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# TRANSIENT ANALYSIS OF ELEVATED INTZE WATER TANK-FLUID- SOIL SYSTEM

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**ABSTRACT**

The conventional analysis (non-interaction analysis) of over head water tank assumes that columns rest on unyielding supports. In reality, the structure is supported by deformable soil strata which deforms unevenly under the action of loads and hence causes redistribution of forces in the components of overhead water tank. In the present work, 3-D interaction analysis of intze type water tank-fluid-layered soil system is carried out using ANSYS software to evaluate the principal stresses in different parts of the tank and supporting layered soil mass. The resultant deflections, Von-mises stress, neural frequency of the tank are calculated and also evaluate acceleration by Transient analysis under different filling conditions of the intze tank. The intze tank, supporting frame, foundation and soil mass are considered to act as single compatible structural unit for more realistic analysis. The tank, foundation and soil are considered to be follow linear stress-strain relationship. The natural frequency of the tank is evaluated for different filling conditions and comparison is made between the non-interaction and interaction analyses.

**Keywords:** Intze tank, fluid-structure interaction, non-interaction analysis, finite element analysis, layered soil, principal stresses, natural frequency, transient analysis, resonance.

## 1. INTRODUCTION

The soil-structure interaction is a complex phenomenon which involves mechanism of interaction between various components of intze type water tank-fluid-layered soil system. In common design practice, interaction between soil, foundation and tank structure is neglected to simplify the structural analysis. A stress analyst generally ignores the influence of the settlements of supporting soil on the structural behavior of the super-structure. In addition to this, the effect of the stiffness of the structure is disregarded in evaluating the foundation settlements. Earlier studies have indicated that interaction effects are quite significant, particularly for the tank resting on highly compressible soils.

The eelevated tanks are subjected to lateral and torsional vibrations due to wind and seismic forces. These lateral forces physically induce two different types of vibration in the water of the tank. A part of the water at the upper portion of tank participates in sloshing motion (convective) with a longer period, while rest of the water at the bottom portion of the tank experiences the same impulsive vibration as the tank container is rigidly attached with container wall. The differential settlements, rotation of shaft or frame and stiffness of the tank cause redistribution of forces/stresses in the tank members. A more rational solution of such an interaction system can be achieved by appropriate analysis.

## 2. LITERATURE REVIEW

A lot of investigations have taken place in the area of soil-structure interaction of over-head and under-ground water tanks. Various investigators have proposed different approaches for solution of interaction problems from time to time in attempt to obtain more realistic analysis. They have quantified the effect of interaction behaviour and established that there is redistribution of forces/stresses in the water tank components.

Housner (1963) considered a model with two uncoupled masses and developed equations to compute the impulsive and sloshing liquid masses along with their location above the tank base and the stiffness of the convective mass spring. Usually only one convective mass is considered in practical design.

Haroun and Housner (1981) developed a three-mass model which takes into the tank-account the wall flexibility only.

Ibrahim *et al*. (2001) presented a broad overview of sloshing dynamics, including both linear and nonlinear analyses, with emphasis on cylindrical and rectangular tanks.

Karamanos *et al*. (2006) and Patkas and Karamanos (2007) developed a mathematical model for calculating linear sloshing effects in the dynamical response of horizontal cylindrical and spherical liquid containers under earthquake excitation.

Livaoglur. R. and Dogangun A. (2006) investigated the effects of foundation embedment on the seismic behaviour of fluid-elevated tank-foundation-soil system with a structural frame supporting the fluid containing tank. Six different soil types defined in the seismic codes were considered. Both the sloshing effects of the fluid and soil-structure interaction of the elevated tanks resting on these six different soils were included in the analyses.

Karamanos *et al*. (2006) proposed a methodology based on a ‘‘convective-impulsive’’ decomposition of the liquid-vessel motion and a semi-analytical solution of sloshing in non-deformable containers by which the seismic forces can be estimated. Additionally, the effects of the support structure flexibility are also considered.

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Livaoglu *et al*. (2007) presented simplified procedures for seismic analysis for elevated tanks considering fluid-structure-soil interaction Ten different models weree analyzed using mechanical and finite-element modelling techniques. The applicability of these ten models for the seismic design of the elevated tanks with four different subsoil classes is emphasized.

Sezen *et al*. (2008) carried out dynamic analysis using a simplified three-mass model and. investigated the seismic performance of elevated cylindrical tanks damaged during the Kocaeli earthquake (1999) in Turkey.

Dutta *et al*. (2009) presented comprehensive study on dynamic characteristics of RC elevated tanks supported by cylindrical shaft staging. The results were validated analytically using finite element analysis and by small-scale experimentation.

Amani *et al.* (2010) evaluated resonant frequencies in an RC elevated spherical container partially filled with water using finite element method and verified the results experimentally. The overall dynamical response of elevated spherical tanks subjected to horizontal base motion and free vibration and containing water at different levels were carried out. He investigated that for spherical tank, essentially three independent mass-motions are necessary; translation (structural), sloshing (convective) and pendulum motions. Therefore, three degrees of freedom is required for the analysis.

Moslemi, M. *et al*. (2011) presented the seismic response of liquid-filled elevated tank and studied the complexities associated with modelling of the conical shaped tanks. The fluid domain is modelled using displacement-based fluid elements (D-Fluid element). Both time history and modal analyses were performed for an elevated tank.

Chaduvula, U. *et al*. (2013) have an experimental investigation made on a 1:4 scale model of cylindrical steel elevated water tank subjected to combined horizontal, vertical and rocking motions, for earthquake excitation (accelerations) of 0.1g and 0.2g and increasing angle of rocking motion. It was investigated that the impulsive base shear and base moment increase with increase in earthquake acceleration, whereas, the convective base shear and base moment increase with increase in earthquake acceleration but decrease with increasing angular motion. Therefore, there is no considerable effect of rocking motion found due to sloshing of water. The nonlinearity is found in the structure, when the impulsive pressure of tank decreases with increase in tank acceleration.

## 3. PROBLEM FOR ANALYSIS

In present problem, an overhead Intze water tank of capacity of $1000m^3$ resting on layered soil mass and subjected to gravity and water loading is analyzed. The elevated tank has a frame supporting structure in which columns are connected by the circumferential beam at regular intervals, at 4m, 8m and 12m height level. The container is filled with water. The container and the supporting structure are being used in most part of India located in earth quack prone zone. To investigate the interaction behaviour, the interaction analyses are carried out for the following four cases:

**Case-1:** The conventional/non-interaction analysis (NIA) considering the columns fixed at their bases.

**Case-2:** The linear interaction analysis of intze water tank with fluid (LIA+FSI) considering the tank foundation resting on layered soil mass consisting of five different soil types.

**Case-3:** The vibration analysis (VIA) of intze water tank-foundation-layered soil mass to evaluate natural frequency, deflection and von-Mises stress for ten modal shapes under different filling condition of the tank.

**Case-4:** Transient analysis of intze water tank-foundation-layered soil system for different filling conditions of water tank to investigate Resonance condition.

