Ductility Analysis of R.C.C.(O.H.T)desigh As per IS 3370(PART II)

Submitted in Partial Fulfilment for the Award of the Degree of

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CERTIFICATE

*This is to certify that the project entitled "**Ductility analysis of R.C.C.(O.H.T.)as per IS 3370PART(11) tank**" being submitted by me, is a bonafide record of my own work carried by me under the guidance and supervision of Assist. Prof. **G.P.Awadhiya**. in partial fulfillment of requirements for the award of the Degree of Master of Engineering (Structural Engineering) in Civil Engineering, from University of Delhi, Delhi.*

The matter embodied in this project has not been submitted for the award of any other degree.

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ABSTRACT

Large capacity elevated intze tanks are used to store a variety of liquids, e.g. water for drinking and fire fighting, petroleum, chemicals, and liquefied natural gas. The liquid storage tanks are particularly subjected to the risk of damage due to earthquake-induced vibrations. A large number of overhead water tanks damaged during past earthquake. Majority of them were shaft staging while a few were on frame staging. Recently the Muzaffarabad earthquake 2005 and Bhuj earthquake 2001 also represented similar damage. Most of the damage was caused because of the tanks were either designed without considering the earthquake forces or inadequate seismic design considerations. To cope with this need the seismic design codes for over head water tanks have been revised and upgraded.

The objective of this dissertation is to shed light on the difference in the design parameters of (a) over head water tanks without having earthquake forces, (b) over head water tanks constructed with a consideration of earthquake forces following two approaches; firstly based on Indian standard code 1893, (1984) i.e. adopting single mass method and second is based on draft code 1893-Part 2, (2005) considering two mass modal i.e. convective and impulsive mode method.

Two types of elevated water tanks namely intze tank supported by frame staging and shaft staging have been considered in this study. These elevated water tanks are first conventionally designed and then seismic analyzed and again redesigned considering earthquake forces. Their strength and ductility have also been evaluated and compared.

It has been observed that time period in frame staging is higher then the shaft staging since the lateral stiffness of shaft staging is much larger. The tank supported on shaft staging has higher strength as compare to tank supported on frame staging but the ductility is low that may be the return of frequent failure of elevated water tank supported on shaft staging.

ABSTRACT

Large capacity elevated intz tanks are used to store water for drinking and fire fighting. The liquid storage tanks are particularly subjected to the risk of damage due to earthquake-induced vibrations. A large number of overhead water tanks damaged during past earthquake. Majority of them were shaft staging while a few were on frame staging. Recently the Muzaffarabad earthquake 2005 and Bhuj earthquake 2001 also represented similar damage. Most of the damage was caused because of the tanks were either designed without considering the earthquake forces or inadequate seismic design considerations. To cope with this need the seismic design codes for over head water tanks have been revised and upgraded.

The objective of this dissertation is to shed light on the difference in the design parameters of (a) over head water tanks (b) over head water tanks designd with earthquake forces, firstly based on Indian standard code 1893, (2002) i.e. adopting lumped mass modal method and second is based on draft code 1893-Part 2, (2005) considering two mass modal i.e. convective and impulsive mode method.

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In this dissertation work of 3D frame is being modeled in SAP 2000 VER. 11 (Advanced) & analyzed . The non-linear static procedure or simply push over analysis is a simple option for estimating the strength capacity in the post-elastic range. This procedure involves applying a predefined lateral load pattern which is distributed along the structure height.

For evaluation of strength and ductility of frame staging, only staging portion has modeled in SAP. Here container portion is quite rigid there fore rigid link is assumed from top of staging to make rigidity. Mass of the rigid link has taken zero and stiffness is quite high compare to the other member of the staging.

CHAPTER 1

NTRODUCTION

1.1 GENERAL

Water tanks are very important components of lifeline. They are critical elements in municipal water supply, fire fighting systems and in many industrial facilities for storage of water. The liquid storage tanks are particularly subjected to the risk of damage due to earthquake-induced vibrations. A large number of overhead water tanks damaged during past earthquake. Majority of them were shaft staging while a few were on frame staging type. Muzaffarabad earthquakes (2005) Kuch and Bhuj earthquakes (2001) are the recent example, as shown in figure 1.1 and 1.2. It is observed from the past earthquake; most of the elevated water tanks undergo damage to their staging.

14Two alternative configurations of staging of intze tank i.e. frame staging and shaft staging has been studied. These tanks are designed by with and without earthquake forces. The seismic analysis of these tanks has been carried out by two different methods; firstly based on Indian standard code 1893,(2002) i.e. adopting lumped mass modal

Figure 1.1: Collapse of water tank in Bhuj

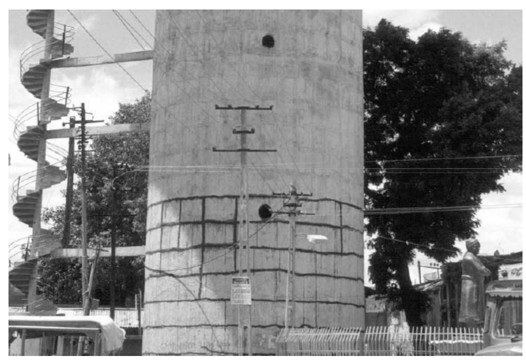


Figure1.2: Flexure cracks in staging

Figure 1.3: Crack in soffit concrete under side the staging beam of tank

Figure 1.4: Crack in soffit concrete along the entire length of a staging

method and second is based on draft code 1893-Part 2,(2005) considering two mass modal (convective and impulsive mode) method. In the present study has also been focus on the evaluation of strength and ductility of different staging namely frame staging and shaft staging with and without considering earthquake forces. Commercial software package SAP2000 has been used for modeling and evaluation the ductility of over head water tank supported on frame and shaft staging.

1.2 LITERATURE REVIEW

1.2.1 LITERATURE CITED

Housner, (1963) proposed the values for equivalent masses with their locations to represent the forces and moments exerted by the liquid on the tank. He says that an analysis of the dynamic behavior of such tanks must take into account the motion of the water relative to the tank as well as the motion of the tank relative to the ground. Some simple expressions are given for the pertinent dynamic property of tanks with free water surface. A simplified dynamic analysis is indicated for the response of elevated water tanks to earthquake ground motion.

Rosman, (1992), analyzed the effect of single lateral load at the top of staging, so that the cantilever shear force is constant along the height and cantilever bending moment varies linearly. He analyzed the effect of lateral load at all ring girders, so that cantilever shear force diagram is step like and the cantilever bending moment diagram is polygonal. He has applied technical flexure theory and obtained result. When analyze bending of the column, the author consider only shear force in the frame tangential planes.

Sameer et al (1994), presents an improved procedure for lateral-load analysis of polygonal braced staging with columns fixed at foundation level. The axial force in the columns is obtained assuming that it is proportional to the distance of the column from the bending axis of the staging. The shear in bracing is then obtained by considering vertical equilibrium of structural units isolated from the staging. Finally, the shear in columns is obtained from moment equilibrium in the plane of bending. The point of inflection is assumed at mid spans of columns in the intermediate panels and bracing girders. A simple formula has also been developed to locate point of inflection in the top and bottom panels more accurately. He has used some trigonometric identities for reduces the design forces in columns and bracings to simple closed forms. Influence of the direction of lateral force on the column and brace forces has also been discussed, and this aspect is duly considered in deriving the expressions, four practical tank staging have been analyzed by the finite-element method to evaluate the accuracy of results obtained by the proposed method.

Dutta, (1995) studied torsional response of RC elevated water tanks supported on axisymmetric frame-type staging. He studied that Elevated water tanks have failed during past earthquakes owing to large torsional response. Considerable torsional response may occur due to accidental eccentricity if the uncoupled torsional and lateral natural periods of the tanks are closely spaced.

Malhotra et al., (2000) provides the theoretical background of a simplified seismic design procedure for cylindrical ground-supported tanks. Seismic responses, base shear, overturning moment, and sloshing wave height are calculated by using the site response spectra and performing a few simple calculations.

Shenton et al., (1999) presents the results of an analytical investigation of the seismic response of isolated elevated water tanks. A discrete three-degree-of-freedom model of the isolated structure is presented that includes the isolation system, tower structure, and sloshing fluid.

Dutta et al., (1996) summarize a recent study on torsional response of R.C. elevated water tank supported on axisymmetric moment resisting frame type staging. They describe response for magnified torsional response in such tank. Four alternate staging configuration are indicated which can be use to achieve this. In general, elevated tanks supported on more number of column and with large number of panels under go less torsional response than those on fewer column and panels. Staging design to respond inelastically under seismic condition (i.e., large ductility reduction factor used in design).

Rashad at al, (1993) has been proposed a displacement and ductility based design procedure for reinforced concrete frames which explicitly considers strength, stiffness, ductility, and structural configuration. The procedure satisfies the design requirements ate two limit states; serviceability limit state (SLS) and ultimate limit state (ULS).

Beeby, (2004) illustrates the importance of ductility in reinforcement design for the ultimate load. As a consequence, the benefits of specifying reinforcement with a higher ductility are unquantifiable and less like to be considered, although they clearly exist in terms of risk reduction. As a consequence, the greater ductility of material used, the greater confidence in the over all performance of the structure under all situation, whether foreseen or unforeseen.

1.2.2 CODES REFERRED

IS: 1893(2002) Criteria for Earthquake Resistant Design of structures, in this code lumped mass modal of water tank has illustrate for analysis of water tank.

- **IS: 1893(Part II)(2005)** Draft Criteria for Earthquake Resistant Design of Structure (Liquid Retaining Tanks), in this draft two mass modal is illustrate for analysis of liquid storage tank.
- **IS: 3370(part II-IV)-1965** Code of practice for concrete structures for the storage of liquids, in this code general requirement and stress for design of liquid storage tank is illustrated.
- **IS: 11682-1985** Criteria for design of RCC staging for over head water tanks, in this code analysis and design for both type of staging frame staging and shaft staging has illustrate.
- **IS: 456-2000** plain and reinforced concrete code of practice, in this code all design parameter for RCC design of different component of elevated water tank.

1.3 OBJECTIVES

- Conventional design of elevated water storage tank supported on frame staging as well as shaft staging tank without considering earthquake.
- Seismic analysis of elevated water storage tank considering lumped mass modal and two mass modal as per IS:(1984) and draft code IS 1893 Part II (2005) respectively
- Redesign of elevated water storage tank supported on frame staging as well as shaft staging by considering earthquake forces as per IS: 1893(2002) Part II.
- Evaluation of strength and ductility of frame staging as well as shaft staging of elevated water storage intze tank using SAP2000.
- Capacity and demand curve of Staging of elevated storage tank

1.4 ORGANIZATION OF THE THESIS WORK

To get the above objectives whole report is divided into six chapters. Chapter 1 introduces the importance of the water tank, performance of the water tank during past earthquake. The past work that's had been carried out by researchers also explained and stipulate the object for this study. Chapter 2 describes the conventional design of elevated water storage tank for different type of staging like frame staging and shaft staging using Indian slandered IS: 3370-Part II(1965), IS: 11682-1985 and IS: 456(2000) without considering earthquake forces. In chapter 3 the seismic analysis of water tanks has been discussed using following two approaches; lumped mass modal and two mass modal. Lumped mass approach has been considered only impulsive case while two mass modal has consider both impulsive and convective mode. In Chapter 4 the water tank has to be designed considering earthquake forces those are calculated in chapter 4 using seismic Indian standard code IS: 1893(2002) and IS:1893 Part 2(draft 2005). Chapter 5 describes strength and ductility evaluation of frame staging and shaft staging by using software package SAP2000 , plotting of capacity and demand curve. Finally in the last chapter result discussion and conclusion have been drawn from the whole study of this dissertation work.

Chapter 2

DESIGN OF ELEVATED WATER TANKS

1 GENERAL

In general, water tanks can be classified under three heads: (i) tank resting on ground (ii) elevated tank (iii) under ground tank. This study has emphases merely on elevated intze tank. Design of liquid retaining structure has to be based on the avoidance of cracking in the concrete having regard to its tensile strength. It has to be ensured that no cracks in the concrete should be formed on the water face.

2.2 GENERAL DESIGN REQUIREMENTS

(Indian standard code practice (IS: 3370-PART II-IV)

Plain concrete member of reinforced concrete liquid structure may be designed against structure failure by allowing tension in plain concrete as per the permissible limit for tension in bending specified in IS:456 (permissible stress in tension in bending may be taken to the same as permissible stress in shear). This will automatically take care of failure due to cracking.

2.2.1 PERMISSIBLE STRESS IN CONCRETE

(a) For resistance to cracking: The permissible tensile stresses due to bending apply to the face of the member in contact with liquid. The member with thickness less than 225 mm and contact with the liquid on one side, these permissible stresses in bending apply also to the face remote from the liquid.

(b) For strength calculation: In strength calculation the usual permissible stress, in accordance with IS: 456-2000, is used. Where the calculated shear stresses in concrete above exceed the permissible value, reinforcement acting in conjunction with diagonal compression in concrete shall be provided to take the whole of the shear.

2.2.2 PERMISSIBLE STRESS IN STEEL

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

(b) For strength calculation: In strength calculation the permissible stress in steel, in accordance IS: 3370 are used.

When water is filled in tank container, the hydrostatic pressure will try to increase the diameter at any section of the tank. However, this increase in the diameter in all along the height of the tank will depend upon the nature of the joints. If the joint is flexible, it will be free to move outward and when the joint is fixed, no movement is possible, then a fixing moment will be induced.

2.3 ANALYSIS AND DESIGN PAREMETERS OF INTZE TANK

In past most of the circular water tanks with flat bottom slab were used because they were easy to construct. In the flat bottom slabs, the thickness and reinforcement is heavy so that they are economical. In case of large diameter tanks and economical alternative would be to reduce its diameter at its bottom by conical dome such tank is known as Intze tank.

2.3.1 PARAMETERS OF CONTAINER

The capacity of Intze type water tank container can be expressed in term of the volume of the water in cylindrical portion and conical portion as given bellow

$$V= (\pi/4)D^2h +(\pi h_0/12)(D^2 + D_0^2 +DD_0) - (\pi h_2^2/3)(3R_2 - h_2) \qquad \text{..... (2.1)}$$

Where:

D = Diameter of cylindrical portion

h = Height of the cylindrical portion which is calculated by above formula or thumb rule is two third of the diameter

h_0 = Height of the conical dome $\approx 3\times D/16$

D_0 = Diameter of the staging $\approx 5\times D/8$

h_1 = Rise of top spherical dome $\approx 1\times D/8$

h_2 = Rise of bottom spherical dome $\approx 1\times D/8$

R_1 = Diameter of the staging $\approx 5\times D/8$

R_1 = Height of the conical dome $\approx 3\times D/16$

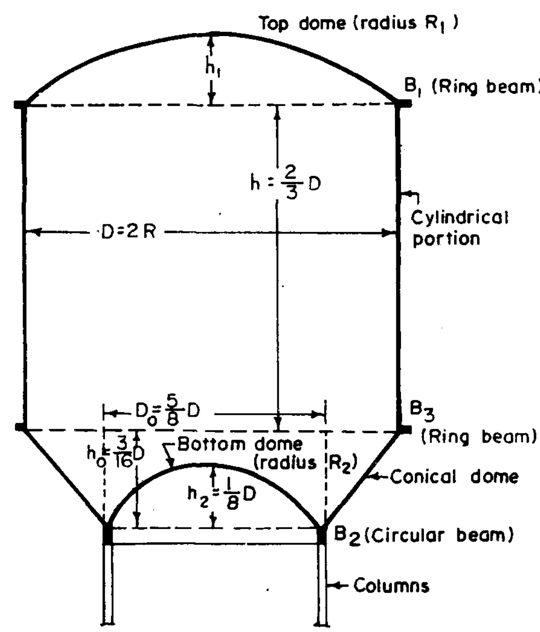


Figure 2.1 Container parameters

2.3.2 TOP SPHERICAL DOME

The top dome is subjected to live load and dead load. The dome is accessible only for maintenance purpose and hence the live load can be taken as 750 N/m^2 as per IS: 875-1964. It is supported by the top ring beam and the cylindrical shell. The rise of top dome (h_1) is kept 0.125 times the diameter of the container (D).

The radius of dome is given by

$$R_1 = (0.25D^2 + h_1^2)/ 2h_1 \quad \text{..... (2.2)}$$

The maximum stresses occur at the edge of the dome. The membrane stress resultants in the dome are

Meridian thrust at edge

$$T1 = P\times R_1/ (1+\cos\Phi_1) \quad \text{..... (2.3)}$$

Maximum hoop stress at centre is

$$= (P\times R_1/t) \times [\cos\Phi_1 -1/(1+\cos\Phi_1)] \quad \text{..... (2.4)}$$

Where: P = Total load = (Live load + Dead load)

$$\Phi_1 = \tan^{-1} [D/2(R_1 - h_1))] \quad \text{..... (2.5)}$$

(According to IS: 3370 maximum hoop stress should be less then 1.2 N/mm^2)

The thickness of the dome is primarily controlled by practical consideration since the stresses are of compressive nature and thickness is small. The thickness normally provided 100mm

The dome is reinforced by nominal reinforcement of 0.24% in each direction in case of tore steel and o.3% in case of mild steel in the form of square mesh placed at top near the springing and brought to the bottom gradually near the crown.

2.3.3 TOP RING BEAM

The top ring beam is providing to resist the horizontal component of the meridional thrust of top dome. The centre line of ring beam is aligned with the centre line of top dome. The hoop tension is given by

$$P_1 = T_1\times\cos\Phi\times D/2 \quad \text{..... (2.6)}$$

Top ring beam also design on no crack basis so as to avoid possible corrosion of the reinforcement and also to provide good stiffness to support the roof dome. The area of cross section of the ring beam is give by

$$A = 1.2/P_1 - (m -1) A_{st} \quad \text{..... (2.7)}$$

$$A_{st} = P_1/\sigma_{st} \quad \text{..... (2.8)}$$

σ_{st} = allowable tensile stress in steel = 150 N/mm^2

2.3.4 CYLINDRICAL WALL

The walls of the tank are assumed to be free both at top and bottom. Due to this, the tank wall will be subjected to hoop tension only, without any B.M. The maximum hoop tension will occur at base and its magnitude it's given by

$P_h = w.h.D/2$ ….. (2.9)

To prevent the rupture of wall due to this hoop tension adequate reinforcement with horizontal ring provided on both face.

$A_{st} = P_h/\sigma_{st}$ ….. (2.10)

In addition to this, vertical reinforcement is provided on the both face in the form of distribution reinforcement.

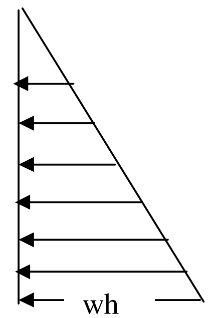


Figure 2.2 Pressure distributions on wall

2.3.5 MIDDLE RING BEAM (B_3)

The purpose of the middle ring beam is to provide a horizontal support to the conical dome. It is provide wide enough so as to function as a balcony for inspection purpose. The design of the beam is governed by the hoop tension caused by the horizontal component of the meridional thrust in conical dome.

The load W transmitted through tank wall, at the top of conical dome per unit length is

W_1 = Load due to top dome = T_1 x $\sin\Phi$

W_2 = Load due to ring beam

W_3 = Load due to tank wall

W_4 = Self weight of ring beam

Total weight $W = W_1 + W_2 + W_3 + W_4$

Horizontal component of dead load of upper structure is given by

$P_W = W.\tan\Phi_0$ ….. (2.11)

Horizontal force due to water pressure is given b

$P_w = w.h.d_3$ ….. (2.12)

Where d_3 = depth of ring beam

Hoop tension trying to burst the ring beam (B_3) is given by

$P_3 = (P_W + P_w).D/2$ ….. (2.13)

Area of reinforcement is given by

$A_{st} = P_3/\sigma_{st}$ ….. (2.14)

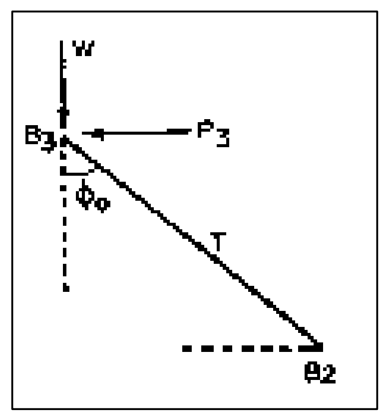


Figure 2.3 Forces in middle ring beam

2.3.6 CONICAL DOME

Conical dome is subjected to both meridional thrust as well as hoop tension.

a) <u>Meridional thrust:</u> The meridional thrust in the conical dome is due to vertical force transferred to it at base.

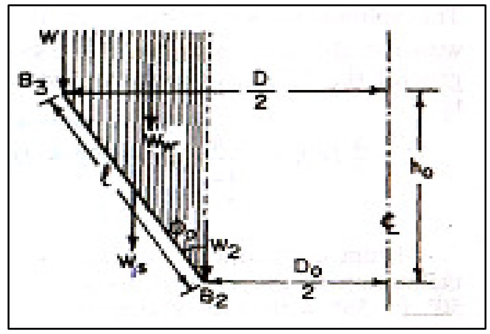


Figure 2.4 Meridional thrust in conical dome

The total load consist of

i) Weight of top dome, Top ring beam (B_1), cylindrical wall and middle ring beam (B_3) = W

ii) Weight of water

$$W_w = [\pi/4(D^2 - D_o^2)\times h\times w + [(\pi/12)\times h_0\times w\times (D^2 + D_o^2 + DDo)] - (\pi/4)\times D_o^2\times ho\times w] \quad \text{.... (2.15)}$$

iii) Self weight

$$Ws = \left(\pi\frac{(D + D_0).l.t_0}{2}\right)\gamma_c \quad \text{..... (2.16)}$$

Total vertical load per meter length is given by

$$W_2 = (\pi DW + W_w + W_s)/\pi D_0 \quad \text{..... (2.17)}$$

Meridional thrust T_0 in the conical dome is given by

$$T_0 = W_2 / \cos\Phi_0 \quad \text{..... (2.18)}$$

b) <u>Hoop tension:</u> Due to water pressure and self weight the conical dome will be subjected to hoop tension. Let p' be the water pressure at any height above the base of conical dome and let D' be the diameter of the conical dome at that depth. The water pressure p will act normal to the inclined slab surface. Let q be the weight of the conical slab per square meter of the surface area.

Then the hoop tension P_0' at height h' above the base is given by

$$Po' = (P/\cos\Phi_0 + q.\tan\Phi_0)\, D'/2 \quad \text{..... (2.19)}$$

Where:

$$D' = D_0 + \frac{(D + D_0)}{2}\times h' \quad \text{..... (2.20)}$$

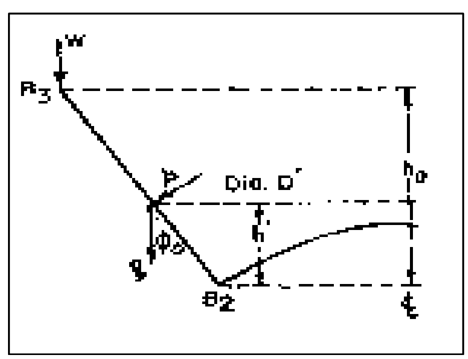


Figure 2.5 Hoop tension in conical dome

With the help of above formula, hoop tension at the top, middle and base of the conical dome is calculated. For bear this hoop tension steel reinforcement has provided.

$$A_{st} = P_0'/\sigma_{st} \quad \text{..... (2.21)}$$

2.3.7 BOTTOM DOME

The bottom dome is of spherical shape and its developing compressive stress both meridional as well as along hoop, due to weight water overt it and self weight. It is supported by a ring beam. The radius of curvature of the dome is

$R_2 = (0.25D_0^2 + h_2^2)/ 2h_2$

Let H_0 be the total depth of water above the edge of the dome. The weight of water above surface of dome is given by

$W_0 = [(\pi/4) \times D_o^2 \times H_o - (\pi/3) \times h_2^2 (3R_2 - h_2)] \times w$ ….. (2.22)

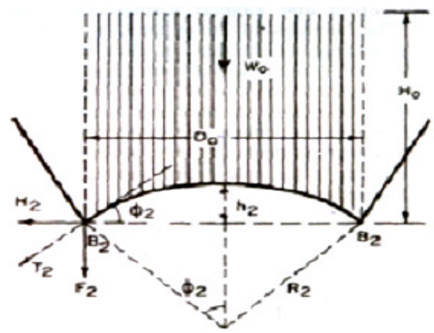


Figure 2.6 Forces on bottom dome

Self weight of dome

$Ws = 2\pi R_2 \times h_2 \times t_2 \times 25$ ….. (2.23)

Total load

$W_t = W_0 + 2\pi R_2 \times h_2 \times t_2 \times 25$ ….. (2.24)

Meridional thrust

$T_2 = W_t / \pi \times D_o \times \sin \Phi_2$ ….. (2.25)

Intensity of load

$P_2 = W_t / 2\pi R_2 h_2$ ….. (2.26)

Maximum hoop stress at centre = $P_2R_2/ 2t_o$

2.3.8 BOTTOM RING BEAM (B2)

The bottom ring beam is considered as a part of the staging because its design is governed by the number of column in the staging. The bottom ring beam subjected to the total vertical load of the container and the water including self weight in the vertical direction. In addition it's also subjected to radial force which is difference of the horizontal components of both of meridional thrust from the conical dome (T_0) and the bottom dome (T_2).

Net horizontal force P is given by

$P = T_0 \sin\Phi_0 - T_2 \cos\Phi_2$ ….. (2.27)

And net vertical force is given by

$W = T_0 \cos\Phi_0 + T_2 \sin\Phi_2$ ….. (2.28)

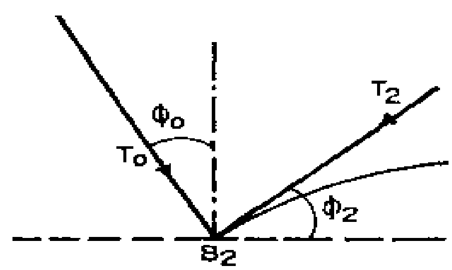


Figure 2.7 Forces in bottom ring beam

If $T_0 \sin\Phi_0 > T_2 \cos\Phi_2$ the beam will be subjected to hoop compression. If however

$T_0 \sin\Phi_0 < T_2 \cos\Phi_2$, it will be subjected to hoop tension that's why dimension of the tank should be so adjusted that p is zero or compressive.

The hoop force is given by

$$P_h = P \times D_0/2 \qquad \text{..... (2.29)}$$

The bottom ring beam is analyzed as a beam curved in plan and continuous over column support. The design is based on the recommendations of the working stress method as per IS: 456-2002

Design forces

Maximum hogging moment at support section is given by

$$M_s = C_1 W R^2 (2\theta) \qquad \text{..... (2.30)}$$

Maximum sagging moment at mid span is given by

$$M_c = C_2 W R^2 (2\theta) \qquad \text{..... (2.31)}$$

Maximum torsional moment is given by

$$M_m^t = C_3 W R^2 (2\theta) \qquad \text{..... (2.32)}$$

C_1 C_2 and C_3 are constant which are depending on no. of column.

2.3.9 COLUMNS

The columns are to be design for vertical load due to the weight of the container, the weight of the water and the self weight as well as horizontal forces due to wind or earthquakes. Generally columns are providing the same cross-sectional area and are placed symmetrically. Therefore vertical loads share equally by all the columns.

The lateral force induced bending moment and shear forces and axial forces in the column. The magnitude of these reactions depends upon the condition of the end fixity of the column at the top and bottom. This problem is statically indeterminate and involves the analysis of the tower as a space frame.

The analysis for horizontal load is done by the equivalent tube analogy method. In this method the tower is assumed to behave as a cantilever under the action of horizontal forces with the neutral axis passing threw the bending axis of the tower. The tower built monolithically with the container base at top, braces in middle and the foundation at the bottom are consider infinitely rigid and there fore, the rotation of columns at these point is consider to be zero. If the bending moment and shear forces due to the lateral load on the tower are calculated on this equivalent vertical cantilever beam at the horizontal section passing threw points of inflexion, then the bending stress in the equivalent vertical cantilever beam will give the vertical forces in the column and the shear stress in the cantilever beam will give the horizontal shear force in the column at their point of inflexion. The detailed formulation has been explained by (1) Jai Krishana and Jain (2001), (2) B.C. Punmia A. K. Jain and Jain (1998).

2.3.10 BRACINGS

Bending moment in a brace is developed at the junction due to shear force acting at the mid height of columns. The bending moment has to be resisted by the two bracing two braces meeting at the junction. The bending moment in the brace about the vertical axis of its section is almost zero and even the twisting moment is negligible. Thus bracing bear the joint moment mostly by the developing bending moment about horizontal axis of its section. These moments in the bracing can easily be calculated by considering the static equilibrium of moments at the joint.

Moment in the bracing due to lateral load:

$$m_1 = \frac{(Q_{w1}.h_1 + Q_{w2}.h_2)\sin(\theta + \pi/n)}{n\sin(2\pi/n)}\cos^2\theta \qquad \text{..... (2.33)}$$

$$m_2 = \frac{(Q_{w1}.h_1 + Q_{w2}.h_2)\sin(\theta - \pi/n)}{n\sin(2\pi/n)}\cos^2\theta$$

Maximum shear force in bracing due to lateral load:

$$(Sb)max = \frac{(Q_{w1}.h_1 + Q_{w2}.h_2)\, 2\cos^2}{L.n.\sin(2\pi/n)}(\pi/n)\sin(2\pi/n) \qquad \text{..... (2.34)}$$

2.3.11 SHAFT STAGING

It is a tower in the form of shaft is called shaft staging. The area enclosed with the shaft may be used for providing stairs, pipes, electrical control panels etc. The shaft staging is to be design for vertical load due to the weight of the container, the weight of the water and the self weight as well as horizontal forces due to wind or earthquakes.

