NONLINEAR STATIC ANALYSIS OF ELEVATED RCC WATER TANK

By

Vyas Prutha Vipulkumar

13MCLC25

DEPARTMENT OF CIVIL ENGINEERING

INSTITUTE OF TECHNOLOGY

NIRMA UNIVERSITY

AHMEDABAD-382481

May-2015

NONLINEAR STATIC ANALYSIS OF ELEVATED RCC WATER TANK

Major Project

Submitted in partial fulfillment of the requirements for the Degree OF

MASTER OF TECHNOLOGY

IN

CIVIL ENGINEERING

(Computer Aided Structural Analysis And Design)

By

Vyas Prutha Vipulkumar

13MCLC25

Guided By

Prof. Jahanvi Suthar

DEPARTMENT OF CIVIL ENGINEERING

INSTITUTE OF TECHNOLOGY

NIRMA UNIVERSITY

AHMEDABAD-382481

Declaration

This is to certify that

i) The thesis comprises my original work towards the Degree of Master of Technology in Civil Engineering (Computer Aided Structural Analysis and Design) at Nirma University and has not been submitted elsewhere for a degree.

ii) Due acknowledgement has been made in the text to all other material used.

Vyas Prutha Vipulkumar

Certificate

This is to certify that the Major Project entitled **"Nonlinear Static Analysis Of Elevated RCC Water Tank"** submitted by **Ms. Vyas Prutha Vipulkumar (Roll No: 13MCLC25)** towards the partial fulfillment of the requirement for the Degree of Master of Technology in Civil Engineering (Computer Aided Structural Analysis And Design) of Nirma University, Ahmedabad is the record of work carried out by him under our guidance and supervision. In our opinion, the work submitted has reached a level required for being accepted for examination. The results embodied in this major project work to the best of our knowledge have not been submitted to any other University or Institution for award of any degree or diploma.

Prof. J. M. Suthar
Guide,
Assistant Professor,
Dept. of Civil Engg.,
Institute of Technology,
Nirma University,
Ahmedabad.

Dr. P. V. Patel
Professor,
Head of Department,
Dept. of Civil Engg.,
Institute of Technology,
Nirma University,
Ahmedabad.

Dr. K. Kotecha
Director
Institute of Technology
Nirma University
Ahmedabad

____
Examiner

____
Date of Examination

Abstract

Elevated water tank is storage system of huge mass of water at certain height. Because of large mass, especially when tank is full lateral forces are more or less govern by tank. So due to earthquake damage or collapse of elevated tank is at risk. Elevated tank should be safe and functional during and after earthquake for providing necessary water supply for drinking and for firefighting. However some repairable damage may be acceptable during sever earthquake not affecting the functionality of tank. Whatever may be the cause of distress but water tanks should fulfil the purpose for which it has been designed and constructed with minimum maintenance throughout its intended life. It is necessary to understand the seismic behaviour of elevated tank. As codal based design of structure is limited up to elastic behaviour of structure. Nonlinear static analysis is useful to understand post elastic behaviour of structure. Achievement of good performance and less damage with less loss of lives, it is necessary to understand nonlinear behaviour of structure. Pushover analysis is a method to perform nonlinear static analysis. In this study, elevated tank is design with limit state method as per IS:3370 (part 1 and 4) -2009. Water tank container should free from cracks so it can avoid leakage problem. Water generates hydrostatic pressure on tank walls. Due to seismic forces hydrodynamic forces also exerted on the walls in addition to hydrostatic forces. These hydrodynamic forces are evaluated with the help of spring mass model of tanks as per IS: 1893:2002 (part-2). Finite element model is generated in SAP2000 as per design of tank. Pushover analysis is performed to obtain pushover curve i.e. base shear v/s roof displacement. Concluded the capacity to resist earthquake by tank.

Acknowledgement

It gives me great pleasure in expressing thanks and deep sense of gratitude to **Prof. J. M. Suthar**, Assistant Professor. I like to give my special thanks to **Dr. P. V. Patel**, Head, Department of Civil Engineering, Institute of Technology, Nirma University, Ahmedabad for his continual kind words of encouragement and motivation throughout the Major Project. I am also thankful to **Dr. K. Kotecha**, Director,Institute of Technology, Nirma University for his kind support in all respect during my study.

I would like to thank The Almighty, my family and all my friends, for supporting and encouraging me in all possible ways throughout the dissertation work.

Vyas Prutha Vipulkumar

13MCLC25

Abbreviations, Notations and Nomenclature

PBD . . . . Performance Based Design

SDOF . . . . Single-Degree of Freedom System

MDOF . . . . Multi-Degree of Freedom System

R.C.C. . . . . Reinforced Cement Concrete

FSL . . . . Full Supply Level

UDL . . . . Uniformly Distibuted Load

A_h . . . . Design Horizontal Seismic Co-efficient

V_B . . . . Base shear

Z . . . . Zone factor

I . . . . Importance Factor

R . . . . Response Reduction factor

T . . . . Time period

m_i . . . . Impulsive mass

m_c . . . . Convective mass

K_c . . . . Spring stiffness of convecctive mode

K_s . . . . Lateral stiffness of elevated tank

h_i . . . . Height of Impulsive mass

h_c . . . . Height of convective mass

h_s . . . Structural height of staging,measured from top of foundation to the bottom of container wall

Contents

List of Tables

List of Figures

Chapter 1

Introduction

1.1 General

Earthquake is the most destructive force that sudden release stored energy in a form of waves. Momentary shaking ground or vibrations of ground caused by slip or volcano or magnitude acting or other sudden stress change in the earth are called an earthquake. An earthquake causes damage of structures. As the unpredictable nature of earthquakes, the design and construction of structures to withstand the effect of earthquake is the only cause of action of ground. The actual earthquake force in considering higher than what the structure are designed for, engineers can not design the structure for the actual value earthquake intensity because the cost of construction shall be high. The essential rule of earthquake resistance design of structure is that the structure should not collapse but damage to the structural components is permitted. The seismic design should be such that it prevents loss of life, ensure continuity, and minimum damage to property. Since a structure is permitted to damage in case of sever shaking, the structure should be designed for seismic shaking if the structure were to remain linearly elastic.

Water is basic need for human life. Water tank is used to store water. It is designed as efficient and economical unit for commercial as well as residential use. Seismic safety of liquid tanks is important. Water tanks should remain functional during and after earthquake to ensure efficient supply of water to earthquake affected regions and also for fire fighting purpose. Industrial liquid containing tanks may contain highly toxic and inflammable liquids and these tanks should not damage or leakage during earthquake.

Tanks are usually of three kinds: Tanks resting on ground, Elevated tank and Underground tank. Tanks resting on ground either circular or rectangular in shape. Elevated water tank is a water storage facility supported by tower. The height of tower of elevated water tank provides the pressure for the water supply system. These structures are depend on the hydrostatic pressure produced by elevation of water and hence supply of water can be done by gravity. This feature of elevated tank, become advantage in case of power outage after earthquake in which pumping system may not able to work without electricity. Elevated water tanks are lifeline structure, expected to remain functional after severe earthquake as a provider of drinking and for other utilities of water as well as firefighting operations.

Water storage tanks are designed as per IS-3370-2009(part- I to IV) [16]. Previous version of this code is IS-3370-1965(part- I to IV) is used to designed water tank with working stress method on the philosophy of no cracking. But as per IS-3370-2009(part-II), of use of limit state method has been permitted with the philosophy of cracking.

1.2 Background

Performance of structure in nonlinear static analysis known as pushover analysis. To know the nonlinear seismic response of elevated tank, pushover analysis is required. Pushover analysis is simplified static nonlinear procedure in which a predefined pattern of earthquake loads is applied incrementally to the structure until a plastic collapse mechanism is reached. A elevated tanks are critical structure during earthquake ground motion that is expected to remain functional after earthquake to serve water network system. It is necessary to study the performance of elevated water tank during earthquake. Elevated water tanks have poor seismic performance during many sever earthquake in the past. Seismic performance is described by designating maximum allowable damage state performance level for an earthquake ground motion.The target performance are two levels non-structural and structural damages,the combination of the two gives the building a combined performance level. Increasingly order of structural displacement the various performance levels that is operational, immediate occupancy, life safety and collapse prevention.

Performance-Base Design (PBD)[5] is modern approach to earthquake resistant design. Performance based engineering is not new. Automobiles, Airplanes, and turbines have been designed and manufactured using this approach for many decade. Nowadays pushover analysis is widely used for performance based design of new structures and retrofitting of existing structures.

1.3 Objective of Work

Performance based design is essential for structure to understand its behaviour and response during earthquake.

1) To design and detail of the Elevated Water Tank using IS:3370-(part 1 and 2)2009 IS-1893 Draft code [14].

2) To study behaviour and seismic response of Elevated Water Tank with nonlinear static analysis.

3) To understand plastic hinge formation in Elevated Water Tank.

1.4 Scope of the Work

1) Design of Elevated water tank.

2) Detailing of Elveated water tank

3) Modelling of tank in SAP2000.

4) Carryout static nonlinear analysis (pushover analysis) for elevated tank.

5) Generate pushover curve (base shear-roof displacement) for Tank.

6) Obtain demand curve.

7) Superposition of capacity curve and demand curve to obtain performance point for a specific level of earthquake.

8) Evaluation of elevated tank performance with reference to performance point.

1.5 Organization of Report

The content of report is divided into seven chapters as follows:

Chapter 2 Discusses the literature review. In this chapter literature regarding seismic behaviour of elevated tanks and performance based analysis basic, also included the fundaments of pushover analysis and information regarding hinge properties.

Chapter 3 Discusses about components of Intze tank type of staging. Also discuss analysis and design of these components and force distribution about staging.

Chapter 4 Presents the design of selected elevated water tank also check for tension crack width and design of foundation.

Chapter 5 Discuss the fundaments of nonlinear static analysis and procedure of pushover analysis.

Chapter 6 Presents the procedure of modelling of elevated tank and application of pushover analysis in SAP2000 and after pushover analysis result shows of elevated tanks.

Chapter7 Discuss summary and conclusion of this project. Also included future scope of work.

Chapter 2

Literature Review

2.1 Seismic Response of Elevated Water Tank

Literature survey is essential to review the work done in the area of performance based engineering.

M.Moslemi, M.R.Kianoush and W.Pogorzelski [1] In this article conical shaped elevated tank is taken and evaluate the performance of tank under seismic loading. Finite element technique is used to investigate the seismic response. Finite element method has advantage that impulsive and convective components separately. For simplicity conical portion of vessel is ignored. To create Finite element model analysis is used tank capacity = 75 m^3, Side shell thickness = 8.8mm, Cone thickness = 24.5mm, tank floor thickness = 330mm, shaft thickness =300mm.Tank is modeled using four node quadrilateral elements. Each node having six degree of freedom. Time history component of 1940 EL Centro ground motion PGA=0.32g. Consider system lumped mass and 2 DOF both resulting having 6 % difference. Reason of different lumped mass practice effect if tank wall flexibility on seismic response cannot be accounted appropriate and also only contribution of first sloshing and impulsive motion are included.

W Shenton and Hampton [2]This paper presents the analytical investigation of seismic response of isolated elevated water tank. Seismic isolation may prove to be a simple and cost effective rehabilitation for such structures. and isolation having benefit that most of retrofit work can be completed at the ground level. Isolated structure exhibited reduced hydrodynamic pressure on shell. In this paper three degree of freedom system model of tank is generated. Impulsive mass, convective mass are determined. The legs of tower are assumed to be bolted to the isolation bearings. The natural frequency and mode shapes are determine and response spectrum analysis is conducted. Comparison of fixed base and isolated tank response. For isolated base tank reduced tower drift, overturning moment and tank wall pressure for full range of tank capacity. Isolation is most effective for the smallest capacity tank. Also advantage is that reduction in base shear for isolated tank.

Sarokolyi, B.Navayineya et al. [3]In this paper, cylindrical concrete water tanks having a central shaft, have been evaluated with considering the effect of the structures interaction with water. Three methods i.e. Added mass method, The Eulerian-Lagrangian method and Lagrangian-lagrangian method are considered for interaction between fluid and structure. ANSYS software is used to analysis tank model. Displacement and hydrostatic pressure are compared by using of theoretical and finite element methods. Time period of tank is calculated for different condition of water i.e. empty tank, 1/3 full tank, 2/3 full tank and full tank. It is concluded that the base shear force reduced of structure in pseudo static analysis according to Iranian code/2800 for empty tank is four times and for tank with water is seven times as much as those from linear dynamic analysis that these difference are reduced from response reduction factor R. Base shear in pseudo dynamic analysis is much higher than that in static analysis.

S K Jain and Sameer.U.S [4] In this paper calculation of staging stiffness and time period with beam flexibility IS:11682-1985 [15]given the criteria for design of RCC staging of such structures. In practice beams are flexible. So, the consideration of column stiffness 12 EI / L^3 is not accurate. It will become overestimated for staging of stiffness. The seismic code all over the world practice is a performance factor, which is 2.5 to 4 times higher for elevated tanks than that for ductile buildings. Calculation for few tanks indicates that the single degree of freedom representation overestimates the lateral design forces, the difference in values depends on the geometrical properties of the tank and relative stiffness of staging. The convective pressure can be a dominant factor for certain proportions of tank and structure. Seismic effects on flexible tanks are substantially greater than those rigid tanks. The point of inflection is assumed to occurs at the mid span of beams and columns. The compatibility requires that lateral deflection is same in all columns of a panel. Conclude that the method for calculation the staging stiffness including beam flexibility and without having to resort to finite element type analysis has been presented. The method is based on the well known portal method which has been suitably developed to incorporate the beam flexibility and three dimensional behavior of staging.

R Ghateh, N R Kianoush, W Pogorzelski [6] In this paper the finite element method is used for nonlinear static pushover analysis of the prototypes of elevated water tanks. Total forty eight prototypes are selected and design as per code. For each prototype pushover curve is generated and response reduction factor is determined. The effect of various parameters such as fundamental period, height to diameter ratio, seismic design category and tank size on seismic response factors of elevated water tank is evaluated.Cracking propagation pattern in RC pedestal is also studied in this paper. Finite element method is used to investigate the nonlinear seismic response of RC pedestal in elevated water tank. ANSYS software is used for finite element modeling of water tank. For study of tank two groups are categorized. In first group 24 prototypes are generated with R=2, and other group of 24 prototypes are generated with R= 3. The result of the study shows that the tank size has a significant effect on seismic response factors elevated tanks. For pushover analysis of water tank first the gravity loads including weight of tank stored water , pedestal wall and other equipment is applied to FEM model. Next gradually increasing lateral load is applied to the model until the structure collapse. The results of pushover shows that taller tank has maximum base shear comparing to shorter tanks. Two types of cracking pattern were observed during pushover analysis. Tank with height to mean diameter ratio is 2 or above it having flexure shear cracking pattern which initiate at the opposite top and bottom corners of RC pedestal, and the tank having height to mean diameter ratio is less than 2 starts near the base parallel to the lateral load direction and gradually extends to the top of pedestal.

2.2 Performance Based Design General

X K Zou, C N Chan [7] Pushover analysis is a simplified, static nonlinear procedure in which load is applied incrementally to structure until a plastic collapse mechanism is reached. Performance based design using nonlinear pushover analysis which involves tedious and intensive computational effort. This paper presents pushover drift performance design of reinforced concrete buildings. Non linear static pushover procedure requires to determine three elements that is capacity, damage and performance. Capacity spectrum, representation of structures ability to resist the seismic demand. Demand spectrum curve representation of the earthquake ground motion and the intersection of the pushover capacity and demand spectrum curve defines the performance point. Lateral drift performance is a principal concern in the seismic design of structure. This paper presents an effective optimization technique for the inelastic drifts performance design of RC building frames under pushover loadings. Most of the plastic hinges are found to have smaller values of plastic rotations within the ranges of the IO and LS and only one hinge is found to be in the more critical ranges between LS and CP level. Axial moments hinges and moment hinges should be considered in the nonlinear of columns and beams can be effectively modeled. At optimum a uniform lateral drift or ductility demand over all stories of buildings with the minimum cost is achieved.

Mehmet, Hyri Baytan and O Zmani [19]Puhover analysis is carried out for either user defined nonlinear hinge properties or default. This paper presents the comparison between user define hinges and default hinges. Four and Seven story buildings are considered to represent low and medium rise buildings. The definite of user defined hinges properties require moment curvature analysis of each element.In pushover analysis the behavior of structure is characterized by a capacity curve that represents the relationship between base shear and displacement of roof. Plastic hinges formation starts with beam and ends at lower stories then propagates to upper stories and continuous with yielding of base columns. It is observed that the base shear capacity does not depend on whether the default or user defined hinges properties are used. Increase in the amount of transverse reinforced improves the displacement capacity. The improvement is more effective for smaller spacing. The observations shows that user defined hinge model is more better than default hinge model which shows nonlinear behavior of element.

ATC-40 [10] Describes the fundaments and procedure of performance based design of reinforced concrete structure. Pushover analysis is basic tool for performance based design. Results of pushover analysis is base shear vs roof displacement curve called capacity curve. The generation of capacity curve defines the capacity of the structure uniquely for an assumed force distribution and displacement pattern. Also describe the methods of obtaining performance point which is point where capacity curve and demand curve meets.

2.3 Summary

In this chapter, review of relevant literature is carried out. The review of literature includes the seismic behaviour of elevated tanks and necessity to understand behaviour of tank during sever shaking due to past earthquake experiences and also included the fundaments of performance based analysis and procedure of pushover analysis. Also understand the consideration of hinge properties.

Chapter 3

Analysis and Design of Elevated Water Tank

3.1 Introduction

Water is basic need of human for daily life. Sufficient distribution of water is necessary in all area as per basic lifestyle of people. Elevated water tank is large water storage container for holding water and supply it at certain height to pressurization the water distribution system. Liquid storage tanks are used extensively by municipalities and industries for storing water, inflammable liquids and other chemicals.

Elevated water tank contains huge mass of water at certain height of staging. It is most critical consideration for the failure of structure during earthquake. Seismic safety of liquid tanks is of considerable importance.Water storage tanks should remain functional also after earthquake to ensure the water supply to earthquake affected region and also need for fire fighting. As per Indian standards water storage tanks are designed with IS:3370-2009 (part 1 and 2) [16]. All tanks are designed as crack free structures for durability and to prevent leakage and concrete should be impervious to eliminate seepage.The reinforced concrete Intze tanks having circular walls are mainly subjected to direct tension in form of hoop force. This tension is carried primarily by the steel and concrete is considered only to provide cover to steel reinforcement. In such type of structures, the values of allowable stresses in steel and concrete are restricted so that strains in steel and concrete are not high, consequently crack widths are limited. This provision minimize the damage of corrosion of steel.

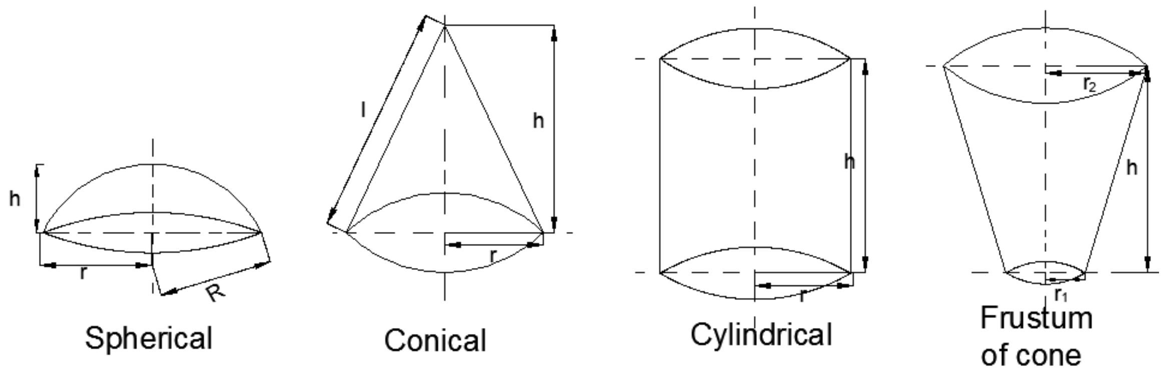

Figure 3.1: Shells of Revoluation of Tank

A wide variety of elevated water tanks can be seen in different shapes and sizes as shown in figure 3.1. The shapes of tank, its height above the ground, the type of supporting structure etc, are decided by functional, structural, aesthetic and economic considerations. A tank is generally an assembly of revolution of shells. To minimum use of materials, maximum structural advantage and economy. Different shapes of elevated tanks are i.e Rectangular, Circular, Conical and Intze types. Rectangular tanks having side walls and bottom slab with flexible and rigid connections. Circular tanks having top dome,cylindrical walls and bottom slab. This cylindrical walls supported on the bottom ring beam the walls of tank are assumed to be free at top as well as bottom. Tank wall is designed for hoop tension only, which is caused by horizontal water pressure,without any bending moment maximum hoop tension will occur at the base of tank wall. Floor slab is designed as two way slab supported on bottom ring beam. It should satisfy the crack width criteria of IS:3370-2009 (part 2).