The tank, foundation and supporting soil mass are considered to behave in linear elastic manner.

The geometric properties of tank, foundation, and soil mass are provided in Table-1. The material properties of tank, foundation, and soil mass are provided in Table-2.

Table-3 provides loading on different parts of the tank which include self weight and imposed load due to water.

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**Table-1.** Geometrical properties of tank, foundation and soil mass.

| Name of component | Description | Data |
|---|---|---|
| Intze water tank | Inside diameter of tank (D) | 12.0 m |
| | The average depth (0.75D) | 9.0 m |
| | Height of cylindrical portion of tank (2/3D ) | 8.0 m |
| | Height of top dome (0.15 to 0.20D ) | 2.0 m |
| | Height of conical dome (0.2D ) | 2.5 m |
| | Height of bottom dome (0.15D ) | 1.8 m |
| | Diameter of staging (0.6 D ) | 7.0 m |
| | Bracing at 0.3D c/c | 4.25 m |
| | Dimension of bracing beam | 500 mm x 700 mm |
| Foundation | Diameter of foundation | 6.0 m |
| | Depth of foundation | 1.2 m |
| Homogeneous soil | Semi-finite extent of soil mass | 21 m x 14 m |

**Table-2.** Material properties of tank and soil mass.

| Material type | Young modulus kN/m$^2$ | Poission's ratio | Density kN/m$^3$ |
|---|---|---|---|
| Concrete | 25490 | 0.20 | 24.00 |
| Soil Type-1 | 35000 | 0.28 | 17.10 |
| Soil Type-2 | 40000 | 0.29 | 17.40 |
| Soil Yype-3 | 45000 | 0.30 | 18.00 |
| Soil Yype-4 | 55000 | 0.32 | 19.20 |
| Soil Type-5 | 60000 | 0.33 | 19.90 |

**Table-3.** Loads on various parts of tank.

| Component | Description | Dead load plus Live load |
|---|---|---|
| Intze water tank | Top dome | 3.85 kN/m$^2$ |
| | Maximum hoop tension at base of side wall | 480.00 kN/m |
| | Bottom ring beam | 86.34 kN/m |
| | Total load at base of conical dome slab | 110.50 kN/m$^2$ |
| | Total load at base of spherical dome | 92.50 kN/m$^2$ |
| | Total design load on circular girder | 753.00 kN/m |
| | Total vertical load on each Column | 2234.64 kN |
| Foundation | Axial load on all columns | 19274.40 kN |

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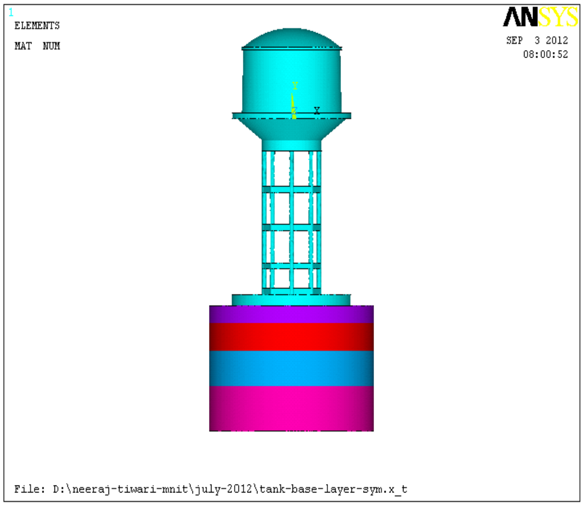


**Figure-1.** Model of intze tank.

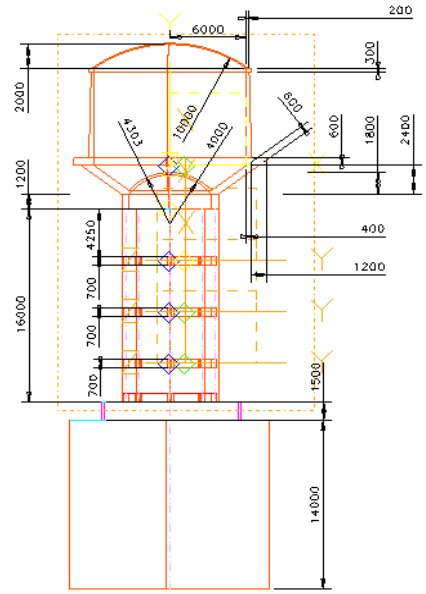


**Figure-2.** Geometry and dimensions.

### 3.1 Calculation for earthquake (Seismic) load

#### 3.1.1 Centre of gravity of empty tank

Taking moment of weight of element about bottom of circular girder

**Table-4.** Centre of gravity for empty tank.

| Element | Weight | Distance | Moment |
|---|---|---|---|
| Top dome | $2\pi$ x 10 x 2 x 3.85 = 438.8 | 11.7 + 2/3 | 5983 |
| Top ring beam | 0.3 x 0.3 x25 x $\pi$ x 12 = 84.8 | 11.7-0.15 | 979.7 |
| Cylindrical side wall | 7.1 x 0.3 x 25 x 13.2 $\pi$ = 2007.5 | 3.7 + 0.6 + 3.16 | 14967 |
| Bottom ring beam | 0.6 x 1.2 x 25 $\pi$ x 13.2 = 746.4 | 3.7 + 0.3 | 2985.8 |
| Conical dome | 1520 | 1.2 + 2.5 x 0.7 | 4484.0 |
| Bottom dome | 2 $\pi$ x 4.3 x 1.8 x5 = 243.2 | 1.2 + 1.8/3 | 437.7 |
| Bottom girder | 18 x $\pi$ x 7 = 395.8 | 0.6 | 237.5 |
| Total | 5481.5 | | 30074.7 |

Centre of gravity = 30074.7/5481.5 = 5.49 m above bottom of circular girder.

#### 3.1.2 Centre of gravity of full tank

Taking moment of weight of element about bottom circular girder.

**Table-5.** Centre of gravity of full tank.

| Element | Weight | Distance | Moment |
|---|---|---|---|
| Empty tank | 5481.5 | | 30074.7 |
| Water in cylindrical wall | $\pi$ x $6^2$ x 8 x 10 = 9047.8 | 3.7 + 4 | 69667.9 |
| Water in conical dome in conical section | $\pi$ x 9.5 x ($2.5^2$/2) x 10 = 932.7 | 3.7 – 2.5/3 | 2673.6 |
| Water in conical dome in cylindrical section | $\pi$ x $3.5^2$ x 0.5 x 10 = 192.4 | 3.7 – 0.25 | 663.9 |
| Water in conical dome between bottom dome and cylindrical portion | [($\pi$ x $3.5^2$ x 2) - ($\pi$ x $2^2$/3)(3 x 4.3 – 2.0)] 10 = 313.3 | 1.2 + 2.0 – 0.15 | 641.9 |
| Total | 15967.5 | | 103722.0 |

Centre of gravity = 103722/15967.5 = 6.5 m above bottom of circular girder.

