment at support= 0.0083wR= (0.0083x14360x3.75) = 447 kN.m

Maximum positive moment at mid span= 0.0041 wR = (0.0041x14360x3.75) = 221 kN.m

Maximum torsional moment= 0.0006x14360x3.75 = 32.3 kN.m

Shear force at support section= V = (571.4x3.75x(π/4))/2 = 841.5 kN

Shear force at section of maximum torsion= V= [841.5 – ((571.4xπx3.75x9.5)/180)]

=486.2 kN

The support section is designed for a maximum negative moment of M= 447 kN.m

Shear force V=841.5 kN

Assuming b= 750 mm

Effective depth d= $\sqrt{\frac{447x10^6}{1.38x750}}$ = 657.2mm

Adopt d= 800 mm with cover= 50 mm

A_{st}= $\frac{447x10^6}{230x0.9x800}$ = 2700 mm^2

Provide 6 bars of 25 mm φ (A_{st}= 2946 mm^2)

τ_{v}= $\frac{841.5x10^6}{750x800}$ = 1.4 N/mm^2

$\frac{100A_{st}}{\text{bd}}$=$\frac{100\text{x}2700}{750\text{x}800}$ = 0.45 N/mm^2

τ_c= 0.26 N/mm^2

Since $\tau_v > \tau_c$, shear reinforcement is required.

Shear taken by concrete= (0.26x750x800/1000) = 156 kN

Balance shear= (841.5-156) = 685.5 kN

Using 12 mm φ 4 legged stirrups, spacing is

S_v= $\frac{230\text{x}4\text{x}113\text{x}800}{685.5\text{x}10^6}$ = 121 mm.

Adopt 120 mm spacing.

Steel required for mid span section

A_{st}=$\frac{221\text{x}10^6}{230\text{x}0.9\text{x}800}$ = 1334.5 mm^2

But, minimum steel A_{st}=$\frac{0.85bd}{f_y}$ =$\frac{0.85\text{x}750\text{x}800}{415}$ = 1228.9 mm^2

Provide 3 bars of 25 mm at mid span section.

The section subjected to maximum torsional moment and shear should be designed for following forces

T= 32.3 kN.m

V= 486.2 kN

D= 850 mm

b= 750 mm

d= 800 mm

$M_t = T\left[\frac{1+(\frac{D}{b})}{1.7}\right] = 40.5$ kN.m

$M_{e1}=(M+M_t)= (0+40.5) = 40.5$ kN.m

$A_{st}=\left(\frac{40.5x10^6}{230x0.9x800}\right)= 244.5$ mm^2

Minimum area of steel= 1334.5 mm^2

Provide 3 bars of 25 mm φ (A_{st}= 1473 mm^2)

Equivalent shear= V_e = (V+ 1.6 T/b) = [486.2 + 1.6x(32.3/0.750)] = 555 kN

$\tau_v=\left(\frac{555x10^3}{750x800}\right) = 0.925$ N/mm^2

$\frac{100A_{st}}{bd}=\frac{100x1473}{750x800} = 0.2455$ N/mm^2

From tables, τ_c= 0.21 N/mm^2

$\tau_v > \tau_c$, therefore shear reinforcement is required.

Balance shear=$\left[555 - \frac{0.21x750x800}{1000}\right] = 429$ kN

Using 12 mm φ 4 legged stirrups, spacing is

$S_v = \left[\frac{A_{sv}.\sigma_{sv}}{(\tau_v-\tau_c).b}\right] = \left[\frac{4x113x230}{(0.925-0.21)x750}\right] = 194$ mm

Adopt 12 mm φ 4 legged stirrups at 190 mm centres

Design of Raft Slab

Maximum projection of raft slab from face of column= ((3-0.75)/2) =1.125 m

Soil pressure= $\left(\frac{143.6}{(5.5^2-2.25^2)x\pi}\right)$ = 181 kN/m^2

Considering 1 m width of raft slab along the circular arc

Maximum Bending Moment= (181x1.1^2)/2 = 109.5 kN.m

$d= \sqrt{\frac{109.5x10^6}{1.38x850}} = 305.5$ mm

Provide 500 mm over all depth with effective depth d= 450 mm to contain the shear stresses within permissible limits.

$A_{st}= \left(\frac{109.5x10^6}{230x0.9x450}\right)= 1175.5\ mm^2$

Provide 25 mm bars at 200 mm centres to reuce shear stresses (A_{st}=2454 mm^2)

Distribution steel= $\left(\frac{0.12x500x1000}{100}\right)$= 600 mm^2 provide 12 φ 180 mm c/c

Shear force at a section 450 mm from face of column is V= (181x0.65x1) = 118 kN

$\tau_v= \left(\frac{118x10^3}{1000x450}\right)= 0.26\ N/mm^2$

$$\frac{100A_{st}}{bd} = \frac{100x2454}{1000x450} = 0.545\ N/mm^2$$

From tables, τ_c= 0.31 N/mm^2

$\tau_v < \tau_c$, therefore no shear reinforcement needed.

Thickness of footing 500 mm decreased to 250 mm towards edges.

