# BEHAVIOUR OF GROUND SUPPORTED CYLINDRICAL WATER TANKS SUBJECTED TO SEISMIC LOADING

*A THESIS*

submitted by

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*for the award of the degree*

*of*

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# CERTIFICATE

This is to certify that the thesis entitled **BEHAVIOUR OF GROUND SUPPORTED CYLINDRICAL WATER TANKS SUBJECTED TO SEISMIC LOADING** submitted by Asha Joseph to the Cochin University of Science and Technology, Kochi for the award of the degree of Doctor of Philosophy is a bonafide record of research work carried out by her under my supervision and guidance at the Division of Civil Engineering, School of Engineering, Cochin University of Science and Technology. The contents of this thesis, in full or in parts, have not been submitted to any other University or Institute for the award of any degree or diploma. All the relevant corrections and modifications suggested by the audience during the pre-synopsis seminar and recommended by the Doctoral Committee have been incorporated in the thesis.

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# DECLARATION

I hereby declare that the work presented in the thesis entitled **BEHAVIOUR OF GROUND SUPPORTED CYLINDRICAL WATER TANKS SUBJECTED TO SEISMIC LOADING** is based on the original research work carried out by me under the supervision and guidance of Dr. Glory Joseph, Professor, Division of Civil Engineering, School of Engineering, Cochin University of Science and Technology, Kochi -22, for the award of degree of Doctor of Philosophy with Cochin University of Science and Technology. I further declare that the contents of this thesis in full or in parts have not been submitted to any other University or Institute for the award any degree or diploma.

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Date:

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# ABSTRACT

**Keywords:** ground supported water tanks, free vibration analysis, aspect ratio, water fill condition, earthquake characteristics, soil - structure interaction


Large liquid containing structures are used nowadays to store different types of liquids such as water, petroleum, chemicals, liquid natural gas and nuclear spent fuel and are important components of human societies and industrial facilities. Water is one of the most precious commodities in tiny metropolitan islands and huge tanks are constructed to meet the water demands. Seismic safety of water tank is of great concern because of the potential adverse economic and environmental impacts associated with failure of the container. To make sure that the water tank is capable to withstand earthquake load, detailed investigation on its seismic behavior is essential.

Dynamic characteristics and seismic responses of ground supported liquid containing structures are affected by configuration of tank, tank wall flexibility, type of wall base joint, characteristics of earthquake and soil structure interaction. An overview on existing codes, standards, and guidelines used in design of liquid storage tanks have been done to check how efficiently these influencing parameters are incorporated in the design guidelines. Detailed review of literature indicated that influence of aspect ratio (height to diameter) and water fill condition of tank under seismic loading are not undergone proper investigation. This study aims at an in-depth analysis on the influence of water fill conditions and aspect ratio of the tank on the dynamic characteristics and seismic response of ground supported concrete cylindrical water tanks through finite element method. The objectives also include to bring out the characteristics of earthquake and soil structure interaction on seismic response by considering the tanks resting on soil of varying properties subjected to seismic

loading of earthquakes of low frequency content Northridge earthquake, medium frequency content Imperial Valley earthquake and high frequency content Koyna earthquake.

Free vibration analyses of both rigid and flexible water tanks have been conducted. Convective mode of vibration is less significant in dynamic behavior of ground supported concrete water tanks and is independent of tank wall flexibility. Fundamental impulsive frequency of water tanks with aspect ratio varying from 0.2 to 2.0 for different water fill conditions is evaluated. The impulsive natural frequency of the tank is observed to be decreased with decrease in water height up to mid height of the tank, and after that the influence of water height on frequency of vibration is marginal, which is not properly incorporated in various codes. Present study proposes a coefficient for determination of impulsive time period of vibration of tanks in any water fill condition.

Seismic responses are studied by performing time history analyses through the evaluation of radial displacement, hoop force, bending moment and base shear. Maximum seismic responses are not always occurred in full fill condition. Under high frequency content earthquake, seismic behaviour of the tank is dominated by characteristics of earthquake whereas for earthquake of low frequency content, the characteristics of the tank determines the seismic behaviour. The fundamental impulsive frequency decreases with decrease in the stiffness of the soil on which it rests. Frequency content of earthquake is the governing factor in the seismic response of tank resting on soil with high stiffness, whereas for tanks on low stiff soil, peak ground acceleration predominates. Results indicate the necessity to include the water fill condition and soil structure interaction effects in the seismic analysis of ground supported tanks.

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# ABBREVIATIONS

| | | |
|---|---|---|
| ACI | – | American Concrete Institute |
| API | – | American Petroleum Institute |
| AR | – | Aspect Ratio |
| AR0.6F | – | Tank with aspect ratio 0.6 in full fill condition |
| AR0.6H | – | Tank with aspect ratio 0.6 in half fill condition |
| AR0.6Q | – | Tank with aspect ratio 0.6 in quarter fill condition |
| AR0.8F | – | Tank with aspect ratio 0.8 in full fill condition |
| AR0.8H | – | Tank with aspect ratio 0.8 in half fill condition |
| AR0.8Q | – | Tank with aspect ratio 0.8 in quarter fill condition |
| AR1.0F | – | Tank with aspect ratio 1.0 in full fill condition |
| AR1.0H | – | Tank with aspect ratio 1.0 in half fill condition |
| AR1.0Q | – | Tank with aspect ratio 1.0 in quarter fill condition |
| AR1.2F | – | Tank with aspect ratio 1.2 in full fill condition |
| AR1.2H | – | Tank with aspect ratio 1.2 in half fill condition |
| AR1.2Q | – | Tank with aspect ratio 1.2 in quarter fill condition |
| AR1.4F | – | Tank with aspect ratio 1.4 in full fill condition |
| AR1.4H | – | Tank with aspect ratio 1.4 in half fill condition |
| AR1.4Q | – | Tank with aspect ratio 1.4 in quarter fill condition |
| ASCE | – | American Society of Civil Engineers |
| BM | – | Bending Moment |
| FE | – | Finite Element |
| FEA | – | Finite Element Analysis |
| FEM | – | Finite Element Method |

| | | |
|---|---|---|
| FSI | – | Fluid Structure Interaction |
| IEQ | – | Imperial valley Earthquake, 1940 |
| IS | – | Indian Standards |
| KEQ | – | Koyna Earthquake, 1967 |
| LCS | – | Liquid Containing Structure |
| NEQ | – | Northridge Earthquake, 1994 |
| NZSEE | – | New Zealand Society for Earthquake Engineering |
| PGA | – | Peak Ground Acceleration |
| PGD | – | Peak Ground Displacement |
| PGV | – | Peak Ground Velocity |
| RMSD | – | Root Mean Square Displacement |
| RMSV | – | Root Mean Square Velocity |
| RSMA | – | Root Mean Square Acceleration |
| SRSS | – | Square Root of Sum of Square |
| SSI | – | Soil Structure Interaction |

# NOTATIONS

| | | |
|---|---|---|
| $\{\ddot{u}_t\}$ | – | Nodal acceleration vector |
| $\{\dot{u}_t\}$ | – | Nodal velocity vector |
| $\{F_t^a\}$ | – | Applied load vector |
| [K] | – | Structural stiffness matrix |
| $\{p\}$ | – | Nodal pressure vector |
| $\{u\}$ | – | Nodal displacement vector |
| [C] | – | Structural damping matrix |
| $[C_F]$ | – | Damping matrix of the acoustic fluid |
| $[C_s]$ | – | Damping matrix of the tank |
| $[K]$ | – | Structural stiffness matrix |
| $[K_F]$ | – | Stiffness matrix of the acoustic fluid |
| $[K_S]$ | – | Stiffness matrix of the tank |
| $[M]$ | – | Structural mass matrix |
| $[M_F]$ | – | Mass matrix of the acoustic fluid |
| $[M_S]$ | – | Mass matrix of the tank |
| $[R]$ | – | Coupling matrix that represents the coupling conditions on the interface between acoustic fluid and structure |
| $C_c$ | – | Coefficient of time period for convective mode |
| $C_i$ | – | Coefficient of time period for impulsive mode |
| $C_{i,\ proposed}$ | – | New coefficient proposed for determination of impulsive frequency |
| $C_l$, $C_w$ | – | Coefficients for determining the fundamental frequency of the tank-liquid system |
| D | – | Inside diameter of circular tank |

| | | |
|---|---|---|
| $E, E_c$ | – | Modulus of elasticity of tank wall |
| f(t) | – | Constant of integration |
| g | – | Acceleration due to gravity |
| H | – | Height of tank wall |
| h | – | Height of water in the tank, maximum depth of liquid |
| h/H | – | Ratio of height of liquid in the tank to height of tank wall |
| hc | – | Height of convective mass above the bottom of tank wall |
| $H_{EQ}$ | – | Maximum hoop force of the tank due to seismic loading |
| $H_F$ | – | Maximum hoop force of full fill tank |
| $H_{F,\ rigid\ base}$ | – | Maximum hoop force of full fill tank with rigid base |
| $H_H$ | – | Maximum hoop force of half fill tank |
| $H_{H,\ rigid\ base}$ | – | Maximum hoop force of half fill tank with rigid base |
| $h_i$ | – | Height of impulsive mass above the bottom of tank wall |
| $H_L$ | – | Design depth of stored liquid |
| $H_S$ | – | Maximum hoop force due to static load |
| Hs | – | Maximum hoop force due to static loading |
| k | – | Bulk modulus of the fluid |
| $K_c$ | – | Spring stiffness of convective mode |
| L | – | Inside length of rectangular tank parallel to the direction of seismic force |
| m | – | Number of axial half waves |
| $m_c$ | – | Convective mass of liquid |
| $M_{EQ}$ | – | Maximum bending moment due to seismic loading |
| $M_F$ | – | Maximum bending moment of full fill tank |

| | | |
|---|---|---|
| $M_{F,\ rigid\ base}$ | – | Maximum bending moment of full fill tank with rigid base |
| $M_H$ | – | Maximum bending moment of half fill tank |
| $M_{H,\ rigid\ base}$ | – | Maximum bending moment of half fill tank with rigid base |
| $m_i$ | – | Impulsive mass of liquid |
| $M_S$ | – | Maximum bending moment due to static load |
| n | – | Number of circumferential waves |
| P | – | Water pressure |
| r | – | Inside radius of the circular tank |
| S1 | – | Soil with properties of hard rock |
| S2 | – | Soil with properties of rock |
| S3 | – | Soil with properties of very dense soil |
| S4 | – | Soil with properties of stiff soil |
| $S_F$ | – | Maximum base shear of full fill tank |
| $S_{F,\ rigid\ base}$ | – | Maximum base shear of full fill tank with rigid base |
| $S_H$ | – | Maximum base shear of half fill tank |
| $S_{H,\ rigid\ base}$ | – | Maximum base shear of half fill tank with rigid base |
| $T_c$ | – | Natural period of convective modes of vibration |
| $T_i$ | – | Fundamental period of oscillation of tank in impulsive mode of vibration |
| $t_w$, t | – | Thickness of tank wall |
| $u$ | – | Fluid velocity in X direction |
| U | – | Maximum radial displacement |
| $U_F$ | – | Maximum radial displacement of full fill tank |
| $U_{F,rigid\ base}$ | – | Maximum radial displacement of full fill tank with rigid base |

| | | |
|---|---|---|
| $U_H$ | – | Maximum radial displacement of half fill tank |
| $U_{H,rigid\ base}$ | – | Maximum radial displacement of half fill tank with rigid base |
| v | – | Fluid velocity in Y direction |
| $v_n(t)$ | – | Velocity component normal to boundary |
| w | – | Fluid velocity in Z direction |
| z | – | Vertical distance measured from bottom of the tank |
| $\gamma_c$ | – | Specific weight of concrete |
| $\eta$ | – | Small displacement of the free liquid surface |
| $\rho$ | – | Mass density of liquid |
| $\Phi$ | – | Velocity potential |
| $\omega_c$ | – | Circular frequency of oscillation of first (convective) mode of sloshing in rad/s |
| $\omega_i$ | – | Circular frequency of impulsive mode of vibration in rad/s |
| $\alpha$, $\beta$ | – | Parameters for Rayleigh damping coefficient |
| $\xi$ | – | Damping ratio |

# CHAPTER 1

# INTRODUCTION

## 1.1 GENERAL

Liquid storage tanks, used for storage of different types of materials such as water, petroleum, chemicals, liquid natural gas and nuclear spent fuel, are important components of human societies and industrial facilities. Water is one of the most precious commodities in metropolitan islands and huge tanks are constructed to meet the needs of the public. The purpose of water storage tanks is to provide storage of water for use in many applications such as drinking water, irrigation, fire suspension, agricultural farming – both for plants and livestock, chemical manufacturing, food preparation as well as many other uses. Required water distribution storage capacity for potable water systems is traditionally met using ground, elevated or standpipe storage tanks or a combination of all three. Though different configurations of liquid storage tanks have been constructed around the world ground supported cylindrical tanks are more numerous than any other type of water tanks because of their simplicity in design and construction, also due to their efficiency in resisting hydrostatic and hydrodynamic applied loads.

Satisfactory performance of the tanks during strong ground shaking is crucial for modern facilities. Water supply is essential immediately following destructive earthquakes, not only to cope with possible subsequent fires, but also to avoid outbreak of diseases. Therefore, large capacity water reservoirs must be safe and need to remain functional after earthquakes. Several tanks have been severely damaged, and some failed with disastrous consequences revealing their vulnerability in major earthquakes and it is essential to ensure the seismic safety of these structures. To make sure that the water tank design is capable to withstand any earthquake load, detailed investigation of complicated fluid-structure interaction must be taken into account. These special considerations account for the hydrodynamic forces exerted by the fluid on the tank wall. Evaluation of hydrodynamic forces requires suitable modelling and dynamic analysis of tank- liquid system, which is rather complex.

## 1.2 BRIEF HISTORY OF FAILURE OF TANKS IN PAST EARTHQUAKES

There are many reports on damage to Liquid Containing Structures (LCS) under some of the major historical earthquakes. The Great Chilean Earthquake (1960), The Niigata Earthquake (1964), The Great Alaska Earthquake (1964), The San Fernando Earthquake (1971), Imperial Valley Earthquake (1979), Northridge earthquake (1994), The Kocaeli Earthquake (1999), The Bhuj earthquake (2001), The Wenchuan earthquake (2008), The Great East Japan (Tohoku) Earthquake (2011) are some of the earthquakes that cause heavy damages to both concrete and steel storage tanks (Soroushina et al., 2011; Hanson, 1973; Cooper, 1997; Jennings, 1971; Nayak, 2013; Suzuki, 2002; Sezen, 2004; Rai, 2002; Krausmann et al., 2010; Hokugo, 2013).

The failure of the tanks and associated disastrous consequences revealed the vulnerability of the liquid storage tanks in major earthquakes. The damage to the tanks may occur due to the combined effect of strong shaking and ground failure, seismic sea wave and conflagration fuelled by destroyed tank oil farms as reported in The Niigata Earthquake (Hanson, 1973). The Niigata and Alaska earthquakes of 1964 resulted in considerable loss in the petroleum storage tanks. The significant losses during earthquake attracted many practicing engineers and researchers to further investigate the seismic behaviour of liquid storage tanks.

The failure mechanism of liquid storage tanks depends on different parameters such as construction material, tank configuration, and the support conditions. It should be noted that failure mechanism of concrete tanks may be different from that of steel tanks under the effect of seismic loads. The main causes of damage to concrete LCS are due to deformations, rupture of tank wall at the location of joints with pipes, cracks in the ground supported reinforced concrete tanks, collapse of supporting tower of elevated tanks, and leakage in the tank wall etc. (Jaiswal et al., 2008; Sezen et al., 2008). The poor performance of critical facilities like water tanks needs careful scrutiny of their design.

The calamities associated with failure of flammable liquid storage tanks are numerous as reported in The Wenchan earthquake, 2008. The failure of pipes of ammonia storage tanks and damage of supporting structure of sulphuric acid storage tanks indicates that the ignition probability of a flammable substance is rather high upon release during an earthquake (Krausmann et al., 2010). The tsunami after an earthquake can have huge impact on liquid containing structures, especially for oil tanks located in the ports as happened in The Great East Japan (Tohoku) earthquake, 2011 (Hokugo, 2013).

The significance of preventing damage to liquid containing structures has led to extensive research study on the dynamic behaviour of liquid containing structures. These studies resulted in a better understanding and knowledge of these structures under seismic loads.

### 1.3 SEISMIC ANALYSIS OF GROUND SUPPORTED TANKS

Seismic analysis of liquid storage tank is complicated due to the complicated fluid-structure interaction of the system. Numerous studies have been carried out on seismic behaviour of ground supported tanks, initial studies were based on rigidity assumption of tank wall, later the flexibility of wall is also taken into account. For rigid tank model, the tank wall is considered to be rigid and experience the same motion as the ground support. The motion of the tank shell wall in the flexible model is no longer the same as that of the ground support, but affected by the ground excitation.

Availability of mechanical models, Housner's (1963) spring - mass model for rigid tank along with modifications by other researchers to incorporate the tank wall flexibility has considerably simplified the analysis procedure of cylindrical tanks. These mechanical models (Jacobson 1949, Housner 1963, Veletsos and Yang 1976, 1977, Haroun and Housner 1981) convert the tank-liquid system into an equivalent spring- mass system. Design codes use these mechanical models to evaluate seismic response of tanks. The parameters get associated with the analysis while using this approach are: pressure distribution on tank, time period of vibration, base shear, moment at base and hoop force. Design codes suggest the expressions for determination of these responses of the tank at design water height.

### 1.3.1 Fluid-structure interaction

Seismic energy is transferred from the ground to the fluid through the motion of the tank. A portion of the liquid accelerates with the tank whereas the remaining liquid is assumed to slosh. Sloshing occurs in the upper part of the liquid, which does not displace laterally with the tank wall. Hydrodynamic response can be separated into impulsive motion, in which liquid is assumed to be rigidly attached to tank and moves in unison with tank wall and convective motion, characterized by long period oscillations and involves vertical displacement of fluid's free surface. The division of the hydrodynamic pressure into the impulsive and convective parts has proved to be of great value in the analysis of tanks excited laterally. The impulsive pressures are associated with the forces of inertia produced by impulsive movements of the walls of the container, and the pressure developed is directly proportional to the acceleration of the container walls. The convective pressure is produced by oscillations of the fluid and is the consequences of the impulsive pressure. Because of large differences in natural periods of impulsive and sloshing responses, these two actions can be considered uncoupled (Housner, 1963; Haroun and Housner, 1981; Malhotra and Veletsos, 1994).

### 1.3.2 Soil-structure interaction

During earthquake, the behaviour of any structure is influenced not only by the response of the superstructure, but also by the response of the soil beneath. A seismic soil-structure interaction (SSI) analysis evaluates the collective response of three linked systems: the structure, the foundation, and the geologic media underlying and surrounding the foundation to a specified free-field ground motion. Structural failures in past have shown the significance of soil-structure interaction (SSI) effects on seismic response of water tanks.

## 1.4 MOTIVATION AND RELEVANCE OF PRESENT STUDY

The failure of large number of water tanks during past earthquakes generated lot of interest among researchers to safeguard these tanks against seismic forces. The necessities for the tank to remain functional after an earthquake make it essential to design the tanks to withstand the hydrodynamic forces developed during an earthquake. This research which focuses to identify the parameters that adversely affect the seismic safety of water tanks is relevant based on lessons learnt from the damages of water tanks during earthquakes. Seismic analysis of the tanks are performed with due consideration to parameters such as aspect ratio, water fill condition, characteristics of earthquake and properties of soil. The seismic behaviour of water tank is very complicated due to combined fluid- structure and soil- structure interaction effects. Proper analysis and design of tanks are essential to minimize the failure of tanks in the future associated with calamities.

### 1.5 ORGANIZATION OF THE THESIS

**Chapter 1** presents general introduction on seismic analysis, modelling of ground supported tank and soil-structure interaction. Discussion on failure of liquid storage tanks during past earthquake discloses the necessity of study on seismic response of water tanks

**Chapter 2** discusses the published works in the proposed area. Literature on different possible failure mechanisms of water tanks identifies the response parameters to be considered for the study. Mathematical model proposed for analysis of rigid and flexible tanks help to distinguish the assumptions incorporated in the mathematical modelling of water tanks and how the real life situation differs from the modelling. Detailed review on literature in proposed area is to assess past studies in this area and to identify the gaps that are to be undergone further investigation.

**Chapter 3** outlines the objectives and scope of the present work based on the extensive review of literature.

**Chapter 4** presents the numerical modelling of the fluid – structure – soil system. The equations governing the fluid modelling is given in detail. The fluid – structure interface and soil – structure interface modelling using the finite element techniques are also included in this chapter.

**Chapter 5** gives the free vibration analyses and dynamic characteristics of ground supported cylindrical concrete water tanks. Effect of wall flexibility and role of convective component on dynamic response of water tanks are investigated. Significance of water fill condition on seismic behaviour of tank is ensured by performing the free vibration analyses of tanks of different aspect ratio and highlighted the necessity to incorporate the water height in the expression for determination of fundamental frequency / time period of vibration suggested by various design codes.

**Chapter 6** discusses the seismic behaviour of ground supported cylindrical water tanks. The effects of water fill condition, aspect ratio and characteristics of earthquake on response of tanks were examined by carrying out time history analyses. Tanks of different aspect ratios under three water fill conditions – full fill, half fill and quarter fill were subjected to the seismic loading. Comparisons are also made with response of the tanks to hydrostatic pressure.

**Chapter 7** explains the role of soil-structure interaction on the seismic performance of water tanks. To achieve this, time history analyses of tanks resting on four different soil types of varying properties were performed. Comparison of obtained results with tank with fixed base slab indicated the role of SSI on seismic response of ground supported water tanks.

**Chapter 8** summarises salient conclusions arrived from the study. The scope of future work is also discussed at the end of the chapter.

# CHAPTER 2

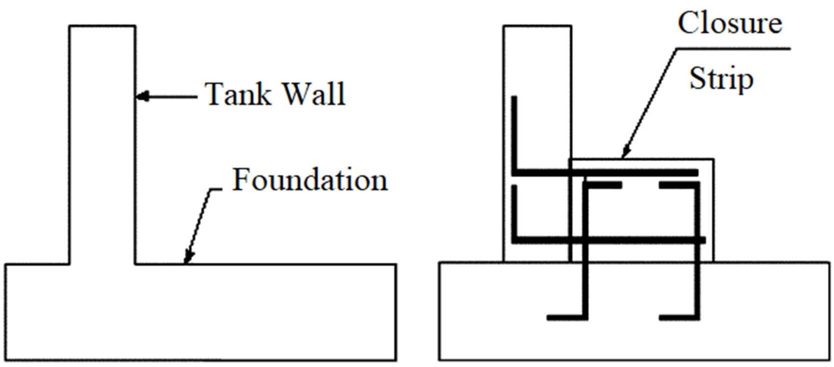
# REVIEW OF LITERATURE

## 2.1 GENERAL

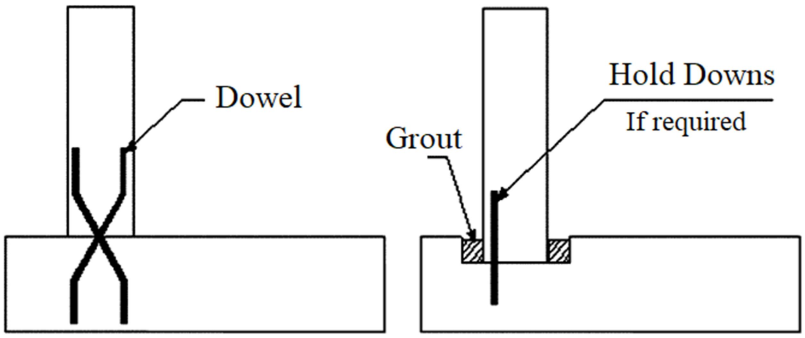
Dynamic behaviour of ground supported liquid containing structures are affected by configuration of tank, type of wall-base joint, wall flexibility, characteristics of load and soil-structure interaction. Critical review on the damages during earthquakes and identification of causes of failure are essential in the design of earthquake resistant design of structures. Hence a systematic review on the dynamic behaviour of various types of liquid retaining structures subjected to earthquake loading has been made in this chapter to identify the need for the present study. An overview on standards and guidelines used in design of liquid storage tanks are also presented.

### 2.1.1 Classification of Ground Supported Tanks

Ground supported concrete tanks can be classified (ACI 350.3: 2006) on the basis of (i) General configuration (rectangular or circular) (ii) Wall-base joint type (fixed, hinged or flexible base) and (iii) Method of construction (reinforced or prestressed concrete). Based on configuration, tanks are classified into rectangular and circular. Fixed and hinged tanks are of non- flexible wall base joints whereas flexible base connections are possible in anchored and unanchored tanks. The ground supported concrete tanks can be either reinforced or of prestressed concrete. Figures 2.1 (a) and (b) show the configurations for two types of nonflexible base connections: fixed base tanks and hinged base tanks respectively. For fixed base support, no movement or rotation are allowed at the wall base. The bending moment at tank base is resisted by vertical reinforcement connecting tank base and tank wall. For fixed base connection with closure strip, no vertical reinforcement extends between the base and the wall, while the fixity between the wall and tank base is provided through the closure strip. For hinged base support, no bending moment is transmitted between the tank wall and base by allowing the rotation. For fixed and hinged base tanks, the earthquake base shear is transmitted partially by membrane (tangential shear) and the rest by radial shear that cause vertical bending (Hafez, 2012).



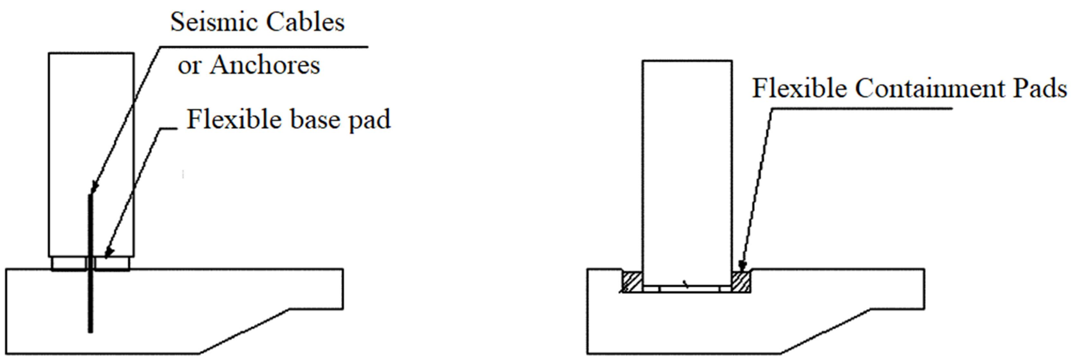
(a) Fixed



(b) Hinged

Fig. 2.1 Non Flexible base connections of ground supported tanks (ACI 350.3-06)

In the case of anchored tanks shown in Fig. 2.2, the tank wall bottom edge is fixed to a proper base which may be a concrete ring foundation. Uplift of the tank bottom edge is not possible. For unanchored, flexible-base tanks, it is assumed that the base shear is transmitted by friction only. The anchored, flexible-base support consists of seismic cables connecting the wall and the footing, as well as elastomeric bearing pads. Because of economic and technical reasons, most medium to large liquid storage tanks in the field are unanchored. Compared to anchored liquid-tank systems, the unanchored liquid-tank systems exhibit much more complicated seismic behaviour. This is because of the possibility of base plate uplift, which occurs when the resultant overturning moment generated from the liquid motion is large enough. For an unanchored tank subjected to a sufficiently large resultant overturning moment, a portion of the base plate will separate from the support foundation. Accompanying with the base plate uplift, extensive deformations with significant out-of-round distortion occur in the tank shell wall.




(a) Anchored (b) Unanchored

Fig. 2.2 Flexible base connections of ground supported tanks (ACI 350.3 – 06)

## 2.2 FAILURE MECHANISM OF LIQUID STORAGE TANKS

Liquid storage tanks involve various modes of failure mechanisms. A wide variety of failure mechanisms are possible depending upon the configuration of tank geometry, possible fluid-structure-soil interaction, the tank material, type of support structure, etc. Characteristics of earthquakes also significantly influence the response of liquid storage tanks. Failure modes of rectangular tank are significantly different from those of cylindrical, spherical, and conical tanks. Similarly, the failure patterns for rigid tank considerably differ from those for flexible tanks. The damage mode in concrete tanks is different from that of steel tanks. Elephant-foot buckling, anchorage system failure, and sloshing damage to the roof and upper shell of the tank are the most common damages in steel tanks (Minoglou, 2013). Stresses caused by large hydrodynamic pressures together with the additional stresses resulted from the large inertial mass of concrete could cause cracking, leakage and ultimately failure of the concrete tank. That is why the design criteria for concrete tanks are based on crack control (Hashemi et al., 2013).

Reported damage to liquid containing structures during past earthquakes fall into one or more of the failure mechanism such as (i) buckling of the shell caused by excessive axial compression of the shell structure due to exerted overturning moment (elephant-foot buckling), (ii) deformation, cracks and leakage in side shell, (iii) damage to the roof or the upper shell of the tank, due to sloshing of the upper portion of the contained liquid in tanks with insufficient free board provided between the liquid free surface and the roof, (iv) spill over of the stored liquid, (v) failure of piping and other accessories connected to the tank because of the relative movement of the flexible shell, (vi) damage to the supporting structure in elevated water tanks, (vii) damage to the anchor bolts and the foundation system and (viii) failure of supporting soil due to over-stressing (Moslemi, 2011). Different combinations of above possible parameters make the failure mechanism more complex.

Failure of the supporting structure is one of the main reasons behind the failure of the elevated concrete water tanks during earthquakes. The cracks and subsequent fracture of elevated water tank due to shear failure of the beams in the RCC staging was reported during Chile earthquake, 1960 (Soroushina et al., 2011). Failure of shaft staging having low seismic energy absorption capacity has caused the collapse of elevated water tanks in Bhuj earthquake, 2001. Majority of these tanks, supported on cylindrical shaft staging, developed circumferential flexural cracks near the base (Rai, 2003; Dutta et al., 2009).

Elephant foot buckling failure mechanism commonly occurring in steel tanks, is an outward bulge just above the tank base which usually occurs in tanks with a low height to radius ratio. By performing experimental study, Niwa and Clough (1982), concluded that the elephant foot buckle mechanism results from the combined action of vertical compressive stresses exceeding the critical stress and hoop tension close to the yield limit. However, Rammerstorfer et al. (1990) attributed the bulge formation to three components; the third being the local bending stresses due to the restraints at the tank base. The elephant foot buckle often extends around the circumference of the tank (Hamdan, 2000).

Unanchored or partially anchored tanks may undergo local uplift when the magnitude of the overturning moments exceeds a critical value. As a result, a strip of the base plate is also lifted from the foundation. Although uplift does not necessarily result in the collapse of the tank, its consequences include serious damage to any piping at the connection with the tank and an increase in the axial stress acting on the tank wall.

Sloshing waves of high amplitude often cause damage to the roofs of tanks and render them temporarily unserviceable. As a consequence, liquid spillage over the roof may either result in fires or in the loss of water supply used in putting out fires. Base sliding can cause extensive damage to inlet/outlet piping unless provisions are made to accommodate vertical and horizontal movements of pipe triggered by base sliding.

## 2.3 DYNAMIC ANALYSIS OF LIQUID CONTAINERS

By evaluating the performance of water tanks during past earthquakes, it is obvious that these tanks are to be designed properly to reduce earthquake effects on liquid storage tanks. The tank and the supporting structure should be designed to withstand the hydrodynamic forces developed during the earthquakes. For this purpose, dynamic behaviour of water tank considering the complicated fluid-structure interaction has to be investigated. Different tank models developed for the dynamic analysis of ground supported tanks and performance evaluation of the tanks are discussed in the following section.

### 2.3.1 Tank Models for Dynamic Analysis

***Rigid Tank Model***

An equivalent single-degree-of-freedom spring-mass model is used to simplify the analysis of water tank and to evaluate the maximum resultant lateral force and overturning moment. In the simplified model, the effect of the impulsive mode is represented by the impulsive mass '$m_i$', attached to the tank shell wall at height '$h_i$' by a rigid bar, whereas the effect of the convective mode is represented by the convective mass '$m_c$', attached to the tank shell wall at height '$h_c$', by a spring with stiffness '$K_c$'. The height '$h_i$' and '$h_c$' are determined such that the simplified model has the same overturning effect as the respective liquid motion they represented. Fig. 2.3 gives the spring – mass model for ground supported circular /rectangular tank.