### 3.1.3 Calculation of seismic coefficient

As per section 5.2 of IS: 1893 - 1984, elevated water tank shall be regarded as a system with a single degree of freedom with the mass concentrated at the centre of gravity. The analysis is to be done for tank full and empty condition.

**a) Tank empty condition**

$\Delta$ = static deflection at tank top = $Wl^3\alpha/EI$

$\propto l$ = 167.5 + 5.49 = 21.99 m, $l$ = 16.5 + 13.7 = 30.2 m,

$\propto$ = 21.99/30.2 = 0.728

Deflection coefficient $\alpha = \propto^2 (3 - \propto) / 6 = 0.728^2 (3 – 0.728) / 6 = 0.2$

$W$ = Weight of empty tank + weight of staging /3 = 5481.5 + (113.1 + 51.8) 8/3 = 5921.2 KN

$E = 5700\sqrt{f_{ck}} = 5700\sqrt{20}$ 25.49 x $10^3$ N/mm$^2$

$I$ = Moment of inertia of staging = (8 x 9.75 x $10^9$) = 2(x $300^2$) (3500 /$\sqrt{2}$)$^2$ = 1.39 x $10^{13}$ mm$^4$

$\Delta$ = 5921.2 x $10^3$ x (30.2 x $10^3$)$^3$ x 0.2/ (25.49 x $10^3$ x 1.39 x $10^{13}$) = 92.06 mm

Free period $T = 2\pi\sqrt{\frac{\Delta}{g}} = 2\pi\sqrt{\frac{92.06}{9.81 \times 1000}} = 0.609$ seconds.

System damping may be assumed as 5% of the critical for concrete structures.

From Figure-2 of IS: 1893 -1984, average acceleration coefficient $S_a / g$ = 0.15

$\beta =$ 1.0 for raft foundation, from Table-2, IS: 1893-1984

$I$ = 1.5 importance factor for water tank from table 4, IS: 1893-1984

Delhi is in zone IV, from Figure-1, IS: 1893-1984

Horizontal seismic coefficient $\alpha_h = \beta I F_0 S_a / g$ = 1 x 1.5 x 0.25 x 0.15 = 0.05625

**b) Tank full condition**

$\propto l$ = 16.5 + 6.5 = 23 m, $\propto$ = 23 / 30.2 = 0.7616

$\alpha = 0.7616^2 (3 – 0.7616) / 6 = 0.216$

$W$ = weight of full tank + weight of staging / 3 = 15967.5 + (113.1 + 51.8) 8/3 = 16407.2

$\Delta$ = 16407.2 x $10^3$ x (30.2 x $10^3$)$^3$ x 0.216/ (25.49 x $10^3$ x 1.39 x 1.39 x $10^{13}$) = 275.5 mm

Free period $T = 2\pi\sqrt{\frac{275.5}{9.81 \times 1000}}$ = 1.053 seconds

Average acceleration coefficient $S_a / g$ = 0.102 for 5% damping

Horizontal seismic coefficient $\alpha_h$ = 1 x 1.5 x 0.25 x 0.102 = 0.03825

### 3.1.4 Checking column under EQ load for tank empty condition

Lateral load = $\propto_h$

$W$ = 0.05625 x 5921.2 = 333.1 KN

Moment at top of foundation = 333.1 x (16 + 5.49) =7158.3 KNm

**a) Checking farthest column**

Axial force on the farthest column = 4 $M/n$ $D$ = 4 x 7158.3 / (8 x 7) = 511.3 KN

Dead load on each column = (5481.5 / 8) + 113.1 + 51.8 = 850.1 KN

Total load on farthest column = 511.3 + 850.1 = 1361.4 KN < 3067.4 KN capacity

Total load on nearest column = 850.1 – 511.3 = 338.8 KN > 0, thus safe against uplift.

**b) Checking column lying on bending axis**

Maximum shear force $V = 2 \sum Q / n$ = 2 x 333.1/8 = 83.3 KN < 265 KN capacity

Moment due to $V = V$ x $l$ /2 = 83.3 x 3.5/2 = 145.78 KNm

Checking the interaction equation,

(2.27/5 x 1.33) + (4.49/7 x 1.33) = 0.822 < 1, hence safe under EQ load.

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**3.1.5 Checking column under EQ load for tank full condition**

Lateral load = $\alpha_h W$ = 0.03825 x 16407.2 = 627.6 KN

Moment at foundation top = 627.6 (16 + 6.5) = 14121 KNm

**a) Checking for farthest column**

Axial force in the farthest column = 4 $M/nD$ = 4 x 14121 / (8 x 7) = 1008.7 KN

Dead load on each column = (15967.5/8) + 113.1 + 51.8 = 2160.8 KN

Total load on the farthest column = 1008.6 + 2160.8 = 3169.5 > 3067.4 KN hence slightly unsafe.

**b) Checking column lying on bending axis**

Maximum shear force $V = 2 \sum Q / n$ = 2 x 627.6/8 = 156.9 KN < 265 KN capacity

Moment due to $V = V \times l/2$ = 156.9 x 3.5/2 = 274.6 KNm

Checking the interaction equation,

(5.77/5 x 1.33) + (8.45/7 x 1.33) = 1.78 >> 1, hence very unsafe under EQ load.

**3.1.6 Foundation size**

The column are supported on a circular girder with a circular raft below it

| | |
|---|---|
| Weight of full tank | = 15967.5 KN |
| Weight of column = 184 x 8 | = 1472.0 KN |
| Weight of brace = $\pi$ x 7 x25 x 3 x 0.5 x 0.7 | = 577.3 KN |
| | =========== |
| Total load on foundation | = 18016.8 KN |

Assuming 10% self weight of foundation and safe bearing capacity of soil = 250 KN/mm$^2$

Area of foundation required = 18016.8 x 1.1/250 = 79.3 m$^2$

Providing a circular raft of 10.5 m diameter, area = 86.6 m$^2$

$I = \pi \times 5.24^4/4 = 596.7$ m$^4$, $Z = I/r = 596.7/5.25 = 113.65$ m$^3$

**3.1.7 Checking foundation for stability under EQ load**

**a) Full tank condition**

Overturning moment at foundation base for tank full = 902.4 (17 + 6.5) = 21206.4 KNm

Upward reaction $q_o = (P/A) \pm (M/Z)$ = (18016.8 x 1.1/86.6) $\pm$ (21206.4/113.65)
= 415.45 KN/mm$^2$, 42.25 KN/mm$^2$

Permissible increase in safe bearing capacity of soil for EQ load for raft foundation is 50%, as per Table-1, IS: 1893-1984,

Permissible bearing capacity = 250 x 1.5 = 375 KN/m$^2$

Hence the foundation slab size must be increased to say 12 m

Area = 113.1 m$^2$, $I = \pi \times 6^4/4 = 1017.9$ m$^4$, $Z$ = 1017.9/6 = 169.9 m$^3$

$q_o$= (18016.8 x 1.1/113.1) $\pm$ (21206.4/169.6) = 300.2 KN/mm$^2$, 50.2 KN/mm$^2$,
Both < 375 KN/mm$^2$, hence safe.