The minimum thickness of concrete shell for staging shall be 150mm. When internal diameter is more than 6m, the minimum thickness in mm shall be

$$t = 150 + \frac{(D_0 - 6000)}{120} \qquad \text{..... (2.35)}$$

When e/r ratio is less than 0.5 the whole section is under compression. Vertical stress in circular shaft is given by (without considering lateral load)

$$\sigma_{cv} = \frac{W}{2\pi rt}(1 + 2e/r) \qquad \text{..... (2.36)}$$

Vertical compressive stress with considering lateral load is

$$\sigma_{cv} = \sigma_{cv}'\left(1 + \frac{t}{2r\cos\beta(\cos\beta - \cos\alpha)}\right) \qquad \text{..... (2.37)}$$

Where

$$\sigma_{cv}' = \frac{W}{2rt}\left(\frac{(\cos\beta - \cos\alpha)}{(1 - p)(\sin\alpha - \alpha\cos\alpha) - (1 - p + mp)(\sin\beta - \beta\cos\alpha) - mp\pi\cos\alpha}\right) \quad ......(2.38)$$

Where

m = modular ratio

p = total area of vertical reinforcement to total area of concrete shell at section under consideration

e = eccentricity

r = radius of the shaft

2.4 ANALYSIS AND DESIGN CONTAINER PORTION OF INTZE TANK FOR CAPACITY (1000KL)

2.4.1 DIMENSIONS OF CONTAINER PORTION OF INTZE TANK

Diameter of cylindrical portion D = 14m

Diameter of bottom ring beam D_0 = 10m

Inclination of conical wall with horizontal is = 45 degree

Rise h_1= 1.75m h_2= 1.75

Radius of top dome R_1 = 14.875m

Radius of bottom dome R_2 = 8.02m

Height of the cylindrical portion h = 5.6m

Height of the staging= 16m

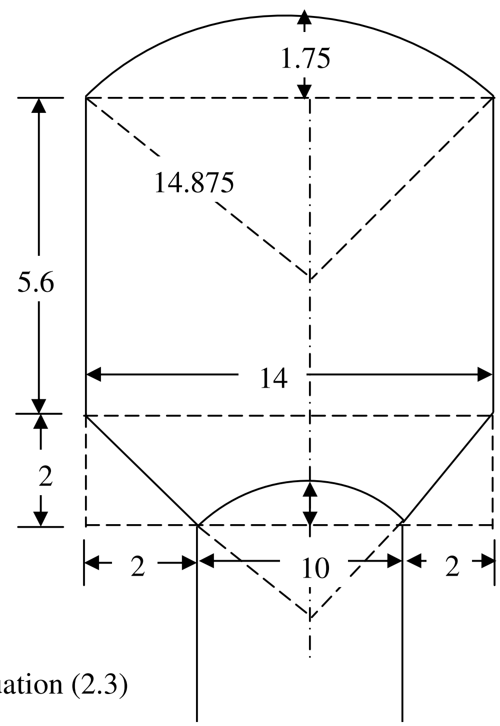


Figure 2.8 Dimensions of tank container

2.4.2 DESIGN OF TOP DOME

R_1 = 14.875m $\sin\Phi_1$ = 0.47 $\cos\Phi_1$ = 0.882

Let thickness 100mm and live load 1.5KN/m^2

Total load P = LL + DL = 4.0 KN/m^2

Meridian thrust at edge

T_1 = 31.615 kN/m   ….from equation (2.3)

Meridian stress =

T_1/t => 0.316 N/mm^2 < 1.2 N/mm^2 (safe)

Maximum hoop stress at centre is

= 0.297 N/ mm^2 ….from equation (2.4)

Allowable stress is less then the permissible limit (1.2 N/mm2) hence it's safe. Since stresses are within limit, hence provide nominal reinforcement @ 0.3%.

Ast = 300 mm^2

Hence provide 8 mm Φ bars @ 160 mm c/c.

2.4.3 DESIGN OF TOP RING BEAM B_1

Vertical component of meridian thrust is = $T_1 \sin\Phi_1$ = 14859.19 N/m

Horizontal component of meridian thrust is = $T_1 \cos\Phi_1$ = 27884.69 N/m

Total tension tending to rupture the beam is P_1 = 195192.83 N …from equation (2.6)

Permissible stress in HYSD bar σ_{st} = 150 N/ mm^2

A_{sh} = 1302.28 mm^2 …from equation (2.8)

Hence use 4 bars of 22mm Φ A_{sh} = 1519.76 mm^2

Area of cross section of ring beam is given by

A = 144423.57 mm^2 ....from equation (2.7)

Provide ring beam of 370mm depth and 400mm width. Tie the 22mm Φ rings by 8mm diameter nominal stirrups @ 200mm c/c spacing.

2.4.4 DESIGN OF CYLINDRICAL WALL

Maximum hoop tension occur at the base of the wall and its magnitude is given by

P_h = 9800×5.6×14/2 = 3814160 N/m height ….from equation (2.9)

Area of steel A_{sh} = 3814160/150 = 2561.06 mm^2 per meter height

Hence provide 12mm Φ rings @ 85mm c/c on both faces.

Actual A_{sh} = 100×113/85 = 1329.42 mm^2

Permissible stress in composite section is 1.2 N/ mm^2 then thickness of cylindrical wall is given by

384160 / (1000×t + 12×1329.41×2) <= 1.2 …from equation (2.7)

t = 288.22mm

Hence provide 300mm thickness and corresponding to this thickness percentage distribution steel is 0 0.243%. Hence A_{sh} = 728.57mm^2

Hence provide 8mm Φ bars @ 135mm c/c on each face.

2.4.5 DESIGN OF MIDDLE RING BEAM (B_3)

This beam connects the tank wall with conical dome. The vertical load at the junction of the wall with conical dome is transferred to ring beam by meridian thrust in the conical dome. The horizontal component of thrust cause hoop tension at the junction.

The load W transmitted through tank wall at the top of conical dome:

i) Load due to top dome = 31616.30×0.470 = 14859.19N/m

ii) Load due to ring beam = 25000×0.37×(0.4 – 0.3) = 925N/m

iii) Load due to tank wall = 25000×0.3 5.6 = 42000N/m

iv) Self weight of ring beam (assume initially size of ring beam is 1m x 0.6m)

$= 25000 \times 0.6\ (1 - 0.3) = 10500$N/m

Total load W =68284.19 N/m

Inclination of conical dome with vertical is $\Phi_0 = 45$ degree

$\sin\Phi_0 = \cos\Phi_0 = 0.707$ $\tan\Phi_0 = 1$

Force due to self weight of upper structure-

$P_W = 68384.19$ N/m …from equation (2.11)

Pressure due to water

$P_w = 32928$ N/m …from equation (2.12)

Hence hoop tension on the ring beam

$P_3 = 708485.33$ N …from equation (2.13)

This hoop tension resisted by the steel hoops, then area of hoops is

$A_{sh} = P_3 / \sigma_{st} = 4723.23\ mm^2$

Hence provide 8 bars of 28mm Φ then actual $A_{sh} = 4948mm^2$

Stress in equivalent steel = $P_h / (A + (m - 1) A_{sh})$

$= 1.07 < 1.2$ Hence safe

8mm distribution bar (vertical) provide in the ring beam @ 150mm c/c.

2.4.6 DESIGN CONICAL DOME

(a) Meridian thrust:

The weight of the water is

$W_w = 4835353.17$ N …from equation (2.15)

Initially we assume thickness of the conical dome is 400mm then self weight is

Ws = 1066131 N …from equation (2.16)

Weight W at ring beam B_3 due to upper element like top dome, top ring beam, cylindrical wall and self weight itself is = 68284.19 N/m

Hence vertical load W_2 per meter run is given by

$W_2 = [(\pi \times 14 \times 68284.19) + 4835353.17 + 1066131] / \pi \times 10$

$= 283543.22$ N/m

Meridian thrust T_0 in the conical dome is

$T_0 = 400990.67$ N/m

Meridian stress = $T_0 / t = 1.0 < 1.2$ Hence safe

(b) Hoop tension

Diameter of conical dome at any height h' above the base is

$D' = 10 + 2h'$

Intensity of water pressure

$P = 9800\times(5.6 + 2 - h') = 9800\times(7.6 - h')\ N / mm^2$

Self weight $q = 0.4\times1\times1\times25000 = 10000$ N/mm2

Hence hoop tension

$P_0' = [9800(7.6 - h')\times\sqrt{2} + 10000x1](10 + 2h')/2$ …from equation (2.19)

$dP_0'/dh' = 46034.18 - 27718.6h = 0$

$h' = 1.66$ m

Hence at h'=1.66m have the maximum hoop tension

Maximum $P_0' = 614879.45$ N

(c) Design of wall

Maximum hoop stress $P_0 = 614879.45$ N

Area of steel $A_{sh} = 4099.196mm^2$

Hence provide 16mm Φ bar @ 95mm c/c on each face.

Actual $A_{st} = 1000x201/95 = 2115.78\ mm^2$

Maximum tensile stress of composite section is

$= 614879.45 / (1000\times400 + 12\times2\times2115.78)$ ....from equation (2.7)

$= 1.36 > 1.2\ N/mm^2$ (Unsafe)

This is more then the permissible limit but its slightly adjustment will be made after considering the effect of continuity.

2.4.7 DESIGN OF BOTTOM DOME

Radius of bottom dome $R_2 = 8.02$ m

Height of the bottom dome $h_2 = 1.75$ m

$Sin\Phi_2 = 0.62 \quad cos\Phi_2 = 0.78$

Weight of the water on the dome is

$W_0 = 5145854.1$ N …from equation (2.22)

Initially we take thickness of the bottom dome is 250mm

Then self weight of bottom dome is

$Ws = 550873.75$ N …from equation (2.23)

Total weight of bottom dome is

$W_t = 5696727.85$ N

Meridian thrust is

$T_2 = 292620.0$ N/m …from equation (2.25)

Meridian stress

$T_2 / t = 1.17 < 1.2$ Hence safe

Intensity of load per unit area

$P_2 = 64632.86$ N/m2 …from equation (2.26)

Maximum hoop stress at centre of the dome is

$= P_2R_2/ 2t_o = 1.03 < 1.2$ Hence safe

So that provides minimum reinforcement for 250 mm thickness dome is 0.26%

$A_s = 650mm^2$ in each direction

Hence provide 10mm Φ bars @ 120mm c/c in both directions.

2.5 ANALYSIS AND DESIGN STAGING PORTION

2.5.1 FRAME STAGING

2.5.1.1 Design of bottom circular ring beam b_2

Net horizontal force is in ring beam is

P = 55299.6 N/m …from equation (2.27)

Hoop compression in beam = P×D/2 = 276498 N

Initially we assume size of the is 600mm×1000mm

Hoop stress = 0.46 N/mm2

Vertical force due to conical dome and spherical dome on the bottom circular ring beam, per meter run is = 464967.6 N/m …from equation (2.28)

Self weight = 15000.0 N/m

Then total vertical load on the beam W = 479967.6 N/m

The beam is supported on 12 equally space column at a mean diameter of 10m mean radius of curved beam will be R = 5m, $2\theta = 30°$, $\theta = 15°$

Value of different coefficient has taken from the IS: 3370

$C_1 = 0.045$ $C_2 = 0.017$ $C_3 = 0.002$ $\Phi_m = 6.25°$

$WR^2 (2\theta) = 6282761.19$ N-m

Maximum –ve BM at support

$M_s = C_1WR^2 (2\theta) = 282724.25$ N-m

Maximum +ve BM at mid span

$M_c = C_2WR^2 (2\theta) = 106806.94$ N-m

Maximum torsional moment

$M_m^t = C_3WR^2 (2\theta) = 12565.52$ N-m

Using M-20 concrete (σ_{cbc} = 7 N/mm2) and HYSD bar σ_{st} = 150 N/mm2

K = 0.38 J = 0.87 R = 1.157

Required effective depth = $\sqrt{M/R\times b}$ = 699.08mm

However we take total depth = 900mm, and width = 500mm

Maximum shear force at support

$F_0 = WR\theta = 628276.12N$

Shear force at any point is given by

$F_0 = WR\ (\theta - \Phi)$

Shear force at maximum torsion moment $\Phi = \Phi_m$

F = 366494.40N

Torsional moment at any point is given by

$M_\Phi{}^t = WR^2\ [\theta\cos\Phi - \theta\cot\theta\ \sin\Phi - (\theta - \Phi)]$

At support $\Phi = 0$ $M_\Phi{}^t = 0$

At mid span $\Phi = \theta = 15$ $M_\Phi{}^t = 0$

BM at point of maximum torsional moment $\Phi = \Phi_m = 6.25$

$M_\Phi{}^t = -3103.35$ N-m = 3103.35 N-m (Hogging)

Hence we take the following combination BM and torsional moment

At the support

M_0 = 282724.25 N-m hogging $M_t = 0$

$M_e = M_0 + M_t$ = 282724.25 N-m

At the mid span

M_0 = 106806.9 N-m $M_t = 0$

$M_e = M_0 + M_t$ = 106806.9 N-m sagging

At the point of maximum torsion

M_0 = 3103.35 N-m M_t = 12565.52 N-m

$M_T = M_t\ [(1+D/b)/1.7]$ = 20696.15 N-m

M_e = 23799.50 N-m

Main longitudinal reinforcement bar

i) Maximum B.M. at support = 282724.25 N-m

A_{st} = 2537 mm^2

Provide 6 Nos of 25 mm Φ bar in two layer at the top of the section at support.

ii) Maximum B.M. at mid span = 106806.9 N-m

A_{st} = 962.87 mm^2

Hence provide 2 bar of 25mm Φ

Transverse reinforcement bar

Maximum shear $V = 628276.12N$

$\tau = V/bd = 1.47\ N/mm^2$

Percentage steel = $100\ A_{st}/bd = 0.55\%$

Corresponding shear stress of concrete (Ref IS: 456)

$\tau_c = 0.31\ N/mm^2$

$\tau_v > \tau_c$ there fore shear reinforcement is required

Balance shear is 493000N

Using 12mm Φ 4-legged stirrups @110mm c/c

2.5.1.2 DESIGN OF COLUMN

The tank is supported on 12 columns symmetrical placed on a circle of 10meter diameter. Height of the staging above ground level is 16meter. It's divided in four panels each of 4meter height.

Vertical load on column

(1) Weight of water = 9981207.27 N

(2) Weight of tank container

i) Weight of top dome + cylindrical wall+ Ring beam B_1, B_3

$$W = 68284.19\times\pi\times 14 = 3001772.99\ N$$

ii) Weight of conical dome $W_s = 1066181.0\ N$

iii) Weight bottom dome $W_d = 550873.75\ N$

iv) Weight of bottom ring beam = $15000\times\pi\times 10 = 471238.89\ N$

Total weight of tank container = 5090016.63 N

Total super imposed load = 15071223.9 N

Load per column P = 15071223.9 / 12 = 1255935.3 N

Let the column be of 800 mm diameter

Weight of column per meter height = $\pi/4\times(0.8)^2\times 1\times 25000 = 12566.37\ N$

Let the bracing be of 300mm × 600mm size

Length of each bracing L = 2.588 meter
Clear length of bracing = 2.588 – 0.8 = 1.788 m

Weight of each bracing = $0.3\times 0.6\times 1.788\times 25000 = 8046\ N$

Area of reinforcement

Longitudinal reinforcement = 0.8% of the cross-sectional area of column

$A_{st} = 4019.2$ mm2

Hence provide 13 bars of 20mm Φ and transverse reinforcement is 10mm Φ bars @300 mm c/c

2.5.1.3 Design of bracing

Initially assume size of bracing is 300mm × 600mm therefore here is only self weight no lateral load so that minimum reinforcements are providing in bracing. Minimum percentage main tension reinforcement, $p \geq 85/fy$

$$Ast = 1029.0 \text{ mm}^2$$

Provide 6 bars of 16mm Φ

Minimum shear reinforcement in the form of vertical stirrups shell be provided such that

$$S < 0.87 f_y A_{sv}/0.4b$$

Use 10mm Φ stirrups @ 250mm spacing c/c.

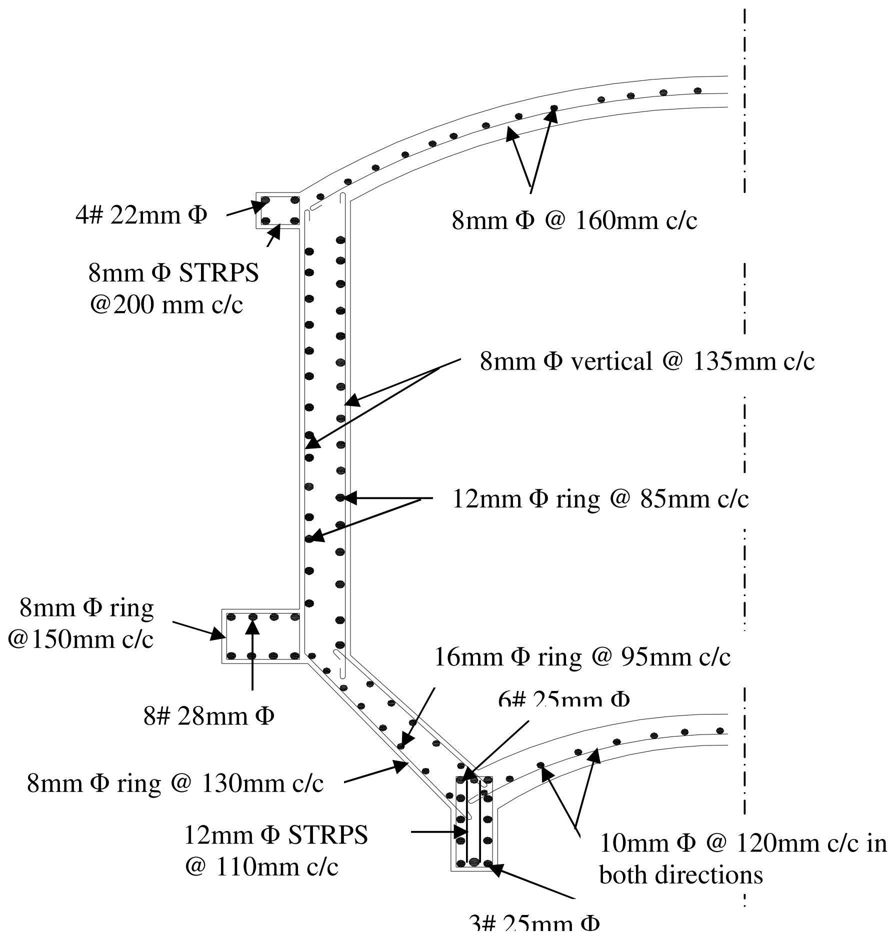


Figure 2.9 Reinforcement detailing in container

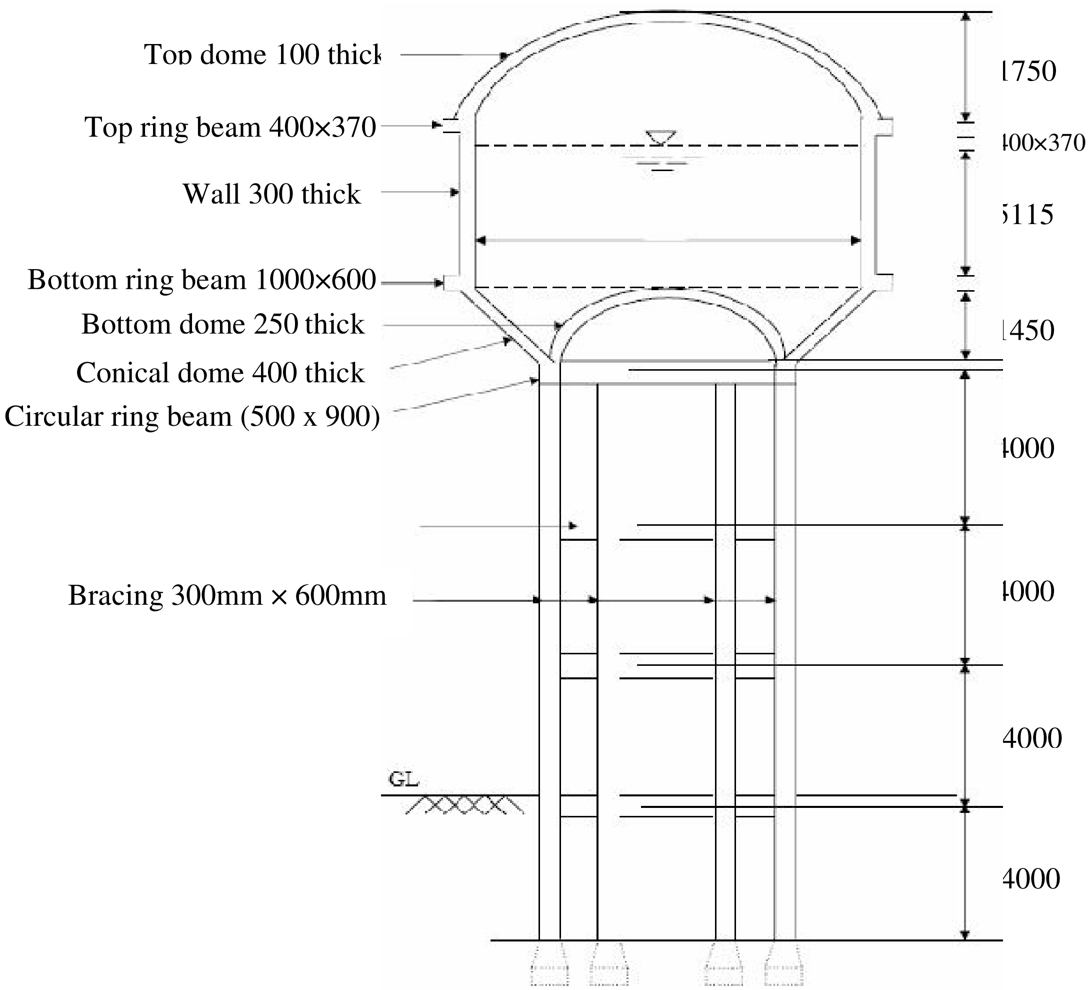


Figure 2.10 Parameter of Intze tank supported on frame staging

2.5.2 SHAFT STAGING

2.5.2.1 Design of bottom circular ring beam B_2

Net horizontal force on circular ring beam is

P = 55299.6 N/m …from equation (2.27)

Hoop compression in beam = P×D/2 = 276498 N

Hoop stress = 276498/A = 1.2

A = 230415 mm2

Hence provide 400mm×600mm ring beam

Vertical load on the beam due to conical dome and spherical dome, per meter run

= 464967.6 N/m …from equation (2.28)

Self weight = 25000×0.4×0.6 = 6000 N/m

Then total vertical load on the beam

$W = 470967.6$ N/m

This vertical load is directly transferred to the foundation threw shaft staging that's why minimum reinforcement in circular ring beam is provided.

Minimum percentage main tension reinforcement, $p \geq 85/fy$

$Ast = 491.56\ mm^2$

Hence provide 5 bars of 12mm Φ in both top and bottom face.

Minimum shear reinforcement in the form of vertical stirrups shell be provided such that

$S < 0.87 f_y A_{sv}/0.4b$

Hence provide 10mm stirrups @ 230mm spacing c/c.

2.5.2.2 Design of shaft staging

Vertical load on staging

1) Weight of water = 9981207.27 N

2) Weight of tank container

i) Weight of top dome + cylindrical wall+ Ring beam B_1, B_3

$W = 3001772.99$ N

ii) Weight of conical dome $W_s = 1066181.0$ N

iii) Weight bottom dome $W_d = 550873.75$ N

iv) Weight of bottom ring beam = 188400 N

Total weight of tank container = 4807227.74 N

Total super imposed load P= 14788435.01 N

Minimum thickness of circular shaft staging is

$= 150 + (D - 6000)/120$

$= 183.33$ mm

Hence we use 220mm thickness concrete shell for staging

Vertical stress $\sigma_{cv} = 14788435.01/2\times\pi\times5000\times220 = 2.1 N/mm2$

This allowable stress is less then the permissible stress ($\sigma_{cv} = 7$) hence it's ok

Vertical reinforcement:

Minimum vertical reinforcement is 0.25% of the concrete area

$Ast = (0.25/100)\times\pi/4\times(10.22^2 - 9.78^2)\times10^6 = 17270 mm^2$

Hence provide 10mm Φ bars @280mm c/c on both face.

Circumferential reinforcement:

Minimum reinforcement of hoop is 0.2% of the concrete area in vertical section.

Area of steel per meter height = $0.002\times220\times1000 = 440 mm^2$

Hence provide 10mm Φ bars @300mm c/c on both face in the form of ring

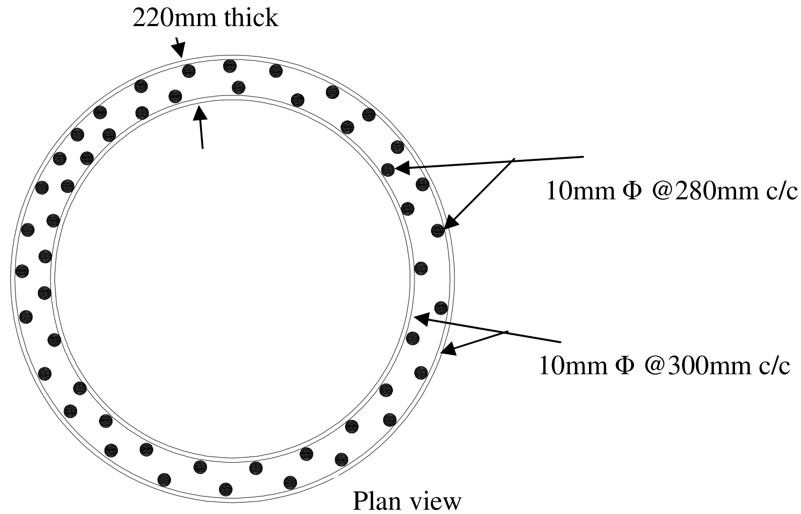


Figure 2.11 Reinforcement Detailing of Shaft Staging

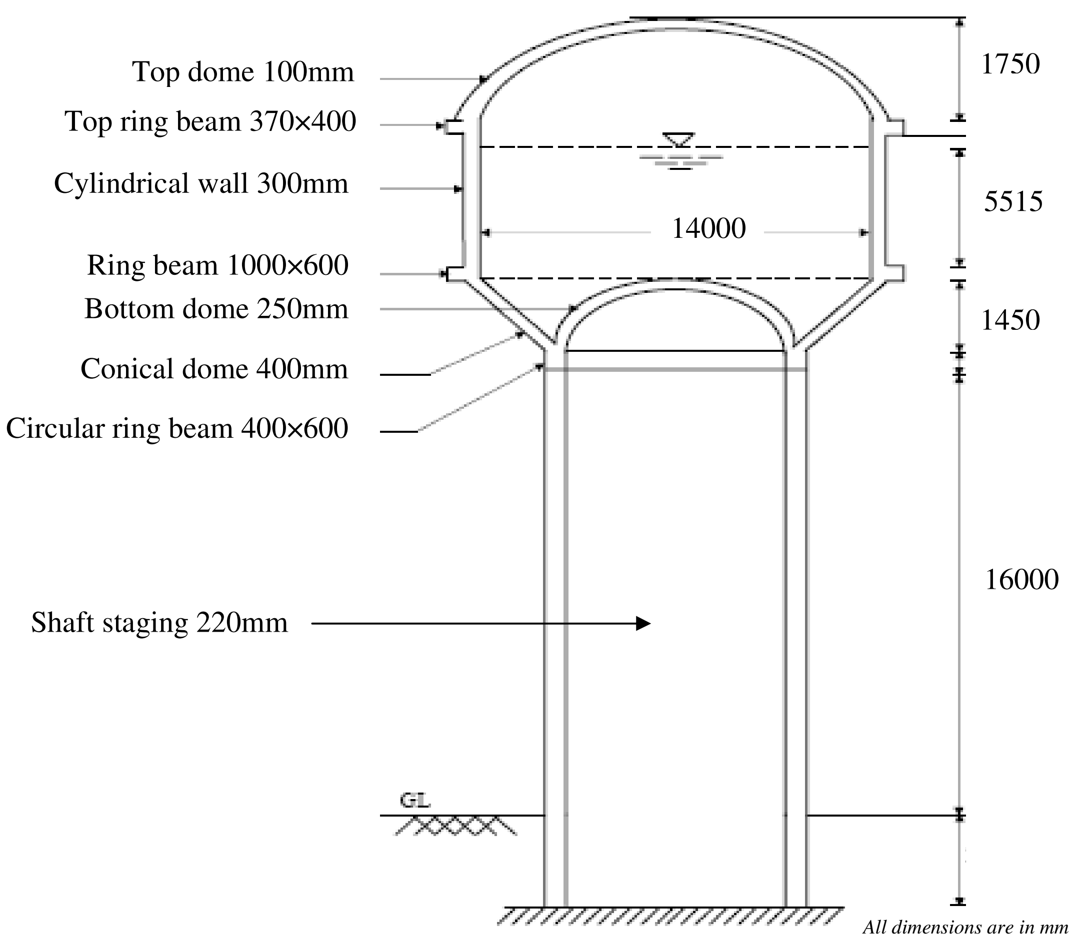


Figure 2.12 Parameter of Intze tank supported on shaft staging

Chapter 3

SEISMIC ANALYSIS OF OVER HEAD WATER TANK

3.1 GENERAL

Analytical studies dealt with the hydrodynamics of liquids in rigid tanks resting on rigid foundations. It was shown that a part of the liquid moves in long-period sloshing motion, while the rest moves rigidly with the tank wall. The latter part of the liquid also known as the impulsive liquid experiences the same acceleration as the ground and contributes predominantly to the base shear and overturning moment. The sloshing liquid determines the height of the free-surface waves, and hence the freeboard requirement. The flexibility of the tank wall may cause the impulsive liquid to experience accelerations that are several times greater than the peak ground acceleration. Thus, the base shear and overturning moment calculated by assuming the tank to be rigid can be neoconservative.

Procedures for the seismic analysis and design of storage tanks are generally based on the Housner, (1963) multi components spring/mass analogy. The analogy allows the complex dynamic behavior of a tank and its contents to be considered in simplified form. The principal modes of response include a short period impulsive mode, with a period of around 0.5 seconds or less, and a number of longer period convective (sloshing) modes with periods up to several seconds. For most tanks, it is the impulsive mode, which dominates the loading on the tank wall. The first convective mode is usually much less significant than the impulsive mode, and the higher order convective modes can be ignored.

Tanks supported on flexible foundations, through rigid base mats, experience base translation and rocking, resulting in longer impulsive periods and generally greater effective damping. These changes may affect the impulsive response significantly. The convective (or sloshing) response is practically insensitive to both the tank wall and the foundation flexibility due to its long period of oscillation.

Failure of tanks during Chilean earthquake of 1960 and Alaska earthquake of 1964 led to beginning of many investigations on seismic analysis of liquid storage tanks. Following two aspects came to forefront.

i) Due consideration should be given to sloshing effect of liquid and flexibility of container wall while evaluating the seismic force of tank.

ii) It is recognized that tanks are less ductile and have low energy absorbing capacity and redundancy compared to the conventional building systems.

All above study is concluded that sloshing effect also must be considered in designing of liquid storage tank. For the purpose of this analysis elevated tanks is considered as a single degree of freedom with their mass concentrated at their centre of gravity.

3.2 LUMPED MASS MODAL METHOD (IS: 1893)2002)

Horizontal pressure

For the purpose of this analysis, elevated tanks shall be regarded as systems with a single degree of freedom with their mass concentrated at their centre of gravity.

The damping in the system may be assumed as 5 percent of the critical for concrete

The Time period T, in seconds, of such structure shall be calculated from the following formula:

$$T=2\pi\sqrt{m/k_s} \qquad \text{…….. (3.1)}$$

Where:

m = mass of the tank container + 1/3rd weight of staging.

k_s = lateral stiffness of the staging

Using time period T calculated in above and 5 percent damping, the spectral acceleration shall be read off from the average acceleration spectra given in Fig.3.2 The design horizontal seismic coefficient.

The lateral force shall be taken equal to:

$$\alpha_h W \qquad \text{…….. (3.2)}$$

Where:

α_h = Design horizontal seismic coefficient

W = Seismic Weight.

The design shall be worked out both when the tank is full and when empty. When empty, the weight W used in the design shall consist of the dead load of the tank and one-third the weight of the staging. When full, the weight of liquid is to be added to the weight under empty condition.

Design horizontal seismic coefficient shell be calculated

W by response spectra method

$$\alpha_h = \beta.I.F_{0.}(S_a/g) \qquad \text{…….. (3.3)}$$

here:

β = a coefficient depending upon the soil foundation

I = a factor depend upon the importance of the structure

F_0 = seismic zone factor for average acceleration spectra

S_a/g = average acceleration coefficient as read from the Fig.3.2 for above time period T and 5 percentage damping

Hydrodynamic Pressure

When a tank containing fluid vibrates the fluid exerts impulsive and convective pressures on the tank. The convective pressures during earthquakes are considerably less in magnitude as compared to impulsive pressures and its effect is a sloshing of the water surface. For the purpose of design only the impulsive pressure may be considered.