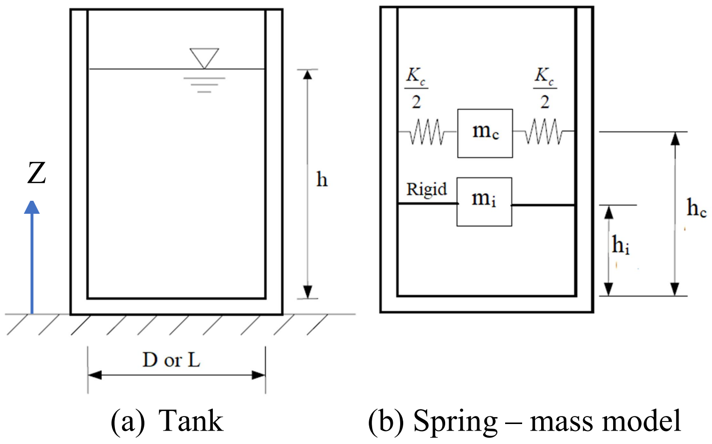



Fig. 2.3 Spring – mass model for ground supported circular and rectangular tank

For rigid tank model, the tank shell wall is considered to be rigid and experience the same motion as the ground support. The boundary conditions of the liquid motion are:

i) At the base plate, ie at z = 0, the vertical velocity of the liquid particles is zero.

ii) At the tank shell wall, the radial velocity of the liquid particles is the same as that of the tank shell wall, which equals to that of the ground motion.

iii) At the liquid free surface, z = h, the liquid pressure is zero.

***Flexible Tank Model***

The impulsive component satisfies the boundary conditions of the liquid motion at the liquid-tank interface and the convective component satisfies the boundary condition at the liquid free surface. The convective liquid motion is of much longer period compared to the tank shell wall vibration and it is assumed that the coupling between the convective liquid motion and the tank shell wall vibration is very weak (Malhotra and Veletsos, 1994; Virella et al., 2006).

Based on the following boundary conditions of the liquid motion for the flexible tank model (Veletos, 1974);

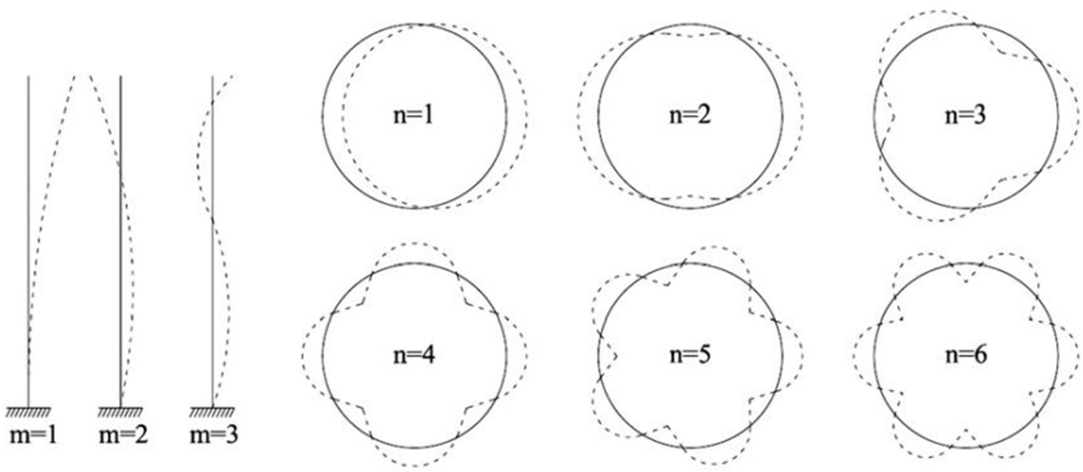
i) at the base plate, ie at $z = 0$, the vertical velocity of liquid particles is zero
ii) at the tank shell wall, the radial velocity of liquid particles is the same as that of the tank shell wall. Because of its flexibility, the motion of the tank shell wall is no longer the same as that of the ground support, but affected by the ground excitation, the hydrodynamic load and the vibration of the shell wall itself and
iii) at the free surface of the liquid content, the liquid pressure is zero. Neglecting the vertical inertia effect of the wave, the hydrodynamic pressure at $z= h$ is equal to the weight of the liquid column above

it has been concluded by earlier researchers that (Veletos,1974; Yang, 1976; Veletsos and Yang, 1976,1977; Veletsos and Kumar, 1984) (i) the flexibility of the tank shell wall will change the temporal variation and the magnitude of the liquid motion, but has little effect on its spatial distribution (ii) the impulsive liquid motion increases from zero at the liquid free surface to the maximum near the bottom of the tank shell wall, (iii) the convective liquid motion decreases from the maximum at the liquid surface with depth. The effect of the higher modes of the convective liquid motion can be neglected.

As compared to the hydrodynamic effect, the contribution of the mass of the tank structure in the inertia effect is proved to be negligible and is thus usually neglected in the formulation (Housner, 1963; Moslemi et al., 2011).

### 2.3.2 Modes of Vibrations of Cylindrical Tank

A cylindrical tank can vibrate in many different modes under dynamic loading. In the axial mode, tank behaves like a vertical cantilever beam whereas is in the circumferential mode, tank shell vibrates in and out (Haroun, 1980). As shown in Fig. 2.4 any of the modes can be specified by two integer parameters ‘m’, the number of axial half waves, and ‘n’, the number of circumferential waves (Amiri and Yazdi, 2011).




Axial modes Circumferential modes

Fig. 2.4 Vibration modes of cylindrical tank (Amiri and Yazdi, 2011)

Barton and Parker (1987) used finite element models of tank-liquid systems to study the seismic response of anchored and unanchored tanks. For cylindrical tanks with height/diameter larger than 0.5 under horizontal excitation, it is stated that those modes involving deformations of the cylinder with the form cos(nθ) and n > 1 have very small participation factors, and are not important in predicting the response Thus, only the cantilever beam mode (i.e. n = 1) would be fundamental in predicting the horizontal seismic response for tanks with height/diameter is greater than 0.5 (Barton and Parker, 1987). Later Virella et al. (2006) showed that even for tanks with an aspect ratio equal to 0.4, which falls outside the range considered by Barton and Parker (1987), modes with n > 1and m ≥ 1 have very small participation factors in the direction of the horizontal motion.

### 2.3.3 Analysis of Ground Supported Tanks

The first solution to the problem of dynamic fluid pressure was by Westergarrd (1933) who determined the pressure on rectangular vertical dam under the action of horizontal acceleration. One of the earlier works on liquid storage tanks were by Hoskins and Jacobsen (1934) for the measurement of impulsive fluid pressure. In the analytical procedures proposed by Hoskins and Jacobsen (l934) and Jacobsen (1949), the fluid is assumed to be incompressible and inviscid, the effect of the gravity waves is excluded from the analysis and only the impulsive motion of the liquid content is considered. Hoskins and Jacobsen (l934) employed the Laplace's differential equation in terms of a potential velocity function to express the liquid motion.

Jacobson (1949) solved the problem of dynamic analysis of cylindrical tanks containing fluid whereas Werner and Sundquist (1949) extended the work to include rectangular fluid containers. A more complete analysis of impulsive and convective pressures in rectangular container was proposed by Graham and Rodriguez (1952). All these analyses were required to find out solution to Laplace equation that satisfies the boundary conditions.

Housner (1957, 1963) investigated the problem of earthquake pressure on rigid fluid containers and derived solutions by approximate method that avoid partial differential equations and series and presented the solution in simple closed form. The liquid was assumed to be incompressible and undergo small displacement. Applying the Hamilton's principle, he had derived the expression for hydrodynamic pressure exerted by fluid on tank wall, moment on wall and frequency of vibration of rectangular, circular, elliptical and composite tanks. Due to simplicity, Housner's two mass approximation has been adopted in many current standards and guides such as ACI 350.3-06 and ACI 371R-16 with some modifications which were the results of subsequent studies by other researchers for seismic design of liquid storage tanks (Hashemi et al., 2013). In Housner model, the shell is assumed to be massless and only mass of water is considered in derivation of equations (Moslemi. et al., 2011).

Dynamic response of flexible tank can be determined by the simple procedure proposed by Veletsos (1974) and Veletsos and Yang (1977), which is formulated on modification of the expressions governing the response of a similarly excited rigid tank. The fundamental basis for this procedure is the assumption that the whole structure mass vibrates in the first mode of vibration and the effect of tank inertia is neglected in this procedure. Only impulsive forces, which are induced based on the assumption of no gravity surface wave is considered. The convective effects cannot be influenced significantly by the flexibility of tank as they are characterized by oscillations of much longer periods than those characterizing the impulsive effects. Forces induced due to the deformation of the tank wall are also included for the determination of equivalent static seismic load. The impulsive components of the response of the flexible tanks are determined by replacing the ground acceleration in the relevant expressions of the rigid tank solutions by the pseudo-acceleration function (Veletsos and Yang, 1977).

Analytical model for the prediction of seismically induced stresses on flexible tank wall, which is valid for excitation peak horizontal displacement as large as 0.065 times radius of tank was proposed by Kana (1979) whereas Housner's rigid cylinder slosh model is valid only up to 0.03 times of radius of tank. The charts that facilitate the calculations of the periods of vibrations of vibration of cylindrical tanks, the effective masses and their centres of gravity were suggested by Haroun and Housner (1981) after studying the behaviour of deformable liquid storage tanks using modal superposition method. Haroun and Ellaithy (1985) extended the mechanical model proposed by Housner to account for rocking motion. The extended model was used by Haroun and Ellaithy (1985, A) to study the response of cylindrical tanks under horizontal excitations.

Seismic responses such as base shear, overturning moment, and sloshing wave height can be calculated by using simplified seismic design procedure for cylindrical ground-supported tanks proposed by Malhotra et al. (2000). The simplified procedure has been adopted in Eurocode 8 considers impulsive and convective vibrations of the liquid in flexible steel or concrete tanks fixed to rigid foundations.

Simplified Equivalent Section Method (ESM), based on replacing the concrete wall and steel shell with a single wall with an equivalent thickness and Young’s modulus, is proposed by Elansary and Damatty (2018) for the prediction of frequency and dynamic behaviour of composite conical tanks.

The effects of various parameters on the dynamic response of ground supported tanks were studied by several researchers, both for flexible and rigid tanks. Since it is difficult to perform the experimental studies, most of them are based on analytical models using finite element software.

The material of the tank may significantly affect the response of the tank. Reinforced concrete tanks are found to be more susceptible to higher dynamic pressure than steel or aluminium tanks upon the action of horizontal excitation (Gupta and Hutchinson, 1989).

The fundamental modes for the cylindrical tank liquid systems are influenced by the geometry of the tank. Clough et al., (1979) classified the tanks based on aspect ratio, the ratio of height of tank wall to diameter of tank, as 'Shallow' ($H/D \leq 1.0$) and 'Deep' ($H/D > 3.0$).

The fundamental modes of cylindrical tank models with aspect ratios larger than 0.63 are very similar to the first mode of a cantilever beam. For the shortest tank, having aspect ratio 0.4, the fundamental mode was observed to be a bending mode with a circumferential wave $n = 1$and an axial half-wave (m) characterized by a bulge formed near the mid-height of the cylinder (Virella et al., 2006). The contributions of modes higher than the first one can be neglected in the computation of sloshing pressure or surface wave amplitude of rectangular ground supported tanks (Virella et al., 2008). Both impulsive and convective modes of vibration of the conical overhead water tank are practically dominated by its fundamental modes (Moslemi et al., 2011).

For the tank with roof, the roof of steel tank does not affect the natural frequency of vibration of tank. This is because, the mass of the tank roof is a very small fraction of the total mass of the tank and the frequency of vibration depends on the total mass of the tank. As the roof does restrain the tank top against the radial deformations; it has considerable influence on the mode shapes of tank (Amiri and Yazdi, 2011).

The ratio of liquid height to tank radius is found to be the most important single parameter governing the uplift response of tanks (Malhotra and Veletsos,1994). Though it is observed by Gupta and Hutchinson (1989) that maximum dynamic pressure developed in partially filled tanks may be more critical than those for the same tank at maximum capacity, studies in this domain is not sufficient to identify the dynamic behaviour of tank with different water fill conditions. For tanks on rigid foundation, the frequencies of beam modes of vibration are found to be reduced by the presence of liquid. Significant dependence of the radial shell mode shapes on the filling ratio is confirmed by both finite element analysis and holographic interferometry by Kruntcheva (2007), by investigating the response of clamped free cylindrical tanks resting on rigid and flexible foundations.

The natural frequency of the intze water tank- fluid- soil system decreases as the weight of water increases in the tank so failure criteria will be different for different filling conditions (Tiwari and Hora, 2015).

Since the sloshing and impulsive components of the response do not reach their peak value at the same time, the effect of considering sloshing on total response might be either increasing or decreasing (Moslemi et al., 2011). For composite conical tanks, sloshing has insignificant effect on calculated base shear forces under horizontal excitations (Elansary and Damatty, 2018).

### 2.3.4 Effect of Wall Flexibility on Dynamic Behaviour of Water Tanks

The flexibility of tank wall has considerable effect on the dynamic behaviour of water tanks. Though Housner's two mass model was initially developed for rigid tanks, studies considering the flexibility of tank wall were performed by many researchers as the assumptions in rigid wall modelling doesn't simulate the real physical condition.

Veletsos (1974) has noticed that the seismic effects in flexible tanks are substantially greater than those in similarly excited rigid tanks. The increased response of flexible tank is due to the magnification of pressures developed in liquid and exerted on tank, thereby increasing the base shear and overturning moment, especially for tanks on stiff soils (Haroun and Izzeddine, 1992). The hydrodynamic effects for flexible tank under vertical excitation may also be larger than those induced in rigid tank of same dimension (Veletsos and Kumar, 1986).

But, later Kianoush and Chen (2006) observed that while considering the vertical acceleration along with horizontal acceleration, the effect of tank wall flexibility can either increase or decrease the response compared with that of rigid wall boundary conditions since the final dynamic response is the combination of the effects due to the rigid wall boundary condition and transverse vibration of the tank wall. This observation is based on studies of seismic response of ground supported rectangular tanks. Moslemi and Kianoush (2012) suggested to consider the wall flexibility in the seismic design of tanks as the wall flexibility resulted in significant increase in impulsive part of response under horizontal vibrations. But the convective part of response was found to be independent of wall flexibility.

The convective effects are characterised by oscillations of much longer periods than of impulsive effects (Veletsos, 1974). The convective component of responses of tanks under harmonic and seismic excitations are insensitive to the flexibilities of the tank walls and may be computed considering the tank wall and the supporting medium to be rigid (Veletsos et al.,1992; Moslemi and Kianoush, 2012).

The sloshing height of liquid inside the tanks is not significantly affected by wall flexibility (Moslemi and Kianoush, 2012). The hydrodynamic pressure in the middle of the flexible storage tanks are generally larger than for rigid storage tanks and it varies not only in vertical direction but also in horizontal direction over the wall surface (Hashemi et al., 2013).

### 2.3.5 Effect of Base Fixity on Dynamic Behaviour of Water Tanks

The mode of failure of liquid storage tank depends upon the connection between the wall and floor as well the fixity at the base of the tank.

The dynamic response for unanchored liquid tank system is determined by the uplift mechanism of base plate and is highly nonlinear to the applied overturning moment (Feng, 1997). The overturning moment developed on unanchored tank is higher than that of anchored tank and the consequent base uplifting completely changes the dynamic characteristics of the system, whereas according to design codes the hydrodynamic overturning moments are assumed to be insensitive to the support condition (Ozdemir et al., 2012). Unanchored tank systems with liquid height equal or less than tank radius is more vulnerable to the seismic load as the effect of liquid motion leads to overturning moment and significant base plate uplift (Feng, 1997). The fundamental natural frequency of unanchored tank is less than the same tank being anchored at base (Maheri et al., 2016).

The sloshing response of fluid free surface is almost insensitive to the support condition at the tank base (Ozdemir et al., 2010). Moslemi and Kianosh (2012) observed that sloshing height is insensitive to the type of connection at the base of the tank. Generally larger structural response values are developed in the walls of a hinged tank as compared to a fixed tank (Moslemi and Kianosh, 2012).

Effect of base isolation on the behaviour of tanks were analysed by Seleemah and Elsharkawy (2011) and detected that for non-isolated tanks the base shear and impulsive displacements are significantly affected by the aspect ratio. The impulsive displacements of non- isolated tanks are considerably increasing with aspect ratio leading to high probability of local buckling occurrences in the steel tanks. On the other hand, for isolated tanks, these responses are significantly reduced and show a negligible increase with aspect ratio.

### 2.3.6 Effect of Vertical Component of Acceleration on Dynamic Behaviour

Though the structures are subjected to the three dimensional earthquake ground motions, only the horizontal motion has been extensively studied and considered in the design process whereas the vertical component of the ground motion has generally been neglected. The vertical component of the ground motion is associated with vertically propagating the P-waves. In most of the current codes, the vertical excitation effect is accounted for by assuming the two thirds of the horizontal response spectrum as the vertical acceleration. The relationship between the arrival times of peak vertical motion with the peak horizontal motion is the governing factor to determine the significance of vertical component of earthquake motion (Shrestha, 2009).

Veletsos and Ashokkumar (1984) proposed a method for the evaluation of the dynamic response of vertically excited cylindrical liquid storage tanks and observed that the hydrodynamic effects for a flexible tank may be substantially larger than those induced in a rigid tank of same dimension.

The tank wall undergoes horizontal displacement in addition to axial displacement due to the vertical excitation. The vertical ground motion produces only 3% of total response displacement and 4.6% total of base moment, when the effects of horizontal and vertical accelerations are combined using SRSS method. But the vertical component of ground motion may have relevance in the analysis of liquid containing tanks, for the near-field zones (Kianoush and Chen, 2006).

The significance of vertical component of earthquake on dynamic behaviour depends on the geometry of the tank too. Kianoush and Ghammaghami (2011) studied the response of 'shallow' and 'tall' rectangular concrete water tanks upon the application of vertical component of four earthquake records. It is found that, for 'shallow' tank with rigid base, the impulsive behaviour is less sensitive to vertical motions whereas convective part increases due to vertical excitation and depends on frequency content of earthquake too. In comparison to 'shallow' tank, the 'tall' tank is more sensitive to vertical component of ground motion and its response varies from one earthquake to another. Since the impulsive response values are so much higher than those of convective, it is concluded that, the effect of vertical excitation is insignificant on overall seismic behaviour of water tanks.

Haroun and Tayel (1985) concluded that sloshing can be neglected in the analysis of tanks subjected to vertical excitation. But the findings of Moslemi and Kianoush, (2012) are contrary to this. According to Moslemi and Kianoush (2012), the vertical ground acceleration has a greater effect on the convective component of response than on the impulsive component. The sloshing height also increases as a result of combined effect of vertical ground motion. Vertical acceleration has relatively greater effect on the response of 'tall' tank as compared to the 'shallow' tank.

Rawat et al. (2015) also noticed that vertical component of an earthquake causes an increase in sloshing displacement of contained liquid. So, it is essential to take into account the increase in sloshing displacement due to vertical component of earthquake while designing freeboard for the tanks on seismic prone regions. The vertical component of the earthquake has pronounced influence on the convective component than the impulsive component of the base shear (Rawat et al., 2015).

The effect of vertical acceleration on impulsive response of the tanks subjected to combined horizontal and vertical acceleration is found to be small, though it has considerable influence on convective response of tank having roof. Since the convective response is small in comparison to impulsive response, the vertical component of ground acceleration can be neglected in the analysis of water tanks.

### 2.3.7 Influence of Characteristics of Earthquake on Dynamic Behaviour

The seismic ground motions are characterised in terms of peak ground motion parameters or integral parameters. The peak ground motion parameters are peak ground acceleration (PGA), peak ground velocity (PGV) and peak ground displacement (PGD). Of these, peak ground acceleration is the parameter most often associated with severity of ground motion. However, examination of recorded seismic events has shown that earthquakes with a very large PGA may not produce appreciable structural damage. Anderson and Bertero (1987), Uang and Bertero (1988), and Bertero et al. (1991) have shown that earthquake ground motion attributes such as frequency content, duration of strong shaking, incremental velocity and incremental displacement can have profound effects on the structural response than the peak ground acceleration, particularly in the inelastic range. The parameters such as Arias intensity, spectrum intensity, root mean square acceleration (RMSA), root mean square velocity (RMSV) and root mean square displacement (RMSD), etc. are reportedly more effective for measuring the energy content of the seismic event. Being the better measures of earthquake destructiveness and these are often used in geotechnical earthquake engineering to assess the deformation and liquefaction potential of soil deposits (Kayen and Mitchell, 1997; Kramer and Mitchell, 2006; Sideras and Kramer, 2012). Power et al. (1986) strongly emphasized the importance of frequency content of ground motion in determining the structural response.

Duration was found to be significant factor influencing structural inelastic response, although secondary in comparison to the influence of frequency content. Bradley (2011) demonstrated that amplitude, frequency content, and duration of a ground motion are correlated. Therefore, different parameters are required to characterize the severity and the damage potential of the earthquake ground motion. The parameter of earthquake which governs the behaviour of the tank depends on the geometry of the tank, support condition and the properties of the soil on which it rests.

Seismic response of unanchored tank is mainly affected by the intensity of the seismic load lest would the frequency content, as the failure of unanchored tank is dominated by the uplift mechanism (Feng, 1997). The high frequency earthquakes result in the highest impulsive response in 'shallow' tanks on rigid foundation model, whereas the intermediate frequency earthquakes highly amplify the 'tall' tank response (Kianoush and Ghammaghami, 2011). Due to significant difference between impulsive and convective fundamental frequencies, a different trend is observed for convective response under same earthquakes.

Konstandakopoulou and Hatzigeorgiou (2017) focussed their study on the inelastic response of water and wastewater steel tanks under both real and artificial repeated earthquakes. Detailed examination of the response of each tank makes clear that the multiplicity of earthquakes strongly affects its seismic response, both for the structure and sloshing of fluid. For this reason, it is recommended to incorporate the provisions for repeated earthquakes in the design codes.

Though the seismic behaviour of tanks under different earthquakes are studied in detail, the influence of characteristic of earthquake on responses of tanks having different aspect ratio with varying water height has not undergone thorough investigation.

### 2.3.8 Influence of Soil-Structure Interaction on Dynamic Behaviour

The deformations of a structure during earthquake shaking are affected by interactions between three linked systems: the structure, the foundation, and the geologic media underlying and surrounding the foundation. A seismic Soil-Structure Interaction (SSI) analysis evaluates the collective response of these systems to a specified free-field ground motion. Discussed alongside are the influences of the soil-structure interaction on seismic behaviour of water tanks.

The effects of soil-structure interaction on the impulsive components of tank response under horizontal shaking may be substantial and should be considered in design as (Veletsos and Tang, 1990). Whereas the soil-structure interaction has negligible effect on convective components of response.

Profound effect on the amplification of hydrodynamic forces and moments exerted on tank structure by SSI were reported by Haroun and Izzeddine (1992). The magnification in seismic response is a factor of both the shear wave velocity of soil as well as the geometric properties of tank. For soft soil with lower shear wave velocity, the soil properties dominated the fundamental frequency of the system whereas for higher shear wave velocity, the flexible shell response is dominant. But observations of Veletsos et al. (1992) reveals the dependence of SSI on geometry of the tank may reduce significantly the critical responses of broad tanks, but may increase those of tall, stiff tanks that have high fundamental natural frequencies.

Later, Kianoush and Ghammaghami (2011) found that the maximum impulsive base shear and base moment may increase or decrease as the soil stiffness changes due to the dynamic pressure variation in the tank wall. This phenomenon is highly dependent on earthquake frequency content and tank configuration. For low frequency content earthquakes, the structural responses increase as the soil stiffness increases. Livaoglu (2008) suggested that for the flexible tanks SSI effect should be accounted for but for the rigid tank this effect may be negligible.

The soil-structure interaction has a negligible effect on the convective component of response and can be ignored in the seismic evaluation of water tanks. (Veletsos et al., 1992, Livaoglu, 2008, Uma et al., 2012).

The effect of foundation embedment on seismic response of fluid – elevated tank – foundation soil system were investigated by Livaoglu and Dogangun (2007) by conducting the time history analysis of reinforced concrete elevated water tank on six different soil types and upon two embedment conditions. It was observed that, as the soil gets softer, the foundation embedment becomes more effective and influences the system behaviour whereas for the very stiff soil considered, the embedment effects virtually do not exist. The overall the magnitude of base shear force increases as the soil gets softer.

Soil-Structure interaction causes rocking motion and fluid - structure interaction causes the hydrodynamic behaviour of water tank. Uma et al. (2012) found that impulsive base shear and base moment increase with increase in earthquake acceleration upon consideration of SSI effects on elevated water tanks.

The study on soil-structure interaction effects on seismic response of tanks is limited, that it is restricted to tanks filled up to design water depth. The influence of geometric features of the tank as well as water fill the conditions on combined FSI – SSI problem of water tank have not been addressed adequately in the past.

### 2.4 CODE SPECIFIED DESIGN GUIDELINES FOR DYNAMIC ANALYSIS

There are various national and international codes for seismic design of cylindrical liquid storage tanks. ACI 350.3, API 650, IS 1893 part 2, NZSEE and Eurocode 8 are most commonly referred ones. All codes and standards suggest mechanical analogues for modelling tank- liquid system, wherein liquid mass is divided into impulsive and convective masses. Two types of mechanical analogues are available: first one is for tank with rigid walls , that represent tank – liquid system as two mass model (Housner 1963, Veletsos and Yang 1977) and the second one for tank with flexible wall , that represent tank – liquid system as three mass model, in which the effect of wall flexibility is included (Haroun and Housner, 1981 and Veletsos 1984). The properties of the mechanical analogue are corresponding to the tank dimensions and fluid properties.

Variation of base shear coefficient with natural period can typically be divided into three time period ranges: acceleration-sensitive (or short-period) range, velocity-sensitive range, and displacement-sensitive (or long-period) range. In most of the codes, impulsive and convective mode base shear coefficients have a different type of variation with natural period and therefore these are discussed separately (Jaiswal et al., 2007).

ACI 350.3 and API 650 use mechanical model of Housner (1963) with modifications of Wozniak and Mitchell (1978). API 650 deals with circular steel tanks, which are flexible tanks. However, since there is no appreciable difference in the parameters of mechanical models of rigid and flexible tank models, these codes evaluate parameters of impulsive and convective modes from rigid tank models. ACI 350.3 also mentions parameters of mechanical model for rectangular tanks. NZSEE guidelines use mechanical model of Veletsos and Yang (1977) for rigid circular tanks and that of Haroun and Housner (1981) for flexible tanks. Eurocode 8 mentions mechanical model of Veletsos and Yang (1977) as an acceptable procedure for rigid circular tanks. For flexible circular tanks, models of Veletsos (1984) and Haroun and Housner (1981) are described along with the procedure of Malhotra et al. (2000). Except Eurocode 8, all the codes suggest SRSS (square root of sum of square) rule to combine impulsive and convective forces. Eurocode 8 suggests use of absolute summation rule.

### 2.4.1 American Concrete Institute (350.3-06, 2006)

ACI classify tanks into three importance categories and importance parameters are assigned accordingly. Instead of assuming a rigid tank directly accelerated by ground acceleration, it assumes amplification of response due to natural frequency of the tank. Rather than combining impulsive and convective modes by algebraic sum, ACI 350 combines these modes by square root- sum-of-the-squares (SRSS). Effects of maximum horizontal and vertical acceleration also shall be combined by the SRSS method. The seismic force levels and response modification factors included in ACI 350 provide results at allowable stress levels. Effect of vertical acceleration is also taken into account and it stipulates that in the absence of a site-specific response spectrum, the ratio of the vertical to horizontal acceleration shall not be less than 2/3. The code suggests 5% damping for impulsive mode and 0.5% for convective mode. Code suggest that for anchored tanks entire base shear is transmitted by membrane shear whereas for fixed or hinged base tanks, base shear is transmitted partially by membrane shear and rest by radial shear that cause vertical bending.

### 2.4.2 IS 1893 Part 2: 2014

The code gives the guidelines for the seismic analysis of both ground supported and elevated tanks. Housner's spring mass model along with modifications as incorporated in ACI 350.3 is used for determining the seismic design forces on ground supported as well as elevated water tanks of any capacity and material of construction namely, reinforced concrete or steel. Code recommends to incorporate the effect of soil flexibility in case of tanks resting in soft soil for the evaluation of impulsive time period, but soil-structure interaction analysis guidelines are not given. Code specifies 5% damping for the impulsive mode of concrete and masonry tanks and 2% damping for steel tanks whereas 0.5% damping for convective mode. As per the provisions, design has to be performed both at full fill and empty conditions. It uses SRSS rule for combining impulsive and convective modes.

### 2.4.3 American Petroleum Institute (API) 650: 2012

API employ mechanical model proposed by Housner (1963) and flexibility of the tank is ignored in the calculation of hydro dynamic pressure. The code suggests 5% damping for impulsive mode and 0.5% for convective mode. As the natural period of impulsive mode for ground supported steel tank is expected to remain in the acceleration sensitive range, the code specifies a constant value of base shear coefficient. This design procedure offers a concise way of determining the maximum applied axial stresses for anchored as well as unanchored tanks based on working stress method. Minimum anchorage requirements are also presented. Guidelines for consideration of vertical acceleration and soil-structure interaction effects in accordance with ASCE 7 are also included. While considering the SSI effects, the tanks shall be mechanically anchored to the foundation. The value of the base shear and overturning moments for the impulsive mode including the effects of soil-structure interaction shall not be less than 80% of the values determined without consideration of soil-structure interaction. Elastic buckling is addressed while elasto-plastic buckling (elephant foot buckle) is not accounted for.

### 2.4.4 EUROCODE 8, 2006

In Eurocode, tanks are assigned three protection levels depending on type of liquid stored and importance values are assigned base on risk to life and environment, economical and social consequence. Eurocode use absolute summation rule for combining the effects of impulsive and convective responses. Curvilinear distribution based on work of Housner for the distribution of impulsive and convective hydrodynamic pressure along wall height is described in the code. Impulsive base shear coefficient given by Eurocode8 is applicable to ground supported as well as elevated tanks as it has specifically prescribed variation in displacement sensitive range also. In this code, two types of spectra, viz elastic spectrum and design spectrum are mentioned and can be used for the determination of base shear coefficient. It specifies 5% damping for the impulsive mode of reinforced concrete and prestressed concrete tanks and 2% damping for steel tanks whereas 0.5% damping for convective mode. Provision for soil- structure interaction is included by presenting modified expression for natural periods of impulsive mode of vibration of rigid and flexible tanks. The convective response are assumed to be not affected by soil – structure interaction. Eurocode specify design forces at strength design level where factored loads corresponding to ultimate level are used. Though the design guidelines for the tanks having different height to radius of rank are given, no specified guidelines for the effect of water fill condition (ratio of height of liquid to height of tank wall) is mentioned.

### 2.4.5 NZS 3106: 2009

New Zealand code, NZS 3106:2009, classify the tanks based on support condition and material of construction. The response modification factor is suggested based on ductility factor and damping ratio. This code uses mechanical model of Veletsos and Yang (1977) for rigid circular tanks and that of Haroun and Housner (1981) model flexible steel tanks. The code specifies 0.5% damping for the convective mode in all types of tanks and for the impulsive mode of the ground supported tanks, it suggests damping values that depend on tank material, aspect ratio of the tank and properties of foundation soil. Curvilinear distribution as well as simplified linear distribution of impulsive and convective hydrodynamic pressure along wall height is described in the code. The basic seismic hazard coefficient, which depend on soil type, corresponds to the elastic design level is the governing factor for the determination of base shear coefficient. It employs graphs for normalised hoop force and bending moment distribution along the tank height developed due to hydrostatic, impulsive rigid and convective pressure components as a function of height to radius and radius to wall thickness ratio. It uses SRSS rule for combining impulsive and convective modes. Though soil-structure interaction is included, the presences of two horizontal components are not considered.