**b) Empty tank condition**

Overturning moment at foundation base for empty tank = 331.1(17 + 5.49) = 7491.4 KNm
Weight of empty tank + staging = 5481.5 + 1472 + 577.3 = 7530.8 KN
$q_o$ = (7530.8 x 1.1/113.1) $\pm$ (7491.4/169.6) = 117.4 KN/m$^2$, 29.07 KN/m$^2$, thus safe.

**3.1.8 Foundation design under EQ load**

Permissible soil pressure acts on the foundation under EQ load for tank full condition.

$q_{eff}$ = (18016.8/113.1) $\pm$ (21206.4/169.6) = 284.3 KN/m$^2$, 34.3 KN/m$^2$

The maximum pressure ordinate on the circular girder under EQ load = 232.2 KN/m$^2$,

And the maximum pressure ordinate on the circular girder under gravity load (18016.8/113.1) = 159.3 KN/m$^2$
(232.2/159.3) = 1.46 > 1.33, hence circular girder will be designed for EQ load.

**Note:** Column lying on bending axis is very unsafe under EQ load so circular girder and column lying on bending axis must be design for EQ load and these are the critical component of the tank.

**4. FINITE ELEMENTMODELING**

The linear interaction analysis (LIA) of the problem is carried out using ANSYS software. The top dome, side wall, bottom ring beam, conical dome, spherical dome, circular girder, column, foundation and soil mass are discretized with SOLID187 element which is a higher order 3-D 10-node element. SOLID187 has a quadratic displacement modelling and is well suited for modelling irregular meshes. The element is defined by 10 nodes having three degrees of freedom at each node: translations in the nodal x, y, and z directions. The element has plasticity, hyper elasticity, creep, stress stiffening, large deflection and large strain capabilities. It also has mixed formulation capability for simulating deformations of nearly incompressible elasto-plastic materials, and fully incompressible hyper elastic materials.

The structure of the water tank and soil mass are modeled with the Solid-187 element as shown in Figure-4 and fluid within the tank are modeled with the FLUID30 (Acoustic-3D30element) as shown in Figure-3.

FLUID30 is used for modelling the fluid medium and the interface in fluid/structure interaction problems. The governing equation for acoustics, namely the 3-D wave equation, has been discretized taking into account the coupling of acoustic pressure and structural motion at the interface. The element has eight corner nodes with four

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degrees of freedom per node: translations in the nodal x, y and z directions and pressure. The translations, however, are applicable only at nodes that are on the interface. The acceleration effects, such as in sloshing of water may be included. The element has the capability to include damping of sound absorbing material at the interface as well as damping within the fluid. The element can be used with other 3-D structural elements to perform unsymmetrical or damped modal, full harmonic and full transient method analyses. When there is no structural motion, the element is also applicable to modal analyses.

The finite element discretization of the problem is shown in Figure-5.

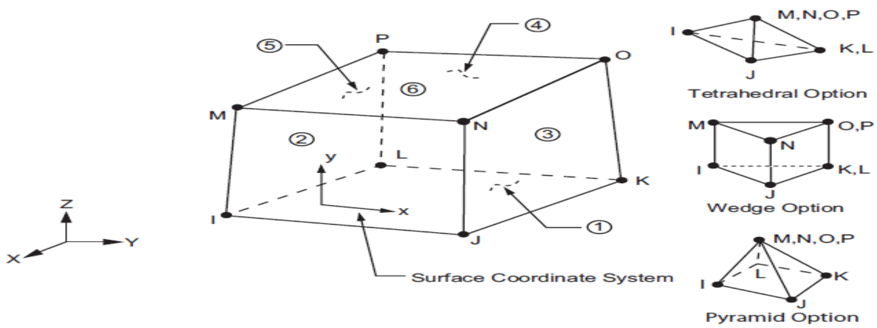


**Figure-3.** Geometry of FLUID 30 element.

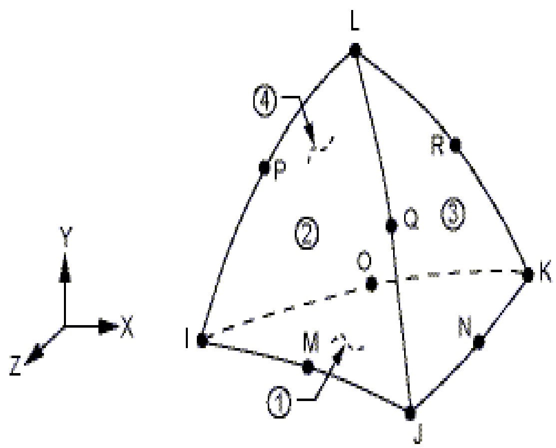

**Figure-4.** Geometry of SOLID187 element.

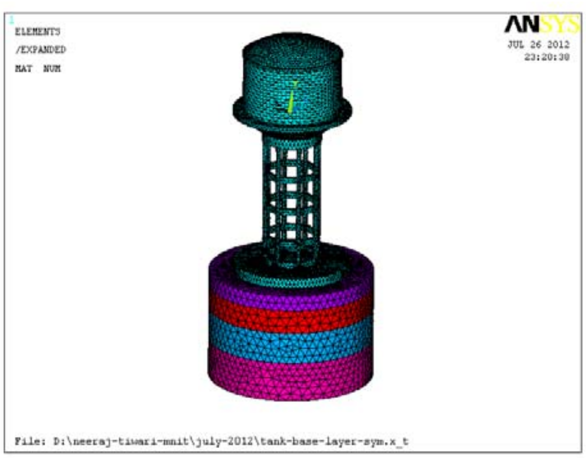


**Figure-5.** Finite element discretization of tank-footing-layered soil system.

The semi-infinite extent of the soil is considered to be of 21m diameter and 14m depth which is achieved by trial and error. The extent of soil mass is decided where vertical and horizontal stresses are found to be negligible due to loading on the superstructure. The vertical displacements in soil mass are restrained at the bottom boundary whereas horizontal displacements are restrained at vertical boundaries.

The Boussinesq's isobar diagram (pressure bulb) is shown in Figure-6.

