ANALYSIS AND DESIGN OF ELEVATED RCC WATER TANK

Under the Guidance of

Mr. Mani Mohan

(Assistant Professor)

by

KARANVIR SINGH RANA-111632

KUNDAN KUMAR-111644

Submitted in Partial Fulfillment of the Requirements for the Degree of

Bachelor of Technology

in

Civil Engineering

To,

DEPARTMENT OF CIVIL ENGINEERING

JAYPEE UNIVERSITY OF INFORMATION TECHNOLOGY,

WAKNAGHAT (H.P)

MAY-15

CERTIFICATE

This is to certify that this project report entitled **" Analysis and design of Elevated RCC Water Tank"** submitted to **Jaypee University of Information Technology,Waknaghat (H.P)** is a bonafide record of work done by "**Kundan Kumar (111644)** and **Karanvir Singh Rana(111632)"** under my supervision,till the end of their 8^{th} Semester of Bachelor of Technology in Civil Engineering.

MR.MANI MOHAN
(Assistant Professor)
Civil Engineering
Department
JUIT,Waknaghat

This is to Certify that above mentioned project work has been carried out by the said group of students.

DR. A.K.GUPTA
(Head of Department,Civil Engineering)
JUIT,Waknaghat

Place:

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KUNDAN KUMAR
(111644)

KARANVIR SINGH RANA
(111632)

Place:

Date:

CONTENTS

LIST OF TABLES

LIST OF FIGURES

ACKNOWLEDGEMENT

It is a genuine pleasure to express our deep sense of thanks and gratitude to our mentor and guide **Mr. Mani Mohan,**(Assistant Professor),Dept. of Civil Engineering, JUIT, Waknaghat. His dedication and keen interest above all his overwhelming attitude had been solely and mainly responsible for completion of our work. His timely and scholarly advice, meticulous scrutiny and scientific approach have helped us to a very great extent to accomplish this task.

We owe a deep sense of gratitude to **Dr.Ashok Kumar Gupta,** (Head of Dept.)**,**Civil Engineering, JUIT, Waknaghat,for keen interest on us at every stage of our project.His prompt inspirations, timely suggestions with kindness,enthusiasm and dynamism have enabled us to complete our project.

We thank profusely all the **Faculty members,**(Dept. of Civil Engineering), JUIT,Waknaghat,for their kind help and co-operation,constant encouragement and providing us necessary technical suggestions throughout our project pursuit.

KARANVIR SINGH RANA(111632)

KUNDAN KUMAR(111644)

ABSTRACT

Elevated Water Tanks are one of the most important lifeline structures in the earthquake regions. In major cities and also in rural areas elevated water tanks forms 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 has large mass concentrated at the top of slender supporting structure hence these structure are especially vulnerable to horizontal forces due to earthquakes. All over the world, the elevated water tanks were collapsed or heavily damaged during the earthquakes because of unsuitable design of supporting system or wrong selection of supporting system and underestimated demand or overestimated strength. So, it is very important to select proper supporting system and also need to study the response of Elevated Water Tanks to dynamic forces and 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 differ 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.

SECTION 1

INTRODUCTION

1.1 General

Indian sub- continent is highly vulnerable to natural disasters like earthquake,draughts, floods, cyclones etc. Majority of states or union territories are prone to one or multiple disasters. These natural calamities are causing many casualties and innumerable property loss every year. Earthquakes occupy first place in vulnerability. Hence, it is necessary to learn to live with these events. According to seismic code IS: 1893(Part I): 2000, more than 60% of India is prone to earthquakes. After an earthquake, property loss can be recovered to some extent however, the life loss cannot. The main reason for life loss is collapse of structures. It is said that earthquake itself never kills people; it is badly constructed structures that kill. Hence it is important to analyze the structure properly for earthquake effects. Water supply is a life line facility that must remain functional following disaster. Most municipalities in India have water supply system which depends on elevated water tanks for storage. Elevated water tank is a large elevated water storage container constructed for the purpose of holding a water supply at a height sufficient to pressurize a water distribution system. These structures have a configuration that is especially vulnerable to horizontal forces like earthquake due to the large total mass concentrated at the top of slender supporting structure. So it is important to check the severity of these forces for particular region.The main purpose of this project is to study the response of elevated water tank to dynamic forces and to find basic design parameters.For seismic analysis, it is necessary to consider the effect of hydrodynamic pressure on sides of container as well as base slab of container. It is also necessary to consider the effect of pressure due to wall inertia & effect of vertical ground acceleration in the seismic analysis of elevated water tank.

1.2 Objective:

- The main objective of our project is:

- To make a study on analysis and design of three different types of elevated water tank i.e,
  1) rectangular
  2) circular and,
  3) intze tank, for a given capacity.
- To carry out the static analysis of the tank finally,
- To know about the design philosophy for the safe and economical design of water tank.

1.3 Introduction to STAAD.pro

The STAAD.pro is explained briefly in the section below.

1.3.1 Introduction

Today's analysis tools allow engineers to refine designs to an unprecedented degree, and as a result, many utilities feel testing is not warranted. However, while great strides have been made in the analysis and design of water towers, differences between analysis results and full-scale tests still occur.

STAAD.Pro features a state-of-the-art user interface, visualization tools, powerful analysis and design engines with advanced finite element and static and dynamic analysis capabilities. From model generation, analysis and design to visualization and result verification, STAAD.Pro is the professional's choice for steel, concrete, timber, aluminum and cold-formed steel design of low and high-rise buildings, culverts, petrochemical plants, tunnels, bridges, piles and much more.The following key STAAD.Pro tools help simplify ordinarily tedious tasks:

- The STAAD.Pro Graphical User Interface incorporates Research Engineers' innovative tabbed page layout. By selecting tabs, starting from the top of the screen and heading down, you input all the necessary data for creating, analyzing and designing a model. Utilizing tabs minimizes the learning curve and helps insure you never miss a step.
- The STAAD.Pro Structure Wizard contains a library of trusses and frames. Use the Structure Wizard to quickly generate models by specifying height, width, breadth and number of bays in each direction. Create any customizable parametric structures for repeated use. Ideal for skyscrapers, bridges and roof structures.

1.3.2 Features of STAAD.Pro

- “Concurrent Engineering” based user environment for model development, analysis, design, visualization and verification
- Full range of analysis including static, P-delta, pushover, response spectrum, time history, cable (linear and non-linear), buckling and steel, concrete and timber design included with no extra charge
- Object-oriented intuitive 2D/3D graphical model generation
- Pull down menus, floating tool bars, tool tip help
- Quick data input through property sheets and spreadsheets

1.3.3 Load Types and Generation

- Categorized load into specific load group types like dead, wind, live, seismic, snow, user-defined, etc. Automatically generate load combinations based on standard loading codes such as ASCE etc.
- One way loading to simulate load distribution on one-way slabs
- Patch and pressure loading on solid (brick) elements
- Element pressure loads can be applied along a global direction on any imaginary surface without having elements located on that surface.

SECTION 2

REVIEW OF LITERATURE

Much of a literature has been presented in the form of technical papers till date on the dynamic analysis of Elevated Water Tanks. Different issues and the points are covered in that analysis i.e. dynamic response to ground motion, sloshing effect on tank, dynamic response of framed staging etc. Some of those are analyzed below:

2.1.1 George W. Housner [1963]

The basic plot behind this paper was the Chilean Earthquake, took place in 1960. In this earthquake, most of the elevated water tanks totally collapsed or badly distorted. This paper clearly speaks about the relation between the motion of water in the tank with respect to tank and motion of whole structure with respect to ground. He has considered three basic conditions for this analysis. He said that if water tank is fully filled i.e. without free board then the sloshing effect of water is neglected, if the tank is empty then no sloshing as water is absent. In above two cases water tower will behave as one-mass structure. But in third case i.e. water tank is partially filled, the effect of sloshing must be considered. In that case the water tower will behave as two-mass structure. Finally he concluded that the tank fully filled is compared with the partially filled tank then it is seen that the maximum force to which the half-full tank is subjected may be significantly less than half the force to which the full tank is subjected. The actual forces may be as little as 1/3 of the forces anticipated on the basic of a completely full tank.

2.1.2 Sudhir Jain K. & U. S. Sameer [1991]

IS code provision for seismic design of elevated water tanks have been revised. It was seen that, due to absence of a suitable value of performance factor for tanks, the code provision for rather low seismic design force for these structure. Simple expressions are derived, which allow calculations of staging stiffness, and hence the time period, while incorporating beam flexibility. The code must include an appropriate value of performance factor, say 3.0 for calculation of seismic design force for water tanks. An earthquake design criteria is incomplete, unless clear specifications are include on how to calculate the time period. A method for calculating the staging stiffness including beam flexibility and without having to resort to finite element type analysis has been presented. This method is based on well-known portal method which has been suitably developed to incorporate the beam flexibility and the three dimensional behavior of the staging.

2.1.3 Sudhir Jain K. & M. S. Medhekar [1993]

The basic plot behind this paper is to modify & suggestion in IS: 1893-1984. The major revisions suggested are:

1. No provision for ground supported tanks with rigid & flexible walls in above IS code. This provision must be included in the seismic analysis.
2. The single degree of freedom idealization of tank is to be replaced by two or three degree of freedom idealization.
3. A performance factor (K) of 3.0 is suggested for all types of tank.
4. The bracing beam flexibility is to be included in the calculation of lateral stiffness of supporting system of tank.
5. In the seismic analysis, the effect of Convective hydrodynamic pressure is to be included.
6. A simplified hydrodynamic pressure distribution is suggested for stress analysis of tank wall.

2.1.4 Sudhir K. Jain & Sajjad Sameer U [1993]

The basic plot behind this paper is to modify and suggestions in IS: 1893-1984 & suggestion given by Sudhir K Jain & M.S.Medhekar. Above author considered all the suggestion given by Sudhir Jain & Medhekar and added some extra suggestions :-

1. In the seismic analysis, the effect of accidental torsion must be included.
2. An expression for calculating sloshing height of water may be introduced in the code.
3. The effect of hydrodynamic pressure for tanks with rigid wall and the tanks with flexible wall should be considered separately, as force in the tanks with flexible wall is higher than those tanks with rigid wall.
4. The stresses due to hydrodynamic pressure in the tank wall and base should be given in the form of table.