## 2.5 NEED FOR THE PRESENT STUDY

Seismic responses of tanks have been studied by several researchers. But most of the studies are on tanks in full fill condition and the effects of water fill condition on seismic response of tanks have not been undergone thorough investigation. Though the responses of broad and slender tanks to seismic loading have been considered, the variation of tank response with aspect ratio has not been addressed quantitatively. The tank response is highly dependent on the characteristics of earthquake such as frequency content and peak ground acceleration. The way in which tanks in different fill condition respond to earthquakes of different characteristics requires thorough examination due to the highly complicated fluid-structure interaction effects. The combined effect of characteristics of earthquake and stiffness of soil on seismic response of water tanks in different fill conditions are to be assessed for proper understanding of the behaviour of tanks to seismic loading.

**2.6 SUMMARY**

In this chapter previous works done in the proposed area are discussed. The failure of water tanks happened in past earthquakes necessities the study on failure mechanism of water tanks and various parameters that effect the response during seismic loading. Seismic response of the tank is highly depended on the flexibility of the tank wall. The tanks with different support conditions behave in different manner to seismic loading. The characteristics of earthquake also play a major role in determining the seismic response of the tanks. Various codes and guidelines for analysis and design of tanks are trying to incorporate the impact of the influencing parameters with in constraints such that catastrophic failure of tanks during earthquake can be avoided. But how these parameters vary with the geometry of the tank and water fill condition are to be undergone further investigation. With the advancement of computing facilities, the usage of finite element techniques for soil-structure interaction modelling will help to assess the response of tanks to real world earthquakes. The objectives and scope of the study given in Chapter 3 are defined based on the review of literature presented in this chapter.

# CHAPTER 3

# OBJECTIVES AND SCOPE OF THE STUDY

## 3.1 GENERAL

Critical review of literature and the need for an in-depth study considering water fill condition, rigidity parameters and soil properties on seismic analysis of water tanks has been brought out in the previous chapter. The objectives and scope of present study are defined in the following sections.

## 3.2 OBJECTIVES

The objectives of present study are:

(i) To study the effect of aspect ratio and water fill conditions on dynamic characteristics of ground supported concrete cylindrical water tank.

(ii) To investigate the influence of characteristics of earthquake on seismic response of ground supported tank with tank wall fixed at bottom for varying aspect ratio and water fill conditions.

(iii) To examine the significance of soil-structure interaction on seismic behavior of water tanks and to evaluate the effect of characteristics of earthquake and soil properties on seismic response.

## 3.3 SCOPE

The scope of the study is limited to the following with respect to tank dimensions and methodology adopted:

(i) Ground supported concrete cylindrical water tanks considered in the analysis are of height 12 m and free at top, with aspect ratio varying from 0.2 to 2.0.

(ii) Only impulsive response due to horizontal component of past three earthquake are considered in the seismic analysis.

(iii) SSI interaction effects on seismic behaviour of water tanks are studied by considering four soil types of different characteristics.

(iv) Studies are carried out through finite element modelling and analysis.

# CHAPTER 4

# MATHEMATICAL FORMULATION AND FINITE ELEMENT MODELLING

## 4.1 GENERAL

Critical review of literature presented in Chapter 2 indicates that several analytical approaches have previously been used to predict the dynamic response of liquid filled tanks of rigid and flexible walls. These techniques developed for tanks having regular geometry are based on several assumptions. Fluid-structure interaction problem is a highly complicated one and experimental investigation on seismic behaviour of tanks is highly expensive. The inclusion of soil – structure interaction along with FSI makes the problem more complex. The parametric study of such a problem by experimental methods is not viable. The numerical methods such as finite difference, boundary element and finite element are now in common use. The finite element method has been successfully applied to dynamic analysis of tanks and has been used in the present investigation. This chapter is dedicated to mathematical and finite element formulations of ground supported water tanks. Fluid-structure interaction is properly addressed in the finite element modelling of the water tanks. Since the influence of soil properties on dynamic response is also investigated as part of the study, soil-structure interaction modelling is also counted in this chapter.

Finite element software ANSYS can us be used with accuracy for modelling and analysing fluid – structure interaction problems. (Kruntcheva, 2007; Sezen et al., 2008; Schubert et al., 2008; Jaiswal et al., 2008; Moslemi and Kianoush, 2012; Goudarzi and Sabbagh Yazdi, 2012; Nicolici and Bilegan, 2013; Elkholy et al., 2015; Ruiz et al., 2015). It also has the capabilities to include the soil modelling and to enable the proper interaction between soil and structure (Livaoglu and Dogangun, 2007; Livaoglu, 2008; Kianoush and Ghaemmaghami, 2011; Uma et al., 2013). Hence in this study, finite element analysis software ANSYS is used for modelling and analysing the water tanks. Using these models, free vibration analyses for the determination of natural frequency and time history analyses for response of the tanks to prescribed seismic loading were performed. While performing these analyses, it is ensured that proper load transfer takes place between the fluid and structure as well as between soil and structure.

## 4.2 MODELLING OF GROUND SUPPORTED TANKS

The initial studies on the dynamic behaviour of liquid-tank systems has focused on anchored liquid-tank systems, in which the tank structure base plate is anchored to the foundation giving simpler seismic response compared to that in unanchored liquid-tank systems. Many analytical models have been proposed (Jacobson 1949, Housner1957, Veletsos and Yang 1976, 1977, Haroun and Housner 1981) and reliable predictions of the dynamic behaviour of anchored liquid- tank systems can be obtained from these models. These analytical models can be categorized into rigid tank models and a flexible tank model according to the different assumptions about the dynamic response of the tank shell wall made in the analysis (Feng, 1992) and is given in Chapter 2.

## 4.3 FINITE ELEMENT FORMULATION OF TANK WALL

The ground supported tank is assumed have idealised fixed boundary condition at bottom such that no sliding or uplift may occur. Therefore, all base nodes located along the floor perimeter are fully restrained in all directions. As a result of perfect anchorage assumption, no bending moment can be transferred from the wall to the floor and vice versa and therefore the tank floor may not be included in FE modelling of such containers (Moslemi and Kianoush, 2012). Hence for the tanks considered in this study, the tank floor is not modeled in FE simulation. The walls of the tank are considered as thin plates made of linearly elastic, homogenous and isotropic material and are assumed to perform transverse bending deflection but no inplane deformation (Hashemi, 2013).

Four noded quadrilateral element, SHELL 181, is used for modelling the tank wall. SHELL181 is suitable for analysing thin to moderately-thick shell structures. The element has six degrees of freedom at each node (translations in the x, y, and z directions, and rotations about the x, y, and z-axes) and both bending and membrane behaviours are permitted. SHELL 181 is well-suited for linear, large rotation, and/or large strain nonlinear applications and can be associated with linear elastic, elastoplastic, creep, or hyperelastic material properties. In the element domain, both full and reduced integration schemes are supported. SHELL 181 includes the linear effects of transverse shear deformation. Finite element model of the ground supported cylindrical water tank having diameter 15m and height 12m, generated in ANSYS software is given in Fig.4.1.

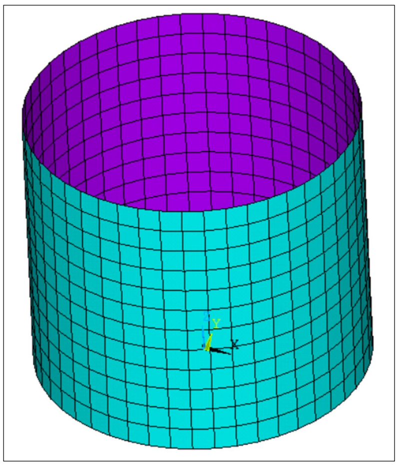


Fig. 4.1 Finite element model of ground supported cylindrical tank

## 4.4 FINITE ELEMENT FORMULATION OF WATER

### 4.4.1 Governing Equations of Liquid Domain

The computation of hydrodynamic pressures in this study is achieved by using the theory of velocity potential. In this theory, it is assumed that stored fluid can be regarded as potential flow. The fluid motion is represented in terms of continuity conditions of contained fluid and boundary conditions of the contact interface between fluid and tank body as well as free surface of the fluid. The fluid is assumed to be incompressible, irrotational and inviscid and also there is no mean flow of the fluid (Ghaemmaghami and Kianoush, 2010; Moslemi and Kianoush, 2012; Avval et al., 2012, Hashemi et al., 2013).

The tanks is assumed to be attached to a rigid base and the contained fluid is inviscid and incompressible resulting in an irrotational flow field. For irrotational flow there exist a velocity potential 'Φ' such that spatial derivation of the velocity potential gives the fluid velocity *u*, *v* and *w* in the x, y and z directions.

$$u = \frac{\partial_\phi}{\partial_x}, \; v = \frac{\partial_\phi}{\partial_y}, \quad w = \frac{\partial_\phi}{\partial_z} \tag{4.1}$$

The velocity potential should satisfy the 3-D Laplace equation at any point of the liquid domain, respecting the assumption of incompressible fluid

$$\nabla^2\phi\ (x,y,z,t) = 0 \tag{4.2}$$

For the inviscid fluid with either steady or unsteady flow, the equations of momentum conservation may be integrated to yield a single scalar equation referred to as Bernoulli's equation:

$$\frac{\partial\phi}{\partial t} + \frac{P}{\rho} + gz + \frac{1}{2}(u^2 + v^2 + w^2) = f(t) \tag{4.3}$$

Where ‘P’ is the pressure, ‘g’ is acceleration due to gravity corresponding to negative Z direction, ‘f(t)’ is constant of integration. Owing to the small volume of containers, the velocity of pressure wave assumed to be infinity, and ‘u’, ‘v’ and ‘w’ components of velocity are assumed to be small, squared value of these quantities are also small compared to first order values and can be neglected. As a result, if the constant value of ‘f(t)’ can be observed into ‘Φ’, the Bernoulli's equation will be linearized to the following form:

$$\frac{\partial \phi}{\partial t} + \frac{P}{\rho} + gz = 0 \tag{4.4}$$

From the equation (4.4), the hydrodynamic pressure acting on tank wall in excess of hydrostatic pressure can be obtained as

$$P\,(x, y, z, t) = -\rho\,\frac{\partial \phi(x,y,z,t)}{\partial t} \tag{4.5}$$

The study of the convective response of the contained liquid is based on the linear theory of sloshing. Sloshing means any motion of the free liquid surface inside its container. It is caused by any disturbance to partially filled liquid containers. Depending on the type of disturbance and container shape, the free liquid surface can experience distinct types of motion including simple planar, nonplanar, rotational, irregular beating, symmetric, asymmetric, quasi-periodic and chaotic. When interacting with its elastic container, or its support structure, the free liquid surface can exhibit fascinating types of motion in the form of energy exchange between interacting modes. Nonlinear effects including amplitude jump, parametric resonance, chaotic liquid surface motion, and nonlinear sloshing mode interaction due to the occurrence of internal resonance among the liquid sloshing modes are not taken into account in linear theory of sloshing (Ibrahim, 2005). The natural frequencies of the free liquid surface appear in the combined boundary condition, kinematic and dynamic, rather than in the fluid continuity equation. For an open surface, which does not completely enclose the field, the boundary conditions usually specify the value of the field at every point on the boundary surface or the normal gradient to the container surface, or both.

As per linear theory of sloshing, fluid particle at the plane surface always remains on the plane surface itself. At the free surface, pressure is equivalent to ambient pressure and is given by

$$P = \rho g \eta \tag{4.6}$$

where $\eta(x,y,t)$ represents the small displacement of the free liquid surface.

This gives the dynamic boundary condition. If the effect of surface waves of the water is neglected (ie if sloshing is not considered), the boundary condition of the free surface may be expressed as P= 0.

Considering the dynamic boundary condition, the unsteady Bernoulli equation can be written as

$$\frac{\partial \phi(x,y,t)}{\partial t} + g\eta(x,y,t) = 0 \tag{4.7}$$

In addition to dynamic boundary condition, the kinematic boundary condition also should be satisfied. Accordingly, liquid at the free surface always remains at the free surface. The kinematic boundary condition at the free surface relates the surface displacement to the vertical component of the velocity at the surface, ie the vertical velocity of a fluid particle located on the free surface should be equated to the vertical velocity of the free surface itself. This condition is known as the kinematic free-surface condition and is given by the following expression

$$\frac{\partial \eta}{\partial t} = \frac{\partial \phi}{\partial z} = w \tag{4.8}$$

At the wetted rigid wall and bottom, the velocity component normal to the boundary must have the same value of the corresponding velocity component of the solid boundary at the point in question, '$v_n(t)$' where 'n' stands for the normal direction. This assumption is only applicable for the cases that the tank motion is not rotational and if the viscous stresses are negligible. The appropriate boundary condition at the interface of liquid and tank is governed by

$$\frac{\partial \phi}{\partial n} = v_n(x, y, z, t) \tag{4.9}$$

For the tanks with rigid walls, $v_n(t)$ equals to grounds velocity. For the case of flexible walls, walls velocity is the summation of the ground velocity and its relative velocity due to wall flexibility.

Taking the time derivative of equation (4.7) and applying (4.8), gives the linearised boundary condition at the free surface may be written as

$$\frac{1}{g}\frac{\partial^2 \phi}{\partial t^2} + \frac{\partial \phi}{\partial z} = 0 \tag{4.10}$$

(Ibrahim, 2005; Kianoush and Ghaemmaghami, 2011; Shekari et al., 2010; Avval et al., 2012; Mandal and Maity, 2016)

For obtaining the impulsive component of the liquid, the boundary condition at free surface of liquid is to be replaced by zero pressure, P = 0 at z = h where 'z' is the vertical distance measured from bottom of the tank. The convective response of the liquid is obtained by subtracting the impulsive response from the total response.

Using these boundary conditions at the free surface of the liquid, the impulsive and convective components can be determined separately in the coupled acoustic-structure FE model (Rawat, 2015).

## 4.5 FLUID-STRUCTURE INTERACTION (FSI) MODELLING

The interaction of the contained fluid and the structure at an interface causes the hydrodynamic pressure to apply a force on the structure and the structural motions produce an effective fluid load (Mandal and Maity, 2015). Fluid-structure interaction problems can be investigated by using different techniques such as added mass, Lagrangian, Eulerian, and Lagrangian–Eulerian approaches in the finite element method (FEM) or by the analytical methods like Housner's two-mass representation or multi-mass representations of Bauer (Livaoglu and Dogangun, 2007). There are two types of FSI simulation: one way, when information from flow simulation is transferred into structure and two way simulation, where data are exchanged between both: fluid and structure.

The boundary conditions of the Laplace's equation are defined by the dynamic response of the tank structure, which is the combination of the vibration in response to the ground excitation and the deformation in response to the hydrodynamic load. Thus the motion of the liquid content and the dynamic response of the tank structure are coupled together (Feng,1992). For most structural dynamics problems of mechanical system, the spatial discretization for the principle of virtual work using finite element method gives equation of motion as

$$[M]\{\ddot{u}_t\} + [C]\{\dot{u}_t\} + [K]\{u_t\} = \{F_t^a\} \tag{4.11}$$

where, $[M]$ structural mass matrix, $[C]$ structural damping matrix, $[K]$ structural stiffness matrix, $\{\ddot{u}_t\}$ nodal acceleration vector, $\{\dot{u}_t\}$ nodal velocity vector, $\{u\}$ nodal displacement vector and $\{F_t^a\}$ applied load vector. To completely describe the FSI problem, the fluid pressure acting at the interface is added to Eq. (4.11) and the structural equation is rewritten as given in Eq. (4.12)

$$[M_F]\{\ddot{p}\} + [C_F]\{\dot{p}\} + [K_F]\{p\} + \rho[R]^T\{\ddot{u}\} = [f_F] \qquad (4.12)$$

The dynamic equilibrium equation of the structure can be expressed as Eq. (4.13)

$$[M_s]\{\ddot{u}\} + [C_s]\{\dot{u}\} + [K_s]\{u\} - [R]\{p\} = \{f_s\} \qquad (4.13)$$

Thus the complete finite element discretized equation for the FSI problem obtained by coupling the acoustic and the structural matrices and are given in Eq. (4.14) where $[M_S]$, $[C_S]$ and $[K_S]$ are the mass, damping and stiffness matrices of the tank respectively and $[M_F]$, $[C_F]$ and $[K_F]$ are the mass, damping and stiffness matrices of the acoustic fluid. The fluid density is denoted by '$\rho$' and $[R]$ gives the coupling matrix which represents the coupling conditions on the interface between acoustic fluid and structure. $\{f_s\}$ and $\{f_F\}$ are structural and fluid load quantities produced at fluid structure interface in terms of unknown nodal displacement $\{u\}$ and pressure $\{p\}$.

$$\begin{bmatrix} [M_S] & [0] \\ \rho[R]^T & [M_F] \end{bmatrix} \begin{Bmatrix} \{\ddot{u}\} \\ \{\ddot{p}\} \end{Bmatrix} + \begin{bmatrix} [C_s] & [0] \\ [0] & [C_F] \end{bmatrix} \begin{Bmatrix} \{\dot{u}\} \\ \{\dot{p}\} \end{Bmatrix} + \begin{bmatrix} [K_S] & -[R] \\ [0] & [K_F] \end{bmatrix} \begin{Bmatrix} \{u\} \\ \{p\} \end{Bmatrix} = \begin{Bmatrix} f_S \\ f_F \end{Bmatrix} \qquad (4.14)$$

### 4.5.1 FE Formulation of Water and Interface in ANSYS

Eight noded acoustic element, FLUID 30, is used for modelling the fluid medium and the interface in fluid-structure interaction problems. The element node has four degrees of freedom per node: translations in the nodal x, y and z directions, and pressure. The translations are applicable only at nodes on the interface. Typical application of the element is the sound wave propagation problem. Elements have the capability to include damping of sound-absorbing material at the interface as well as damping within the fluid. The elements can be used with or without other 3-D structural elements to perform symmetric, unsymmetric or damped modal, full harmonic, and full transient method analyses.

Acceleration effects like those in sloshing problems can be included. If free surface effects are present vertical acceleration is necessary to specify, even for a modal analysis. The speed of sound in the fluid is input as sonic velocity, given by $\sqrt{\frac{k}{\rho}}$ where 'k' is the bulk modulus of the fluid and 'ρ' is the mean fluid density. 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.

Fluid nodes should be coupled at all interfaces with containing structure; as a result all fluid nodes located at the interface with the tank floor should be coupled with the base nodes. For interfaces where structure is not present such as fluid free surface, no FSI flags need to be assigned. In building the FE model, the layer of fluid elements in touch with structural elements are modeled assuming x, y, and z translational degrees of freedom and with the FSI turned on. To specify the FSI flag, first all nodes located on the interface should be selected. Then, the fluid elements attached to this set of nodes are selected. The selected nodes are then specified as fluid–structure interface nodes. This will set the appropriate boundary conditions on different interfaces surrounding the fluid domain (ANSYS).

Fluid-structure interfaces (FSIs) can be flagged by surface loads at the element faces. Specifying the FSI label, couples the structural motion and fluid pressure at the interface. One-way coupling from structure to acoustics is more computationally efficient, while the acoustic effect on the structure can be neglected, the structural solution is performed first. Finite element model of tank having diameter 15m and height 12m filled with water for a height of 11m is given in Fig. 4.2.

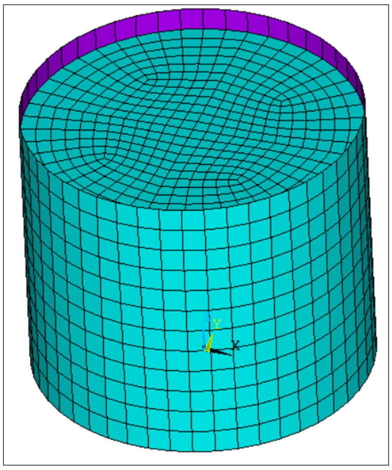
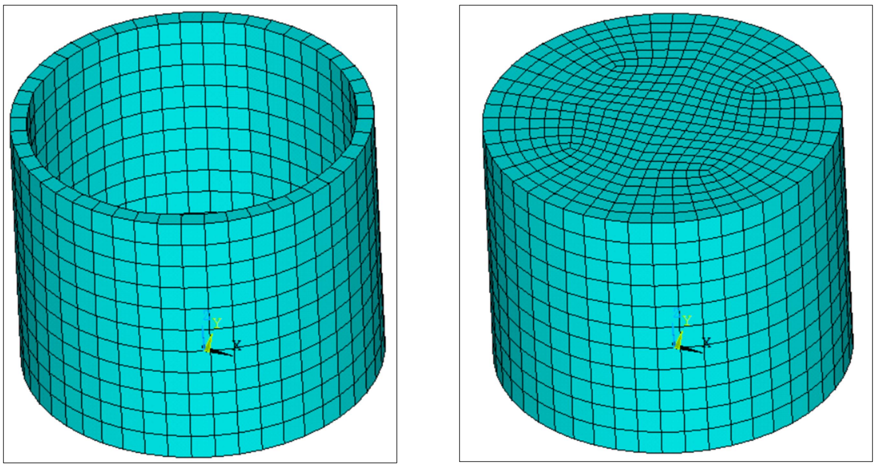


Fig. 4.2 Finite element model of tank filled with water

The finite elements of water having four degrees of freedom constituting the outer volume of water body is given in Fig. 4.3 (a) and the entire water body is shown in Fig. 4.3 (b).



(a) (b)

Fig. 4.3 (a) Outer elements of water body and (b) acoustic elements constituting water body inside the tank having diameter 15m

Fluid structure interface modelled in ANSYS is given in Fig. 4.4

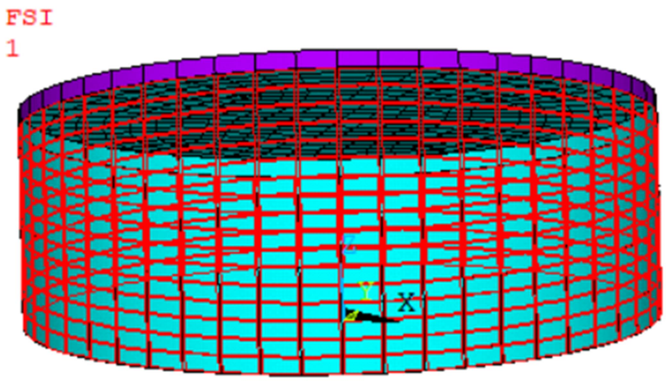



Fig. 4.4 Numerical modelling of fluid-structure interaction (D – 34m, h – 11m)

### 4.6 SOIL-STRUCTURE INTERACTION MODELLING

The deformation of a structure during earthquake is affected by interactions between three linked systems: the structure, the foundation, and the geologic media underlying and surrounding the foundation. A seismic Soil-Structure Interaction (SSI) analysis evaluates the collective response of these systems to a specified free-field ground motion (Domagala and Lisowski, 2011). The degree of influence of SSI on seismic response of structure depends on (i) stiffness of soil, (ii) dynamic characteristics of structure and (iii) characteristics of earthquake. SSI effects can be classified into two: kinematic and inertial interactions. SSI effect which is associated with the stiffness of the structure with the omission of mass of structure is termed as kinematic interaction. Inertial interaction effects are due to the mass of the foundation-superstructure system, which is caused by inertial forces generated in the structure due to movement of masses of the structure during vibration. These inertial forces imparted onto the surrounding soil causes the foundation to experience a response different from the foundation input motion. (Wolf, 1985; Wolf and Song, 1996; Uma et.al, 2013).

SSI problems are classified into two main categories: direct method and substructure method. In direct method, response of the entire structure foundation–soil system is analysed in a single step. Whereas in substructure method, analysis of parts of whole structural system is performed in several steps and the final response is obtained by applying the principle of superposition (Jayalekshmi and Chinmayi, 2006).

To investigate the effects of soil-foundation-structure interaction, the effect of soil can be included implicitly or explicitly. In implicit methods, the effects of the soil are added to the analysis using springs and dampers without modelling the soil itself. Different implicit analysis techniques use different assumptions and are suitable for specific problems. In an explicit analysis method, however, the soil itself is modelled with finite elements. The soil body should be large enough to be accurate and, therefore, it is more time-consuming compared to the implicit method (Austin, 2017).

The simulation of the infinite medium in the numerical method is very important in the dynamic soil- structure interaction problems. When modelling a dynamic problem involving soil–structure interaction, particular attention must be given to the soil boundary conditions. One of the major challenges in this area is to correctly model radiation damping in the unbounded soil domain (Aslmand et al., 2018). Ideally, infinite boundary condition should be surrounding the excited zone. Propagation of energy will occur from the interior to the exterior region. Since the exterior region is non reflecting, it absorbs all of the incoming energy. In finite element analysis, it is constrained into apply finite size boundaries of soil. Those boundaries in turn will reflect the elastic waves which is contrary to the physics of the problem. Since outgoing deformation waves are reflected once they arrive at the fictitious boundaries of the meshed region, the conventional finite element method is not directly applicable. To overcome this problem, various open boundaries have been developed, which can be used together with the finite element method (Bescos, 1987; Givoli, 1991; Tsynkov, 1998; Astley, 2000). Local absorbing boundaries are desirable for ease of implementation and simplicity, but must be placed sufficiently far away from the region of interest to maintain accuracy (Aslmand, 2018).

In a dynamic problem, it may be insufficient to prescribe a zero displacement at a large distance from the structure, as it is routinely done in static problems. However, a sufficiently large soil model is considered to be adequate to include the effect of soil-structure interaction (Wolf, 1985; Livaoglu and Dogangun, 2007; Domagala and Lisowski, 2011; Kianoush 2011; Clough and Penzin, 2015). More appropriate approximations include utilization of artificial and/or transmitting boundaries and hence reflecting and radiation effects of the propagating waves from the structure-foundation layer may be avoided by means of these types of boundaries (Livaoglu and Dogangun, 2007). Providing a suitable damping model for soil, the spurious reflections from the model can be suppressed if the model is wide enough (Shih et al., 2016).

Direct method of SSI analysis has been followed in the study. In the direct method, the modelling and analysis of the entire structure–foundation–soil system is carried out in a single step. The structure and the finite bounded soil zone adjacent to the structure (near field) are modelled and the effect of the surrounding unbounded soil (far field) is imposing by the viscous boundaries to prevent the reflection of waves at the boundaries (Jayalekshmi and Chinmayi, 2006). Structure–foundation–soil system modelled using direct method consists of tank filled with water, base slab, interface between water and tank, proper coupling between base slab and soil, soil of appropriate dimensions to represent near filed and viscous dampers to absorb the wave energy reflected by the system. The viscous boundaries can be used with finite element mesh. At the bottom of soil mass, all degrees of freedom are arrested to prevent the rigid body motion as adopted in studies of Uno et al. (2008) and Shih et al. (2016).

To account for the soil–structure interaction, it is necessary to apply the inertial loads only to the structure and not to the soil foundation. Most computer commercial programs automatically apply the seismic loading to all mass degrees of freedom within the model and cannot solve the SSFI problem. This lack of capability has led to the development of the massless foundation model in which the inertia forces within the foundation material are neglected (Kianoush and Ghaemmagham, 2011). Since, most of the studies done on the behaviour of liquid storage tanks including soil–structure interaction are based on this approximation, the same technique is adopted in the present study.

A viscous damping model, based on Rayleigh damping, is used for the calculations with the FE model. The Rayleigh damping is based on two parameters ‘α’ and ‘β’, which allow the damping matrix ‘C’ to be determined from the mass and stiffness matrices, [M] and [K] using the expression given in Eq. 4.15.

$$C = \alpha\,[M] + \beta\,[K] \tag{4.15}$$

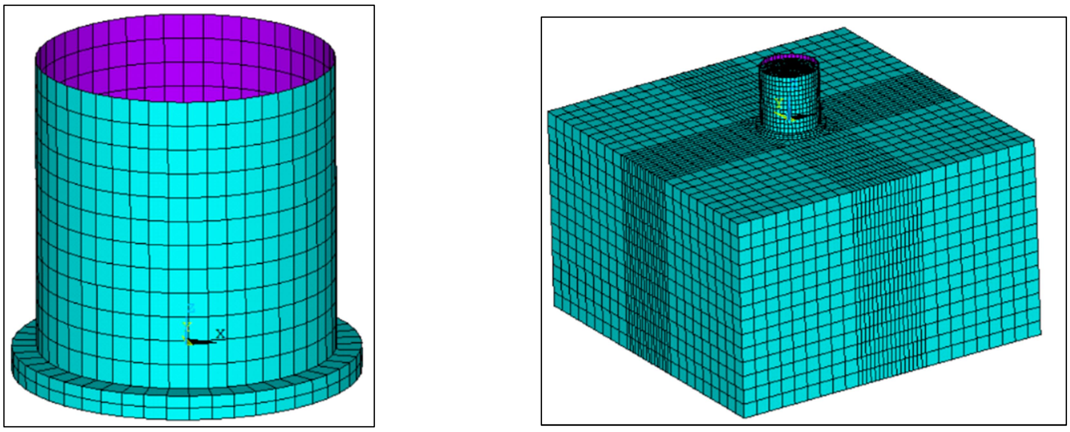
The damping ratio $'\xi'$ is evaluated from Eq.4.16

$$\xi = \frac{\alpha}{2\omega} + \frac{\beta\omega}{2} \tag{4.16}$$

where ω is the frequency at which $\xi$ is applied.

A classical Kelvin-Voigt viscous damping model, in which the damping matrix is proportional to the stiffness matrix, suffers from spurious reflections at the domain boundaries at low frequencies, whereas, a mass-proportional damping model gives a decay with distance that is independent of frequency. Mass-proportional Rayleigh damping can be obtained by setting β equal to zero (Shih et al., 2016) and is adopted in this study.

Here, in the soil modelling, width of soil block considered is five times the diameter of the base slab and depth is three times diameter of base slab such that sufficiently large soil mass is available. SOLID185, defined by eight nodes having three degrees of freedom at each node: translations in the nodal x, y, and z directions is used for 3-D modelling of the base slab and the soil beneath. The element has plasticity, hyperelasticity, stress stiffening, creep, large deflection and large strain capabilities. It also has mixed formulation capability for simulating deformations of nearly incompressible elastoplastic materials, and fully incompressible hyperelastic materials. Finite element models of tank (diameter -12m, height- 12m) having base slab (diameter – 14m, thickness -1m) and tank resting on soil are given in Fig. 4.5 (a) and Fig. 4.5 (b) respectively.



(a) (b)

Fig. 4.5 Finite element model of (a) tank with base slab and (b) tank resting on soil

By providing the viscous boundaries, initially proposed by Lysmer and Kuhlemeyer (1969), the energy of the wave transmitted out of the soil is absorbed by boundary dashpots. Thus reflecting and radiation effects of the propagating waves from the structure -foundation layer may be avoided by means of these types of boundaries. The viscous boundaries are modelled using COMBIN 14 element available in ANSYS. COMBIN 14 has longitudinal or torsional capability in 1-D, 2-D, or 3-D applications. The longitudinal spring-damper option is a uniaxial tension-compression element with up to three degrees of freedom at each node: translations in the nodal x, y, and z directions; no bending or torsion is considered. The torsional spring-damper option is a purely rotational element with three degrees of freedom at each node: rotations about the nodal x, y, and z axes; no bending or axial loads are considered. The spring-damper element has no mass (ANSYS).

The equivalent stiffness and damping coefficients in normal direction and tangential directions are (Lin et al., 2006) given by Eq. 4.17 and 4.18 respectively

$$K_n = \frac{4G}{R} A \quad and \quad C_n = \rho V_p A \tag{4.17}$$

$$K_t = \frac{2G}{R} A \quad and \quad C_t = \rho V_s A \tag{4.18}$$

Where $K_n$ and $K_t$ – Stiffness coefficients in normal and tangential directions, $C_n$ and $C_t$ – damping coefficients in normal and tangential directions, G - the shear modulus of the soil, ρ – density of soil , $V_p$ = P wave velocity, $V_s$ – shear wave velocity, A – effective nodal area of the node that is connected to the damper, and R – radial coordinate.

Three dimensional finite element model of water tank – base slab – soil system having viscous boundary is given in Fig. 4.6.