3.2 Intze Tank

For large storage capacity overhead tanks, circular tanks are economical.Top and bottom slabs of such tanks becomes thick or requires beams to reduce their spans. Intze tanks arethe solution for such tanks where domes are provided of at slabs. Components of Intze tank are below:

a. Top spherical dome

b. Top ring beam

c. Circular side walls

d. Bottom ring beam

e. Conical dome

f. Bottom spherical dome

g. Circular ring beam

h. Staging

i. Foundation

For economical design, the following optimum dimensions may be considered.
In Figure 3.2

D = Diameter of cylindrical dome

R_1 = Spherical radius up to top dome

R_2 = Spherical radius up to bottom dome

h_1= Rise of top dome

h= Height of cylindrical tank

h_0= Height of conical dome

h_2= Rise of bottom spherical dome

D_0 = Diameter of bottom circular ring beam

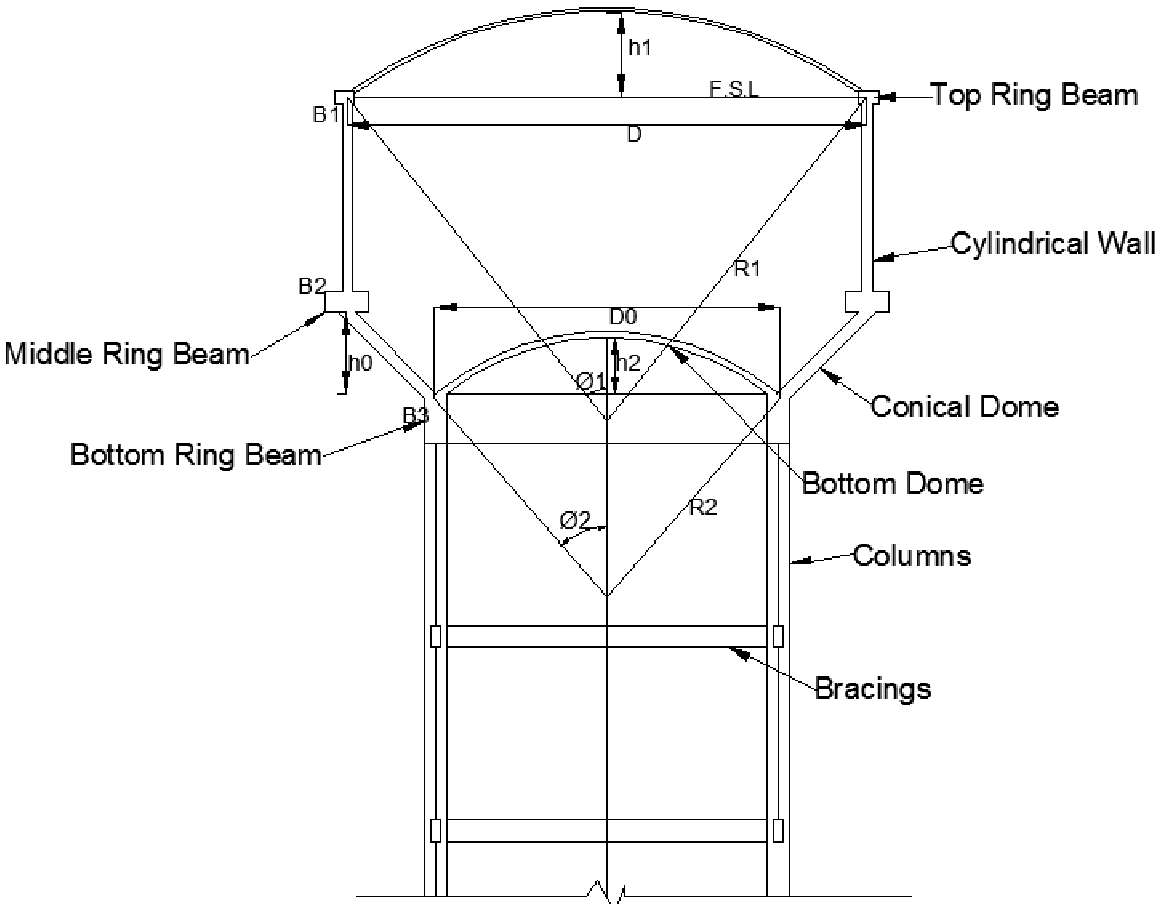

Figure 3.2: Components of Elevated Tank

3.2.1 Design Principles of Intze Tank Container

Top spherical dome :

The loads acting on dome consist of dead load, live load, snow load and wind loads. The domes are usually designed for vertical loads only, as stresses due to wind are very complex and difficult to determine. The live load is tanken from 1 to 1.5 kN/m^2 of the surface area. Two types of stresses are generated.

Meridional thrust and Hoop tension.

H_1 = Hoop tension due to concentrated load W

$$H_1 = \frac{W}{2\pi R sin^2\Theta} \tag{3.1}$$

H_2 = Hoop tension due to UDL (w)

$$H_2 = (cos\Theta - \frac{1}{1 + cos\Theta}) \tag{3.2}$$

Here,

W = concentrated load

w = UDL

R = Spherical radius of dome

The reinforcement on the dome are designed for maximum meridional thrust and circumfential forces as above.

Top ring beam (B_1)

The meridional thrust (T) of top dome at the level of top ring beam B_1 has two components viz. vertical component (T_1 $sin\Theta$) and horizontal component ($T_1 cos\Theta$) The beam is supported vertically through out by side circular wall. Thus, the vertical component which is the downward load (DL +LL) of the dome gets transferred through cylindrical walls. The horizontal component $T_1 cos\Theta$ includes hoop tension in beam B_1 for which the beam shall be designed.

Cylindrical walls

The side cylindrical walls assumed as free to move at top and bottom is subjected to hoop tension due to water load. Hoop tension is increased with depth. Thickness of walls is designed for maximum hoop tension at level of middle ring beam (B_2) and may be reduced with reduction of hoop tension.A shown in figure 3.4. In figure 3.4 'h' is the depth water up to the centre of the beam, 'd' is the depth of middle ring beam and 'γ_w' is density of water.

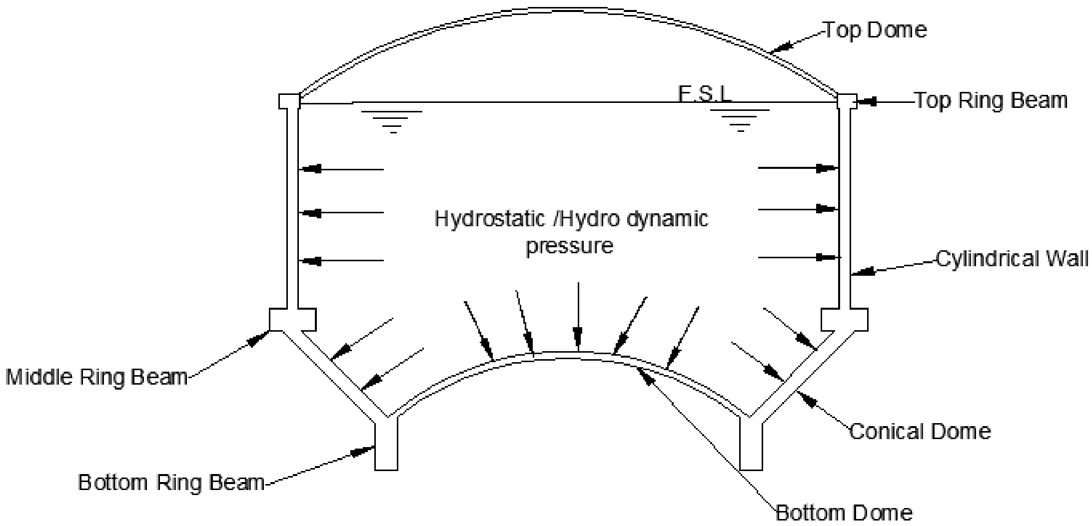

Figure 3.3: Components of Elevated Tank: Pressure Acting on Tank

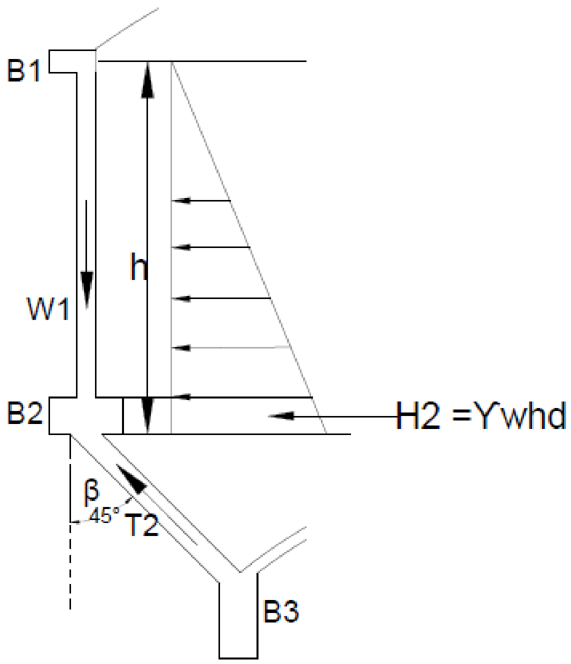

Figure 3.4: Loads at Cylindrical Wall

Middle ring beam (B_2)

The vertical load acting on ring beam B_2 consist of loads from upper elements i.e. top dome, top ring beam and cylindrical walls and also included self weight of middle ring beam. This load get transferred to conical dome by thrust T_2 in conical dome. As shown in figure 3.4. As the conical dome has an inclination of β with vertical T_2 can be found out by equilibrium conditions at joint B_2.

$\sum V = 0$

$W_1 = T_2 \cos\beta$

$$T_2 = W_1 \sec\beta \tag{3.3}$$

The vertical component of T_2 is balanced by vertical load W_1 however, its horizontal component which is equal to $T_2 \sin\beta$ which induce hoop tension in the beam and gets self-balanced. The hoop tension H_1 per metre length of beam.

$$H_1 = T_2 \sin\beta = W_1 \sec\beta \sin\beta \tag{3.4}$$

$$H_1 = W_1 \tan\beta \tag{3.5}$$

At the depth of beam the water pressure induces the hoop tension. Although this pressure is variable, assume to be in uniform with average value equal to water pressure at the centre of the beam.

Let, h = depth water up to the centre of the beam

d = depth of beam B_3.

Water pressure at this level = γ_w h

Horizontal force per metre length of beam H_2= γ_w hd

The beam B_2 is thus subjected to horizontal force, H per metre length where :

$$H = H_1 + H_2 \tag{3.6}$$

$$H = w_1 \tan\beta + \gamma_w hd \tag{3.7}$$

This horizontal force induces hoop tension in the beam for which it should be designed. To serve walk way around the tank, the width of beam usually kept more.

Conical dome:

The thrust T_2 transferred to the conical dome at level B_2-B_2 continue increases due to the water load on conical dome and becomes maximum at level B_3-B_3. The total vertical load acting on bottom ring beam B_3-B_3 is consist of loads of top dome top ring cylindrical wall middle ring beam and conical dome. This loads is calculated acting per metre length (w_2)

Meridional thrust T_3 in conical dome :

$$T_3 = \frac{W_2}{cos\beta} \tag{3.8}$$

In conical dome with meridional force hoop is also subjected. This hoop is subjected due to self weight and water load. The self weight of the dome acts in vertically downward direction where as the water pressure acts in the direction perpendicular to inclined surface.Hoop tension at different levels can be determined and design may be carried out. Also check the compressive stress in meridional direction and provide minimum reinforcement.

Bottom spherical dome

Bottom spherical dome is designed to support the loading of water above it in addition to its self weight. The dome is in turn supported by bottom circular beam B_3. It transferred meridional thrust T_4 to the circular beam B_3.

Bottom circular ring beam (B_3)

Bottom circular ring beam is designed as per beam curved in plan. The beam B_3 is supported on columns. It gives the supports the conical dome.

The beam is subjected to horizontal as well as vertical loads as follows :

Horizontal loads :-

Inward thrust due to conical dome = $T_3 = cos\alpha$

Outward thrust due to spherical dome =T_4 $cos\phi_2$

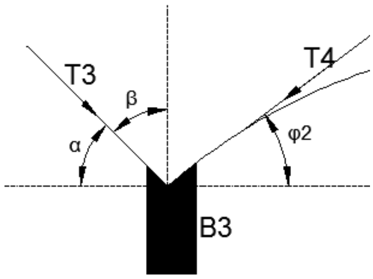

Figure 3.5: Loads on Bottom Ring Beam B_3

Net thrust on B_3= T_3 $cos\alpha$ - $T_4 cos\phi_2$ .... in compression

If this net thrust is negative. It will create hoop tension in beam B_3.

Dimensions of tank and angle of domes are so adjusted as to create hoop compression in beam B_3 and check the beam for hoop compression.

Vertical loads:-

Vertical loads consist of vertical components of thrusts T_3 and T_4 and self weight of the beam. The beam is designed for these loads as the beam is circular in plan.

3.2.2 Design Principles of Intze Tank Staging

Elevated water tanks consist of huge water mass at top of a slender staging. The consideration of staging is most critical for the failure of elevated tank during earthquake. Due to lack of knowledge of supporting system water tanks can be heavily damaged during earthquake. So there is need to focus on seismic safety of supporting system of structure which are safe during earthquake and also can take more design forces. Two types of staging are frame type and shaft types as shown in figure 3.6.

In shaft type staging, the tower may be in the form of single cylindrical shaft circular or polygonal in plan. Shafts are usually hollow and generally used for medium size elevated tanks.The space enclosed within the shaft may be used for providing pipes,stairs and electrical control panels.

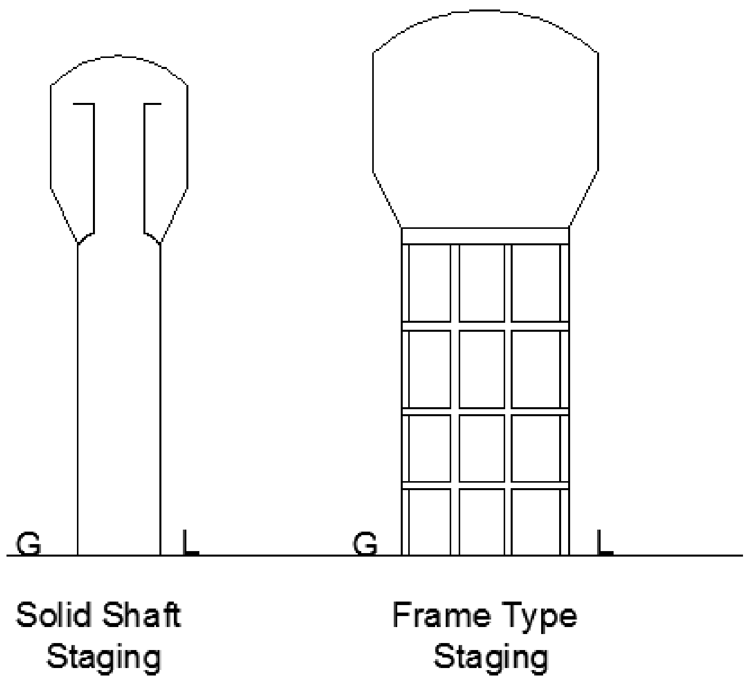

Figure 3.6: Types of Staging

In frame type staging, frame type staging are most commonly used staging in practice. Main components of frame type staging are columns and braces. Columns are arranged on periphery and it is connected internally by bracing at various levels. The staging behaves like a bridge between container and foundation for transfer of loads acting on the tanks. Frame type staging are more better than shaft type staging for lateral resistance because of their large redundancy and greater capacity to absorb seismic energy through inelastic actions. Framed staging have many flexural members in the form of braces and columns to resist lateral loads. Frame members and the brace column joints are to be designed and detailed for inelastic deformation, or else a collapse of staging may occur under seismic overloads.

3.3 Design of Staging

a. **Design of Columns**

(1) *Gravity Loads*

Gravity loads on column consist of dead loads and water load. Thus, loads on column are determined for tank empty and tank full conditions.

(2) *Wind Loads*

While calculating wind loads on circular container the shape factor of 0.7 for circular shape shall be used.It can be seen by figure that the wind loads produce tension, on windward columns, compression in leeward columns and no axial force in columns on the line of neutral axis.

(3) *Axial forces in columns*

Wind forces on windward side are calculated on container, on columns and on bracing. To determined the axial columns determine the sum of moments of all these forces about the neutral axis at the bottom of the columns.

Horizontal shear :

Calculate the total horizontal force at required level and divide it equally in n columns i.e.

$$H = \frac{\sum Hw}{n} \tag{3.9}$$

Wind moment in column:

Wind moment in the column is given by :

M = Horizontal shear x l/2

Where, l = c/c lenght of bracings.

$$M = \frac{Hl}{2} = \frac{\sum Hw}{n} \times \frac{l}{2} \tag{3.10}$$

b. **Design of Bracing**

The moment in column as determined above is about the axis, perpendicular to the direction of wind.As the wind direction can change. The maximum moment in given bracing will be induced for particular direction of wind.

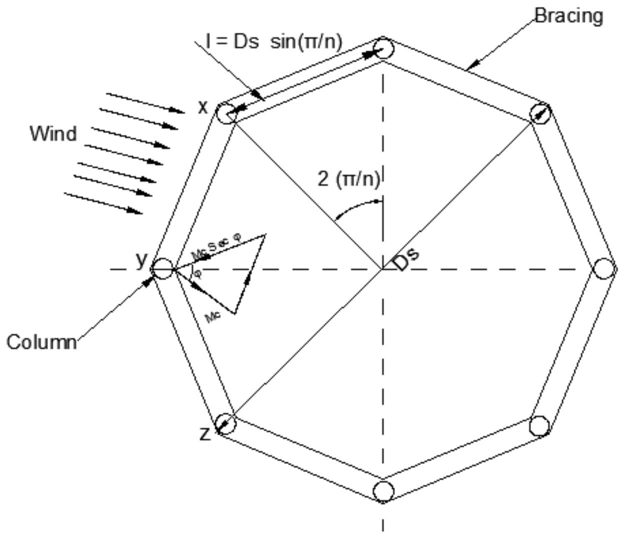

Figure 3.7: Section through staging

In figure 3.7 ,

D_s = Diameter of Staging

n =Number of column

For example as shown in figure 3.7, the maximum moment in yz bracing of is induced when the wind blows perpendicular to xy bracing. If M_c is the moment in column it can be seen from the moment triangle of that the moment M_b in equally spaced bracings (from upper and lower column) will be

$$M_b = 2 \times M_c \sec\theta \tag{3.11}$$

If effective length of bracing is l, the shear V_b in bracing assuming the point of contraflexure :

$$V_b = \frac{M_b}{\frac{l}{2}} = \frac{2M_b}{l} \tag{3.12}$$

The bracing shall be designed for moment M_b and shear V_b the nature of moment and shear can be reversed when wind blows from the opposite side.

c. **Foundation :**

Usually circular solid or annular raft foundation is used for Intze types of tanks.

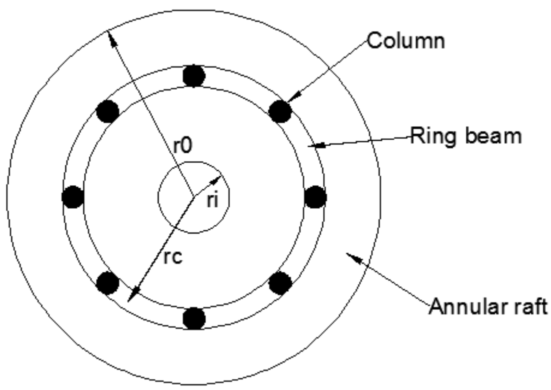

Figure 3.8: Plan of Foundation

3.4 Model Provisions

Water generates hydrostatic pressure on tank walls. Due to seismic forces hydrodynamic forces also exerted on the walls in addition to hydrostatic forces. These hydrodynamic forces are evaluated with the help of spring mass model of tanks.

Spring Mass Model for Seismic Analysis:-

As per Indian standards code IS: 1893-2002 (part II) [14] the dynamic analysis of liquid containing tank is evaluated by spring mass model of tank. When a tank containing liquid vibrates, the liquid exerts implsive and convective hydrodynamic pressure on tank wall. To analyse liquid containing tank including the effects of hydrodynamic pressure can be idealised by equivalent spring mass model as shown in figure 3.9 which also includes the effect of tank-wall interaction. The parameters of this model depends on geometry of the tank and its flexibility.

When a tank containing liquid with free surface is subjected to horizontal earthquake ground motion. Due to horizontal earthquake ground motion both tank wall and liquid are subjected to horizontal acceleration. The liquid in lower region of tank behave like rigidly connected to tank wall. The mass is termed as impulsive hydrodynamic pressure on the tank wall. Liquid in upper region of tank undergoes in sloshing motion. This mass is termed as convective liquid mass exerts convective hydrodynamic pressure on tank wall.

In spring mass model as shown in figure 3.9, the convection mass (m_c) is attached to the tank wall by spring having stiffness (k_c), and impulsive masses (m_i) is rigidly attached to tank wall.