2.1.5 O. R. Jaiswal & S. K. Jain [2005]

Recognizing the limitations and shortcomings in the provision of IS:1893-1984, Jain and Medhekar, Jain and Sameer set of provisions on seismic design of liquid storage tanks, the author has given some recommendations –

1. Design horizontal seismic coefficient given in revised IS: 1893(Part-1)-2002 is used and values of response reduction factor for different types of tanks are proposed.
2. Different spring-mass model for tanks with rigid & flexible wall are done away with; instead, a single spring-mass model for both types of tank is proposed.
3. Expressions for convective hydrodynamic pressure are corrected.
4. Simple expression for sloshing wave height is used.
5. New provisions are included to consider the effect of vertical excitation and to describe critical direction of earthquake loading for elevated water tanks with frame type staging.

2.1.6 Gareane A. I. Algreane, S. A. Osman & O. A. Karim [2008]

This paper is related with the soil & water behavior of elevated concrete water tank under seismic load. An artificial seismic excitation has been generated according to Gasparini and Vanmarcke approach, at the bedrock, and then consideration of the seismic excitation based on one dimension nonlinear local site has been carried out. Author has chosen seven cases to make comparisons with direct nonlinear dynamic analysis, mechanical models with and without soil structure interaction for single degree of freedom, two degree of freedom, and finite element method (FEM) models. The analysis is based on superposition model dynamic analysis. Soil structure interaction and fluid structure interaction have been accounted using direct approach and added mass approach respectively.

2.1.7 W. H. Boyce

The response of a simple steel water tank has been measured during earthquakes and vibration tests. Calculations of the period of vibration of the tank have been made taking ground yielding and water sloshing into account. Excellent agreement has been obtained between measured and calculated results. The response of the tower during the earthquake motion has been calculated from ground accelerogram and the agreement between measured and calculated response was found to be reasonable.

From his experimental study he conclude that –

(1) Water sloshing must be considered when calculating the period of vibration of water towers.The use of total water mass in 2-DOF simplification is not valid.
(2) The simplification to 2-DOF system where ground yielding effects are accounted for equivalent spring stiffness of the tower is adequate and produces the results agreeing well with experimental values.
(3) The analytical producers used to calculate the response of structure from ground accelerograms provide a responsible prediction of structure response.

2.1.8 Dr. Suchita Hirde & Dr. Manoj Hedaoo [2011]

This paper presents the study of seismic performance of the elevated water tank for various seismic zones of India for various heights and capacity of elevated water tanks for different soil conditions. The effect of height of water tank, earthquake zones and soil conditions on earthquake forces have been presented in this paper with the help of analysis of 240 models of various parameters.
In this paper, the study is carried out on RCC circular elevated water tank with M-20 grade of concrete and Fe-415 grade of steel & SMRF are considered for analysis. Elevated water tank having 50,000 liters and 100,000 liters capacity with staging height 12 m. 16 m, 20 m, 24 m, 28 m considering 4 m height of each panels are considered for the study. Author has given following conclusions from his analysis – (1) Seismic forces are directly proportional to the Seismic Zones. (2) Seismic forces are inversely proportional to the height of supporting system. (3) Seismic forces are directly proportional to the capacity of water tank. (4) Seismic forces are higher in soft soil than medium soil, higher in medium soil than hard soil. Earthquake forces for soft soil is about 40-41% greater than that of hard soil for all earthquake zones and tank full and tank empty condition.

2.2 Conclusion based upon literature survey

Analysis & design of elevated water tanks against earthquake effect is of considerable importance. These structures must remain functional even after an earthquake. Elevated water tanks, which typically consist of a large mass supported on the top of a slender staging, are particularly susceptible to earthquake damage. Thus, analysis & design of such structures against the earthquake effect is of considerable importance.
After detailed study of all the papers, following points are to be considered at the time of seismic analysis of elevated water tank

1. In India, there is only one IS code i.e. IS 1893: 1984, in which provisions for seismic design of elevated water tanks are given. IS 1893(Part-1): 2002 is the fifth revision of IS 1893, still it is under revision. So detail criteria for aseismic analysis of elevated water tank are not mentioned in above IS code. Thus, the recommendations & suggestions given by all the above author has to be considered at the time of analysis. IITK-GSDMA has given some guidelines for seismic design of elevated water tank that should consider at the time of analysis.
2. Most elevated water tank are never completely filled with water. Hence, a two – mass idealization of the tank is more appropriate as compared to one-mass idealization.
3. Basically, there are three cases that are generally considered while analyse the elevated water tank – (1) Empty condition. (2) Partially filled condition. (3) Fully filled condition. For (1) & (3) case, the tank will behave as a one-mass structure and for (2) case the tank will behave as a two-mass structure.
4. If we compared the case (1) & (3) with case (2) for maximum earthquake force, the maximum force to which the partially filled tank is subjected may be less than half the force to which the fully filled tank is subjected. Actual forces may be as little as 1/3 of the forces anticipated on the basis of a fully filled tank.
5. During the earthquake, water in the tank get vibrates. Due to this vibration water exerts impulsive & convective hydrodynamic pressure on the tank wall and the tank base in addition to the hydrostatic pressure. The effect of impulsive & convective hydrodynamic pressure should consider in the analysis of tanks. For small capacity tanks, the impulsive pressure is always greater than the convective pressure, but it is vice-versa for tanks with large capacity. Magnitudes of both the pressure are different.
6. The effect of water sloshing must be considered in the analysis. Free board to be provided in the tank may be based on maximum value of sloshing wave height. If sufficient free board is not provided, roof structure should be designed to resist the uplift pressure due to sloshing of water.
7. Earthquake forces increases with increase in Zone factor & decreases with increasing staging height. Earthquake force also depends on the soil condition.

SECTION 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:

Water consumption rate (Per Capita Demand in litres per day per head)
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 demands, which a city may have, may be broken into following class

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	Trasmission 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 foe 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 sewered houses.

• **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 unauthorised 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 <u>Design Periods & Population Forecast</u>

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.

SECTION 4

DESIGN CONSIDERATIONS

4.1 DESIGN REQUIREMENT OF CONCRETE (IS-3370)

In water retaining structure a dense impermeable concrete is required therefore, proportion of fine and course 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 1. For calculation purposes the cover is also taken into concrete area. Cracking may be caused due to restraint to shrinkage, expansion and contraction of concrete due to temperature or shrinkage and swelling due to moisture effects.

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.3.1 (b)**.**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.3.1(a).

(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.3.1(c). This type of joint requires the provision of an initial gap between the adjoining parts of a structure which by closing or opening accommodates the expansion or contraction of the structure.

(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 Fig. 3.1(d).This type of joint is provided between wall and floor in some cylindrical tank designs.

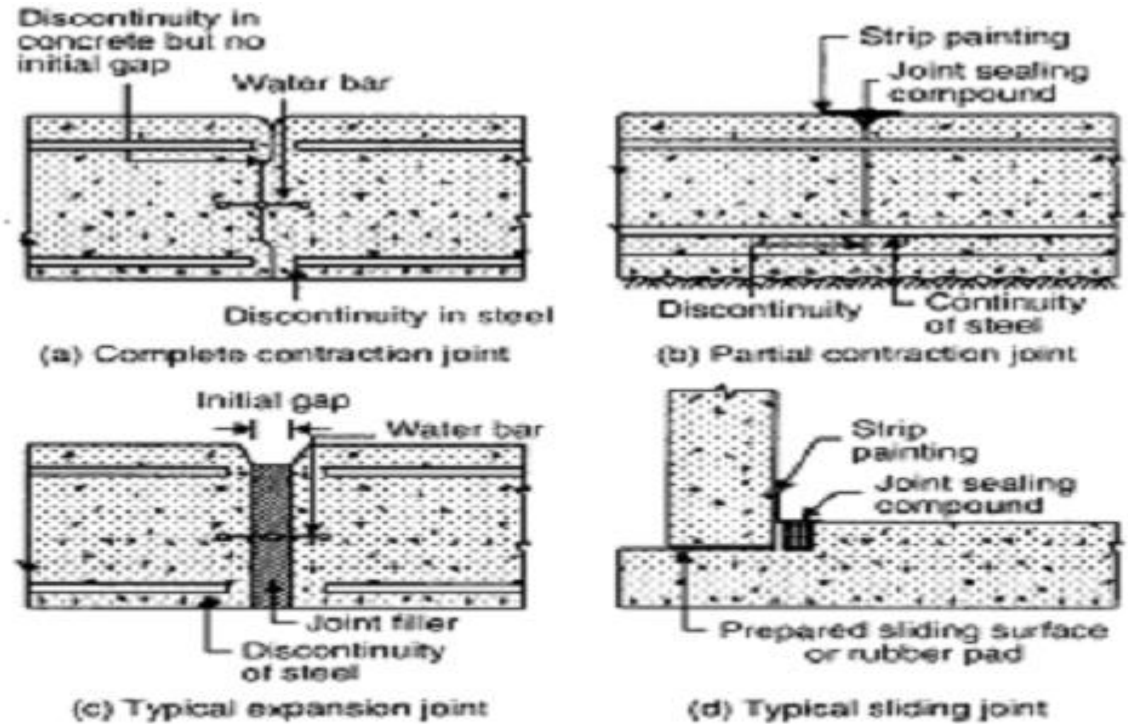

Fig 4.1 Different types of joints

4.2.2 Contraction Joints:

This type of joint is provided for convenience in construction. Arrangement is made to achieve subsequent continuity without relative movement. One application of these joints is between successive lifts in a reservoir wall. A typical joint is shown in Fig.3.2.The number of joints should be as small as possible and these joints should be kept from possibility of percolation of water.

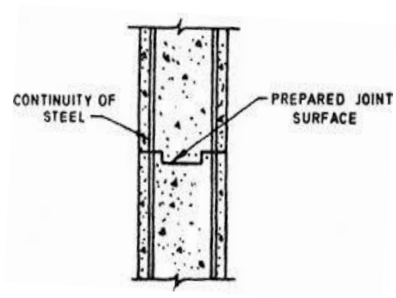

Fig.4.2-A typical contraction joint

4.2.3 Temporary Joints:

A gap is sometimes left temporarily between the concrete of adjoining parts of a structure which after a suitable interval and before the structure is put to use, is filled with mortar or concrete completely as in Fig.3.3 or with suitable jointing materials. In the first case width of the gap should be sufficient to allow the sides to be prepared before filling.