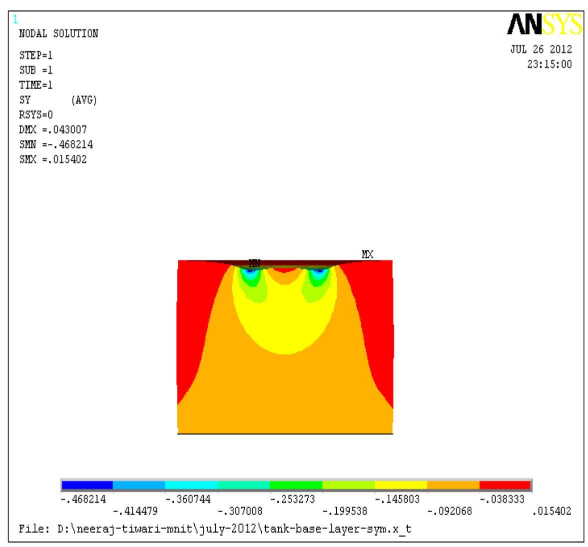


**Figure-6.** Formation of pressure bulb.

It is found that for uniformly loaded circular area the vertical pressure intensity becomes negligible at a distance of 1.5 times the diameter of raft in the vertical direction and nearly 3 times the diameter in the horizontal direction. The soil mass is idealized as isotropic, homogeneous. The element size is taken as 300 mm. The soil mass is discretized with finer meshes in close vicinity of footing where stresses are of higher order.

## 5. SUITABILITY OF ELEMENTS AND MESH CONVERGENCE

The solid element (SOLID 187) is chosen for discretization of tank instead of surface element whereas the shell element is taken for modeling of surfaces. The thickness of side wall varies from top to bottom hence shell element is not suitable. The mesh size of 300 mm is finally adopted since principal stresses are found to converge for this mesh size. Figure-7 shows the plot of element size and major principal stress.

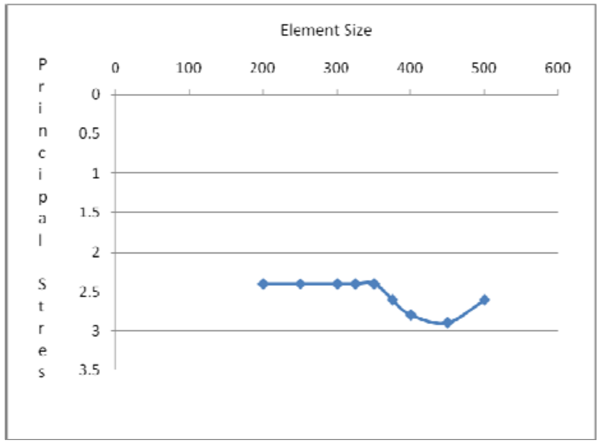


**Figure-7.** Plot of element size and major principal stress.

## 6. VALIDATION OF RESULTS

The results obtained by the ANSYS software are validated with the results obtained by analytical method using IS-code method for the same geometry, material properties and loading conditions.

**Table-6.** Comparison of major principal stress in various components.

| S. No. | Components | Major principal stress N/mm$^2$ | | |
|---|---|---|---|---|
| | | Analytical value (1) | FEM (NIA) (2) | % Difference (1) **and** (2) |
| 1 | Top dome | 0.193 | 0.183 | 5.18% |
| 2. | Top ring beam | 0.912 | 0.903 | 1% |
| 3 | Side wall | 0.960 | 0.986 | -1.25% |
| 4 | Bottom ring beam | 1.000 | 0.957 | 4.3% |
| 5 | Conical dome | 1.160 | 1. 118 | 3.6% |
| 6 | Bottom spherical dome | 0.990 | 1.009 | -1.88% |
| 7 | Bottom circular girder | 1.119 | 1.201 | -06.82 |
| 8 | Column | 1.200 | 1.126 | 5.99% |

Table-6 shows the values of major principal stress obtained by analytical method using IS-code) and finite element method (FEM). The values are found to be in close agreement. The percentage difference between analytical values and FEM values are found to be less than 6% for all components, and the values obtained are found to be within permissible range. It is found that maximum value of major principal stress occurs in the bottom circular girder, that's why this is a critical component of the tank.

## 7. INTERACTION ANALYSIS

In the present work, the maximum principal stress and natural frequency under five modes of water tank with different filling conditions in various components of tank-fluid-soil-foundation system are evaluated due to NIA and LIA+FSI and discussed subsequently. The finite element analyses are carried out using added mass approach for fluid-structure interaction with the distributed mass techniques. An equivalent cylinder is considered for the estimation of equivalent masses and stiffness of fluid. The impulsive mass obtained for the fluid is added to the mass of the container. In the present analysis the hydrodynamic pressure distribution acting on container wall is estimated by (Housner [09]).

The damping values for the reinforced concrete are taken as 5% for the impulsive mode and 0.5% for the convective mode as recommended in most of the literature.

The analyses are also carried out on layered model considering five different types of soil (Type-1 to Type-5) as supporting media. The soil properties considered are provided in Table-2.

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### 7.1 Maximum major principal stress in various parts of the tank

**Table-7.** Shows the values of maximum major principal stress in various parts of the tank for NIA and LIA.

| S. No. | Component of the tank | Major principal stress N/mm$^2$ | | | |
|---|---|---|---|---|---|
| | | Analytical values (I S code) | FEM values | | % Difference in NIA and (LIA+FSI) |
| | | | NIA | LIA+FSI | |
| 1 | Top dome | 0.193 | 0.183 | 0.052 | -71.58 |
| 2. | Side wall | 0.960 | 0.986 | 0.313 | -68.25 |
| 3. | Bottom ring beam | 1.000 | 0.957 | 0.484 | -49.42 |
| 4. | Conical dome | 1.160 | 1. 118 | 1. 301 | 16.36 |
| 5. | Spherical dome | 0.990 | 1.009 | 1.448 | 42.60 |
| 6. | Circular girder | 1.119 | 1.201 | 1.823 | 51.79 |
| 7. | Column | 1.200 | 1.126 | 0.9212 | -18.18 |
| 8. | Foundation | - | - | -0.3271 | - |
| 9. | Soil | - | - | -0.0196 | - |

Table-7 shows the values of of the maximum principal stress in various components of the interaction system and analytical values (IS- Code). The values of maximum principal stress obtained using analytical method and FEM are found to be in close agreement. The interaction analysis causes significant decrease in the maximum principal stress in top dome, side wall, bottom ring beam and column whereas significant increase is found in the conical dome, spherical dome and circular girder. The interaction analysis causes significant decrease of nearly 71% in the maximum principal stress in the top dome, 68% in the side wall, 49% in the bottom ring beam and 18% in the columns. The increase of nearly 16% is found in the maximum principal stress in conical dome, 42% in spherical dome, 51% in circular girder.

The magnitude of major principal stress is quite higher in Circular girder as compared to other components and therefore it is a critical component of the tank.

Livaoglu and Dogangu [11] considered 10 different models and calculated time period under sloshing mode and impulsive mode they evaluated the maximum value for of base shear and the overturning moment for different models. The present analyses evaluates maximum principal stresses in different parts of the tank which shows that the circular girder of the tank is the most critical component and is prone to failure under vertical loading.

The first principal stress and Von-mises stress in the tank-foundation-soil system are shown in Figure-8 and Figure-9 respectively.