Two mass idealization is preferred for elevated tank, which consists of two degree of freedom system. Spring mass model idealization is closer to unity. Most

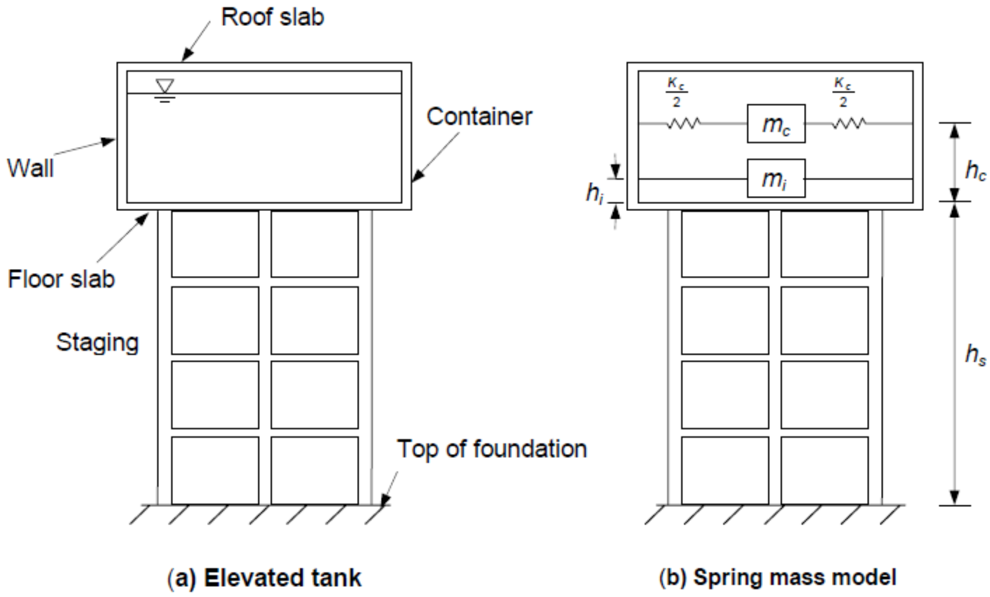

Figure 3.9: Spring Mass Model for Elevated Tank

of elevated tanks are never completely filled with liquid. Hence two mass idealization of tank is more preferred than one mass idealization as used in IS : 1893-1984 (part-I) [12].

For two mass idealization m_i, m_c, h_i, h_c, h_s, h_c^* These parameters are evaluated by charts as well as empirical formulae are given in IS 1893-2009(part -II). These parameters of this model are depends on geometry of the tank and its flexibility.

3.5 Summary

In this chapter, discuss about components of Intze tank and type of staging. Also included the analysis and design of container and staging of tank. Also discuss about evaluation of hydrodynamic pressure with two degree of freedom system.

Chapter 4

Design of RCC Elevated Water Tank

4.1 Input Parameters

Some Input parameters are below:

Capacity of tank= 6 lac liters

Height of Satging=18 meter

Concrete Grade= M 30

Steel Grade = Fe 415

As per IS-3370-2009(part-2) Permissible stresses are :

σ_{ct} =1.5 N/mm^2

σ_{cb} =2 N/mm^2

σ_{st} = 130 N/mm^2

σ_{cc} = 8 N/mm^2

σ_{cbc} =10 N/mm^2

Dimensions of Componenets:

Diameter of cylindrical portion (D) = 12 m

Diameter of bottom ring beam (D_0) =8 m

Rise of Top dome (h_1) = 2 m

Rise of bottom dome (h_2) = 1.5 m

Height of conical portion (h_0) = 2 m

Height of cylindrical wall (h) =5 m

Staging data:

No of column =8

No of brace ties =3

Size Diameter of columns = 500 mm

Size of Bracing Beam =250 × 500 mm

Dimensions of all components of elevated tank is shown in figurer 4.1

4.2 Design of Container

Container design is as per guildines of IS:3370 (part-2) 2009 [16].

4.2.1 Top Dome

Radius of top dome (r_1) = 6 m

Assume thickness of dome (t)= 100 mm

Spherical Radius for Top dome (R_1) = $h_1/2+r_1^2/2h_1$= 10 m

Geometrical Parameters :

Surface area of dome = $2\pi R_1 h_1$ = 125.66 m^2

Substended angle at springing for top dome (Θ_1)= $Sin^{-1}(r_1/R_1)$

= 0.636 radians

= 36.47 degree

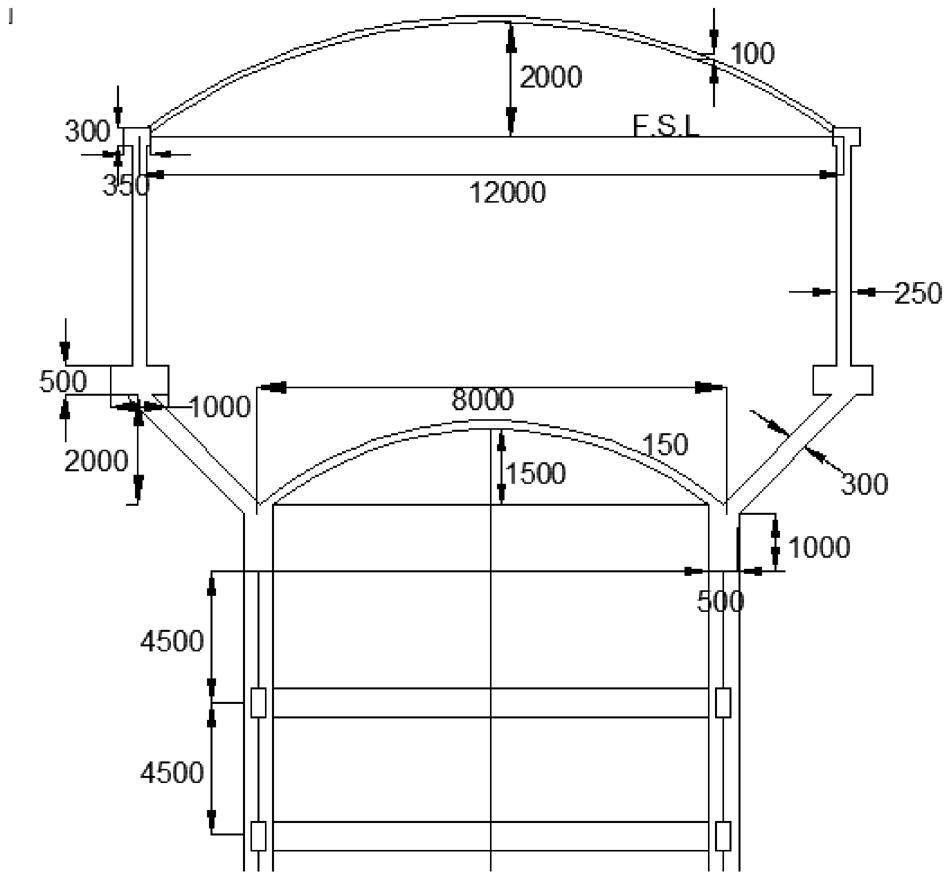

Figure 4.1: Size of Elevated Tank : All Dimensions are in mm

Loads:

1) Dead load: - i) Self weight =0.1 × 25 = 2.5 kN/m^2

ii) weight of inside Plaster =1.6 kN/m^2

2) Live load =1.5 kN/m^2

Total UDL =5.6 kN/m^2

Total load = Total UDL × surface area of dome

=703.66 kN

Meridional Stress:

Perimeter = $2\pi r_1$ = 2×π×6 = 37.8 m

Vertical Load per meter of perimeter (V)= 18.67 kN/m

Meridional Thrust (T) : V/sinΘ_1 = 31.11 kN/m

Area of concrete (A_c)= 100000 mm^2

Meridional Comressive Stress = 0.311mm^2 < 8mm^2.......OK

Hoop Stress:

H_1 = Hoop Tension due to concentrated Load using equation 3.1.

H_2 = Hoop Compression due to UDL using equation 3.2.

Table 4.1: Hoop Stress of Top Dome

Seg.	**Radius**	**SinΘ_1**	**cosΘ_1**	**Thickness**	H_1	H_2	**H**	**Stress**
	m			**mm**	**kN/m**	**kN/m**	**kN/m**	**N/mm^2**
					-ve	**+ve**		
1	1	0.1002	0.995	100	0	27.649	27.649	0.28
2	2	0.2013	0.980	100	0	26.583	26.583	0.27
3	3	0.3045	0.954	100	0	24.764	24.764	0.25
4	4	0.4108	0.917	100	0	22.127	22.127	0.22
5	5	0.5211	0.867	100	0	18.577	18.577	0.19
6	6	0.6367	0.804	100	0	13.988	13.988	0.14

Provide minimum Reinforcement 0.3 %

A_{st} = 300 mm^2.

Provided Spacing = 150 mm

Provide 8 mm ϕ @ 150 mm c/c in radial direction and circumferential direction. (A_{st}= 335 mm^2).

Table 4.2: Summary of Top Dome:

Segments	**Thickness**	P_t	**Provided bars**	A_{st}
	mm	**%**	**mm**	mm^2
1 to 6	100	0.3	8ϕ-150 c/c	353
Provide 10ϕ-150 mm (i.e. extra top reinforcement in radial,direction from r=6 m with 8ϕ-150 mm c/c hoop reinforcement as distribution bars.				

4.2.2 Top Ring Beam

Assume depth of ring beam = 300 mm

Reactions on substructure

1) Self weight of dome = $2\pi r_1 h_1$t =320 kN

2) Plaster weight = 100.53 kN

Total Weight (W) = 420 kN

Vertical reaction per unit perimeter (V) = W/$2\pi r_1$ = 11.15 kN/m

Thrust inclined at angle Θ_1 (T_1)= V/sinΘ_1 = 18.58 kN/m

Horizontal Componenet Thrust (H) = T_1cosΘ_1 = 25.01 kN/m

Hoop tension = H $\times$ r_1 = 150.09 kN.

A_{st} = 1154.54 mm^2.

Width of top ring beam (B) = 350 mm

Provide 6 no - 16 mm ϕ. (A_{st} = 1206.33 mm^2).

Shear reinforcement

Use 8 mm ϕ - 2 legged stirrups. (A_{sv} = 100mm^2)

S_v = 0.87 $\times$ f_y $\times$ A_{sv} /0.4b = 302.47 mm

Provided Spacing = 300 mm

Provide 8 mm ϕ 2-legged stirrups - @ 300 mm c/c

Size of Top Ring Beam = 350 x 300 mm.

Table 4.3: Summary of Top Ring Beam

M-25 Concrete	**Reinforcement**
Width = 350 mm	16 mm ϕ - 6 no –Hoop bars
Depth = 300 mm	8 mm ϕ- 2 legged stirrups @ 300 mm c/c

4.2.3 Cylindrical Wall

Assume thickness of wall = 250 mm

Height of cylindrical wall portion = 5 m

Hoop reinforcement

Height (d_h)= Height of segment

h= height of wall

Hoop Tension (H) = $\rho d_h r_1$h

Thickness of section (t) =(H/σ_{ct}-(m-1)A_c)/d_h

Provided thickness must be more than required thickness .

Table 4.4: Hoop Reinforcement in Cylindrical Walls

Seg	Segment level	Height	Hoop tension	Ast	Bar dia	No of bars	Provided bars	Ast Provided	Thickness check	Provided thickness	check
	mm	m	kN	mm^2	mm		no	mm^2	mm	mm	
1	0-300	0.3	2.7	20.77	8	0.413	4	201.062	-1.341	250	ok
2	300-1300	1	48	369.23	8	7.346	10	502.655	30.725	250	ok
3	1300-2300	1	143.964	1107.42	10	14.100	16	1178.097	96.216	250	ok
4	2300-3300	1	223.944	1722.65	10	21.933	22	1727.876	150.960	250	ok
5	3300-5000	1.7	523.4691	4026.69	12	35.604	36	4071.504	207.333	250	ok

Vertical Reinforcement

As per IS:3370-2009(part -2) cl 7.1.1 The minimum reinforcement in walls ,floors and roofs in each of two directions at right angles shall have an area of 0.3 % of the concrete section in that direction for section up to 100 mm thickness.

For section of thickness more than 100 mm and less than 450 mm the minimum reinforcement in each of two directions shall be linearly reduced from 0.3% for 100 thick and 0.2 % for 450 mm thick section.

For section having more than 450mm thickness,the minimum reinforcement in each of two directions shall be kept at 0.2% .

Here concrete wall section having 250mm thickness.

Provide minimum reinforcement as per thickness of section =p_t= 0.2571 %.

Provide 8mm ϕ bars for vertical reinforcement.

A_{st}= 0.257 x 1000 x 250 / 100

= 642.56 mm^2. for both face reinforement

= 321.45 mm^2. for one each face reinforement.

Spacing = a_{st} x 1000/A_{st}

= 50.26x 1000/321.45 = 156.32 mm

Provided Sapcing = 150 mm

provide 8 mm ϕ –@ 150 mm c/c on each face vertically outer side of main reinformcemt. (A_{st}=335.10 mm^2).

As per IS:3370-2009 (Part -2) Apendix -B,B-6 crackwidth in concrete due to direct tension should be checked.

For limiting design surface crack width should not be more than 0.2 mm

Crack width Calculation for Tension Face:

for each segment the calculation of crack width is required.

For segment 3 :

Hoop tension = 143.98 kN

Thickness (t) = 250 mm

Clear Cover (c) = 30 mm

Dimeter bars (ϕ) = 10 mm

Spaicng of bars (s)= 62.5 mm

Modulas of Elasticity (E)= 200000 N/mm^2

A_s = ($\pi\phi^2/4$)/ Spacing of bars = 1256.64 mm^2

Permissible strain in steel = (0.87 f_y / E) = 0.001805

Permissible Stress in concrete = 0.45 f_{ck} = 13.5 N/mm^2

Effective depth = Thickness - clear cover - $\phi/2$ = 215 mm

Stress in Steel (f_s = (Hoop tension / 2 A_s) = 57.281 N/mm^2

Strain in Steel (ϵ_1) = (f_s/E) =0.00043

Tension stiffening effect (ϵ_2) = (2 $\times$ thickenss of wall)/(3EA_s)= 0.000332

Average surface strain (ϵ_m) = ϵ_1-ϵ_2 = 9.80381E-05

$$acr = \sqrt{(\frac{s}{2})^2 + (c + (\frac{\phi}{2})^2)} - \frac{\phi}{2} \tag{4.1}$$

acr = 41.92

w = 3$\times$acr$\times\epsilon_m$ = 0.01233 < 0.2 .....ok

Same procedure follow for other segments.

Crackwidth check calculation of cylindrical wall is shown in table 4.6

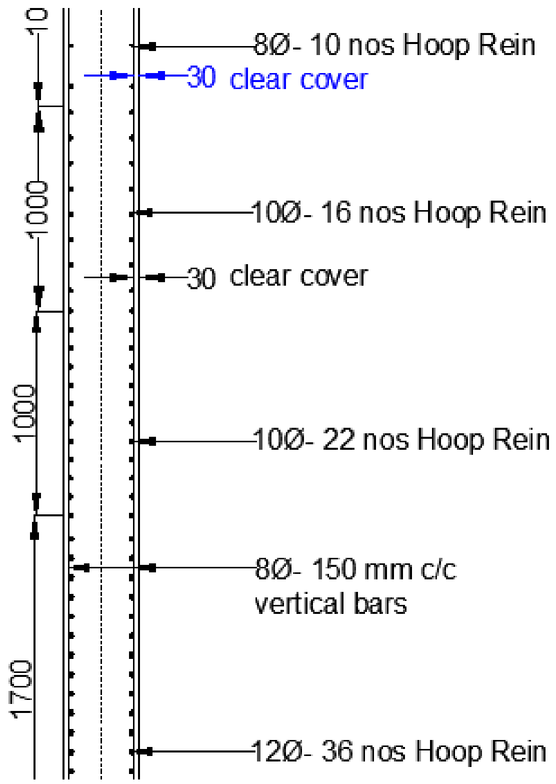

Figure 4.2: Reinforcement of cylindrical wall– crack width calculation

Table 4.5: Summary of Cylindrical wall

Segment	**Height**	**Hoop bars**		**vertical Bars**
		Dia	**No**	
	m	**mm**		
1	0.3	8	4	8ϕ-150 mm c/c
2	1	8	10	8ϕ-150 mm c/c
3	1	10	16	8ϕ-150 mm c/c
4	1	10	22	8ϕ-150 mm c/c
5	1.7	12	36	8ϕ-150 mm c/c

Table 4.6: Crack width check calculation of cylindrical wall

	Segment 2	**Segment 3**	**Segment 4**	**Segment 5**
Hoop tension (kN)	48	143.96	223.944	523.47
Thickness (mm)	100	250	250	250
Cover (mm)	30	30	30	30
Diameter of bars (mm)	8	10	10	12
Spacing (mm)	100	62.5	45.455	27.78
E (N/mm^2)	200000	200000	200000	200000
As (mm^2)	502.65	1256.64	1727.876	4071.50
Permissible strain in steel	0.00181	0.00181	0.00181	0.00181
Permissible stress in concrete	13.5	13.5	13.5	13.50
Effective depth (mm)	66	215	215	214
Stress in steel	71.620	57.281	97.205	96.427
ϵ_1	0.00036	0.00043	0.00049	0.00048
ϵ_2	0.00033	0.000332	0.0002411	0.00010
ϵ_m	2.653E-05	9.804E-05	0.0002449	0.00038
acr	56.46	41.92	36.73	32.586
Crack width (mm)	0.00449	0.01233	0.0270	0.037
	ok	ok	ok	ok

4.2.4 Middle Ring Beam

Assume the size of middle ring beam :

width = 1000 mm

Depth=500 mm

Angle at conical tank with ring beam (Θ)= 45 Degree

Vertical Loads on ring Beam

Table 4.7: Vertical Loads on ring Beam

Sr no	Element	Load (kN)
1	Roof dome	420.528
2	Top ring beam	98.96017
3	Cylindrical wall	1107.411
4	Self weight	471.2389
5	Water on ring beam	471.2389
	Total (W)	2569.377

UDL per unit length of perimeter (V) = W/$2\pi r_1$ = 68.15 kN/m

Horizontal component (H)= VcotΘ = 68.15 kN/m

Hoop tension (P) = H x r = 408.9 kN

Hoop tension due to pressure on ring = 70.5 kN

Total load = 408.9 kN + 70.5 kN = 479.4 kN

Use 20mm ϕ bar

A_{st} = 3687.692 mm^2 .

Provided no of bars = 12 no

Hoop bars –Provide 12 no bars - 20 mm ϕ. (A_{st}= 3769.91 mm^2).

shear reinforcement

use 8 mm ϕ bars - 4 legged stirrups

S_v = 181.48 mm^2

Provide 8 mm ϕ bars - 4 legged stirrups @ 180 mm c/c.

Table 4.8: Summary of Middle ring beaam

M-25 Concrete	**Reinforcement**
Width = 1000 mm	20 mmϕ -12 no - Hoop bars
Depth = 500 mm	8 mm ϕ- 4 legged stirrups @ 180 mm c/c.

4.2.5 Conical Container

Inclination cone angle = 45 Degree

Assume thickness of slab = 300 mm

Height of cone =2 m

Radius at top of conical dome (r_1) = 6 m

Radius at bottom (r_2) = 4 m

Derivation of Hoop tension in Conical Dome

Consider element ring of conical dome at depth 'h' from FSL :

Consider equilibrium of element of unit lenght of this ring,

Here t is thickness of conical dome.

W_1 = Weight of element = $\rho_{concrete}$tcosecΘ

W_2 = Force due to water pressure = ρ_{water}tcosecΘ

P= Hoop Force

F_1 and F_2 are meridional forces @ top and bottom

Resolving Forces vertically and horizontally:

a. (F_2-F_1).sinΘ = W_2.cosΘ + W_1

(F_2-F_1) = W_2.cotΘ + W_1cosecΘ

b. P= (F_2-F_1) cosΘ + W_2sinΘ

Substituting for (F_2-F_1) ,

P =(W_2.cotΘ + W_1cosecΘ) cosΘ + W_2.sinΘ

P= W_2 (cotΘ.cosΘ +sinΘ) + W_1cosecΘ.cosΘ

P = (W_2 +W_1 .cosΘ) cosecΘ

Next , Substituting For W_1 and W_2 ,

P =(ρ_{water}.h+ $\rho_{concrete}$.t .cosΘ) + $(cosec\Theta)^2$

hoop Tension = H = P× r = ρ_{water}hr $(cosec\Theta)^2$ + $\rho_{concrete}$t cosΘ $(cosec\Theta)^2$

so, H_1 = components due to water pressure = ρ_{water}hr $(cosec\Theta)^2$

H_2 = components due to self weight = $\rho_{concrete}$t cosΘ cosecΘ

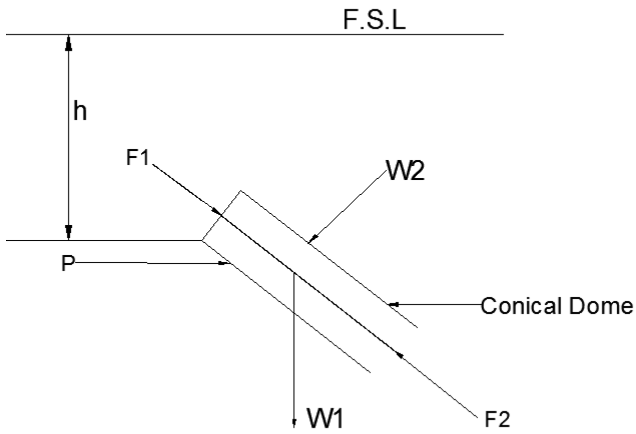

Figure 4.3: Forces on Element of Conical Dome

Evaluation of Hoop Forces

Provide 16 mm ϕ bars as hoop bars

$$a_{st} = 201.12\ mm^2$$

Table 4.9: Hoop Bars Calculation of Conical Container

Segment	**Height**	**Height from CG**	**Radius**	H_1	H_2	**H**	**Ast**	**No of Bars**	**Provided bars**	**Ast Provided**	
	m	**m**	**m**	**kN**	**kN**	**kN**	**mm2**			mm^2	
1	1	2.8	5	280	53.03	333.03	2561.792	12.74	14	2814.867	ok Ast
2	1	3.8	4	304	42.43	346.43	2664.819	13.25	14	2814.867	ok Ast

Meridional Steel

percentage of steel provided as per thickess is 300mm = 0.242 %

Table 4.10: Meridional Steel in Conical Container

segment	**Perimeter**	**Ast**	**Bar dia**	**no of bars**	**provided**
	m	**mm2**	**mm**		**bars**
1	31415.93	22888.75	10	291.43	300
2	25132.74	18311	10	233.14	300

Meridional Stresses in Concrete

For working out meridianal forces at the level of junction of conical dome with circular girder, evaluate vertical loads transffered to the girder through conical container.