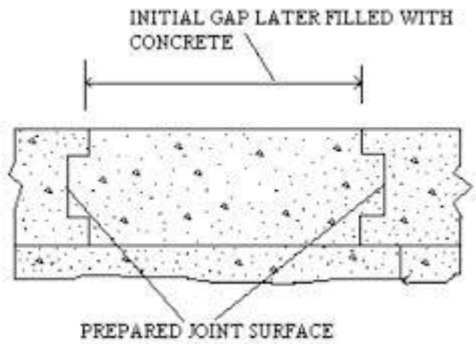

Fig. 4.3-A typical temporary joint

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 1. The permissible tensile stresses due to bending apply to the face of the member in contact with the liquid. In members less than 225mm. thick and in contact with liquid on one side these permissible stresses in bending apply also to the face remote from the liquid.

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

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

4.3.2 ***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 in Table 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 3.2.1.

(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 selfweight.

(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.

4.3.4 Walls

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 1)

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

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

4.3.5 Minimum Reinforcement

(a)The minimum reinforcement in walls, floors and roofs in each of two directions at right angles shall have an area of 0.3 per cent of the concrete section in that direction for sections up to 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.

SECTION 5

DESIGN OF RECTANGULAR AND CIRCULAR WATER TANK

5.1 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 = 276,000 liters/day

We design the tank for 2.5 days capacity

Volume of the tank: 276000*2.5 = 690000 liters = 690 m^3

5.2 Design of rectangular water tank-:

Taking L/B < 2

Assume H= 6 m

We get L= 14.5 m and B= 8 m

We use M25 concrete and Fe 415

h'= max.(H/4,1)= 1.5 m

w = (H-h')γ_w = 4.5*10 = 45 kN/m

Fixed end moment:

Long wall: $\frac{wL^2}{12} = \frac{45\times 14.5^2}{12} =$ 788.4 kN-m

Short wall: $\frac{wB^2}{12} = \frac{45\times 8^2}{12} =$ 240 kN-m

Applying moment distribution for the tank wall at joint A

Joint	A	
Member	AB	AD
Distribution factor	0.35	0.65
FEM	788.4	-240
Final moment	596.5	-596.5

Support moment: Long wall = 596.5 kN-m

Short wall = -596.5 kN-m

Span moment: Long wall = $\frac{wL^2}{8} - $ FEM = 1183-596.5 = 586 kN-m

Short wall = $\frac{wL^2}{8} - $ FEM = 360-596.5 = -236 kN-m

Calculation for tension on water tank wall:

Tension: Long wall = $\frac{wB}{2} = \frac{45\times8}{2} =$ 180 kN-m

Short wall = $\frac{wL}{2} = \frac{45\times14.5}{2} =$ 326.3 kN-m

	Long wall	**Short wall**
Support	596.5 kN-m	-596.5 kN-m
Span	586 kN-m	-236.5 kN-m
Tension	180 kN-m	326.3 kN-m

Calculate the thickness of rectangular wall:

Thickness, t = $\sqrt{\frac{M}{0.5\times\sigma_{cbc}\times k_b\times j_b\times B}}$

Where,

M = Design moment in kN-m × 10^6

σ_{cbc}= permissible stress in concrete ($\frac{f_{ck}}{3}$)

k_b = depth of neutral axis for balanced section = $\frac{280}{280+\sigma_{st}}$

$j_b = 1 - \frac{k_b}{3}$

B = width of rectangular wall = 1000 mm

t= $\sqrt{\frac{596.5\times10^6}{0.5\times8.33\times0.39\times0.87\times1000}}$ = 650 mm

As we can see that the thickness of wall is very large and hence it is uneconomical and impractical to construct the tank with rectangular geometry.

Hence we will now design the water tank for cylindrical geometry.

5.3 Design of circular water tank

Assuming rigid base:

Grade of concrete f_{ck} = 25 N/mm^2

Grade of steel f_y = 415 N/mm^2

Height of water tank = 4.5 m

Volume = 690 m^3

Diameter of water tank = $\sqrt{\frac{690}{\pi\times0.25\times4.5}} = 14$ m

Assuming, $\frac{H^2}{Dt}$ = 8

Thickness, t = $\frac{4.5^2}{14\times8}$ = 180 mm

$\sigma_{cbc} = 8.33\ N/mm^2$

$\sigma_{st} = 150\ N/mm^2$

$\sigma_{cbt} = 1.7\ N/mm^2$ $\{from\ table\ 2\ IS{:}3370\ partII\ 2009\}$

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

τ_c = 1.7 N/mm^2

Modular ratio, m = 13.33

Tension steel (in form of rings)

Depth	Coefficient	T	Ast req	Ast	Dia	Spacing	Ast prov	Fct
0								
0.45	0.104	32.76	218.4	399.6	8	250	402	**Safe**
0.9	0.218	68.67	457.8	457.8	8	210	479	**Safe**
1.35	0.335	105.525	703.5	703.5	8	140	718	**Safe**
1.8	0.443	139.545	930.3	930.3	10	160	982	**Safe**
2.25	0.534	168.21	1121.4	1121.4	12	200	1131	**Safe**
2.7	0.575	181.125	1207.5	1207.5	12	180	1257	**Safe**
3.15	0.53	166.95	1113	1113	12	200	1131	**Safe**
3.6	0.381	120.015	800.1	800.1	10	190	827	**Safe**
4.05	0.151	47.565	317.1	399.6	8	250	402	**Safe**
4.5								

Bending steel (in form of vertical bars)

Depth	Coefficient	M	Abs M	Ast req	Ast	Dia	Spacing	Ast prov
0								
0.45	0	0	0					
0.9	0.0001	0.091125	0.091125	3.5	399.6	10	190	413
1.35	0.0002	0.18225	0.18225	7.1	399.6	10	190	413
1.8	0.0008	0.729	0.729	28.3	399.6	10	190	413
2.25	0.0016	1.458	1.458	56.7	399.6	10	190	413
2.7	0.0028	2.5515	2.5515	99.2	399.6	10	190	413
3.15	0.0038	3.46275	3.46275	134.6	399.6	10	190	413
3.6	0.0029	2.642625	2.642625	102.7	399.6	10	190	413

4.05	-0.0022	-2.00475	2.00475	77.9	399.6	10	190	413
4.5	-0.0146	-13.3043	13.30425	517.3	517.3	10	150	524

p_{tmin} = 0.22 %

A_{stmin}= 399.6 mm^2

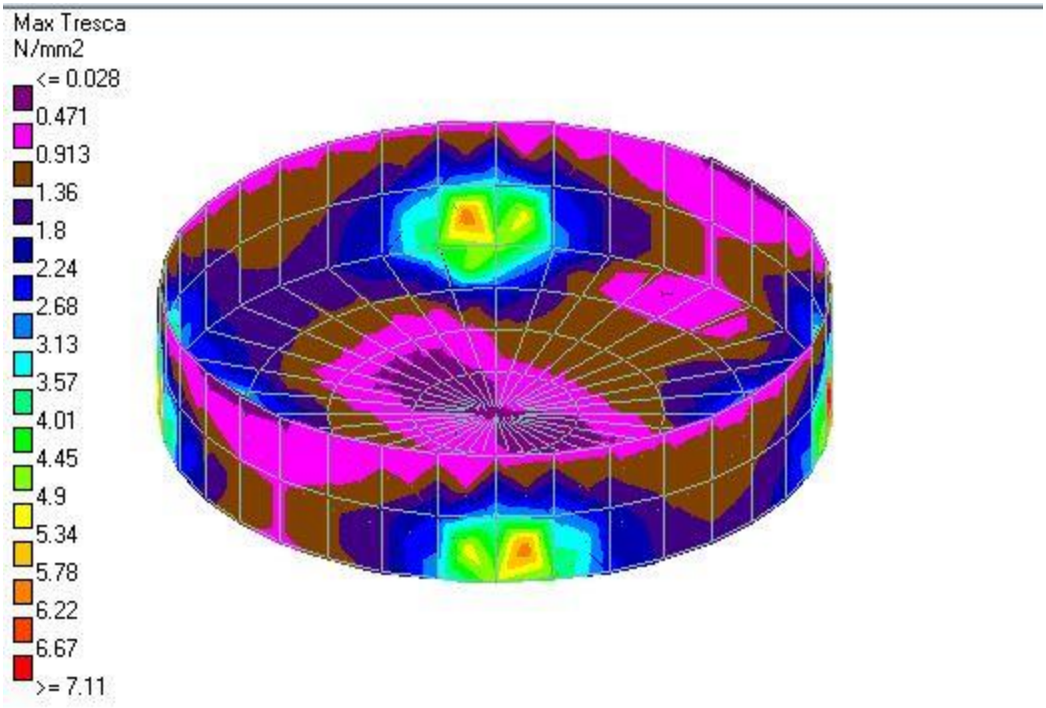

Fig. 5. 1-Stress in plates of cylindrical tank

5.4 Design of supporting structure:

f_{ck} = 25 N/mm^2

f_y = 415 N/mm^2

Total dead load of tank: $\left[\frac{\pi(D^2-d^2)}{4} + \frac{2\times\pi D^2}{4}\right] \times 25$ = 8926 kN