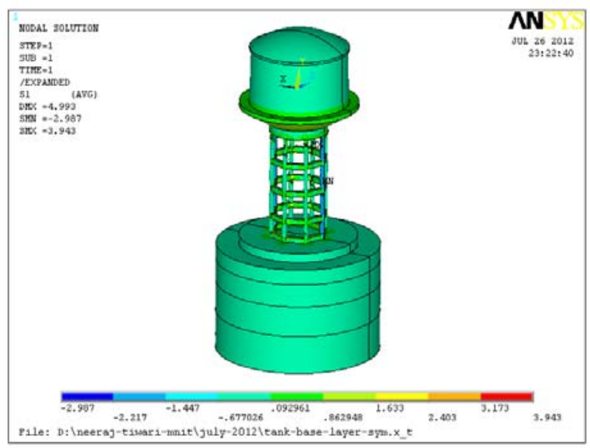


**Figure-8.** Major principal stress.

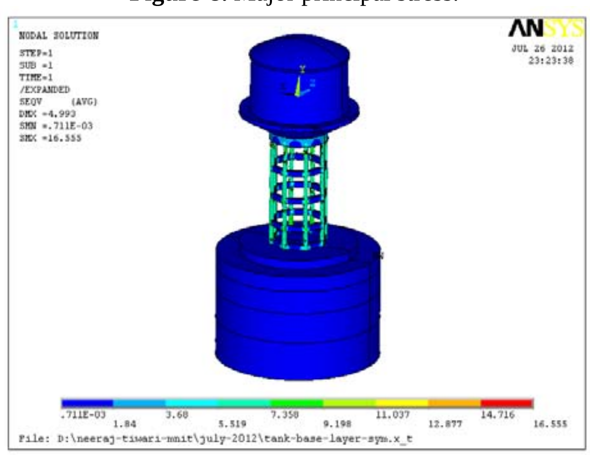


**Figure-9.** Von-mises stress.

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The pressure distribution (pressure bulb) below the soil mass is shown in Figure-10; by this pressure bulb the depth below ground level is found where the vertical pressure intensity in soil is found to be negligible.

The soil mass is not considered in NIA and analyses by IS-code method so these values are not shown in Table-6.

In layered soil mass, softer soil (type-1) is placed just blow the foundation and than soil type-2, type-3, type-4 and type-5 (harder soil) are placed.

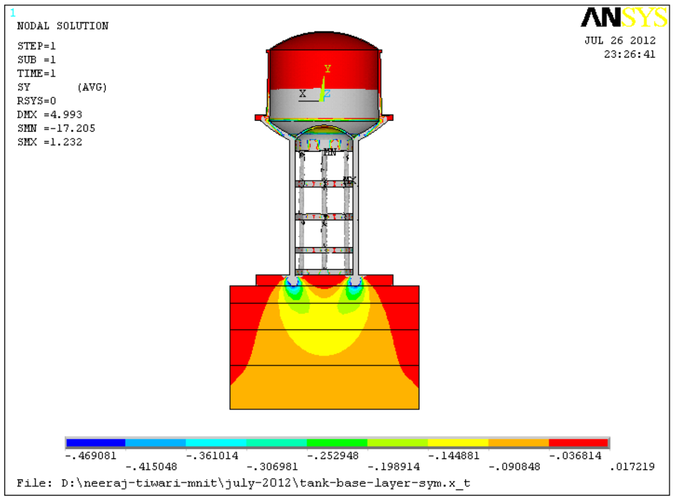


**Figure-10.** Stress distribution in soil mass.

### 7.2 Modal analysis of elevated water tank with sloshing behaviour of water:

Elevated water tank is analyzed for first 10 mode for different filling condition. Table-8 shows the values of natural frequency for LIA+FSI systems for 10 mode shape.

These are the natural frequency of the tank considering sloshing behaviour of water. These natural frequencies are much different from frequency of tank without considering sloshing behaviour of water

**Table-8.** Natural Frequency of the tank for first 10 modes for different filling condition with sloshing behaviour of water. (layered Model).

| S. No. | No. of mode | Natural Frequency of empty tank | Natural Frequency of 20% filled tank | Natural Frequency of 40% filled tank | Natural Frequency of 60% filled tank | Natural Frequency of 80% filled tank | Natural Frequency of full tank |
|---|---|---|---|---|---|---|---|
| 1 | 1 | 1.1343 | 1.0127 | 0.88442 | 0.78894 | 0.70942 | 0.64322 |
| 2 | 2 | 5.9248 | 5.3646 | 5.7228 | 5.6902 | 5.5310 | 5.4010 |
| 3 | 3 | 10.116 | 10.056 | 10.071 | 10.082 | 9.9747 | 6.6657 |
| 4 | 4 | 11.475 | 16.184 | 16.174 | 16.185 | 16.140 | 10.043 |
| 5 | 5 | 16.185 | 20.195 | 20.182 | 20.192 | 16.637 | 15.370 |
| 6 | 6 | 20.200 | 24.731 | 24.727 | 20.307 | 17.367 | 16.178 |
| 7 | 7 | 24.732 | 29.565 | 26.306 | 21.400 | 17.443 | 16.322 |
| 8 | 8 | 30.352 | 30.141 | 28.368 | 22.303 | 20.069 | 16.835 |
| 9 | 9 | 30.868 | 30.846 | 30.892 | 24.620 | 20.098 | 18.348 |
| 10 | 10 | 31.097 | 31.02 | 31.954 | 24.746 | 24.692 | 19.482 |

Table-8 shows the natural frequency of tank for ten mode shape for different filling condition of the tank, Table shows that the value of natural frequency decreases with increase in percentage filling in the tank. Table shows that sloshing effect is maximum between 20-60% fill conditions. When the frequency of external force match with the natural frequency of tank, resonance occurs. So resonance condition will be different for different filling condition of elevated water tank. For the first 10 modes natural frequency of elevated water tank is in the range of

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0.5 to 32. Natural frequency up to $10^{th}$ mode is required for harmonic analysis because earthquake occurs for few second but magnitude of seismic load is much higher than wind load.

### 7.3 Transient analysis of elevated water tank

Transient analysis is the time dependent analysis. For elevated water tank seismic load is the function of time. Generally earthquake occurs for few second. So transient analysis is important analysis when earthquake occurs. Present analysis assumes that earthquake occurs for 5 second in the interval of 1 second and after removal of load effect of seismic load will be analyse. All the steps are used in ANSIS with Load step option: time- time step. The magnitude of Seismic load is calculated for different filling condition of water. Seismic load in term of pressure is applied on projected area of the tank, where pressure is calculated by dividing seismic load by projected area.

#### 7.3.1 Transient analysis of empty tank

Seismic load for empty tank is 333.1 kN and this load is applied in term of pressure (0.002208 N/mm$^2$) on projected area (half portion of tank).