Table 4.11: Weight of Concrete and Plaster

Sr no	**Element**	**Concrete**	**Plaster**	**Total**
		kN	kN	kN
1	Top dome	320	100.528	420.528
2	top ring beam	98.960		98.96
3	cylindrical wall	1107.41	60.318	1167.73
4	Middle ring beam	471.238		471.23
5	conical container	666.43	142.1723	808.607
	Total	2664.049	303.08	2967.06

Weight of Water On Conical Dome

Table 4.12: Weight of Water On Conical Dome

	Element		**kN**
1	cylindrical container	$\pi\ r_1^2$ h	5654.867
2	Conical contianer	$\pi/3(r_1^2{+}r_2^2{+}r_1^2 r_2^2)$h	1591.74
	total		7246.607

In above Table 4.12, r_1 = 6 m, r_2 = 4 m and h = 5 m

Total vertical load through conical part = 2967.061 + 7246.607 = 10213.66 kN

UDL per unit length of perimeter (V)= W/($2\pi r_1$) = 270.92 kN/m

Horizontal component of thrust on girder (H) = V/cotΘ =270.92 kN/m

Meridional force in Conical Dome (T) =V/cosecΘ = 383.147 kN/m

Meridional compressive stress = T/ A_c= 1.2771 N/mm^2 < 8 N/mm^2 .... OK

Crackwidth check for Conical dome

Table 4.13: Crackwidth Check for Conical Dome

	Segment 1	Segment 2
Hoop tension (kN)	333.03	346.43
Thickness of section (mm)	300	300
Clear Cover (mm)	30	30
Dia bars (mm)	16	16
Spacing (mm)	66.66	66.666
E (N/mm^2)	200000	200000
A_s (mm^2)	2814.87	2814.87
Permissible strain in steel	0.00180	0.0018
Permissible stress in concrete	13.5	13.5
Effective depth (mm)	262	262
Stress in steel (T/2A_s)	82.818	86.149
ϵ_1	0.00041	0.0004
ϵ_2	0.0002	0.0002
ϵ_m	0.0003	0.0003
a_{cr}	44.15	44.15
crack width (w) mm	0.035	0.0375

Summary of conical part

Inclined Angle with horizontal = 45 degree

Radius at top r_1 = 6 m

Radius at bottom r_2 = 4 m

Height of frustum of cone = 2 m

Table 4.14: Summary of Conical Dome of Reinforcement

Segment	**Thickness**	**Hoop Reinforcement**		**Meridional Reinforcement**	
	mm	**Dia Bars (mm)**	**Nos**	**Dia Bars (mm)**	**Nos**
1	300	16	14	10	300
2	300	16	14	10	300

4.2.6 Bottom Spherical Dome

Thickness of bottom spherical dome = 150 mm

Rise of spherical dome h_2 = 1.5 m

Radius of bottom dome r_2 = 4 m

Sperical radius (R_2) = (h_2/2) + (r_2/2 h_2) = 6.08 m

Geometrical Parameters

Surface area of dome =$2\pi R_2 h_2$= 57.30 m^2

Angle at Springing (Θ_2) = sin^{-1} (r_2/R_2) = 40.47 Degree

Loads

Dead Load (Self Weight) = 3.75 kN/mm^2

Plaster = 1.6 kN/mm^2

Total dead load = 5.35 kN/mm^2

Water Load = ρ_{water} × height of water up to FSL =53 kN/mm^2

Total Load UDL (w)= Total dead load + water load = 58.35 kN/mm^2

Meridional Stress

Total UDL= total load × surface area of dome = 3343.60 kN

Perimeter = $2\pi r_2$= 25.12 m

Vertical load per meter of Perimeter (V)= 133.038 kN/m

Merional thrust per meter, (T) = V/$sin\Theta_1$ = 204.959 kN/m

Area of concrete = A_c = 150 × 1000 = 150000 mm^2

Meridional Compressive stress = 1.37 N/mm^2 < 8 N/mm^2 .... OK

Meridional reinforcement

Provide minimum reinforcement as per thickness of dome 150 mm = 0.29 % .

A_{st} = 428.57 mm^2

Provided Spacing = 100 mm

Provide 8 mm ϕ bar - @ 100 mm c/c . (A_{st} = 502.654 mm^2).

Extra reinforcement : -

Meridional reinforcement at top 10 mm ϕ bar 100 mm c/c with 30 mm cover

Hoop reinforcement

H_1 = hoop tension due to concentreated Load = -P/($2\pi R_2 sin\Theta_2^2$)

H_2 = hoop compression due to UDL = wR_2(-1+$cos\Theta_2$+$cos\Theta_2^2$)/(1+$cos\Theta_2$)

Bottom spherical dome has to divide in segments as per radius up to springings .

Table 4.15: Hoop reinforcement of Bottom spherical dome

Seg.	**Radius**	**SinΘ**	**CosΘ**	**Thickness**	H_1	H_2	**H**	**Stress**
					Full	**Full**	**Full**	
					kN/m (-ve)	**kN/m**	**kN/m**	**N/mm^2**
1	1	0.165	0.986	150	0	171.34	171.34	1.14
2	2	0.335	0.944	150	0	152.60	152.60	1.02
3	2.5	0.423	0.912	150	0	137.96	137.96	0.92
4	3	0.514	0.871	150	0	119.36	119.36	0.8
5	3.5	0.608	0.821	150	0	96.35	96.35	0.64
6	4	0.706	0.761	150	0	68.38	68.38	0.46
					Max	=	171.33704	

Seg.	H_2	**H**	**Avg force**	**Hoop reinforcement**	**Bar dia**	**Provided no of bars**	**Provided Ast**
	Empty	**kN/m**	**kN**	mm^2	**mm**		m^2
1	15.710	15.710					
2	13.992	13.992	14.851	114.237	12	3	339.29
3	12.649	12.649	6.660	51.234	12	3	339.29
4	10.944	10.944	5.898	45.372	8	10	502.65
5	8.834	8.834	4.945	38.035	8	10	502.65
6	6.270	6.270	3.776	29.047	8	10	502.65

Table 4.16: Summary of Bottom Spherical dome

Seg	**r=(m)**	**r=(m)**	**Hoop Bars**		**position**	**Meridional bars**	
			Dia	**No.pitch**		**Dia**	**No.pitch**
			mm	**mm**		**mm**	**mm**
1	at support r=4	2.5	8ϕ-100c/c		lowest layer with 30 mm clear cover	8ϕ-100c/c	
2	2.5	2.0	12ϕ-2No		lowest layer with 30 mm clear cover	8ϕ-100c/c	
3	2.0	1.0	12ϕ-3No		From slab beam	8ϕ-100c/c	
4	at support r=3.5	2.0	8ϕ spacers for radial bars at 200c/c		below radial bars	10ϕ-100c/c with 30mm cover from top	

4.2.7 Bottom Circular Girder

Radius of bottom dome (r_2) = 4 m

Assume size of girder = 500 $\times$ 1000 mm

Angle between column (Θ) = 45 degree

$\alpha = \Theta/2 = 0.393$ radian

Arc distance between columns =AL = $r_2\Theta = 3.141$

Chord Lenght L = $2r_2 cot\Theta\ /2 = 19.31$ m

Dead Loads on Bottom ring beam calculated shown in Table 4.17

Table 4.17: Dead Loads from elements on the bottom girder

Sr no	**Element**	**kN**
1	Top dome	420.528
2	Top ring beam	98.96
3	Cylindrical wall	1107.41
4	Middle ring beam	471.23
5	Conical dome	808.60
6	Self wt of bottom ring beam	314.15
7	Bottom Dome	306.56
	Total	3527.47

Water Load on Bottom ring beam calculated shown in Table 4.18

Table 4.18: water Load on bottom girder

Sr no	Element	r_1	r_2	h		Load
		m	m	m		kN
1	Conical dome	6	4	2.5	πh(r_1^2+r_2^2+r_1r_2)/3	1989.67
2	Cylindrical wall	6	0	4.5	πh(r_1^2-r_2^2)	5089.38
	Total					7079.05

Total Load

Dead Load = 3527.47 kN

Water Load = 7079.05 kN

Total Load (W) =10606.52 kN

Analysis of Circular Girder for Flexure :

Bottom Circular girder is act as beam curved in plan .

UDL on Girder per meter (w) = W/2πr_2 = 422.02 kN/m

B.M = wr^2(-1+$\alpha sin\phi$+$\alpha cos\phi cot\phi$)

	Mid span	Support
α	0.393	0.393
ϕ	0.393	0.000
B.M. (kN/m)	176.72	-350.719
pt	0.099	0.2010
Ast	497.94	1005.435
bar dia mm	16	20
provided no of bars	4	4
provided A_{st} (mm^2)	804.24	1256.63

At mid span provide 4 no - 16 mm ϕ bars

At support provide 4 no - 20 mm ϕ bars

Circular girder design for Shear and Torsion

Critical section for shear occurs at a distance equal to effective lenght d_e from the face of the support . In case of circular column the effective face of column be taken at a distance equal to 2/3 times the radius of column from the center of column. Thus, Shear force be worked out at a distance x = 2/3 (radius of column)+ d_e

here , d_e = effecctive depth of column

Load per unit lenght = W/$2\pi r_2$ = 422.02 kN/m

distance x from center line of column = 1.15 m

load per unit lenght = 422.02 × 1.15 = 485.32 kN

f=x/r_2 = 0.29 radian

Distance form mid span = α - f = 0.11 radian

Shear orce at critical section (V)= 177.58 kN

Torsion at angle ϕ = T

$$T_\phi = wr_2^2(\phi - \alpha + \alpha\cos\phi - \alpha\cot\alpha\sin\phi) \tag{4.2}$$

T_ϕ =5303.26 kN

Design Shear =

$$V_e = V + \frac{1.6T}{b} \tag{4.3}$$

V_e = 194.55 N

τ_v = 0.3891 N/mm^2

pt= 0.264 %

τ_c= 0.23 N/mm^2 (IS -456-2000, Table -23)

Shear resisted by concrete = V_c = 109. 25 kN

Shear resisted by reinforcement (balance shear) = 85.3 kN

Provide 8mm ϕ bar - 2 legged stirrups

A_{sv} = 100.530 mm^2

S_v = 0.87 ×f_y × A_{sv}/V_c = 425.49 kN

As per IS-456-2000 cl.26.5.17 Transverse reinforcement

1) $0.75d_e = 712.5$ mm

2) 300 mm

3) $S_v = 0.87f_yA_{sv}/0.4$b $= 181.48$ mm

Spacing should be minimum of above three

Provide 8 mm ϕ -2 legged stirrups @ 180 mm c/c

Table 4.19: Summary of Bottom Ring Beam

Section			
	Width	500 mm	
	Depth	1000 mm	
Reinforcement			
		Dia (mm)	**No**
Main Rein	at bottom	20	4
	at top	12	2
Support reinforcement	at bottom	20	2
	at top	20	6
Stirrups		8 mm ϕ -2 legged @ 180 mm c/c	

4.2.8 Center of mass of Container

Computation of Center of Mass of container when Tank Empty

Table 4.20: CG of container Tank Empty

Sr no	**Element**		**Load**	**Distance from top of staging**	**Moment**
			kN	m	kN-m
1	Roof dome	plaster included	420.528	9	3784.75
2	Top ring beam		98.96	7.3	722.408
3	Cylindrical wall	Self weight	1107.41	5.75	6367.60
4		Plaster	60.32	5.75	346.84
5	Middle ring beam		471.24	3.25	1531.53
6	Conical dome	plaster included	808.6047	1.5	1212.90
7	Bottom dome	plaster included	306.57	1	306.57
8	Bottom ring beam		314.16	0.5	157.08
	Total		3587.793	87.65	14429.69

CG of container when tank is empty = 4.02 m

Computation of Center Of Mass when tank is full of water

Water Load

Table 4.21: Computation for Water Load

Sr no	**Element**	**Load**	**Distance from top of staging**	**Moment**
		kN	**m**	**kN-m**
1	Conical dome	1591.74	1.5	2387.61
2	Cylindrical art	5654.86	5.5	31101.76
	Total	7246.60		33489.37
Distance of CG = 4.62 m				

Table 4.22: Computation for center of mass - Tank Full

Sr no	**Element**	**Load**	**Distance from top of staging**	**Moment**
		kN	**m**	**kN-m**
1	Dead load of container	3587.79	4.02	14422.93
2	Water in container	7246.61	4.620	33479.32
	total	10834.40		47902.25

CG of container when tank is Full = 4.42 m

Summary of Center of Gravity of Container

Table 4.23: Summary of Center of Gravity of container

Sr no	**Condition of Tank**	**Load**	**Distance of CG**
		kN	**m**
1	Empty	3587.79	4.02
2	Full	10834.4	4.42

4.3 Design of Staging

Geometrical Data of staging are below:

No of column = 8 nos

Radius of bottom dome (r_2) = 4 m

Diameter of column = 500 mm

Number of bracing level = 3

Size of braces = 250 x 500 mm

Bottom girder = 500 x 1000 mm

Raft beam at Foundation = 630 x 900 mm

Levels

Levels at center lines of various elements are noted below with refrence of G.L

Table 4.24: Levels of Elements From Groundlevel

Levels	m		m
G.L	**0**	**LSL**	**18**
Foundation	-3	Girder bottom	16.33
raft beam c/c	-2.55	top raft	-2
col ht bet braces	4.58	Column ht total	18.33
Brace 1	2.58	Girder c/c	16.83
Brace 2	7.16	CG of tank empty	20.35
Brace 3	11.74	CG of tank full	20.75
		FSL	24.83

G.L.= Ground Level

Self Weight of Staging

Angle between each Column = 45 Degree

Lenght of one brace = $D_s$sin(π/n)- D_c = 2.46 m

here, D_s = Diameter of Satging

D_c = Diameter of column

n= number of columns

Table 4.25: Self Weight of Staging

sr no	Element	No	Length	b	d	weight
			m	**m**	**m**	**kN**
1	Column	8	18.425	0.5		723.548
2	Braces	24	2.461	0.25	0.5	192.11
	Total					915.65

Vertical Loads on Staging Table 4.26 Shows Vertical Loads on Satging and Vertical Load per column.

Wind Load

Basic Wind Speed (V_b) = 47 m/sec

k_1 = 1.06 (IS:875 (3), Table 1)

k_2 = 1.03 (IS:875 (3), Table 2)

Table 4.26: Vertical Loads on Staging

Sr no	**Element**	**Loads on Stagging**	**Load per column**
		kN	**kN**
1	Self wt of container	3587.79	
2	Self wt of stagging	915.6	
	Total Load empty	4503.45	562.93
3	Water load	7246.60	
	Total load (full)	11750.05	1468.75

$k_3 = 1$ (IS:875 (3), cl. 5.3.3)

$V_z = 51.314$ m/sec

$P_z = 1580$ N/m^2

Shape factor = 0.7

Effective Wind Pressure = 1106 N/m^2

Table 4.27: Computation of Wind force on Container

Sr no	**Element**	**Dimensions**		**Area**	**Wind**	**Distance from**	**Moment**
		b	**d or ht**		**Force**	**top of column**	
		m	**m**	m^2	**kN**	**m**	**kN-m**
1	Top dome	12	1.7	20.4	22.56	9	203
2	Top ring beam	12.35	0.3	3.70	4.09	7.3	30
3	Cylinderical wall	12.25	5	61.25	67.74	5.75	390
4	Middle ring beam	13	0.5	6.5	7.18	3.25	23
5	Conical Dome	10	2	20	22.12	1.5	33
6	Bottom Ring beam	8	1	8	8.84	0.5	4
	Total				132.55		683

Wind Force On Satging

Table no 4.28 Shows the Wind force acting on tank Staging .

Table 4.28: Computation Wind Force On Staging

Sr No.	**Element**	**Dimensions**			**Area**	**Wind Force**	**Distance from**	**Moment**
		b	**d**	**no**			**top of column**	
		m	**m**		m^2	**kN**	**m**	**kN-m**
1	Half col of 4th story	2.303	0.6	8	11.055	12.226	16.325	199.60
2	Half col of 3rd and 4th storey	4.60	0.6	8	22.11	24.453	11.718	286.56
3	Brace- 4	8	0.5	2	8	8.848		0
4	Half col of 2rd and 3th storey	4.60625	0.6	8	22.11	24.453	7.112	173.9
5	Brace- 3	8	0.5	2	8	8.848		0
6	Half col of 1st and 2nd storey	4.60	0.6	8	22.11	24.45	2.50	61.28
7	Brace- 2	8	0.5	2	8	8.84		0
8	Half col of 1st	1.60	0.6	8	7.71	8.527	0.9	7.6745
	Total wind load on stagging					120.65	6.042	729.05
	Total wind load on Container					132.55	21.4	2847.036
	Wind load for ESR and moment at GL					253.21		3576.09
	Moment at foundation level						17.12	4335.75

Total wind load on structure = 253.21 kN

Moment at Foundation Level = 4335.74 kN-m

Seismic Mass at CG of container

Table 4.29: Seismic Mass at CG of Container

Sr no	**Element**	**Tank Empty**	**Tank Full**
		Tonnes	**Tonnes**
1	Container	365.7	365.72
2	Water		738.69
3	Stagging	125.02	
	1/3 of Staging	41.67	41.67
	Total	407.40	1146.1

Shear per column = 31.65 kN

Maximum moment in column = 72.89 kN-m

Factored Moment in Column = 109.34 kN-m

4.3.1 Design of Column

Size of Column = 500 mm ϕ circular column

length of column = 4.5 mm

Axial Load (P_u) = 1468.75 kN

Critical bending Moment (M_u) = 102.57 kN-m

Clear Cover = 40 mm

Design of Axial Load and Bending Moment using

$P_u\ /f_{ck}D^2 = 0.235$

$M_u/f_{ck}D^3 = 0.04924$

d'/D = 0.08

Using Chart no 56 of SP-16 , Value of $p_t/f_{ck} = 0.02$

Mimimum reinforcement is provided (p_t) = 0.8 %

Use 8 no - 20 mm ϕ bar (A_{st} provided = 2512 mm^2)

Lateral Ties

Pitch

1) 16 x 20 = 320 mm

2) 500 mm

3) 300 mm

Use 8 mm ϕ ties @ 300 mm c/c

Table 4.30: Summary of Column of staging

Size	500 mm Diameter circular column	
Main reinforcement	**Dia (mm)**	**No**
	20	8
Ties	8 mm ϕ-300 mm c/c	

4.3.2 Bracing Design

Size = 250 x 500 mm

Clear length of member (L_c) = 2.561 m

Total length (L) c/c = clear length (L_c)+ diameter of column = 3.08 m

Maximum bending moment (M) = 193.417 kN-m

Design values at face of column (ML_c/L) = 159.22 kN-m

Shear force(S)= 2M/L = 124.32 kN

Design values at face of column (SL_c/L) = 102.34 kN

Design for Bending Moment

Effective depth = 450 mm

pt = 0.816 %

Use 3 no – 25 mm ϕ bas. (Provided A_{st} = 2454.31 mm^2)

Design for shear

Shear force = 102.34 kN

Nominal shear stress (τ_v) = 0.81 N/mm^2

pt = 1.17 %

$\tau_c = 0.4$

Provide 8 mm ϕ bars 2 -legged stirrups @ 240 mm c/c

Table 4.31: Summary of Bracing

Section		
	Width	250 mm
	Depth	500 mm
Main reinforcement	25mmϕ- 3 no top and bottom	
Stirrups	8mm ϕ - 2legged @ 240 mm c/c	

4.3.3 Stiffness of Staging

For evaluating Seismic Forces, it is necessary to evaluate period of vibration of the system, which depends upon stiffness of the structure. Stiffness is defined as force required to produce unit displacement at the position and direction of the force. For any supporting system of a water-tank, the mass is assumed to be concentrated at the C.G. of the container, which is found out separately for the Tank Full and Tank Empty conditions. Staging is assumed to be having only stiffness and 1/3rd of its mass is assumed to be lumped at the C.G. of the container.

Computation of stiffness with the help of FEM Software: After generating the geometrical model for shaft or staging in any software based on Finite Element Technique, like STADD or SAP the force is to be applied at a node situated above the topmost element of the system. The effect of a force at a joint situated beyond the geometrical dimensions of the model is modeled by placing a master node at the location of C.G. of the container and applying the seismic force at the point. The topmost nodes of the model at the location of beam-column junction are considered as slave nodes. Deflection of point of application of force being arbitrary actual displacement at that point in the direction of load is not real, hence the displacement and rotation at the top of staging are recorded from the results of SAP analysis used for working out stiffness of the system.