Total dead load due to water: V*γ_w= 690*9.81 = 6768.9 kN

Total dead load = 15695 kN

Total dead load on each column = 1961 kN

Self weight of column of cross section (300x500) = 39 kN

Total vertical load on each column = 2006.25 kN

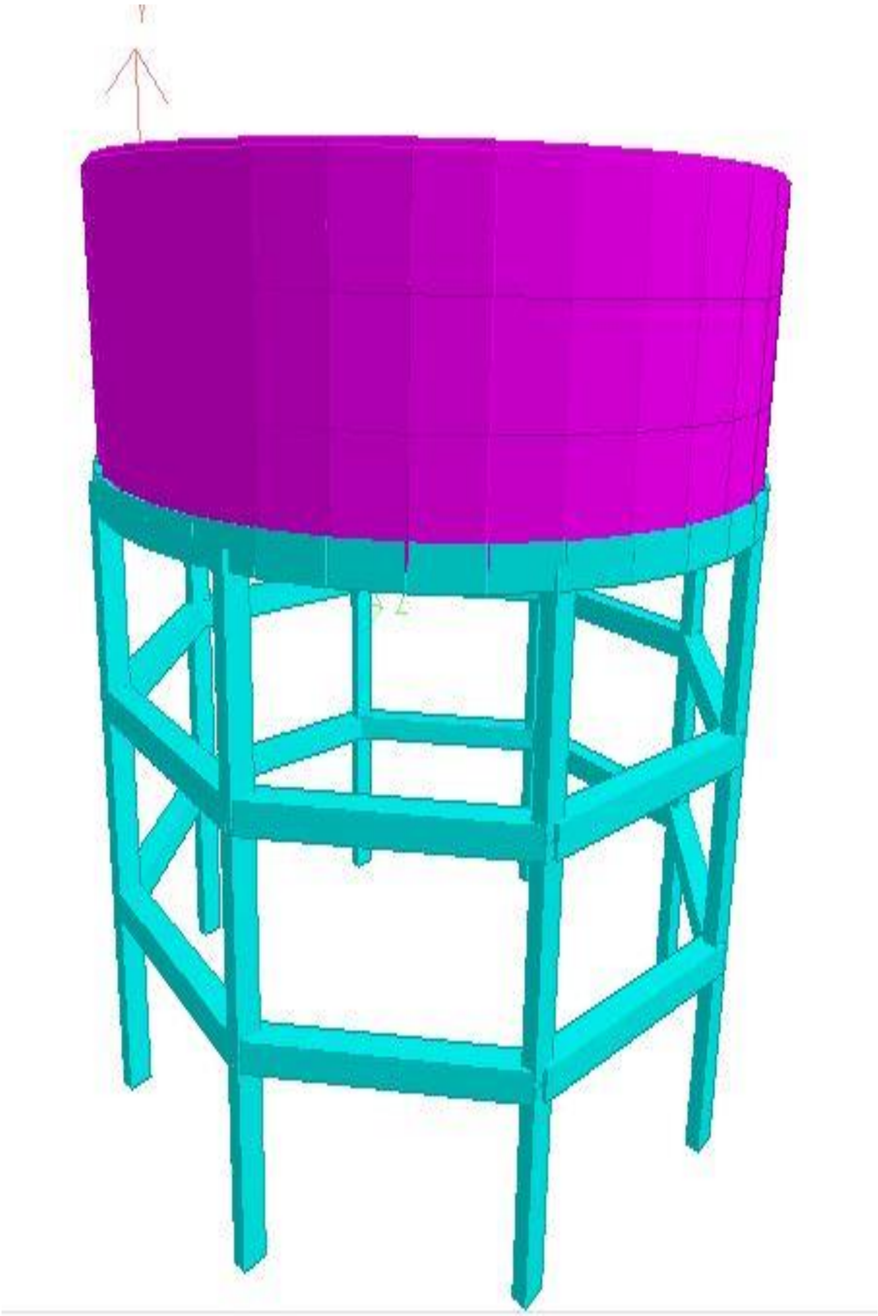

Fig. 5. 2-Designed Cylindrical tank

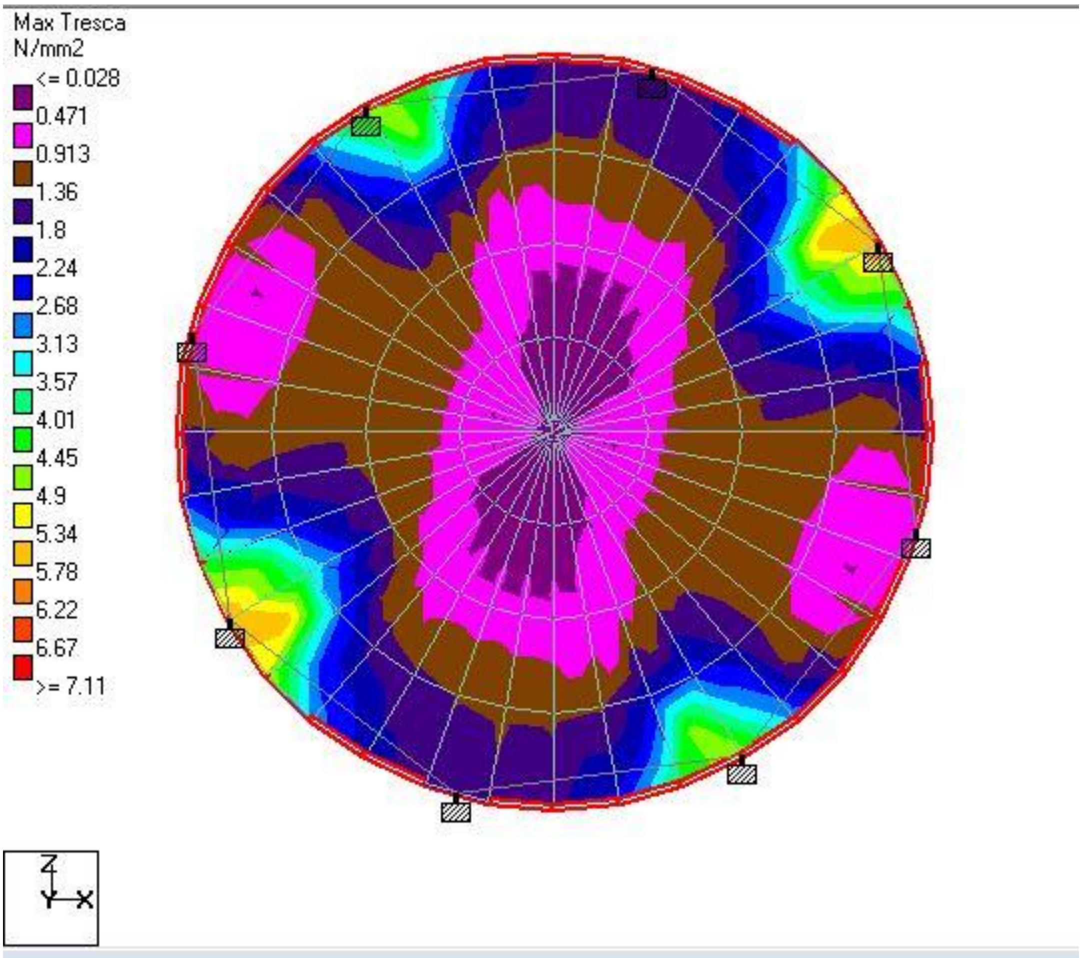

Fig. 5. 3-Top view of plate stress

5.5 Calculation of wind load

Design wind speed is calculated as :

$$V_z = V_b k_1 k_2 k_3$$

k_1= risk coefficient: 1.07 {terrain category - 2}

k_2= terrain height: 1.05

k_3= topography factor: 1

$V_z = 52.81\ m/s$

p_d= 0.6 $V^2{}_z$ = 1.7 kN/m^2

Reduction coefficient for circular shape = 0.7

Wind force on cylindrical wall: 8 x 25 x 0.7 = 75 kN

Wind load (kN)	Distance from base
W = 79 kN	12.75 m
$\boldsymbol{W_1}$**= 31.2 kN**	10.5 m
$\boldsymbol{W_2}$**= 20.83 kN**	7 m
$\boldsymbol{W_3}$**= 10.4 Kn**	3.5 m

Loads and Moments:-

M = Moment at the base of column:

M =$[\boldsymbol{W} + W_1 + W_2 + W_3]$ x $\left(\frac{h}{2}\right)$ = 240.5 kN

Moment due to wind load about base:

$M_1 = (10 + 2.25) \times 75 + 10.5 \times 31.2 + 7 \times 20.83 + 10.4 \times 3.5$

M_1= 1466.07 kN-m

$M_1 = \sum M + \frac{V_1}{r_1}[\sum r^2]$

1466 = 240.5 + $\frac{V}{7}\left[2 \times 7^2 + 4 \times (\frac{7}{\sqrt{2}})^2\right]$

V = 44 kN

Total load on leeward column at base = 2006.25 +89 = 2095 kN

= 2100 kN

Moment in column at base = $\frac{240}{8}$ = 30 kN-m

Moment in direction of brace:

$M_{BC} = (M_{BA} + M_{BD}) \sec 45$ = 82 kN-m

Length of brace = $2r$ sin 22.5 = 5.36 m

Shear force in brace: $\frac{M}{\frac{1}{2} \times l} =$ 30.6 kN

Design of column section:-

P_u = 1.5 x 2100 = 3150 kN

M_u= 1.5 x 30 = 45 kN-m

Assuming 25 mm bars with 40 mm cover

$\grave{d}$ = 52.5 mm

$\frac{\grave{d}}{D}$ = 5.25/50 = 0.105

$\frac{P_u}{f_{ck}bD} = \frac{3150\times10^3}{25\times300\times500}$ = 0.84

$\frac{M_u}{f_{ck}bD^2} = \frac{3150\times10^6}{25\times300\times500^2}$ = 0.025

Refer to interaction chart:

$\frac{p}{f_{ck}}$= 0.14

P = 0.14 x 25 = 3.5%

$A_{st} = \frac{3.5\times300\times500}{100}$ = 5250 mm^2

Provide 12 bars of 25mm diameter

Transverse reinforcement:-

Use 10 mm φ bars

Spacing of ties: min (300, 16Φ, 48 φ)

Spacing = 300 mm

Design of braces:

M_u = 1.5 x 82 =123kN-m

V_u = 30 x 1.5 = 45 kN

b = 500mm, d = 450mm, D = 500mm

M_{ulim}= 349 kN-m

$A_{st} = \frac{0.36\times25\times500\times0.48\times450}{0.87\times415} = 2692\ mm^2$