The pressure on the wall would be:

$$P_w = \alpha_h wh \sqrt{3}\cos\Phi' \left[\frac{y}{h} - 0.5\left(\frac{y}{h}\right)^2 \right] \tanh\sqrt{3}\,(R/h) \qquad \ldots\ldots (3.4)$$

The pressure on the bottom of the tank on a strip of width 2l would be:

$$P_b = \alpha_h wh \frac{\sqrt{3}}{2} \left(\frac{\sinh\sqrt{3}\,(x/h)}{\cosh\sqrt{3}\,(l'/h)} \right) \qquad \ldots\ldots (3.5)$$

Where:

x, y, l, R and ***h*** are as defined in Fig.3.1 and w is unit weight of water .

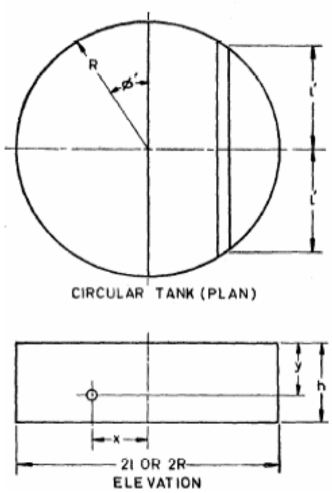


Figure 3.1: Hydrodynamic parameters on circular water tank

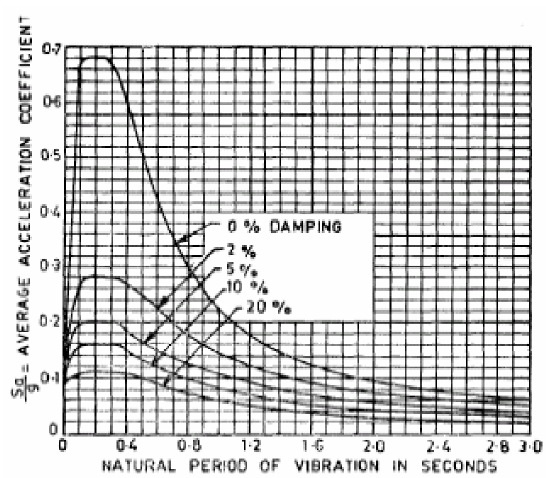


Figure 3.2: Average acceleration spectra

3.3 TWO MASS MODAL METHOD (IS: 1893- 2002)(PART 2 DRAFT)

Most elevated tanks are never completely filled with liquid. Hence a two-mass idealization of the tank is more appropriate as compared to a one mass idealization. Failure of tanks during Chilean earthquake of 1960 and Alaska earthquake of 1964 led to beginning of many investigations on seismic analysis of liquid storage tanks and this aspect came to forefront that consideration should be given to sloshing (convective) effect of liquid and flexibility of container wall while evaluating the seismic force of tank.

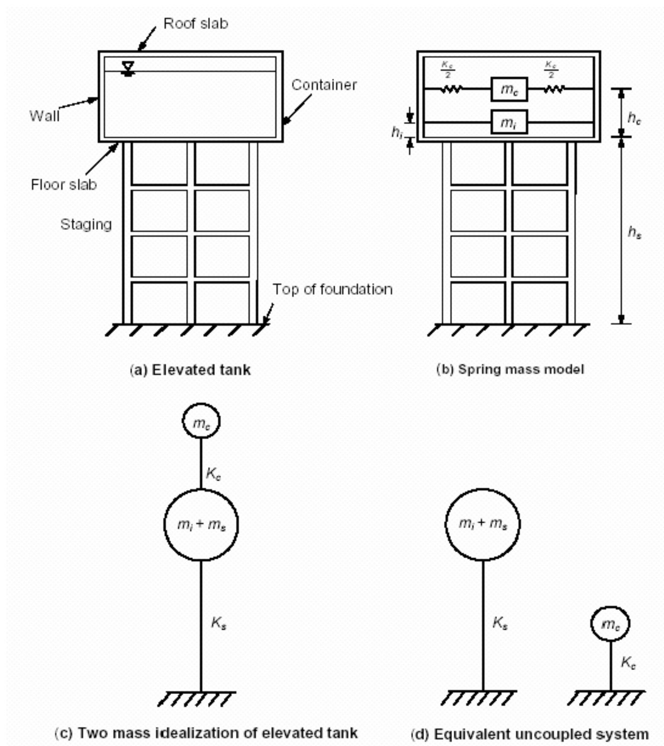


Figure 3.3: Two mass idealization for elevated water tank

Time Period

(1) <u>Impulsive mode</u>

Time period of impulsive mode

$T_i = 2\pi\sqrt{(m_i+m_s/k_s)}$ …… (3.6)

Where:

m_i = impulsive mass

m_s = mass of the staging

k_s = lateral stiffness of the staging

(2) Convective mode

Time period of convective mode, in seconds, is given by

$T_c = 2\pi\sqrt{(m_c/k_c)}$ ...... (3.7)

The expressions for convective mode time period of circular and rectangular tanks are taken from ACI 350.3 (2001), which is based on work of Housner (1963).

$T_c = C_c\sqrt{(D/g)}$ ......(3.8)

Where

C_c = Coefficient of time period for convective mode.

$$C_c = \frac{2\pi}{\sqrt{3.68\tanh(3.68\,(h/D))}} \quad \text{...... (3.9)}$$

D = inner diameter of tank

Damping

Damping in the convective mode for all types of liquids and for all types of tanks shall be taken as 0.5% of the critical. Damping in the impulsive mode shall be taken as 2% of the critical for steel tanks and 5% of the critical for concrete or masonry tanks.

Design Horizontal Seismic Coefficient

Design horizontal seismic coefficient, A_h shall be obtained by the following expression,

$A_h = (Z/2)(I/R)(S_a/g)$ ...... (3.10)

Where

Z= zone factor

I = importance factor

R = response reduction factor

S_a/g = average acceleration coefficient

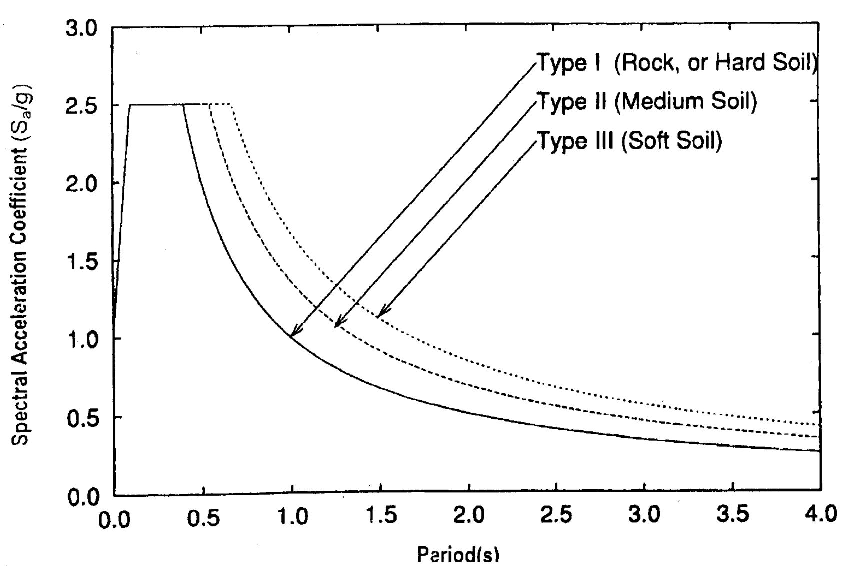


Figure 3.4 Response spectra for 5% damping

Base Shear

$V_i = (A_h)(m_i+m_s)g$ ….. Impulsive mode (3.11)

$V_c = (A_h)(m_c)g$ …..Convective mode (3.12)

Where:

m_s= mass of the container + 1/3[rd] mass of staging

Total base shear

$V = \sqrt{(V_i^2+V_c^2)}$ ..… (3.13)

Base moment

Overturning moment at the base of staging is

$M_i^* = (A_h)_i(m_i(h_i^* + h_s) + m_s h_{cg})g$ ....in impulsive mode (3.14)

$M_c^* = (A_h)m_c(h_c^* + h_s)g$ …….in convective mode (3.15)

Where:

h_s= Structural height of staging, measured from top of footing of staging to the bottom of tank wall,

h_{cg}= Height of center of gravity of empty container, measured from base of staging

Total overturning moment is

$M^* = \sqrt{(M^{*2}_i + M^{*2}_c)}$ …… (3.16)

Note: For elevated tanks, the design shall be worked out for tank empty and tank full both conditions. In case of empty condition, convective mode of vibration will not be generated. Thus, empty elevated tank has to be analyzed as a single degree of freedom system wherein, mass of empty container and one-third mass of staging must be considered. While in case of full condition, both convective and impulsive mode is considered.

Hydrodynamics Pressure

During lateral base excitation, tank wall is subjected to lateral hydrodynamic pressure and tank base is subjected to hydrodynamic pressure in vertical direction

Impulsive Hydrodynamics force:

The impulsive hydrodynamic pressure exerted by the liquid on the tank wall and base is given by:

Lateral hydrodynamic pressure on wall is given by

$P_{iw} = Q_{iw}(y)(A_h)_I \rho ghcos\Phi$ …… (3.17)

$Q_{iw}(y) = 0.866[1 - (y/h)^2]tanh(0.866D/h)$ …… (3.18)

Where

ρ = mass density of liquid

Φ = circumferential angle

Y = vertical distance to a point on tank wall from the bottom of tank wall.

Hydrodynamic pressure in vertical direction on base slab is given by

$$P_{ib} = 0.866\,(A_h)_i\,\rho g h\,\frac{Sinh\left(0.866\,(x/h)\right)}{cosh\left(0.866(l'/h)\right)} \qquad \ldots\ldots (3.19)$$

Where:

x = Horizontal distance of a point on base of tank in the direction of seismic force, from the center of tank.

Convective hydrodynamic pressure:

The convective pressure exerted by the oscillating liquid on the tank wall and base is given by:

Lateral hydrodynamic pressure on wall is given by

$$P_{cw} = Q_{cw}(y)(A_h)_c\rho g D\,[1 - (1/3)cos^2\Phi]\,cos\Phi \qquad \ldots\ldots (3.20)$$

Where:

$$Q_{wc}\,(y) = 0.5625\,\frac{cosh\left(3.674\,y/D\right)}{cosh\left(3.674\,h/D\right)} \qquad \ldots\ldots (3.21)$$

Hydrodynamic pressure in vertical direction on base slab is given by

$$P_{cb} = Q_{cb}(x)(A_h)_c\rho g D \qquad \ldots\ldots (3.22)$$

Where:

$$Q_{cb}(x) = 1.12\left[\frac{x}{D} - \frac{4}{3}\left(\frac{x}{D}\right)^3\right] sech\left(3.674\,\frac{h}{D}\right) \qquad \ldots\ldots (3.23)$$

Pressure Due to Wall Inertia

Pressure due to wall inertia will act in the same direction as that of seismic force. For steel tanks, wall inertia may not be significant. However, for concrete tanks, wall inertia may be substantial. Pressure due to wall inertia, which is constant along the wall height for walls of uniform thickness, should be added to impulsive hydrodynamic pressure.

Pressure on tank wall due to wall inertia is:

$$P_{ww} = (A_h)_i t\rho_m g \qquad \ldots\ldots (3.24)$$

Where:

ρ_m= mass density of tank

T = wall thickness

Effect of Vertical Ground Acceleration

Due to vertical ground acceleration, effective weight of liquid increases, this induces additional pressure on tank wall, whose distribution is similar to that of hydrostatic pressure which describe above.

Hydrodynamic pressure on tank wall due to vertical ground acceleration is:

$$P_v = (A_v)\,\rho g h(1 - y/h) \qquad \text{...... (3.25)}$$

$$A_v = \frac{2}{3}\left(\frac{Z}{2} \times \frac{I}{R} \times \frac{S_a}{g}\right) \qquad \text{...... (3.26)}$$

- Distribution of hydrodynamic pressure due to vertical ground acceleration is similar to that of hydrostatic pressure.
- Design vertical acceleration spectrum is taken as two-third of design horizontal acceleration
- To avoid complexities associated with the evaluation of time period of vertical mode, time period of vertical mode is assumed as 0.3 seconds for all types of tanks.
- While considering the vertical acceleration, due to this effect of increase in weight density of tank and its content.

The maximum value of hydrodynamic pressure should be obtained by combining pressure due to horizontal and vertical excitation through square root of sum of squares (SRSS) rule, which is given as:

$$P = \sqrt{(P_{tw} + P_{ww} + P_{cw})^2 + P_v^{\,2}} \qquad \text{...... (3.27)}$$

Sloshing Wave Height

$$d_{max} = (A_h)_c\, R\, D / 2 \qquad \text{...... (3.28)}$$

Table 3.1 Expression for parameters of spring mass model

Circular tank	Rectangular tank
$\frac{m_i}{m} = \frac{tanh\ \ 0.866\frac{D}{h}}{0.866\frac{D}{h}}$	$\frac{m_i}{m} = \frac{\tanh\left(0.866\frac{L}{h}\right)}{0.866\frac{L}{h}}$
$\frac{h_i}{h} = 0.375$ for $h/D > 0.75$ $= 0.5 - \frac{0.09375}{h/D}$ for $h/D > 0.75$	$\frac{h_i}{h} = 0.375$ for $h/L \le 0.75$ $= 0.5 - \frac{0.09375}{h/L}$ for $h/L > 0.75$
$\frac{h_i\ ^*}{h} = \frac{0.866\frac{D}{h}}{2\,tanh\ \ 0.866\frac{D}{h}} \quad 0.125$ for $h/D \le 1.33$ $= 0.45$ for $h/D > 1.33$	$\frac{h_i\,^*}{h} = \frac{0.866\frac{L}{h}}{2\tanh\left(0.866\frac{L}{h}\right)} - 0.125$ for $h/L \le 1.33$ $= 0.45$ for $h/L > 1.33$
$\frac{m_c}{m} = 0.23\frac{tanh\ \ 3.68\frac{h}{D}}{\frac{h}{D}}$	$\frac{m_c}{m} = 0.264\frac{\tanh\left(3.16\frac{h}{L}\right)}{\frac{h}{L}}$
$\frac{h_c}{h} = 1 - \frac{\cosh\left(3.68\frac{h}{D}\right) - 1.0}{3.68\frac{h}{D}\sinh\left(3.68\frac{h}{D}\right)}$	$\frac{h_c}{h} = 1 - \frac{\cosh\left(3.16\frac{h}{L}\right) - 1.0}{3.16\frac{h}{L}\sinh\left(3.16\frac{h}{L}\right)}$
$\frac{h_c\ ^*}{h} = 1 - \frac{\cosh\left(3.68\frac{h}{D}\right) - 2.01}{3.68\frac{h}{D}\sinh\left(3.68\frac{h}{D}\right)}$	$\frac{h_c\,^*}{h} = 1 - \frac{\cosh\left(3.16\frac{h}{L}\right) - 2.01}{3.16\frac{h}{L}\sinh\left(3.16\frac{h}{L}\right)}$
$K_c = 0.836\frac{mg}{h}\tanh^2\left(3.68\frac{h}{D}\right)$	$K_c = 0.833\frac{mg}{h}\tanh^2\left(3.16\frac{h}{L}\right)$

3.4 SEISMIC ANALYSIS OF INTZE TANK WITH FRAME STAGING

Capacity of tank = 1000 kiloliter and Supported on R.C. frame staging of 12 columns with horizontal bracing

3.4.1 PRELIMINARY DATA FROM CONVENTIONAL DESIGN

Details of sizes of various components and geometry are shown below

Component		**Size (mm)**
Top Dome	-	100 thick
Top Ring Beam B1	-	370×400
Cylindrical Wall	-	300 thick
Bottom Ring Beam B3	-	1000×600
Conical dome	-	400 thick
Bottom dome	-	250 thick
Circular Ring Beam B2	-	500×900
Column	-	800mm dia.
Bracing	-	300×600

3.4.2 WEIGHT CALCULATION OF VARIOUS COMPONENTS

Table 3.2 Weight calculation frame staging tank

Component	Calculation	Weight kN
Top Dome	$2\pi R_1 \times h_1 \times t \times 25$ $2\pi \times 14.875 \times 1.75 \times 0.1 \times 25$	397.21
Top Ring Beam B1	$\pi \times (14+0.4) \times 0.4 \times 0.37 \times 25$	167.38
Cylindrical Wall	$\pi \times 14.3 \times 0.3 \times 5.6 \times 25$	1886.84
Bottom Ring Beam B3	$\pi \times (14+1) \times 1 \times 0.6 \times 25$	706.86
Conical dome	$\pi \times [(14+10)/2] \times 2.8 \times 0.4 \times 25$	1055.57
Bottom Dome	$2 \times \pi \times 8.02 \times 1.75 \times 0.25 \times 25$	551.15
Circular Ring Beam B2	$\pi \times 10 \times 0.9 \times 0.5 \times 25$	353.43
Column	$\pi/4 \times (0.8)^2 \times 16 \times 12 \times 25$	2412.74
Bracing	$0.3 \times 0.6 \times 2.588 \times 12 \times 4 \times 25$	559.0

Weight of water

$$W = [\pi/4 \times (D^2) \times h + \pi/12 \times h_0 \times (D^2 + D_0^2 + D\,D_0) - \pi/3 \times h_2^2 (3R_2 - h_2)] \times g$$

$= 9993.51$ kN

Weight of empty container = 5118.44 kN

Weight of staging = 2412.74 + 559.0= 2971.74 kN

Hence, weight of empty container + one third weight of staging = 6109.02 kN

3.4.3 CENTER OF GRAVITY OF EMPTY CONTAINER

Components of empty container are: top dome, top ring beam, cylindrical wall, bottom ring beam, bottom dome, conical dome and circular ring beam. Then height of the C.G. of tank container above the top of circular ring beam B_2.

i) C.G. of top dome = $R_1 \sin \Phi_1 / \Phi_1 = 14.29$m

CG its own axis $\breve{y} = 14.29 - 13.125 = 1.165$m

ii) CG of bottom dome = $R_2 \sin\Phi_2 / \Phi_2 = 7.439$m

$\breve{y} = 7.439 - 6.27 = 1.169$m

iii) CG of conical dome = $2/3 \times h = 1.33$m

Height of the CG of empty container from top of circular ring beam

$CG = \sum w\breve{y} / \sum w = 3.7$m

Height of the CG of empty container from top of footing is

$h_{cg} = 16 + 0.45 + 3.7 = 20.15$ meter

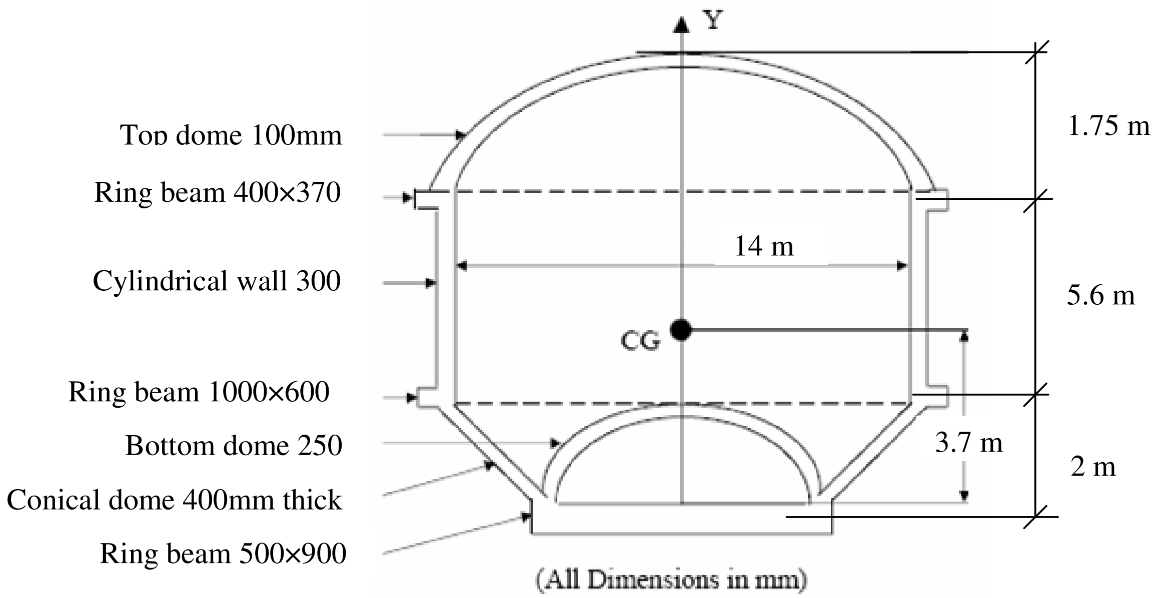


Figure 3.5 Container parameters in frame staging tank

3.4.4 LATERAL STIFFNESS OF THE STAGING

Modulus of elasticity for M-20 concrete is

$E = 5000\sqrt{fck}$

$= 22360$ MPa $= 22.36\times10^6$ kN/m^2

Lateral stiffness of staging is defined as the force required to be applied at the CG of tank so as to get a corresponding unit deflection. CG of tank is the combined CG of empty container and impulsive mass. However CG of tank is taken as CG of empty container. STAAD.Pro2004 software is used to model the staging (Refer Figure 3.5). The stiffness of the staging is calculated by using software STAAD Pro.

Modulus of elasticity for M20 concrete is obtained as

$E = 5{,}000\sqrt{fck}$

$= 22{,}360$ MPa $= 22.36\times10^6$ kN/m^2.

Since container portion is quite rigid, a rigid link is assumed from top of staging to the CG of tank. From the analysis deflection of CG of tank due to an arbitrary 1000 kN force is obtained as 19.257 mm.

Thus, lateral stiffness of staging,

$Ks = 1000/19.257\times10^{-3} = 51929.168$ kN/m

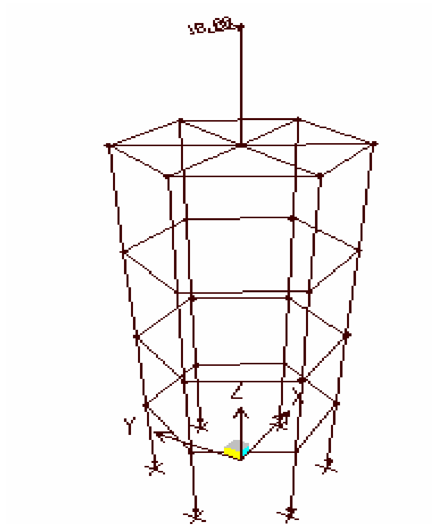


Figure 3.6 Modeling of Frame Staging

3.4.5 LUMPED MASS MODAL MASS METHOD IS: 1893.(2002)

Considering zone IV and soil condition is hard rocky strata

3.4.5.1 Time period

<u>For empty condition</u>

$T = 0.69$ sec. ……from equation (3.1)

$\beta = 1.0$ $I = 1.5$ $F_0 = 0.25$ for Zone IV

Sa/g = 0.14 for 5% damping and 0.69sec time period

$\alpha_h = 0.0525$ ……from equation (3.3)

Lateral force = 320.72 kN ……from equation (3.2)

<u>When tank is full</u>

$T = 1.12$ sec. ……from equation (3.1)

Corresponding this time period Sa/g = 0.10 and all other data are same as empty case

Lateral force = 603.84 kN ……from equation (3.2)

Lateral force is more when tank is full as compare to when tank is empty so that, when tank is full this case is useful for seismic analysis.

Equivalent height of intze tank as a circular cylinder

Volume of water = $(\pi/4)D^2h$

$1018.70 = (\pi/4)14^2 h$

h = 6.62 m

3.4.5.2 Hydrodynamic Pressure

During earthquake tank containing fluid vibrates in two parts. A part the fluid moves in long period sloshing motion, while the rest part moves rigidly with the tank wall. The pressure due to first part is called convective hydrodynamics pressure and pressure due to second part is called impulsive hydrodynamics pressure. In the past purpose of design only impulsive pressure may be considered.

<u>Pressure on the wall of the tank</u>

$P_w = 4007[y/6.62 - 0.5(y/6.62)^2]$ ……from equation (3.4)

Table 3.3 Hydrodynamic pressure on the wall

Y(From Top)	P_w (N/m^2)
0	0
1	550
2	1020
3	1390
4	1670
5	1860
6.62	1980

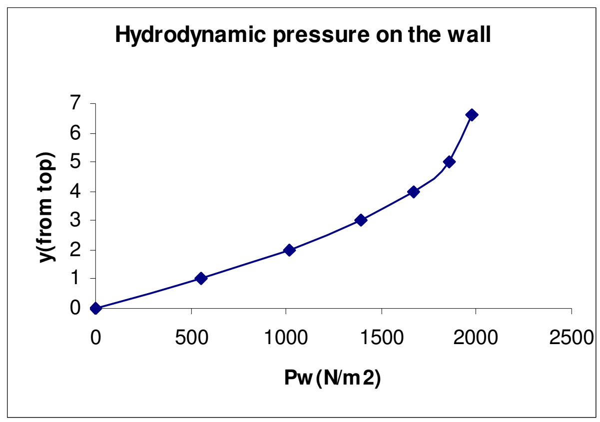


Figure 3.7

<u>Pressure on the bottom of the tank</u>

$P_b = 658\sinh\sqrt{3}(x/6.62)$ ……from equation (3.5)

Table 3.4 Hydrodynamic pressure on the bottom of the tank

X	x/h	P_b (N/m^2)
0	0	0
1	0.15	173
2	0.30	357
3	0.45	567
4	0.604	815
5	0.755	1117
6	0.906	1496
7	1.057	1977

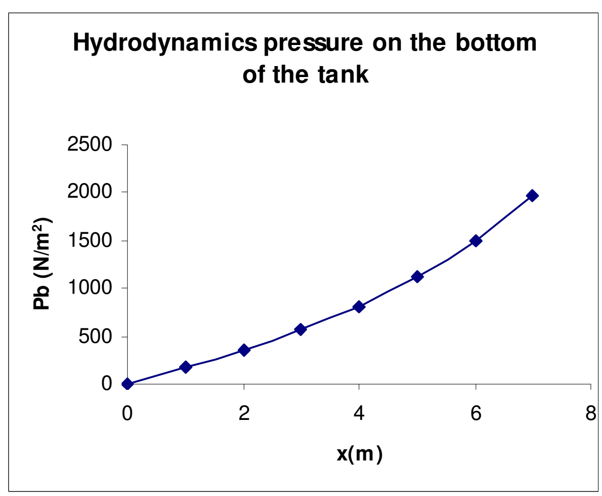


Figure 3.8

3.4.6 TWO MASS MODAL

3.4.6.1 Parameter of Spring Mass Modal

Total weight of water = 9993.5 kN

Volume of water = 9993.5/9.81 = 1018.7 m^3

Mass of water m = 10.188×10^5 Kg

Inner diameter D = 14m

Height of the equivalent circular cylinder =6.62m

For: *h/D* = 0.47 (from table 3.1)

m_i = 528403.73Kg m_c = 468107.79 Kg

hi = 2.482   hc = 3.95m

hi* = 5.55m   hc* = 5.344m

m_s = mass of empty container + 1/3rd weight of staging = 622733.94 Kg

Lateral stiffness of staging k_s = 252885.6 kN/m

3.4.6.2 Time Period

Time period of impulsive mode:

Ti = 0.935 sec. ……from equation (3.6)

Time period of convective mode:

Tc = 4.0 sec. ……from equation (3.8)

3.4.6.3 Design Seismic Horizontal Coefficient

Design seismic horizontal coefficient for impulsive mode,

$Z = 0.24$ (IS 1893(Part 1): Table 2; Zone IV)

$I = 1.5$ (Table 1)

Since staging has special moment resisting frames (SMRF), R is taken as 5(Table 2)

For Ti = 0.935 sec, Site is hard soil and Damping is 5%,

$(S_a/g)_i$ = 1.07 ……from figure (3.4)

$(A_h)_i$ = 0.038 ……from equation (3.10)

Design seismic horizontal coefficient for convective mode,

For Tc = 4.0 sec, Site has hard soil and Damping is 0.5%, (Multiplying factor of 1.75 is used to obtain Sa/g values for 0.5% damping from that for 5% damping).

$(S_a/g)_c$ = 0.4375 ……from figure (3.4)

$(A_h)_c$ = 0.0157 ……from equation (3.10)

3.4.6.4 Base Shear

V_i = 429.121 kN ……from equation (3.11)

V_c = 72.096 kN ……from equation (3.12)

Total base shear

V = 435.135 kN ……from equation (3.13)

3.4.6.5 Base Moment

M_i* = 8913.46 kN-m ……from equation (3.14)

Mc* =1538.83 kN-m ……from equation (3.15)

Total overturning moment

M = 9045.31 kN-m ……from equation (3.16)

3.4.6.6 Hydrodynamics Pressure:

a) Impulsive Hydrodynamics pressure

Impulsive hydrodynamic pressure on wall

Maximum pressure will occur at Φ=0, cosΦ = 1

$P_{iw}(y) = 2030.18[1 - (y/h)^2]$ ……from equation (3.17)

Table 3.5 Impulsive Hydrodynamic pressure on wall

y/h	$P_{iw}(N/m^2)$
0	2030.18
0.2	1948.97
0.4	1705.35
0.6	1299.32
0.8	730.86
1.0	0

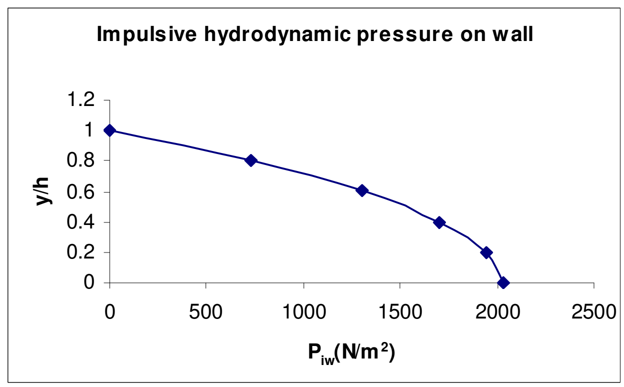


Figure 3.9

Impulsive hydrodynamic pressure on the base slab

P_{ib} = 667.54 sinh(0.131x) ……from equation (3.19)

Table 3.6 Impulsive hydrodynamic pressure on the bottom of tank

X	$P_{ib}(N/m^2)$
0	0
1	87.69
2	176.90
3	269.15
4	366.02
5	469.18
6	580.40
7	701.60

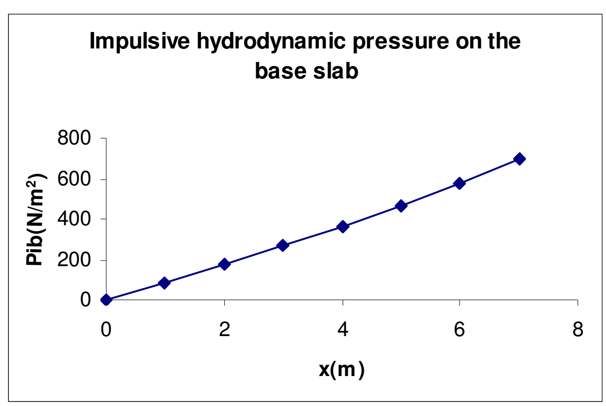


Figure 3.10

b) Convective Hydrodynamics pressure

Convective hydrodynamic pressure on the wall

$P_{cw}(y) = P_{cw}(y) = 275.58\cosh(3.674y/D)$ ……from equation (3.20)

Table 3.7 Convective hydrodynamic pressure on the wall

Y	y/D	P_{cw}(N/m^2)
0	0	275.58
1.5	0.107	297.15
3.0	0.21	361.75
4.5	0.32	489.01
6.0	0.43	697.23
6.62	0.47	799.31

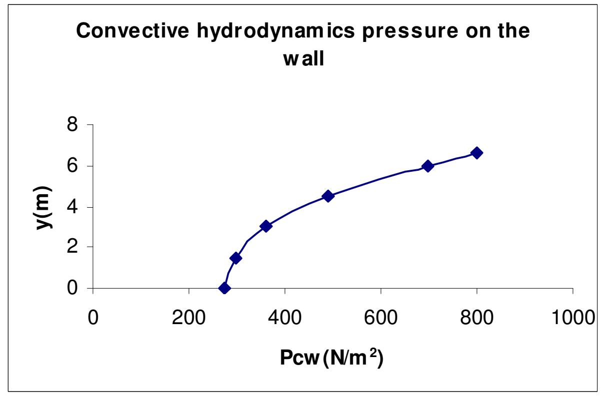


Figure 3.11

Convective hydrodynamic pressure on the base slab

$P_{cb} = 828.19[0.1x - 0.0013x^3]$ ……from equation (3.22)

Table 3.8 Convective hydrodynamic pressure on the base slab

X	P_{cb} (N/m^2)
0	0
1	58.39
2	114.36
3	165.44
4	209.23
5	243.28
6	265.15
7	272.41

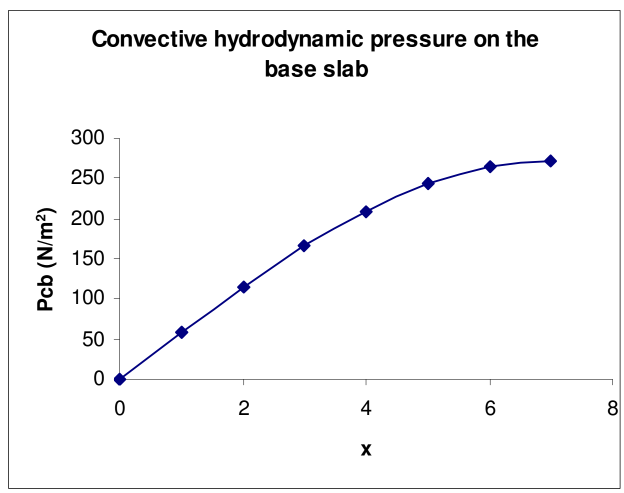


Figure 3.12

c) Pressure due to wall Inertia

$P_{ww} = 285$ N/m^2 ……from equation (3.24)

d) Pressure due to vertical excitation

Time period of vertical mode of vibration is recommended as 0.3 sec for 5 % damping, then *Sa /g* value is 2.5 for this time period, damping and hard rocky site condition.