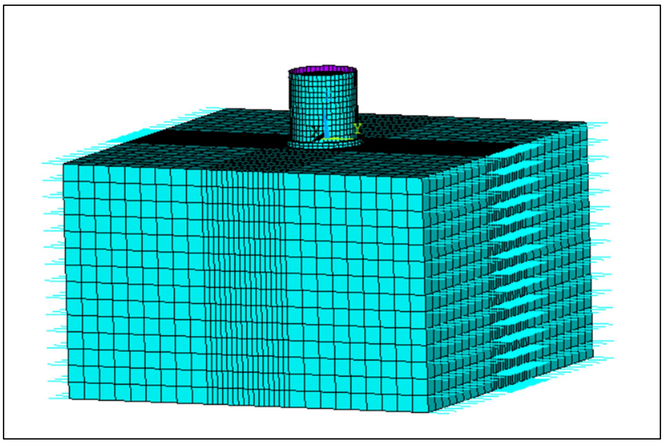


Fig. 4.6 Finite element model of tank resting on soil with viscous boundaries

## 4.7 SUMMARY

Parametric study was performed to assess the role of aspect ratio, water fill condition, frequency content of earthquake and properties of soil on seismic response of water tanks. Finite element analysis performed on different tank models generated in ANSYS help to identify the influence of theses parameters on the dynamic response of water tanks. Free vibration analysis is done for the determination of natural frequency of vibration and to characterise the dynamic behaviour of tanks and the results are presented in Chapter 5. Results of seismic analysis of ground supported tanks as well as those resting on different soil presented in Chapter 6 and Chapter 7 are based on finite element analysis of water tank modelled as per techniques described in this chapter.

# CHAPTER 5

# FREE VIBRATION ANALYSIS OF GROUND SUPPORTED CYLINDRICAL WATER TANK

## 5.1 GENERAL

Prior to perform the seismic analysis, it is necessary to do the free vibration analysis of the tanks to define the dynamic characteristics of the tank. By performing the free vibration analysis of ground supported tanks, natural frequency in both impulsive and convective modes of vibration and corresponding mode shapes can be obtained. The fundamental modes can be identified as those with the largest participation factors in the translational directions. The significance of convective mode of vibration is estimated from the results of free vibration analysis of the rigid and flexible tanks. The dependence of impulsive frequency on tank wall flexibility, aspect ratio (height to diameter ratio) and water fill condition can also be assessed from the free vibration analysis. Comparisons of results with guidelines in standard codes are reviewed to establish the influence of various parameters on both impulsive and convective frequencies of the tank.

## 5.2 SUITABILITY OF FINITE ELEMENT MODEL FOR DYNAMIC ANALYSIS

In order to ascertain that the proposed finite element model can be used for the dynamic analysis of tanks, free vibration analysis of present study is compared with results published in literature: comparison is separately done for both flexible and rigid tanks. For this purpose, cylindrical ground supported tank of 35m outer diameter and 12m height with tank wall thickness 0.5m was considered in line with Moslemi and Kianoush (2012). The design height of water is 11m. The tanks are modelled as described in section 4.2. Tank walls are assumed to be in the idealized case of fixed at bottom. The properties of tank and water considered are: density of concrete 2400 kg/m$^3$, Modulus of elasticity of concrete – 24.86GPa, Poisson's ratio of concrete – 0.16, density of water – 1000 kg/m$^3$ and bulk modulus of water – 2.0 GPa.

The impulsive and convective frequencies of vibrations presented in Table 5.1 indicate that the present finite element analysis are in good agreement with the values calculated using standard codes ACI 350.3-06 and IS 1893: Part2 (2014) and procedure proposed by Malhotra et al. (2003).

The fundamental frequency proposed by ACI 350.3-06 for impulsive and convective mode of vibration of ground supported circular tank is given in equation 5.1 and 5.2 respectively.

$$\omega_i = C_l \frac{1}{H_L} \sqrt{10^3 E_C \frac{g}{\gamma_c}} \tag{5.1}$$

$$\omega_c = \frac{\lambda}{\sqrt{D}} \tag{5.2}$$

where $C_l = C_w \sqrt{\frac{t_w}{10r}}$ and $\lambda = \sqrt{3.68 g \tanh[3.68 \frac{H_L}{D}]}$

$\omega_i$ and $\omega_c$ are impulsive and sloshing frequency of vibration in rad/s., $C_l$, $C_w$- coefficients for determining the fundamental frequency of the tank-liquid system, $H_L$- design depth of stored liquid, D - inside diameter of circular tank, $\gamma_c$ - density of concrete, $E_c$ - modulus of elasticity of concrete $t_w$ – thickness of tank wall, r- radius of the tank.

$C_w$ is given in Eq.5.3 and is also depicted in Fig. 5.1.

for $D/H_L > 0.667$

$$C_w = 9.375x10^{-2} + 0.2039\left(\frac{H_L}{D}\right) - 0.1034\left(\frac{H_L}{D}\right)^2 - 0.1253\left(\frac{H_L}{D}\right)^3 + 0.1267\left(\frac{H_L}{D}\right)^4 - 3.186x10^{-2}\left(\frac{H_L}{D}\right)^5 \quad (5.3)$$

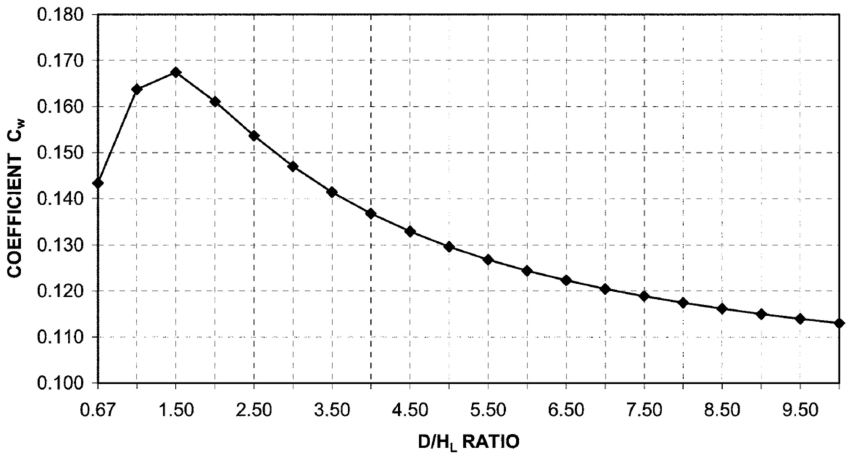



Fig. 5.1 Coefficient $C_w$ for circular tanks (ACI 350.3 – 06, 2006)

Equations 5.4 and 5.5 give the time period of impulsive ($T_i$) and convective ($T_c$) modes of vibration given by IS 1893: Part 2 (2014)

$$T_i = C_i \frac{h\sqrt{\rho}}{\sqrt{\frac{t}{D}}\sqrt{E}} \quad (5.4)$$

$$T_c = C_c\sqrt{\frac{D}{g}} \quad (5.5)$$

where 'h' is maximum depth of liquid, 'D' is inner diameter of tank, 't' is thickness of tank wall, E = modulus of elasticity of tank wall and 'ρ' is mass density of liquid. $C_i$ and $C_c$ are coefficient of time period for impulsive mode and convective mode respectively (Fig. 5.2).

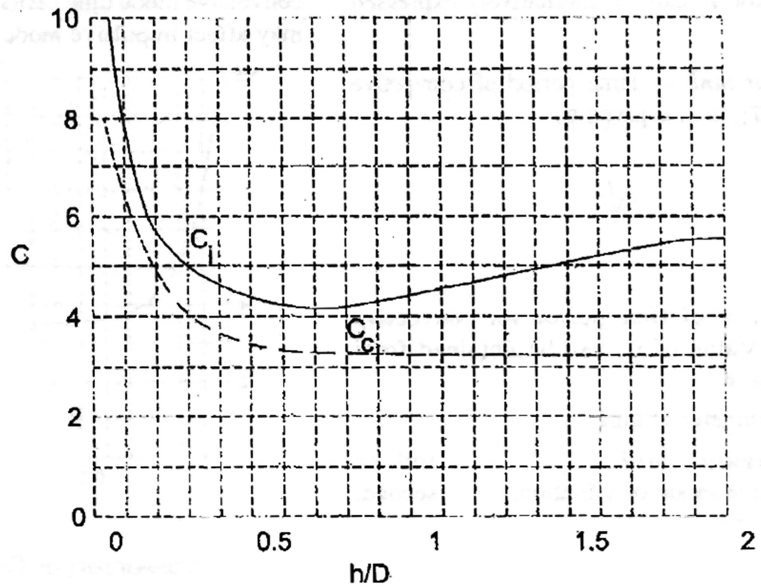



Fig. 5.2 Coefficient of impulsive and convective time period (IS 1893: Part 2, 2014)

Table 5.1 Natural frequencies of flexible cylindrical tank

| Mode | Type | Frequency (Hz) | | | |
|---|---|---|---|---|---|
| | | Present Study | Malhotra et al. (2000) | ACI 350.3 | IS 1893: Part 2 |
| 1 | Impulsive | 12.73 | 10.76 | 11.66 | 11.60 |
| 2 | | 22.09 | | | |
| 1 | Convective | 0.150 | 0.147 | 0.150 | 0.141 |

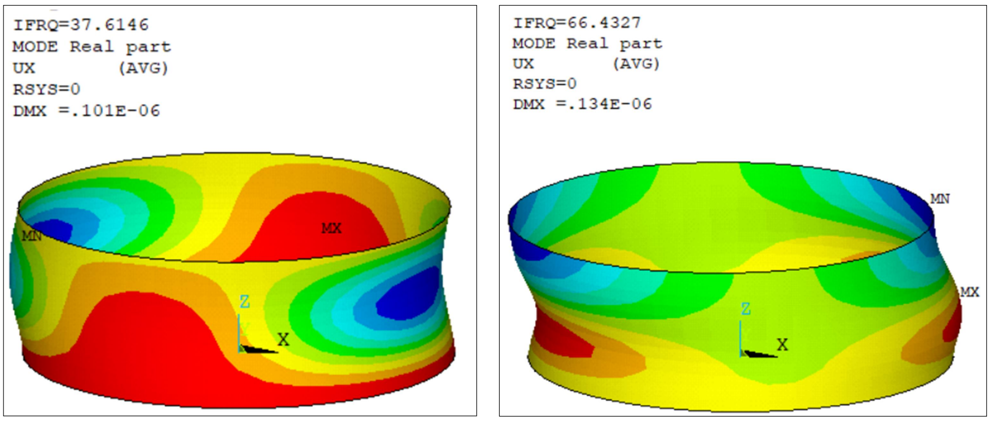
By performing parametric study on tanks of different material property, Moslemi and Kianoush (2012) observed that for tank with modulus of elasticity is equal to 10 times modulus of elasticity of concrete (10Ec), concrete tanks of similar dimension behave as rigid tanks. In order to check whether the same model can be used for tanks with rigid wall, modal analysis was further carried out with rigid wall consideration by taking elastic modulus of the tank wall material as $10E_{c.}$ The results are also compared with the analytical approach introduced by Housner (1963) and produced by Moslemi and Kianoush (2012) for tanks with rigid wall and is given in

Table 5.2. Housner's method is applicable only to tanks with rigid wall boundary condition, in which the tank shell is assumed to be massless and only the mass of water is considered (Housner, 1963; Moslemi et al., 2011).

Table 5.2 Natural frequencies of rigid cylindrical tank

| Mode | Type | Frequency (Hz) | | | | |
|---|---|---|---|---|---|---|
| | | Moslemi and Kianoush (2012) | | Present study | ACI 350.3 | IS 1893: Part 2 |
| | | Analytical Approach | FEA | | | |
| 1 | Impulsive | 36.0 | 30.53 | 37.61 | 36.87 | 36.59 |
| 2 | | | 61.78 | 66.43 | | |
| 1 | Convective | 0.149 | 0.149 | 0.150 | 0.150 | 0.141 |

Mode shapes corresponding to first two impulsive mods of vibration of rigid tank , considered for validation study, is given in Fig. 5.3




(a) mode 1 (b) mode 2

Fig 5.3 Impulsive modes of vibration of rigid tank (D – 34m, h -11m)

Since the obtained results of finite element analysis are in good agreement with that reported in literature, it can be concluded that the proposed FE model can be employed with high accuracy to study the fluid–structure interaction problems of liquid containing tanks with flexible as well as rigid wall boundary conditions.

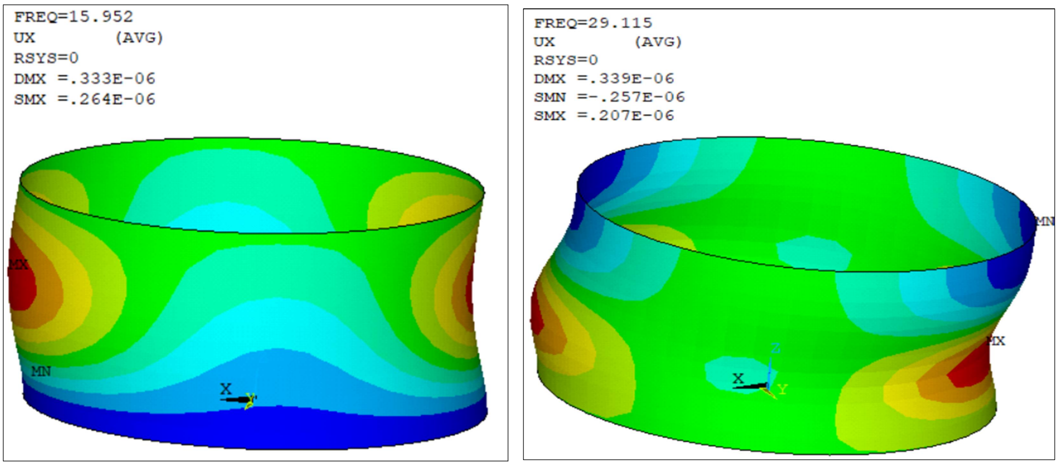
## 5.3 EFFECT OF WALL FLEXIBILITY ON DYNAMIC BEHAVIOUR

Having been checked the suitability of FEA model, discussion on analysis and results of the present study is presented from this section onwards. To investigate the dynamic behaviour of circular tanks, tanks of 12m height with 0.5m wall thickness covering over a wide range of aspect ratio with properties of concrete and water as given Table 5.3 is considered for the study. In order to investigate the effect of wall flexibility on impulsive and convective modes of vibration of the tank, modal analyses were performed on cylindrical ground supported tanks of outer diameter 35m, 25m and 15m.

Table 5.3 Properties of concrete and water

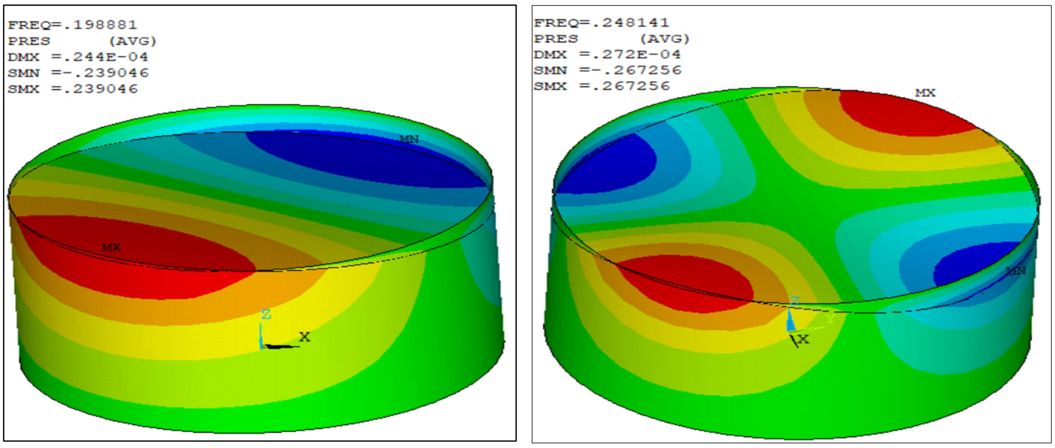
| Concrete | | | Water | |
|---|---|---|---|---|
| Density ($kg/m^3$) | Modulus of Elasticity (GPa) | Poisson's Ratio | Density ($kg/m^3$) | Bulk modulus (GPa) |
| 2500 | 27.83 | 0.16 | 1000 | 2.2 |

The natural frequencies and modal response values for the impulsive and convective modes with the highest participation factors among all modes of vibration has been noted. The fundamental impulsive and convective modes are identified as those with the largest participation factors in the horizontal direction. The first two impulsive modes of vibrations for flexible tank of 25m diameter with water height of 11m (considered as full fill condition/design height) are given in Fig. 5.4. Fig. 5.5 represents first two convective modes of vibration of same tank.




(a) mode1 (b) mode 2

Fig 5.4 Impulsive modes of vibration of tank (D – 25m, h -11m)




(a) mode 1 (b) mode 2

Fig. 5.5 Convective modes of vibration of tank (D – 25m, h -11m)

The fundamental impulsive frequencies of flexible and rigid water tanks (E= 10 Ec) for full fill and empty conditions are given in Table 5.4. for tank of same dimension and mass. It can be noticed that for tanks of same height, low diameter tanks vibrate with higher frequency i.e. as aspect ratio (height of tank/inner diameter) increases from 0.35 to 0.86, impulsive frequency increases.

Table 5.4 Fundamental impulsive frequency of flexible and rigid circular tanks

| Fill condition | Fundamental impulsive frequency (Hz) | | | | | |
|---|---|---|---|---|---|---|
| | Inner Diameter =34 m<br>Aspect ratio = 0.35 | | Inner Diameter =24m<br>Aspect ratio = 0.5 | | Inner Diameter=14m<br>Aspect ratio = 0.86 | |
| | Flexible | Rigid | Flexible | Rigid | Flexible | Rigid |
| Full fill | 13.36 | 39.16 | 15.95 | 45.71 | 22.20 | 62.34 |
| Empty tank | 23.66 | 74.83 | 26.69 | 84.42 | 27.55 | 87.14 |

The increase in rigidity of tank wall or assumption of rigid wall leads to an increase in impulsive frequency as expected. While the impulsive frequency of empty rigid tank is about 3.16 times of flexible tank, for rigid tanks with full fill condition, the corresponding values are 2.93, 2.83 and 2.8 times of flexible tanks of 35m, 25m and 15m diameters respectively indicating the influence of fluid-structure interaction.

It has been observed that all impulsive modes contributing to the response to excitation are cantilever beam type modes with circumferential wave number (n) equal to 1 in which shell cross section remains circular while vibrating. Results of free vibration analyses show that the participation factors and effective masses associated with the cos(nh) type modes (n > 1) are very small and therefore can be neglected in determining the response to excitations as stated by various research works (Barton and Parker, 1987; Virella et al, 2006; Chen and Kianoush, 2009) for cylindrical tanks.

The translation of tank wall in radial direction along the height of both flexible and rigid wall tanks in the first two modes is depicted in Fig.5. 6. It can be noticed that variation in radial displacement along tank height is more profound for flexible tank for both modes of vibrations. For the first mode of vibration of 25m diameter tank filled with design water height, maximum radial displacement occurs at a height of 7m from tank bottom for both flexible and rigid tanks and for the second mode of vibration, maximum is at top of tank wall for flexible tank and at a height of 10m from bottom for rigid tank.

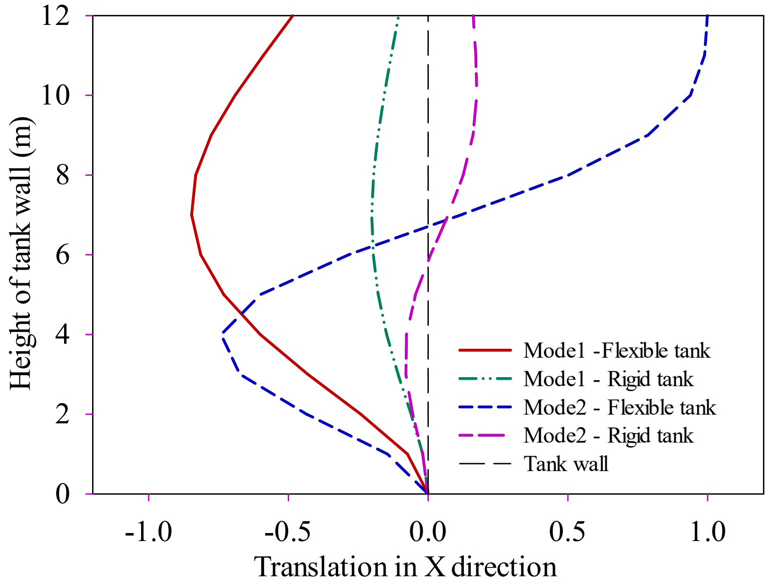



Fig. 5.6 Radial displacement of tank wall at first two modes

The fundamental frequency in convective modes of vibration for different fill conditions are noted and is presented in Table 5.5. By performing modal analysis on flexible and rigid water tanks for different water heights, it is observed that convective frequency obtained for both tanks are same and is independent on the rigidity parameter of the tank for any water height and aspect ratio.

Table 5.5 Fundamental convective frequencies of circular tanks with varying water heights

| Height of water (m) | Fundamental convective frequency (Hz) | | |
|---|---|---|---|
| | 35 m Diameter Tank<br>Aspect ratio = 0.35 | 25 m Diameter Tank<br>Aspect ratio = 0.5 | 15 m Diameter Tank<br>Aspect ratio = 0.86 |
| 11 | 0.150 | 0.199 | 0.248 |
| 6 | 0.203 | 0.227 | 0.251 |
| 3 | 0.285 | 0.297 | 0.282 |

Fig.5.7 gives the variation in pressure degree of freedom along the diameter on the free liquid surface of tank having 25m diameter and 11m water height.

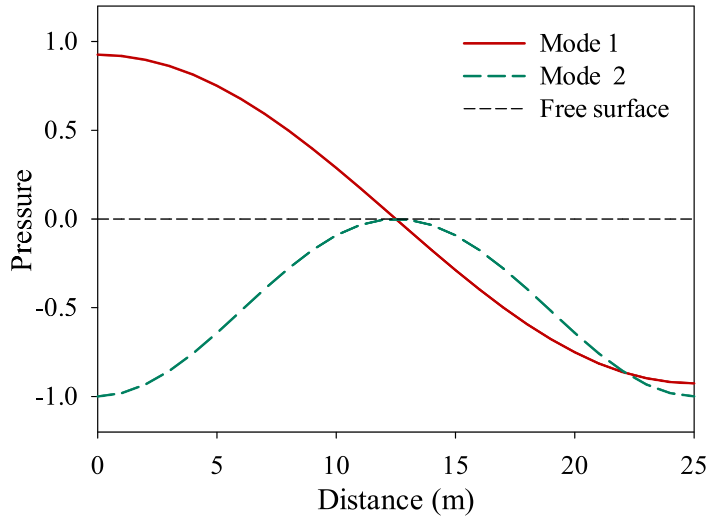



Fig. 5.7 Variation of pressure along the free surface of water (D -25m, full fill)

It can be noticed that, convective vibrations are of low frequency and the influence of aspect ratio, and water height on convective frequency are less significant as noted by other researchers (Moslemi and Kianoush, 2012; Rawat et al., 2015). Hence it is concluded that the influence of convective response can be ignored in seismic response analysis of ground supported cylindrical water tanks which are free at top. In the following studies, influence of various parameters on impulsive response of tanks are only considered. Also, it is observed from Fig.5.6 that impulsive response of rigid tank is much lower than flexible tank, rigidity assumption postulated by earlier researchers in mathematical formulation is not incorporated in further finite element analyses.

## 5.4 INFLUENCE OF ASPECT RATIO ON IMPULSIVE FREQUENCY

Section 5.3 implies that fundamental frequency of tank is also affected by aspect ratio of tank and water fill conditions. Hence the influence of aspect ratio on the fundamental frequency of ground supported tank was studied by performing the free vibration analysis on cylindrical tanks having idealised condition of tank walls fixed at bottom with aspect ratio 0.2, 0.4, 0.6, 0.8, 1.0, 1.2, 1.4, 1.6, 1.8 and 2.0 for different water fill conditions. All ground supported concrete tanks are of height 12m with wall thickness of 0.5m. The properties of tank and water considered are same as given in Table 5.3.

The variation of fundamental frequency of the empty tank with aspect ratio is presented in Fig. 5.8. The fundamental frequency increases for tanks upto aspect ratio 0.8 and decreases with further increase in aspect ratio. This is due to less reduction in mass with respect to reduction in stiffness as the aspect ratio increases.

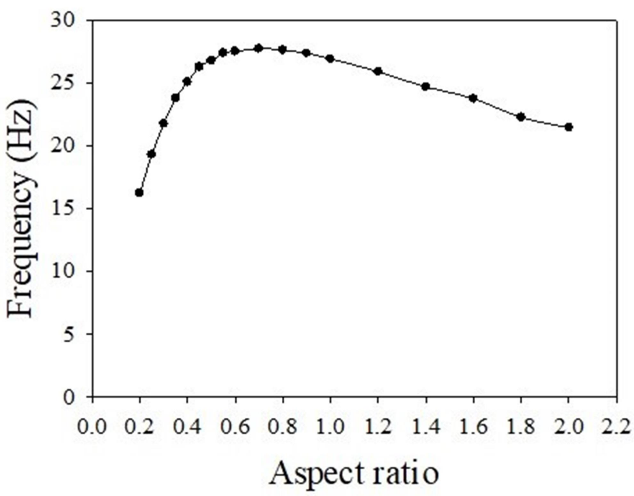



Fig. 5.8 Variation of fundamental frequency of empty tank with aspect ratio

The fundamental impulsive frequency of the tanks with design water height obtained from finite element analysis (FEA) for tanks with varying aspect ratios along with the values computed using IS and ACI codes are presented in Fig.5.9.

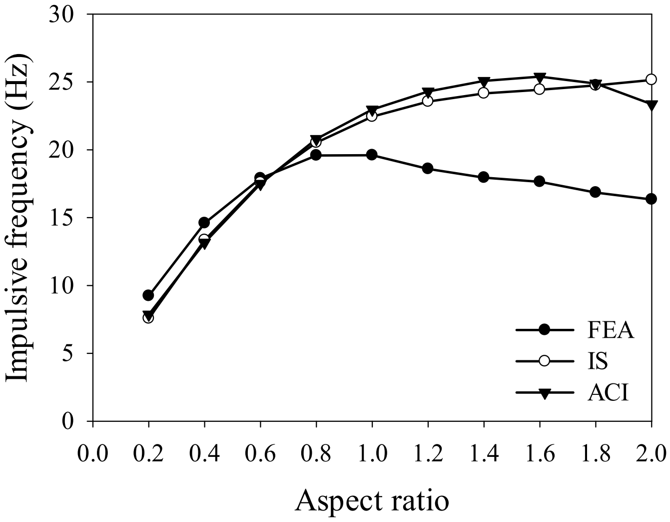
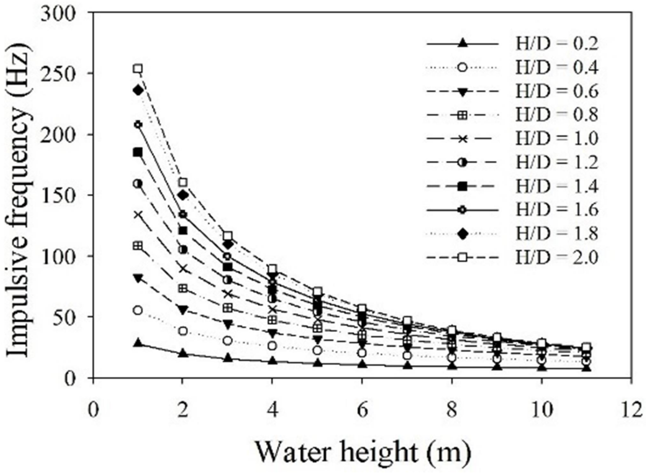



Fig. 5.9 Variation of fundamental impulsive frequency of tanks at design water height with aspect ratio

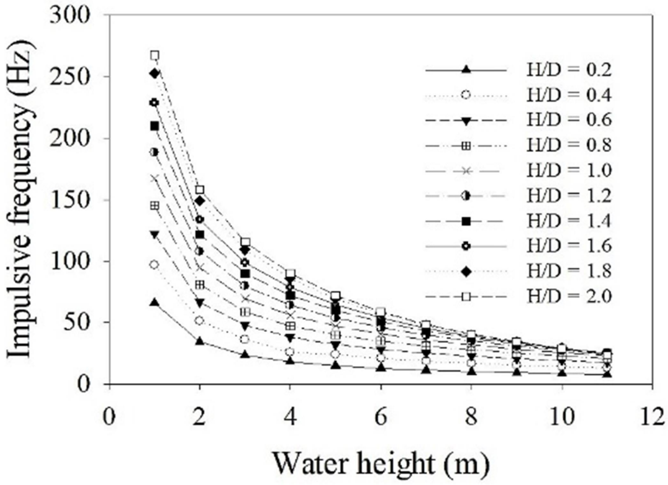
It can be noted from FE analysis that fundamental frequency increases with increase in aspect ratio from 0.2 to 1 for tank filled with designed water height and after that the influence of aspect ratio on fundamental frequency is quite marginal. When the tank with aspect ratio 0.6 is filled with design water depth, the impulsive frequency values given by FEA, ACI 350.3 and IS 1893:Part 2 are covenant. But for other aspect ratios there is variation from the values calculated by the formula proposed in the codes and this variation increases with deviation from aspect ratio 1.0. For slender tanks, the mass of tank wall is not negligible in comparison with mass of water, as in Housner's rigid mass assumption.

## 5.5 INFLUENCE OF WATER HEIGHT ON IMPULSIVE FREQUENCY

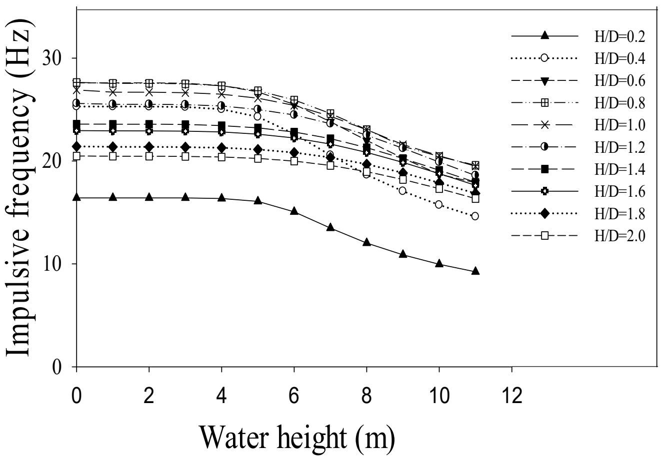
Fig. 5.10 (a-c) gives the variation in impulsive frequency of tank with variation in water height for different aspect ratios. Fig. 5.10 (a) and (b) are plotted to check whether the formulae specified in IS 1893: Part2 and ACI 350.3 can be applied for water fill conditions other than design/maximum water height.




(a) IS formula




(b) ACI formula




(c) Finite element analysis

Fig. 5.10 Variation of natural frequency with water depth for tanks of various aspect ratios

A comparison of Fig.5.10 (a) and Fig.5.10 (b) with Fig. 5.8 shows that for low water level in the tank, impulsive frequencies calculated using codal formulae are significantly higher than that of empty tank indicating that the expressions for the determination of impulsive frequency given by codes are applicable only for design/ maximum water height. Though the formulae include the design depth of water and thickness of tank wall, it is insufficient to incorporate the effect of mass and stiffness contributed by the tank wall for tanks filled much below the design depth.

However, from the present finite element analyses (Fig. 5.10(c)) it can be observed that, as the water height decreases from full fill condition to half fill condition, the frequency increases and reaches a maximum value, but further reduction of water in the tank does not cause significant variation in impulsive frequency. This behaviour is due to the complicated fluid-structure interaction, which is not properly incorporated in various codes. As impulsive mass is considered to be concentrated at the tank bottom, percentage of mass participating in impulsive mode of vibration is observed to increase with reduction in water height. Also, it can be observed that impulsive frequency of tank with 1m water height as given by finite element analysis is almost near to that of empty tank whereas the impulsive frequency of tank with 1m water height calculated from standard codes instead of design height are significantly higher than that of empty tank.