For evaluating stiffness of a structure of given geometry, an arbitrary unit load (Say 10 tonnes) is applied at the C.G. of the container (which is at distance of ' h' from top of staging, value of h is different for Tank Full and tank Empty Conditions). As shown in the figure 4.4. Stiffness of the system is obtained by dividing the value of arbitrary force with the displacement obtained in the analysis by any software.

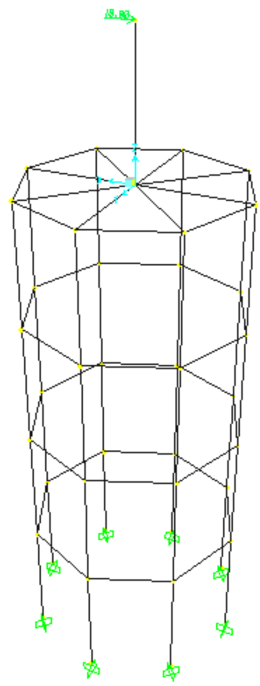

Figure 4.4: Software model for computation of stiffness

Stiffness of staging

Horizontal force applieed (W) = 10 kN

Average displacement of top stagging = 0.996 mm (Result from SAP)

Stiffness of structure = 10040.16 kN/m

4.3.4 Single Degree of Freedom

Zone factor (Z) = 0.24 (for Zone IV)

Importance factor (I) = 1.5 (IS 1893-(1)-2002 , Table -6)

Response Reduction factor (R) = 2.5 (IS 1893-(2)-2009 , Table -2)

Table 4.32: Base shear with Single Degree of Freedom System

		Tank Empty	**Tank Full**
1	Equivalent load at CG (kN)	3893.012	11139.62
2	Time period (Sec)	1.25	2.11
3	S_a/g	0.8	0.4
4	Seismic coefficient (α_h)	0.057	0.028
5	Base shear (kN)	224.23	320.82

4.3.5 Two Degree Freedom System

Evaluation of Parameters

As per provision of of draft code IS-1893:2002(part -2) the overhead water tanks have to be designed on principle of two mass idealisation,seperation of the total mass of water in to two parts, the impulsive mass that oscillates in tune with the dead weight of the system and the other convective mass that oscillates independently like a wave in the ocean. Two Degree Freedom system is applicable to only Tank Full condition. In Tank Empty condition the structure behaves as Single Degree Freedom syastem because there is no water to contribute to second degree of freedom.

Total Mass of water (Weight) = 738695.92 kg = 7246.60 kN

Total Mass of water (Volume) =738.7 m^3

Diameter of tank (D) = 12 m

Equivalent height of tank (h) [V/πD^2 /4]= 6.53 m

Ratio D/h = 1.83

Ratio h/D = 0.54

h_{cg} = 22.42 m

(IS -1893 (Part -2) table C1)

Step- I:

$$\frac{m_i}{m} = \frac{\tanh(0.86\frac{D}{h})}{0.86\frac{D}{h}} \tag{4.4}$$

m_i/m = 0.578

m_i = 0.578 x 7246.6

m_i = 4191.6 kN

Step- II:

$$\frac{h_i}{h} = 0.375. \qquad for\frac{h}{D} \leq 0.75 \tag{4.5}$$

$$\frac{hi}{h} = 0.5 - \frac{0.0937}{\frac{h}{D}} \qquad for\frac{h}{D} > 0.75 \tag{4.6}$$

h_i/h = 0.375

h_i = 2.44 m

Step- III:

$$\frac{h_i^*}{h} = \frac{\frac{D}{h}}{2\tanh(0.866\frac{D}{h})} - 0.125 \qquad for\frac{h}{D} \leq 1.33 \tag{4.7}$$

$$\frac{h_i^*}{h} = 0.45 \qquad for\frac{h}{D} > 1.33 \tag{4.8}$$

h_i^*/h = 0.73

h_i^* = 4.82 m

Step- IV:

$$\frac{m_c}{m} = \frac{0.23\tanh(3.68\frac{h}{D})}{\frac{h}{D}} \tag{4.9}$$

m_c/m = 0.407

m_c = 2952.6 kN

Step- V:

$$\frac{h_c}{h} = 1 - \frac{\cosh(3.68\frac{h}{D}) - 1}{3.68\frac{h}{D}sinh(3.68\frac{h}{D})} \tag{4.10}$$

h_c/h = 0.619

h_c = 4.046 m

Step- VI:

$$\frac{h_c^*}{h} = 1 - \frac{cosh(3.68\frac{h}{D}) - 2.01}{3.68\frac{h}{D}sinh(3.68\frac{h}{D})} \tag{4.11}$$

h_c^*/h = 0.7580

h_c^* = 4.95 m

Step- VII:

$$Kc = 0.836\frac{mg}{h}\tanh^2(3.68\frac{h}{D}) \tag{4.12}$$

K_c =8459.87 kN/m

Self wt of Container = 3587.79 kN

1/3 Self wt of Stagging = 305.21 kN

Impulsive mass of water = 4191.65 kN

m_s = 3587.792+305.2193 = 3893.01 kN

Total M_i = 8084.66 kN

Stiffness of staging = 10040.16 kN/m

Impulsive time period= T_i = 1.80 Sec

Convective time period= T_c = 3.68 Sec

Sa/g impulsive = 0.6 (IS 1893-(I)-2002 ,Figure -2 -medium Soil)

Sa/g convective = 0.525 (IS 1893-(I)-2002 ,Figure -2 -medium Soil)

Base Shear due to Impulsive Load = V_i = 349.257 kN

Base Shear due to Convective Load = V_c = 111.610 kN

As per IS-1893 (part-2) Draft code cl.4.6.3 specifies application of SRSS rule for getting resultant of Impulsive and Convective Base Shear ,

Total Base shear V = 366.657 kN

Base Moment due to Impulsive Load M_i =7904.503 kN-m

Base Moment due to Convective Load M_c =2460.566 kN-m

Total Base Moment M^* = 8278.62 kN-m

Table 4.33: Overturning Moment on Raft

Sr no.		**Ah**	**W**	**Depth from tank bottom**	**Staging height from CG OF Raft**	**Distance from Foundation**	**Moment**
			kN	**m**	**m**	**m**	**kN-m**
1	Self wt of container	0.043	3642.65	4.02	20.45	24.47	3850.66
2	1/3 wt of stagging	0.043	309.89	4.02	20.45	24.47	327.58
3	Impulsive mass of water	0.043	4191.65	2.45	20.45	22.899	4146.59
4	Convective mass of water	0.0378	300.98	4.951	20.450	25.4	288.9
	Total						8613.837

4.4 Analysis and Design of Foundation

4.4.1 Design of Circular Foundation Girder

Analysis and Design for Flxure

Assume size of girder :

Width (b) = 600 mm

Depth(d) = 900 mm

No of column = 8

Load (W) = 11750.05 kN

Radius (r) = 4 m

Column width = 450 mm

Critical section for -ve BM occurs (x) = 0.15 m

$$M_x = Wr[\frac{sin\frac{x}{r}}{2N} - cos\frac{x}{r}(\frac{\frac{N}{\pi} - cot\frac{\pi}{N}}{2N}) - (\frac{1 - cos\frac{x}{r}}{2\pi})] \tag{4.13}$$

M_x (-ve) = -283.38 kN-m

$$p_t = \frac{50f_{ck}}{f_y}[1 - (1 - \sqrt{\frac{4.6M}{f_{ck}bd^2}})] \tag{4.14}$$

p_t = 0.166 %

A_{st} = 897.29 mm^2

Use 20 mm diameter bars (ast = 314.15 mm^2)

Provided bars = 4

Provided A_{st} = 1256.637 mm^2

Provide 4 no- 20 ϕ bars.

At Mid Span

At mid span (x) = πr/N = 1.570 m

Using equation 4.13

Mx (+ve) = 195.776 kN-m

p_t = 0.113 %

A_{st} = 614.39 mm2

Use 20 mm diameter bars (ast = 314.15 mm^2)

Provided bars = 2

Provided A_{st} = 628.319 mm^2

Provide 2 no- 20 ϕ bars.

Shear check

Critical section from edge of support (x) = 2/3 × width of Girder × Effective Depth of Girder

x = 1.05 m

Shear force V_x =

$$V_x = \frac{W}{2N} - \frac{Wx}{2r\pi} \tag{4.15}$$

V_x = 243.48 kN

Torsion T_x=

$$T_x = \frac{Wr}{2\pi}[\frac{x}{r} - \frac{\pi}{N} + \frac{\pi cos\frac{x}{r}}{N} - \frac{\pi sin\frac{x}{r}}{\frac{tan\frac{x}{r}}{N}}] \tag{4.16}$$

T_x = 38.496 kN-m

Check for Shear Stress

Equivalent SF @ sec x = V_x × x +1.6T_x/b = 358.313 kN

Nominal shear stress τ_v = 0.66354 N/mm^2

P_t = 0.124 %

τ_v = 0.19 N/mm^2 (IS:456-2000, Table -23)

shear reinforcement required

Shear taken by concrete (V_c) = 102.6 kN

Balance shear (V_s) = 140.88 kN

Provide 8 mm ϕ- 2 legged stirrups @ 100 mm c/c

Table 4.34: summary of Foundation Girder

Section		
	Width	600 mm
	Depth	900 mm
Longitudinal reinforcement	**At support**	20 mm ϕ - 4 No
	At mid span	20 mmϕ - 2 No
Stirrups		8ϕ 2-legged @ 100 mm c/c

4.4.2 Annular Raft

SBC of soil = 200 kN/m^3

Soil density = 18 kN/m^3

load on foundation = 11750.05 kN

10 % load = 1175.005 kN

Total = 12925.06 kN

Area of footing = 64.625 mm2

Dimensions:

outer Radius (a) = 5.5 m

mean radius (b) = 4 m

Inner Radius (c) = 2.5 m

Plan Area (A) = 75 m^2

Modulus (Z) = 125.09 m^2

Design Loads

Dead load (W) = 11750.05 kN

Assume thickness = 0.65 m

Add self weigth = Plan Area $\times$ thickness $\times$ density = 341.25 kN

Girder weight = 339.292 kN

Total Weight = 341.25+ 339.292 = 12430.59 kN

Moment on raft(M) =4435.593 kN

Check Pressure

p_1 = (W/A)= 156.667 kN/m^2

p_2 =(M/Z) = 68.84 kN/m^2

maximum pressure = p_1 +p_2= 192.12 kN < 300 kN

Minimum pressure =p_1 - p_2= 121.20 kN

Stability Checks:

Check for overturning moments:

Overturning moment @ bending M = 4435.593 kN-m

Critical moment (1.4M) = 6209.83 kN-m

Stability force (W) = 12430.59 kN

Lever arm for stability force (a) = 5.5 m

Critical stability moment (0.9$\times$ W$\times$ a) = 61531.46 kN-m > 1.4M ok

Check for sliding:

Horizontal force causing sliding (H) = 366.657 kN

Sliding vertical force (W) = 12430.59 kN

Coefficient of friction (μ) = 0.5

Critical stability force (0.9$\times$ W$\times\mu$) = 5593.76 kN

FOS against sliding = 15.256 > 1.4 ok

Analysis of Raft Foundation

outer Radius (a) = 5.5 m

mean Radius (b) = 4 m

Inner Radius (c) = 2.5 m

p = (W/A)= 156.667 kN/m

q =(M/Z) = 68.84 kN-m

β = b/a = 0.7272

α = c/a = 0.4545

Moments on Circular raft by two types a) Direct Axial Load b) Applied Moment

Coefficent due to Direct Axial Load :

$$Y_1 = \beta^4 + 8\alpha^2\beta^2 ln\beta - \beta^2 Y_2 - Y_3 ln\beta \tag{4.17}$$

$$Y_2 = 5.48\alpha^2 - 2.52 - 2.96\beta^2 - 8ln\beta + 8\alpha^4 ln\alpha/(\alpha^2 - 1) \tag{4.18}$$

$$Y_3 = \alpha^2[-6.82 - 8\beta^2 - 21.65ln\beta + 21.65\alpha^2 ln\alpha/(\alpha^2 - 1)] \tag{4.19}$$

$$Y_4 = -8\alpha^2 \tag{4.20}$$

$$Y_5 = -\beta^4 + 8\beta^2 ln\beta - \beta^2 Y_6 - Y_7 ln\beta \tag{4.21}$$

$$Y_6 = 5.48 - 2.52\alpha^2 - 2.96\beta^2 - 8\alpha^2 lnbeta + 8\alpha^4 ln\alpha/(\alpha^2 - 1) \tag{4.22}$$

$$Y_7 = -6.82\alpha^2 - 8\beta^2 + 21.65\alpha^4 ln\alpha/(\alpha^2 - 1) - 21.65\alpha^2 ln\beta \tag{4.23}$$

$$Y_8 = -8 \tag{4.24}$$

Coefficeint Due to Applied Moment:

$$Y_1 = -\beta^4 - \beta^2 Y_2 - \frac{1Y_3}{\beta^2} - ln\beta Y_4 \tag{4.25}$$

$$Y_2 = -5.46[1 + \frac{\alpha^4}{(\alpha^2+1)}] - 0.81\beta^2 + 3/\beta^2 \tag{4.26}$$

$$Y_3 = 3\beta^2\alpha^4 - 11.12\alpha^4/\beta^2 + 20.24\alpha^4/(\alpha^2+1) \tag{4.27}$$

$$Y_4 = 12\alpha^4 \tag{4.28}$$

$$Y_5 = -\beta^4 - \beta^2 Y_6 - \frac{1Y_7}{\beta^2} - ln\beta Y_8 \tag{4.29}$$

$$Y_6 = -5.46[1 - \alpha^4/(\alpha^2+1)] - 0.81\beta^2 + \frac{3\alpha^4}{\beta^2} \tag{4.30}$$

$$Y_7 = 3\beta^2 - 11.12\alpha^4/\beta^2 + 20.24\alpha^4/(\alpha^2+1) \tag{4.31}$$

$$Y_8 = 12 \tag{4.32}$$

Radial Moments Due to Direct Axial Load (M_r)

$$M_{ri} = \frac{pa^2}{64}[-12.6f^2 - 2.3Y_2 + \frac{0.85Y_3}{f^2} - (3.15 + 2.3lnf)Y_4] \tag{4.33}$$

$$M_{re} = \frac{pa^2}{64}[-12.6f^2 - 2.3Y_6 + \frac{0.85Y_7}{f^2} - (3.15 + 2.3lnf)Y_8] \tag{4.34}$$

Tangential Moments Due to Direct Axial Load(M_0)

Table 4.35: Results for Coefficent of Moments of Circular raft

	Coefficent due to direct Load	**Coefficent due to Moment**
Y_1	0.036	0.315
Y_2	-0.124	-0.410
Y_3	-0.0976	-0.1137
Y_4	-1.652	0.512
Y_5	-4.390	3.775
Y_6	4.201	-5.464
Y_7	-3.454	1.405
Y_8	-8	12

$$M_{ti} = \frac{pa^2}{64}[-5.8f^2 - 2.3Y_2 - \frac{0.85Y_3}{f^2} - (1.45 + 2.3lnf)Y_4] \quad (4.35)$$

$$M_{te} = \frac{pa^2}{64}[-5.8f^2 - 2.3Y_6 - \frac{0.85Y_7}{f^2} - (1.45 + 2.3lnf)Y_8] \quad (4.36)$$

Radial Moments Due to Applied Moment(M_r)

$$M_{ri} = \frac{qa^2}{192}[-20.6f^3 - 6.3fY_2 - \frac{1.7Y_3}{f^3} - \frac{1.15Y_4}{f}]cos\Theta \quad (4.37)$$

$$M_{re} = \frac{qa^2}{192}[-20.6f^3 - 6.3fY_6 - \frac{1.7Y_7}{f^3} - \frac{1.15Y_8}{f}]cos\Theta \quad (4.38)$$

Tangential moments Due to Applied Moment (M_0)

$$M_{ti} = \frac{qa^2}{192}[-7f^3 - 2.9fY_2 + \frac{1.7Y_3}{f^3} - \frac{1.15Y_4}{f}]cos\Theta \quad (4.39)$$

$$M_{te} = \frac{qa^2}{192}[-7f^3 - 2.9fY_6 + \frac{1.7Y_7}{f^3} - \frac{1.15Y_8}{f}]cos\Theta \quad (4.40)$$

Values of Radial and Tengential moments at different points on Raft

Table 4.36: Values of Radial and Tengential moments at different points on Raft

		Moment Due to Direct load		**Moment Due to Moment**		**Design values**		
Radius	**f**	**Mr**	**M0**	**Mr**	**M0**	**Mr**	**M0 (+ve)**	**M0 (-ve)**
	r/a	**kN-m**	**kN-m**	**kN-m**	**kN-m**		**kN-m**	**kN-m**
For r > b								
2.5	0.45	37.74	82.27	0.00	37.66	37.74	119.92	44.61
2.8	0.51	48.84	78.97	11.90	31.90	60.74	110.87	47.07
3	0.55	62.14	79.08	19.79	29.93	81.93	109.00	49.15
3.3	0.60	90.02	82.67	32.42	29.03	122.44	111.69	53.64
3.5	0.64	113.51	87.31	41.68	29.55	155.19	116.85	57.76
3.7	0.67	140.68	93.70	51.82	30.84	192.50	124.54	62.87
For r < b								
4	0.73	188.02	106.52	87.60	42.66	275.62	149.18	63.85
4.4	0.80	90.44	95.71	53.42	37.87	143.86	133.58	57.84
4.68	0.85	45.36	87.19	37.92	34.38	83.28	121.57	52.81
4.8	0.87	31.13	83.68	33.16	33.00	64.29	116.68	50.67
5.15	0.94	5.16	74.71	25.24	29.69	30.40	104.41	45.02
5.5	1.00	0.02	68.61	25.66	27.82	25.68	96.43	40.78

Design for radial moment M_r:

Maximum design value ofM_r @ distance = 3.7 m

maximum moment (M) = 192.4 kN-m

Select depth (d) = 650 mm

width at section b (2πr) = 34.55 m

M_u/bd^2 = 0.455

p_t = 0.130 % (From SP-16- Table-1)

A_{st} = 845 mm^2

bar size = 16 mm

Spacing = 237.940 mm so, Provided spacing = 200 mm

No of bars = 172.7875

Provided no of bars = 173

Radial Direction Provide 16 mm ϕ 173 nos 200 mm c/c spacing

Table 4.37: Design for Circumferential Moment M_0

Seg.	From	To	Width	Depth	Design Moment		Average Moment	Pt	Ast	Dia Bars	Provided Bars	Provided Ast
					(-ve)	(+ve)	kN-m	%	mm2	mm		mm2
					kN-m	kN-m						
1	3	3.7	700	650	63.67	94.49	79.08	0.075	341.3698	12	4	452.389
					77.83	109.58	93.70	0.089	405.4816	16	2	402.124
2	4.4	5.5	1100	650	76.21	115.21	95.71	0.057	411.9809	16	3	603.186
					54.28	82.94	68.609	0.041	294.5097	16	2	402.124

4.5 Summary of Elevated Tank Design

Sr	**Details of concrete**			**Details of Reinforcement**			
1	**Roof Dome**						
	Radius (mm)	=	4000	8 ϕ - 150 mm c/c in both way			
	Rise of Dome (mm)	=	2000	10 ϕ-150 mm extra reinforcement			
2	**Top ring beam**						
	Radius (mm)	=	6000		**Dia (mm)**	**No**	
	Width (mm)	=	350	**Main**	16	6	
	Depth (mm)	=	300	**Stirrups**	8 ϕ- 300 mm c/c		
3	**Cylindrical Container**						
				Hoop Bars			
				Dia (mm)	**No**		
				8	4		
	Thickness (mm)	=	250	8	10		
				10	16		
				10	22		
				12	36		
				Vertical bars– 8ϕ- 150 mm c/c			
4	**Middle ring beam**						
	radius (mm)	=	6000		**Dia (mm)**	**No**	
	Width (mm)	=	1000	**Main**	20	12	
	Depth (mm)	=	500	**Stirrups**	8 ϕ- 180 mm c/c		
5	**Conical container**						
	Radius at top	=	6000	**Hoop Reinforcement**		**Meridional Reinforcement**	
	Radius at bottom (mm)	=	4000	**Dia Bars (mm)**	**Nos**	**Dia Bars (mm)**	**Nos.**
	Thickness (mm)	=	300	16	14	10	300
				16	14	10	300

6	**Bottom Sherical Dome**							
	Radius	=	4000	**r=(m)**	**Hoop Bars**		**Meridional bars**	
	Rise of dome	=	1500		**Dia (mm)**	**No. pitch**	**Dia**	**No. pitch**
	Thickness (mm)	=	150	4 to 2.5	8ϕ-100c/c		8ϕ-100c/c	
				2.5 to 2	12 ϕ-2 No		8ϕ-100c/c	
				2 to 1	12 ϕ-3 No		8ϕ-100c/c	
				3.5 to 2	8 ϕ spacers for radial bars, at 200c/c		10ϕ-100 c/c with 30mm cover from top	
7	**Bottom Circular girder**							
	Width (mm)	=	500			**Dia (mm)**	**No**	
	Depth (mm)	=	1000	**Main Rein**				
					At bottom	20	4	
					At top	12	2	
				Support rein	At bottom	20	2	
					At top	20	6	
				Stirrups		8ϕ -2 legged @ 180 mm c/c		
8	**Columns**							
	No of columns	=	8			**Dia (mm)**	**no**	
	Diameter of column (mm)	=	500		**Main Rein**	20	8	
	Length of column (m)	=	18		**Ties**	8ϕ- 300 mm c/c		
	Radius of staging	=	4000					
9	**Bracings**							
	Width (mm)	=	250		**Main rein**	25 ϕ- 3 no top and bottom		
	Depth (mm)	=	500		**Stirrups**	8 ϕ - 2 legged @ 240 mm c/c		
	Foundation Girder							
10	Width (mm)	=	600			**Dia (mm)**	**Nos**	
	Depth (mm)	=	900		**At support**	20	4	
					At mid Span	20	2	
					Stirrups	8 ϕ - 2 legged @ 100 mm c/c		

4.6 Summary

In this chapter, the design of elevated tank is included. Design of container as per selected capacity of tank and design of frame staging and annular raft foundation.