$A_v = 0.06$ ……from equation (3.26)

$Pv = 3896.53(1 - y/h)$ ……from equation (3.25)

Table 3.9 Pressure due to vertical excitation

y/h	P_v (N/m^2)
0	3896.53
0.2	3117.22
0.4	2337.92
0.6	1558.61
0.8	779.31
1.0	0

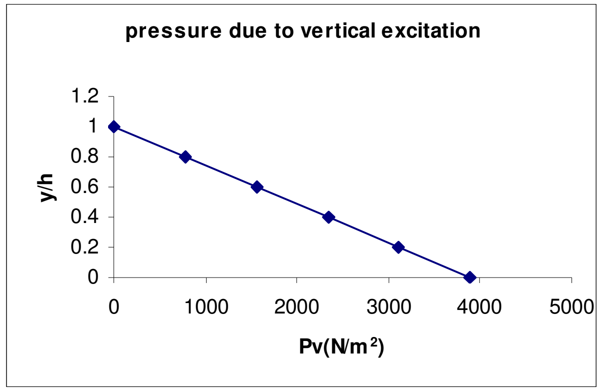


Figure 3.13

Maximum Hydrodynamic Pressure

$P = 4679.20$ N/m^2 ……from equation (3.27)

Sloshing Wave Height

$d_{max} = 0.45$ meter ……from equation (3.28)

During preliminary design only 13cm free board is available. During earthquake in convective mode sloshing wave height will be up to 45cm so that free board is required 45cm. in such case height of cylindrical portion is increased.

3.5 SEISMIC ANALYSIS OF INTZE TANK WITH SHAFT STAGING

Capacity of tank = 1000 kiloliter and Supported on R.C. Shaft staging

3.5.1 PRELIMINARY DATA FROM CONVENTIONAL DESIGN

Details of sizes of various components and geometry are shown below

Component		**Size (mm)**
Top Dome	-	100 thick
Top Ring Beam B1	-	370×400
Cylindrical Wall	-	300 thick
Bottom Ring Beam B3	-	1000×600
Conical dome	-	400 thick
Bottom dome	-	250 thick
Circular Ring Beam B2	-	400×600
Shaft Staging	-	220 thick.

3.5.2 WEIGHT CALCULATION OF VARIOUS COMPONENTS

Table 3.10 Weight calculation shaft staging tank

Component	**Calculation**	**Weight KN**
Top Dome	$2\pi R_1 \times h_1 \times t \times 25$ $2\pi \times 14.875 \times 1.75 \times 0.1 \times 25$	397.21
Top Ring Beam B1	$\pi \times (14+0.4) \times 0.4 \times 0.37 \times 25$	167.38
Cylindrical Wall	$\pi \times 14.3 \times 0.3 \times 5.6 \times 25$	1886.84
Bottom Ring Beam B3	$\pi \times (14+1) \times 1 \times 0.6 \times 25$	706.86
Conical dome	$\pi \times [(14+10)/2] \times 2.8 \times 0.4 \times 25$	1055.57
Bottom Dome	$2 \times \pi \times 8.02 \times 1.75 \times 0.25 \times 25$	551.15
Circular Ring Beam B2	$\pi \times 10 \times 0.6 \times 0.4 \times 25$	188.49
Shaft Staging	$\pi \times 10 \times 0.22 \times 16 \times 25$	2764.60

Weight of water W = 9993.51 KN

Weight of empty container = 4953.5 kN

Weight of staging = 2764.60 kN

Hence, weight of empty container + one third weight of staging

= 5875.03 kN

3.5.3 CENTER OF GRAVITY OF EMPTY CONTAINER

Components of empty container are: top dome, top ring beam, cylindrical wall, bottom ring beam, bottom dome, conical dome and circular ring beam. Then height of the C.G. of tank container above the top of circular ring beam B_2.

i) C.G. of top dome = $R_1 \sin \Phi_1 / \Phi_1 = 14.29$m

CG its own axis $\breve{y} = 14.29 - 13.125 = 1.165$m

ii) CG of bottom dome = $R_2 \sin\Phi_2 / \Phi_2 = 7.439$m

$\breve{y} = 7.439 - 6.27 = 1.169$m

iii) CG of conical dome = $2/3 \times h = 1.33$m

Height of the CG of empty container from top of circular ring beam

CG = $\sum w\breve{y} / \sum w = 3.85$m

Height of the CG of empty container from top of footing is

hcg = 16 + 0.3 + 3.85 = 20.15 meter

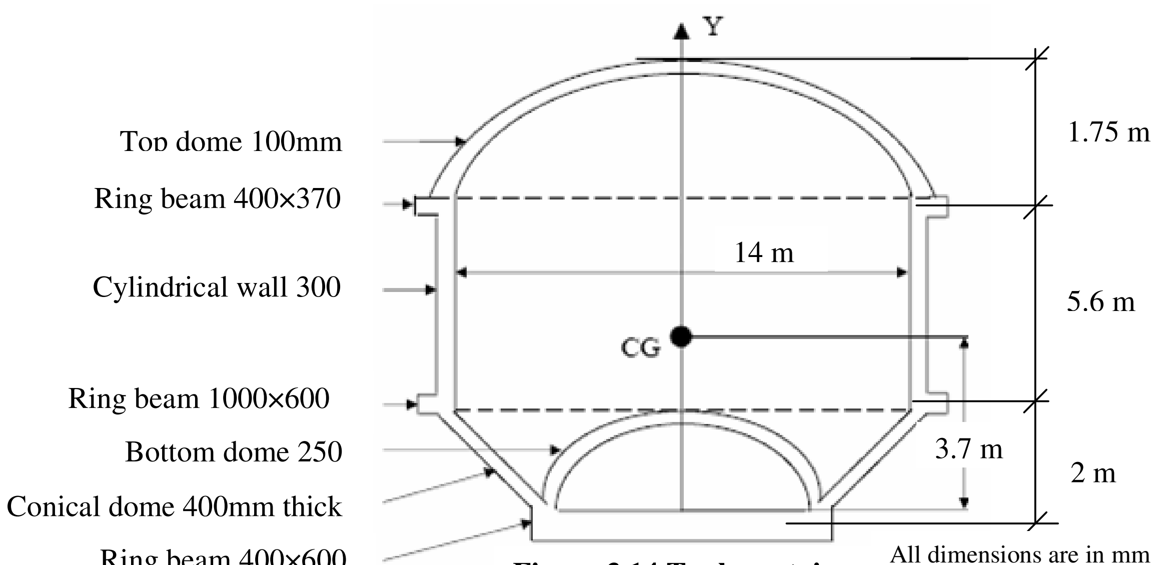


Figure 3.14 Tank container

3.5.4 LATERAL STIFFNESS OF THE STAGING (SP: 22-1982)

Lateral stiffness of staging is defined as the force required to be applied at the CG of tank so as to get a corresponding unit deflection. Shaft is considered as cantilever of length 16 meter from top of the footing up to bottom of circular ring beam.

$Ks = 3EI/L^3$

Modulus of elasticity for M20 concrete is obtained as

$E = 5{,}000\sqrt{fck}$

$= 22{,}360 \text{ MPa} = 22.36\times10^6 \text{ kN/m2}.$

Moment of inertia of shaft section is

$I = (\pi/64)\,(D^4 - d^4) = (\pi/64)\,(10.22^4 - 9.78^4)$

$= 86.44 \text{ m}^4$

Thus, lateral stiffness of staging,

$Ks = 1.41\times10^6 \text{ kN/m}$

3.5.5 LUMPED MODAL MASS METHOD (Ref IS: 1893-2002)

Take zone IV and soil condition is hard rocky strata

3.5.5.1 Time period

<u>For empty condition</u>

$m = 5875.03 \text{ kN} = 598882.09 \text{ Kg}$

$T = 0.129 \text{ sec.}$ ……from equation (3.1)

$\beta = 1.0$, $I = 1.5$, $F_0 = 0.25$ for Zone IV and $Sa/g = 0.2$ for 5% damping and 0.129sec time period from Fig.

$\alpha_h = 0.075$ ……from equation (3.3)

Lateral force = 440.62 kN ……from equation (3.2)

When tank is full

m =15868.54 kN = 1617588.17 Kg

T = 0.213 sec. ……from equation (3.1)

Corresponding this time period $Sa/g = 0.2$ from Fig. 4.2 and other data are same as empty case.

Lateral force = 1190.14 kN ……from equation (3.2)

Lateral force is higher in case of full condition as compare to empty condition, that's why full condition is useful for seismic analysis.

Equivalent height of intze tank as a circular cylinder

$(\pi/4)D^2h$ = volume

$(\pi/4)14^2h = 1018.70$ $h = 6.62$ m

3.5.5.2 Hydrodynamics pressure

Pressure on the wall would be

$P_w = 8014.17[y/6.62 - 0.5(y/6.62)^2]$ ……from equation (3.4)

Table 3.11 Hydrodynamic Pressure on the wall

Y(From Top)	P_w (N/m^2)
0	0
1	1119.19
2	2055.59
3	2809.19
4	3379.99
5	3767.99
6.62	4008.60

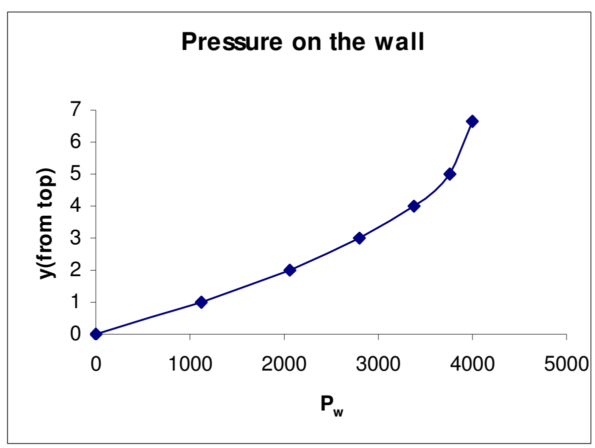


Figure 3.15

Pressure on the bottom of the tank

$P_b = 1316 \sinh\sqrt{3}(x/6.62)$ ……from equation (4.5)

Table 3.12 Pressure on the bottom of the tank

X	x/h	P_b (N/m^2)
0	0	0
1	0.15	348.20
2	0.30	720.37
3	0.45	1142.13
4	0.604	1642.48
5	0.755	2255.89
6	0.906	3024.56
7	1.057	4001.40

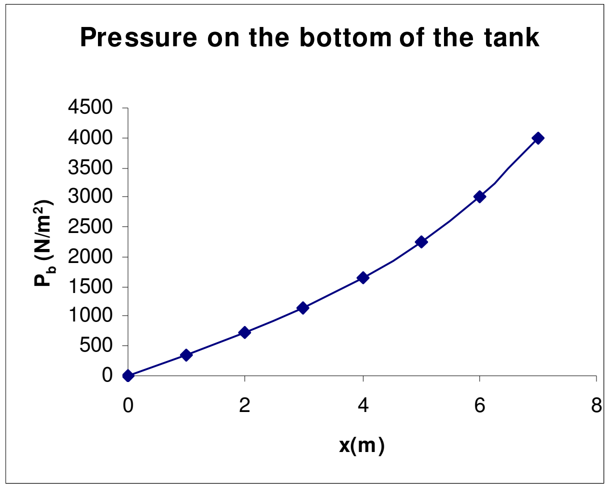


Figure 3.16

3.5.6 TWO MASS MODAL

3.5.6.1 Parameter of Spring Mass Modal

Total weight of water = 9993.5 kN

Volume of water = 9993.5/9.81 = 1018.7 m^3

Mass of water m = 10.188×10^5 Kg

Inner diameter D = 14m

Height of the equivalent circular cylinder =6.62m

For h/D = 0.47 (From table 3.1)

m_i = 528403.73Kg m_c = 468107.79 Kg

hi = 2.482 *hc* = 3.95m

*hi** = 5.55m hc* = 5.344m

m_s = 598882.09 Kg

Lateral stiffness of staging $k_s = 1.41\times10^9$ N/m

3.5.6.2 Time Period

Time period of impulsive mode:

Ti = 0.177 sec. ……from equation (3.6)

Time period of convective mode:

Tc = 4.0 sec. ……from equation (3.8)

3.5.6.3 Design Seismic Horizontal Coefficient

Design seismic horizontal coefficient for impulsive mode,

Z = 0.24 (IS 1893(Part 1): Table 2; Zone IV)

I = 1.5 (Table 1)

Since staging is a special moment resisting frames (SMRF), *R* is taken as 5 from (Table2), time period in impulsive case is 0.177 sec, site is hard soil and damping is 5% then

$(S_a/g)_i = 2.5$ ……from figure (3.4)

$(A_h)_i = 0.09$ ……from equation (3.10)

Design seismic horizontal coefficient for convective mode

For Tc = 4.0 sec, Site has hard soil and Damping is 0.5%, (Multiplying factor of 1.75 is used to obtain *Sa /g* values for 0.5% damping from that for 5% damping). For convective mode, value of *R* is taken same as that for impulsive mode.

$(S_a/g)_c = 0.4375$ ……from figure (3.4)

$(A_h)_c = 0.016$ ……from equation (3.10)

3.5.6.4 Base Shear

$V_i = 995.28$ kN ……from equation (3.11)

$V_c = 73.47$ kN ……from equation (3.12)

Total base shear

V = 997.98 kN ……from equation (3.13)

3.5.6.5 Base Moment

$M_i^* = 20708.04$ kN-m ……from equation (3.14)

$Mc^* = 1568.23$ kN-m ……from equation (3.15)

Total overturning moment

$M = 20767.34$ kN-m ……from equation (3.16)

3.5.6.6 Hydrodynamics Pressure:

a) Impulsive Hydrodynamics pressure

Hydrodynamic pressure on wall

Maximum pressure will occur at $\Phi=0$, $\cos\Phi = 1$

$P_{iw}(y) = 4804.42[1 - (y/h)^2]$ ……from equation (3.17)

Table 3.13 Impulsive hydrodynamic pressure on the wall

y/h	$P_{iw}(N/m^2)$
0	4804.42
0.2	4612.24
0.4	4035.71
0.6	3074.83
0.8	1729.59
1.0	0

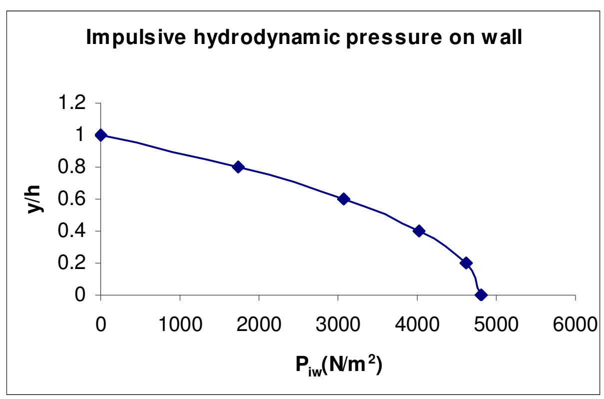


Figure 3.17

Hydrodynamic pressure on the base slab

$P_{ib} = 1581.02 \sinh(0.1308x)$ ……from equation (3.19)

Table 3.14 Impulsive hydrodynamic pressure on the base slab

X(m)	$P_{ib}(N/m^2)$
0	0
1	207.39
2	418.32
3	636.44
4	865.45
5	1109.29
6	1372.13
7	1658.48

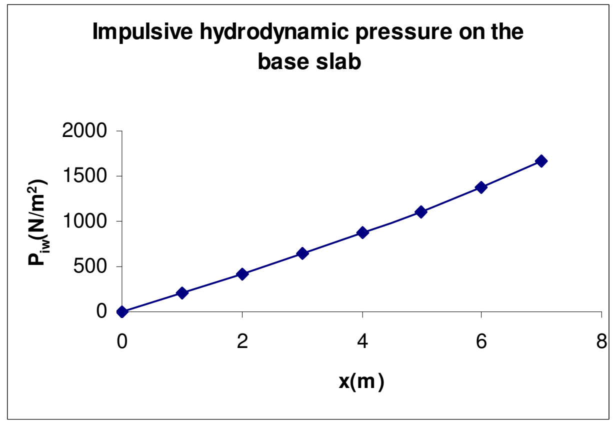


Figure 3.18

b) Convective Hydrodynamics pressure

Hydrodynamic pressure on the wall

$P_{cw}(y) = 275.58\cosh(3.674y/D)$ ……from equation (3.20)

Table 3.15 Convective hydrodynamic pressure on the wall

Y	y/D	$P_{cw}(N/m^2)$
0	0	275.58
1.5	0.107	297.15
3.0	0.21	361.75
4.5	0.32	489.01
6.0	0.43	697.23
6.62	0.47	799.31

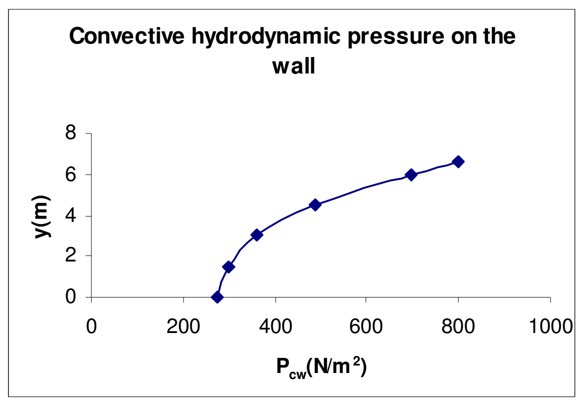


Figure 3.19

Hydrodynamic pressure on the base slab

$P_{cb} = 828.19[0.1x - 0.0013x^3]$ ……from equation (3.22)

Table 3.16 Convective hydrodynamic pressure on the base slab

X	P_{cb} (N/m^2)
0	0
1	58.39
2	114.36
3	165.44
4	209.23
5	243.28

6	265.15
7	272.41

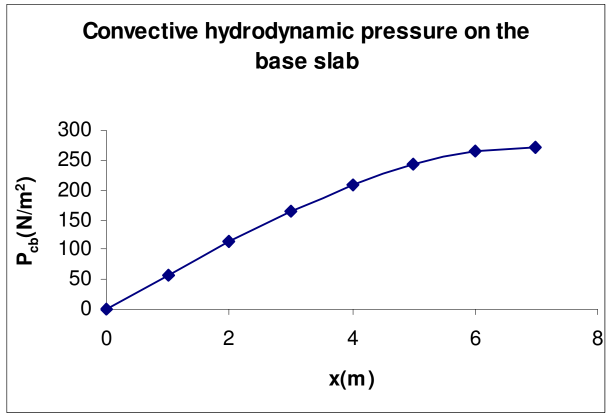


Figure 3.20

c) Pressure due to wall Inertia

$P_{ww} = 675 \ N/m^2$ ……from equation (3.24)

d) Pressure due to vertical excitation

Time period of vertical mode of vibration is recommended as 0.3 sec for 5 % damping, then *Sa /g* value is 2.5 for this time period, damping and hard rocky site condition.

$A_v = 0.06$ ……from equation (3.26)

$Pv = 3896.53(1 – y/h)$ ……from equation (3.25)

Table 3.17 Pressure due to vertical excitation

y/h	P_v (N/m^2)
0	3896.53
0.2	3117.22
0.4	2337.92
0.6	1558.61
0.8	779.31
1.0	0

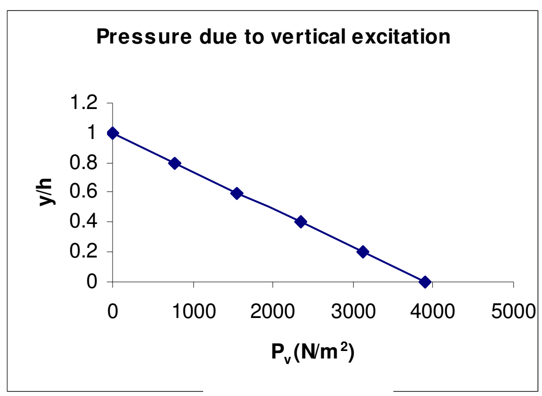


Figure 3.21

e) Maximum Hydrodynamic Pressure

$P = 6950.03 \ N/m^2$ ……from equation (3.27)

f) Sloshing Wave Height$d_{max} = 0.549$ meter ……from equation (3.28

during preliminary design only 13cm free board is available. During earthquake in convective mode sloshing wave height will be up to 55cm so that free board is required 55cm. in such case height of cylindrical portion is increased

Chapter 4

DESIGN OF OVER HEAD WATER TANKS CONSIDERING EARTHQUAKE FORCES

4.1 GENERAL

In past, performance of liquid storage tank during earthquake was observed, most of the water tanks failure was during earthquake.

Failure of tanks during Chilean earthquake of 1960 and Alaska earthquake of 1964 led to beginning of many investigations on seismic analysis of liquid storage tanks. There are two aspects.

i) Due consideration should be given to sloshing effect of liquid and flexibility of container wall while evaluating the seismic force of tank.

ii) It is recognized that tanks are less ductile and have low energy absorbing capacity and redundancy compared to the conventional building systems.

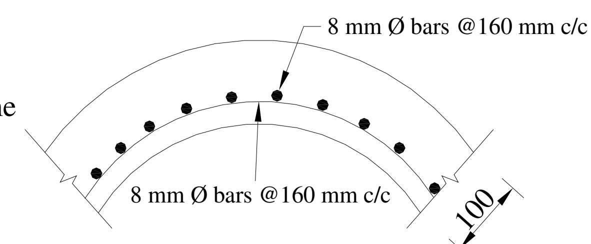
For considering all above aspect I have to decide design of liquid storage tank considering forces due to both impulsive as well as convective mode. Design consideration of tank is same as previous chapter 2.

4.2 REDESIGN OF INTZE TANK WITH FRAME STAGING CONSIDER EARTHQUAKE FORCES

4.2.1 TOP DOME

There is no effect of Earthquake so its design is same as conventional design.

Hence provide 8 mm Φ bars @ 160 mm c/c.

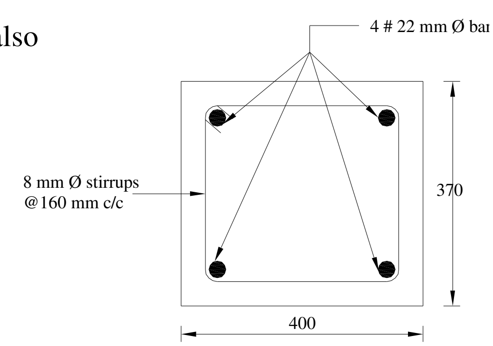


SECTION OF TOP DOME

4.2.2 RING BEAM (B1)

There is also no effect of earthquake so its design also same as conventional design Provide ring beam of 370mm depth and 400mm width.

Tie the 22mm Φ rings by 8mm diameter nominal stirrups @ 200mm c/c spacing.



SECTION OF TOP RING BEAM

4.2.3 DESIGN OF CYLINDRICAL WALL

Hoop tension without considering earth quake

At top = 0

At bottom = whD/2 = 412020 N/m height

Hoop tension due to earth quake

(a) For impulsive mode

At top = 0

At bottom = 14211.26 N/m

(b) For convective mode

At top = 5595.17 N/m

At bottom = 1926.05 N/m

Total hoop tension

At top = 5595.17 N/m

At bottom = 428157.31 N/m

Area of steel $A_{sh} = P_h / \sigma_{st} = 2854.38\ mm^2$ per meter height

Hence provide 16mm Φ rings @ 140mm c/c.

Actual $A_{sh} = 100 \times 201/140 = 1435.71\ mm^2$

Permissible stress in composite section is 1.2 N/ mm^2 the

$428157.31 / (1000 \times t + 12 \times 1435.71 \times 2) <= 1.2$

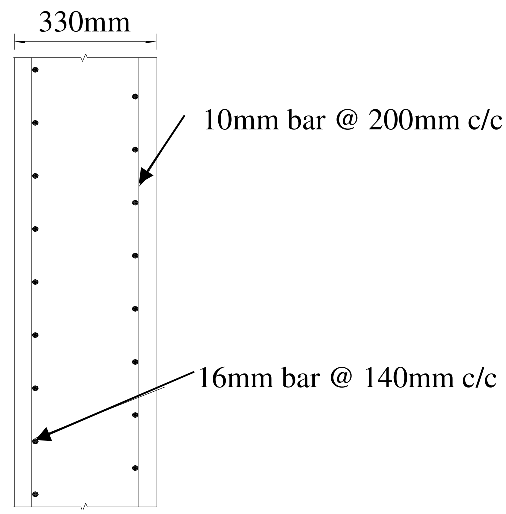
t = 322.34 mm

Hence provide 330mm thickness.

Percentage distribution steel = 0.3% for 100mm thickness and 0.2% for 450mm thickness. Hence for 300mm thickness percentage distribution is 0.234%

$A_{sh} = 773.14 mm^2$

Hence provide 10mm Φ bars @ 200mm c/c on each face.



4.2.4 DESIGN OF MIDDLE RING BEAM B_3

This beam connects the tank wall with conical dome. The vertical load at the junction of the wall with conical dome is transferred to ring beam by meridian thrust in the conical dome. The horizontal component of thrust cause hoop tension at the junction.

The load W transmitted through tank wall at the top of conical dome is

i) Load due to top dome = T_1 x sinΦ = 14859.19N/m

ii) Load due to ring beam = 25000×0.37×(0.4 – 0.3) = 925N/m

iii) Load due to tank wall = 25000×0.33×6.0 = 49500N/m

iv) Self weight of ring beam (assume initially size of ring beam is 1m x 0.6m)

$= 25000 \times 0.6\ (1 - 0.3) = 10500$N/m

Total load W =75784.19 N/m

Inclination of conical dome with vertical is $\Phi_0 = 45$ degree

$\sin\Phi_0 = \cos\Phi_0 = 0.707$ $\tan\Phi_0 = 1$

Force due to self weight of upper structure $P_W = W.\tan\Phi_0 = 75784.19$ N/m

Pressure due to water $P_w = w.h.d_3 = 35280$ N/m

Pressure due to earthquake $P_{eq} = (Piw + Pcw) \times d_3 = (2030.18 + 275.15) \times 0.6$

$= 1383.19$ N/m

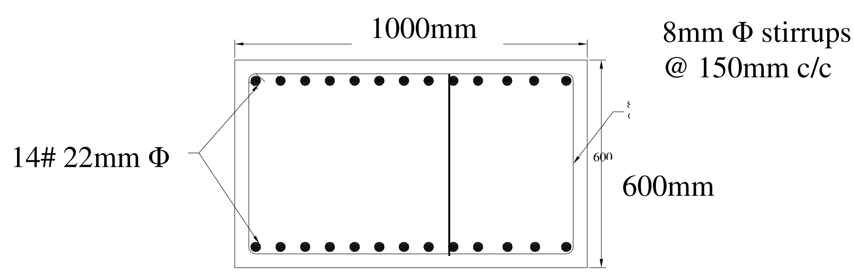
Hence hoop tension on the ring beam $P_3 = (P_W + P_w + P_{eq}).D/2 = 787131.72$ N

This hoop tension resisted by the steel hoops, then area of hoops 5247.54 mm^2

Hence provide 14 bars of 22mm Φ then actual $A_{sh} = 5319.16$mm^2

Stress in equivalent steel = $P_3 / (A + (m - 1) A_{sh}) = 1.18 < 1.2$ Hence safe

Hence provide 1000mm×600mm ring beam with 14 bars of 22 mm Φ and distribution bars of 8mm Φ @150mm c/c.



Section of middle ring

4.2.5 DESIGN CONICAL DOME

(a) <u>Meridian thrust</u>

The weight of the water is

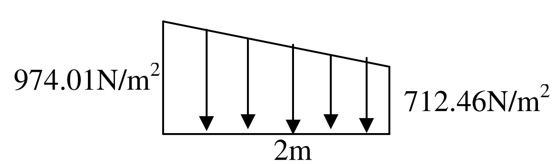
$W_w = 5128666.66$ N

Initially we assume thickness of the conical dome is 400mm then self weight is

Ws = 1066131 N

Weight W at ring beam B_3 due to upper element like top dome top, ring beam, cylindrical wall and self weight itself is = 75784.19 N/m

Weight due to Earthquake:



W_{eq} = 1686.47 N/m

Hence total vertical load W per meter run is given by

$W = [(75784.19\times\pi\times14) + 5128666.66 + 1066131 + (1686.47\times\pi\times12.1)] / \pi\times10v$

$= 305426.08$ N/m

Meridian thrust T_0 in the conical dome is

$T_0 = W / \cos\Phi_0 = 432002.94$ N/m

Meridian stress

$T_0 / t = 432002.94 / 1000\times400$

$1.08 < 1.2$ Hence safe

(b) Hoop tension

Diameter of conical dome at any height h’ above the base is

$D' = 10 + 2h'$

Intensity of water pressure at any height h’ above the base is

$P = 9800\times(8 - h')$ N / mm^2

Self weight q = 10000 N/ mm^2

Hence hoop tension

$P_0' = [9800(8 - h')\times\sqrt{2} + 10000x1](10 + 2h')/2$ …… from equation (3.19)

$dP_0'/dh' = 51577.84 - 27718.6h' = 0$

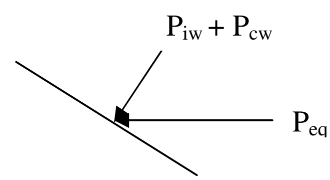
$h' = 1.86$ m

Hence at h’=1.86m have the maximum hoop tension

Maximum $P_0' = 604371.72 + 51577.84\times1.86 - 13859.3\times1.86^2 = 652358.87$ N

(c) Design of wall

Maximum hoop stress = 652358.87 N at the h’=1.86 meter above the base



Pressure due to earthquake at h’=1.86 meter

$P_{eq} = 10463.41$ N

Total hoop stress = 662822.28 N

Area of steel

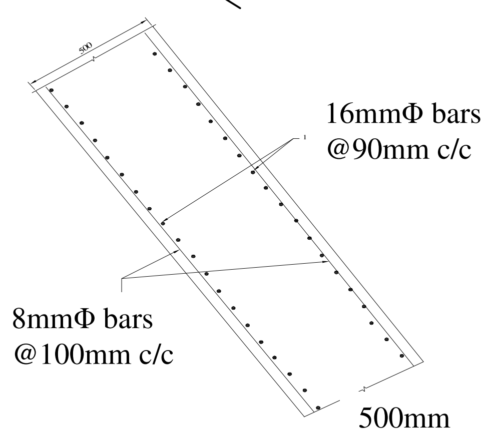
$A_{st} = 662822.28 / 150 = 4418.82$ mm^2

Hence provide 16mm Φ bar @ 90mm c/c on each face.