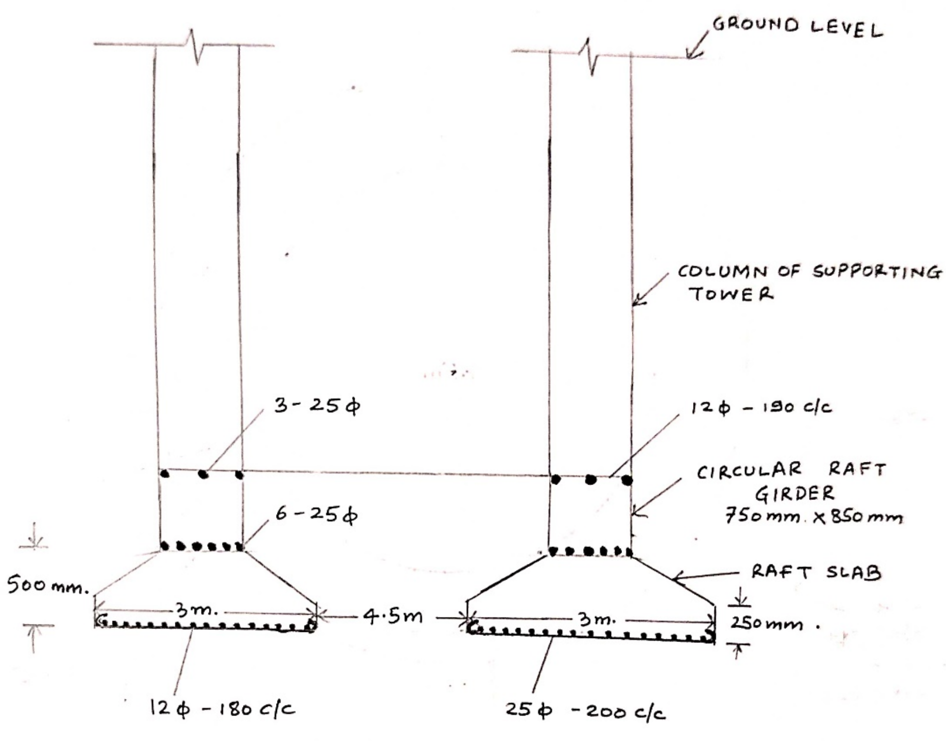

Fig 5.8 Section of Raft Foundation for elevated water tank

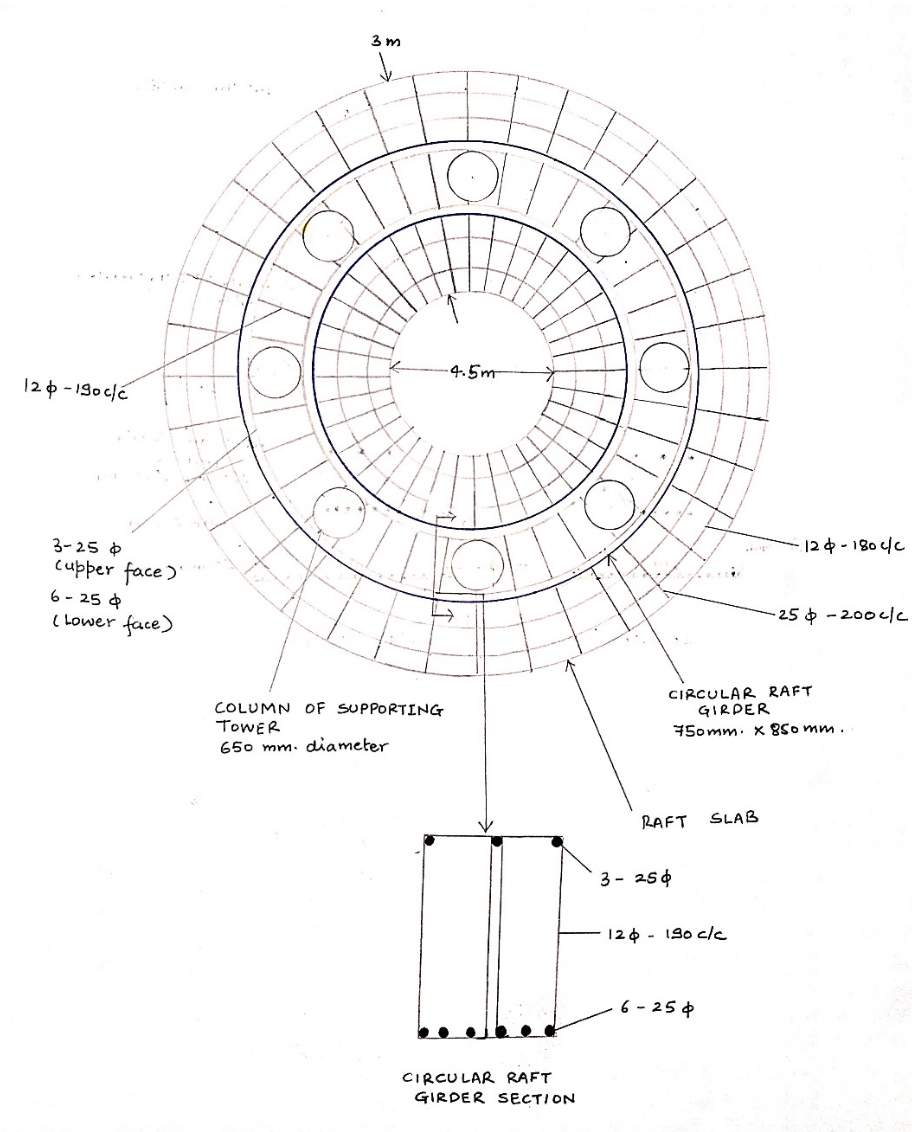

Fig 5.9 Plan of Raft Foundation for elevated water tank

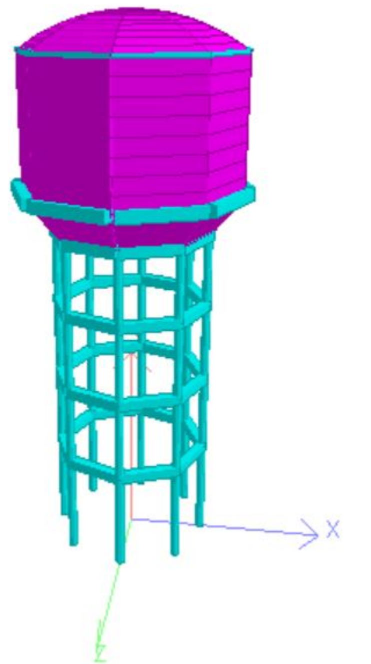

Fig 5.10 Intze water tank modelled on STAAD Pro

Member	Size of Cross Section(mm.)	Main Bars Size(mm.)	Size of Distribution Reinforcements(mm.)
Top Dome	100 mm. thickness	6- 8 mm. dia. bars circumferentially and meridionally	
Top Ring Beam	300 x 200	6-12 mm. dia. bars	6 mm. stirrups 150 mm. c/c spacing
Cylindrical Wall	200 mm. thickness at top and 250 mm. thickness at bottom	given in table	given in table
Bottom Ring Beam	1200 x 600	12-20 mm. dia. bars	10 mm. stirrups 180 mm. c/c spacing
Bottom Conical Dome	600 mm. thickness	8- 25 mm. 180 mm. c/c both faces	10 mm. 130 mm. c/c along meridians
Bottom Spherical Dome	300 mm. thickness	8- 12 mm. 120 mm. c/c circumferencially and meridionally	
Bottom Circular Girder	600 x 1200	**Support section**: 8-20 mm. dia. bars **Mid Span**: 6-20 mm. dia. bars **Max Torsion**: 4- 20 mm. dia. Bars.	**Support section**: 12 mm. 4 legged 80 mm. c/c spacing. **Mid Span**: 10 mm. 4 legged 250 mm. c/c spacing. **Max Torsion**: 10 mm. 4 legged 80 mm. c/c spacing.
Column of supporting tower	650 mm. diameter	8- 32 mm. dia. bars	10 mm. stirrups 300mm. c/c spacing
Bracings	500 x 500	4- 25 mm. dia. bars	10 mm. 2 legged 330 mm. c/c spacing
Foundation	**Ring Girder** (750 x 850) **Raft Slab** 500 mm. thickness decreased to 250 mm. towards edges	**Ring Girder**: *Upper face* 3- 25 mm dia. bars. *Lower face* 6-25 mm. dia. bars **Raft Slab**: 25 mm. dia. bars 200 c/c spacing meridionally and 12 mm dia. bars 180 c/c spacing circumferencially	12 mm dia. 4 legged stirrups

Details of reinforcements in water tank walls

Distance From top (m)	Main hoop steel each face (mm. c/c)	Vertical distribution steel, each face (mm. c/c)
0-2	10-180	10-300
2-4	16-200	10-250
4-8	20-180	10-180

Summary of the Designed Members of the Intze Water Tank

Fig 5.11 Design details of different components of elevated water tank

CHAPTER 6

ANALYSIS OF WATER TANK

6.1 INTRODUCTION