It can be noticed that the ACI formula for determination of fundamental impulsive frequency and IS code formula for fundamental period exclude the height of tank wall and the formulae can be applied only for design depth/maximum depth of water as specified in the codes and not for all water heights. So, the design formula given by standard codes based on Housner's model without considering the weight of tank wall cannot be applied for tanks with low water fill condition. Hence it is recommended to incorporate the effect of water fill condition in the codal provisions for the determination of impulsive frequency so that it can be used by the designer for tanks of any water fill condition.

## 5.6 PROPOSED COEFFICIENT FOR IMPULSIVE FREQUENCY

Based on these studies, a new coefficient for determination of impulsive frequency is formulated in the present study that take into account the effect of water fill condition. The proposed coefficient is designated as '$C_{i,\ Proposed}$', such that by incorporating the coefficient in the expression for time period of impulsive mode of vibration specified by IS 1893: Part2 (2014), as given in Equation 5.4 can be applied for all water fill conditions with sufficient accuracy. The computation of the coefficient '$C_{i,Proposed}$' for the for tank with aspect ratio 0.6 is given in Table 5.6 and corresponding equation is plotted in Fig. 5.11. The same procedure is performed for tanks of all aspect ratios.

Table 5.6 Proposed coefficient for time period of impulsive mode of vibration

| Water Height (h) (m) | Water height to Tank height (h/H) | Impulsive frequency IS 1893: Part 2 (Hz) | $C_i$ (as per IS 1893: Part 2) | Impulsive frequency FEA (Hz) | $C_{i,\ Proposed}$ | $\frac{C_{i,Proposed}}{C_i}$ |
|---|---|---|---|---|---|---|
| 11 | 0.92 | 17.59 | 4.28 | 17.9 | 4.20 | 0.98 |
| 10 | 0.83 | 19.12 | 4.33 | 18.71 | 4.42 | 1.02 |
| 9 | 0.75 | 20.88 | 4.40 | 20.27 | 4.54 | 1.03 |
| 8 | 0.67 | 22.91 | 4.51 | 21.99 | 4.70 | 1.04 |
| 7 | 0.58 | 25.4 | 4.65 | 23.81 | 4.96 | 1.07 |
| 6 | 0.5 | 28.41 | 4.86 | 25.5 | 5.41 | 1.11 |
| 5 | 0.42 | 32.21 | 5.14 | 26.72 | 6.19 | 1.21 |
| 4 | 0.33 | 37.25 | 5.55 | 27.33 | 7.57 | 1.36 |
| 3 | 0.25 | 44.5 | 6.20 | 27.55 | 10.01 | 1.62 |
| 2 | 0.17 | 56.35 | 7.34 | 27.6 | 14.99 | 2.04 |
| 1 | 0.08 | 82.37 | 10.05 | 27.6 | 29.98 | 2.98 |

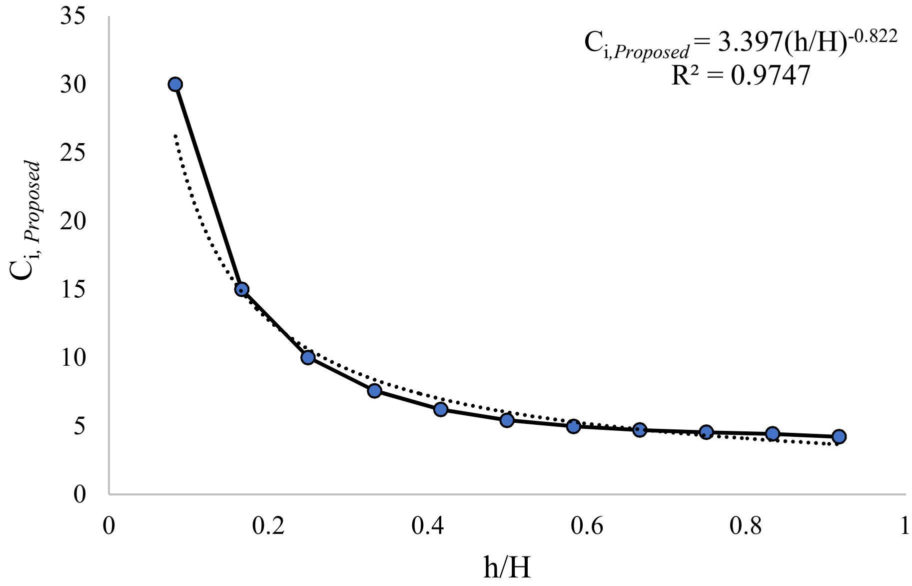



Fig. 5.11 Proposed coefficient of impulsive time period for circular tank for tank with aspect ratio 0.6

The coefficient '$C_{i,\ Proposed}$' is based on the aspect ratio of tank. The expressions for the determination of coefficients are derived for tanks of (i) aspect ratio upto 0.6 (ii) aspect ratio 1.2 and (iii) aspect ratio 2.0. The proposed value of coefficient of time period for impulsive mode of vibration $C_{i,Proposed}$ for any aspect ratio can be obtained from Fig. 5.12 and the corresponding formulae are given in equation 5.6 – 5.8.

$$C_{i,\,proposed} = 3.397\frac{h}{H}^{-0.822}, \quad for\ \frac{H}{D} upto\ 0.6 \tag{5.6}$$

$$= 4.252\frac{h}{H}^{-0.885}, \quad for\ \frac{H}{D} = 1.2 \tag{5.7}$$

$$= 6.73\ \frac{h}{H}^{-0.923}, \quad for\ \frac{H}{D} = 2.0 \tag{5.8}$$

where $\frac{h}{H}$ is the ratio of height of liquid in the tank to height of tank wall

The equation 5.4 for determination of time period of impulsive mode of vibration given by IS 1893: part 2 (2014) is modified so that it can be used for any water fill condition and is given in equation 5.9.

$$T_i = C_{i,proposed} \frac{h\sqrt{\rho}}{\sqrt{\frac{t}{D}}\sqrt{E}} \tag{5.9}$$

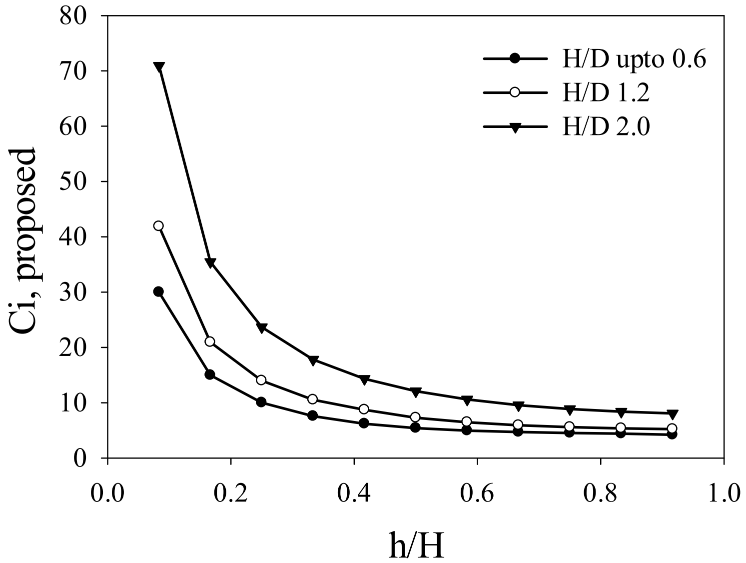


Fig. 5.12 Proposed coefficient of impulsive time period for circular tank

The impulsive frequency of the concrete tank having diameter 16m, height of tank wall 8m and wall thickness 0.4m is calculated using the proposed formula for different water fill conditions. Material properties of tank and water are same as given in Table 5.3. The results obtained from finite element analysis are compared with proposed formula and IS code formula (applied for different water height) and are presented in Table 5.7 to know how efficiently the proposed formula can be used for determination of impulsive frequency of water tanks in different fill conditions.

Table 5.7 Comparison of impulsive frequency by FEA with proposed formulae and IS code values

| Water height (m) | Fundamental Impulsive frequency (Hz) | | |
|---|---|---|---|
| | Height of Tank - 8 m; Tank diameter – 16 m (Aspect ratio – 0.5) | | |
| | FEA | $C_{i,proposed}\frac{h\sqrt{\rho}}{\sqrt{\frac{t}{D}}\sqrt{E}}$ | IS 1893: Part 2 $C_i\frac{h\sqrt{\rho}}{\sqrt{\frac{t}{D}}\sqrt{E}}$ |
| 7 | 26.29 | 26.75 | 26.71 |
| 6 | 29.86 | 30.40 | 30.14 |
| 5 | 34.05 | 34.34 | 34.49 |
| 4 | 37.55 | 38.25 | 40.26 |
| 3 | 38.69 | 39.98 | 48.50 |
| 2 | 40.6 | 41.33 | 61.96 |
| 1 | 40.6 | 41.37 | 91.34 |

The proposed formula is derived by performing modal analysis of cylindrical concrete tanks having 12m height and of aspect ratio varying from 0.2 to 2.0. Table 5.7 infers that the proposed formula can be applied for determination of impulsive frequency of concrete tanks of height other than 12m, and having ratio less than 2.0.

The time period of impulsive mode of vibration depends on several parameters: aspect ratio, water height, thickness of tank wall, density of liquid, material of tank wall and fixity condition. So further study may be required on tanks of different materials and fixity conditions for determination of coefficient of impulsive time period for circular tank to accommodate all liquid fill conditions, which is beyond the scope of this work.

## 5.7 SUMMARY

Free vibration analyses of the ground supported cylindrical tanks were carried out to define the dynamic characteristics of ground supported cylindrical concrete water tanks with varying aspect ratio with different water fill conditions. The convective mode of vibration is found to have less significance on dynamic characteristics of the tanks. The flexibility of tank wall influences the dynamic response of the tank as the response of flexible tank is more than that of rigid tank of same dimensions. The fundamental impulsive frequency of full fill tank obtained from finite element analysis matches well with values obtained by standard codes but there is large discrepancy at low fill conditions. Since the codal expressions are insufficient to incorporate the effect of water fill condition, a modified coefficient for determination of time period of impulsive mode of vibration is proposed such that same expression can be used for tank at any fill condition.

As it is observed that the free response of the tank is being altered by the fill condition, seismic analysis of the tanks at various water depths are to be performed to know how these tanks at different fill conditions respond to earthquakes. The response of the tanks at various water fill condition to seismic loading are discussed in detail in Chapter 6.

# CHAPTER 6

# SEISMIC ANALYSIS OF GROUND SUPPORTED CYLINDRICAL WATER TANKS

## 6.1 GENERAL

Availability of water supply after earthquakes is crucial to meet various demands of mankind. Therefore, large capacity water reservoirs must be safe and need to remain functional after earthquakes. The earthquake responses of the tanks are evaluated by performing the time history analysis in which tanks are subjected to real earthquake accelerograms. This chapter provides an overview of the response of ground supported circular tanks, such as hoop force, bending moment, and base shear due to hydrodynamic pressure exerted on the wall due to the seismic loading.

Seismic response of the tank is affected by aspect ratio, water fill condition, support condition of tank, characteristics of earthquake and properties of soil underneath. The study presented in this chapter is limited to the response of ground supported cylindrical tank having ideal condition of tank wall fixed at bottom subjected to seismic loading.

### 6.1.1 Geometric and Material Properties of Tanks

The effect of dimensions of tank on the seismic response of ground supported cylindrical water tanks, is studied by performing seismic analysis on tanks with different aspect ratio. The effect of water fill condition on the seismic response of water tanks are studied by considering the tank in three water fill conditions – full fill (design water height of 11m), half fill conditions (water height - 6m) and quarter fill (water height of 3m). All the ground supported concrete water tanks are of height

12m with wall thickness 0.5m and of varying diameter such that aspect ratios (AR) are 0.6, 0.8, 1.0, 1.2 and 1.4. In this aspect ratio range, $C_i$ for determination of impulsive time period given by IS 1893: Part 2 and $C_w$ for determination of impulsive natural frequency for circular tanks by ACI 350.3 show reversal in slope of the curve as can be seen from Fig. 5.1 and Fig.5.2. Table 6.1 gives the description of tanks along with water fill conditions considered for the study. The material properties of the ground supported tanks are the same as given in Table 5.3.

Table 6.1 Description and designation of the tanks

| Inner diameter (m) | Aspect ratio (Height of tank =12 m) | Water height | Tank Designation |
|---|---|---|---|
| | | Design height | AR0.6F |
| 20 | 0.6 | Half fill | AR0.6H |
| | | Quarter fill | AR0.6Q |
| | | Design height | AR0.8F |
| 15 | 0.8 | Half fill | AR0.8H |
| | | Quarter fill | AR0.8Q |
| | | Design height | AR1.0F |
| 12 | 1 | Half fill | AR1.0H |
| | | Quarter fill | AR1.0Q |
| | | Design height | AR1.2F |
| 10 | 1.2 | Half fill | AR1.2H |
| | | Quarter fill | AR1.2Q |
| | | Design height | AR1.4F |
| 8.5 | 1.4 | Half fill | AR1.4H |
| | | Quarter fill | AR1.4Q |

### 6.1.2 Finite Element Modelling of the Tanks

Finite element models of all tanks given in Table 6.1 are generated using the methodology described in chapter 4. The structural motion and fluid pressure at the interface are coupled to ensure the effect of fluid-structure interaction. All tanks are having idealised fixed boundary condition. Since the tanks are having idealised fixed base condition, soil-structure interaction effect is not taken into account.

Since the convective component is observed to have a negligible effect on the overall seismic behaviour of the tank as noticed by other researchers (Moslemi and Kianoush, 2012; Rawat et al., 2015) only the impulsive part of response is considered in this study and the effect of sloshing is neglected.

## 6.2 STATIC ANALYSIS OF GROUND SUPPORTED TANKS

Prior to perform the seismic analysis of the tanks, static analysis was performed on all tanks. The radial displacement, hoop force, bending moment and base shear developed due to the hydrostatic pressure on tank wall is given in Table 6.2. These results are further used to quantify the consequence of seismic event on response parameters of the tank.

All response parameters are observed to be decreased with increase in aspect ratio for tank with height of 12 m. Also, response of full fill tank is higher than that of half fill and quarter fill conditions for tanks with any aspect ratio as expected.

Table 6.2 Results of static analysis of tanks of varying aspect ratio

| Tank Designation | Max. Displacement (mm) | Max. Hoop force (kN/m) | Max. Bending moment (kNm/m) | Max. base shear (kN/m) |
|---|---|---|---|---|
| AR0.6F | 0.52 | 697.6 | 68.4 | 118.6 |
| AR0.6H | 0.18 | 224.7 | 28.7 | 58.8 |
| AR0.6Q | 0.03 | 36.5 | 6.2 | 19.9 |
| AR0.8F | 0.31 | 565.5 | 47.7 | 97.8 |
| AR0.8H | 0.12 | 199.9 | 20.9 | 49.5 |
| AR0.8Q | 0.02 | 35.0 | 4.9 | 17.9 |
| AR1.0F | 0.21 | 470.4 | 35.6 | 83.8 |
| AR1.0H | 0.08 | 180.4 | 16.1 | 42.7 |
| AR1.0Q | 0.02 | 32.9 | 4.22 | 16.29 |
| AR1.2F | 0.15 | 398.4 | 27.8 | 73.3 |
| AR1.2H | 0.06 | 162.9 | 12.7 | 37.5 |
| AR1.2Q | 0.02 | 31.1 | 3.9 | 14.9 |
| AR1.4F | 0.12 | 353.6 | 22.2 | 64.8 |
| AR1.4H | 0.05 | 147.3 | 10.3 | 33.4 |
| AR1.4Q | 0.01 | 30.7 | 3.5 | 13.7 |

Structural design of the water tank of outer diameter 20m with design water height condition (AR0.6F) has been done based on the maximum response of static analysis and is given in Appendix. It is noticed that the assumed wall thickness of 500mm is sufficient to withstand the structural demands. Hence, for the seismic analysis of the tanks of aspect ratio 0.6, 0.8, 1.0, 1.2 and 1.4, uniform wall thickness of 500 mm is adopted.

## 6.3 TIME HISTORY ANALYSIS OF GROUND SUPPORTED TANKS

Time history dynamic analysis is a powerful and reliable method for seismic assessment of structures especially for those involving various sources of nonlinear behaviour. In this method, dynamic behaviour of tank is analysed under earthquake accelerogram that is exerted as a function of time, and response history are obtained. Selection of appropriate accelerograms and determination of tank responses under them result in accurate evaluation of seismic performance of water tank.

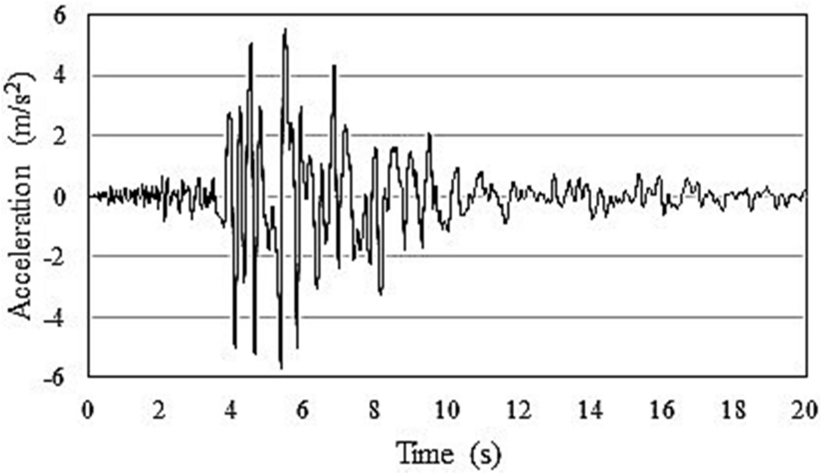
### 6.3.1 Seismic Loading

To investigate the effect of frequency content of earthquake on seismic response of ground supported tanks, same tank model is subjected to past three earthquakes ground motions separately, having different characteristics. The ratio of peak ground acceleration (PGA) to peak ground velocity (PGV) is considered as a good indicator of the frequency characteristics of the earthquake. According to the ratio of PGA in units of '*g*' to PGV in units of 'm/s' (PGV), earthquakes are categorized as that of low frequency when PGA/PGV < 0.8, intermediate frequency when 1.2 < PGA/PGV < 0.8 and high frequency when PGA/PGV > 1.2 (Kianoush and Ghaemmaghami, 2011). Northridge record, 1994 having low frequency content, the Imperial Valley record, 1940 of intermediate frequency content and Koyna record, 1967 of high frequency content are the three earthquakes considered in the study and its characteristics are given in Table 6.3. As it is observed by Moslemi (2005) that though the pure vertical ground motion could cause dynamic effects as high as those of the horizontal motion when considered separately, it is of less importance when horizontal and vertical earthquake components are applied together, only horizontal component of earthquake are considered as input motion in this study. An integration step of 0.02 second is used to characterize the tank response.

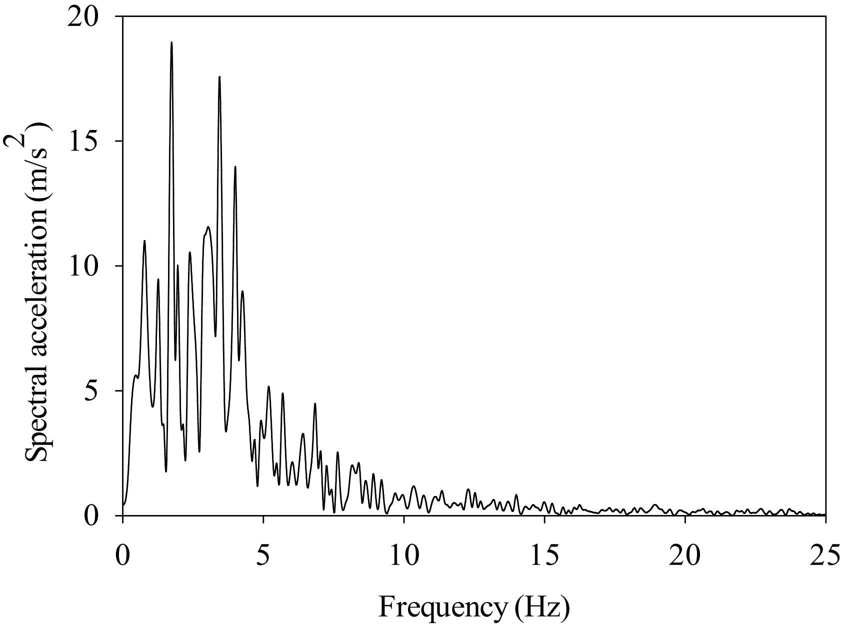
Table 6.3. Characteristics of past earthquakes considered for transient analysis

| Name of Earthquake (Year) | Recording station | Design ation | Moment Magnitude | PGA | PGA/ PGV | Category of earthquake based on frequency content |
|---|---|---|---|---|---|---|
| Northridge (1994), | New Hall La county Fire Station, California | NEQ | 6.7 | 0.583 g | 0.51 | Low frequency |
| Imperial Valley (1940) | El Centro, California | IEQ | 6.9 | 0.349 g | 0.88 | Medium frequency |
| Koyna (1967) | Monolith1A, Koyna dam, India | KEQ | 6.6 | 0.489 g | 2.49 | High frequency |

Acceleration time history and response spectra of horizontal component of first 20s of Northridge earthquake, 15s of Imperial Valley earthquake and 10.76s of Koyna earthquake considered in the present study are given in Fig 6.1, Fig. 6.2 and Fig.6.3 respectively.

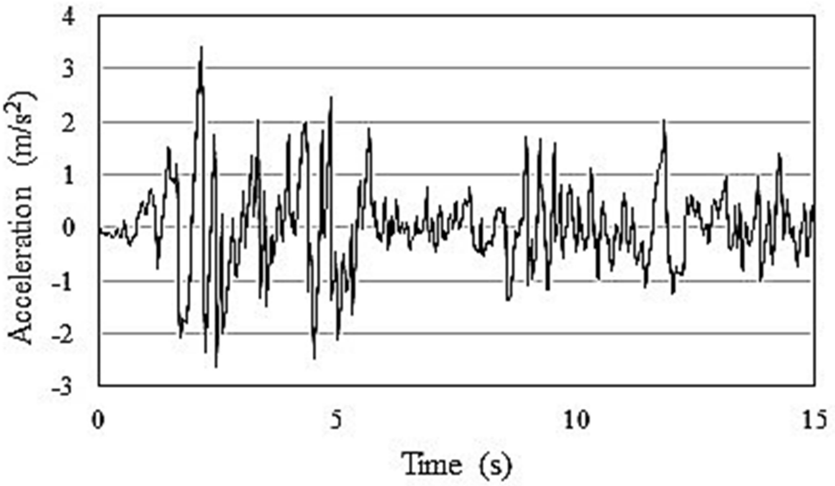



(a)

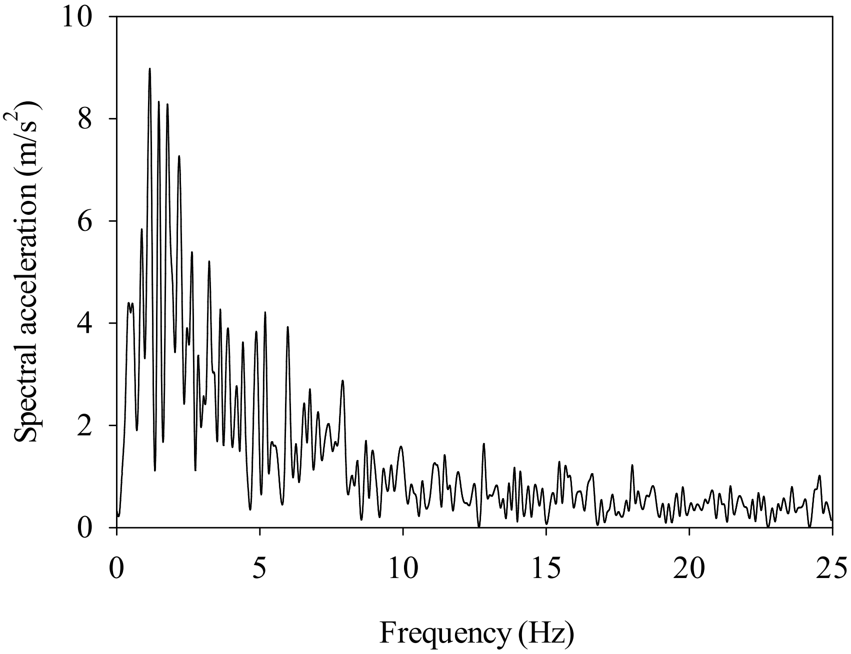



(b)

Fig. 6.1 (a) Acceleration time history and (b) response spectra of Northridge earthquake, 1994

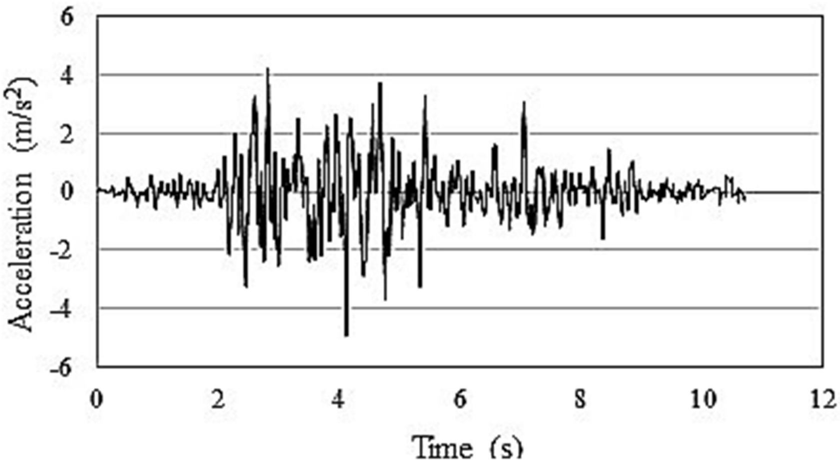



(a)

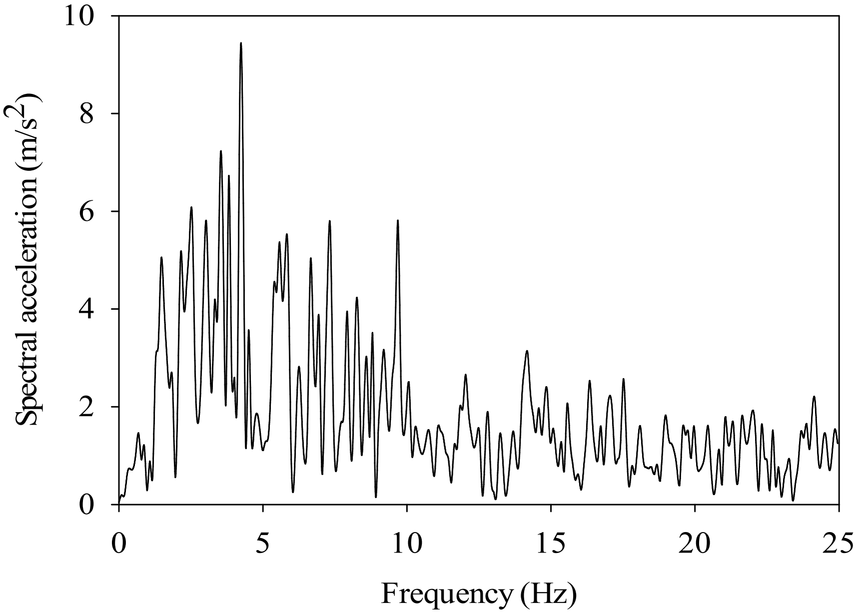



(b)

Fig. 6.2 (a) Acceleration time history and (b) response spectra of Imperial Valley earthquake, 1940




(a)




(b)

Fig. 6.3 (a) Acceleration time history and (b) response spectra of Koyna earthquake, 1967

## 6.4 RESPONSE OF TANKS TO NORTHRIDGE EARTHQUAKE OF LOW FREQUENCY CONTENT

The tanks having different aspect ratios under different water fill conditions are subjected to time history of Northridge earthquake acceleration for 20s. Responses of the tank such as radial displacement, hoop force, bending moment and base shear due to hydrodynamic pressure exerted by the liquid on the tank wall are examined and presented. The maximum values of response anywhere on the tank wall during seismic loading were noted and are presented in the form of bar charts. The time at which maximum response occurring was observed to be different for tanks with different aspect ratio.

### 6.4.1 Effect of Aspect Ratio and Water Height on Response Parameters of Tank due to Northridge Earthquake

***Radial Displacement***

The maximum radial displacement of the tank under Northridge earthquake is presented in Fig. 6.4. It can be seen that peak displacement in radial direction occurs in full fill tank having aspect ratio (AR) 0.6 and is lowest for tank with aspect ratio 1.0. The maximum radial displacement of AR0.6F is 20.69% more than that in AR1.0F.

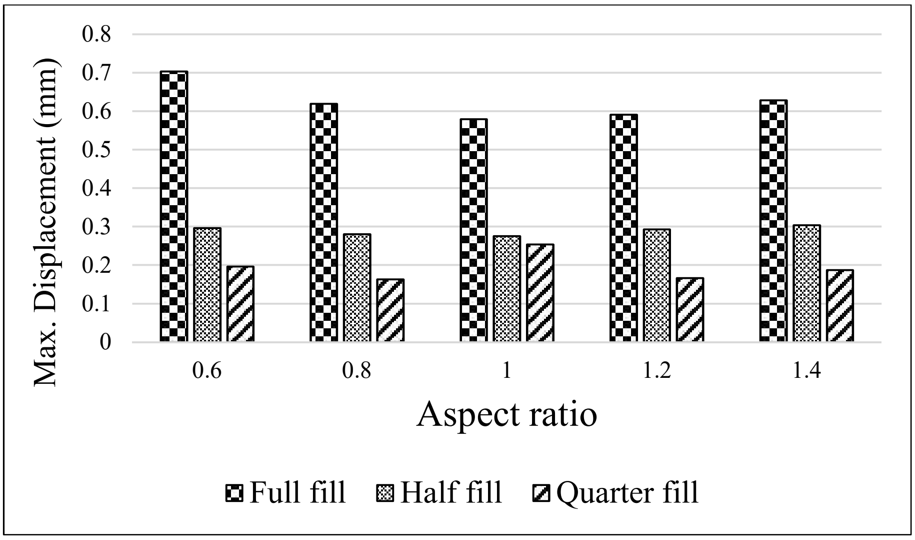
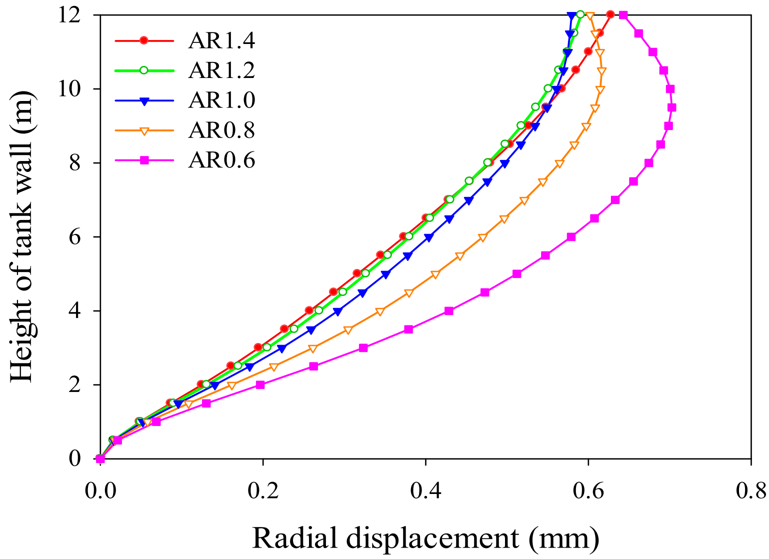
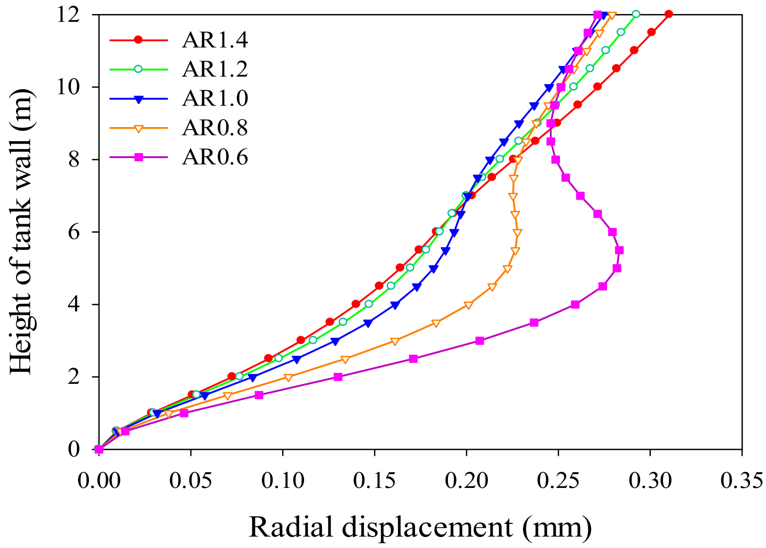



Fig. 6.4 Maximum radial displacement of the tank due to Northridge earthquake

Variation of radial displacement along the height of tank wall for tanks of various aspect ratios, for full fill and half fill conditions at the instant of maximum displacement on tank wall, are plotted in Fig.6.5.