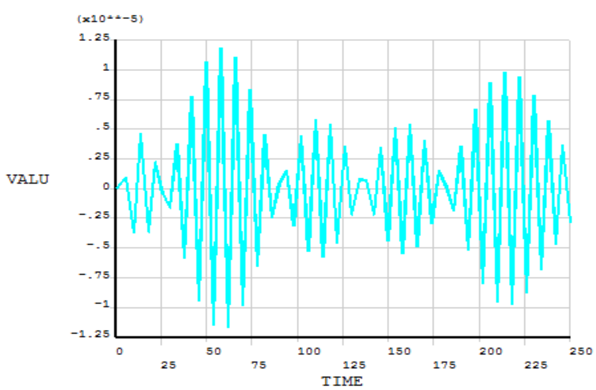


**Figure-11.** Graph between time (s) and acceleration (mm/s$^2$) in horizontal direction at tank empty condition.

Figure-11 shows that value (acceleration) will be maximum at 50s and minimum at 80s, after that acceleration increases or decreases with time and magnitude of acceleration decreases with increasing time. Magnitude of acceleration is very low in case of empty tank condition comparison to other filling condition but amplitude is higher at this condition.

#### 7.3.2 Transient analysis of 20% filled water tank condition

Seismic load for 20% filled water tank condition is 392.99 kN and this load is applied in term of pressure (0.002605 N/mm$^2$) on projected area (half portion of tank).

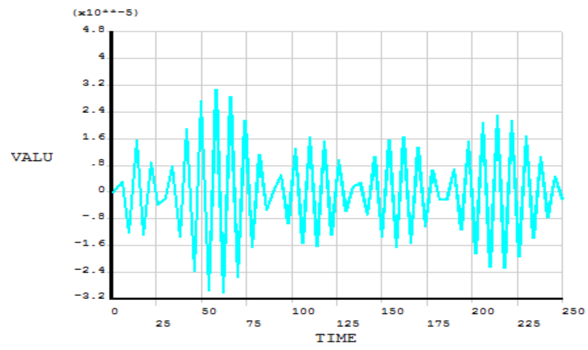


**Figure-12.** Graph between time (s) and acceleration (mm/s$^2$) in horizontal direction at 20% filled condition.

Figure-12 shows that value (acceleration) is maximum at 50 sec. and minimum at 90 sec. after that acceleration increases or decreases with time and magnitude of acceleration decreases with increasing time. Magnitude of acceleration and amplitude is higher in comparison to tank empty condition.

#### 7.3.3 Transient analysis of 40% filled water tank condition

Seismic load for 40% filled water tank condition is 453.93 kN and this load is applied in term of pressure (0.003009 N/mm$^2$) on projected area (half portion of tank).

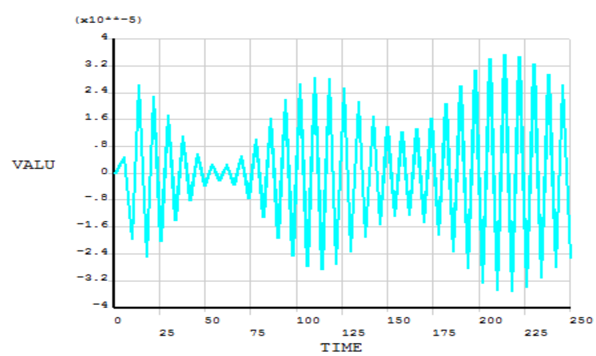


**Figure-13.** Graph between time (s) and acceleration (mm/s$^2$) in horizontal direction at 40% filled condition.

Figure-13 shows that value (acceleration) is maximum at 15s and minimum at 60s after that acceleration increases or decreases with higher rate due to sloshing behaviour of water. Magnitude of acceleration is higher in comparison to 20% filled water tank condition.

#### 7.3.4 Transient analysis of 60% filled water tank condition

Seismic load for 60% filled water tank condition is 515.18 kN and this load is applied in term of pressure (0.003415 N/mm$^2$) on projected area (half portion of tank).

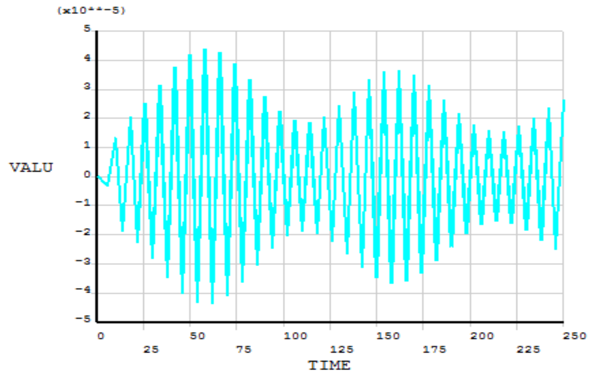


**Figure-14.** Graph between time (s) and acceleration (mm/s$^2$) in horizontal direction at 60% filled condition.

Figure-14 shows that value (acceleration) is maximum at 50 sec. and minimum at 120 sec. after that acceleration increases or decreases with lower rate. The minimum value of acceleration is not nearer to zero at 120s. due to sloshing behaviour of water. Magnitude of acceleration is higher in comparison to 40% filled tank.

### 7.3.5 Transient analysis of 80% filled water tank condition

Seismic load for 80% filled water tank condition is 577.04 kN and this load is applied in term of pressure (0.003825 N/mm$^2$) on projected area (half portion of tank).

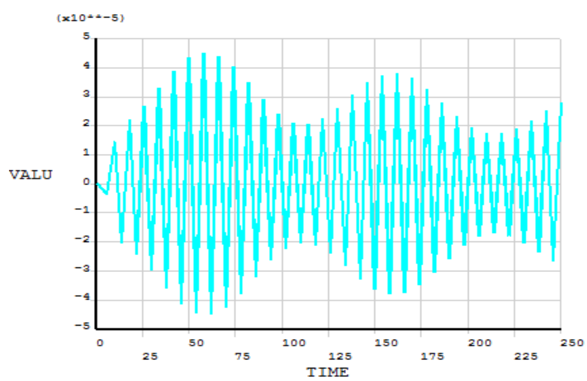


**Figure-15.** Graph between time (s) and acceleration (mm/s$^2$) in horizontal direction at 80% filled condition.

Figure-15 shows that value (acceleration) is maximum at 50 sec. and minimum at 110 sec. after that acceleration increases or decreases with lower rate. The minimum value of acceleration is not nearer to zero at 110 sec. due to sloshing behaviour of water but sloshing affect is low for this filling condition. Magnitude of acceleration is higher in comparison to 60% filled tank.

### 7.3.6 Transient analysis of tank full condition

Seismic load for tank full condition is 626.82 kN and this load is applied in term of pressure (0.004155 N/mm$^2$) on projected area (half portion of tank).