There are two forms of structural analysis: static and dynamic. In static analysis, applied forces are constant, while in dynamic analysis, applied forces differ. Static analysis is not suitable for water tanks since it indicates high scale values. It is uneconomical to obtain over-reinforced sections. As a consequence, dynamic analysis is used to obtain more appropriate outcomes. The tank is modelled as a two-mass system, with the assumption that it is still partially filled. The liquid in the lower part of the tank acts like a mass that is rigidly attached to the tank wall. Impulsive Liquid Mass is the name given to this mass. This liquid mass accelerates in lockstep with the tank wall, causing hydrodynamic pressure to build up on the tank wall and foundation. Convective Liquid Mass refers to the sloshing motion of liquid mass in the upper field.

Dynamic analysis classified on the basis of materials can be of two types:

1. Linear Analysis: Loads are within elastic range of deformation and the material obeys Hooke's law. Response spectrum analysis is a type of linear dynamic analysis.
2. Non Linear Analysis: Loads are beyond elastic range of deformation. Time History analysis is a type of non linear dynamic analysis.

Seismic Zone Factor, Z	0.16 (Seismic Zone 3)
Importance Factor, I	1.5 (for lifeline structures)
Response Reduction Factor, R	5.0 (Special RC Moment Resisting Frames SMRF)
Structural Response Factors, Sa/g	2.5 (for medium soil sites)
Damping	5%

Table 6.1: Values considered for Seismic Analysis of the designed water tank (according to IS 1893 Part 1:2016)

6.1.1 Response Spectrum Analysis

For a given damping, the Response Spectrum is a plot of the maximum response of linear Single Degree of Freedom system oscillators as a function of natural time. All of the following responses are appropriate:

1. Displacement
2. Velocity
3. Acceleration

For a structure being analysed with STAAD Pro, various mode shapes are obtained.

When a part vibrates at its natural frequency, its mode shape defines the deformation it will exhibit. It shows how the framework responds to a dynamic load. The mode form is ideal for assessing the structural component's dynamics qualitatively.

The sum by which any mode of vibration contributes to the overall vibration of the structure under horizontal and vertical earthquake ground motions is known as the modal participation factor.