Actual $A_{sh} = 2233.33$ mm^2

Maximum tensile stress of composite section is

$P_h / (A + (m - 1) A_{sh}) = 1.2$



$662822.28 / (1000\times t + 12\times 2\times 2233.33) = 1.2$

$t = 498.75$ mm

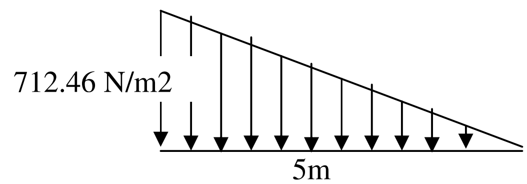
Hence provide 500mm thick wall of conical dome

4.2.6 DESIGN BOTTOM DOME: (Ref: N Krishna Raju)

Radius $R_2 = 8.02$ $\quad \sin\Phi_2 = 0.62$ $\quad \cos\Phi_2 = 0.78$

Weight of the water on the dome is $W_0 = 5456723.60$ N

Initially we take thickness of the bottom dome is 250mm



Then self weight of bottom dome is Ws= 550873.75 N

Additional weight due to earthquake $W_{eq} = 37285.40$ N

Total weight of bottom dome is W = 6044882.75 N

Load per unit area $W_t = 77004.87$ N/m^2

Meridian thrust $T_2 = 346954.55$ N/m ….from equation (3.25)

Meridian stress = $T_2 / t = 1.38 > 1.2$

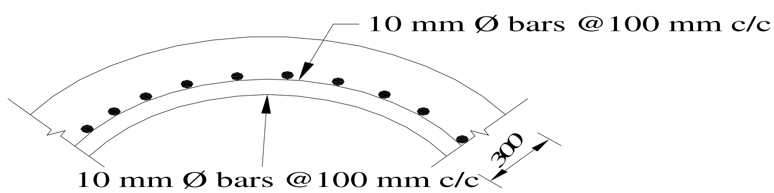
Hence increase the thickness of the dome. Hence use 300 mm thickness of the dome

Circumference force = $W_t R(\cos\theta - 1/ 1+ \cos\theta) = 308789.53$ N/m

Hoop stress = 1.02 <1.2 hence ok.

Provide nominal reinforcement = 0.24 % for thickness 300mm, hence area of steel in each direction is 720 mm^2

Hence provide 10mm Φ bars @ 100mm c/c in both directions



SECTION OF BOTTOM DOME

4.2.7 DESIGN OF BOTTOM CIRCULAR RING BEAM

Horizontal force

P = 34847.65 N/m ….from equation (2.27)

Hoop compression in the beam = P×D/2 = 174238.25 N

Hoop stress = 174238.25 / 500×900 = 0.387 <1.2 hence safe

Vertical load on ring beam W = 520584.03 N/m ….from equation (2.28)

Self weight of ring beam = 11250.0 N/m

Total vertical load W = 531834.03 N/m

The beam is supported on 12 equally spaced columns at mean diameter 10meter and mean radius of curvature R = 5m, $2\theta = 30$, $\theta = 15$

Value of different coefficient has taken from the IS: 3370

$C_1 = 0.045$; $C_2 = 0.017$; $C_3 = 0.002$; $\Phi_m = 6.25^o$

$WR^2 (2\theta) = 6958161.88$ N-m

Maximum –ve BM at support $M_s = C_1WR^2 (2\theta) = 313117.28$ N-m

Maximum +ve BM at mid span $M_c = C_2WR^2 (2\theta) = 118288.75$ N-m

Maximum torsional moment $M_m^t = C_3WR^2 (2\theta) = 13916.32$ N-m

Using M-20 concrete ($\sigma_{cbc} = 7$ N/mm2) and HYSD bar $\sigma_{st} = 150$ N/mm2

K = 0.38, J = 0.87 and R = 1.156

Required effective depth = $\sqrt{M/R \times b}$ = 736.02mm

Hence we provide total depth is 900mm and effective depth is 850mm then it's safe.

Effective depth d = 850mm

(a) Design at support section

d = 850mm; cover = 50mm

Longitudinal reinforcement $A_{st} = Ms/\sigma_{st}.j.d = 2793.88$ mm^2

Provide 6 bars of 25mm Φ

Maximum shear force at support $F_0 = WR\theta = 695816.19$ N

Shear stress at support $\tau_v = F_0/bd = 1.64$ N/mm2

Percentage steel at support = $100\times A_{st}$ / bd = 0.66 %

Corresponding this percentage steel, shear resisting capacity of concrete is (Ref. IS: 456)

$\tau_c = 0.33$ N/mm2

Here $\tau_v > \tau_c$ so that shear reinforcement is required

Shear taken by concrete = 140250 N

Balance shear = 555566.19 N

Hence provide 12mm Φ 4-leged stirrups @ 100mm c/c near support.

(b) Design of mid span section

Moment at mid section is, Mc = 118288.75 N-m

Area of steel at mid span is $A_{st} = Mc/\sigma_{st}.j.d = 1055$ mm2

Minimum area of steel is 0.3% = 1275 mm2

Provide 4 bars of 20mm Φ at mid span section and 10mm 4-leged stirrups @ 300mm c/c

(c) Design of section subjected to maximum torsion

Maximum torsional moment, $T = M_t = 13916.32$ N-m

Shear force at maximum torsion moment

$V_t = WR\,(\theta - \Phi m) = 405892.77$ N

d = 850mm   D = 900mm   b = 500mm

BM at the point of maximum torsion moment at, $\Phi = \Phi m = 25/4$

$M = M_\Phi^t = WR^2\,[\theta\sin\Phi + \theta\cot\theta\,\cos\Phi - 1] = -6647.92$ N-m

$= 6647.92$ N-m   (Hogging)

$M_T = M_t\,[(1+D/b)/1.7] = 22920.99$ N-m

$M_e = 29568.92$ N-m

Area of reinforcement for this moment is, $A_{st} = Me/\sigma_{st}.j.d = 263.84$ mm2

Minimum reinforcement = 0.3% = 1275 mm2

Provide 4bars of 20mm Φ

Equivalent shear

$Ve = (V + 1.6T/b) = 450425.00$ N

$\tau_{ve} = Ve/bd = 1.05$ N/mm^2

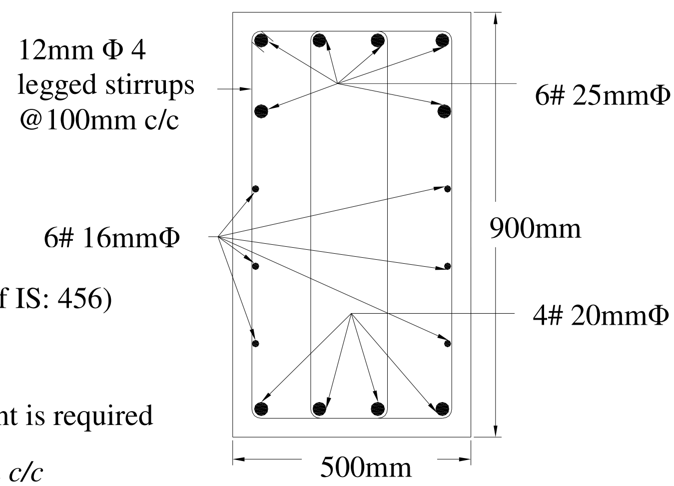
Percentage steel = 100 A_{st} /bd = 0.3%

Corresponding shear stress of concrete (Ref IS: 456)

$\tau_c = 0.236$ N/mm2

$\tau_v > \tau_c$ there fore shear reinforcement is required

Using 10mm Φ 4-legged stirrups @200mm c/c



4.2.8 DESIGN OF COLUMN

Total axial load due to upper structure = 16699588.54 N

Provide 12 columns on the periphery of 10 meter diameter. Then axial load on each column is

P = 1391632.38 N

Total overturning moment due to earthquake is 9045.31 kN-m hence over turning moment on each column will be

M = 753.77 kN-m

Use 20 bars of 30mm Φ Then area of steel $A_{st} = 14130$ mm^2

Percentage of steel = $14130 \times 100 / (\pi/4) \times 800^2 = 2.81\%$

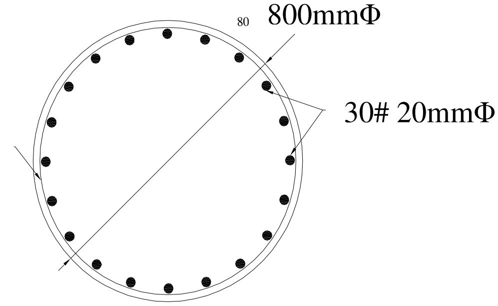
Equivalent area of column:

$A_e = \pi/4 \times D^2 + (m-1)\times A_{st} = 671960 \text{ mm}^2$

Equivalent moment of inertia:

$$I_e = \pi/64 \times D^4 + \frac{(m-1)\times A_{st}\times(d^2)}{8}$$

$$= 3.1083\times10^{10}$$



Axial compressive stress of column

$$\sigma_c' = P/Ae = 2.07 \text{ N/mm2}$$

Bending compressive stress in column

$$\sigma_{cbc}' = \frac{M\times D/2}{I_e}$$

$$= 9.7$$

$$(\sigma_c'/\sigma_c) + (\sigma_{cbc}'/\sigma_{cbc}) = 2.07/5 + 9.7/7$$

$$= 1.79 > 1.33$$

Hence increase the diameter of the column

4.2.9 DESIGN OF BRACING

Each and every junction of column and bracing, the column imposed moment on the joint. This moment has to be resisted by two bracing meeting at each joint thus the bracing are subjected to bending moment and twisting moment at each joint. Maximum BM has occurred at the lowest junction of column and bracing.

$$m = \frac{(420.137\times4 + 435.17\times4)\sin(\theta + \pi/12)\cos^2\theta}{12\sin(2\pi/12)} \quad \text{….from equation (2.33)}$$

$$= 570.2 \sin(\theta + \pi/12)\cos^2\theta$$

$$dm/d\theta = 570.2[\cos(\theta + \pi/12)\cos^2\theta - 2\sin(\theta + \pi/12)\cos\theta\sin\theta] = 0$$

$$\theta = 28$$

$$m = 420.30 \text{ kN-m}$$

Twisting moment = 5% of the maximum moment

$$T = 21.02 \text{ kN-m}$$

Using M-20 concrete ($\sigma_{cbc} = 7$ N/mm2) and HYSD bar $\sigma_{st} = 230$ N/mm2

K = 0.283 J = 1 – 0.283/3 = 0.906 R = 0.897

Depth of neutral axis = 0.283d

$A_{st} = A_{sc} = pbd$ where p is %steel and dc= 0.1d

Equating the equal moment of area

$$0.5b(kd)2 + (m-1)pbd(kd-dc) = mpbd(d-kd)$$

$$p = 0.0056$$

Since bracing is subjected to BM and twisting moment, we have

$Me = m + m^t$

$m^t = T(1 + D/b)/1.7 = 70.58$ kN-m

For calculate the depth of the bracing, equate the resisting moment caring capacity of the bracing to the external moment in the bracing due to earthquake.

$$0.5bkd\sigma_{cbc}\,[d - kd/3] + (m_c - 1)Asc\,c'(d - dc) = Me$$

$$0.5\times b\times 0.283d\times 9.31\times [d - 0.283d/3] + (19.5 - 1)\,(0.0056bd)\,6.02(d - 0.1d) = 457.39\times 10^6$$

$$1.754\,bd^2 = 457.39\times 10^6$$

$b = 400$mm $\quad d = 700$mm

Hence area of steel is, $Asc = Ast = pbd = 1568\text{mm}^2$

Provide 5 bars of 20mm Φ each top and bottom

Maximum shear force in bracing is

$$(Sb)max = \frac{(420.137\times 4 + 435.17\times 4)\times 2\cos^2(\pi/12)\,\sin(2\pi/12)}{2.588\times 12\times \sin(2\pi/12)} \quad \text{..from equation (2.34)}$$

$$= 205.56 \text{ kN}$$

$Ve = V + 1.6T/b = 289.64$ kN

$\tau_v = 1.03 < \tau_{c\,max}(=1.8)$

Shear caring capacity of the section without shear reinforcement is

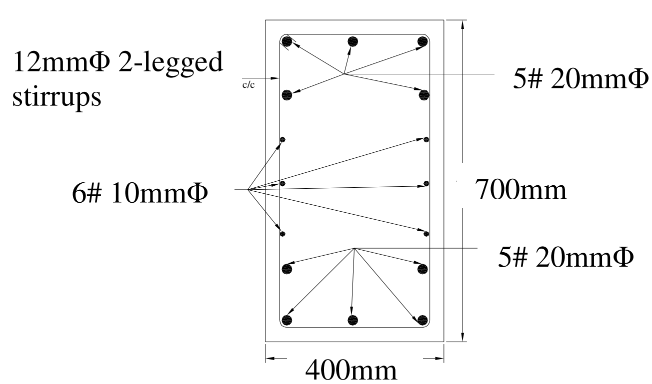
$\tau_c = 0.36$ kN/m2

τ_v is less then $\tau_{c.max}$ but more then τ_c so that transverse reinforcement is required

$$Asv = \frac{T\,Sv}{b_1 d_1\,\sigma_{sv}} + \frac{V\,Sv}{2.5d_1\,\sigma_{sv}}$$

$b_1 = 400 - 2\times 25 - 20 = 330$, $\quad d_1 = 700 - 2\times 25 - 20 = 630$

Hence provide 12mm Φ 2 legged stirrups @ 220mm c/c



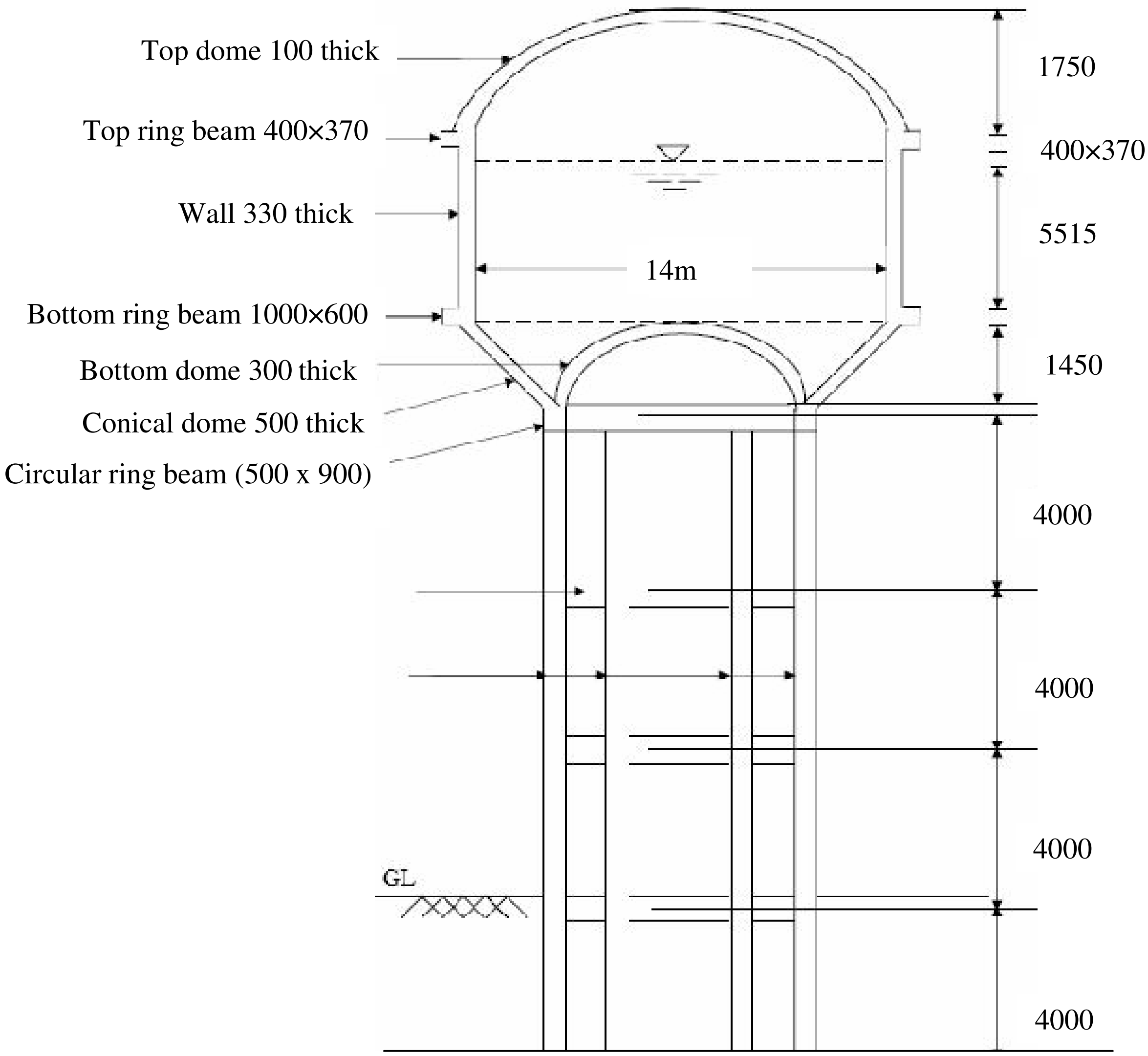


Figure 4.1 Geometric parameters of frame staging tank after considering earthquake

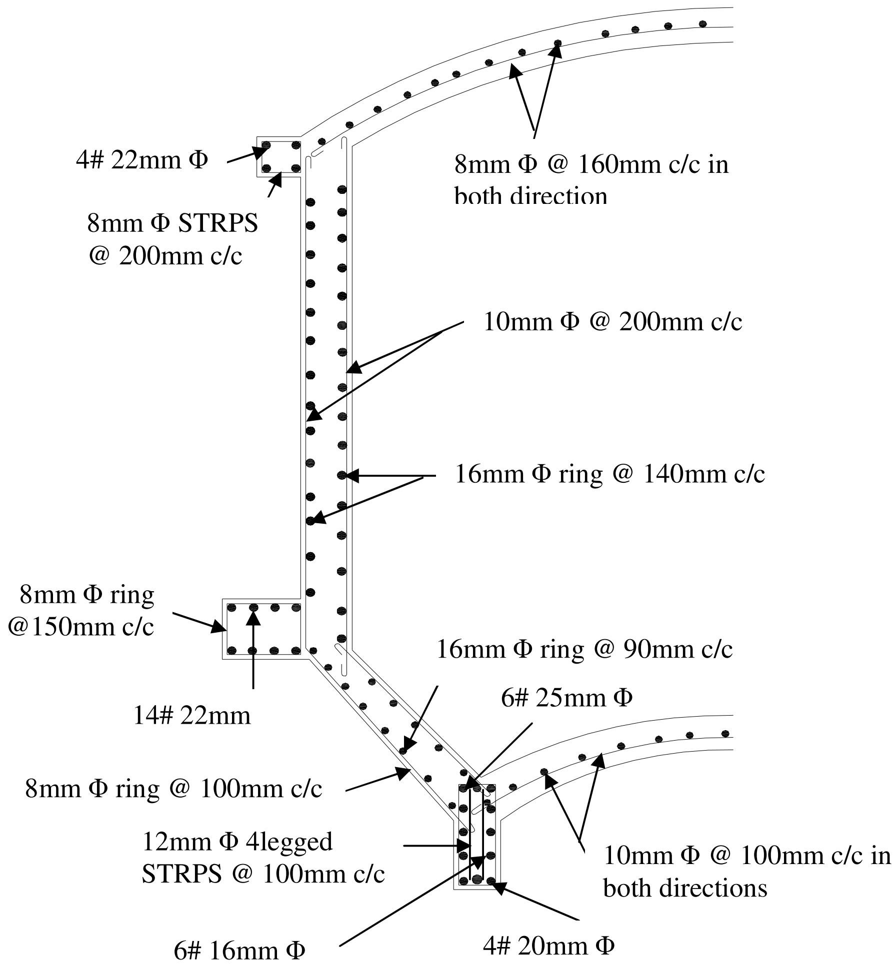


Figure 4.2 Reinforcement Detailing in Container of Frame Staging

4.3 REDESIGN OF INTZE TANK WITH SHAFT STAGING CONSIDERING EARTHQUAKE FORCES

4.3.1 TOP DOME

There is no effect of Earthquake so its design is same as conventional design.

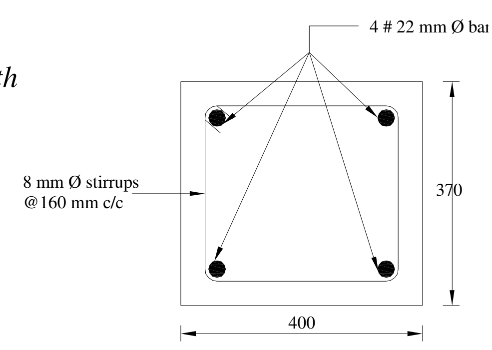
Hence provide 8 mm Φ bars @ 160 mm c/c.

4.3.2 RING BEAM B_1

There is also no effect of earthquake so its design also same as conventional design in chapter 3rd.

Provide ring beam of 370mm depth and 400mm width

Tie the 22mm Φ rings by 8mm diameter nominal stirrups @ 200mm c/c spacing.



SECTION OF TOP RING BEAM

4.3.3 DESIGN OF CYLINDRICAL WALL

During earthquake the height of sloshing wave in convective mode is reached up to 0.549m from their actual position so that water tank should have sufficient free board that's why this wave don't affect the water tank.

Actual height of the tank is h = 5.47m

Free board required = 0.549m

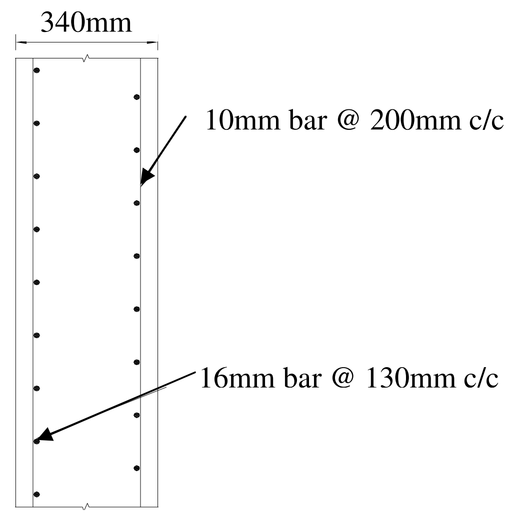
Required height of the tank is = 5.47 + 0.549 = 6.019m

Hence we provide height of the cylindrical wall is 6.1 meter

Hoop tension without considering earth quake

At top = 0

At bottom = whD/2 = 418887 N/m height



Hoop tension due to earth quake

(a) For impulsive mode

At top = 0

At bottom = 33630.94 N/m

(b) For convective mode-

At top = 5595.17 N/m

At bottom = 1926.05 N/m

Total hoop tension-

At top = 5595.17 N/m

At bottom = 452793.52 N/m

Area of steel

$A_{sh} = P_h / \sigma_{st} = 3018.62\ mm^2$ per meter height

Hence provide 16mm Φ rings @ 130mm c/c on both faces.

Actual $A_{sh} = 1546.15\ mm^2$

Permissible stress in composite section is 1.2 N/ mm^2 the

$P_h / (A + (m -1) A_{sh}) <= 1.2$

$452793.52 / (1000 \times t + 12 \times 1546.15 \times 2) <= 1.2$

t = 340.00 mm

Hence provide 340mm thickness.

Percentage distribution steel for 340mm thickness is 0.23% = 782 mm^2

Hence provide 10mm Φ bars @ 200mm c/c on each face

4.3.4 DESIGN OF MIDDLE RING BEAM B_3

This beam connects the tank wall with conical dome. The vertical load at the junction of the wall with conical dome is transferred to ring beam by meridian thrust in the conical dome. The horizontal component of thrust cause hoop tension at the junction.

The load W transmitted through tank wall at the top of conical dome is-

i) Load due to top dome = 14859.19N/m

ii) Load due to ring beam = 925N/m

iii) Load due to tank wall = 51850N/m

iv) Self weight of ring beam (assume initially size of ring beam is 1m x 0.6m)

= 25000×0.6 (1 - 0.3) = 10500N/m

Total load W =78134.19 N/m

Inclination of conical dome with vertical is Φ_0 = 45 degree

$\sin\Phi_0 = \cos\Phi_0 = 0.707$ $\tan\Phi_0 = 1$

Hoop tension due to self weight of upper structure $P_W = W.\tan\Phi_0$ = 78134.19 N/m

Pressure due to water P_w = w.h.d3 = 35868 N/m

Pressure due to earthquake $P_{eq} = (P_{iw} + P_{cw})\times d_3$ = 3048 N/m

Hence hoop tension on the ring beam $P_3 = (P_W + P_w + P_{eq}).D/2$ = 819351.33 N

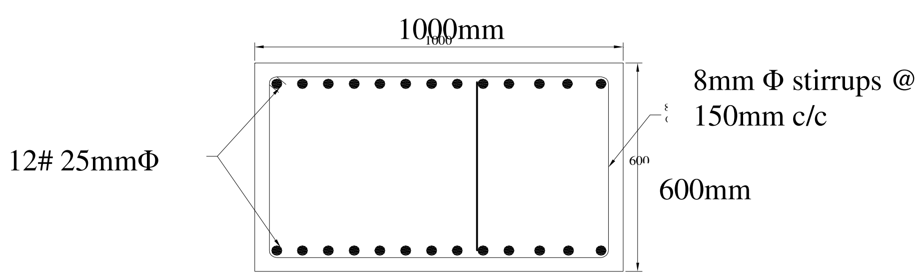
This hoop tension resisted by the steel hoops, then area of hoops is

$A_{sh} = P_3 / \sigma_{st}$ = 5462.34 mm^2

Hence provide 12 bars of 25mm Φ then actual A_{sh} = 5887.5mm^2

Stress in equivalent steel = $P_h / (A + (m-1) A_{sh})$ = 1.19 <1.2 Hence safe

Hence provide 1000mm×600mm ring beam with 12 bars of 25 mm Φ and distribution bars of 8mm Φ @150mm c/c.



Section of middle ring beam

4.3.5 DESIGN CONICAL DOME

(a) Meridian thrust

The weight of the water is

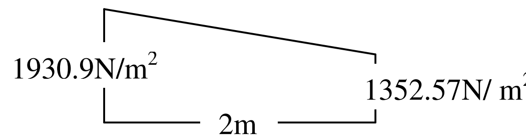
$W_w = 5202519.46$ N

Initially we assume thickness of the conical dome is 400mm then self weight is

$Ws = 1066131$ N

Weight W at ring beam B_3 due to upper element like top dome top, ring beam, cylindrical wall and self weight itself is

$W = 78134.19$ N/m



Weight due to Earthquake:

$W_{eq} = 3283.47$ N/m

Hence total vertical load W per meter run is given by

$$W = [(78134.19\times\pi\times14) + 5202519.46 + 1066131 + (3283.47\times\pi\times12.12)] / \pi\times10$$

$$= 313005.98 \text{ N/m}$$

Meridian thrust T_0 in the conical dome is

$T_0 = W / \cos\Phi_0 = 442724.16$ N/m

Meridian stress

$T_0 / t = 1.10 < 1.2$ Hence safe

(b) Hoop tension

Diameter of conical dome at any height h' above the base is

$D' = 10 + 2h'$

Intensity of water pressure

$P = 9800\times(6.1 + 2 - h') = 9800\times(8.1 - h')$ N / mm^2

Self weight q = 10000 N/ mm^2

Hence hoop tension $Po' = (P/\cos\Phi_0 + q.\tan\Phi_0)\, D'/2$

$= 604371.72 + 51577.84h' - 13859.3h'^2$

$dPo'/dh' = 51577.84 - 27718.6h' = 0$

$h' = 1.86$ m

Hence at h'=1.86m have the maximum hoop tension

Maximum $P_0' = 652358.87$ N

(c) Design of wall

Maximum hoop stress = 652358.87 N at the h'=1.86 meter above the base

Pressure due to earthquake at h'=1.86 meter

$$Peq = \frac{(Piw + Pcw)D'/2}{\cos\Phi_0}$$

$Peq = 45935.89$ N

Total hoop stress = 698294.28 N

Area of steel $A_{sh} = 4655.29$ mm^2

Hence provide 18mm Φ bar @ 100mm c/c on each face.

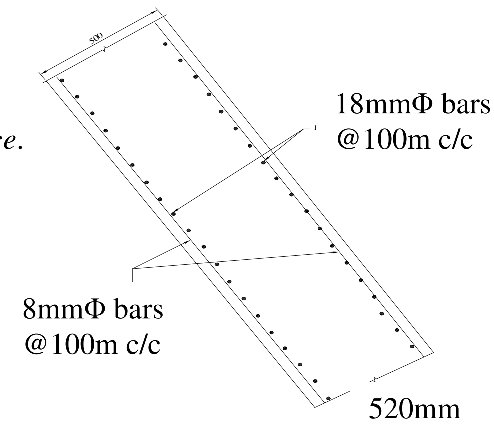
Actual $A_{sh} = 2543.4$ mm^2

Maximum tensile stress of composite section is

$P_h / (A + (m - 1) A_{sh}) = 1.2$

t = 520 mm

Hence provide 520mm thick wall of conical dome



4.3.6 DESIGN BOTTOM DOME: (Ref: N Krishna Raju)

Radius $R_2 = 8.02$ $\sin\Phi_2 = 0.62$ $\cos\Phi_2 = 0.78$

Weight of the water on the dome is

$W_0 = 5530532.0$ N

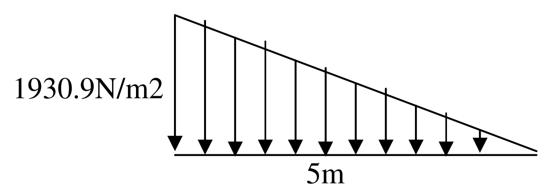
Initially we take thickness of the bottom dome is 250mm

Then self weight of bottom dome is

$W_s = 550873.75$ N

Additional weight due to earthquake

$W_{eq} = 101050.43$ N



Total weight on bottom dome is

W = 6182456.18 N

Load per unit area

$W_t = 77444.24$ N/m^2

Meridian thrust

$T_2 = W_t \times R_2 / (1 + \cos\Phi_2) = 348934.14$ N/m

Meridian stress = T_2 / t = 1.39 > 1.2

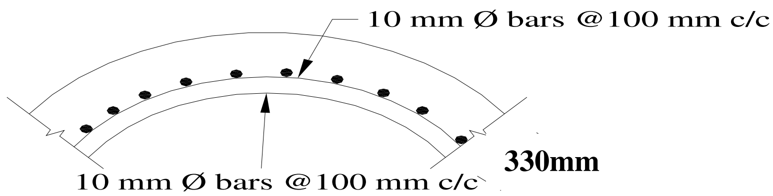
Hence increase the thickness of the dome. Hence use 300 mm thickness of the dome

Circumference force = 310551.40 N/m

Hoop stress = 310551.40 / 300×1000 = 1.03 <1.2 hence ok.