Use 6 bars of 25 φ on each side

$\tau_v = \frac{V_u}{bd}$ = 0.2 N/mm^2

$$p_t = \frac{100A_{st}}{bd} = 1.2\%$$

τ_c= 0.688 > τ_v

We use 10mm-φ 2-legged stirrups at 250 mm c/c

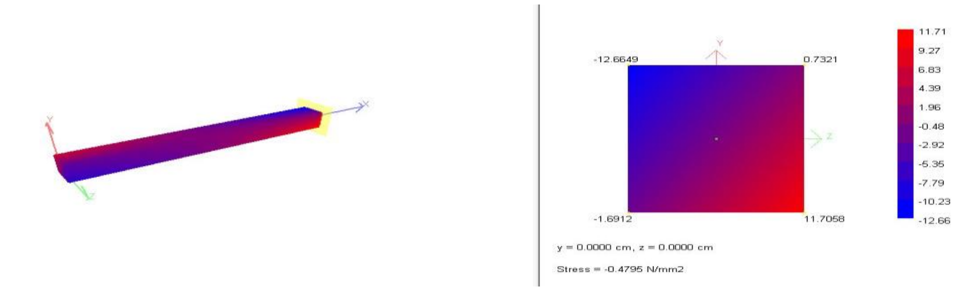

Fig. 5. 4-Stresses on bracings

Table 5. 1-Beam forces summary

All | Summary | Envelope

	Beam	L/C	Node	Fx kN	Fy kN	Fz kN	Mx kNm	My kNm	Mz kNm
Max Fx	829	8 GENERATE	372	**3716.157**	196.309	-43.335	-0.555	-48.143	-530.387
Min Fx	797	13 GENERAT	243	**-782.537**	-209.590	21.072	-76.151	-15.284	-573.417
Max Fy	779	13 GENERAT	63	636.654	**829.722**	-34.965	199.828	28.036	598.200
Min Fy	815	13 GENERAT	63	2987.794	**-799.630**	-36.422	-8.559	-22.720	-1684.550
Max Fz	815	12 GENERAT	63	2176.992	618.494	**201.620**	14.991	-414.027	1140.145
Min Fz	839	13 GENERAT	243	2177.000	-618.496	**-201.620**	14.991	414.027	-1140.147
Max Mx	788	12 GENERAT	153	160.411	122.717	-41.836	**285.310**	26.775	-104.109
Min Mx	787	12 GENERAT	143	614.997	-658.482	46.393	**-282.448**	-22.618	-357.786
Max My	839	13 GENERAT	243	2177.000	-618.496	-201.620	14.991	**414.027**	-1140.147
Min My	815	12 GENERAT	63	2176.992	618.494	201.620	14.991	**-414.027**	1140.145
Max Mz	839	12 GENERAT	243	2987.781	799.629	36.422	-8.559	22.720	**1684.547**
Min Mz	815	13 GENERAT	63	2987.794	-799.630	-36.422	-8.559	-22.720	**-1684.550**

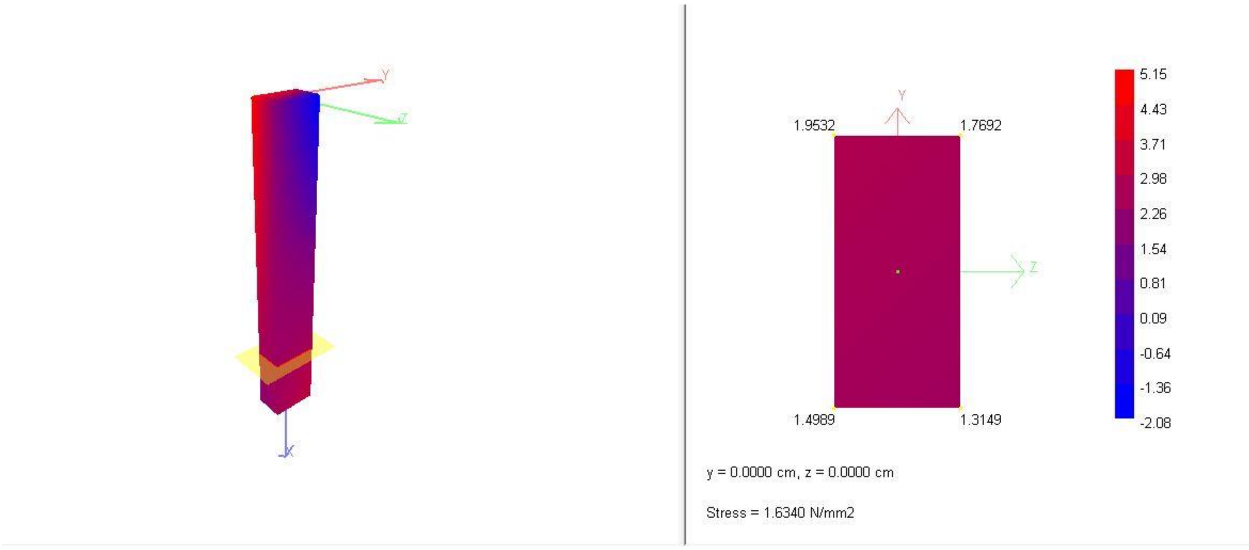

Fig. 5. 5-Stresses on Columns of supporting tower

Table 5. 2-Node displacement summary

All Summary

			Horizontal	Vertical	Horizontal	Resultant	Rotational		
	Node	L/C	X mm	Y mm	Z mm	mm	rX rad	rY rad	rZ rad
Max X	84	12 GENERAT	**160.277**	-8.358	-0.773	160.497	-0.000	0.000	-0.001
Min X	264	13 GENERAT	**-160.277**	-8.358	0.773	160.497	0.000	0.000	0.001
Max Y	14	1 LOAD CAS	106.572	**2.458**	-0.586	106.602	-0.000	-0.000	-0.000
Min Y	9	5 GENERATE	-0.000	**-317.338**	0.000	317.338	0.000	0.000	-0.000
Max Z	380	12 GENERAT	102.457	0.256	**15.760**	103.662	0.004	0.001	-0.004
Min Z	366	13 GENERAT	-102.457	0.256	**-15.760**	103.662	-0.004	0.001	0.004
Max rX	277	5 GENERATE	-0.260	-187.036	-0.279	187.036	**0.064**	0.000	0.000
Min rX	97	5 GENERATE	0.260	-187.036	0.279	187.036	**-0.064**	0.000	-0.000
Max rY	363	12 GENERAT	86.030	-3.957	-3.359	86.186	-0.003	**0.002**	-0.012
Min rY	366	12 GENERAT	99.832	-0.500	1.422	99.843	0.002	**-0.002**	-0.004
Max rZ	187	5 GENERATE	0.268	-186.917	-0.262	186.918	-0.000	0.000	**0.064**
Min rZ	5	5 GENERATE	-0.269	-186.917	0.262	186.917	0.000	0.000	**-0.064**
Max Rs	9	13 GENERAT	-156.964	-288.197	0.712	**328.170**	-0.000	0.000	-0.001

SECTION 6

DESIGN OF INTZE TANK

6.1 Introduction

Overhead water tanks of various shapes can be used as service reservoirs, as a balancing tank in water supply schemes and for replenishing the tanks for various purposes.

Reinforced concrete water towers have distinct advantages as they are not affected by climatic changes, are leak proof, provide greater rigidity and are adoptable for all shapes.

Components of a water tower consists of-
(a) Tank portion with -
(1) Roof and roof beams (if any)
(2) sidewalls
(3) Floor or bottom slab
(4) floor beams,includingcircular girder
(b) Staging portion, consisting of-
(5) Columns
(6)Bracings and
(7)Foundations

Types of water Tanks may be –
(a) Square-open or with cover at top (b)Rectangular-open or with cover at top
(c) Circular-open or with cover at which may be flat or domed.

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

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

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

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

The design of the tank will involve the following

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

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

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

(4) **Ring beam at the junction of the cylindrical walls and the conical wall.**

This ring beam is provided to resist the horizontal component of the reaction of the conical wall on the cylindrical wall. The ring beam will be designed for the induced hoop tension.

(5) **Conical slab**, This will be designed for hoop tension due to water pressure.

The slab will also be designed as a slab spanning between the ring beam at top and the ring girder at bottom.

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

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

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

(9) **Foundations :** A combined footing is usuals provided for all supporting columns.

When this is done it is usual to make the foundation consisting of a ring girder and a circular slab.

6.2 DOMES

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

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

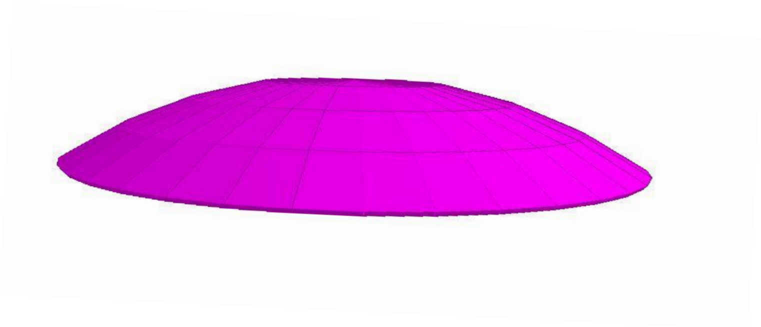

Fig. 6. 1 –Top Dome of an Intze tank

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 shells must be thick enough to allow space and protection for two layers of reinforcement. From this point of view, 80 mm is considered as minimum thickness of shell.

6.3 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 steel provided is

0.15% of the sectional area in each direction along the meridians as well as along the latitudes. The 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 1.1 N/mm2 to 1.70 N/mm2 for direct tension and 1.5 N/mm2 to 2.40 N/mm2 for tension due to bending in liquid resisting structure depending on the grade of concrete.

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

In case of domes with lantern opening with concentrated load acting there, ring beam has to be provided at the periphery of the opening. The edge beam there will, however, be subjected to hoop compression in place of hoop tension. Openings may be provided in the dome as required from other functional or architectural requirements. However, reinforcement has to be provided all around the opening.The meridian and hoop reinforcement reaching the opening should be well anchored to such reinforcement.

Minimum reinforcement of each of two directions at right angles shall have an area of 0.3% for 100 mm thick concrete to 0.2% for 450 mm thick concrete wall. In floor slabs, minimum reinforcement to be provided is 0.15%. The minimum reinforcement as specified above may be decreased by 20%), if high strength deformed bars are used. Minimum cover to reinforcement on the liquid face is 25 mm or diameter of the bar, whichever is larger and should be increased by 12 mm for tanks for sea water or liquid of corrosive character.