Chapter 5

Fundaments of Nonlinear Analysis

5.1 Introduction

In past earthquake, many reinforced concrete structures were severely damaged. It indicates that the need for seismic adequacy of structures and seismic rehabilitation of structures which, are vulnerable to damage must be identified and acceptable level of safety should be estimated. Achievement of good performance under earthquake load may require study of nonlinearity either geometry or material. To satisfy these requirements linear elastic models are not adequate. Thus, structural engineers have developed a new generation of design and seismic procedures that incorporate performance based design structure.

This approach basically moves away from linear elastic methods and more concern about nonlinear analysis. Nonlinear static analysis has been developed over the past twenty years and has become the popular analysis produce for design and seismic performance evaluation purpose.The procedure is relatively simple and consider post elastic behavior.

5.2 Methods of Analysis

For seismic performance evaluation of structure, the analysis methods are linear and nonlinear.

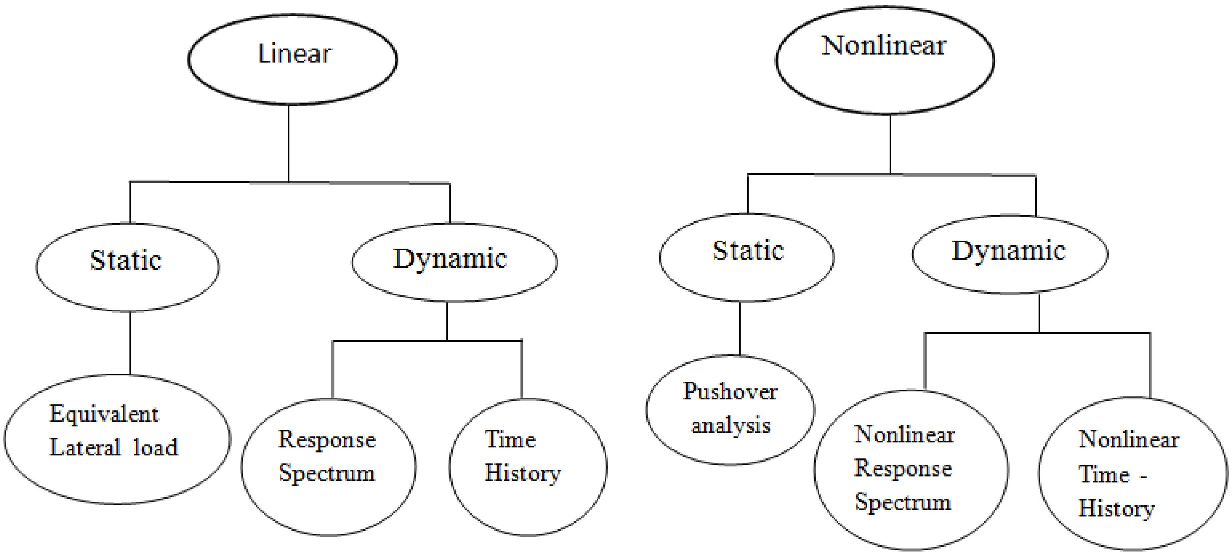

Figure 5.1: Methods of Analysis

1) Linear Analysis Method:

Standard code methods incorporate both static and dynamic analysis methods. The code static lateral load method is commonly used by the structural engineers for design of structure. In this method, the code prescribes a formula that determine lateral loads. These loads applied in such a manner that determine the adequacy of structural system. If some of the elements of designed structure are not adequate, the design is revised and redesign until all the provision of code are satisfied.

This method based on that structural components are evaluated for strength and serviceability in the elastic range. Also additional requirements are prescribed such as ductile detailing and energy dissipation characteristics to the structural components to survive into the inelastic range of lateral displacements during major earthquake.

In some cases, the structure requires dynamic lateral load methods which may be either response spectrum analysis or an elastic time history analysis. But structural elements are still design for serviceability in the elastic range of strength and deformation.

Linear elastic methods can predict elastic capacity of structure and indicates only where first yielding point will occur. But this method do not show the failure mechanism. The distribution of design forces are based on initial estimate of stiffness is dependent on the strength of element. Also the distribution of seismic forces between elements based on initial stiffness is illogical. Due to above drawbacks of force based procedure, require to develop nonlinear analysis procedure.

2) Nonlinear Analysis Method:

For seismic design of structures, the new philosophy is applied by engineers that is Performance Based Design (PBD). This new seismic design, provision will require structural engineers to perform nonlinear analysis of structure. During strong earthquake structure suffers significant inelastic deformation. Dynamic characteristics of structural change with time, so to understand the performance of structure require nonlinear analysis. Nonlinear analysis basically includes nonlinear static analysis, which also known as pushover analysis and nonlinear dynamic analysis i.e. nonlinear time history analysis.

Nonlinear static analysis that is pushover analysis, has been preferred method for seismic performance evaluation due to its simplicity. Pushover analysis is a tool which can easily apply to structure in practice. It gives an idea about post yield behaviour of structures, plastic hinges and energy dissipation through plastic hinge formation.

5.3 Performance Levels:

For a given structure performance levels are expressed in terms of limiting damage conditions for satisfactory consideration. The target performance objectives is divided into structural performance level and non-structural performance levels.

5.3.1 Structural Performance Level

Structural performance levels are defined as:

1) Immediate Occupancy (SP-1): With basic vertical and lateral structural force limiting structural damage during earthquake.The risk of life-threatening injury from structural failure is negligible.

2) Damage Control (SP-2): This term is actually not a specified value but damage is considered somewhere between Immediate Occupancy and Life Safety.

3) Life Safety (SP-3): Significant damage with some margin against total or partial collapse. Injuries may occur with the risk of life threatening injury being low. Repair may not be economical.

4) Limited Safety (SP-4): This term is actually not a specific level. It is somewhere between Life Safety and structure stability.

5) Structural Stability (SP-5): Substantial structural damage in which the structural system is on the verge of experiencing partial or total collapse. Significant risk of life. Repair of structure may not be economical.

6) Not considered (SP-6): Situations where have to performed non structure seismic evaluation or retrofit the structure.

5.3.2 Non-structural Performance Level

Non-Structural performance levels are defined as:

1) Operational (NP-A):Non-structural elements are generally functional. The failure of external utilities, communications and transportation, so it need to repair.

2) Immediate Occupancy (NP-B): Non-structural elements are generally in place but may not be functional.

3) Life Safety (NP-C): Considerable damage to non-structural components and systems but not failure of heavy items. Secondary hazards such as breaks in high-pressure, toxic or fire suppression piping should not be present.

4) Reduced Hazards (NP-D): Extensive damage to non-structural components but should not include collapse of large and heavy items that can cause significant injury to groups of people.

5) Not considered (NP-E): Non-structural elements, other than those that have an effect on structural response, are not evaluated.

5.4 Fundaments of Pushover Anlysis:

Pushover analysis in recent years is becoming a popular method for performance evaluation of existing and new structures. It is recommended by FEMA 273 [11] and Capacity Spectrum analysis method [ATC -40] [10].

Pushover analysis is an approximate analysis method in which structure is subjected to monotonically increasing lateral forces with height-wise distribution until a target displacement is reached.

Pushover analysis consists of a series of sequential elastic analysis. First gravity load is applied to the structure, after gravity load predefined lateral load pattern is distributed along the height of the structure is applied. The lateral forces are increased by some members yield. The process is continued until a control displacement achieved at the top of the structure. The roof displacement is plotted with base shear to obtain the capacity curve.

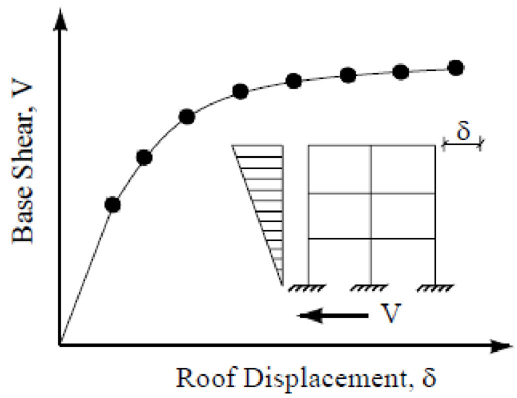

Figure 5.2: Global Capacity (Pushover) Curve of a Structure

As per above performance levels, force versus deformation curve is generated. This curve divided as shown in figure 5.2

Five points are labeled a A,B,C,D and E. it shows the performance levels are on curve. The point of localized damage in structure is often called hinge. These points shows the force deflection behavior of the hinge and three points labeled as IO,LS and CP are used to define the acceptance criteria for the hinge.

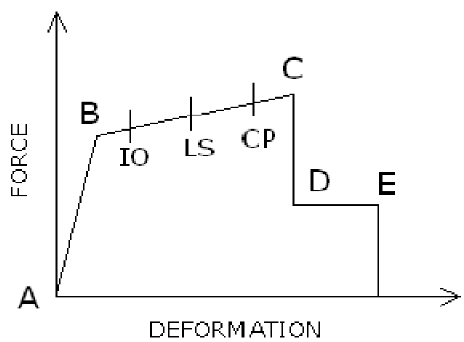

Figure 5.3: Force-deformation for pushover hinge

Where,

IO = Immediate Occupancy

LS = Life Safety

CP= collapse prevention

C= Strength Degradation

C-D = Initial failure of the component

D-E = Residual Resistance

Pushover analysis can be performed as force-controlled or displacement controlled. In force controlled analysis, load is known that is gravity loading. Also the target displacement is associated with a very small positive or even negative lateral stiffness because of the development of mechanism and P-Δ effects. In displacement control method, displacement is known, but force is unknown. Also the structure is expected to lose strength and become unstable. Generally, roof displacement of the center of mass of structure is chosen as control displacement.

5.5 Pushover Analysis by SAP2000: Software Impementation

5.5.1 Modeling of Structure

Modeling of any structure is software is very important for any type of analysis. Modeling is a very critical part of analysis, even a small mistake in modeling can change the results of the analysis. Modeling of structure includes the creating grid systems, adding frame members or shell area members to create components of structure, it is includes the assigning loads that is point load, UDL, area loads etc. After generating grid system lines in X ,Y and Z directions, the structural properties of any structural elements are material property, section property are identified. Material property defines mass density, weight, modules of elasticity and poison ratio. Section property defines geometric data that is length, width and height. Each properties is to be named and assigned to the respective structural elements.

5.5.2 Defining Static Load Cases

After the model is to be created and assigning the properties of the components, load cases should defined. Static liner load i.e. dead load, live load, earthquake loads, wind loads. And for static nonlinear load case push is defined. Dead load is calculated by software automatically. Earthquake load is applied in both X and Y directions as per IS 1893:2002 (part 1)[12]. For earthquake load zone factor, time period, soil type other parameters should defines as per code. Same as for wind load case for both X and Y direction. As shown in figure 5.4.

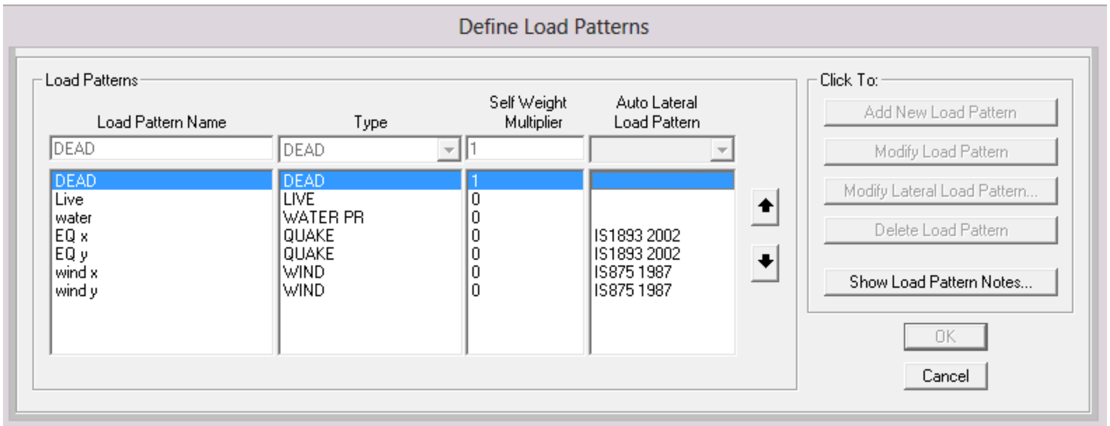

Figure 5.4: Defining Static Load Cases

5.5.3 Defining Response Spectrum Case

The response spectrum given in IS: 1893-2002 (part 1)[12] for 5 % damping is to be defined and same is to be used for performing Response Spectrum Analysis. In SAP 2000, material combination option are available with the software are complete Quadratic Combinations(CCSC), Square Root of Sum of Squares (SRSS), Absolute Method (ABS) and General Model combination (GMC). As shown in figure 5.5.

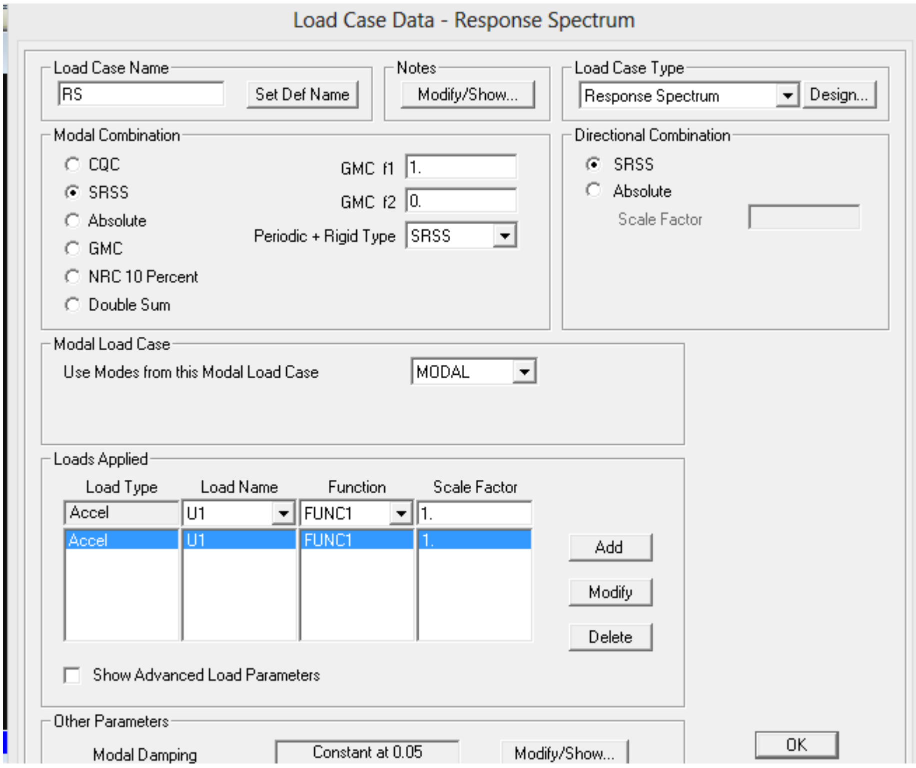

Figure 5.5: Response Spectrum Case in SAP2000

5.5.4 Defining Frame Nonlinear Hinge Property:

Defining locations in structural components based on possibility of damage. Such locations, known as hinge. In practice FEMA -273[11] and ATC -40[10] documents specified default hinge properties are used due to simplicity. There are three types of hinge properties in software: Default hinge property, user defined hinge property and generated hinge property.

For frame elements default or user defined hinge property can be assigned. Hinge types are Axial P, Shear V3, Torsion T, Moment M2 Moment M3. The suffix 2 and 3 indicates the local axe direction. P-M2-M3 hinge type is coupled hinge. Default hinge properties can not be modified. Hinge properties are assigned to frame elements and after analysis it shows the effect of default hinge properties on structure.

5.5.5 Defining Static Nonlinear cases

Nonlinear static pushover analysis is very powerful feature available in SAP 2000. Pushover analysis can consist of any number of pushover cases and each pushover case can have a different distribution of lateral load on structure. A pushover case may starts from zero initial conditions or it may be start from other nonlinear pervious case.

Generally, two static nonlinear cases are defined, one for gravity load and other for lateral load. For case of lateral load, the static nonlinear analysis starts from previous case of gravity nonlinear case. As shown in figure 5.6 and 5.7.

In SAP 2000 pushover can perform either force controlled or displacement controlled. In load application parameters"Full load" option button is used to perform a force controlled analysis. With "Displacement Controlled" button pushover case can perform with displacement control type.In this method pushover perform with displacement control type in a specific direction at a specific joint. As shown in figure 5.8.

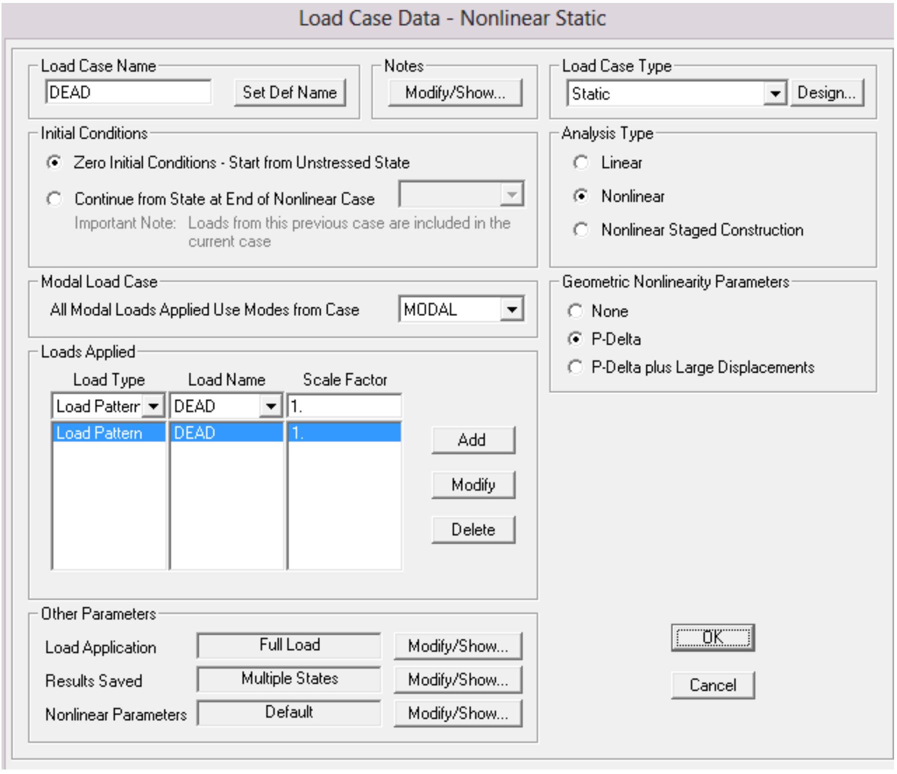

Figure 5.6: NonLinear Static case for Gravity Load

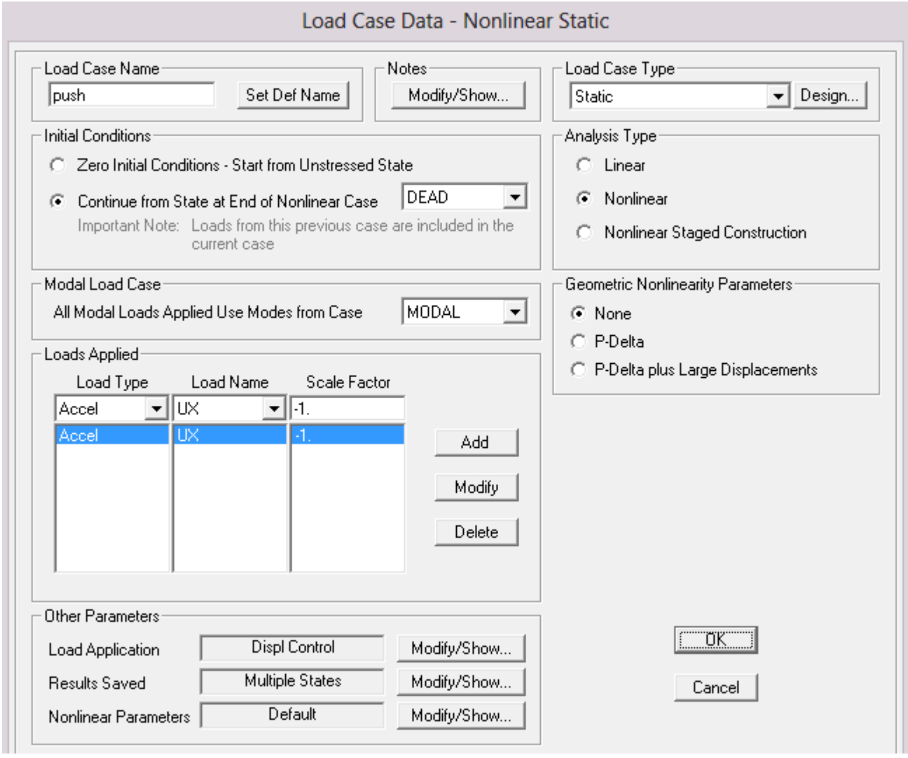

Figure 5.7: NonLinear Static case for Push case

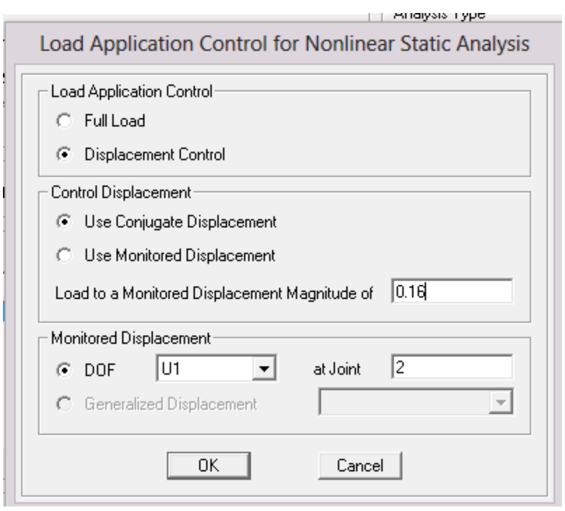

Figure 5.8: NonLinear Static case for Push case : Displacement Control

5.5.6 Performing Pushover Analysis:

Before performing pushover analysis, linear static and dynamic analysis is to be performed. Now Run for pushover cases. After completing pushover analysis it results shows, static pushover curve. This pushover curve of base shear versus displacement. The ideal pushover curve is shown in figure 5.2

A-B linear range, B-C is nonlinear range which includes different performances levels such as IO, IS and CP. Point C indicates the ultimate failure after which the residential strength remains indicated by point D. point E is the final displacement under residual strength. Pushover curve can be display in tabular format from file menu.