Provide nominal reinforcement for bottom dome is 0.24 % = 720 mm^2 each direction

Hence provide 10mm Φ bars @ 100mm c/c in both directions



SECTION OF BOTTOM DOME

4.3.7 DESIGN BOTTOM CIRCULAR RING BEAM B_2

In ward thrust from conical dome = $T_0 \sin\Phi_0$ = 313005.98 N/m

Out ward thrust from spherical dome = $T_2 \cos\Phi_2$ = 272168.63 N/m

Net horizontal force

$$H = T_0 \sin\Phi_0 + T_2 \cos\Phi_2 = 40837.35 \text{ N/m}$$

Hoop compression in the beam

$$= H.D/2 = 204186.75 \text{ N}$$

Hoop stress = 204186.75 / 400×600 = 0.85 <1.2 hence safe

Vertical load on ring beam = $T_0 \cos\Phi_0 + T_2 \sin\Phi_2$ = 529345.14 N/m

Self weight of ring beam = 6000 N/m

Total vertical load

$$W = 535345.14 \text{ N/m}$$

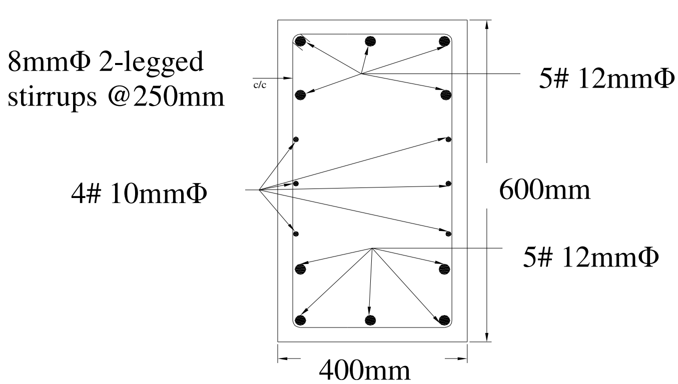
This ring beam is supported on shaft type staging that's why all vertical load is transferred to foundation threw cylindrical shaft.

This ring beam is design only for hoop compression in such case minimum reinforcement is provided.

Minimum reinforcement according IS: 456

$$A_{st} = 0.85bd/fy = 491.56 \text{ mm}^2$$

Use 5 bar of 12mm Φ



4.3.8 DESIGN SHAFT STAGING

Vertical load threw upper structure = 16818.36 kN

Initially thick ness of shaft is 220 mm

Self weight of staging = $\pi \times 10 \times 0.22 \times 16 \times 25$ = 2764.60 kN

Total vertical load on the base W = 19582.96 kN

Base shear V= 997.98 kN

Base moment M= 20767.34 kN

Eccentricity e = M/W = 1.06

<u>Vertical stress on circular shaft (Ref: IS: 11682-1985)</u>

$e/R = 0.202 < 0.5$

$\sigma_{cv} = W/2\pi Rt\ [1 + 2e/R]$

$= 4036.79\ kN/m^2 = 4.036\ N/mm^2$

Maximum permissible value = $0.4\sigma cv$ = 3.74 N/mm^2

Maximum permissible vertical stress is greater then the allowable stress so that thickness of circular shaft has increased and redesign of shaft

Now take thickness of shaft is 300mm

Self weight of circular shaft is = 3769.91 kN

Total vertical load on the base W = 20588.27 kN

Base shear V= 997.98 kN

Base moment M= 20767.34 kN

Eccentricity e = M/W = 1.01

Vertical stress on circular shaft (Ref: IS: 11682-1985)

$e/R = 0.202 < 0.5$

$\sigma cv = W/2\pi Rt\ [1 + 2e/R]$

$= 3068.57 kN/m^2 = 3.07\ N/mm^2$

Maximum permissible value = $0.4\sigma cv$ = 3.74

Permissible stress is greater then the allowable stress hence it's ok

Assume percentage steel is 1% of area of concrete

Area of steel

$Ast = 0.01 \times \pi/4[10.3^2 - 9.7^2] \times 10^6 = 94200\ mm^2$

Hence provide 20 mm Φ bars @200mm c/c on both face

Actual Ast = 98596 mm^2

Percentage of steel p = $98596 \times 100/\ \pi/4[10.3^2 + 9.7^2] \times 10^6 = 1.05\%$

Tensile stress in steel (Ref: IS: 11682-1985)

$$\sigma sv = m\ \sigma cv \frac{[1 - \cos\alpha]}{[1 + \cos\alpha]}$$

α = one half the central angle subtended by neutral axis as a chord on the circle of radius R, in degree and it's calculate by following equation using trial method.

$$e/R = \frac{0.5(1-p)(\alpha - \sin\alpha \cos\alpha) - 0.5(1 - p + mp)(\beta + \sin\beta \cos\beta - 2\cos\alpha \sin\beta) + 0.5\pi mp}{(1 - p)(\sin\alpha - \alpha\cos\alpha) - (1 - p + mp)(\sin\beta - \beta\cos\beta) + \pi mp\cos\alpha}$$

Here e/R = 0.202, β = 0, p = 0.01, m = 13

Then α = 82

$\sigma sv = 30.15\ N/mm^2$

Maximum permissible stress is = 0.60× (1.33×230) = 184 N/mm2

Permissible stress is greater then the allowable stress hence ok

<u>Stress in horizontal reinforcement</u>

Horizontal steel (Hoop steel) is used 0.4% of sectional area

Then area of steel per meter height = 0.004×300×1000 = 1200 mm^2

Hence provide 12mm Φ bar @180mm c/c on both faces and cover is 30mm

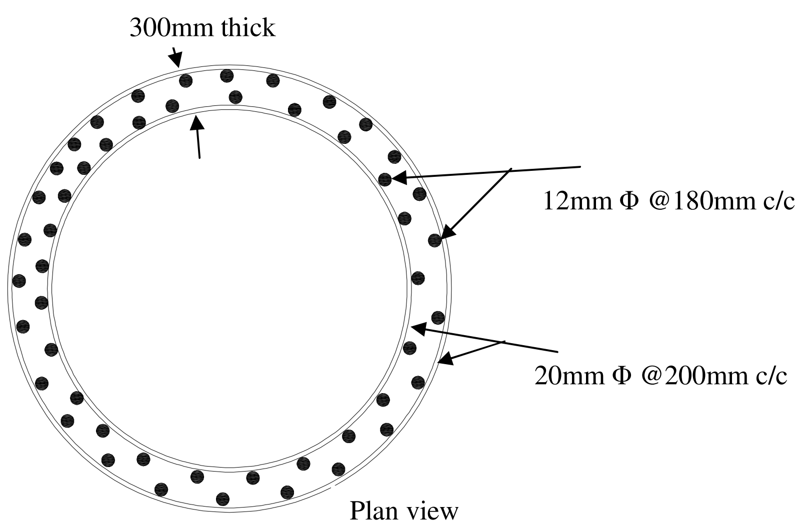


Figure 4.3 Reinforcement Detailing of Shaft Staging

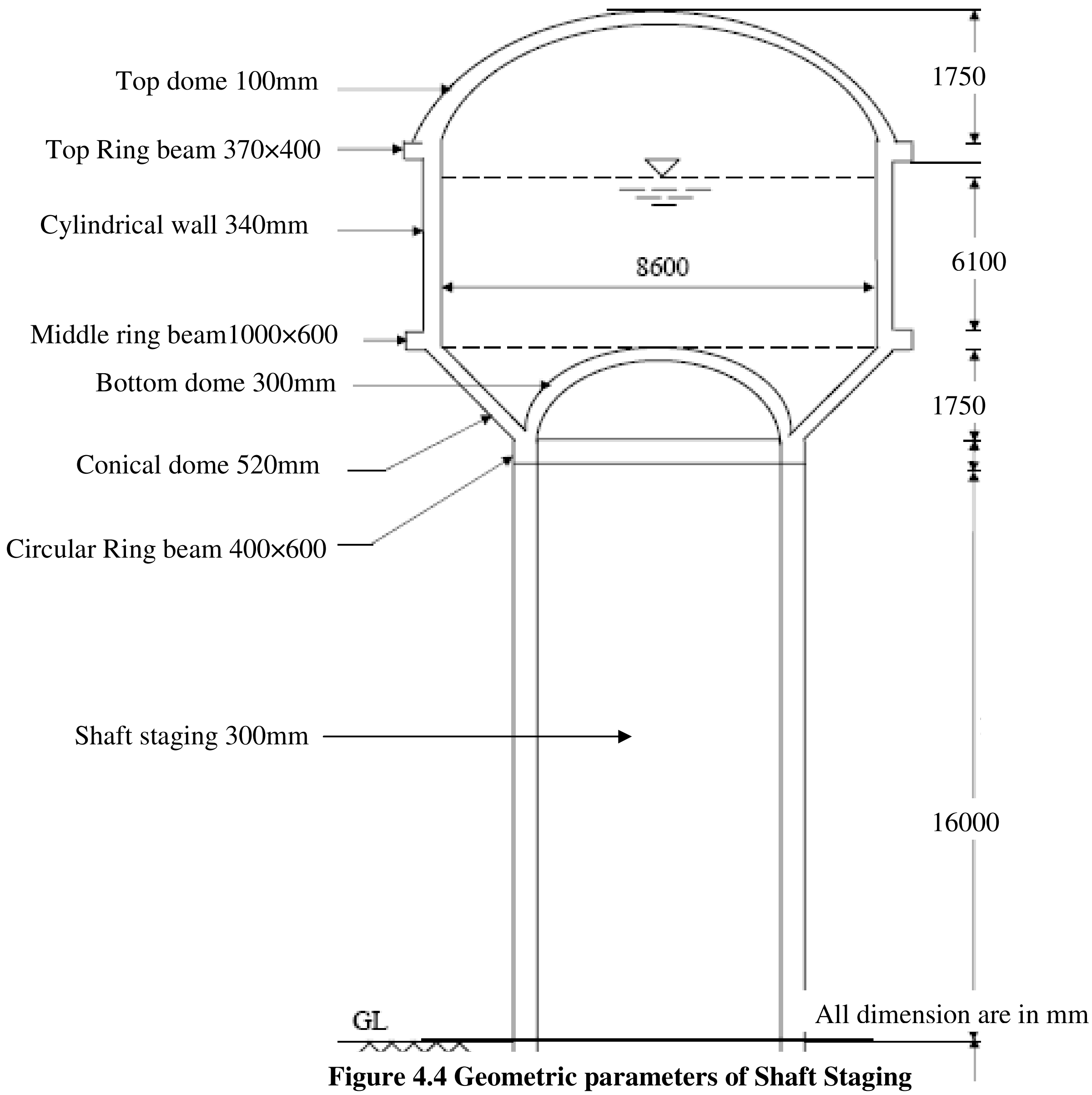


Figure 4.4 Geometric parameters of Shaft Staging Tank after considering earthquake

Chapter 5

Pushover analysis

Push over Analysis

The non-linear static procedure or simply push over analysis is a simple option for estimating the strength capacity in the post-elastic range. This procedure involves applying a predefined lateral load pattern which is distributed along the structure height.

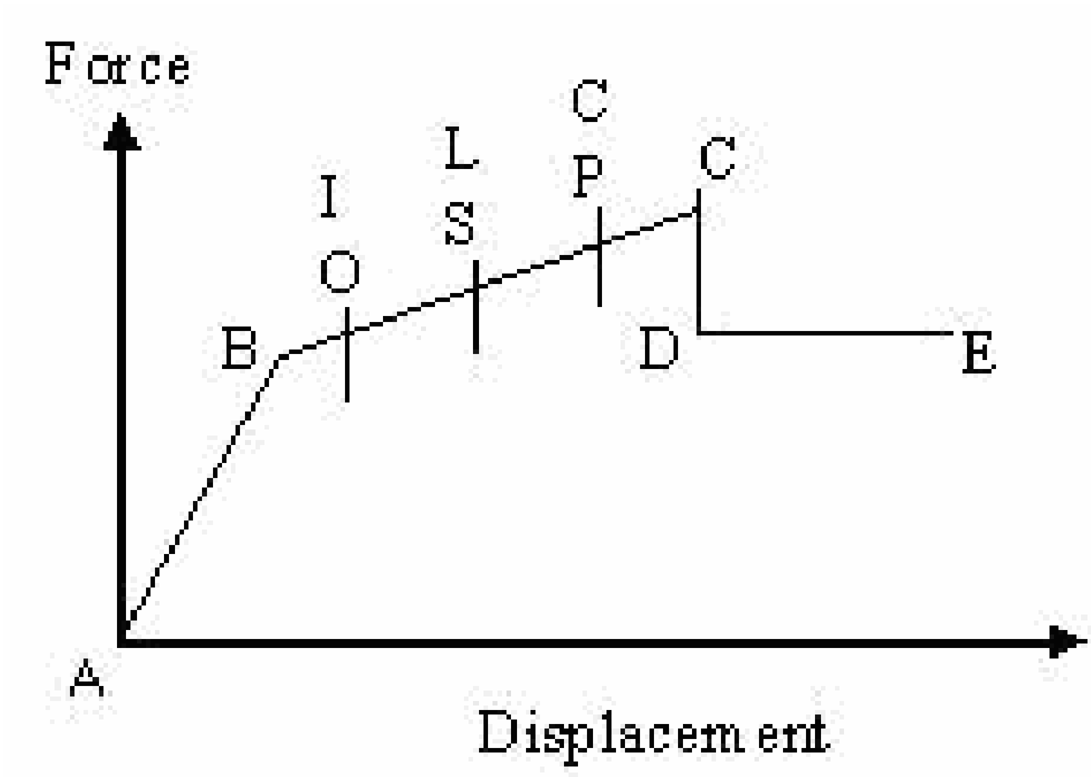
The lateral forces are then monotonically increased in constant proportion with a displacement control node of the building until a certain level of deformation is reached. The applied base shear and the associated lateral displacement at each load increment are plotted. Based on the capacity curve, a target displacement which is an estimate of the displacement that the design earthquake will produce on the building is determined. The extent of damage experienced by the building at this target displacement is considered representative of the damage experienced by the building when subjected to design level ground shaking

The most frequently used terms in pushover analysis as given in ATC-40 are.

5.11 Capacity curve

It is the plot of the lateral force V on a structure, against the lateral deflection d, of the roof of the structure. This is often referred to as the ‘push over’ curve. Performance point and location of hinges in various stages can be obtained from pushover curves as shown in the fig. The range AB is elastic range, B to IO is the range of immediate occupancy IO to LS is the range of life safety and LS to CP is the range of collapse prevention.

Fig 1.3 Different stages of plastic hinge



5.12Capacity-spectrum

It is the capacity curve transformed from shear force vs. roof displacement (V vs. d) coordinates into spectral acceleration vs. spectral displacement (Sa vs. Sd) coordinates.

5.13 Demand

It is a representation of the earthquake ground motion or shaking that the building is subjected to. In nonlinear static analysis procedures, demand is represented by an estimation of the displacements or deformations that the structure is expected to undergo.

This is in contrast to conventional, linear elastic analysis procedures in which demand is represented by prescribed lateral forces applied to the structure.

5.14 Demand spectrum

It is the reduced response spectrum used to represent the earthquake ground motion in the capacity spectrum method.

5.15 Displacement-based analysis

It refers to analysis procedures, such as the non linear static analysis procedures, whose basis lies in estimating the realistic, and generally inelastic, lateral displacements or deformations expected due to actual earthquake ground motion. Component forces are then determined based on the deformations.

5.16 Elastic response spectrum

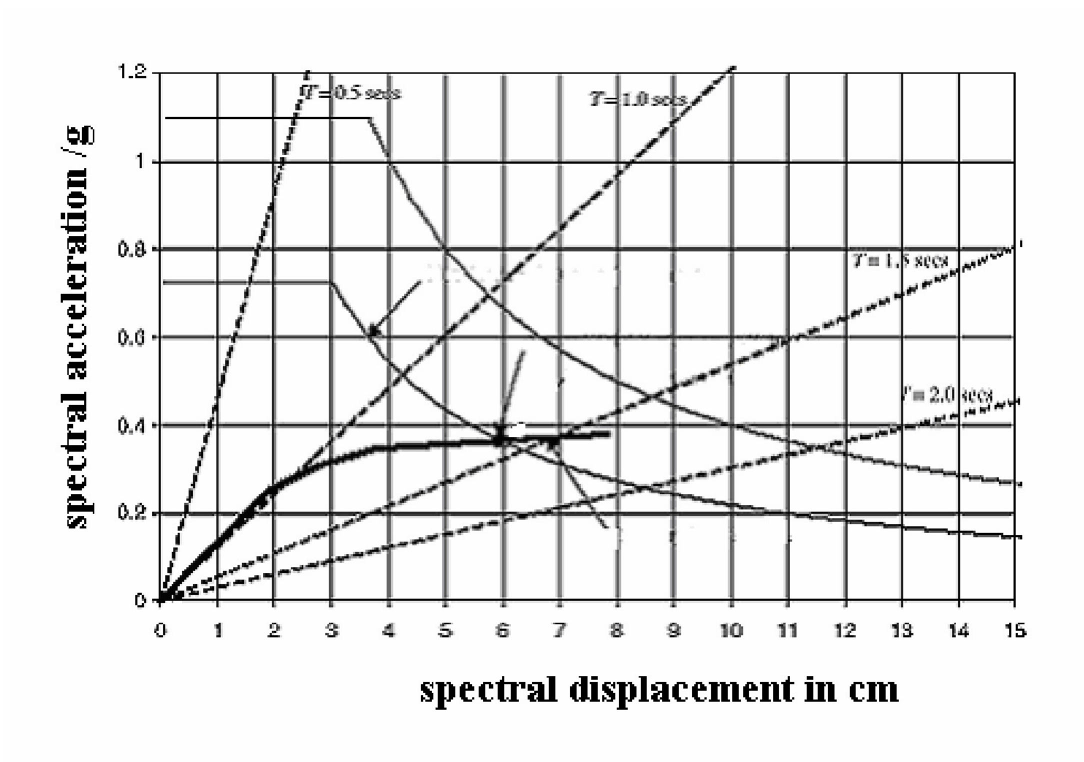
It is the 5% damped response spectrum for the (each) seismic hazard level of interest, representing the maximum response of the structure, in terms of spectral acceleration Sa, at any time during an earthquake as a function of period of vibration T.

5.17 Performance level

A limiting damage state or condition described by the physical damage within the building, the threat to life safety of the building's occupants due to the damage, and the post earthquake serviceability of the building. A building performance level is that combination of a structural performance level and a nonstructural performance level

5.18 Performance point

The intersection of the capacity spectrum with the appropriate demand spectrum in the capacity spectrum method (the displacement at the performance at the performance point is equivalent to the target displacement in the coefficient method). To have desired performance, every structure has to be designed for this level of forces. Desired performance with different damping ratios have been shown below:



Determination of performance point

5.19 Yield (effective yield) point

The point along the capacity spectrum where the ultimate capacity is reached and the initial linear elastic force-deformation relationship ends and effective stiffness begins to decrease.

5.20 Structure Performance levels

A performance level describes a limiting damage condition which may be considered satisfactory for a given structure and a given ground motion. The limiting condition is described by the physical damage within the building, the threat to life safety of the building's occupants created by the damage, and the post earthquake serviceability of the building.

5.21 Immediate occupancy

The earthquake damage state in which only very limited structural damage has occurred. The basic vertical and lateral forces resisting systems of the building retain nearly all of their pre- earthquake characteristics and capacities. The risk of life threatening injury from structural failure is negligible.

5.22 Life safety

The post-earthquake damage state in which significant damage to the structure may have occurred but in which some margin against either total or partial collapse remains. Major structural components have not become dislodged and fallen, threatening life safety either within or outside the building. While injuries during the earthquake may occur, the risk of life threatening injury from structural damage is very low. It should be expected that extensive structural repairs will likely be necessary prior to reoccupation of the building, although the damage may not always be economically repairable.

5.23 Collapse prevention level

This building performance level consists of the structural collapse prevention level with no consideration of nonstructural vulnerabilities, except that parapets and heavy appendages are rehabilitated.

5.24 **Primary elements**

Refer to those structural components or elements that provide a significant portion of the structure's lateral force resisting stiffness and strength at the performance point.

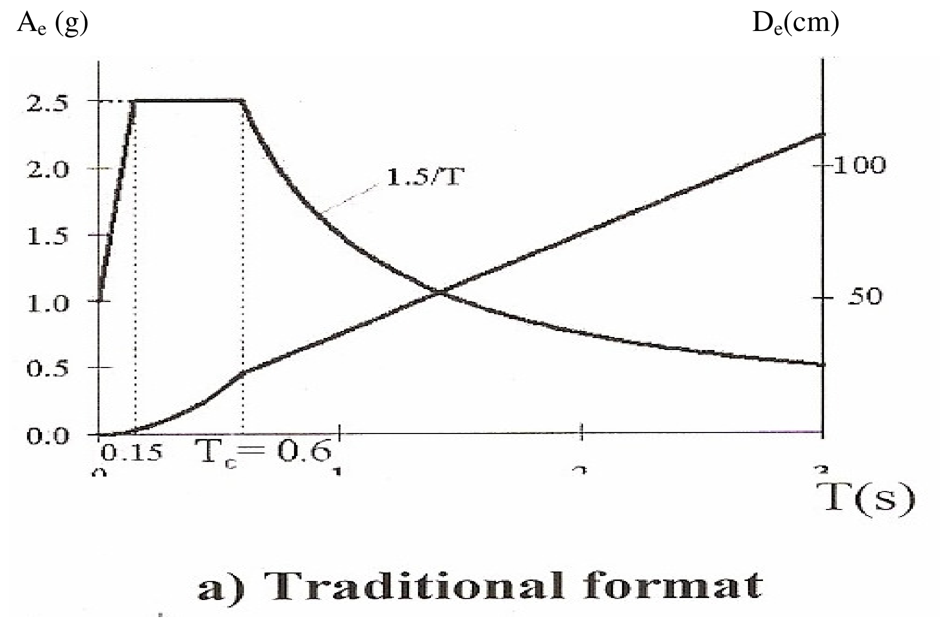
These are the elements that are needed to resist lateral loads after several cycles of inelastic response to the earthquake ground motion.

5.25 Secondary elements

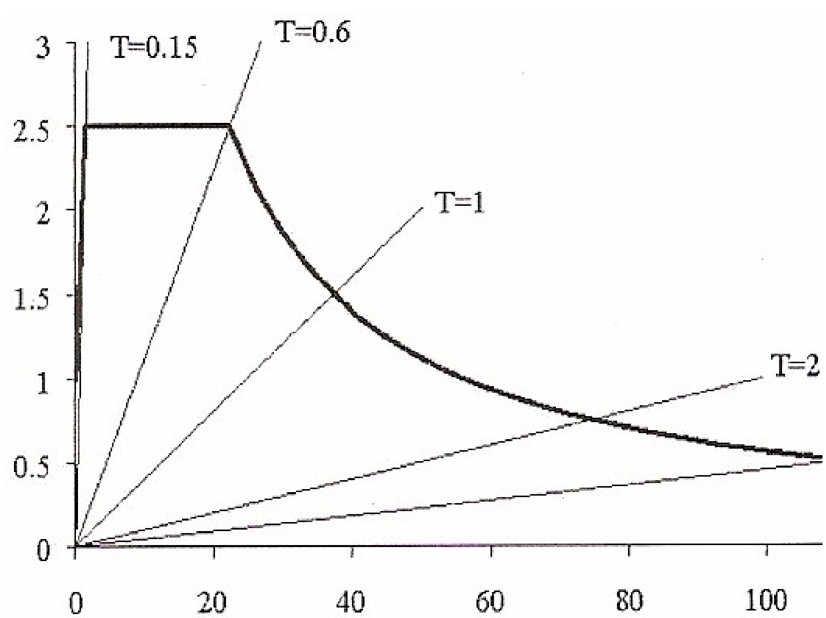
Refer to those structural components or elements that are not, or are not needed to be, primary elements of the lateral load resisting system. However, secondary elements may be needed to support vertical gravity loads and may resist some lateral loads.

SEISMIC DEMAND IN A-D FORMAT

Starting from the acceleration spectrum, we will determine the inelastic spectra in acceleration –displacement (AD) format. For an elastic SDOF system, the following relationship is applies-



Traditional Format



Typical elastic acceleration A_e and displacement spectrum D_e for 5% damping normalized to 1.0g peak ground acceleration (Peter Fajfar, M.EERI)

Where A_e =elastic acceleration,

D_e =Elastic displacement spectrum,

Corresponding to the period T and a fixed viscous damping ratio. A typical smooth elastic acceleration spectrum for 5% damping, normalized to a peak ground acceleration of 1.0g, and the corresponding elastic displacement spectrum.

For an inelastic SDOF system with a bilinear force-deformation relationship, the acceleration spectrum (A_I) and the displacement spectrum (D_I)can be determined as-

$$A_I = \frac{A_e}{R_y}$$

$$D_I = \frac{\mu}{R_\mu} D_e$$

$R_y - \mu - T_n$ Equations:

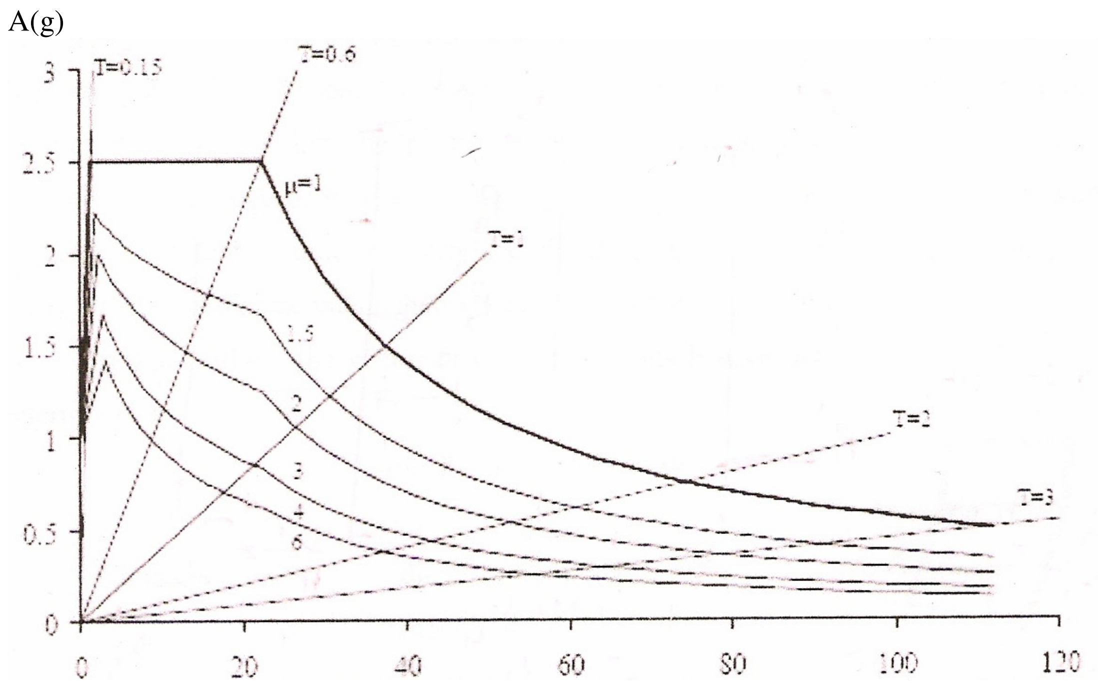
$$R_y = \begin{cases} 1 & T_n < T_a \\ \sqrt{2\mu - 1} & T_b < T_n < T'_c \\ \mu & T_n > T_c \end{cases}$$

Where Yield strength reduction factor= $R_y = \frac{f_0}{f_y} = \frac{u_0}{u_y}$

Ductility factor= $\mu = \frac{u_m}{u_y}$

T_c =Characteristic period of the ground motion (it is typically defined as the transition period where the constant acceleration segment of the response spectrum passes to the constant velocity segment of the spectrum.)

Starting from the elastic design spectrum the demand spectra (for the constant ductility factor (μ)) in A-D format can be obtained



Demand spectra for constant ductility in AD format normalized to 1.0g peak ground acceleration (Peter Fajfar, M.EERI)

SAP2000 FEATURES AND CAPABILITIES

SAP2000 represent the most sophisticated and user friendly release of the SAP series of computer programs. SAP2000 is a full-featured program which can be used for simplest problem or the most complex projects. SAP analyzes and designs the structure using model that is defined in the graphical user interface. The model consists primarily of the following types of component:

- Units
- Objects
- Groups
- Coordinates system and grids
- Properties
- Load cases functions
- Analysis case
- Combinations
- Design settings
- Output and display definitions

In general, a solution of any structure may be broken in to the following three stages, as given under.

- **Preprocessing**: In this step of analysis, the element type is selected. Properties are assigned to different parts of the structure. Thereafter, modeling of geometry is carried out.
- **Solution**: In this step, first of all analysis type is defined. The analysis type may be static, modal or harmonic etc. and displacement constraints and loads are applied on the modal according to the desired boundary conditions.
- **Post-processing**: In this step, the deformed shape of the sandwich beam is plotted and the nodal solution at the required position is listed. Plotting of graph is carried out to interpret the results.

5.26 ELEMENTS DESCRIPTION

- **Point objects:** Joint objects: these are automatically created at the corner or ends of all other types of objects below, and they can be explicitly added to model support or other localized behavior.
- **Line object:** Frame/cable objects: used to model beam, column, braces, trusses and/or cable members.
- **Area objects:** Used to model walls, floors, and other thin walled members, as well as two-dimensional solid

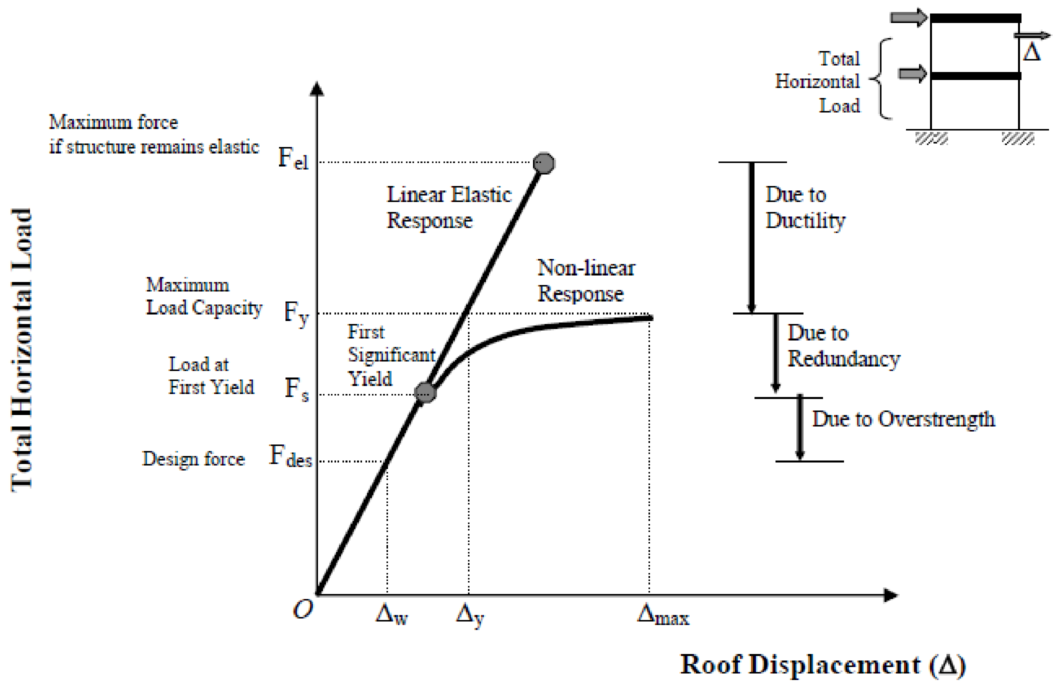
5.3 MODELING OF ELEVATED WATER TANK IN SAP2000

Here the response spectrum analysis is performed for the water tank by using the software package SAP2000. Special care needs to be taken while modeling water tank in SAP2000 in defining properties and orientation of various elements. It is also necessary to choose the proper element type and specify ply orientation. Here line elements are used for modeling the ring beam, bracing, column and area elements (shell) are used for top dome, bottom dome, cylindrical wall and shaft type staging.