6.4 <u>Design of Intze tank</u>

Design of Intze tank for a capacity of 690,000 lts.

Assuming height of tank above GL 16m.

No. of columns=8

Depth of foundation=1 m below GL.

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: σ_{st} = 150N/mm^2

Dimensions:-

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

Dia.,D =10.5m

Height of cylindrical portion = 6.5m

Depth of conical dome = 1.75m

Spacing of bracings = 4m

Dia. of supporting tower = 7.5m

6.4.1 <u>Design of top Dome</u>

Assume thickness of dome slab =100mm

Live load on dome =1.5 kN/m^2

Self wt. = 2.4 kN/m^2

Finishes = 0.1 kN/m^2

Total load,w = 4 kN/m^2

Central rise,r =1.75m

Radius of dome,R: r(2R-r) =(0.5D)^2

R=8.75m

cosΘ = cos(36.86) =0.8

Meridional Thrust,T_1 =$\frac{wR}{1+cos\theta}$ = 19.44 kN/m

Circumferential force,T_2 =$wR[cos\theta - \frac{1}{1+cos\theta}]$ = 8.55 kN/m

Meridian Stress =0.19 N/mm^2 <6 N/mm^2

Hoop Stress =0.85 N/mm^2 <6 N/mm^2

Stresses are within safe limits, providing nominal reinforcement of 0.3% ,

A_{st} =300 mm^2

Provide6- 8mm dia. bars at 160mm c/c on both faces.

6.4.2 Design of Top ring beam:

Hoop tension,F_t = $\frac{T1\cos\Theta.D}{2}$ =81.65 kN

A_{st} = $\frac{Ft}{\sigma st}$ = 544 mm^2

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

If A_c is cross-sectional area of ring beam,$\frac{81.65*1000}{Ac+11*680}$ =1.3

A_c =55327.7 mm^2

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

Shear force along the edge =$T_1\sin\Theta$ =11.66kN

Shear stress along the edge =0.116 N/mm^2 – very low.

6.4.3 Design of cylindrical wall:

Maximum hoop tension at the base of wall, F_t =$\frac{whD}{2}$, where,w =unit wt. of water=10kN/m^3

h=height of water

F_t =341.25 kN/m

A_{st} =2275mm^2/m height.

Provide 8-20mm dia. bars @180mm c/c on each face.(A_{st} =2500mm^2)

A_{st} required at 1.75m below top =612.5 mm^2

Provide 10-10mm bars @180mm c/c on each face.

Let, t =thickness of side wall at bottom

$\frac{341.25*1000}{1000t+11*2275}$ =1.3

t =237.475 mm

Adopt 250mm thick wall at bottom gradually reducing to 200mm at top.

Distribution steel:
At bottom, A_{st} =0.2% of cross-sectional area =500mm^2
Provide 6-10mm dia. bars at 200mm c/c.

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

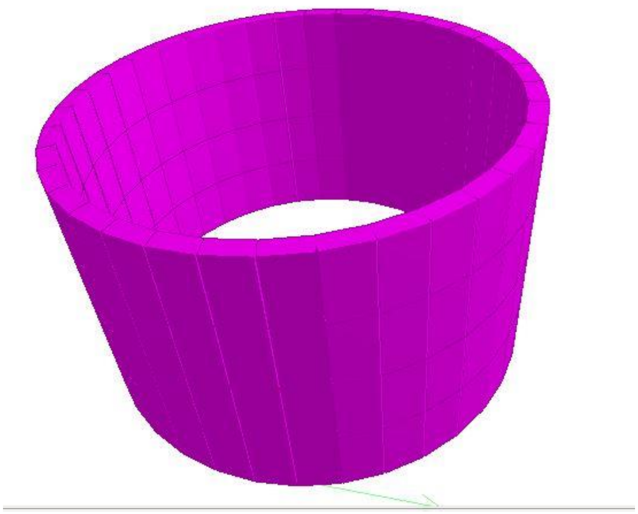

Fig. 6. 2- Cylindrical portion of an Intze tank

6.4.4 Design of bottom ring beam:

Load due to top dome = $T_1 \sin\Theta$ =11.66kN/m

Load due to top ring beam =0.3*0.2*24 =1.44kN/m

Self wt. of ring beam(assuming 1.2m*0.6m) =17.28kN/m

Load due to cylindrical wall =35.1kN/m

Total vertical load V=65.28kN/m

Hoop tension due to vertical loads,$H_v = \frac{VD}{2}$ =343.77 kN

Hoop tension due to water pressure,$H_w = \frac{whdD}{2}$ =204.75 kN

Total hoop tension = $H_v + H_w$ =548.52 kN
A_{st} =3656.8 mm^2

Provide 12 bars of 20 mm dia.(A_{st} =3770 mm^2)

Maximum tensile stress =$\frac{548.52*1000}{1200*600+11*3656}$ =0.75 N/mm^2 <1.3 N/mm^2

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

6.4.5 Design of conical dome:

Average dia = $\frac{10.5+7.5}{2} = 9$

Average depth of water = 6.5+$\frac{1.75}{2} = 7.375\ m$

Weight of water above conical dome

= π×9×7.375×1.75× 8.25 = 3010 KN

Assuming 600mm thick slab,

Self weight of slab = π×2.30×9×0.6×24 = 938.4 KN

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

= π×10.5×65.28

= 2153.37 KN

Total load at base of conical slab = 6102 KN

Load/ unit length, $V_2 = \frac{6102}{\pi\times6.5} = 300$ KN/m

Meridian thrust = T =$V_2 cosec\theta$= 300×cosec 45°=425 KN

Meridional stress =$\frac{425\times10^3}{600\times1000} = 0.708$ N/ mm^2

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

Hoop tension (H) =(pcosecθ + qcosecθ)D/2

Water pressure, p=10×6.5= 65 KN/ mm^2

Weight of conical dome slab / m^2,

q = 0.6×24=14.4 $KN/\ mm^2$

θ= 45°, D= 10.5 m

H= (65×cosec45° + 14.4×cot45°) 10.5/2
= 558.20 KN

$A_{st} = \frac{558.20\times10^3}{150}$ = 3721 mm^2

Provide 8-25mm φ bars @ 180mm c/c

$A_{st}(3926\ mm^2)$ On both faces of slab

Distribution steel:$\frac{0.2\times600\times1000}{100} = 1200\ mm^2$

Provide 10mm φ at 130mm c/c on both faces along the meridions.

Max. tensile stress = $\frac{558.20\times10^3}{(600\times1000)+(11\times3926)}$

= 0.86<1.3(safe)

6.4.6 <u>Design of bottom spherical dome:</u>

Thickness of dome slab (assume) = 300mm

Dia at base =7.5m

Central rise = 1/5 ×7.5= 1.5m

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

R= $\frac{(D/2)^2+r^2}{2r} = \frac{(7.5/2)^2+1.5^2}{3}$=5.4375m=5.44m

Self weight of dome slab

= 2π×5.44×1.5×0.3×24 = 370 KN

Volume of water above the dome

=π×3.25^2×(6.5+2)+π×$3.25^2/3$×(5.44-1.5)

2π($\frac{5.44\times1.5}{3}$)=308.6 m^3

Weight of water = 3080 KN

Total load on dome = 3080+370=3450 KN

Load/ unit area = $\frac{3450}{\pi \times 3.25^2}$

= 103.9 KN/m^2

Meridional thrust, T=$\frac{wR}{1+cos\theta}$

Cosθ= $\frac{3.94}{5.44}$ =0.724

θ = 44.5°

$T_1 = \frac{104 \times 5.44}{1.724}$ = 328.16 KN/m

Meridional stress=$\frac{328.16 \times 10^3}{300 \times 1000}$

=1.09(safe)

Circumference force = wR (cosθ- $\frac{1}{1+cos\theta}$)

= 104×5.44(0.724-1/0.724)

= 81.44 KN/m

Hoop stress=$\frac{81.44 \times 10^3}{300 \times 1000}$=0.27 N/$mm^2$ (safe)

Provide nominal reinforcement of 0.3%,

$A_{st} = \frac{0.3 \times 300 \times 1000}{100}$ = 900mm^2

Provide 8- 12mm φ bars @ 120 mm c/c circumferentially and along the meridions.

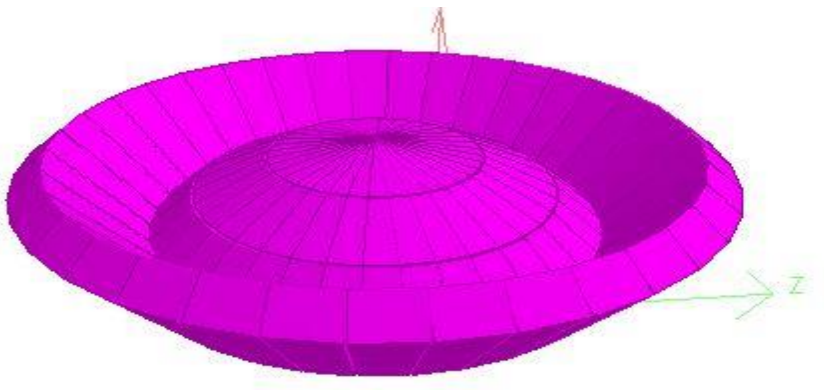

Fig. 6. 3-Bottom conical and spherical dome of an Intze tank

6.4.7 Design of bottom circular Girder

Thrust from conical dome, T_1= 425 KN/m

Thrust from spherical dome, T_2= 328 KN/m

Net horizontal force on ring beam

$T_1 cos\alpha - T_2 cos\beta$ =(425×0.707-328×0.713) = 66.71 KN

Hoop compression in beam = $\frac{66.61\times65}{2}$ = 216.5 KN

Assuming the size of girder as 600 mm wide and 1200 mm deep

hoop stress = $\frac{216.5\times10^3}{600\times1200}$ = 0.3 N/mm^2

Vertical load on ring beam = 425×0.707+328×0.70

= 530.07 KN/m

Self weight of beam = 0.6×1.2×24 = 17.28 KN/m

Total load = 530+17.28 = 547.3 KN/m

Total design load on ring girder

W= πDw = π×7.5×547.3

= 12895.4 KN

The circular girder is supported on 8 columns using the moment coefficient

Maximum –ve BM at support section = 0.083wR

= 0.0083×125895.4×3.75

$= 401.4$ kN-m

maximum +ve BM at mid span section

$=0.0041wR$

$= 0.0041 \times 12895.4 \times 3.75$

$= 198.27$ kN-m

Torsional moment $= 0.0006wR$

$= 0.0006 \times 12895.4 \times 3.75$

$=29.01$ kN-m

Shear force @ support section

$V=\frac{wR\times\frac{\pi}{4}}{2} \quad = \frac{547.3\times3.75\times\frac{\pi}{4}}{2\times4} =805.96$ kN