(a) full fill condition




(b) half fill condition

Fig. 6.5 Variation of radial displacement along height of tank wall (Northridge earthquake)

From Fig.6.5 (a), it can be noticed that, in full fill condition, radial displacement is decreased with increase in aspect ratio. There is marginal reduction in displacement near the top of tank wall with AR1.0 in comparison to AR1.2 and 1.4, attributable to the variation in hoop effects and bending effects. For wider tanks of AR0.6 and AR0.8 with full fill condition, the maximum displacement is at a height of 9.5m and 10.5m from the bottom of tank respectively. For tanks with high aspect ratio, the maximum displacement at top of tank wall may be attributed to fact that for tanks with high aspect ratio, the tank and water body act as single mass.

For tanks in half fill condition, (Fig.6.5(b)) except for tank with AR0.6, the maximum displacement occurs at top of tank wall. For the wider tank with AR0.6, displacement is maximum at a height of 5.5m from bottom of the tank.

The response displacement decreases with decrease in water height for tanks of all aspect ratios (Fig. 6.4) considered. The maximum response displacement of the tank in half fill condition is 0.41, 0.45, 0.48, 0.49 and 0.48 times of maximum displacement of the full fill tank whereas for quarter fill condition, maximum values are 0.29, 0.26, 0.43, 0.29 and 0.3 times of maximum displacement in full fill condition for tanks of AR 0.6, 0.8, 1.0, 1.2 and 1.4 respectively.

Time history response of radial displacement at the top of wall of tank with AR 1.2 in full fill and half fill conditions are given in Fig 6.6. Maximum radial displacement of 0.59mm occurs at 5.82s for full fill and 0.29mm at 5.8s for half fill conditions of the tank. From the figure, it is evident that the responses of half fill tank is always less than full fill tank.

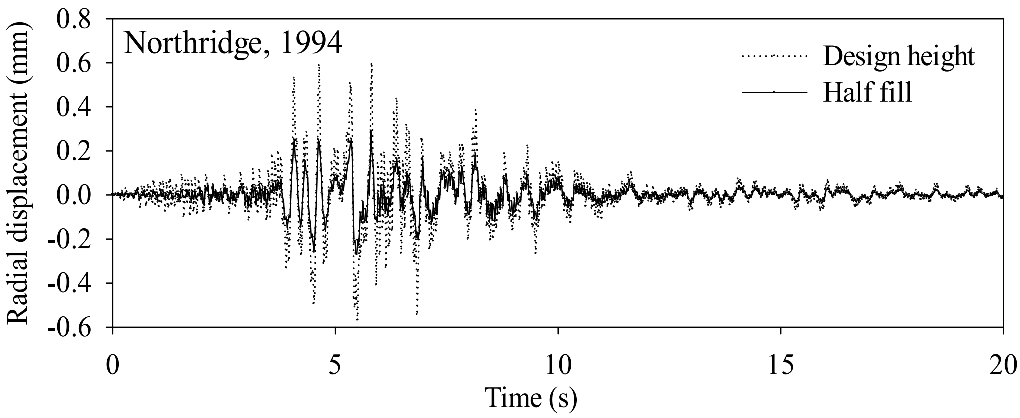



Fig. 6.6 Time history response of radial displacement of tank with AR 1.2 in full fill and half fill conditions

***Hoop force***

The maximum hoop force is plotted in Fig. 6.7 for tanks of different aspect ratios and is observed to be highest for the tank with AR 0.6 for full fill condition. The maximum hoop force decreases as aspect ratio increases from 0.6 to 1.4 for both full fill, half fill and quarter fill conditions as observed in static analysis. The maximum hoop force of tank with AR 0.6 in full fill, half fill and quarter fill conditions are 3.59, 2.8 and 1.65 times of corresponding value of tank with AR 1.4.

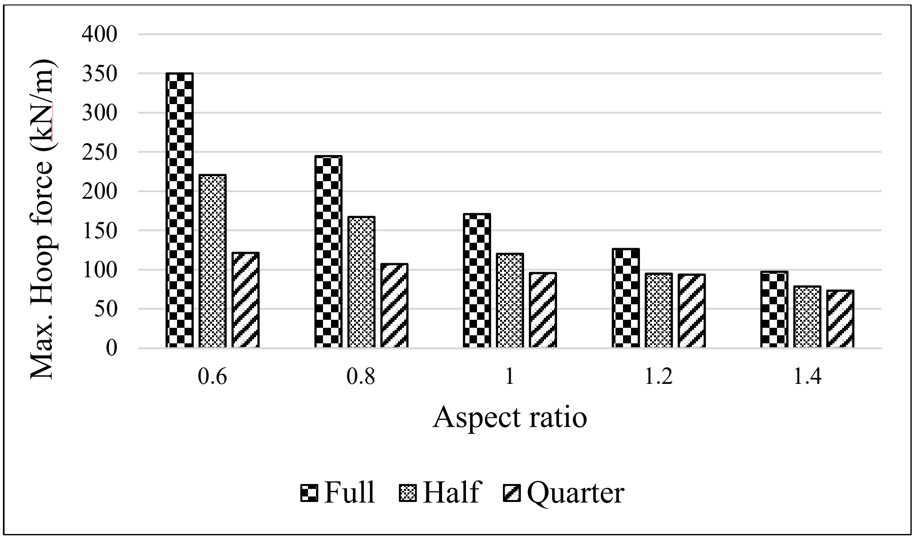
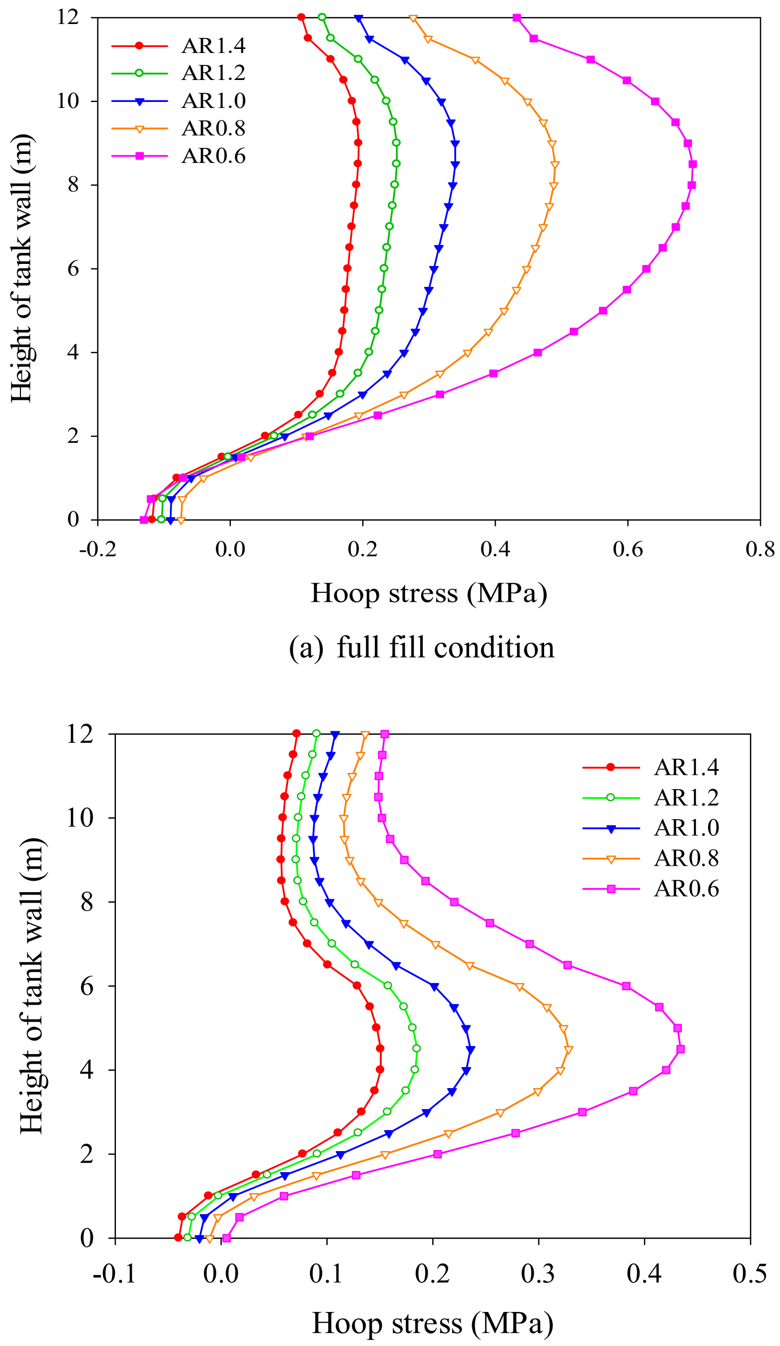



Fig. 6.7 Maximum hoop force of the tank due to Northridge earthquake

Variation of hoop stress along the height of tank wall at the instant of maximum response in the tank wall for various aspect ratio of tank for full fill and half fill conditions are given in Fig.6.8 (a) and Fig.6.8(b) respectively. Hoop stress is maximum at a height of 8.5m to 9m from bottom of tank for full fill condition and it is at a height of 4m from the tank bottom for half fill condition. All tanks show similar hoop behaviour under Northridge earthquake.




(a) full fill condition

(b) half fill condition

Fig. 6.8 Variation hoop stress along height of tank wall (Northridge earthquake)

Time history plot of hoop force of the tank with AR1.2 in full fill condition, at a height of 9m from bottom of tank is given in Fig. 6.9. The maximum response of 126.27 kN/m occurs at 5.82s.

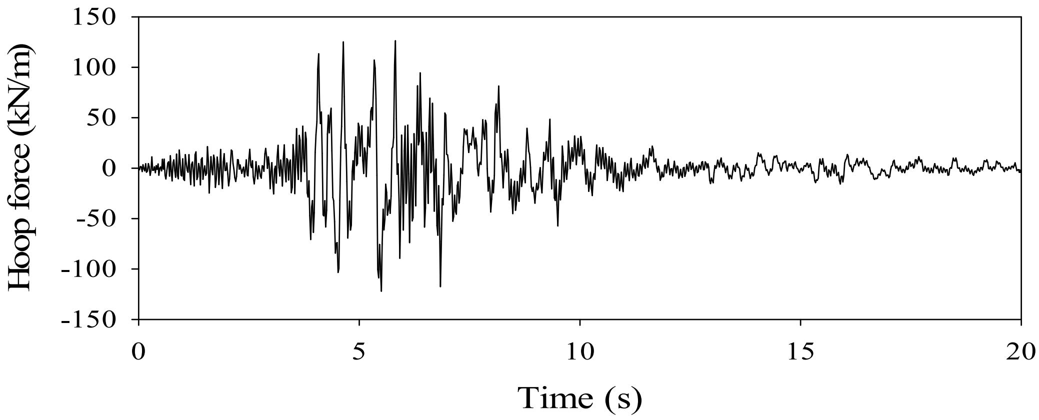



Fig. 6.9 Time history response of hoop force of tank with aspect ratio 1.2 in full fill condition

It can also be noticed from Fig. 6.7 that the hoop force of half fill tank with AR 0.6 is 63% of hoop force in full fill condition while for tank with AR 1.4, 80.7% of hoop force in full fill condition occurs for half fill condition. Variation in maximum hoop force with water height reduces with increase in aspect ratio.

***Bending moment***

As in case of hoop force, maximum bending moment also observed to decrease as aspect ratio increases from 0.6 to 1.4 for tanks in full fill, half fill and quarter fill conditions (Fig. 6.10). The maximum bending moment of AR0.6F is 1.33 times AR1.4F and of AR0.6 in half and quarter fill conditions are about 1.5 times of AR1.4 in half fill and quarter fill conditions. This indicates that bending is more predominant in wider tanks of low aspect ratio.

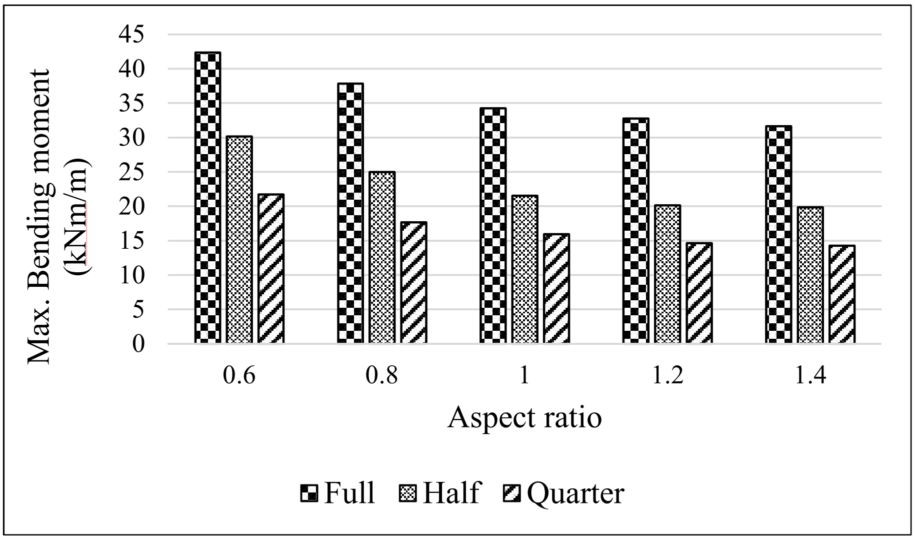



Fig. 6.10 Maximum bending moment of the tank due to Northridge earthquake

Bending moment decreases with reduction in water height for all tanks, and maximum bending moment of tank in half fill condition is 61.58 % to 71.2% of full fill condition, depending upon the aspect ratio of the tanks. Time history plot of bending moment at base of the tank with AR 1.2 in full fill condition is given in Fig. 6.11.

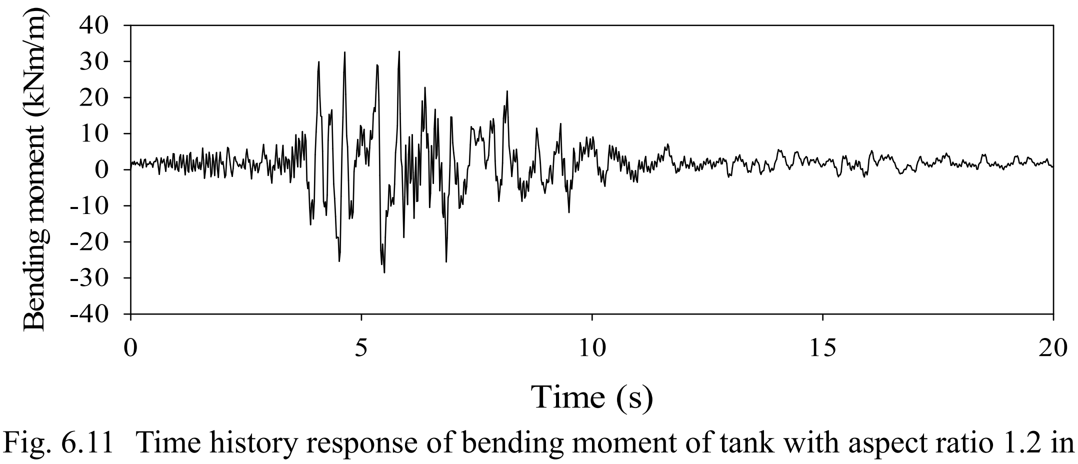



Fig. 6.11 Time history response of bending moment of tank with aspect ratio 1.2 in full fill condition

***Base shear***

The maximum response values are presented in Fig. 6.12. The maximum value of base shear in full fill condition increases with increase in aspect ratio. But the variation with aspect ratio is not significant under Northridge earthquake as it is less than 10%. However, in half fill and quarter fill conditions, tank with low aspect ratio 0.6 indicated base shear marginally higher than the tanks with higher aspect ratio.

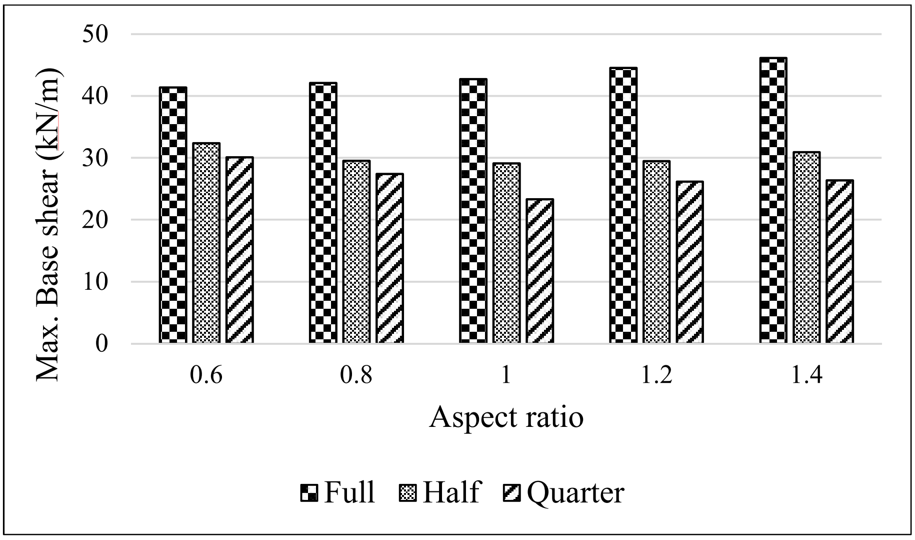



Fig. 6.12 Maximum base shear of the tank due to Northridge earthquake

There is reduction in maximum base shear with water height for all tanks under consideration as expected. The percentage reduction in base shear with reduction in water height from full fill to half fill condition are 21.8, 29.8, 31.8, 33.75 and 32.93 for tanks with aspect ratio 0.6, 0.8, 1.0, 1.2 and 1.4 respectively. The variation of base shear, at the location of maximum response, with time for tank with aspect ratio 1.2 is given in Fig. 6.13. The maximum base shear of 44.52 kN/m occurs at 5.02s whereas for the input motion, maximum acceleration is at 4.64s.

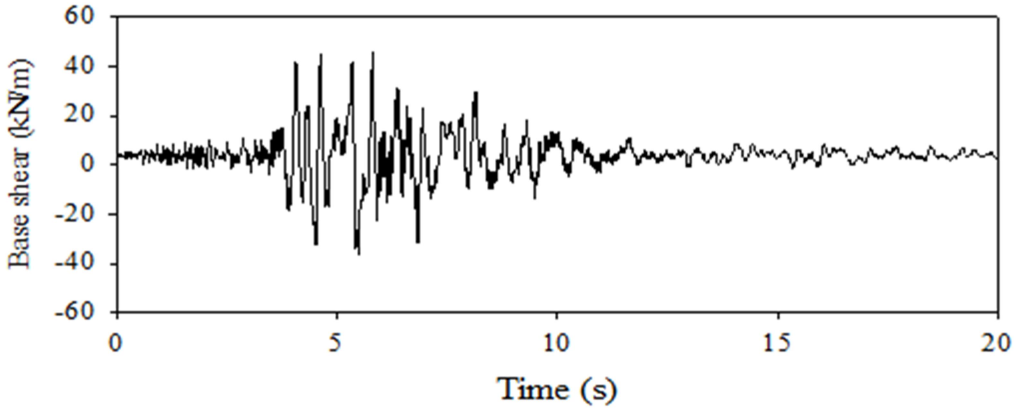



Fig. 6.13 Time history response of base shear of tank with aspect ratio 1.2 in full fill condition

### 6.4.2 Ratio of Response of Tank due to Seismic Loading of Northridge Earthquake to Hydrostatic Loading

Ratio of the seismic response to the static response is determined for evaluating and identifying the effect of earthquake over static loading. Effect of aspect ratio on maximum hoop force due to earthquake loading to hoop force due to static load ($\frac{H_{EQ}}{H_S}$) and maximum bending moment due to earthquake loading to bending moment due to static load ($\frac{M_{EQ}}{M_S}$) are presented in Fig 6.14 and Fig.6.15 respectively.

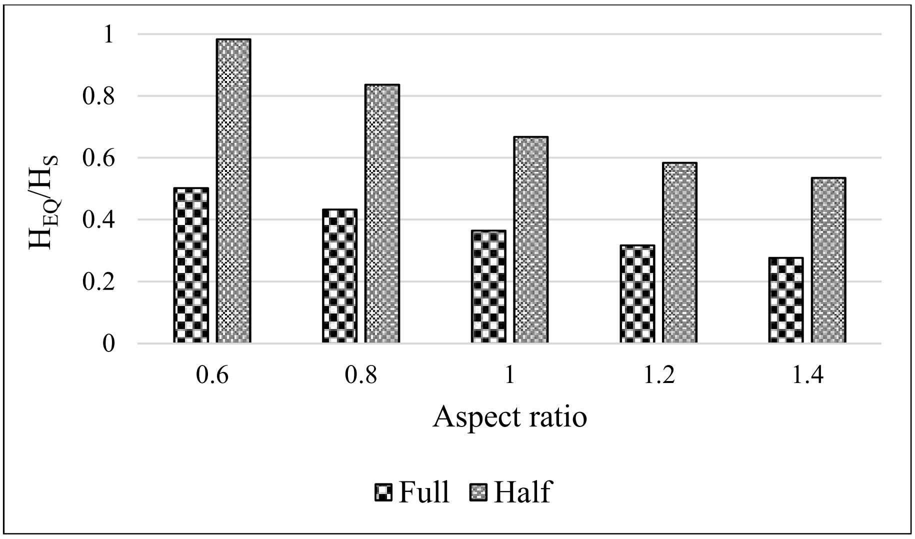



Fig. 6.14 Variation of $\frac{H_{EQ}}{H_S}$ with aspect ratio (Northridge earthquake)

The ratio of hoop force $\frac{H_{EQ}}{H_S}$ (Fig. 6.14) is observed to be decreased with increase in aspect ratio for both full fill and half fill conditions indicating that the seismic force has higher impact on hoop force developed in tank with low aspect ratio both in half fill and full fill conditions. The influence of earthquake is more pronounced in tank with half fill condition than full fill tank. In tank with half fill condition $\frac{H_{EQ}}{H_S}$ increases from 0.53 to 0.98 whereas in full fill tank the ratio increases 0.27 to 0.5 with higher value attributed to low aspect ratio. The higher influence of seismic force on tank with low water height is due to the reduction in mass participating in vibration which leads to increased frequency of vibration.

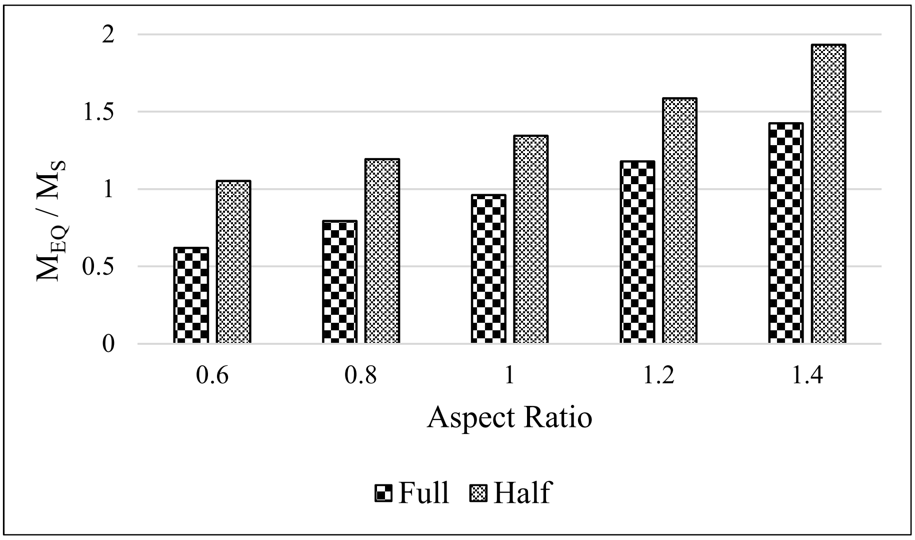



Fig. 6.15 Variation of $\frac{M_{EQ}}{M_S}$ with aspect ratio (Northridge Earthquake)

Fig. 6.15 indicates that the ratio of moment due to earthquake to static moment, $\frac{M_{EQ}}{M_S}$ increases with increase in aspect ratio, which is opposite to hoop behaviour with aspect ratio. Bending behaviour in half fill condition is influenced more than full fill condition. The ratio $\frac{M_{EQ}}{M_S}$ varies from 0.62 to 1.43 for full fill condition and 1.05 to

1.93 for half fill condition. Kianoush and Ghaemmaghami (2011) also observed reduced seismic response of rectangular tank in comparison to the static response under Northridge earthquake.

## 6.5 RESPONSE OF TANKS TO IMPERIAL VALLEY EARTHQUAKE OF MEDIUM FREQUENCY CONTENT

Response parameters of the tank such as radial displacement, hoop force, bending moment and base shear in the full fill, half fill and quarter fill conditions under Imperial Valley earthquake are noted.

### 6.5.1 Effect of Aspect Ratio and Water Height on Response Parameters of Tank due to Imperial Valley Earthquake

***Radial Displacement***

Maximum response displacement of the tanks in different fill conditions is presented in Fig. 6.16. Displacement in the radial direction of the tank in full and half fill conditions are the highest for tank with AR 0.6 whereas for tank in quarter fill condition, it is maximum for tank with AR 1.4. The maximum value of radial displacement of tank with AR 0.6 is 1.23 and 1.14 times of corresponding value of the tank with AR 1.4 for full fill and half fill conditions respectively. Displacement in tank with aspect ratio 1.4 is more than that of tank with aspect ratio 1.0 and 1.2. There is no direct relation of maximum radial displacement with aspect ratio as observed in the static case or tank subjected to earthquake of low frequency content.

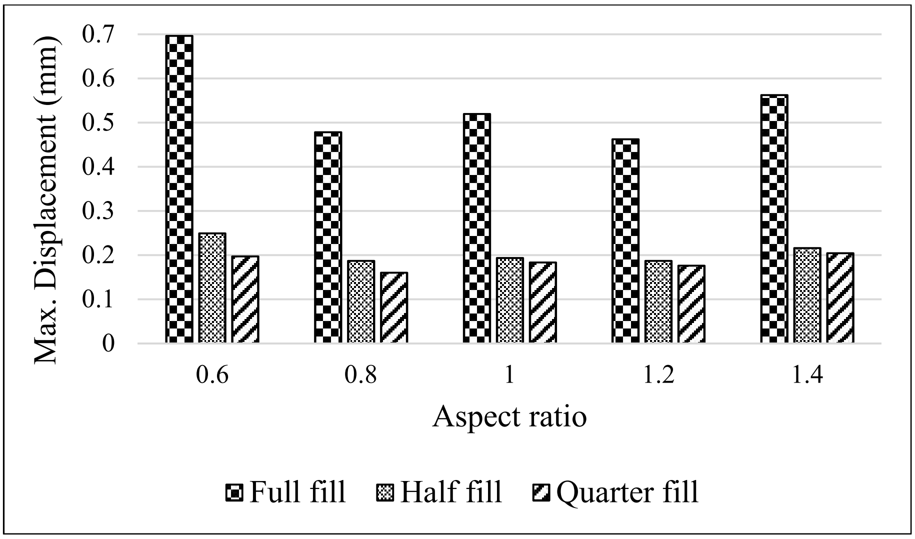



Fig. 6.16 Maximum radial displacement of the tank due to Imperial Valley earthquake

Under the excitation of Imperial Valley earthquake, tank in full fill condition have produced higher radial displacement than half fill and quarter fill conditions for tanks of all aspect ratios considered. Maximum response displacement of tank with AR 0.6 in half fill and quarter fill conditions are 0.36 and 0.29 times of the displacement in full fill condition whereas the corresponding value for tank with AR 1.4 is 0.39 and 0.36 respectively.

Time history response of radial displacement at top of wall of tank with AR 1.2 in full fill and half fill conditions is given in Fig 6.17. Maximum displacement of 0.46mm occurs at 4.84s for full fill and 0.19mm at 5.0s for half fill conditions under Imperial Valley earthquake. The response of the half fill tank is observed to be less than that in full fill condition as expected.

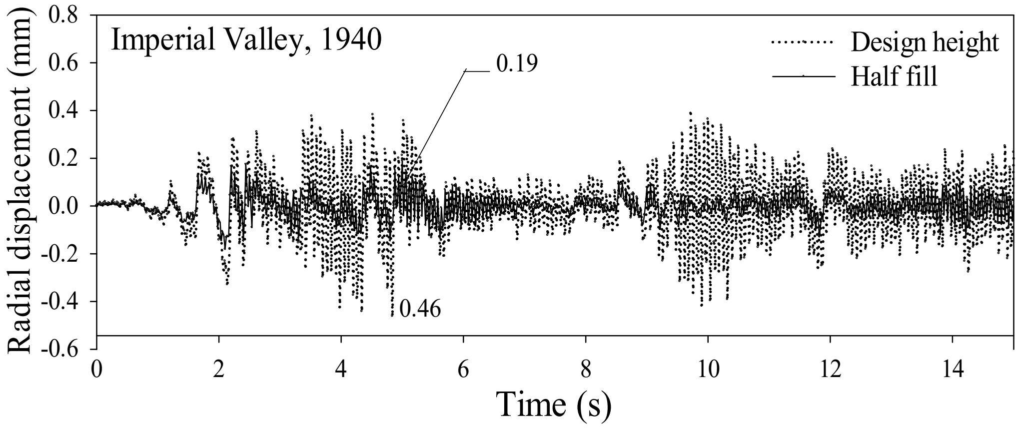



Fig. 6.17 Time history response of radial displacement of tank with aspect ratio 1.2 in full fill and half fill conditions under Imperial Valley earthquake

***Hoop Force***

As aspect ratio increases, hoop force decreases for all fill conditions as evident from Fig.6.18. Maximum hoop force of the tank with AR 0.6 in full fill, half fill and quarter fill conditions are respectively 3.89, 3.42 and 2.52 times of hoop force of the tank with AR 1.4.