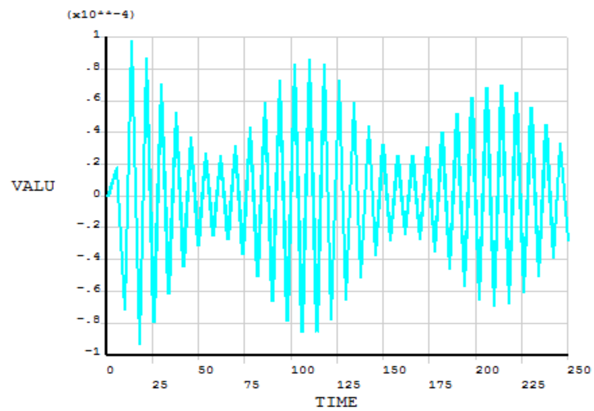


**Figure-16.** Graph between time (s) and acceleration (mm/s$^2$) in horizontal direction at tank empty condition.

Figure-16 shows that value (acceleration) is maximum at 15 sec. and minimum at 60 sec. after that acceleration increases or decreases with lower rate. Magnitude of acceleration is higher in comparison to 80% filled water tank condition but amplitude is higher at this condition. For this condition minimum value of acceleration is nearer to zero at 60 sec. since no sloshing affect occur for this filling condition.

**Table-9.** Shows the values of acceleration and magnitude of acceleration for Transient analysis.

| S. No. | Tank filling condition | Seismic load | Maximum acceleration | Minimum acceleration | Magnitude of acceleration |
|---|---|---|---|---|---|
| 1 | Empty condition | 333.1 kN | 50sec | 80sec | 1.20 |
| 2 | 20% filled with water | 392.99 kN | 50sec | 90sec | 3.10 |
| 3 | 40% filled with water | 453.93 kN | 15sec | 60sec | 3.80 |
| 4 | 60% filled with water | 515.18 kN | 50sec | 120sec | 4.20 |
| 5 | 80% filled with water | 577.04 kN | 50sec | 110sec | 4.40 |
| 6 | Fully filled with water | 626.82 kN | 15sec | 60sec | 9.0 |

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Table-8 shows that Seismic load is maximum at tank empty condition and it increases with %filling in the tank increases since Horizontal seismic coefficient $\alpha_h$ depends upon the weight of fill water in the tank. Maximum acceleration is almost same for different filling condition except 40% filled tank condition since Seismic effect is maximum at this condition, but minimum acceleration is different for different filling condition and maximum value of minimum acceleration occurs at 60% filled tank condition. Above table shows that the magnitude of acceleration increases with % filling in the tank increases but amplitude of vibration decreases when %filling in the tank increases.

## 8. RESULTS AND DISSCUSSIONS

In cylindrical containers, most of the lower liquid mass moves rigidly attached to the container walls (impulsive/bulging mass concept). It is a simplified model of two lumped-masses and is sufficient to describe the overall dynamical behaviour of the interaction system.

There are essentially three independent mass-motion in case of spherical tank which should be considered for a wide range of liquid levels since impulsive mass is not rigidly attached to container walls as it is assumed in case of cylindrical containers.

Curadelli *et al.* [3] proposed three mode shapes viz structural (Translational), sloshing (convecting) and particularly in spherical tank with 20% water filled condition. It is found that the third vibration mode (highest frequency) dominates the dynamical behaviour. The small liquid mass moves as a pendulum out of phase with the structure. When this tank is nearly 50% full, the second vibration mode (intermediate frequency) is the most important. This mode shape is distinguished predominantly by a sloshing liquid mass. If this tank is filled with 80%, then most of the liquid mass oscillates in pendular motion in-phase with the translation of the structure. This is first mode shape (lowest frequency).

In spherical containers, the free liquid surface changes with the filling condition of the tank but in Intze type tank upper portion of the tank is cylindrical and bottom portion of the tank is conical with central spherical dome, therefore, three or at most five modal shapes are sufficient to describe the dynamic behaviour of the interaction system. But for harmonic analysis frequency up to ten mode shape is required for finding out resonance condition.

At higher liquid level where natural frequency is low, the free surface remains plane and most of the liquid mass oscillates in pendular motion in phase with the structure, which is not a critical condition for intze type tank. Thus, in intze type tank first mode shape (lowest frequency) due to oscillating pendular motion is not significant, therefore, only translation and sloshing mode (highest frequency) dominates the dynamic behaviour.

Curadelli *et al*. [3] evaluated natural frequency of the tank system in the range of 1-5 Hz for different water levels by free vibration tests having small vibration amplitudes.

In the present study all possible modes in the range of 0.5-32Hz for different water levels are evaluated by free vibration with small vibration amplitudes since in different seismic zone, frequency may be higher at the time of earthquake. Therefore, non-interaction and interaction analyses have been carried out for calculating natural frequency for ten mode shapes for six different filling conditions of the tank

Table-8 shows that highest natural frequency occurs for tank with empty condition. This natural frequency decreases as water level in the tank increases. The lowest natural frequency occurs under tank full condition. The magnitude of natural frequency is more for higher mode shape. The mode shape with exactly similar natural frequencies represent the same mode in two orthogonal directions x and y).

Table-9 shows that magnitude of acceleration increases with %filling in the tank increases but amplitude of vibration decreases when %filling in the tank increases.

## 9. CONCLUSIONS

In the present work, the linear interaction analysis of intze water tank-fluid-soil system considering layered soil mass is carried out to investigate the interaction behaviour and time dependent (Transient) analysis using the finite element method. The following important research findings are summarized below:

a) The interaction effect causes increase in the stresses in the range of 16-51% in various components of the tank. The maximum principal stress occurs in the circular girder portion. The decrease of nearly 18% is found in the maximum principal stress in the columns.
b) The insignificant interaction effect is found on principal stresses, deflection and natural frequency in various components due to layered soil mass.
c) The natural frequency of the interaction system decreases as the weight of water increases in the tank so failure criteria will be different for different filling condition. The natural frequency of the vibration up to tenth mode contributes to the dynamical response in the range of 0.5-32Hz for different filling conditions. Up to third mode NIA gives all most same natural frequency but natural frequency for (LIA+FSI) increases from 0.5-32Hz up to tenth mode shape. These natural frequencies are very useful for harmonic and transient analysis since the tank collapses when wind load frequency matches with the natural frequency of tank for any mode causing resonance to occur.
d) The interaction analysis causes significant increase in stress of various components of the interaction system. The increase of nearly 16% is found in conical dome, 42% in spherical dome and 51% in the circular girder. The significant decrease of nearly 18% is found in the maximum principal stress the columns. The interaction effect causes decrease in the principal

stress in top dome, side wall and bottom ring beam. The maximum principal stress occurs in the circular girder portion.

e) Principal stresses are same in the various component of the tank for different type of homogeneous soil mass below the foundation or layered soil mass below the foundation of the tank.
f) The maximum von-Mises stresses and maximum deflection take place between 20 to 60% fill condition for all most all ten modes since at this condition sloshing effect is maximum. Deflection and maximum von- mises stresses increases with increasing mode shape.
g) Tank for all filling condition is safe up to 10 mode. But column lying on bending axis are very unsafe under EQ load so circular girder and column lying on bending axis must be design for EQ load and these are the critical component of the tank.
h) Transient analysis shows that acceleration is maximum in case of full tank condition and up to 250s acceleration is not nearer to zero for 40% to 80% filled water tank condition since sloshing effect is maximum in this range.

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