Total number of hinge generated in each performance it shows levels and at each step of push base shear and displacement.

High points also can be seen on the structure with Deformed shape of structure-Push. Option It shows hinge points on the frame section at different performance levels with different colors. Within A to B level hinge pont is defined by pink color, After point B up to IO level hinge points are defined by blue color. Up to LS level it is defined by sky blue color. Up to CP level hinge points are defined by green color. Up to C point it is shows by yellow color. Up to point D hinge points shows with orange color and at failure level point E hinge points are red color.

5.5.7 Obtaing Performance Point

After performing pushover analysis, the result obtain is pushover curve. That is Base shear v/s Controlled displacement. Convert this pushover curve to capacity spectrum curve. That is Acceleration Displacement Response Spectrum (ADRS) curve.

S_a = (V/W)/α_1

S_d=δ_{roof}/(P$F_1\phi_{1,roof}$)

Where, α_1 and PF_1 are the modal mass coefficient and participation factors for the first natural mode of the structure respectively. $\phi_{1,roof}$ is the roof level amplitude

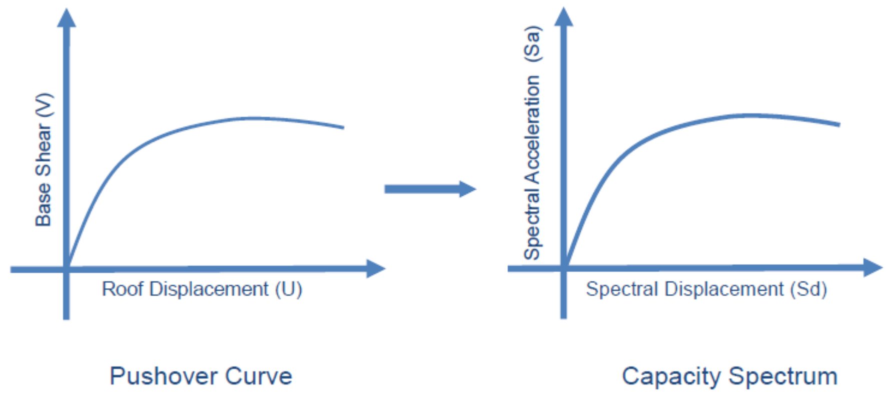

Figure 5.9: pushover Curve convert to Capacity curve

of the first mode.

Obtain the equivalent damping based on the expected performance level. Get the design response spectra for different levels of damping and adjust the spectra for nonlinearity based on the damping in the capacity spectrum. The capacity spectrum and the design response spectra can be plotted together when they are expressed in the ARDS format.

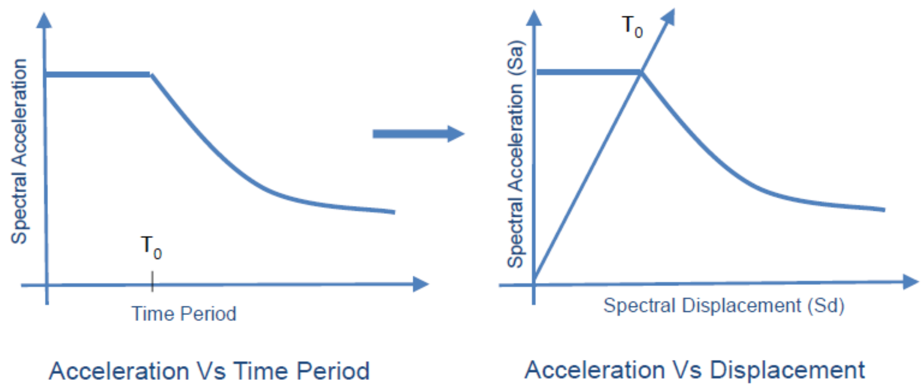

Figure 5.10: Demand Spectrum : Response Spectral Conversion

S_d=$S_a T^2/4\pi^2$

Where,

S_a =Spectra Acceleration

T =Time period

S_d =Spectra Dispalcement.

The intersection of the capacity spectrum and the response spectra defines the performance levels. In pushover curve a point on the curve defines a specific damage state for the structure, since the deformation of all components can be related to the global displacement of structure. By correlating this capacity curve to the seismic demand generated by a specific earthquake or ground shaking intensity, a point can be found defines performance point as shown in figure 5.11.

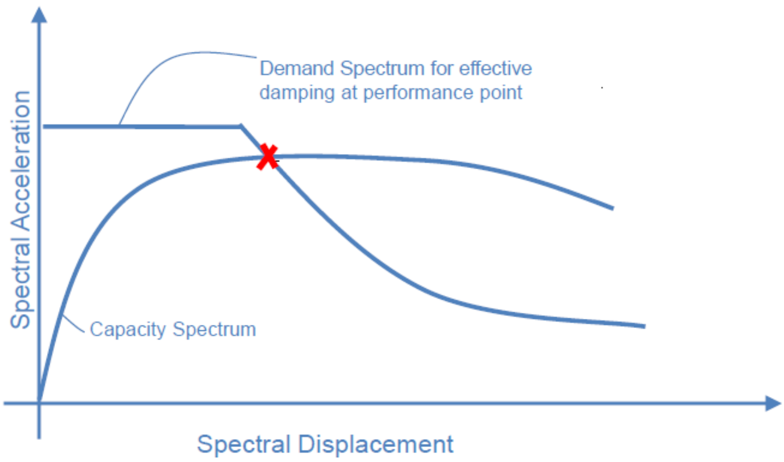

Figure 5.11: Performance Point

5.6 Summary

In this chapter discuss the fundaments of nonlinear static analysis and procedure of pushover analysis in SAP2000. With pushover analysis capacity curve i.e. base shear v/s roof dispalcement, demand curve and performance point can be obtained.

Chapter 6

Modeling and Analysis of Elevated Tank

6.1 Introduction

During earthquake any structure undergoes nonlinear state. To know the post elastic behavior of structure, it is requires for analysis by nonlinear static analysis. As water tank is important structure it is necessary to remain functional during and after earthquake. Pushover analysis is carried out for elevated RCC water tank.

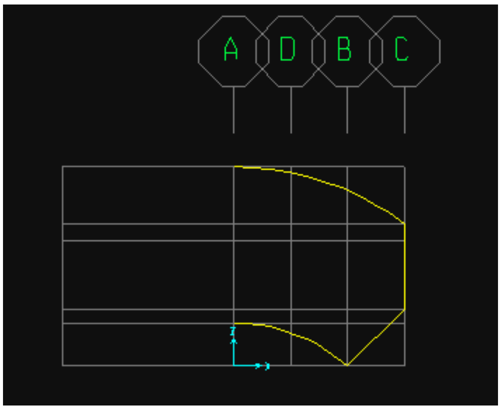

Figure 6.1: Generation of Grid Lines for Tank Container

6.2 Modeling of Elevated Tank in SAP 2000

6.2.1 Grid lines modeling

Define coordinate system, grid lines is generated for container of tank as per dimension of tank. For this selected tank. From bottom of the container, the conical container height is 2 m, cylindrical wall height is 5 m and rise of top is 2 m, and radios of top dome is 6 m and bottom dome is 4 m. So the co-ordinate system is as shown in figure 6.1.

6.2.2 Modeling of Components

The model was created, element properties were assigned and support conditions were given fixed. For elevated tank top dome, cylindrical wall, conical container and bottom dome is defined as Area section "Thin shell" element.And top ring beam, middle ring beam, bottom ring beam, bracing and columns are defined as frame element. Also bracing and columns are defined as frame element. Table 6.1 shows the thickness of shell elements of selected tank and table 6.2 shows size of frame elemnts.

Table 6.1: Thickness of Shell Elements

Element	**Thickness**
Top Dome	100 mm
Cylindrical wall	250 mm
Conical Dome	300 mm
Bottom Spherical Dome	150 mm

Table 6.2: Size of Frame Elements in Frame Elements

Element	**Size (mm)**
Top ring beam	350 x 300
Middle ring beam	1000 x 500
Bottom ring beam	500 x 1000
Bracing	250 x 500
Column	500 Diameter

6.3 Define Static Loads

Elevated water tank subjected to three types of static loads dead Load, live Load and water Load. Dead load is taken by software automatically. Live Load of 1.5 kN/m^2 acts on top of the dome which is model by applying uniform area load command as shown in figure 6.2. Water load above the bottom dome is also modeled by uniform area load command. This acts in gravitational direction. In SAP2000 software gravitational direction is the (-Z) direction.

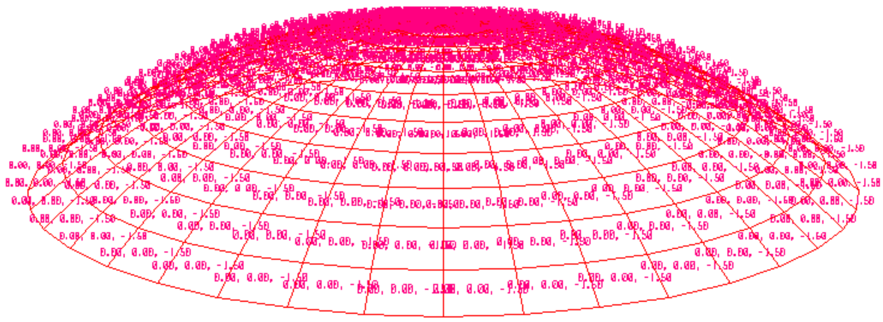

Figure 6.2: Live Load on Top Dome

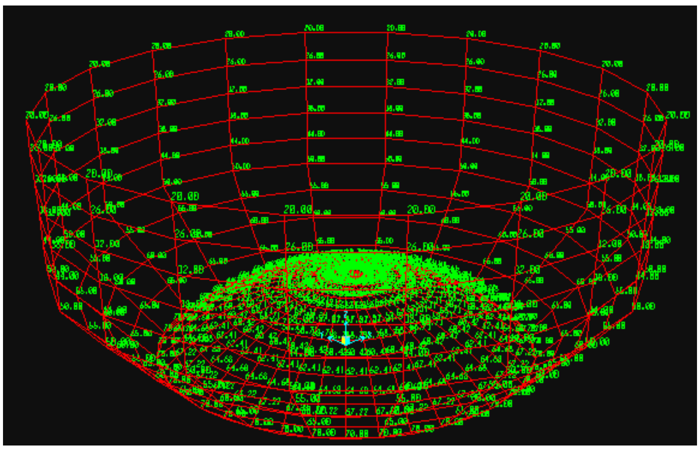

Figure 6.3: Hydrostatic Force with Joint Pattern on Wall of Tank

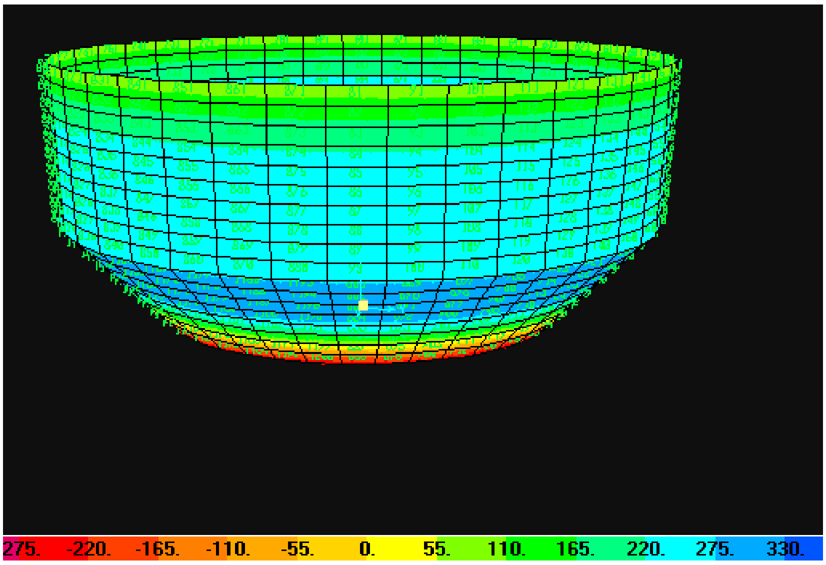

Figure 6.4: Hydrostatic Force on Wall of Tank

The hydrostatic pressure varies according to height linearly and such type of loading can be modeled in SAP 2000 software with help joint pattern command. Using joint pattern command one can generate load pattern according to requirement and then applied on surface.

Earthquake load is defined in both X and Y direction as per IS 1893:2002 (part 1)[12]. Zone factor (Z) = 0.24 , Soil type : II, Importance factor = 1.5 , Response Reduction Factor (R) = 2.5. Wind load is defined in both X and Y direction as per IS : 875-1987 (part-3).

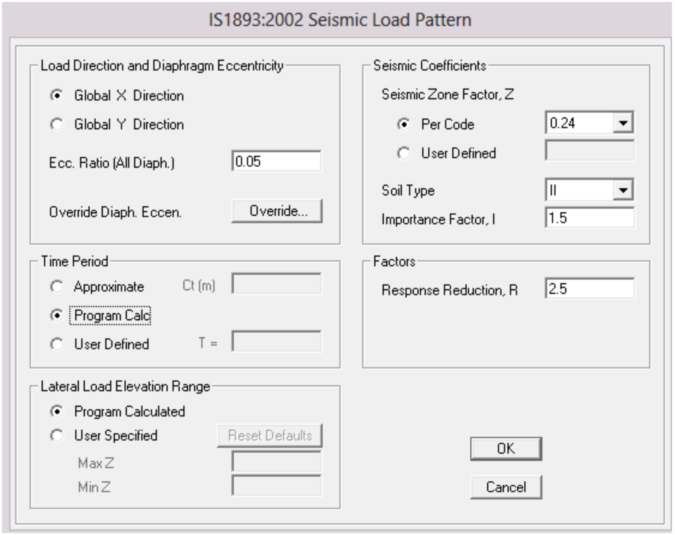

Figure 6.5: Defined Earthquake Load case

6.4 Defining Response Spectrum Case

The response spectrum case is defined as IS: 1893-2002 (part 1) [12]. For 5 % damping factor response spectrum case is defined with SRSS method as shown in figure 6.6.

6.5 Assigning Hinge Properties for Frame Elements

For bracing elements RCC beam moment M3 hinges are considered. As hinge properties are default hinge the location of hinge properties is located at relative distance zero and one. That is at the both the ends of beams and column hinge property P-M2-M3 is defined at zero relative distance. Hinge properties is defined in SAP as shown in figure 6.7.

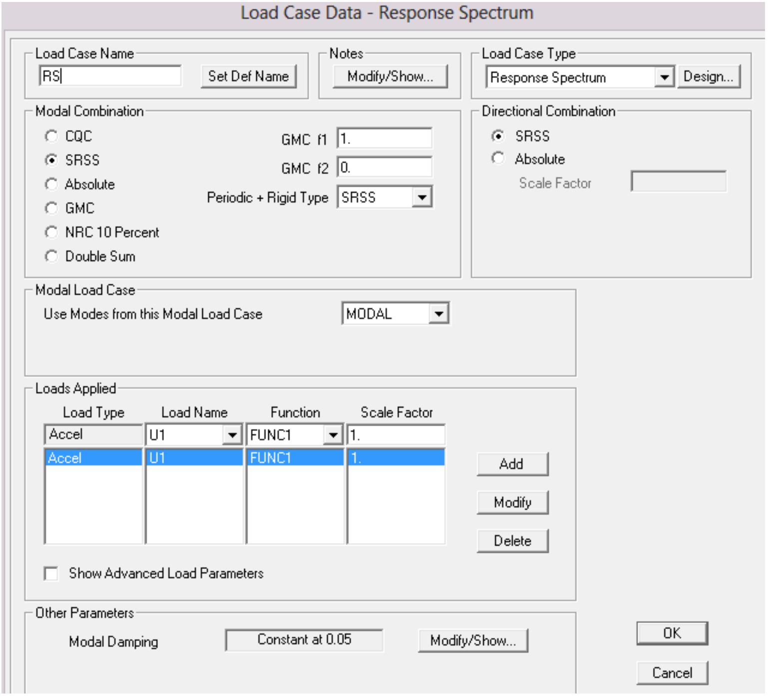

Figure 6.6: Defining Response Spectrum Case

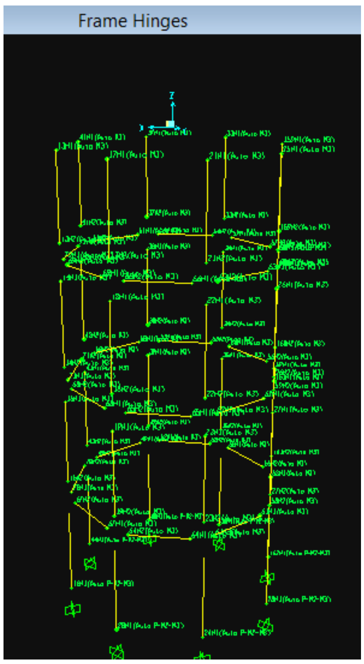

Figure 6.7: Assigning Hinge Properties

6.6 Static Nonlinear Case

Two static nonlinear case are defined. Case 1 is for dead load as shown in figure 6.8. As the load is known that is gravity load and the structure is expressed to be able to support the load, the type of nonlinear push case is force controlled. Member unloading method was unloading entire structure, geometrically nonlinearity was include as P-Δ and load was applied to the added elements. In case 2,the load control was conjugate displacement control and only lateral load case was considered in the load pattern. Load case 2 is starts from the end of the case 1 as show in figure 6.9.