6.2 RESULTS

			Horizontal	Vertical	Horizontal	Resultant	Rotational		
	Node	L/C	X mm	Y mm	Z mm	mm	rX rad	rY rad	rZ rad
Max X	330	1 EQ-X	22.215	0	0	22.215	0	0	0
Min X	330	6 HP	-259.594	0.26	0	259.594	0	0	0.002
Max Y	238	6 HP	-251.569	13.027	0	251.907	0	0	0.002
Min Y	256	6 HP	-255.208	-12.845	0	255.531	0	0	0.003
Max Z	330	2 EQ-Z	0	0	22.215	22.215	0	0	0
Min Z	205	6 HP	-243.292	9.205	-0.241	243.467	0	0	0.003
Max rX	14	6 HP	-60.229	-2.533	-0.115	60.283	0.003	0	0.012
Min rX	12	6 HP	-60.229	-2.533	0.115	60.283	-0.003	0	0.012
Max rY	45	6 HP	-234.575	-0.021	-0.034	234.575	0	0	0.003
Min rY	41	6 HP	-234.575	-0.021	0.034	234.575	0	0	0.003
Max rZ	13	6 HP	-60.114	-3.58	0	60.22	0	0	0.015
Min rZ	9	1 EQ-X	5.296	-0.293	0	5.304	0	0	-0.001
Max Rst	330	6 HP	-259.594	0.26	0	259.594	0	0	0.002

Node Displacement Summary

	Beam	L/C	Node	Fx kN	Fy kN	Fz kN	Mx kNm	My kNm	Mz kNm
Max Fx	45	6 HP	5	5159.753	-398.585	0	0	0	-1568.441
Min Fx	41	6 HP	1	-5141.854	-398.669	0	0	0	-1568.572
Max Fy	22	6 HP	22	-20.635	1324.42	0.766	85.772	-1.167	1949.657
Min Fy	18	6 HP	18	24.406	-1324.42	-0.766	-85.772	1.167	-1949.978
Max Fz	50	6 HP	10	-2892.915	-532.009	353.76	1.562	-739.129	-1091.164
Min Fz	56	6 HP	16	-2892.915	-532.009	-353.76	-1.562	739.129	-1091.164
Max Mx	14	6 HP	14	118.014	1263.777	-1.709	110.648	3.013	1877.091
Min Mx	10	6 HP	10	-118.742	-1263.777	1.709	-110.648	-3.013	-1877.027
Max My	56	6 HP	16	-2892.915	-532.009	-353.76	-1.562	739.129	-1091.164
Min My	50	6 HP	10	-2892.915	-532.009	353.76	1.562	-739.129	-1091.164
Max Mz	19	6 HP	20	-20.635	-1324.419	-0.766	-85.772	-1.167	1949.657
Min Mz	43	6 HP	3	8.949	-665.726	-0.042	-1.578	0.065	-2009.775

Beam Forces Summary

Table 6.2: Node Displacement summary and Beam Forces summary for the elevated water tank obtained from STAAD Pro