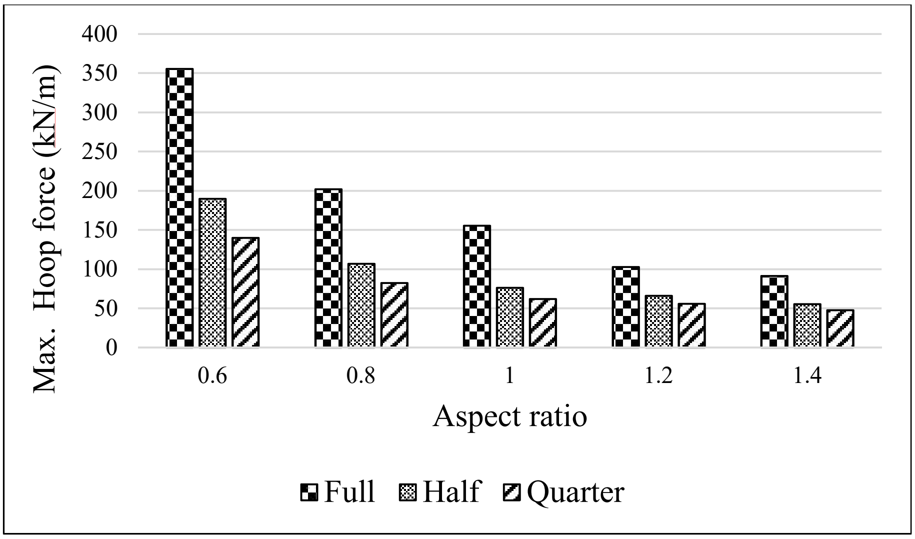



Fig. 6.18 Maximum hoop force of the tank due to Imperial Valley earthquake

Hoop response of the tank in full fill condition is higher than that of half fill condition. Also, response of tank in half fill condition is higher than that in quarter fill condition as expected. Maximum hoop force of tank in half fill condition is 0.49 to 0.64 times of the hoop force in full fill condition and for quarter fill condition, the maximum values are 0.33 to 0.54 times of the hoop force in full fill condition, depending upon the aspect ratio.

***Bending Moment***

Maximum bending moment occurring during seismic loading of medium frequency content Imperial Valley earthquake presented in Fig. 6.19 indicates no direct relation of bending moment with aspect ratio.  Bending moment in all the three fill conditions are the highest for tank with aspect ratio 0.6.  The maximum bending moment of the full fill tank with AR 0.8 to 1.4 varies from 70% to 74% of bending moment of the tank with AR 0.6 indicating that bending effect is more predominant in wider tank of aspect ratio 0.6 or less. Results also indicate that the influence of aspect ratio on the maximum bending moment of the tank with similar water fill condition is marginal for tanks with aspect ratio greater than 0.8.

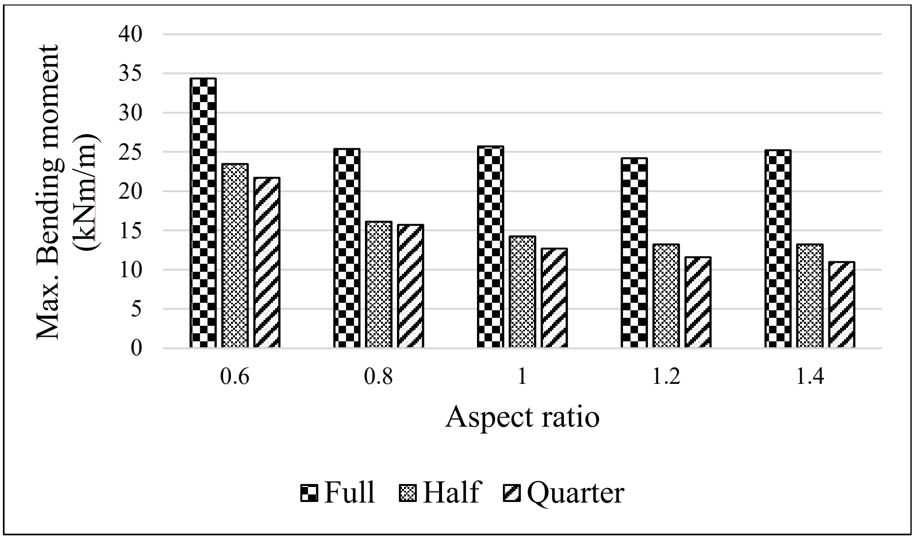



Fig. 6.19 Maximum bending moment of the tank due to Imperial Valley earthquake

Maximum bending moment of half fill tank is 0.52 to 0.68 times of the bending moment in full fill condition and for quarter fill condition, 0.43 to 0.63 times of the bending moment in full fill condition, depending upon the aspect ratio.

***Base Shear***

Base shear response also is not directly related to aspect ratio of the tank (Fig. 6.20) as the case of bending moment. For full fill condition, base shear is highest for tank with AR 1.4 whereas for half fill and quarter fill condition, peak response is for tank with aspect ratio 0.6. It can also be noticed that the variation of base shear with aspect ratio is not significant.

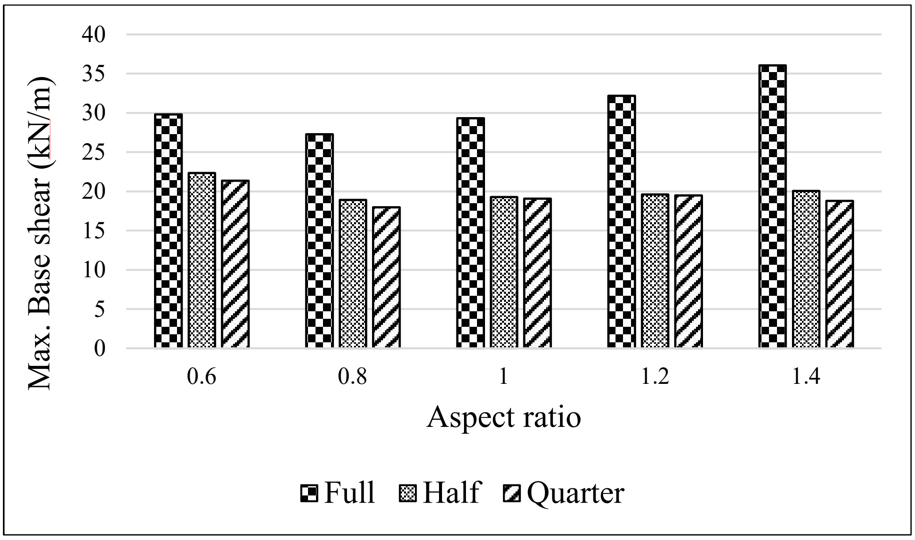



Fig. 6.20 Maximum base shear of the tank due to Imperial Valley earthquake

For all tanks considered, the base shear in full fill condition is more than that of half fill and quarter fill conditions, but the difference in base shear for half fill and quarter fill conditions are quite marginal, indicative of characteristics of earthquake loading. For tank with aspect ratio 0.6, the base shear in half fill condition is 25% less than that of full fill condition, whereas difference in base shear between half fill and quarter fill condition is only 4.4%.

### 6.5.2 Ratio of Response of Tank due to Seismic Loading of Imperial Valley Earthquake to Hydrostatic Loading

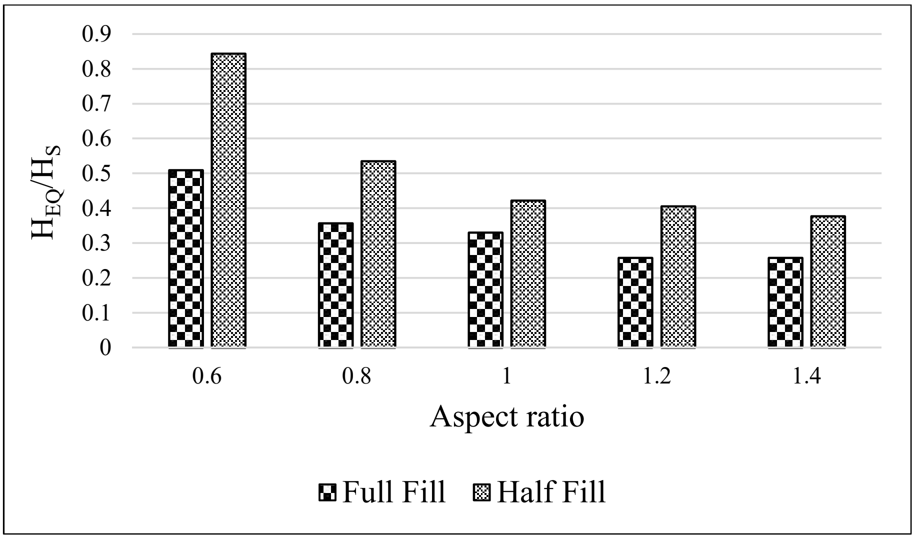



Fig.6.21 Variation of $\frac{H_{EQ}}{H_S}$ with aspect ratio (Imperial Valley earthquake)

From the plot of maximum hoop force due to Imperial Valley earthquake to hoop force due to static load, $\frac{H_{EQ}}{H_S}$ vs aspect ratio given in Fig.6.21, it is evident that as the aspect ratio increases, the effect of seismic force on the hoop force developed is decreased for both full fill and half fill conditions. This behaviour is similar to the response of tanks to seismic loading having low frequency content. The influence of earthquake is more pronounced in tank with half fill condition than full fill tank as in response to low frequency earthquake. For tank in half fill condition, the ratio of maximum hoop force due to seismic loading to static load decreases from 0.84 to 0.37 whereas in full fill tank the ratio decreases from 0.51 to 0.26 as aspect ratio increases from 0.6 to 1.4. This shows that the seismic event has major influence on hoop behaviour of tanks with low aspect ratio. Maximum hydrodynamic pressure reported by Moslemi and Kianoush (2012) on cylindrical tank wall (diameter -34 m; aspect ratio- 0.35) under El Centro earthquake is about 68% of maximum hydrostatic pressure.

The maximum bending moment due to seismic force to static load is given in Fig. 6.22 for tanks of various aspect ratios. For the tanks in full fill condition, $\frac{M_{EQ}}{M_S}$ increases with aspect ratio. This indicate that bending of tank with high aspect ratio (slender tank) is greatly affected by the seismic force.

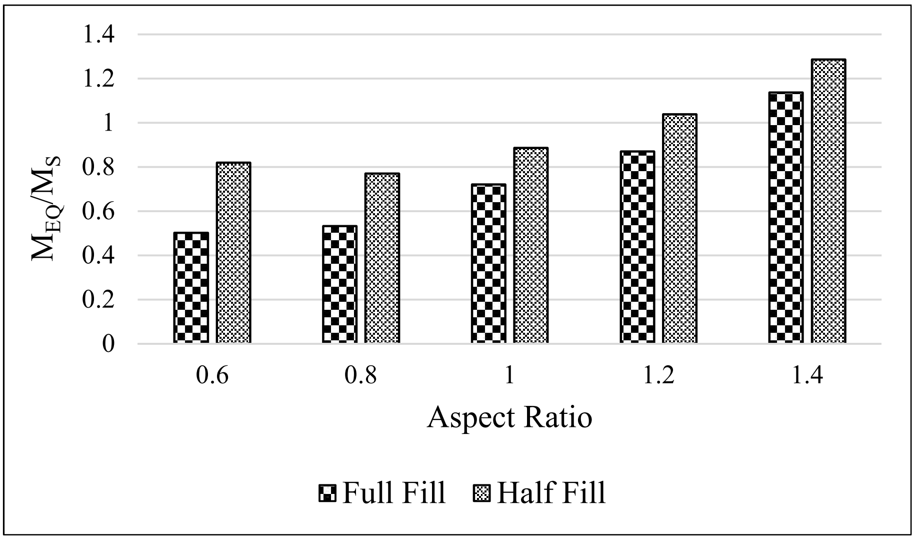



Fig. 6.22 Variation of $\frac{M_{EQ}}{M_S}$ with aspect ratio (Imperial Valley earthquake)

Though aspect ratio can not be considered as prime factor that determines the seismic response of water tanks under earthquake of medium frequency content, the response displacement, hoop force and bending moment are the highest for tank having lowest aspect ratio. Hence, for tanks having aspect ratio less than 1.0, characteristics of the tank can be considered as the governing factor that controls the seismic response of the water tanks under medium frequency earthquake. For tanks with higher aspect ratio, the earthquake characteristics influences more than the aspect ratio of the tank. As in case of low frequency earthquake, base shear is not found to be direct function of aspect ratio, also its variation with aspect ratio is less in comparison to the other response parameters.

## 6.6 RESPONSE OF TANKS TO KOYNA EARTHQUAKE OF HIGH FREQUENCY CONTENT

The five tanks in three different water fill conditions are subjected to the high frequency content Koyna earthquake for 10.76s. The response parameters are analysed and influence of aspect ratio and water height on seismic response of the tanks under Koyna earthquake of high frequency content are identified.

### 6.6.1 Effect of Aspect Ratio and Water Height on Response Parameters of Tank due to Koyna Earthquake

***Radial Displacement***

Fig. 6.23 indicates no direct relation of radial displacement with aspect ratio under Koyna earthquake. Displacement in the radial direction is highest for the tank with AR 1.2 in full fill condition and for AR 1.4 for half fill and quarter fill conditions. Maximum displacement experienced by the wider tank is the least among the five tanks analysed. The maximum radial displacement of tank with AR 1.2 in full fill condition is 2.6 times of the corresponding value of the tank with AR 0.6.

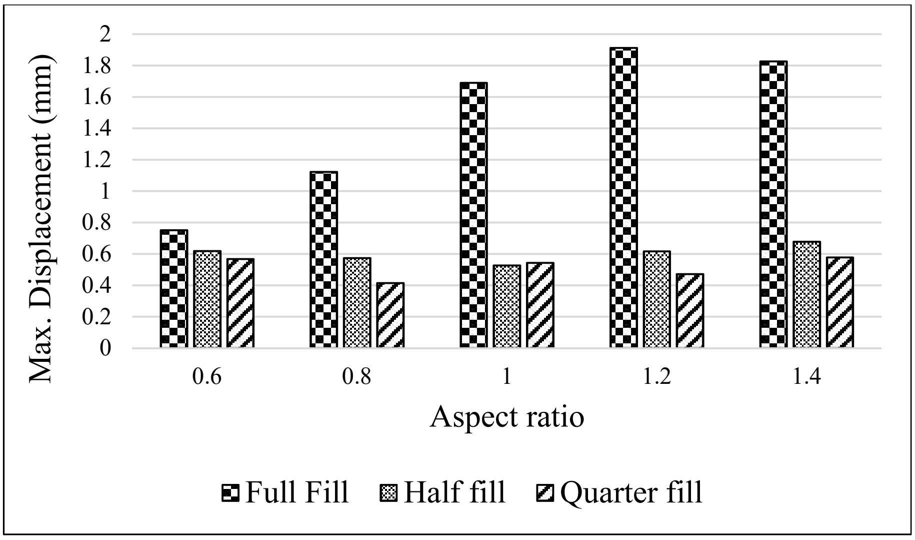



Fig. 6.23 Maximum radial displacement of the tank due to Koyna earthquake

In general, response displacement is observed to be decreased with decrease in water height under Koyna earthquake except in tank with AR1.0 where maximum response of quarter fill tank is marginally higher than half fill tank. Radial displacement of tank with AR 0.6 in half fill and quarter fill conditions are 0.83 and 0.76 times of the displacement full fill condition whereas for tank with AR 1.4, the corresponding values are 0.39 and 0.32.

Time history of displacement of the tank having AR 1.2 in full fill and half fill conditions under Koyna earthquake, indicating maximum responses are given in Fig. 6.24. Maximum displacement of 1.91 mm in full fill tank occurs at 4.78s and 0.62 mm in half fill tank at 4.7s. From the figure, it can be seen that response of half fill tank is much less than the full fill condition.

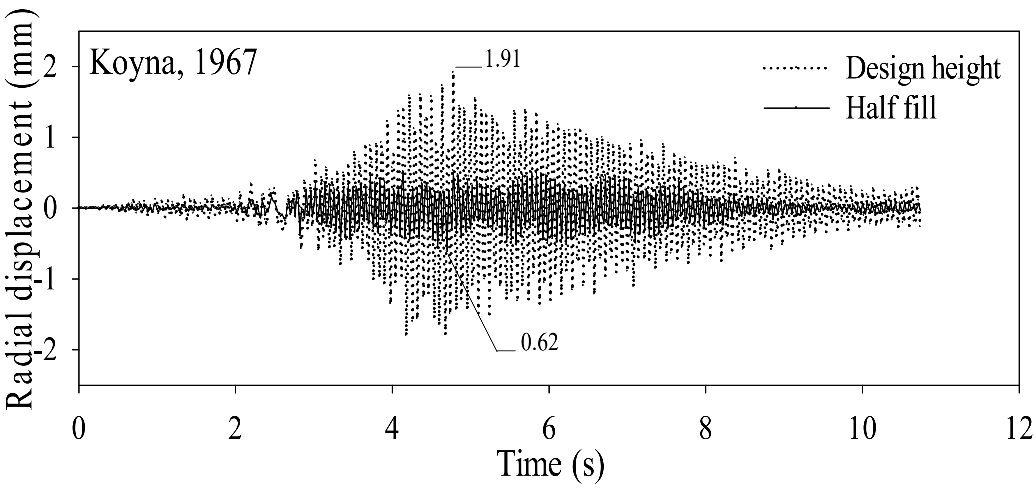



Fig. 6.24 Time history response of radial displacement of tank with aspect ratio 1.2 in full fill and half fill conditions under Koyna earthquake

***Hoop force***

Fig. 6.25 indicates that the behaviour of maximum hoop response to high frequency content Koyna earthquake is different from those observed in medium or low frequency content earthquakes. For the full fill tank, peak hoop force is for AR1.0.

But for half fill and quarter fill condition, peak occurs for AR0.6. Maximum hoop force values of tank with AR 0.8, 1.0, 1.2, and 1.4 in full fill condition are 1.22, 1.32, 1.10 and 0.78 times of corresponding value of AR0.6F (Fig. 6.25).

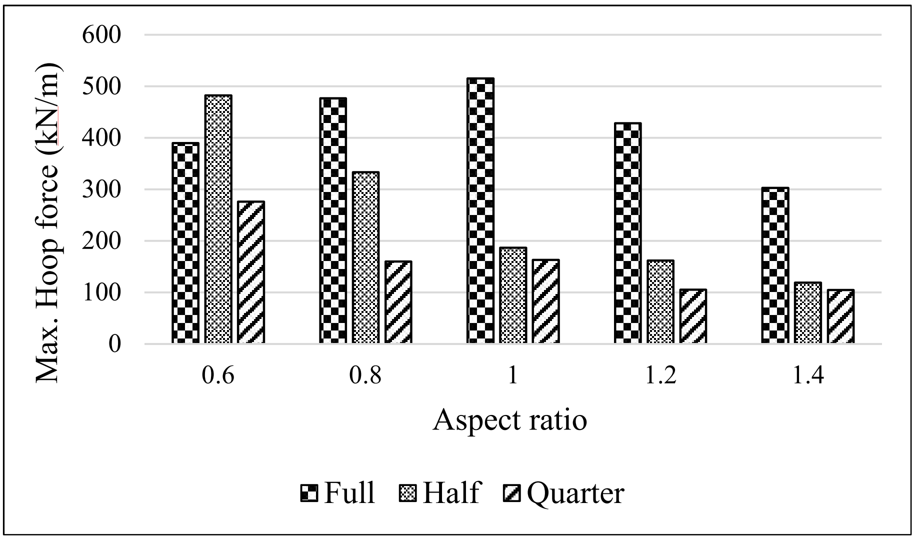



Fig. 6.25 Maximum hoop force of the tank due to Koyna earthquake

Maximum hoop force in the tank with AR 0.6 in half fill condition is more than that of full fill condition when subjected to Koyna earthquake. Maximum hoop force of AR0.6H is 24% more than that AR0.6F, may be due to rocking motion of tank wall in half fill condition under Koyna earthquake.

For all other tanks, the maximum response in full fill condition is higher than that for half fill condition. This indicates that maximum seismic response of tank cannot always be expected for tanks filled for design water height, as in the case of response to low and medium frequency content earthquakes as well as to static loading.

***Bending moment***

Bending moment variation with aspect ratio (Fig. 6.26) cannot be directly correlated, similar to other response such as displacement and hoop force. Upon Koyna earthquake, bending moment is the highest for tank with AR1.2 in full fill condition and with aspect ratio 0.6 for half fill and quarter fill conditions. Maximum bending moment of AR1.2F is 2.1 times of maximum of AR0.6F. This indicates that bending effect is more predominant in tank with aspect ratio greater than 1.0 which is opposite to the observation in tank subjected to low and medium frequency content earthquakes.

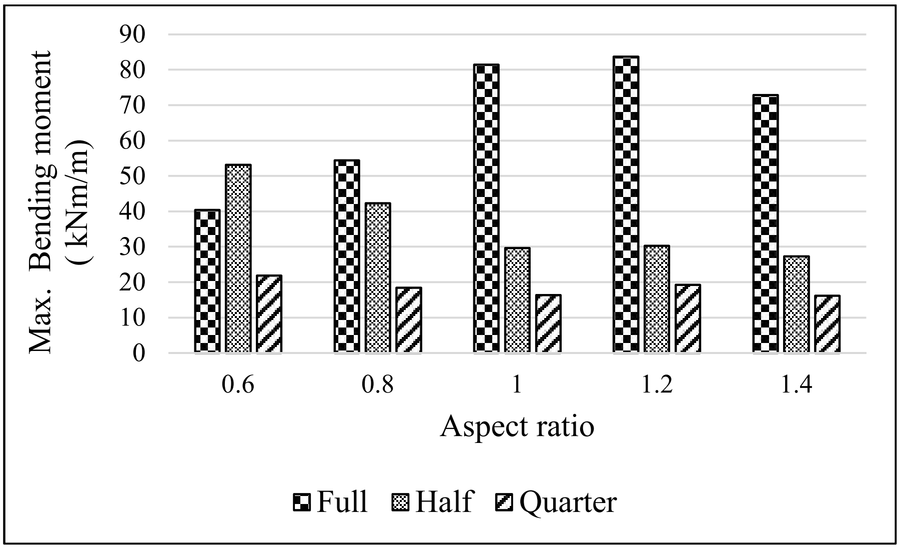



Fig. 6.26 Maximum bending moment of the tank due to Koyna earthquake

Similar to hoop response of the tank, bending moment of tanks with all aspect ratios except 0.6, show reduction in magnitude with decrease in water height. But for AR0.6H response is 32% more than AR0.6F. This is due to high influence of frequency content of earthquake on seismic behaviour of the tank, that being altered by the water fill condition too.

***Base shear***

The maximum response values of base shear are depicted in (Fig.6.27). Contrary to response to low and medium frequency content earthquakes, variation in maximum base shear with aspect ratio is significant for tanks under earthquake of high frequency content. But, the variation cannot be directly related to aspect ratio for all fill conditions.

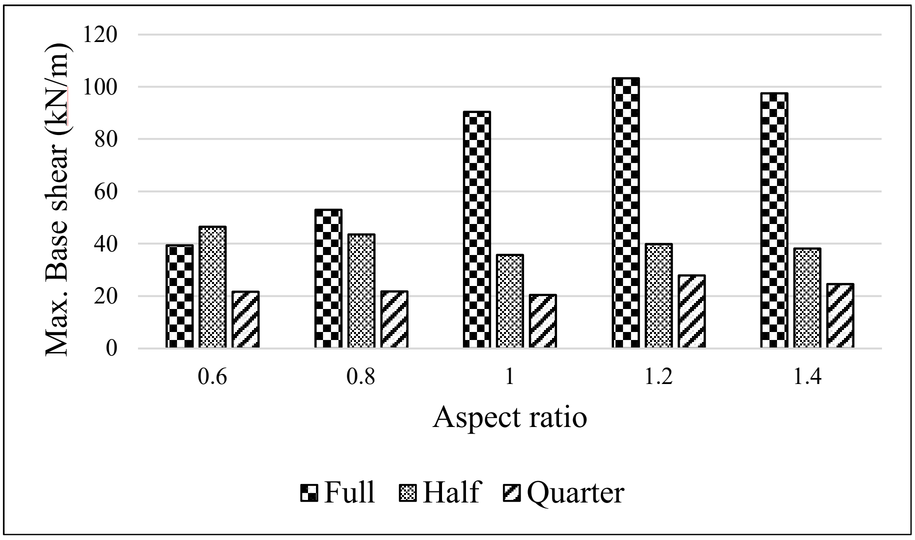
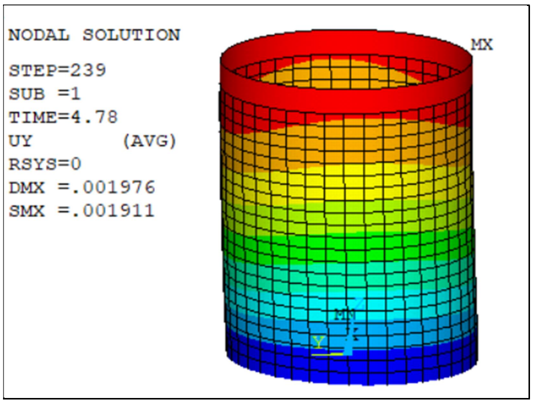


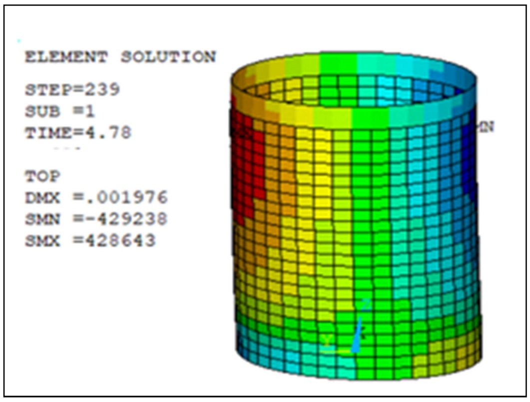

Fig. 6.27 Maximum base shear of the tank due to Koyna earthquake

Maximum base shear of AR0.6F is less than AR0.6H demonstrating the necessity to include the effect of water fill condition on seismic analysis of water tanks. Maximum base shear of AR0.6H 18% more than that AR0.6F. For all other tanks, base shear decreases with decrease in water height.

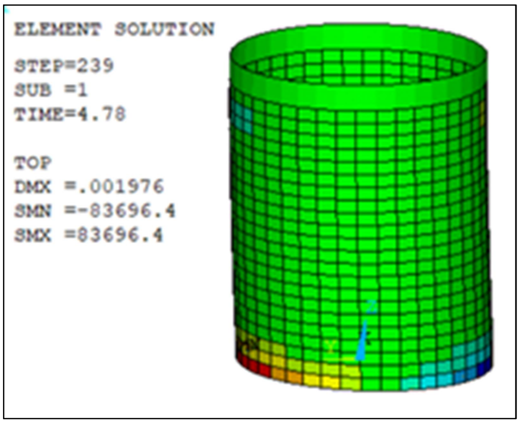
Response contours of the tank gives the response of the entire tank at any particular instant of time to seismic loading. This will provide information regarding distribution of the response parameter over the tank shell. Response contours of the tank with AR1.2, at the instant of maximum response of particular parameter such as radial displacement, hoop force, bending moment and base shear are given in Fig.6.28 and Fig.6.29 for full fill and half fill conditions respectively.



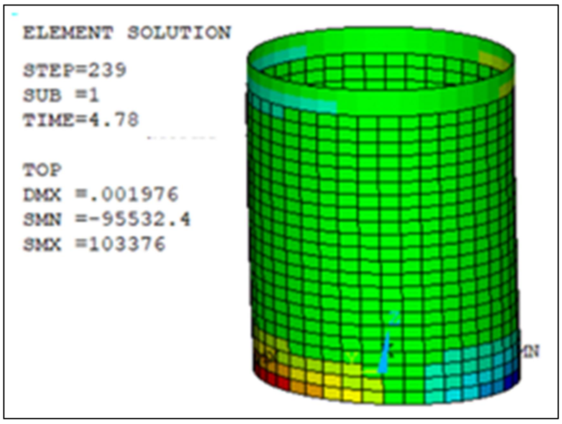

(a) Radial displacement




(b) Hoop force

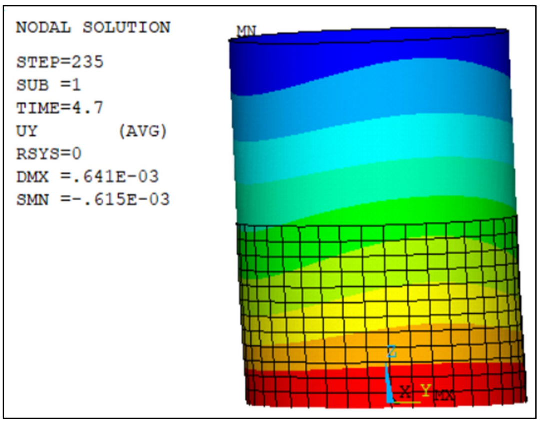



(c) Bending moment

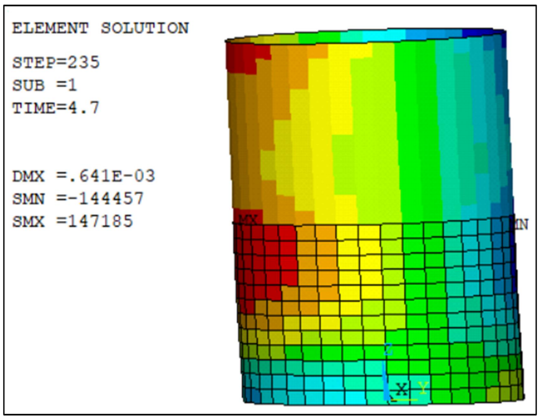



(d) Base shear

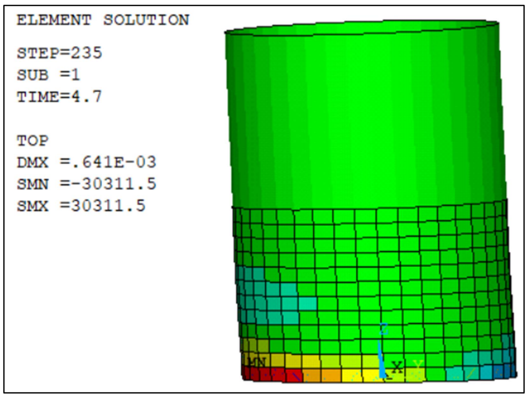
Fig.6.28 Response contours of tank with aspect ratio 1.2 in full fill condition (Koyna Earthquake)



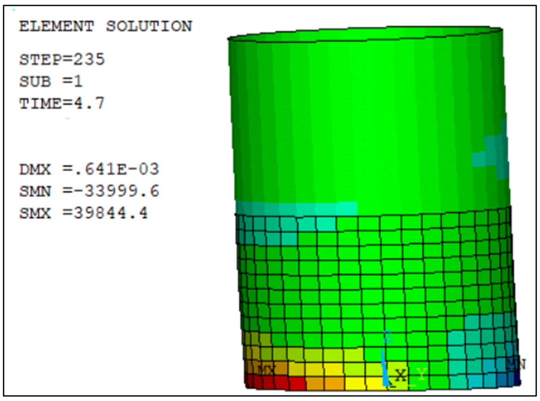

(a) Radial displacement




(b) Hoop force




(c) Bending moment




(d) Base shear

Fig. 6.29 Response contours of tank with aspect ratio 1.2 in half fill condition (Koyna Earthquake)

### 6.6.2 Ratio of Response of Tank due to Seismic Loading of Koyna Earthquake to Hydrostatic Loading

The seismic responses under Koyna earthquake is compared with the response due to hydrostatic loading to have an insight into the effect of seismic forces on the tank response. The plot of $\frac{H_{EQ}}{H_S}$ vs aspect ratio, given in Fig.6.30 also leads to the conclusion that, unlike low and medium frequency content earthquakes, the tank response under high frequency earthquake cannot directly correlated to aspect ratio. For full fill condition, the ratio varies between 0.56 to 1.1, the lowest corresponds to AR 0.6 and highest for AR1.0. Seismic effects are predominant for tank in half fill condition as the ratio $\frac{H_{EQ}}{H_S}$ varies between 0.81 to 2.15. For tank with aspect ratio 0.6, the seismic hoop force is more than two times the static response.