Shear force at section of maximum torsion

$805.96-\frac{547.3*3.14*3.75*9.5}{180} =465.66$ kN

<u>Design of support section</u>

M $=401.4$ kN-m

V$=805.96$ kN

$k_b =0.39$

$j_b =0.87$, $Q=1.38$

$d=\sqrt{\frac{401.4*10^6}{1.38*600}}=696.3$ mm

Effective depth =800 mm (taking cover of 50 mm)

$A_{st} =\frac{401.4*10^6}{150*1.38*800} =2423.9 \text{ mm}^2$

Providing 8-20mm dia bars ($A_{st} =2513\text{mm}^2$)

$\tau_v = \frac{805.96*10^6}{600*800} = 1.67 \text{ N/mm}^2$

$\frac{100Ast}{bd} =\frac{100*2513}{600*800} =0.523$

$\tau_c =0.31\text{N/mm}^2$

$\tau_c < \tau_v$, hence shear reinforcement required

Shear taken by concrete $=\frac{0.31*600*800}{1000}$ =148.8 kN

Balance shear =805.9-148.8=657 kN

Using 12mm dia.,4-legged stirrups,

spacing, $s_v=\frac{150*4*113*800}{657*10^3}$ =82.55 mm

Adopt 12mm dia.,4-legged stirrups at 80mm c/c near supports.

Design of mid-span section:

Maximum +ve moment =198.27 kN-m

$A_{st}=\frac{198.27*10^6}{150*0.9*750}$ =1836mm^2

minimum area of steel in section = $\frac{0.24*600*800}{100}$ =1152mm^2

Provide 6 bars of 20mm dia at mid-span (A_{st} =1884mm^2)

Adopt 10mm dia 4-legged stirrups at 250mm c/c.

Design of section subjected to maximum torsion:

T=29 kN-m D=800mm

V=465.66kN b=600mm, d=750mm

M=0kN-m

$M_t=T\left[\frac{1+\frac{D}{b}}{1.7}\right]$ = 40kN-m

Equivalent moment,M_e =M+ M_t =40kN-m

$A_{st}=\frac{40*10^6}{150*0.9*750}$ =395 mm^2

But minimum area of steel =1152mm^2

Provide 4 bars of 20mm dia.(1256mm^2)

Equivalent shear $=V_e=V+1.6\frac{T}{d}$ $=465.66+1.6(\frac{29}{0.75})$ =528kN

$T_{ve}=\frac{Ve}{bd}=\frac{528*10^3}{600*750}$ =1.17N/mm^2

$\frac{100Ast}{bd}=\frac{100*1256}{600*750}$ =0.279 N/mm^2 $=\tau_c$

$T_{ve}>\tau_c$,shear reinforcement is required.

Using 10mm dia.,4-legged stirrups with side covers of 25mm and top and bottom covers of 50mm,spacing

$s_v = \frac{4*78.5*150}{(1.17-0.28)*600}$ =88mm

Adopt 10mm dia.,4-legged stirrups at 80mm c/c.

6.4.8 <u>Design of columns of supporting tower</u>

8 columns equally spaced on 7.5 m diameter circle

Vertical load on each column = $\frac{12895.4}{8}$ = 1611 KN

Self weight of column of height 16m & diameter 650mm

$$= \frac{\pi}{4} \times 0.65^2 \times 16 \times 24 = 127 \text{ KN}$$

Self weight of bracings (3 No's of 4 m intervals, size 500mmx500mm)

$$=3 \times 0.5 \times 0.5 \times \frac{\pi}{8} \times 24 = 57\ kN$$

Total vertical load = 1795kN

Wind forces: Intensity = 1.7kN/m^2

Reduction coefficient = 0.7

- Wind forces on top dome & cylindrical wall

  = (6.5+1.75) x0.7x1.7x10.5

  = 103.08kN

- On conical dome: 1.7x0.7x9x2 = 21.42 kN
- On bottom ring beam: 1.7x0.7x1.2x8
  = 11.424 KN
- On 5 columns: 5x0.65x16x0.7x1.7
  = 61.88 KN
- On bracings: 1.7x0.5x3x7 = 17.85 KN

Total horizontal wind force = 215.65 KN

- Moment at the base of columns

  $$M = \frac{215.65 \times 4}{2} = 431 \text{ KN-m}$$

  (Assuming point of contra-flexure at mid of column)
- Moment @ base of columns due to wind loads
  = 26x103 + 21.42x17.5 + 17.75+ 12.5x12 + 12.5(8+4)
  = 3555.63 KN-m

$3555.63 = 431 + \frac{V}{3.75}(2 \times 3.75^2 + 3.75(\frac{3.75}{\sqrt{2}})^2)$

V = 214.98 KN = 215 KN

Total load on leeward column at base = 1795+215 = 2010 KN

Moment in each column at base = $\frac{431}{8}$= 53.875 KN-m

- Reinforcement in column:

  Axial load, P=2010 KN

  Moment, M = 53.875 KN-m

  Eccentricity, e = M/P= 26.8mm

Eccentricity is small, hence direct stress is predominant

8 bars- 32mmφ & lateral ties of 10mmφ at 300 mm c/c

A_{sc}=6432 mm^2

$A_c = \frac{\pi \times 650^2}{4}$ + 1.5x13x6432

= 0.45x 10^6 mm^2

$I_c = \frac{\pi \times 3254}{4}$ + 1.5x13(2x804x275^2+4x804$(275/\sqrt{2})^2$)

=13.48 x 10^9 mm^4

Direct compressive stress = $\sigma_{cc} = \frac{2010}{0.45 \times 10^6}$= 4.467 N/$mm^2$

Bending stress = $\sigma_{cb} = \frac{53.875 \times 10^6 \times 325}{13.48 \times 10^9}$ = 1.2989 N/mm^2

Permissible stresses in concrete are increased by 33.33%

$$\frac{\acute{\sigma}_{cc}}{\sigma_{cc}} + \frac{\acute{\sigma}_{cb}}{\sigma_{cb}} < 1$$

$$\frac{4.467}{6} + \frac{1.2989}{8.5} = 0.89 < 1$$

Design of bracing

Moment = 2x53.875x$\sqrt{2}$

= 152.38 KN-m

Section of brace = 500x500

B=500mm, d= 450mm

$M_1 = 1.38 \times 500 \times 450^2$ = 139.725 KN-m

Balance moment = 12.3 KN-m

$$A_{s_{t1}} = \frac{139.725 \times 10^6}{230 \times j \times 450} = 1575\ mm^2$$