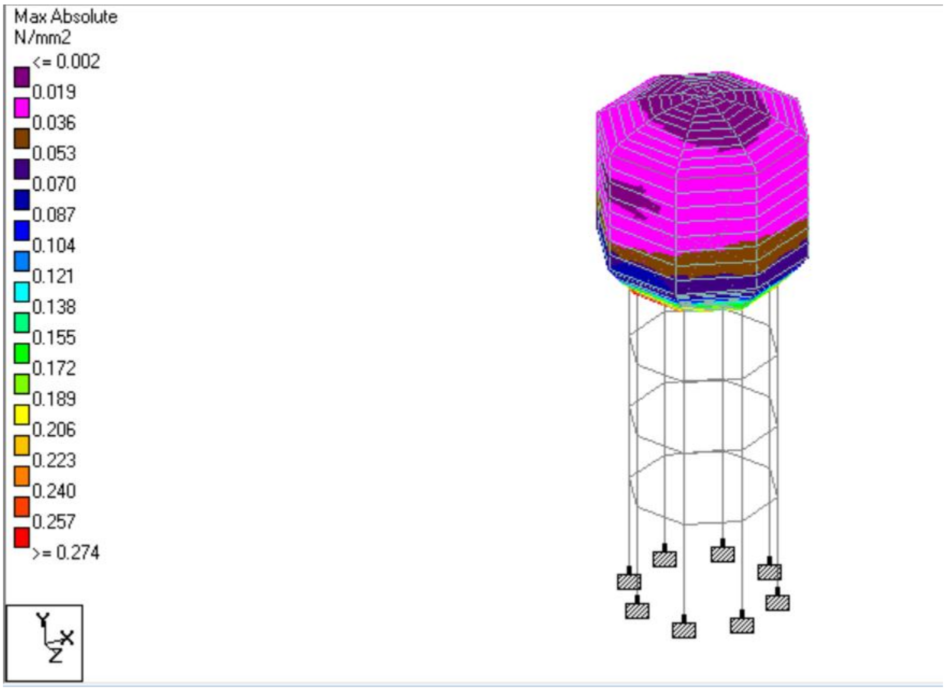

Fig 6.1 Maximum Absolute Pressure in the tank due to Earthquake loads

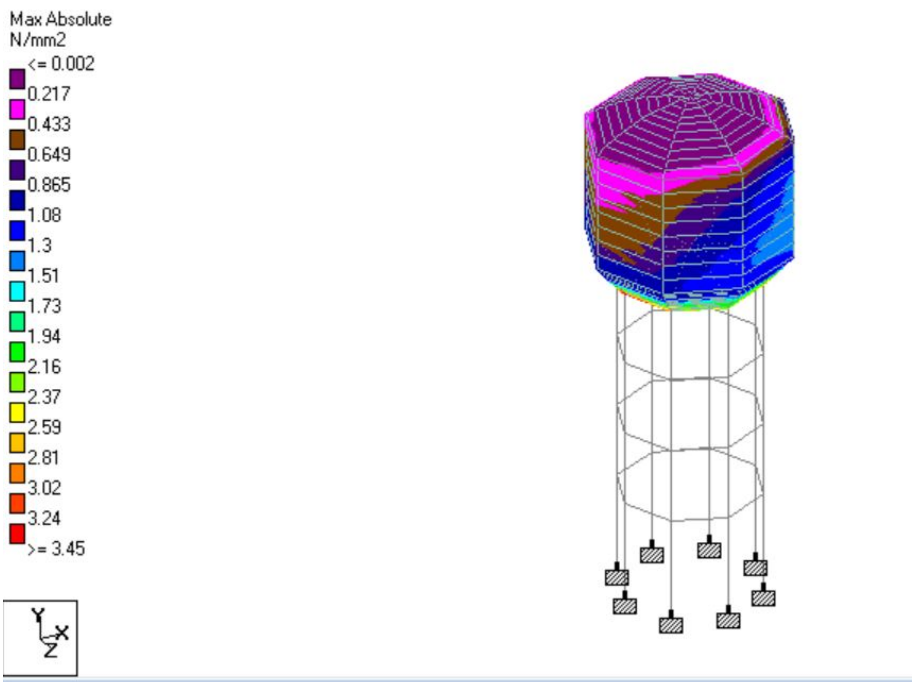

Fig 6.2 Maximum Absolute Pressure in the tank due to Hydrostatic loads

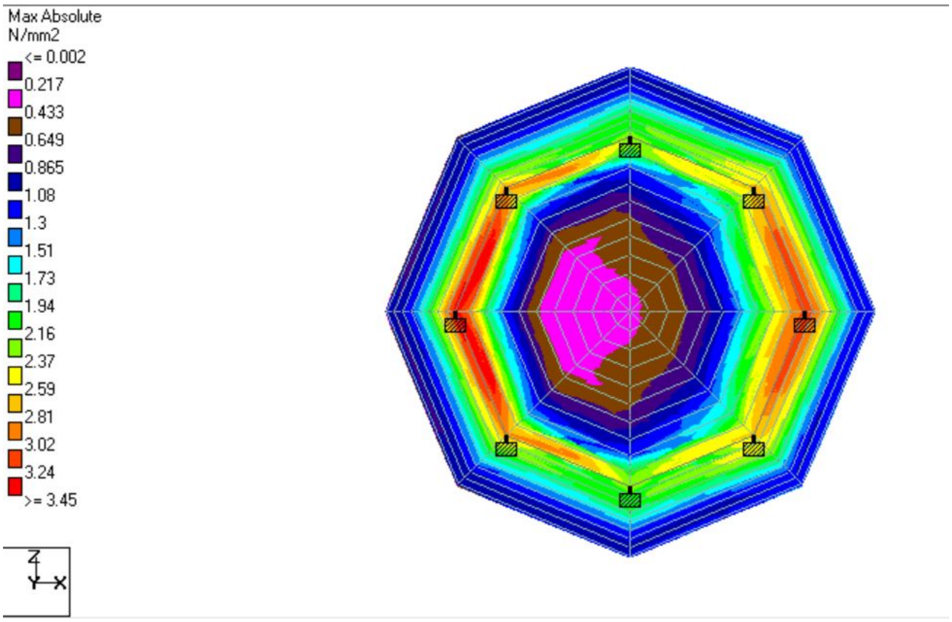

Fig 6.3 Maximum Absolute Pressure on the bottom dome due to Hydrostatic loads

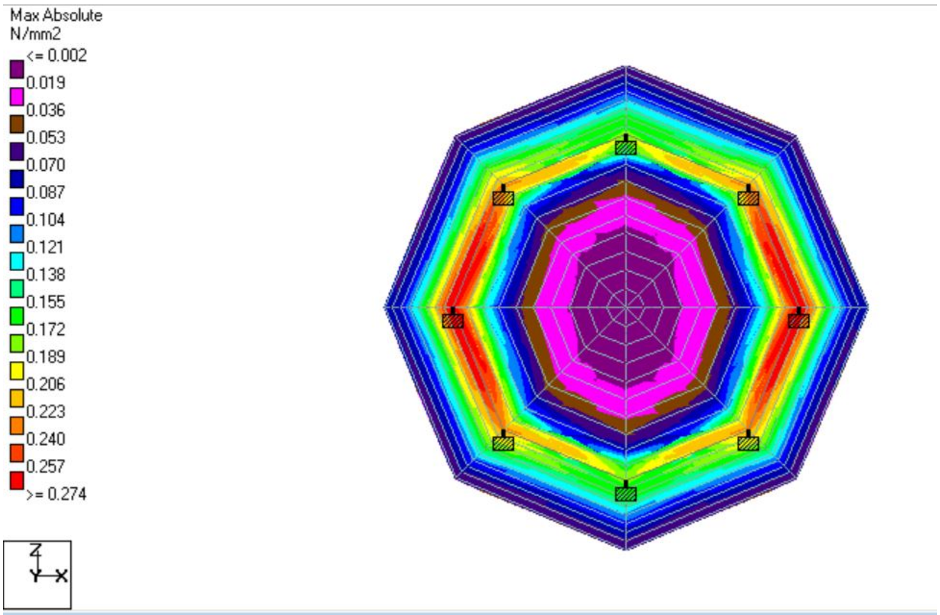

Fig 6.4 Maximum Absolute Pressure on the bottom dome due to Earthquake loads

Fig 6.5 Mode Shapes obtained from the Response Spectrum Analysis of the Elevated water tank