<u>Steps followed for modeling the staging and tank container SAP2000</u>:

- Define element type: Here frame/cable type element is used for ring beam, bracing, column and area element (shell) is used for top dome, bottom dome, cylindrical wall and shaft type staging.
- Define Material properties: Here we provide the properties of the beam and shell such as elastic modulus, shear modulus, Poisson's ratio, weight density, etc
- Define sections: Frame sections define the width and depth for line element and area sections define thickness for shell element.
- Modeling geometry: Here water tank geometry is model by using grid system.
- Apply loads and boundary condition: Define menu provides option for specifying the boundary conditions and loads.
- Deflection results: The solution is obtained using the display option in main menu.

5.4 EVALUATION STRENGTH AND DUCTILITY



Seismic demands are computed by nonlinear static analysis of the structure subjected to monotonically increasing forces until a predetermined target displacement is reached.

More recently, a modal pushover analysis procedure based on structural dynamics theory has been developed (Chopra and Goel 2002), where seismic demands due to individual terms in the modal expansion of the effective earthquake forces are determined by a pushover analysis using the inertia force distributions associated with each mode up to a ''modal'' target displacement. The target roof displacement in all of these pushover procedures is determined from the peak deformation of an inelastic single-degree-of-freedom (SDF) System with its force-deformation relation defined from the pushover curve.

Nonlinear static pushover analysis capabilities are provided in the nonlinear version of SAP2000. The nonlinear behavior occurs in discrete user-defined hinges. Currently, hinges can be introduced into frame objects only and assigned at any location along the frame object. Uncoupled moment, torsion, axial force and shear hinges are available.

There is also a coupled P-M2-M3 hinge that yields based on the interaction of axial force and bending moments at the hinge location. More than one type of hinge can exist at the same location; for example, both an M3 (moment) and a V2 (shear) hinge may be assigned to the same end of a frame object.

5.4.1 FRAME STAGING TANK WITHOUT EARTHQUAKE CONSIDERATION

1. Geometric parameters for frame staging tank without considering earthquake:

Component	**Size (mm)**	**Component**	**Size (mm)**
Top Dome	100 thick	Bottom dome	250 thick
Top Ring Beam B_1	370×400	Bottom Ring Beam B_2	500×900
Cylindrical Wall	300 thick	Column	800 dia.
Middle Ring Beam B_3	1000×600	Bracing	300×600
Conical dome	400 thick		

2. Material properties of concrete:

Mass per unit volume = 2.54 Kg/m^3 Poisson's ratio = 0.17

Weight per unit volume = 25 kN/m^3

Modulus of elasticity = 2.236×10^7 kN/m

Concrete compressive strength fc = 20×10^3kN/m^3

Bending Reinf. Yield stress fy = 415×10^3 kN/m^3

3. Loading:

Self weight of each component

Water pressure

4. Modeling:

3D Geometric modeling of intze tank with frame staging is shown in (figure 5.2)

5. Computational value of intze tank with frame staging through software package SAP

Table 5.1 Modal Periods and Frequencies for frame staging without considering earthquake

StepType	StepNum	Period	Frequency	CircFreq	Eigenvalue
Text	Unitless	Sec	Cyc/sec	rad/sec	rad^2/sec^2
Mode	1	0.722142	1.3848	8.7008	75.703
Mode	2	0.331498	3.0166	18.954	359.25
Mode	3	0.223748	4.4693	28.082	788.57
Mode	4	0.109811	9.1066	57.218	3273.9
Mode	5	0.109811	9.1066	57.218	3273.9
Mode	6	0.107618	9.2922	58.384	3408.7
Mode	7	0.088541	11.294	70.963	5035.8

Mode	8	0.087621	11.413	71.709	5142.1
Mode	9	0.087621	11.413	71.709	5142.1
Mode	10	0.043081	23.212	145.85	21271
Mode	11	0.03457	28.927	181.75	33034
Mode	12	0.026196	38.174	239.85	57529

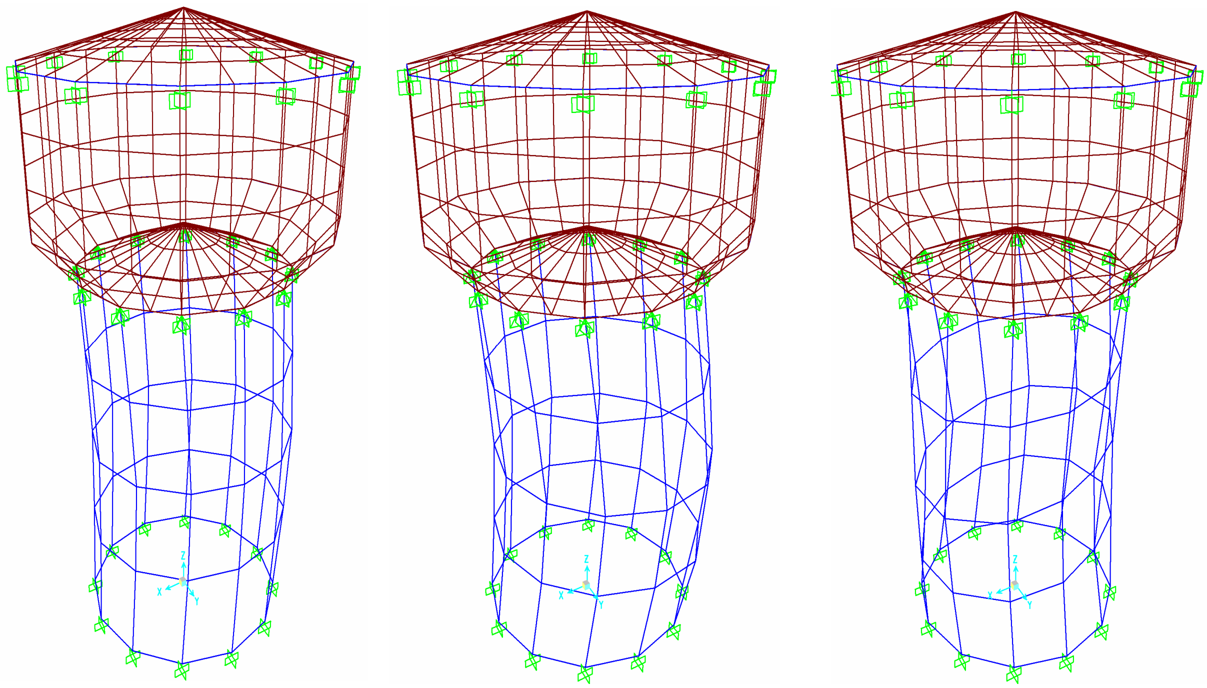


Figure 5.1 Mode shape of frame staging tank

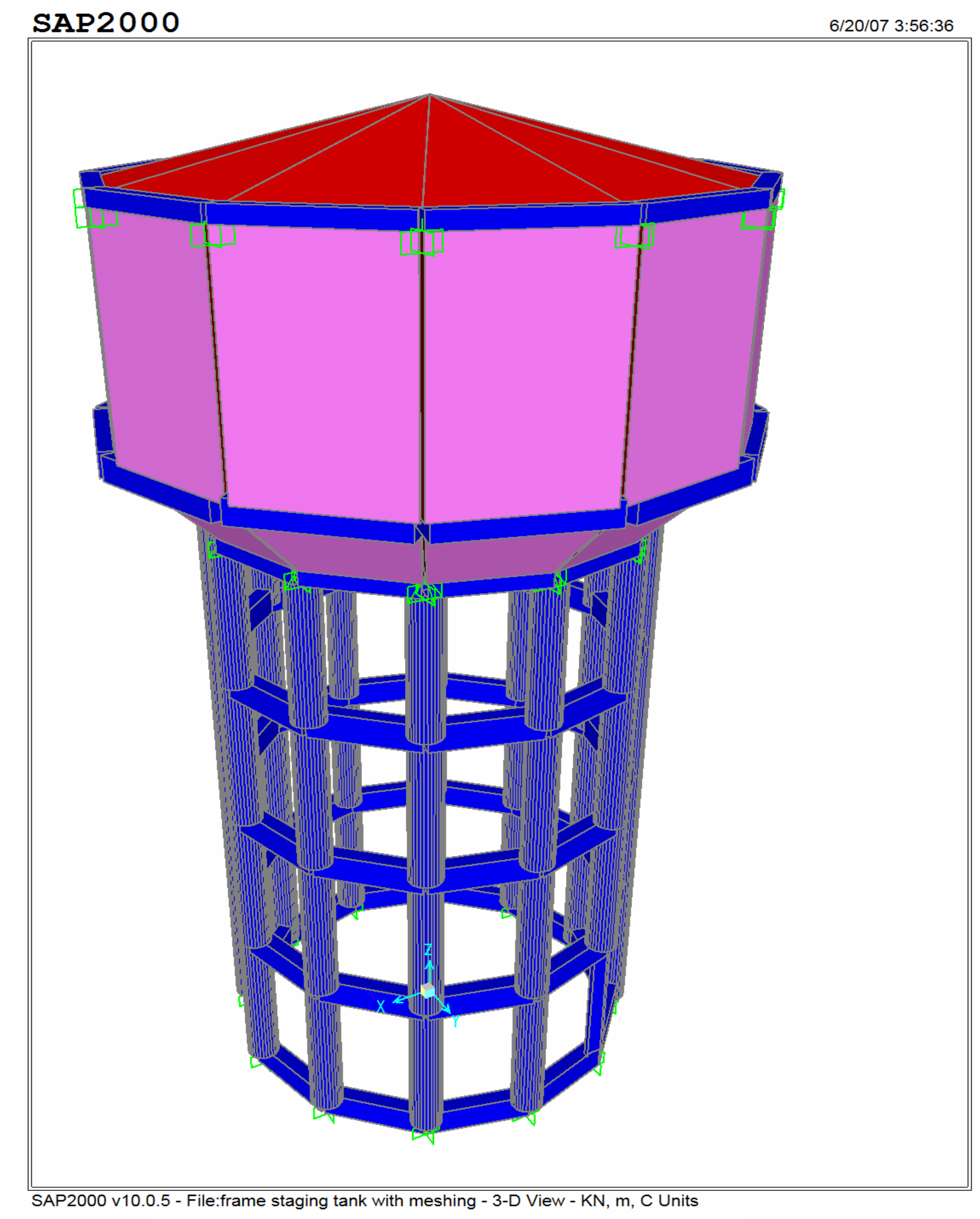


Figure 5.2 Intze tank supported on Frame Staging

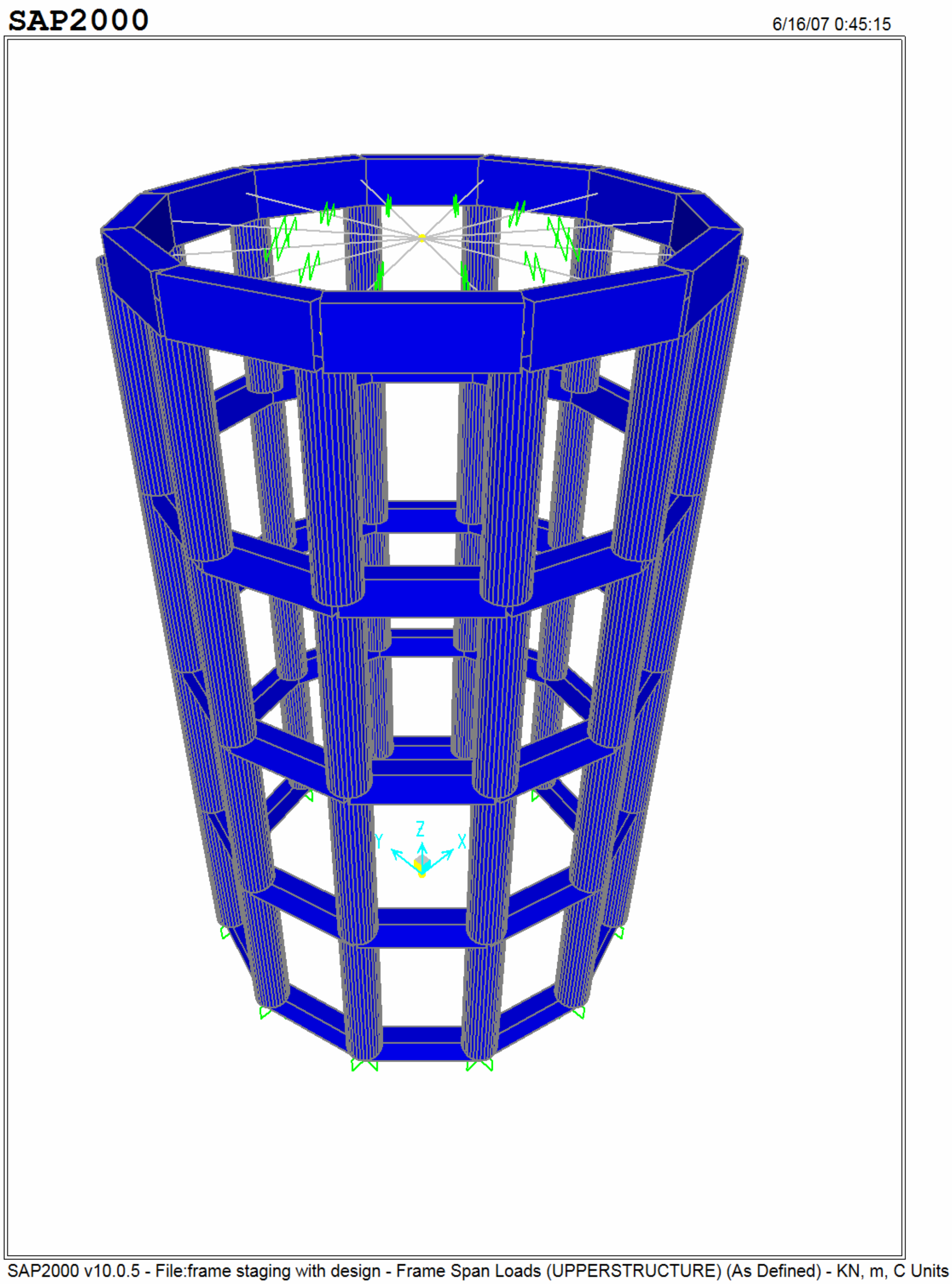


Figure 5.3 Frame staging

6. Evaluation of strength and ductility of Frame Staging without considering earthquake:

For evaluation of strength and ductility of frame staging, only staging portion has modeled in SAP. Here container portion is quite rigid there fore rigid link is assumed from top of staging to make rigidity. Mass of the rigid link has taken zero and stiffness is quite high compare to the other member of the staging. 3D modal of frame staging is shown in (figure 5.3). Strength and ductility of frame staging is evaluated by static nonlinear pushover analysis. Pushover analysis provides a curve between base shear and deflection.

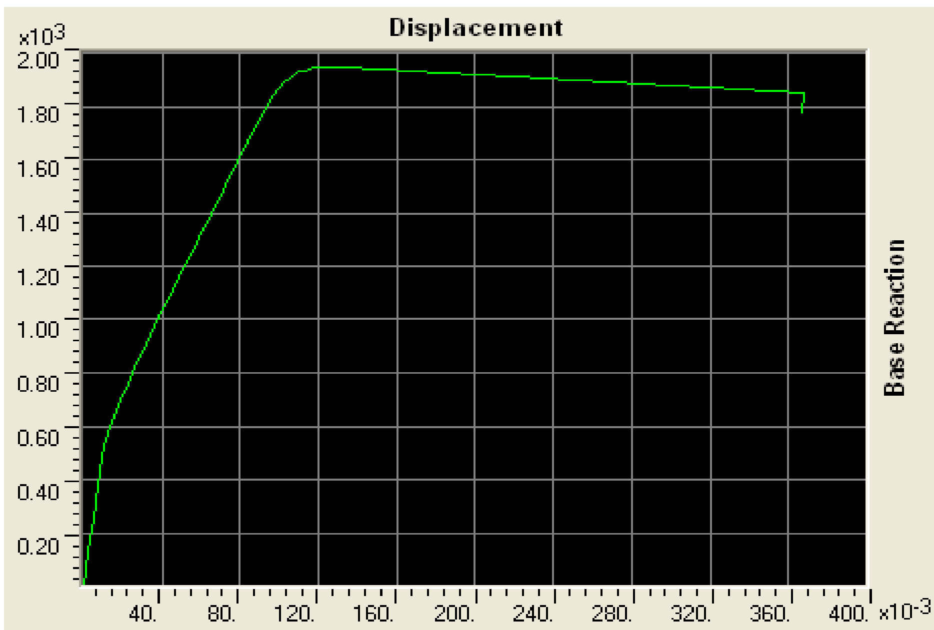


Figure 5.4 Pushover Curve for Frame Staging without Considering Earthquake

Ductility $\mu = \Delta_{ultimate}/\Delta_{yield}$

Where

$\Delta_{ultimate}$ = deflection at ultimate point = 368×10^{-3}

Δ_{yield} = deflection at yield point = 106×10^{-3}

Hence ductility of frame staging without considering earthquake is $\mu = 3.47$ and strength of frame staging is 1.89×10^3 kN

Different steps of forming plastic hinge in frame staging are shown in figure 5.4

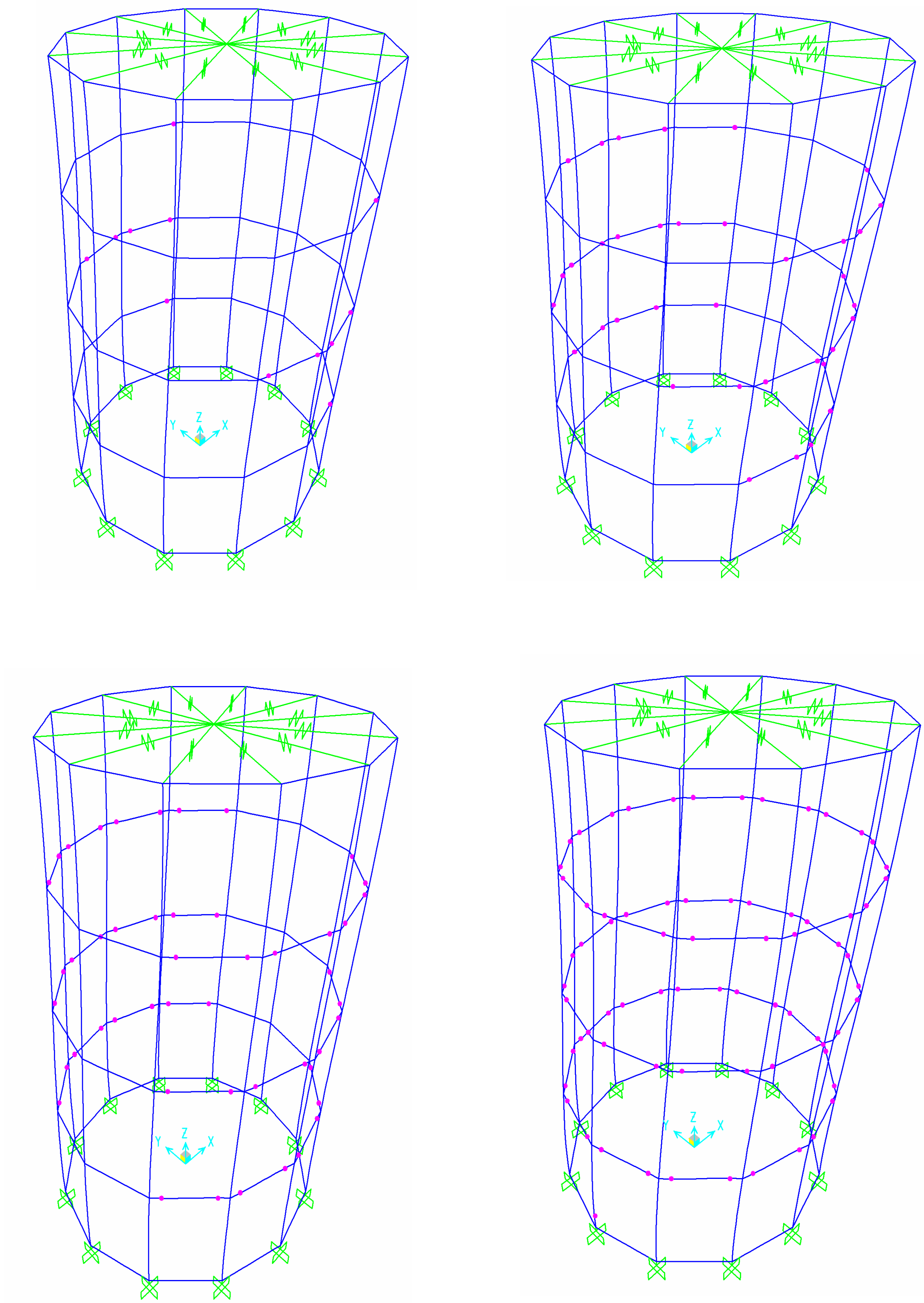


Figure 5.5 Different Steps of Forming Plastic Hinge in frame staging without Earthquake

5.4.2 FRAME STAGING TANK CONSIDERING EARTHQUAKE

1. Geometric parameters for frame staging tank with considering earthquake:

Component	Size (mm)	Component	Size (mm)
Top Dome	100 thick	Bottom dome	300 thick
Top Ring Beam B_1	370×400	Bottom Ring Beam B_2	500×900
Cylindrical Wall	330 thick	Column	800 dia.
Middle Ring Beam B_3	1000×600	Bracing	400×700
Conical dome	500 thick		

2. Material properties of concrete:

Mass per unit volume = 2.54 Kg/m^3 Poisson's ratio = 0.17

Weight per unit volume = 25 kN/m^3

Modulus of elasticity = 2.236×10^7 kN/m

Concrete compressive strength fc = $20 \times 10^3 kN/m^3$

Bending Reinf. Yield stress fy = 415×10^3 kN/m^3

3. Loading:

Self weight of each component

Water pressure

Earthquake forces

4. Modeling:

3D Geometric modeling of intze tank with frame staging is shown in (figure 5.2)

5. Computational value of intze tank with frame staging through software package SAP.

Table 5.2 Modal Periods and Frequencies for frame staging with considering Earthquake

StepType	StepNum	Period	Frequency	CircFreq	Eigenvalue
Text	Unitless	Sec	Cyc/sec	rad/sec	rad2/sec2
Mode	1	0.376834	2.6537	16.674	278.01
Mode	2	0.272825	3.6653	23.03	530.38
Mode	3	0.190511	5.249	32.981	1087.7
Mode	4	0.097717	10.234	64.3	4134.5
Mode	5	0.097486	10.258	64.452	4154.1
Mode	6	0.090651	11.031	69.312	4804.2
Mode	7	0.079357	12.601	79.176	6268.9
Mode	8	0.068598	14.578	91.594	8389.5
Mode	9	0.068206	14.662	92.121	8486.3
Mode	10	0.060749	16.461	103.43	10698
Mode	11	0.047018	21.269	133.63	17858
Mode	12	0.046944	21.302	133.84	17914

6. Evaluation of strength and ductility of Frame Staging:

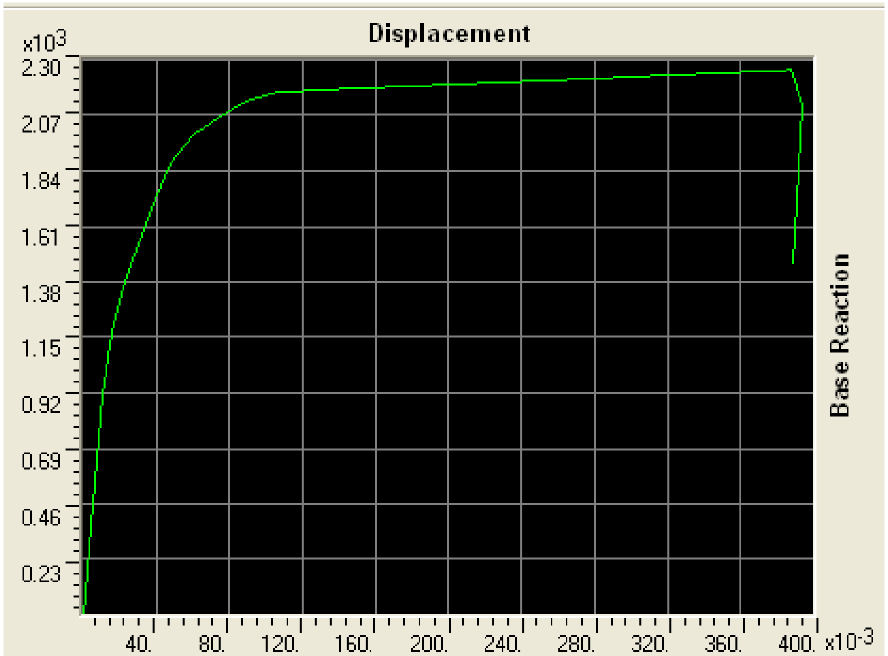


Figure 5.6 Pushover Curve for Frame staging with considering earthquake

Ductility $\mu = \Delta_{ultimate}/\Delta_{yield}$

Where

$\Delta_{ultimate}$ = deflection at ultimate point = 390×10^{-3}

Δ_{yield} = deflection at yield point = 80×10^{-3}

Hence ductility of frame staging with considering earthquake forces is $\mu = 4.9$ and strength of frame staging is 2.16×10^{3} kN

During push over analysis initially plastic hinges are formed in bracing near the ground and after that in fifth steps plastic hinge are formed in bottom most column. Forming of plastic hinge are shown in figure 5.7

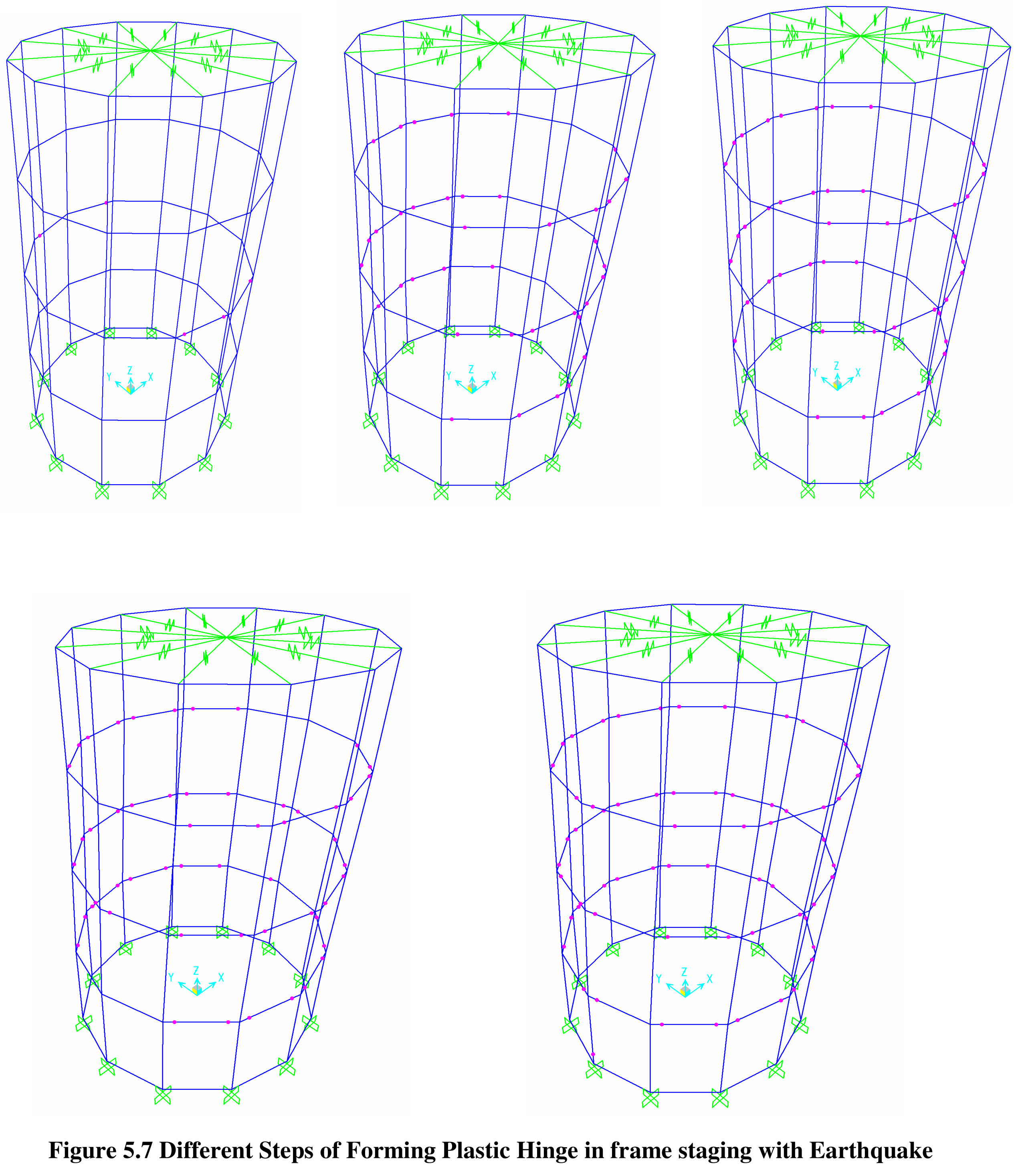


Figure 5.7 Different Steps of Forming Plastic Hinge in frame staging with Earthquake

5.4.3 SHAFT STAGING TANK WITHOUT CONSIDERING EARTHQUAKE

1. Geometric parameters for Shaft staging tank without considering earthquake:

Component	**Size (mm)**	**Component**	**Size (mm)**
Top Dome	100 thick	Conical dome	400 thick
Top Ring Beam B1	370×400	Bottom dome	250 thick
Cylindrical Wall	300 thick	Circular Ring Beam B2	400×600
Middle Ring Beam B3	1000×600	Shaft Staging	220 thick.