$$A_{s_{t2}} = \frac{12.3 \times 10^6}{230 \times j \times 400} = 156\ mm^2$$

$$A_{s_t} = A_{s_{t1}} + A_{s_{t2}}$$
$$= 1731 \ mm^2$$

Provide 4 bars- 25 mm φ @top and bottom

Length of brace = 2x3.75xsin22.5

= 2.87 m

- Maximum shear force in brace

$$\frac{moment\ in\ brace}{0.5 \times length\ of\ brace} = 106.2 \text{ KN}$$

$$\tau_v = \frac{106.2 \times 10^3}{500 \times 450} = 0.472 \text{ N}/mm^2$$

$$\frac{100A_{s_t}}{bd} = 0.769, \quad \tau_c = 0.38 \text{ N}/mm^2$$

$\tau_v > \tau_c$, hence shear reinforcement is required

Shear carried by concrete = $\frac{0.38 \times 500 \times 450}{1000}$=85.5 KN

Balance shear = 20.7 KN

Using 10mm φ – 2 legged stirrups,

Spacing, $s_v = \frac{230 \times 2 \times 79 \times 450}{20.6 \times 10^6}$

= 793mm

0.75d = 337.5, therefore s_v= 337.5 mm

Adopt 10mm φ – 2 legged stirrups @ 330 mm/cc

Fig. 6. 4-Columns and bracings for an Intze tank

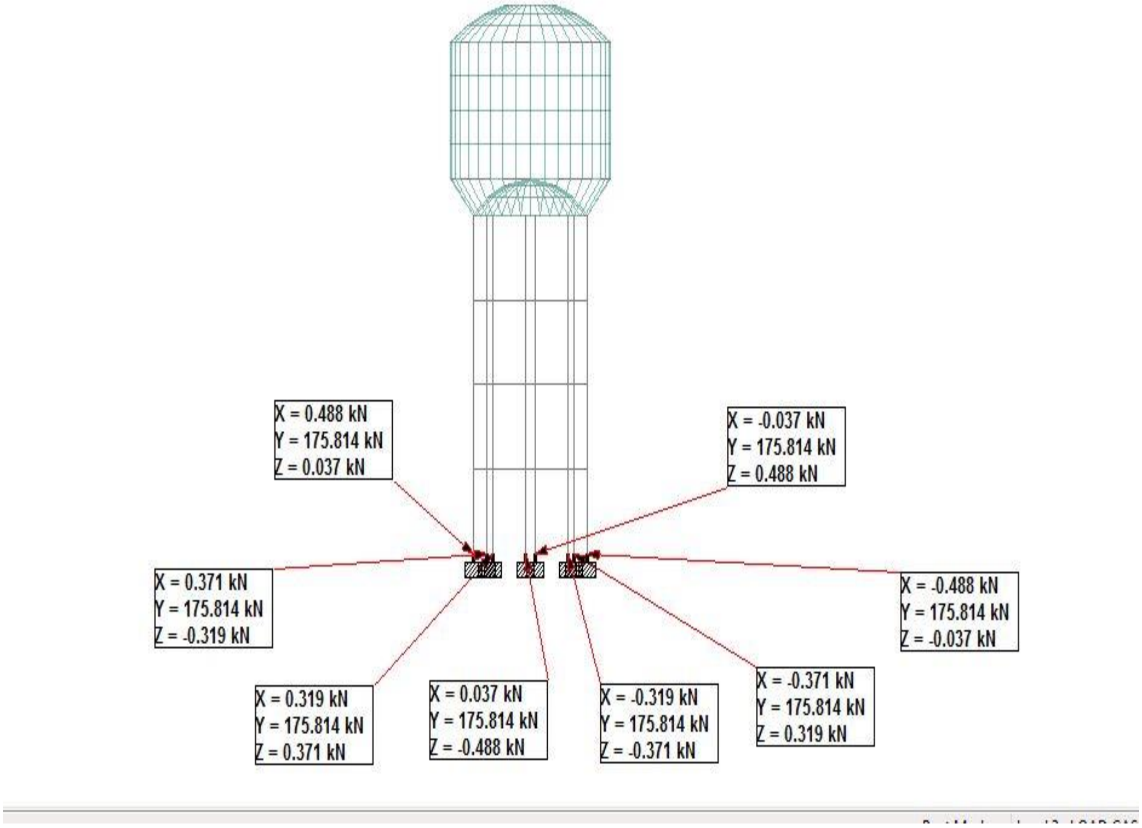

Fig. 6. 5-Reaction forces on the designed Intze tank

Table6. 1-Node displacement summary

All | Summary

			Horizontal	Vertical	Horizontal	Resultant	Rotational		
	Node	L/C	X mm	Y mm	Z mm	mm	rX rad	rY rad	rZ rad
Max X	307	14 GENERAT	**535.92112E**	-36.90044E6	-40.00180E6	538.67731E	-4881.937	-8037.286	-63758.898
Min X	307	13 GENERAT	**-535.94556E**	-40.21009E6	40.04894E6	538.94194E	4887.689	8033.233	63762.688
Max Y	205	15 GENERAT	-414.79897E	**305.82991E**	-10.41899E6	515.45966E	4886.239	8034.253	63761.949
Min Y	11	13 GENERAT	-346.65750E	**-382.95494E**	42.17400E6	385.27038E	4887.652	8033.154	63762.840
Max Z	2	13 GENERAT	-414.80519E	-382.95491E	**73.94373E6**	569.37288E	4887.649	8033.149	63762.836
Min Z	2	14 GENERAT	414.78056E	305.81506E	**-73.92761E6**	520.60644E	-4881.896	-8037.202	-63759.047
Max rX	195	13 GENERAT	-457.28728E	208.33589E	2.70639E6	502.51650E	**4888.402**	8033.074	63763.090
Min rX	195	14 GENERAT	457.25684E	-285.44450E	-2.64811E6	539.04488E	**-4882.649**	-8037.126	-63759.297
Max rY	240	15 GENERAT	-511.12459E	139.78798E	17.88436E6	530.19700E	4886.340	**8034.667**	63762.379
Min rY	240	12 GENERAT	511.10087E	-216.88989E	-17.82907E6	555.50269E	-4880.587	**-8038.720**	-63758.586
Max rZ	218	13 GENERAT	-503.46016E	138.00508E	15.10643E6	522.25066E	4888.325	8033.014	**63763.375**
Min rZ	218	14 GENERAT	503.43256E	-215.11117E	-15.04972E6	547.67112E	-4882.572	-8037.066	**-63759.582**
Max Rs	216	12 GENERAT	422.10225E	-382.29131E	9.83621E6	**569.57325E**	-4880.413	-8038.087	-63757.910

Table6. 2-Beam forces summary

All | Summary | Envelope

	Beam	L/C	Node	Fx kN	Fy kN	Fz kN	Mx kNm	My kNm	Mz kNm
Max Fx	981	13 GENERAT	191	**3.46332E6**	3.90830E6	3.06956E6	-149.50870E	-350.18187E	316.92916E
Min Fx	981	14 GENERAT	191	**-3.46161E6**	-3.90820E6	-3.06945E6	149.50409E	350.17803E	-316.81291E
Max Fy	990	14 GENERAT	224	-1.63739E6	**4.94605E6**	3.68202E6	-185.08462E	-1.38698E6	1.79239E6
Min Fy	990	13 GENERAT	224	1.63895E6	**-4.94647E6**	-3.68233E6	185.10017E	1.38709E6	-1.79240E6
Max Fz	990	14 GENERAT	224	-1.63739E6	4.94605E6	**3.68202E6**	-185.08462E	-1.38698E6	1.79239E6
Min Fz	990	13 GENERAT	224	1.63895E6	-4.94647E6	**-3.68233E6**	185.10017E	1.38709E6	-1.79240E6
Max Mx	984	13 GENERAT	202	-856.08800E	-4.35945E6	-3.42504E6	**192.60897E**	977.14381E	-1.26934E6
Min Mx	984	14 GENERAT	202	857.44163E	4.35923E6	3.42488E6	**-192.60041E**	-977.10800E	1.26943E6
Max My	981	13 GENERAT	202	3.46332E6	3.90830E6	3.06956E6	-149.50870E	**1.65630E6**	-2.23782E6
Min My	981	14 GENERAT	202	-3.46161E6	-3.90820E6	-3.06945E6	149.50409E	**-1.65623E6**	2.23786E6
Max Mz	981	12 GENERAT	202	-3.46118E6	-3.90817E6	-3.06942E6	149.50295E	-1.65621E6	**2.23788E6**
Min Mz	981	15 GENERAT	202	3.46289E6	3.90828E6	3.06953E6	-149.50756E	1.65629E6	**-2.23783E6**

RESULTS

Design of circular water tank:

Height = 4.5 m; diameter = 14 m; thickness of wall = 180 mm

Tension steel

Depth	Coefficient	T	Ast req	Ast	Dia	Spacing	Ast prov	Fct
0								
0.45	0.104	32.76	218.4	399.6	8	250	402	**Safe**
0.9	0.218	68.67	457.8	457.8	8	210	479	**Safe**
1.35	0.335	105.525	703.5	703.5	8	140	718	**Safe**
1.8	0.443	139.545	930.3	930.3	10	160	982	**Safe**
2.25	0.534	168.21	1121.4	1121.4	12	200	1131	**Safe**
2.7	0.575	181.125	1207.5	1207.5	12	180	1257	**Safe**
3.15	0.53	166.95	1113	1113	12	200	1131	**Safe**
3.6	0.381	120.015	800.1	800.1	10	190	827	**Safe**
4.05	0.151	47.565	317.1	399.6	8	250	402	**Safe**
4.5								

Bending steel

Depth	Coefficient	M	Abs M	Ast req	Ast	Dia	Spacing	Ast prov
0								
0.45	0	0	0					
0.9	0.0001	0.091125	0.091125	3.5	399.6	10	190	413
1.35	0.0002	0.18225	0.18225	7.1	399.6	10	190	413
1.8	0.0008	0.729	0.729	28.3	399.6	10	190	413
2.25	0.0016	1.458	1.458	56.7	399.6	10	190	413
2.7	0.0028	2.5515	2.5515	99.2	399.6	10	190	413

3.15	0.0038	3.46275	3.46275	134.6	399.6	10	190	413
3.6	0.0029	2.642625	2.642625	102.7	399.6	10	190	413
4.05	-0.0022	-2.00475	2.00475	77.9	399.6	10	190	413
4.5	-0.0146	-13.3043	13.30425	517.3	517.3	10	150	524

Design of support structure

Height of column = 10.5 m; size of column = 500x300 mm

Ast = 5250mm^2; provide 12-25mmφ bars,Bracing: Ast = 2692mm^2, provide 6 bars 25mm φ

Design of intze water tank:

Dia.,D =10.5m

Height of cylindrical portion = 6.5m,

Thickness of cylindrical portion =250 mm.

Depth of conical dome = 1.75m

Spacing of bracings = 4m

Dia. of supporting tower = 7.5m

300mm*200mm size top ring beam, with 6 bars of 12mm dia. as main reinforcement and 6 mm dia. stirrups at 150mm c/c.

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

Size of bottom circular girder = 600 mm wide and 1200 mm deep

	Top spherical dome	**Conical dome**	**Bottom spherical dome**
Slab thickness	100 mm	600 mm	300 mm
Central rise	1.75 m	-	1.5 m
Radius	8.75 m	4.5 m	5.44 m
A_{st}	300 mm^2;6-8mmφ bars @ 160 mm c/c	3721mm^2;8-25mmφ bars@180mm c/c; Dist.steel-1200 mm^2 .	900 mm^2;8-12 mm φ bars.

CONCLUSIONS

- While designing rectangular tank for the capacity of 690 cu.m, thickness of the rectangular wall comes out to be 650mm which quite high.Water tanks having walls of such thickness are uneconomical and impractical to design or build.
- Usually,rectangular water tanks are used for design capacity of upto 200 cu.m .Rectangular water tanks are economical for smaller capacities.
- However,cylindrical or intz tanks might be more economical and a viable choice in case of larger capacities.
- We have carried out design of cylindrical water tank and its wall thickness comes out to be 180mm.. For circular tank, design of cylindrical is performed using MS-Excel and the thickness of wall is obtained. The tank along with the support structure was safe and displacement obtained were within permissible limits.
- For Intze tank, its various components like top and bottom spherical domes, conical dome ,bottom circular girder, circular ring beams were designed using the hoop stress principle and membrane shell theory and their dimension were obtained. The structure was modelled in Staad Pro, and satisfactory results with notional deviation was obtained.

SCOPE OF OUR PROJECT

1. We have designed a cylindrical and an Intze tank for a particular capacity and performed its static analysis on Staad Pro. In future, dynamic analysis for the same can be performed and the behaviour of the liquid storage structure can be evaluated under dynamic loads.
2. The detailed quantity and cost estimation of the elevated water tanks for a given capacity can be performed, and thus the most economical one can be decided.
3. Recent earthquakes in Nepal and Northern parts of India, is a wake up call for the structural engineers of the world. During an earthquake, water tanks, being of utmost public importance, should not fail. Hence, seismic analysis of elvated water tanks can be performed in order to decide the most appropriate design in earthquake prone areas.
4. The height of the supporting towers, we have designed is 16m. In future, the behaviour of the elevated water tanks of same capacity but varying supporting tower heights, under seismic loads can be studied.

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