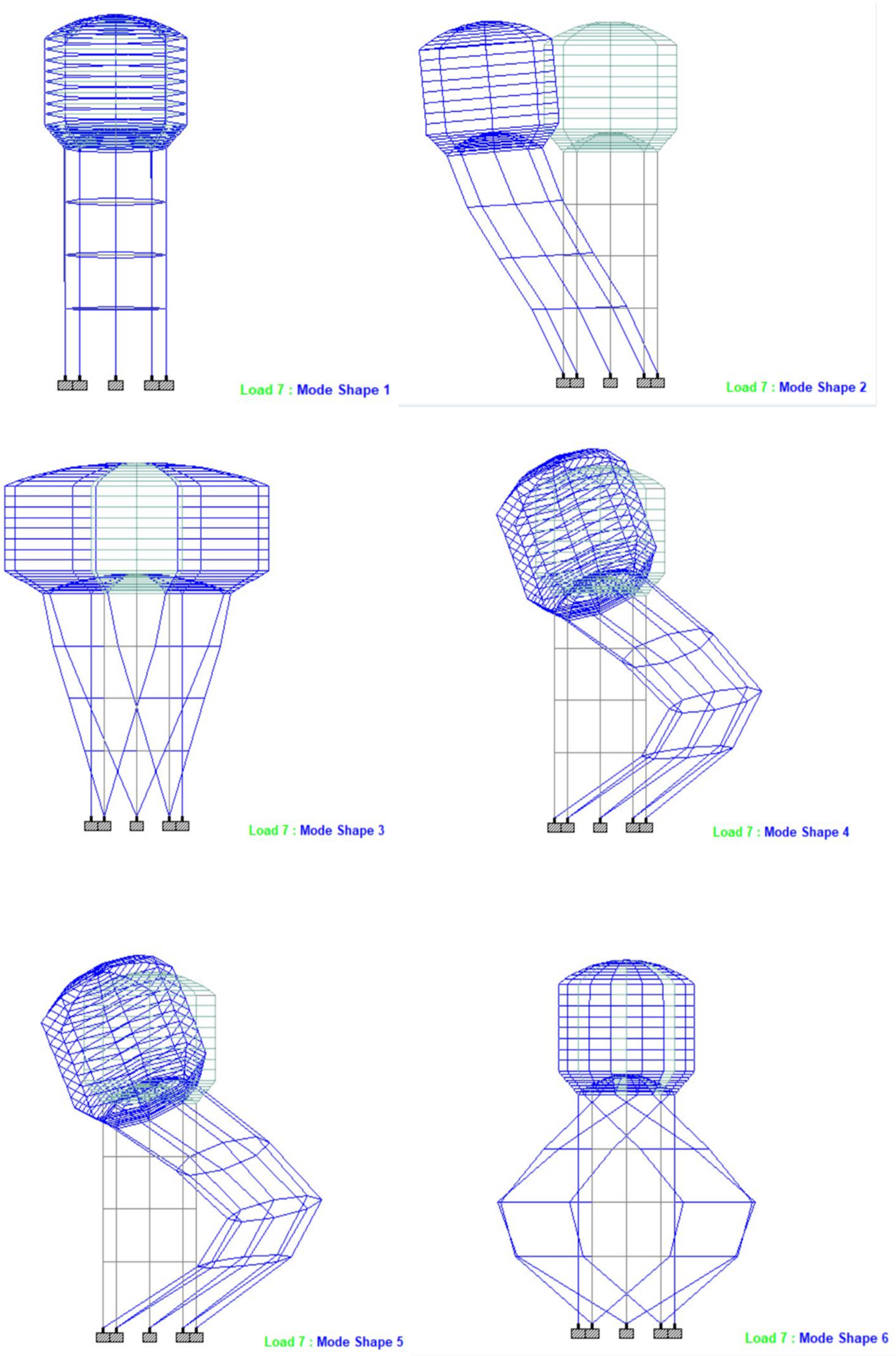

Fig 6.5 Mode Shapes obtained from the Response Spectrum Analysis of the Elevated water tank

Mode	Frequency Hz	Period seconds	Participation X %	Participation Y %	Participation Z %	Type
1	0.723	1.384	0.001	0	93.691	Elastic
2	0.723	1.384	93.691	0	0.001	Elastic
3	0.829	1.206	0	0	0	Elastic
4	4.278	0.234	2.323	0	2.643	Elastic
5	4.278	0.234	2.643	0	2.323	Elastic
6	5.539	0.181	0	0	0	Elastic
			98.658		98.658	

Table 6.3: Participation Factor of Mode Shapes in the X-direction and Z-direction obtained by Response Spectrum Analysis.