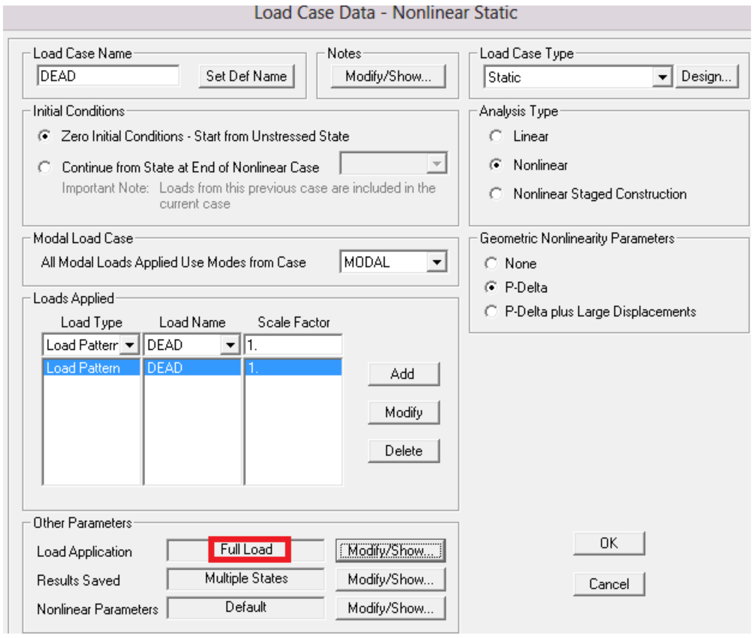

Figure 6.8: Static Nonlinear case1 for Gravity Load

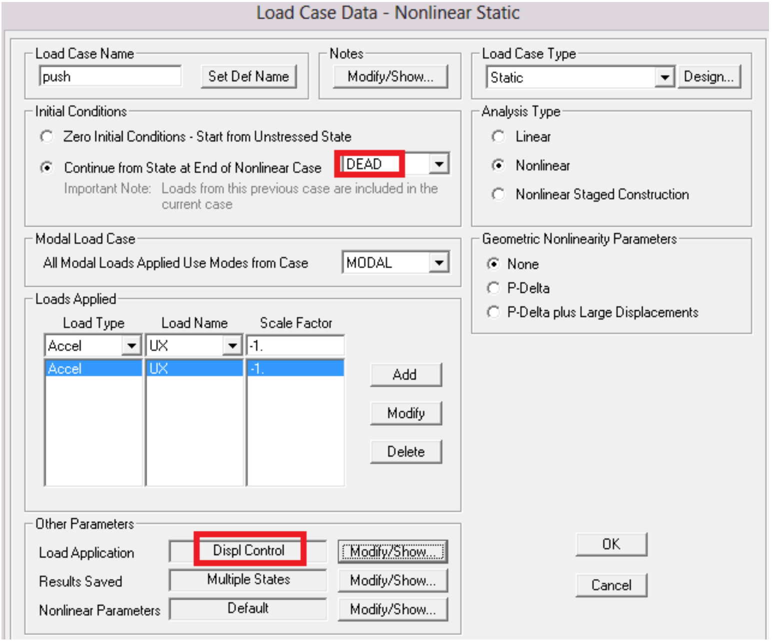

Figure 6.9: Static Nonlinear case2 for Push case

6.7 Result of Pushover Analysis of Elevated Tank

Pushover analysis is performed on elevated water tank.

The results obtained after nonlinear static analysis are, Pushover curve (Base Shear Vs Roof Displacement), Capacity Spectrum Curve (ADRS Format), Performance Point, Tabular format of pushover curve.

Pushover curve obtained as shown in figure 6.10. The ultimate base shear of the structure can take before failure is around 700 kN which is 1.9 times more than elastic base shear and the corresponding roof displacement is 180mm.

The capacity spectrum curve of the model is shown in figure 6.11, green curve in the shows the response spectrum curve for various damping values. The Response Spectrum curves are governed by the values of Coefficient of Acceleration (Ca) and

Coefficient of Velocity (Cv). For getting the response spectrum curve as per IS:1893-2002 (part I), the value of Ca and Cv were calculated and assigned to the software. The values of Ca and Cv for all type of soils. For medium soil and Zone 4, Ca is 0.24 and Cv is 0.33.

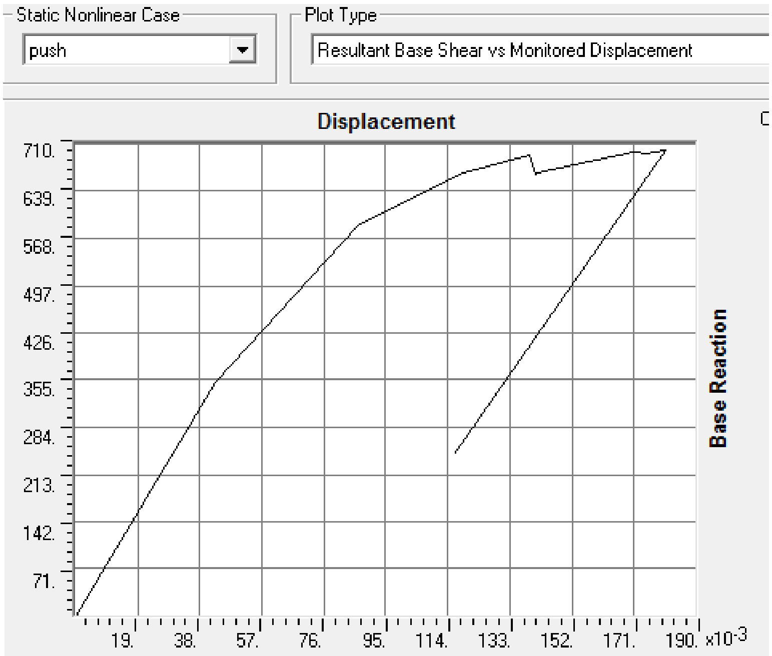

Figure 6.10: Pushover curve for Elevated Tank

Gray curve is capacity spectrum curve and red curve is is Single Demand Spectra. The intersection point of Single Demand Spectra with the Capacity Spectrum Curve is the performance point. At performance point base shear is 487.75 kN and displacement is 68mm.

In pushover analysis total ten steps are there. First hinge is generated in base column. Initially hinges were in B- IO level subsequently proceeding to IO-LS and LS-CP stage. At performance point total hinges generated is 104. Out of 104 hinges, 87 hinges are generated in A-B level and 14,3 hinges are in B-IO and IO-LS level respectively.As at performance point of structure three hinged has formed at column and of stage IO-LS, overall performance of structure is of life safety stage and

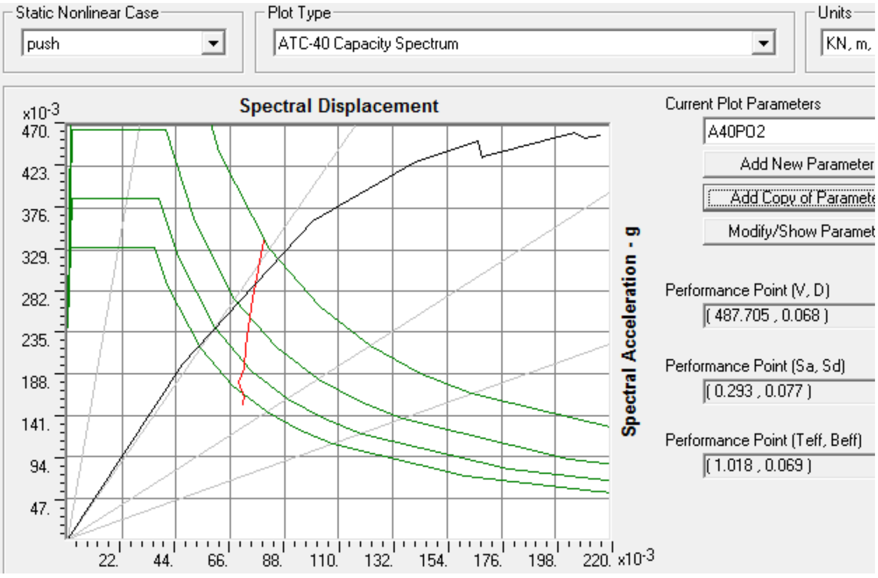

Figure 6.11: Capacity Spectrum Curve for Elevated Tank

hence, the structure has good capacity to resist future earthquake as demand seen less.Generated hinges at performance point is shown in figure 6.12.

Table 6.3: tabular formate of pushover curve for Elevated tank

Step	**Displacement**	**Base Force**	**A to B**	**B to IO**	**IO to LS**	**LS to CP**	**CP to C**	**C to D**	**D to E**	**Beyond E**	**Total**
	m	**KN**									
0	0.000106	0	104	0	0	0	0	0	0	0	104
1	0.042934	352.379	103	1	0	0	0	0	0	0	104
2	0.086431	587.336	87	14	3	0	0	0	0	0	104
3	0.118765	667.25	72	25	6	1	0	0	0	0	104
4	0.138695	693.98	64	31	6	2	0	1	0	0	104
5	0.140685	664.934	64	31	6	2	0	0	0	1	104
6	0.141568	667.627	64	31	6	2	0	0	0	1	104
7	0.170649	696.925	60	34	7	0	1	1	0	1	104
8	0.174925	695.076	60	33	8	0	1	0	0	2	104
9	0.180524	700.581	60	33	8	0	0	1	0	2	104
10	0.115912	243.559	60	33	8	0	0	0	0	3	104

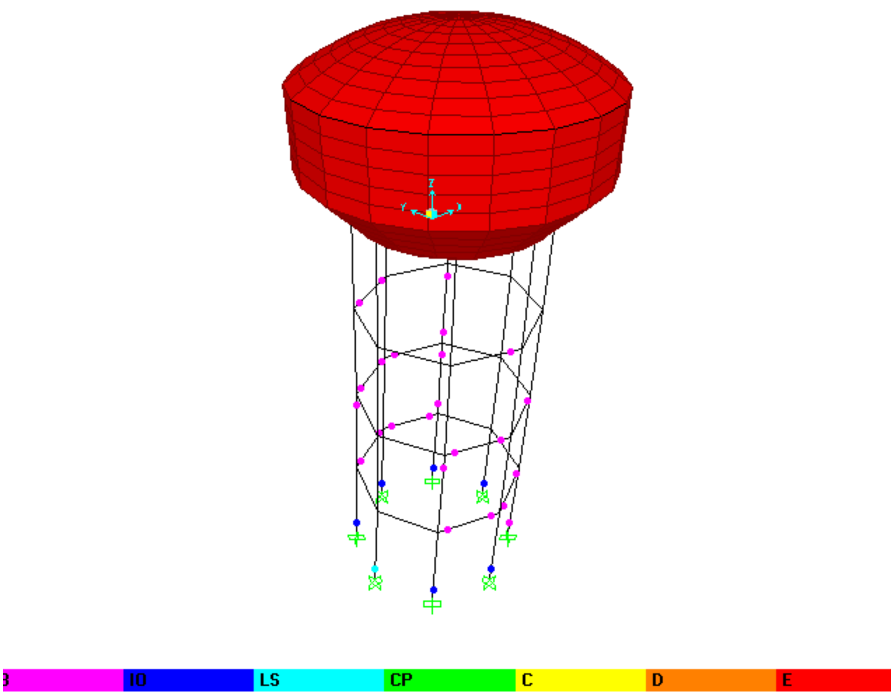

Figure 6.12: Hinge Formation at Perfomance Point for Elevated Tank

6.8 Summary

This chapter presents the procedure of modelling of elevated tank and application of pushover analysis in SAP2000 and after pushover analysis result shows of elevated thank, i.e. capacity curve,demand curve and performance point.

Chapter 7

Summary and Conclusions

7.1 Summary

As codal based design of structure is limited up to elastic behaviour of structure. Nonlinear static analysis is useful to understand post elastic behaviour of structure. To achieve of good performance and less damage with less loss of lives it is necessary to understand nonlinear behaviour of structure.

Elevated water tank is large water storage container with certain height. Elevated water tank is critical structure. During earthquake failing of elevated tank is at risk. Elevated tank should remain functional during and after earthquake to provide water supply efficiently for drinking water and firefighting purpose.

Pushover analysis is a tool for performing nonlinear static analysis. With pushover analysis base shear vs roof displacement curve can be obtained. And also understand the plastic hinge formation in structure. In practice, ATC- 40[10] documents are used for RCC structures with default hinge properties due to convenience and simplicity.

In present study one RCC Elevated water tank is design with limit state method by using IS:3370-2009 (part 1 and 2)[16].Also design to check crack width of components of Intze tank. Excel Sheet is generated for design of Elevated tank. Two mass model is consider for tank and determine stiffness of tank, Impulsive mass, Convective mass of tank, Time period of both impulsive and convective mode, base shear is determine for both impulsive and convective mode. Overturning moment on raft is obtained.

Nonlinear static pushover analysis is carried out on this elevated tank considering default hinges of SAP2000 as per ATC-40[10].The results are presented in terms of capacity, demand, performance point, pattern of hinge formation. The results help in identifying damage level of structure. Overall performance of structure is of life safety stage and hence, the structure has good capacity to resist future earthquake.

7.2 Conclusions

Based on the current study, following conclusion are made:

- Pushover analysis is a simple tool for performing nonlinear static analysis for structures.
- First hinge formation is in base column and also more number of hinges generated in the column.
- Overall performance of the structure is on life safety stage and hence, the non-structural elements are severely damaged, but should not include the falling or collapse of heavy structural elements so less possibility of threatening of life. It may cause injury but not cause loss of life.

7.3 Future Scope of Work

1) Nonlinear time history can be performed on elevated tank to understand seismic behaviour of tank.

2) Nonlinear static analysis can be carried out for elevated tank with change in height of staging, with change in capacity and compare behaviour of seismic analysis of tanks.

3) Design the shape of other than Intze type tank i.e.conical shape tank, rectangular type tank and apply pushover analysis on these tanks and compare seismic behaviour of tanks.

References

[1] M. Moselemi, M R Kianosh and W Pogorzelski, "Seismic Response of liquid Filled Elevated Tanks", Engineering structures Volume 33 (2011) page 2074-2084.

[2] Harry W Shenton, Francis P Hampton, "Seismic Response of Isolated Elevated Water Tanks", Journal of structural engineering, volume 125, September 1999.

[3] L Kalani Sarokolayani, B Navayaieya, M Hosainalibegi, J Vaseghi Amiri, "Dynamic Analysis of water Tanks with interaction between Fluid and Structure" 14th World Conference on Earthquake Engineering , October 2008, China.

[4] Sudhir K Jain, Sammer U. S., "Seismic Design of Frame Staging for Elevated Water Tanks". Journal of Structural Engineering, volume 120, Number 5, May 1994.

[5] Ahmed Ghobherab, "Performance based design in Earthquake Engineering State of Development", Journal of Engineering Structures 2011.

[6] R. Ghateh, M R Kainosh, W pagorzelski, "Seismic Response Factors of Reinforcement Concrete pedestal in Elevated Water Tanks",Journal of Engineering Structures, volume 87, October 2013.

[7] X K Zou and C M Chan, "Optimal Seismic Performance based design of Reinforced Concrete building using non linear Pushover analysis",volume 27, April 2005.

[8] Uma Chudavala, Deepam Patel, N Gopalkrishan, “Fluid Structure Soil Interaction Effects on Seismic Behavior of Elevated Water Tanks”, Nirma University International Conference on Engineering 2012.

[9] IS 456 : 2000, Indian Standard, “Plain and Reinforced Concrete Code for Practice”, fourth revision edition, Bureau of Indian Standards, New Delhi.

[10] Applied Technology Council (ATC) -40, “NEHRP Guidelines for the seismic Rehabilitation of Buildings”, Building Seismic Safety Council,Washington D C October 1997.

[11] Federal Emergency Management Agency (FEMA) -273, “Displacement-based seismic design for buildings”, Journal of the Chinese Institute of Engineers, 25(1):89-98, 2002.

[12] IS 1893 (Part 1) : 2002, Indian Standard, “Criteria for Earthquake Resistant Design of Structures”, Part 1 General Provisions and Buildings, fifth revision edition. Bureau of Indian Standards, New Delhi.

[13] Mehmet Inel ,Hayri Baytan Ozmrn, “Effects of Plastic Hinge properties in non linear analysis of reinforced concrete buildings”,Journal of Engineering Structures, 28(2006) pages 1494-1502.

[14] IS 1893-2007 (part 2) Draft code, “ IITK-GSDMA Guidelines for Seismic Design of Liquid Storage Tank”, Indian Institute of Technology Kanpur, India.

[15] IS 11682- 1985, “Criteria for Design of RCC Staging for Overhead Tanks, Bureau of Indian Standard”, New Delhi.

[16] IS 3370 (part 1 and 2) -2009, “Concrete Structures for storage of Liquids- code practice”, Bureau of Indian Standards, New Delhi.

[17] Somnath Dutta, Aparna Mandal, Sekhar Chnadra Dutta, “Soil Structure interaction in dynamic behavior of elevated tanks with alternate frame staging configurations”,Journal of sound and vibrations 2003.

[18] S.C.Dutta, S.K.Jain, C.V.Murty, “Alternate tank staging configurations with reduced torsional vulnerability”,Soil dynamics and earthquake engineering 1999.

[19] Mehmet Inel, Hayri Baytan Ozmen, “Effects of plastic hinge properties in nonlinear analysis of reinforced concrete buildings”, Engineering structures 28(2006) 1494-1502.

[20] “Structural Analysis Program SAP2000, Users manual , Computers and Structures, Inc., Berkeley, Calif.

Appendix A

Calculation of C_a and C_v

For each earthquake hazard level of interest at a site is based on the site seismic coefficients C_a and C_v. The seismic coefficient C_a represents the effective peak acceleration (EPA) of the ground. A factor of about 2.5 times Ca represents the average value of peak response of a 5 % damped short period system in the acceleration domain. The seismic coefficient C_v represents 5 % damped response of a 1-second system and when divided by period defines acceleration response in the velocity domain.

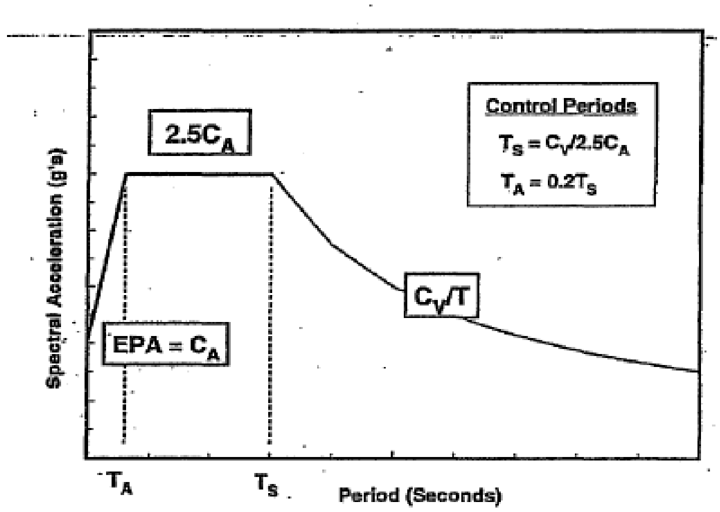

Figure A.1: Construction of 5% damped Elastic Response Spectra (ATC 40)

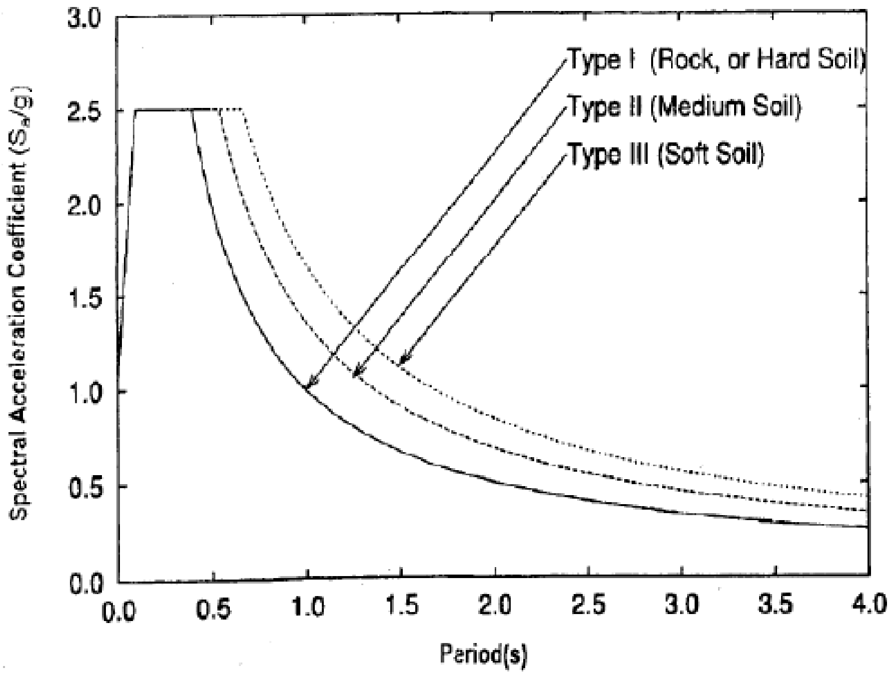

Figure A.2: Response Spectra for Rock and Soil sites for 5% damping (IS 1893-2002)(part-1) [12]

Coefficient of acceleration (C_a) = Zone factore (Z)

Coefficient of velocity (C_v) = 2.5 $\times C_a \times T_s$

$C_v = 0.24$

For Zone factore IV (IS:1893:2002)

$T_s = 0.40$ (Type I - Rocky or Hard soil sites)

$T_s = 0.55$ (Type II - Medium soil sites)

$T_s = 0.67$ (Type III - Soft soil sites)

Cv = 2.5 $\times C_a \times T_s$

$= 2.5 \times 0.24 \times 0.40 = 0.24$ (Rocky or Hard soil sites)

$= 2.5 \times 0.24 \times 0.55 = 0.33$ (Medium soil sites)

$= 2.5 \times 0.24 \times 0.67 = 0.40$ (Soft soil sites)

Table A shows value of all the zones for considering soil conditions.

Table A.1: Coefficient of Acceleration and Coefficient of Velocity

Seismic coefficient (C_a)				
Soil	**Zone II (0.10)**	**Zone III (0.16)**	**Zone IV (0.24)**	**Zone V (0.36)**
Type I	0.1	0.16	0.24	0.36
Type II	0.1	0.16	0.24	0.36
Type III	0.1	0.16	0.24	0.36
Seismic coefficient (C_v)				
Type I	0.1	0.16	0.24	0.36
Type II	0.136	0.22	0.33	0.495
Type III	0.167	0.26	0.402	0.603

Appendix B

Excel sheet of Elevated Watertank

Appendix C

Reinforcement detailing of all components of Elevated Water Tank