2. Material properties of concrete:

Mass per unit volume = 2.54 Kg/m^3

Weight per unit volume = 25 kN/m^3

Modulus of elasticity = 2.236×10^7 kN/m

Poisson's ratio = 0.17

Concrete compressive strength fc = $20\times10^3 kN/m^3$

Bending Reinf. Yield stress fy = 415×10^3 kN/m^3

3. Loading:

Self weight of each component v

Water pressure

4. Modeling:

3D Geometric modeling of intze tank with shaft staging is shown in figure 5.8

5. Computational value of intze tank with frame staging through software package SAP.

Table 5.3: Modal Periods and Frequencies for shaft staging without considering earthquake

Step Type	**Step Num**	**Period**	**Frequency**	**Circ Freq**	**Eigenvalue**
Text	Unit less	Sec	Cyc/sec	rad/sec	rad^2/sec^2
Mode	1	0.239496	4.1754	26.235	688.28
Mode	2	0.089339	11.193	70.329	4946.2
Mode	3	0.036701	27.247	171.2	29309
Mode	4	0.027215	36.745	230.87	53302
Mode	5	0.022447	44.549	279.91	78350
Mode	6	0.01549	64.559	405.63	164540

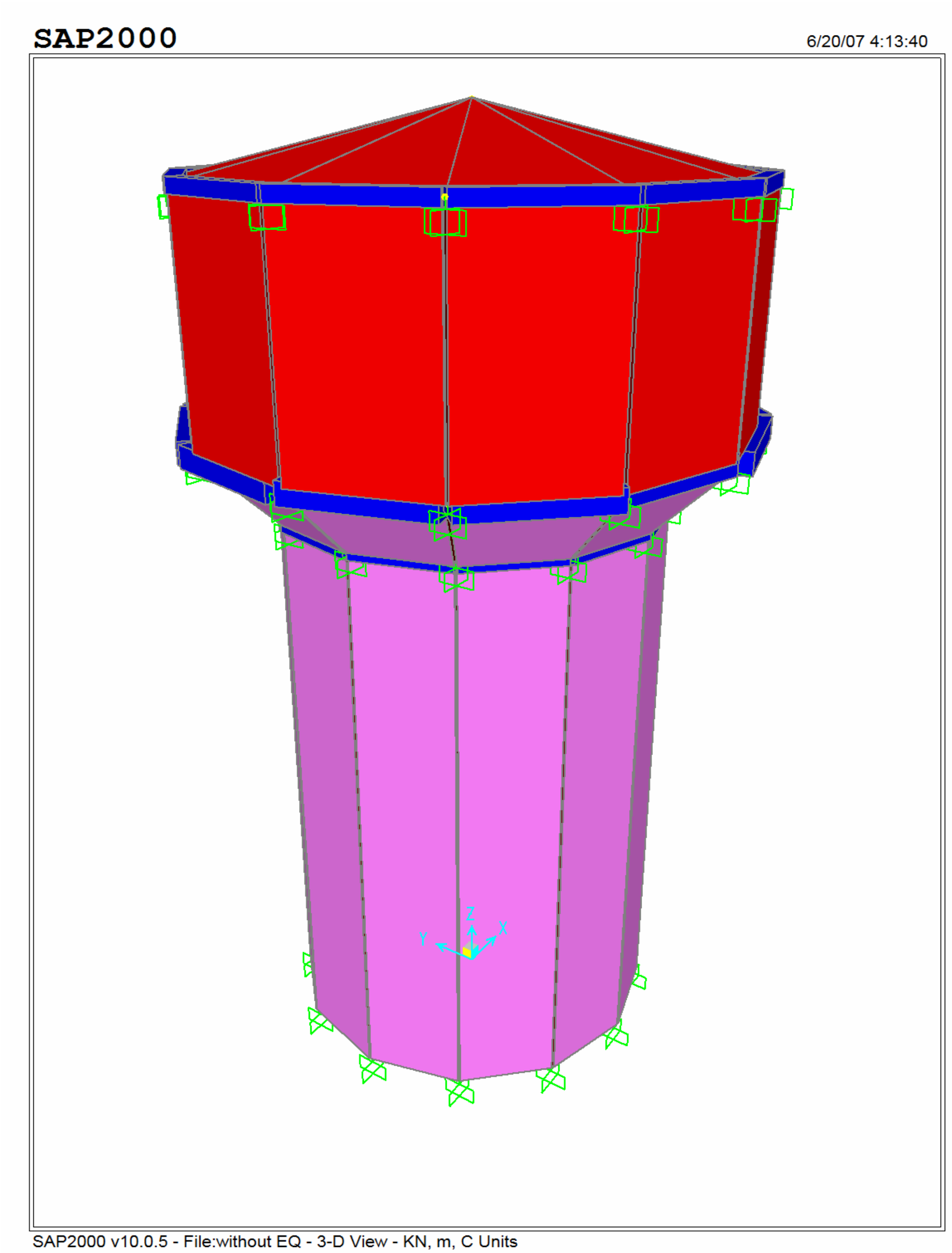


Figure 5.8 Intze tank supported on Shaft Staging

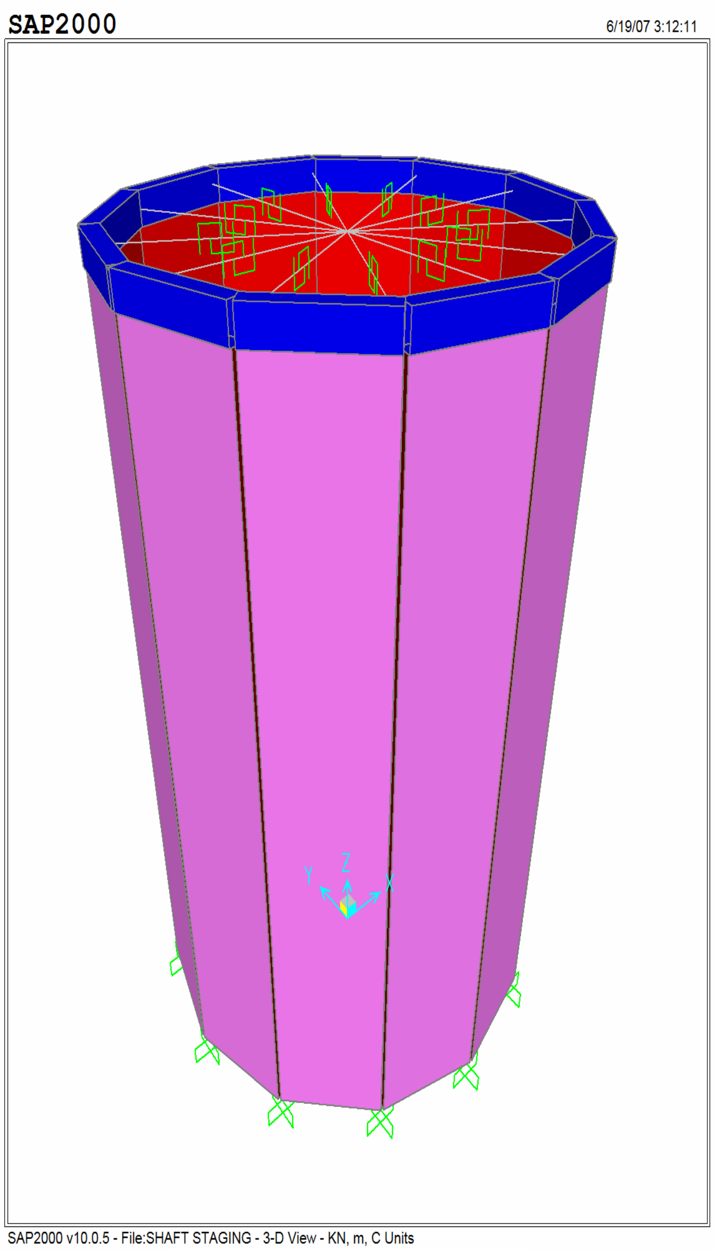


Figure 5.9 Shaft Staging

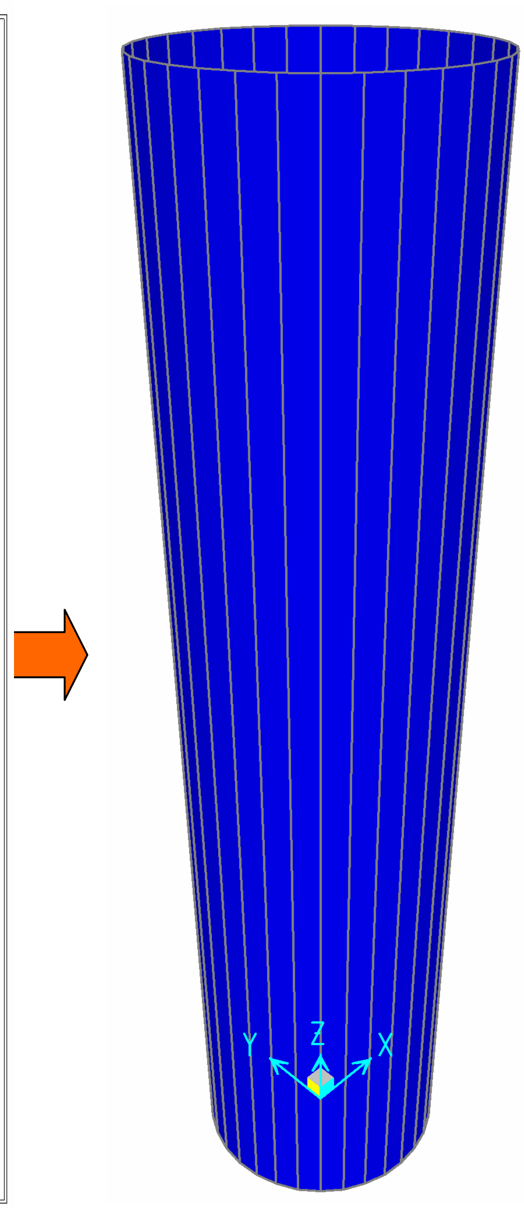


Figure 5.10 Solid Column equivalents to Shaft Staging

6. Evaluation of strength and ductility of shaft Staging:

Evaluation of strength and ductility of shaft staging, an equivalent solid column is modeled in place of cylindrical shaft because in pushover analysis hinges are defined before analysis. In SAP2000 hinges can not be defined in shell element so that an equivalent solid circular column has modeled in place of circular shaft. 3D modal of shaft staging and equivalent solid column is shown in figure 5.9 and 5.10

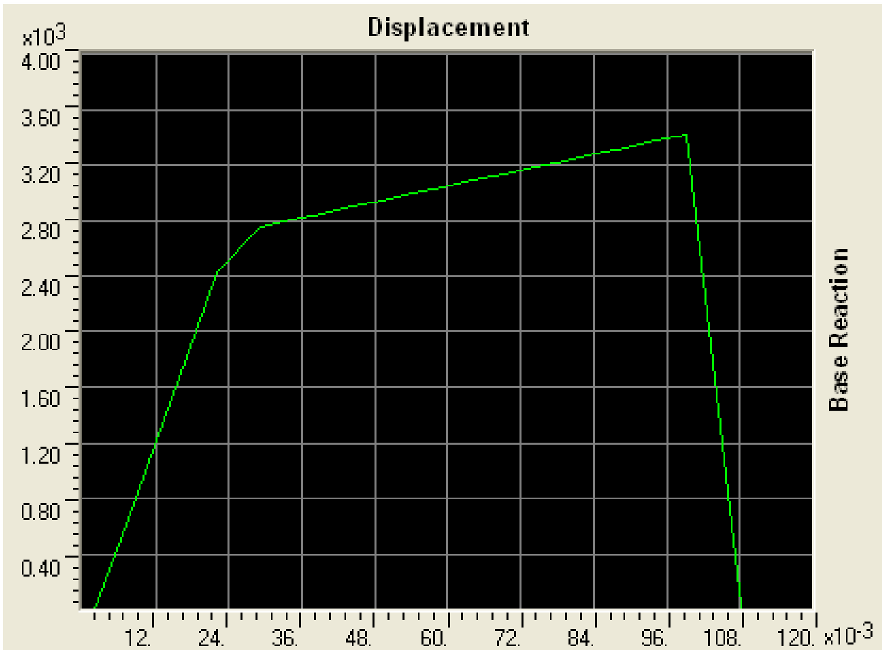


Figure 5.11 Pushover Curve for Shaft staging without considering earthquake

$\Delta_{ultimate}$ = deflection at ultimate point = 99×10^{-3}

Δ_{yield} = deflection at yield point = 29×10^{-3}

Ductility $\mu = \Delta_{ultimate} / \Delta_{yield}$

$= 3.37$

Hence ductility of shaft staging with considering earthquake forces is $\mu = 3.37$ and strength of frame staging is 2.806×10^3 kN

5.4.4 SHAFT STAGING TANK CONSIDERING EARTHQUAKE FORCES

1. Geometric parameters for Shaft staging tank with considering earthquake:

Component	Size (mm)	Component	Size (mm)
Top Dome	100 thick	Conical dome	520 thick
Top Ring Beam B1	370×400	Bottom dome	300 thick
Cylindrical Wall	340 thick	Circular Ring Beam B2	400×600
Bottom Ring Beam B3	1000×600	Shaft Staging	300 thick.

2. Material properties of concrete:

Mass per unit volume = 2.54 Kg/m^3

Weight per unit volume = 25 kN/m^3

Modulus of elasticity = 2.236×10^7 kN/m

Poisson's ratio = 0.17

Concrete compressive strength fc = $20\times10^3 kN/m^3$

Bending Reinf. Yield stress fy = 415×10^3 kN/m^3

3. Loading:

Self weight of each component

Water pressure

Earthquake forces

4. Modeling:

3D Geometric modeling of intze tank with shaft staging is shown in figure 5.8

5. Computational value of intze tank with frame staging through software package SAP.

Table 5.4 Modal Periods and Frequencies of shaft staging tank with considering earthquake

Step Type	Step Num	Period	Frequency	Circ Freq	Eigenvalue
Text	Unit less	Sec	Cyc/sec	rad/sec	rad^2/sec^2
Mode	1	0.235479	4.2467	26.683	711.96
Mode	2	0.087888	11.378	71.491	5111
Mode	3	0.036088	27.71	174.11	30314
Mode	4	0.025702	38.908	244.47	59764
Mode	5	0.022035	45.382	285.15	81308
Mode	6	0.015238	65.625	412.33	170020

6. Evaluation of strength and ductility of Shaft Staging:

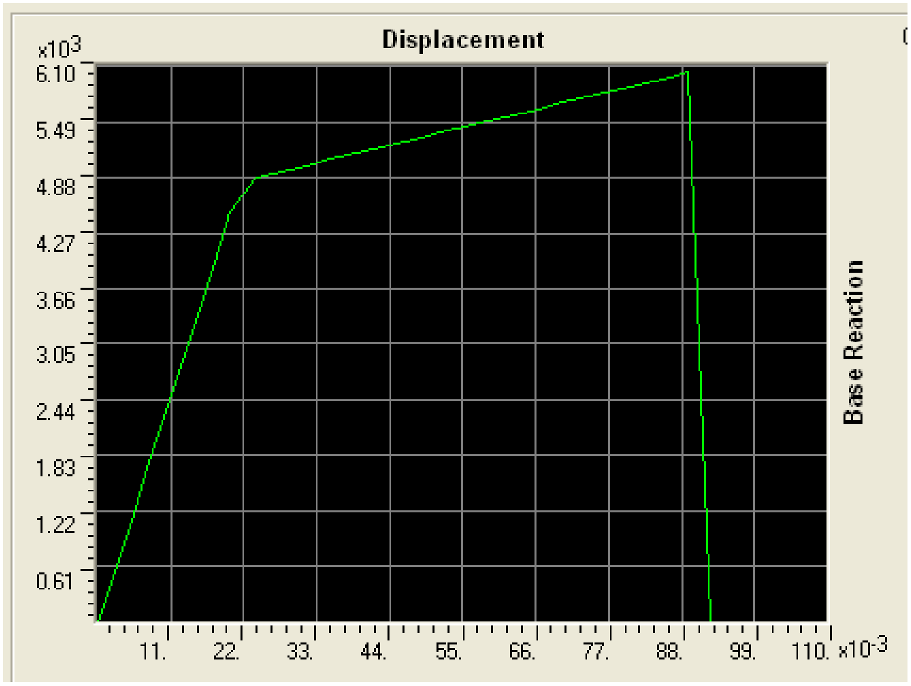


Figure 5.12 Pushover Curve for Shaft staging with considering earthquake

$\Delta_{ultimate}$ = deflection at ultimate point = 89×10^{-3}

Δ_{yield} = deflection at yield point = 23×10^{-3}

Ductility $\mu = \Delta_{ultimate} / \Delta_{yield}$

$= 3.86$

Hence ductility of shaft staging with considering earthquake forces is μ = 3.86 and strength of frame staging is 4.859×10^3 kN

chapter 6

RESULTS DISCUSSIONS AND CONCLUSION

6.1 GENERAL

In India different types of tanks are available for storage of water. Most of them are reinforced concrete and only very few are structural steel. Water tank is a slender top heavy structure and its natural period of vibration is quite high. Therefore, it is necessary to consider the hydrodynamic behavior while designing such structures.

The present study looks into the possibility of changing time period and ductility by adopting a few alternate configurations. Two alternative configurations of staging of elevated water storage tank are studied: intze tank supported on frame staging and shaft staging. This tank is also being designed by considering with and without earthquake forces. The seismic analysis of these tanks has been done by two different methods, first lumped mass modal and second is two mass modal methods. Strength and ductility of frame and shaft staging has also been evaluated by considering with and without earthquake forces. The outcome of this study can be briefly summarized as follows:

Comparison of different seismic analysis parameters of intze tank supported on frame staging and shaft staging has shown in table 6.1. In this table all parameter for single mass modal as well two mass modal for frame staging and shaft staging are summarized.

Table 6.1 Comparison of Seismic Analysis Parameter of Intze Tank Supported On Frame Staging and Shaft Staging

S.No	Component	Frame staging	Shaft staging
A	Lateral Stiffness (K_s)	51929.168 kN/m	1.41×10^6 kN/m
B	**lumped mass modal**		
1	Time period (a) tank is empty (b) tank is full	0.69 sec 1.12 sec	0.129 sec 0.213 sec.
2	Base shear (a) tank is empty (b) tank is full	320.72 kN 603.84 Kn	440.62 kN 1190.14 kN
3	Hydrodynamics pressure on the wall	1980 N/m^2	4008.60 N/m^2
4	Hydrodynamics pressure on the base	1977 N/m^2	4001.40 N/m^2
C	**Two mass modal**		

1	Time period (a) impulsive mode (b) convective mode	0.935 sec 4.0 sec.	0.177 sec 4.0 sec
2	Base shear (a) impulsive mode (b) convective mode	429.121 kN 72.096 kN	995.28 kN 73.47 kN
3	Overturning moment (a) impulsive mode (b) convective mode	 8913.46 kN-m 1538.83 kN-m	 20708.04 kN-m 1568.23 kN-m
4	Hydrodynamics pressure on the wall (a) impulsive mode (b) convective mode	 0 (Top) 2030.18 N/m^2 (Bottom) 799.31 N/m^2 (Top) 275.58 N/m^2 (Bottom)	 0 (Top) 4804.42 N/m^2 (Bottom) 799.31 N/m^2 (Top) 275.58 N/m^2 (Bottom)
5	Hydrodynamics pressure on the base (a) impulsive mode (b) convective mode	 0 (at centre) 701.60 N/m^2 (at wall) 0 (at centre) 272.41 N/m^2 (at wall)	 0 (at centre) 1658.48 (at centre) 0 (at centre) 272.41 N/m^2 (at wall)
6	Pressure due to wall Inertia	285 N/m^2	675 N/m^2
7	Pressure due to vertical excitation	0 (Top) 3896.53N/m^2 (Bottom)	0 (Top) 3896.53 N/m^2 (Bottom)
7	Sloshing Wave Height	0.45 meter	0.549 meter

Time period of water tank supported on frame staging is higher (five times) as compare to tank supported on shaft staging because lateral stiffness of shaft staging is much higher than the frame staging. Lateral stiffness of shaft staging is 25 times more than the frame staging. Due to very much time difference the hydrodynamic pressure also higher in shaft staging. Graphical representations of hydrodynamic pressure on cylindrical wall as well as bottom of the tank for lumped mass modal and two mass modal are shown in figure 6.1 to 6.4.

When earthquake forces are considering in designing elevated water storage tank, then dimensions and reinforcement of different component of intze tanks like cylindrical wall, conical dome bottom has changed. This variation of thickness and reinforcement has shown in table 6.2 and 6.3.

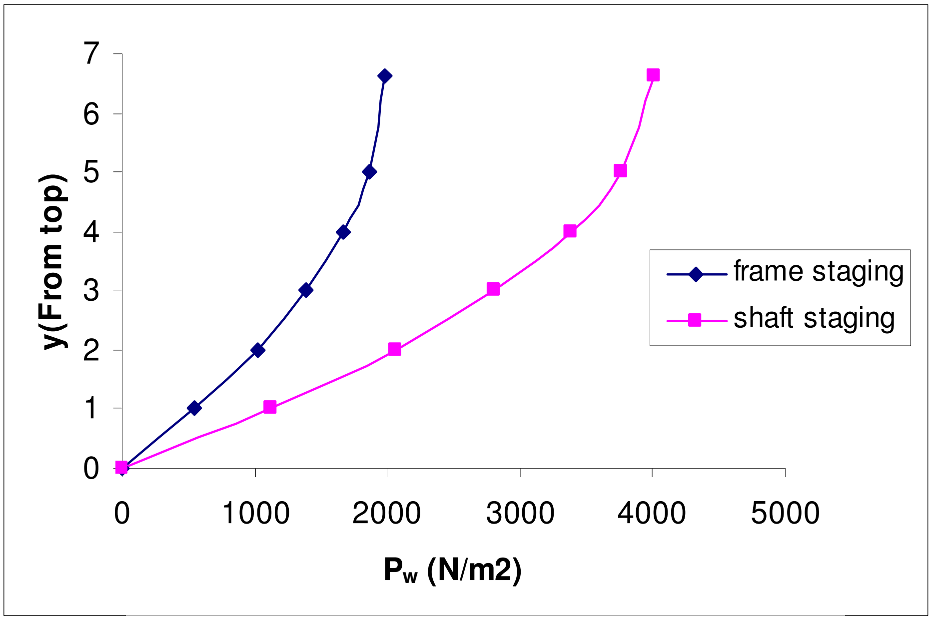


Figure 6.1 Hydrodynamic pressures on the tank wall

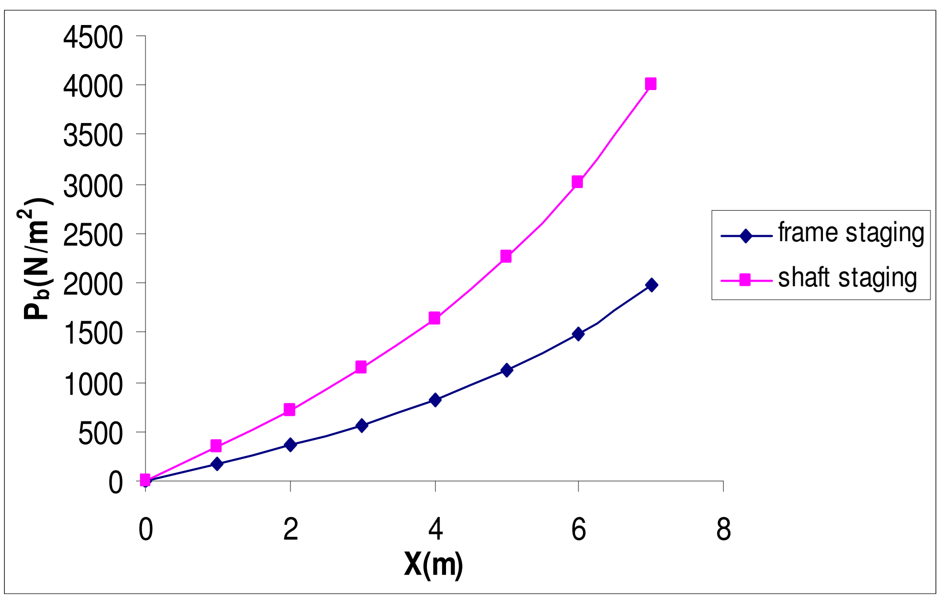


Figure 6.2 Hydrodynamic pressures on the bottom of the tank

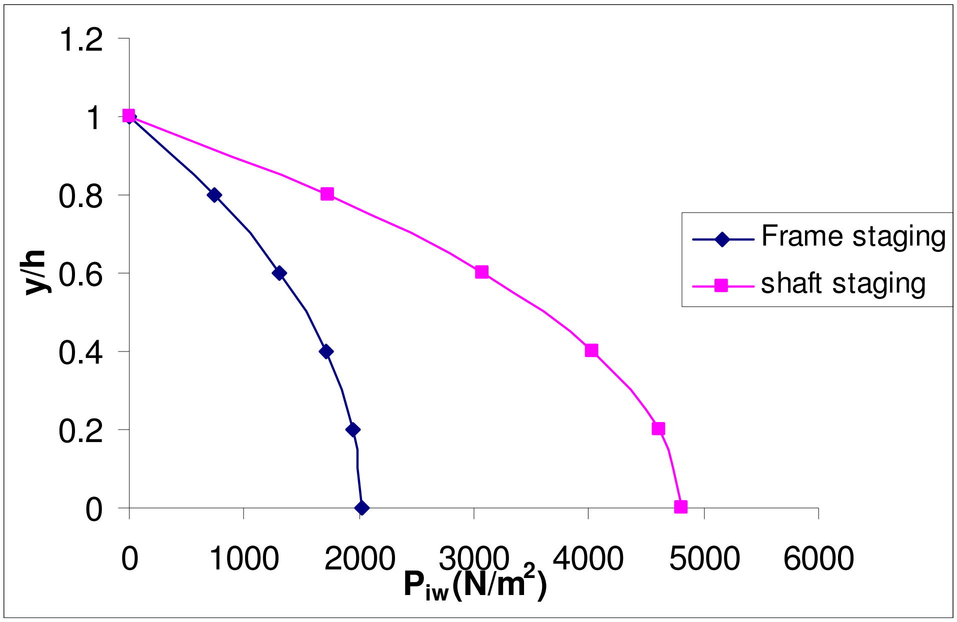


Figure 6.3 Impulsive Hydrodynamic pressures on the tank wall

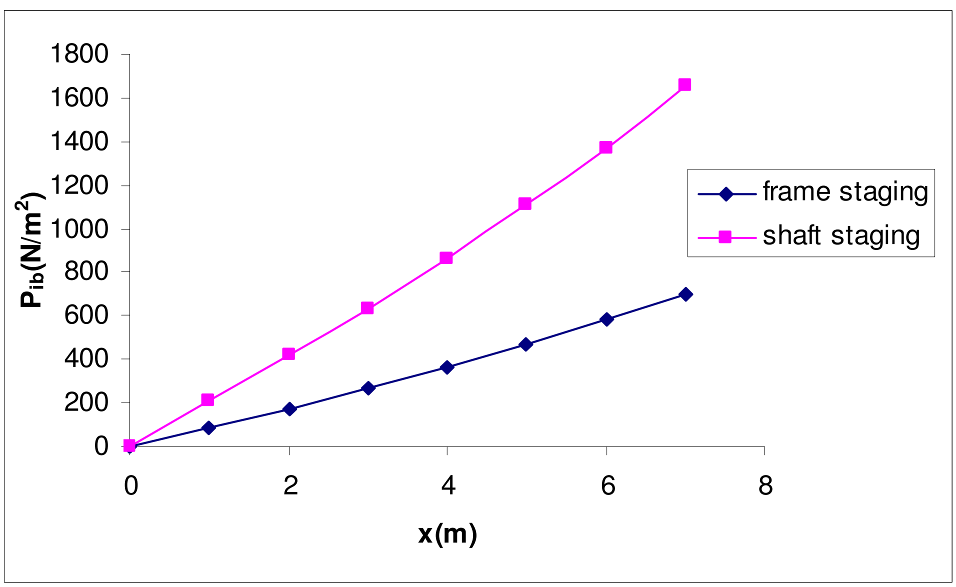


Figure 6.4 Impulsive Hydrodynamic pressures on the bottom of the tank

Table 6.2 Comparisons of Forces without Considering and Considering Earthquake in Intze Tank Supported On Frame Staging

No.	Component	Without Considering Earthquake		With Considering Earthquake	
		Forces	Reinforcement (mm^2)	Forces	Reinforcen (mm^2)
1	Top dome	31615 N/m	300	31615 N/m	300
2	Top ring beam B_1	195192.83 N	1519.76	195192.83 N	1519.7
3	Cylindrical wall	384160 N/m	2658.84	428157.31 N/m	2871.4
4	Middle ring beam B_3	708485.33 N	4948	787131.72 N	5319.1
5	Conical dome	614879.45 N	2115.78	662822.28 N	2233.3
6	Bottom spherical dome	292620.0 N/m	650	346954.55 N/m	720
7	Bottom ring beam B_2	276498 N	2537	174238.25 N	2826
8	Column	P = 1255935.3N	4019.2	P = 391632.38N M=753.77 kN-m	14130
9	Bracing	only self weight	1029.0	70580 N-m	2800

Table 6.3 Comparisons of Forces without Considering and Considering Earthquake in Intze Tank Supported On Shaft Staging

S. No.	Component	Without Considering Earthquake		With Consid
		Forces	Reinforcement (mm^2)	Forces
1	Top dome	31615 N/m	300	31615 N/m
2	Top ring beam B_1	195192.83 N	1519.76	195192.83 N
3	Cylindrical wall	384160 N/m	2658.84	452793.52 N/m
4	Middle ring beam B_3	708485.33 N	4948	819351.33 N
5	Conical dome	614879.45 N	2115.78	698294.28 N
6	Bottom spherical dome	292620.0 N/m	650	348934.14 N/m
7	Bottom ring beam B_2	276498 N	491.56	204186.75 N

8	Shaft staging	14788435.0 N	17270 vertical 440 horizontal	W = 19582.96 kN V= 997.9 M= 20767.34 kN

Table 6.4 Comparison Strength and Ductility of Staging

S.No.	Component	Without considering earthquake	Considering earthquake
A	**Frame staging**		
1	Ductility	3.24	4.9
2	Strength	1.89×10^{3} kN	2.16×10^{3} kN
B	**Shaft staging**		
1	Ductility	3.37	3.86
2	Strength	2.806×10^{3}	4.859×10^{3}

Ductility of frame staging is around 22% higher than the shaft staging therefore, during earthquake shaft staging type tanks are more crumble compare to frame staging tank. Shaft type of staging has poor ductility because of thin shell sections has lack of redundancy of load paths and toughness. When earthquake forces are considered in design i.e. ductility has been increased 33% in frame staging and 13% in shaft staging. The response reduction factor for shaft supported elevated tanks should be smaller then the frame staging, because thin shells of shafts are not ductile and devoid of advantages of redundancy of load paths.

6.2 CONCLUSIONS

1. Time period of elevated water tank supported by frame staging is (5 times) higher than the tank supported by shaft staging. Since the lateral stiffness of shaft staging is much higher (25 times) than the frame staging.
2. Due to higher lateral stiffness of shaft staging, lateral forces are also high as compare to frame staging. The study shows that the base shear in tanks supported by shaft staging is approximately 2 times higher than the tank supported on frame staging.
3. Impulsive hydrodynamic pressure in shaft staging is 2.3 times higher then the frame staging, while the convective hydrodynamic pressure is same in both type of staging.
4. After non-linear static push over analysis of frame staging and shaft staging, it is indicate that the ductility of shaft staging is lower than the frame staging but it has higher strength. The ductility of shaft staging is 3.37 without considering earthquake forces and 3.86 after considering earthquake forces. The ductility of frame staging is 3.24 without considering earthquake forces and 4.9 after considering earthquake forces.
5. The designed tank (i.e. without earthquake forces) has low strength and ductility as compare to tank designed on the basis of earthquake forces.

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