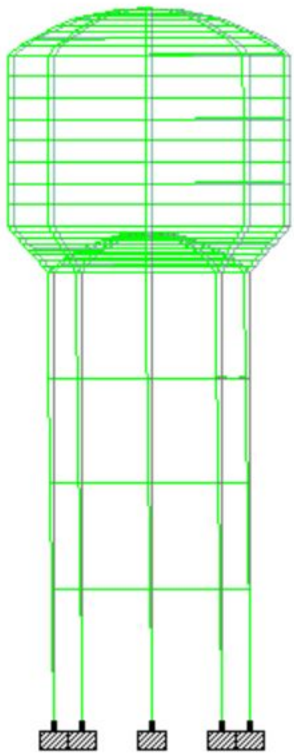

Fig 6.6 Deflection of the tank due to Hydrostatic Loads

STORY	HEIGHT	LOAD	AVG. DISP.(cm.) X-direction	AVG. DISP.(cm.) Z-direction	DRIFT X-direction	DRIFT Z-direction	RATIO	STATUS
1	0	1	0	0	0	0	L /999999	PASS
1	0	2	0	0	0	0	L /999999	PASS
2	5	1	0.5306	0	0.5306	0	L /942	PASS
2	5	2	0	0.5306	0	0.5306	L /942	PASS
3	9	1	1.0977	0	0.5671	0	L / 705	PASS
3	9	2	0	1.0977	0	0.5671	L / 705	PASS
4	13	1	1.6479	0	0.5501	0	L / 727	PASS
4	13	2	0	1.6479	0	0.5501	L / 727	PASS
5	17	1	2.021	0	0.3732	0	L / 1072	PASS
5	17	2	0	2.021	0	0.3732	L / 1072	PASS
6	17.22	1	2.0258	0	0.0048	0	L / 4573	PASS
6	17.22	2	0	2.0258	0	0.0048	L / 4572	PASS
7	17.27	1	2.0264	0	0.0006	0	L / 9143	PASS
7	17.27	2	0	2.0264	0	0.0006	L / 9148	PASS
8	17.44	1	2.0305	0	0.0041	0	L / 4051	PASS
8	17.44	2	0	2.0305	0	0.0041	L / 4050	PASS
9	17.52	1	2.0311	0	0.0006	0	L / 12663	PASS
9	17.52	2	0	2.0311	0	0.0006	L / 12660	PASS
10	17.66	1	2.0351	0	0.004	0	L / 3512	PASS
10	17.66	2	0	2.0351	0	0.004	L / 3511	PASS
11	17.74	1	2.0352	0	0	0	L /258673	PASS
11	17.74	2	0	2.0352	0	0	L /261247	PASS
12	17.88	1	2.0397	0	0.0045	0	L / 3074	PASS
12	17.88	2	0	2.0397	0	0.0045	L / 3074	PASS
13	17.93	1	2.0386	0	0.0011	0	L / 5000	PASS
13	17.93	2	0	2.0386	0	0.0011	L / 5000	PASS
14	18.09	1	2.0429	0	0.0042	0	L / 3892	PASS
14	18.09	2	0	2.0429	0	0.0042	L / 3892	PASS
15	18.23	1	2.0439	0	0.001	0	L / 13090	PASS
15	18.23	2	0	2.0439	0	0.001	L / 13091	PASS
16	18.31	1	2.0487	0	0.0048	0	L / 1704	PASS
16	18.31	2	0	2.0487	0	0.0048	L / 1704	PASS
17	18.34	1	2.0458	0	0.0029	0	L / 889	PASS
17	18.34	2	0	2.0458	0	0.0029	L / 889	PASS
18	18.42	1	2.0471	0	0.0013	0	L / 5722	PASS
18	18.42	2	0	2.0471	0	0.0013	L / 5722	PASS
19	18.46	1	2.048	0	0.0008	0	L / 5506	PASS
19	18.46	2	0	2.048	0	0.0008	L / 5506	PASS
20	18.53	1	2.0531	0	0.0052	0	L / 1353	PASS
20	18.53	2	0	2.0531	0	0.0052	L / 1353	PASS
21	18.75	1	2.0575	0	0.0044	0	L / 4963	PASS
21	18.75	2	0	2.0575	0	0.0044	L / 4963	PASS
22	19.56	1	2.0737	0	0.0162	0	L / 5021	PASS
22	19.56	2	0	2.0737	0	0.0162	L / 5021	PASS
23	20.38	1	2.0897	0	0.016	0	L / 5066	PASS
23	20.38	2	0	2.0897	0	0.016	L / 5066	PASS
24	21.19	1	2.1058	0	0.0161	0	L / 5046	PASS
24	21.19	2	0	2.1058	0	0.0161	L / 5046	PASS
25	22	1	2.122	0	0.0162	0	L / 5029	PASS
25	22	2	0	2.122	0	0.0162	L / 5029	PASS
26	22.81	1	2.1382	0	0.0162	0	L / 5027	PASS
26	22.81	2	0	2.1382	0	0.0162	L / 5028	PASS
27	23.62	1	2.1543	0	0.0161	0	L / 5032	PASS
27	23.62	2	0	2.1543	0	0.0161	L / 5032	PASS
28	24.44	1	2.1704	0	0.0161	0	L / 5032	PASS
28	24.44	2	0	2.1704	0	0.0161	L / 5032	PASS
29	25.25	1	2.1866	0	0.0162	0	L / 5030	PASS
29	25.25	2	0	2.1866	0	0.0162	L / 5030	PASS
30	25.57	1	2.1931	0	0.0065	0	L / 4989	PASS
30	25.57	2	0	2.1931	0	0.0065	L / 4989	PASS
31	25.87	1	2.1989	0	0.0058	0	L / 5010	PASS
31	25.87	2	0	2.1989	0	0.0058	L / 5010	PASS
32	26.13	1	2.2041	0	0.0052	0	L / 5022	PASS
32	26.13	2	0	2.2041	0	0.0052	L / 5022	PASS
33	26.36	1	2.2087	0	0.0046	0	L / 5024	PASS
33	26.36	2	0	2.2087	0	0.0046	L / 5024	PASS
34	26.55	1	2.2126	0	0.0039	0	L / 5025	PASS
34	26.55	2	0	2.2126	0	0.0039	L / 5025	PASS
35	26.71	1	2.2158	0	0.0032	0	L / 5028	PASS
35	26.71	2	0	2.2158	0	0.0032	L / 5028	PASS
36	26.84	1	2.2183	0	0.0025	0	L / 5031	PASS
36	26.84	2	0	2.2183	0	0.0025	L / 5031	PASS
37	26.93	1	2.22	0	0.0018	0	L / 5031	PASS
37	26.93	2	0	2.22	0	0.0018	L / 5032	PASS
38	26.98	1	2.2212	0	0.0011	0	L / 4855	PASS
38	26.98	2	0	2.2212	0	0.0011	L / 4855	PASS

Table 6.4: Story Drift results for Earthquake Loading obtained from STAAD Pro(Load 1: Earthquake Loading in X-direction; Load 2:Earthquake Loading in Z-direction; Allowable Drift: L/250)

CONCLUSION

- Rectangular and circular elevated water tanks are not designed to store significant amounts of water. For broad capacities, they are uneconomical and impractical.
- For large water storage capacities, Intze water tanks are the most suitable and cost-effective option.
- The maximum amount of node displacement and beam forces are accounted for by earthquake and hydrostatic loads. Due to hydrostatic and earthquake loads, the conical bottom carries the highest absolute pressure.
- The water tank's Response Spectrum Analysis yielded the mode shapes. According to IS 1893 Part 1:2016, six mode shapes are considered if the number of their participation factors is greater than 90%.
- In the Z and X directions, mode shape 1 and mode shape 2 have the highest participation (93.691 percent). As a result, when the water tank is excited, it will vibrate in the manner represented by mode shapes 1 and 2 in the majority of cases.
- The design used in this project was found to be practical and cost-effective, according to the provisions of the IS code. The storey drifted within permissible drift (=L/250) due to earthquake loading. As a result, the tank met the IS code's storey drift requirements.

SCOPE OF THE PROJECT

- According to IS 1893 Part 1: 2000, more than 60% of India is prone to earthquakes. Also, the population of India is increasing rapidly. Hence elevated water tanks of large capacities are the need of the hour.
- The design of these structure should be such that they are durable as well as economical. The design carried out can serve both the purposes very well.
- Further, cost optimization studies can be carried out so as to make these structure more economical.
- Prestressed concrete tank designs can be done which will be even more economical and will be able to hold greater capacities of water.

REFERENCES

[1]. George W. Housner, 1963 "The Dynamic Behaviour of Water Tank" *Bulletin of the Seismological Society of America. Vol.53, No.2,* pp. 381-387.

[2]. Dr Suchita Hirde, Dr Manoj Hedaoo, 2011 "Seismic Performance of Elevated Water Tank", *International Journal of Advanced Engineering Research and Studies Vol.1, Issue 1,* pp.78-87.

[3]. R. Livaoglu and A. Dogangun, 2007 "An Investigation About Effects of supporting systems on Fluid-elevated tanks Interaction" *SS: Special Structures Paper ID: SS148,* Tehran, Iraq.

[4]. Prasad S. Barve, Ruchi P. Barve, 2015 "Parametric Study to understand the Seismic Behaviour of Intze Tank Supported on Shaft" *International journal of engineering sciences & research technology Barve, 4(7):* July, 2015, pp. 161-168.

[5].More Varsha T., Mor Vyankatesh K., 2017 "Comparative Study on Dynamic Analysis of Elevated Water Tank Frame Staging and Concrete Shaft Supported",*IOSR Journal of Mechanical and Civil Engineering (IOSR-JMCE), Volume 14, Issue 1 Ver. I,* pp. 38-46.

[6]. Dona Rose K J , Sreekumar M & Anumod A S , 2015 "A Study of Overhead Water Tanks Subjected to Dynamic Loads" *International Journal of Engineering Trends and Technology (IJETT)*, Vol.28, Issue 7, pp.344-348.

[7]. Urmila Ronad, Raghu K.S, Guruprasad T.N, 2016 "Seismic Analysis of Circular Elevated Tank" *International Research Journal of Engineering and Technology*, Vol. 3, Issue 9, pp.903-907.

[8]. Nandagopan.M., Shinu Shajee, April 2017 "Dynamic Analysis of RCC Water Tanks with Varying Height of Water Level" *International Journal of Innovative Research in Science, Engineering and Technology,* Vol. 6, Issue 4,pp.6819-6826.

[9]. KeerthiGowda B.S, Gururaj M.H, Raghavendra.G, May-June 2014 "Dynamic analysis of overhead water tank under shaft Staging" *International Journal of Advanced Scientific and Technical Research,* Issue 4, volume 3, pp.505-511.

[10]. Dr. Abdulamir Atalla, Mawahib A. Gate'a, June 2015 "Dynamic analysis of elevated tanks having various supporting frame configurations" *Journal of University of Thi Qar,* Vol. (10), No. (2), pp.1-13.

[11]. IS 1893. "Indian Standard Criteria for Earthquake Resistant Design of Structures, Part 1-General Provisions and Buildings (Fifth Revision)", *Bureau of Indian Standards*, New Delhi, India, 2002.

[12].IS 1893. "Indian Standard Criteria for Earthquake Resistant Design of Structures, Part 2-Liquid Retaining Tanks (Revision of IS 1893(Part 2) Bureau of Indian Standards, New Delhi, India, 2006.

[13]. IITK-GSDMA guidelines for seismic design of liquid storage tanks.

[14]. IS: 456-2000, Indian Standard Code of Practice for Plain and Reinforced Concrete, Bureau of Indian Standards, New Delhi.

[15]. IS: 3370 (Part 1)-2009, General Requirements, Indian Standard Code of Practice for Concrete Structures for the Storage of Liquids, Bureau of Indian Standards, New Delhi.

[16]. IS: 3370 (Part 2)-2009, Reinforced Concrete Structures, Indian Standard Code of Practice for Concrete Structures for the Storage of Liquids, Bureau of Indian Standards, New Delhi.

[17]. IS: 3370 (Part 4)-2009, Design Tables, Indian Standard Code of Practice for Concrete Structures for the Storage of Liquids, Bureau of Indian Standards, New Delhi.

[18]. IS: 11682-1985, Indian Standard Criteria for Design of RCC Staging for Overhead Water Tanks, Bureau of Indian Standards, New Delhi.

[20]. IS: 875(Part 3), Indian Standard Code of Practice for Design Loads (other than Earthquake), for Buildings and Structures, Bureau of Indian Standards, New Delhi.

[21]. Advanced Reinforced Concrete Design ,N. Krishna Raju, Book Code: 023118, ISBN 8123912250. ***Publication*** Year: 2010, Cbs Publisher, New Delhi.

[22]."RCC Designs (Reinforced Concrete Structures)", B. C. Punmia, Ashok Kumar, ***Book Code***: 001644. ISBN: 8170088534. ***Publication*** Year: 2006, Laxmi ***Publication, New Delhi.***

[23] *.IS: 1172-1993,* Indian Standard Code of Basic Requirements for Water Supply, Drainage and Sanitation, Bureau of Indian Standards, New Delhi.

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