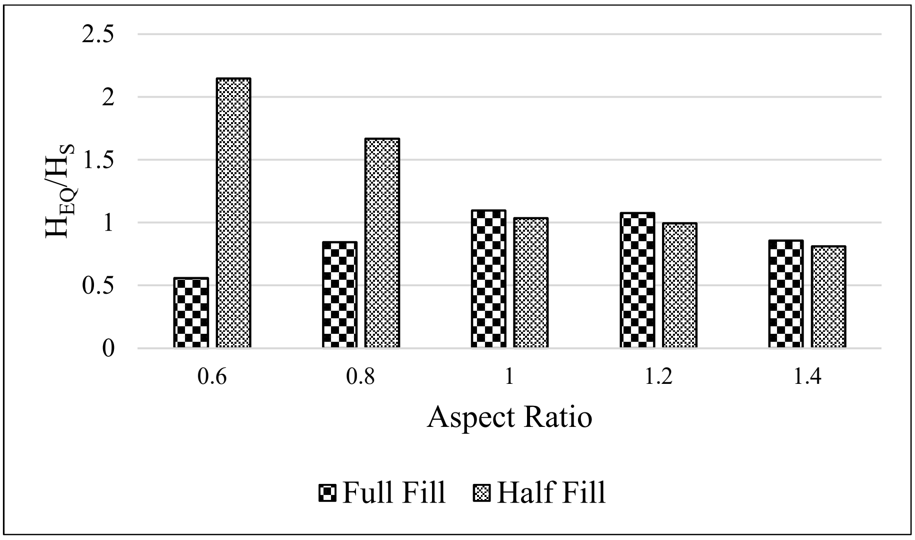



Fig. 6.30 Variation of $\frac{H_{EQ}}{H_S}$ with aspect ratio (Koyna earthquake)

For the tanks in full fill condition, $\frac{M_{EQ}}{M_S}$ increases with increase in aspect ratio whereas for half fill tanks such an observation can not be made (Fig.6.31). For wider tanks, seismic effects are predominant in half fill conditions whereas for tanks with low aspect ratio, full fill tank responses are more effected. $\frac{M_{EQ}}{M_S}$ varies from 0.59 to 3.29 in full fill condition, higher value corresponds to tank with low aspect ratio. Hence it can be concluded that bending behavior of tank with aspect ratio ≥1.0 is more influenced by earthquake of high frequency content than tanks low aspect ratio. Variation of $\frac{M_{EQ}}{M_S}$ for half fill condition is between 0.85 and 2.66.

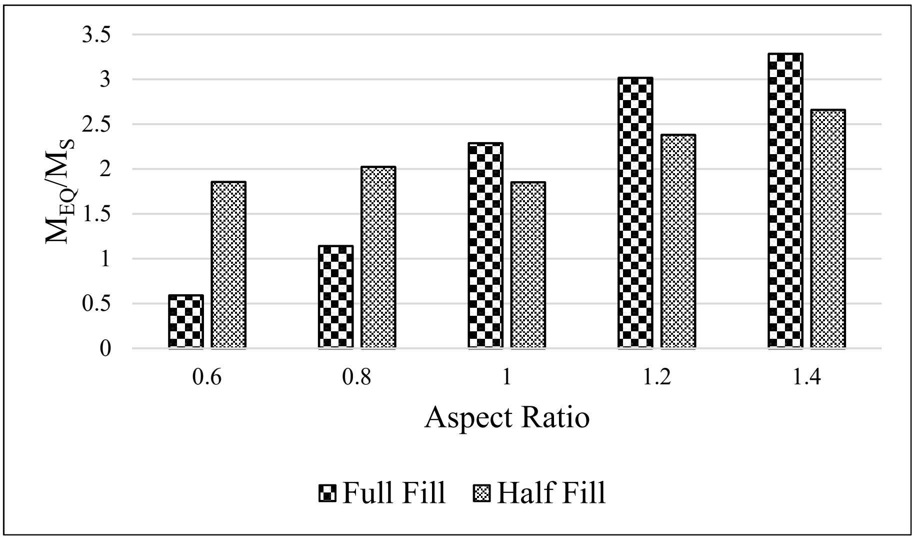



Fig. 6.31 Variation of $\frac{M_{EQ}}{M_S}$ with aspect ratio (Koyna earthquake)

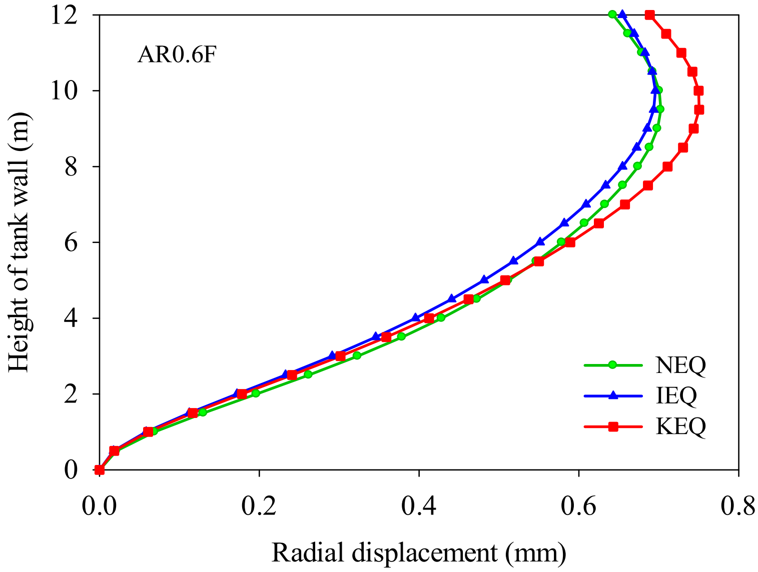
## 6.7 INFLUENCE OF EARTHQUAKE CHARACTERISTICS ON RESPONSES

From the above discussions on 6.4, 6.5 and 6.6, it is clear that seismic response of the water tank is highly depended on the frequency content of the earthquake. To clearly recognize how the characteristics of earthquake influences the seismic response of the tanks, its effect on seismic response parameters are further studied.

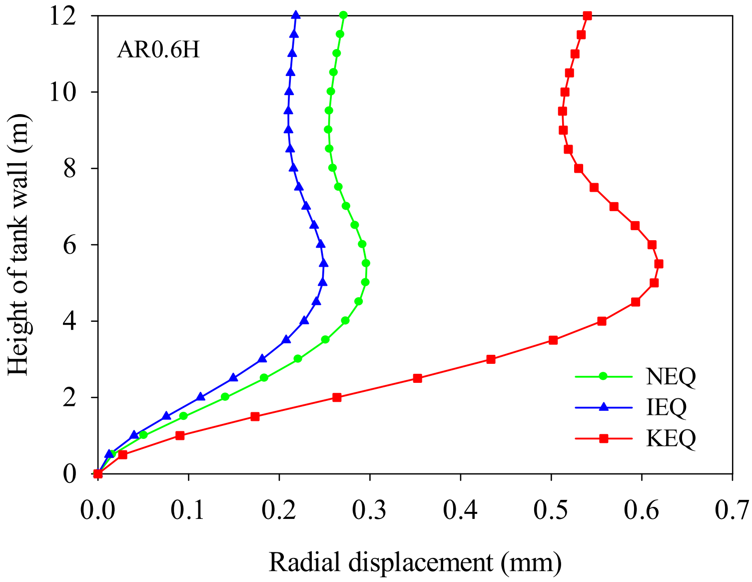
***Radial Displacement***

For all the five tanks and for all water fill conditions, response displacement is more for Koyna earthquake of high frequency content than low and medium frequency content earthquakes. The highest radial displacement of 1.91mm in AR1.2F under Koyna earthquake is observed at the top of the tank wall. The displacement in response to the three earthquakes in design water height condition is higher than that of half fill and quarter fill condition for all tanks under consideration.

To identify the role of frequency content of earthquake on seismic response of tank, the variation of radial displacement along the height of the tank wall, at the event of maximum response of the tank, under three earthquakes are plotted and is given in Fig.6.32 for AR 0.6 and Fig.6.33 for AR 1.0.

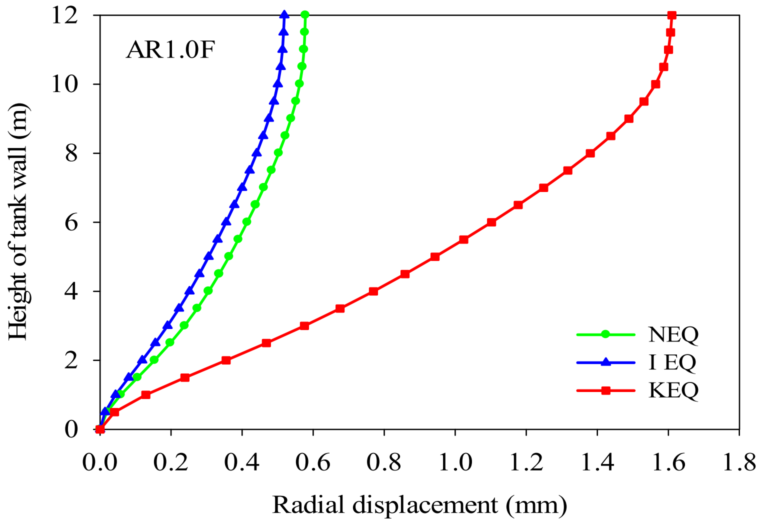



(a) Full fill

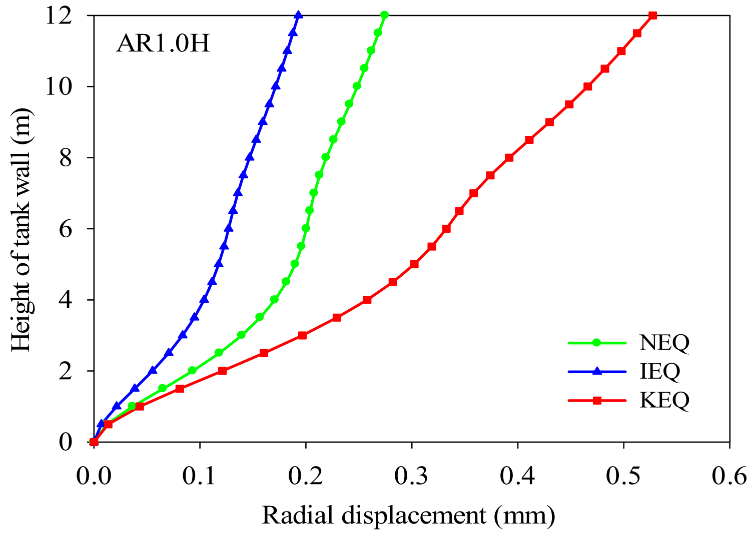



(b) Half fill

Fig. 6.32 Radial displacement along height of tank wall due to seismic loading (AR0.6)




(a) Full fill



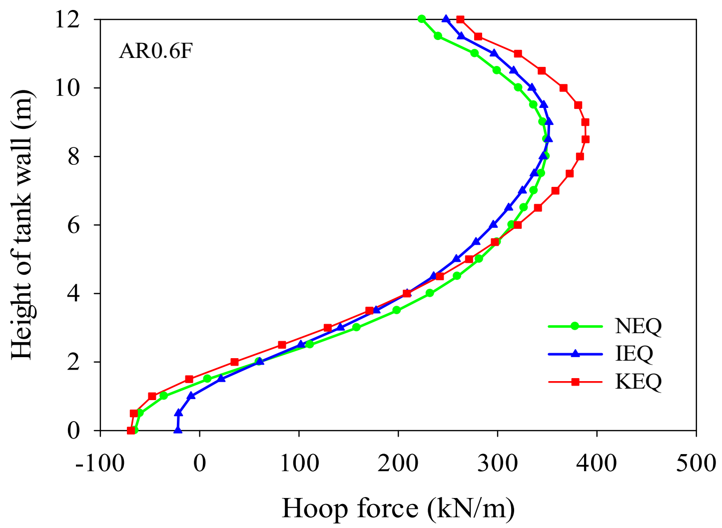

(b) Half fill

Fig. 6.33 Radial displacement along height of tank wall due to seismic loading (AR1.0)

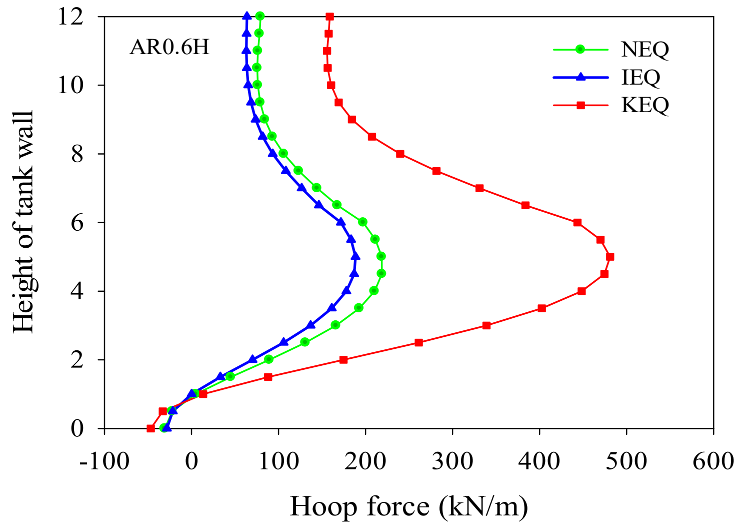
It can be noticed from Fig. 6.32 and Fig.6.33 that response to Koyna earthquake having high frequency content is more than that of Northridge and Imperial Valley earthquakes. Under the three earthquakes, for tank with AR 1.0 maximum response is at top of tank wall for both full fill and half fill conditions. Displacement is maximum at a height of 9.5m from bottom of tank for AR 0.6 in full fill and at 5.5 from bottom in half fill conditions. It has been noticed that the location of maximum displacement depends on the aspect ratio of the tank and its water fill condition. Comparison of Fig. 6.32 (a) and Fig. 6.33(b) indicate that tanks with low aspect ratio in full fill condition, responses are less influenced by the frequency content of earthquake as the mass vibrating system is huge. Slightly higher response to Northridge earthquake (low frequency content) than Imperial Valley earthquake (medium frequency content) may be attributed to the higher PGA of Northridge earthquake.

***Hoop Force***

Variation of hoop force along height of tank wall for full fill and half fill conditions are depicted in Fig. 6.34 for tank with aspect ratio 0.6 and in Fig. 6.35 for aspect ratio 1.0.

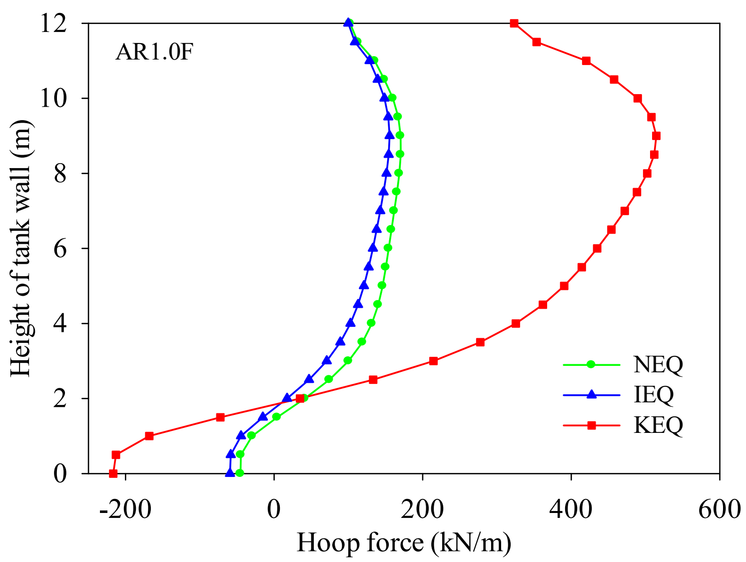



(a) Full fill

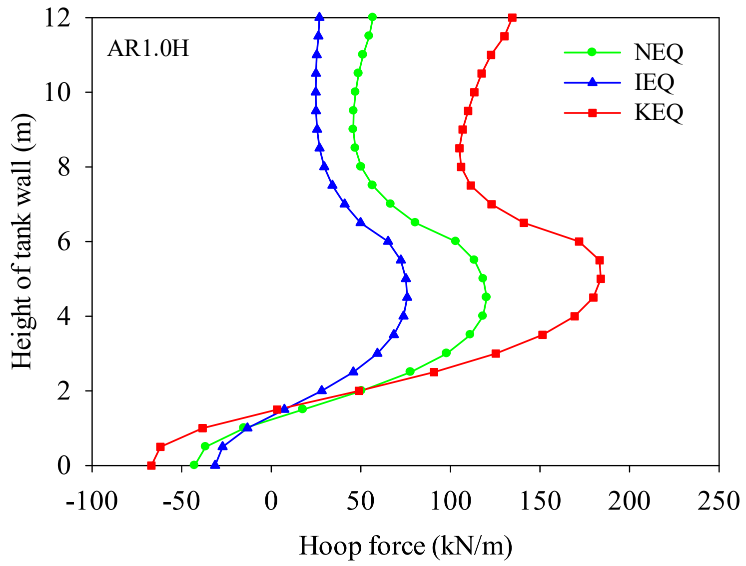



(b) Half fill

Fig. 6.34 Hoop force along height of tank wall due to seismic loading (AR0.6)




(a) Full fill



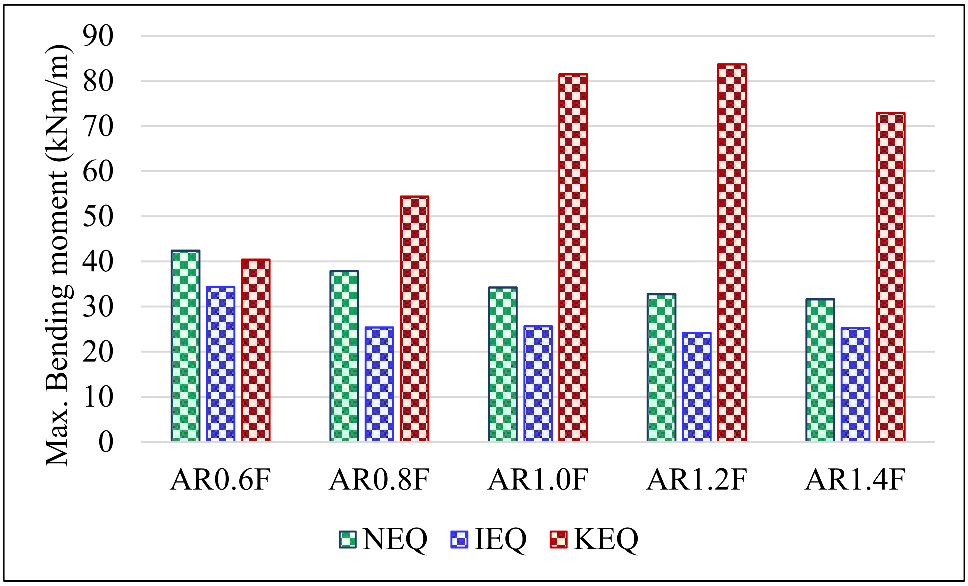

(b) Half fill

Fig. 6.35 Hoop force along height of tank wall due to seismic loading (AR1.0)

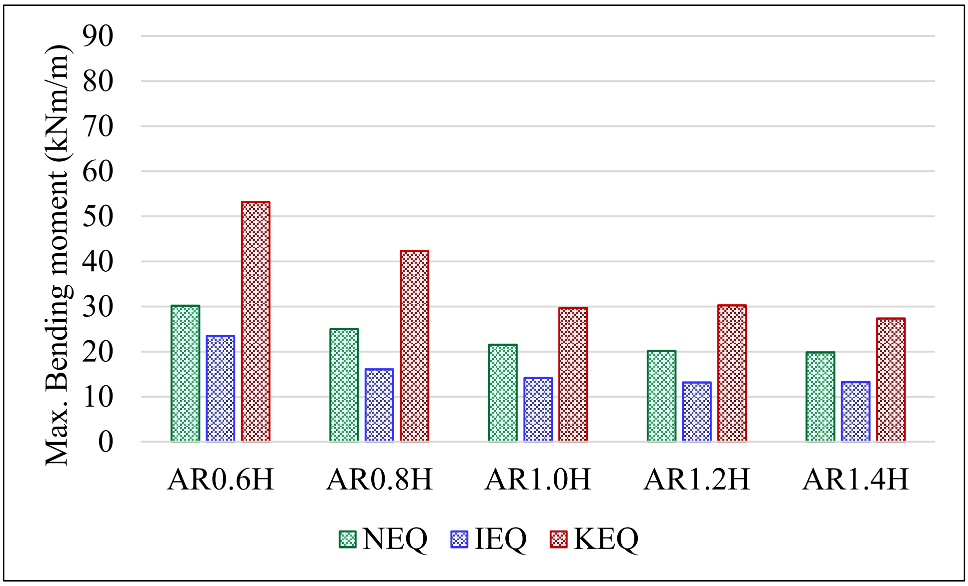
Hoop force developed in response to Koyna earthquake with high frequency content is more than low frequency and medium frequency content earthquakes. The maximum value of hoop force in full tank has been noticed at a height of 8.5 to 9 m from the bottom and 4.5 to 5m from bottom for half fill condition for all earthquakes. Hoop force of AR0.6 in full fill condition under high frequency content earthquake is only 11% and 9% more than those due to low and medium frequency content earthquakes respectively, whereas AR0.6 in half fill condition under Koyna earthquake is 2.17 and 2.52 times of response due to Northridge and Imperial Valley earthquakes. For tank with AR1.0 in full fill condition, maximum hoop force due to Koyna earthquake of high frequency content is 3.02 and 3.31 times low and medium frequency content earthquakes respectively. This also indicate that tanks with low aspect ratio in full fill condition, responses are less influenced by the frequency content of earthquake. It may also be noted that tank wall experiences compressive force at bottom 1.5m height of tank in all cases of seismic loading,

***Bending Moment***

For all tanks under consideration, maximum bending moment in the tank is observed at the bottom of the tank as expected. Fig. 6.36 infers that maximum bending moment occurs in response to Koyna earthquake except for tank with AR0.6 in full fill condition. However for AR 0.6 in full fill condition, Northridge earthquake response is found to be higher than that of Imperial Valley and Koyna earthquakes. For all tanks in any fill condition, response to low frequency content Northridge earthquake is more than to medium frequency content Imperial Valley earthquake, attributable to high PGA of Northridge earthquake.




(a) Full fill




(b) Half fill

Fig. 6.36 Maximum bending moment of tank with different aspect ratio due to transient loading

Maximum bending moment of AR0.6 in full fill condition under high frequency earthquake is 0.95 and 1.18 times of maximum bending moment due to low and medium frequency earthquakes, but for AR0.6H these values are 1.76 and 2.26 indicating higher impact of frequency of earthquakes on tanks in half fill condition.

For tank with AR1.4 in full fill condition, maximum bending moment due to high frequency Koyna earthquake is 2.3 and 2.88 times of low and medium frequency earthquakes. This also indicate the higher influence of frequency content of earthquake on bending behaviour of tank with high aspect ratio.

## 6.8 SUMMARY

By performing the time history analysis on ground supported cylindrical water tanks in three water fill conditions, the effect of aspect ratio, water fill condition and frequency content of earthquake on seismic response of the tanks have been studied. The static analysis performed prior to time history analysis helped to identify the response of tanks to hydrostatic loading and how the behaviour differs under seismic loading. The response parameters examined are radial displacement, hoop force, bending moment and base shear. Under low frequency earthquakes, the aspect ratio has direct relation to the seismic response of water tanks. Radial displacement, hoop force, and bending moment of the tank due to hydrodynamic effects are found to be decreased with increase in aspect ratio of the tank as for hydrostatic loading. Tanks with low aspect ratio in full fill condition, responses are less influenced by the frequency content of earthquake but by the geometry and water fill condition. But for tanks under earthquake of high frequency content, seismic behaviour is dominated by characteristics of earthquake rather than geometric features and fill condition of the tank. Contribution of aspect ratio towards base shear is found to be less. The responses of ground supported cylindrical water tanks to seismic loadings depend not only on the frequency content of earthquake but also on aspect ratio of the tank and its water fill condition.

The effect of soil-structure interaction is not taken in to account here, as only the response on tanks having idealized case of fixity at bottom of tank wall to seismic loading is studied. But the response of tank is really influenced by the properties of the soil on which it is rested. The roles of soil-structure interaction on seismic behaviour of tanks are dealt in Chapter 7.

# CHAPTER 7

# FLUID - STRUCTURE- SOIL INTERACTION ON SEISMIC BEHAVIOUR OF CYLINDRICAL WATER TANKS

## 7.1 GENERAL

Dynamic characteristics and seismic response of any structure depends on the characteristics of soil on which it is founded and hence the effect due to soil-structure interaction. Seismic analysis of the tanks considering the combined fluid-structure interaction (FSI) and soil-structure interaction (SSI) effects are required for proper understanding of the responses of tank resting on soil of varying properties. This chapter evaluates the responses of cylindrical water tanks resting on four different soil types which are subjected to seismic loads of past three earthquakes.

### 7.1.1 Description of the Tank and Soil System under Consideration

Ground supported concrete cylindrical tank having aspect ratio 1.0 is considered to study the effect of SSI on the seismic behaviour of water tanks. The geometric features of the tank are: inner diameter – 12m, height of tank wall – 12m and thickness of tank wall – 0.5m. The modelling of the tank and soil is performed in accordance with details given in Chapter 4. The past three earthquakes, for seismic loading are same as those considered in Chapter 6 for the time history analysis of tanks without consideration of SSI. The seismic response of the tank under two water fill conditions, full fill (design height of 11m) and half fill (water height of 6m) are investigated. The tank is provided with a base slab of 1m thick having 1 m projection around the tank wall. The material properties of concrete and water are same as given in Table 5.3. The base slab is set upto solid soil model and hence the embedment effect of tank foundation is not taken into consideration as done by other researchers (Livaoglu and Dogangun, 2007).

Four different soil conditions mentioned in literature; hard rock, rock, very dense soil (soft rock) and stiff soil (lateritic soil), (Livaoglu, 2008; Kianoush and Ghaemmaghami, 2011) are considered for soil-structure interaction analysisand its properties are given in Table 7.1. Time history analysis of tanks resting on these soil types are performed and compared with the response of tank having rigid base to study the significance of SSI on seismic behaviour of water tanks. The width and the thickness of the soil medium should be such that sufficiently large soil mass is available to include the SSI effects (Clough and Penzenin, 1993; Potts and Zdravkovic, 1999; Livaoglu, 2007). The pressure isobars based on Boussinesq equation for a footing gives that pressure bulbs are within a width of 1.5 times width of foundation from the centre of footing (Bowles, 2001). In the opinion of Bhatia (2008), the length, breadth and depth of soil domain modelled below the foundation should be upto three to five times the width of foundation (Al-Nakdy et al., 2014). Livagolu and Dogangun (2007) adopted the soil model with width 2.22 times and depth 1.11 times diameter of mat foundation in the seismic analysis of over headed water tank. Chinmayi and Jayalekshmi (2013, 2014), has chosen the width and the thickness of the soil medium as 1.5 times and 2 times the least width of the raft foundation, beyond which it shows a negligible influence on the settlement and the contact pressure, as reported by Maharaj et al. (2004). Thangaraj and Ilamparuthi (2010) and Kianoush and Ghaemmaghami (2011) have performed parametric study to fix the dimensions of the soil block such that displacements of nodes on lateral boundaries are almost zero. In the present study, the width and depth of soil medium considered are 5 times and 3 times the diameter of base slab respectively such that the displacement along the boundaries are almost zero. The near field is modelled using finite elements, and the far field is treated by viscous boundary elements to model it as non-reflecting boundaries. All translations were restricted at the bottom of soil block. Details of soil modelling has been dealt in Chapter 4.The finite element models of tank with rigid base and tank resting on soil are given in Fig. 4.4.

Table 7.1 Properties of soil

| | S1 | S2 | S3 | S4 |
|---|---|---|---|---|
| | Hard rock | Rock | Very Dense soil (soft rock) | Stiff soil (lateritic soil) |
| Density (kg/m$^3$) | 2000 | 2000 | 1900 | 1900 |
| Modulus of Elasticity (MPa) | 7000 | 2000 | 500 | 150 |
| Poisson's ratio | 0.3 | 0.3 | 0.35 | 0.35 |
| Bulk Modulus (MPa) | 9423.08 | 2692.31 | 673.08 | 201.92 |
| Shear wave velocity (m/s) | 1149.10 | 614.25 | 309.22 | 169.36 |
| Dilatational wave velocity (m/s) | 2149.89 | 1149.16 | 643.68 | 352.56 |

### 7.2 FREE VIBRATION ANALYSIS OF FLUID – TANK – SOIL SYSTEM

Prior to perform the time history analysis, free vibration analysis is carried out on tank with rigid base and tank resting on different soil conditions. The natural frequencies and the modal responses are obtained from the modal analysis. Since the SSI has negligible effect on convective response (Veletsos and Tang. 1990; Livagolu, 2008; Kianoush and Ghaemmaghami, 2011), the fundamental impulsive modes of the tanks with largest participation factor in horizontal direction are identified and are tabulated in Table 7.2.

Table 7.2 Fundamental impulsive frequency of the tanks on different soil

| Tank resting condition | Natural frequency (Hz) | | % increase in frequency in half fill condition |
|---|---|---|---|
| | Full fill | Half fill | |
| Tank with rigid base | 14.39 | 20.24 | 40.65 |
| Tank on S1 | 12.04 | 15.82 | 31.39 |
| Tank on S2 | 8.52 | 9.17 | 7.63 |
| Tank on S3 | 4.67 | 5.10 | 9.2 |
| Tank on S4 | 2.58 | 2.8 | 8.52 |

From these results, it is very clear that as the stiffness of the soil on which the tank rests decreases, the fundamental impulsive frequency also decreases, and is in agreement with well proven facts (Kramer, 2007; Larkin, 2008; Kianoush and Ghaemmaghami, 2011). The frequency of full fill tank on hard rock, rock, very dense soil and stiff soil respectively are 84%, 59%, 32% and 18% of the tank with rigid base, whereas for half fill tank the corresponding values are 78%, 45%, 25% and 14% of tank with rigid base. SSI interaction is observed to have vital role for tanks founded on different soil condition. The reduction in modulus of elasticity from 7000 MPa to 150 MPa causes reduction of fundamental frequency of full fill tank from 12.04Hz to 2.58Hz. As stiffness ratio, the ratio of stiffness of structure to stiffness of soil increases, natural frequency gets reduced.

For all soil conditions, the impulsive frequency of the tank in half fill condition is higher than that of full fill condition. The impulsive frequency of the tank with rigid base in half fill condition is 1.4 times that of full fill tank, the variation reduces when soil-structure interaction is taken into account as the frequency of half fill tank varies from 1.31 to 1.08 times of full fill tank for the soil types considered (Table 7.2).

### 7.3 TIME HISTORY ANALYSIS OF FLUID- TANK - SOIL SYSTEM

Tank resting on four different soil conditions with full fill (design height of 11m) and half fill conditions are considered for the investigation of influence of soil-structure interaction on seismic behaviour of ground supported water tanks. In the seismic analyses, it is assumed that tanks are subjected to horizontal components of the past three earthquakes separately, the characteristics of the which are as given in Table 6.3. The obtained results are compared with the tank with rigid base. The displacement, hoop force, bending moment and base shear developed in the tank wall are also noted.

### 7.4 FSI – SSI EFFECTS ON RESPONSE UNDER NORTHRIDGE EARTHQUAKE OF LOW FREQUENCY CONTENT

Response of cylindrical water tank on different soil types in full fill and half fill water conditions were obtained by performing the time history analyses. Response parameters of the tank such as radial displacement, hoop force, bending moment and base shear due to hydrodynamic pressure exerted by low frequency content record of Northridge earthquake were examined and the influence of soil properties and water height on the seismic behaviour of tank is evaluated.

#### 7.4.1 Influence of Soil Properties on Responses under Northridge Earthquake

***Radial Displacement***

The maximum radial displacement (U) response that occurs at the top of tank wall in full fill ($U_F$) and half fill conditions ($U_H$) during seismic loading are presented in Table 7.3. The ratio of maximum response displacement of the tank on different soil conditions to maximum response displacement of tank with rigid base are computed and tabulated in Table 7.3 to assess the significance of SSI on response of tanks on different soil conditions.

Table 7.3 Maximum radial displacement under Northridge earthquake

| Support /soil type | Displacement (mm) | | $\frac{U_F}{U_{F,\text{rigid base}}}$ | $\frac{U_H}{U_{H,\text{rigid base}}}$ | Displacement at half fill as % of full fill case $\frac{U_H}{U_F} \text{x } 100$ |
|---|---|---|---|---|---|
| | Full fill ($U_F$) | Half fill ($U_H$) | | | |
| Rigid base | 1.52 | 0.61 | | | 40.13 |
| S1 | 1.8 | 0.91 | 1.18 | 1.49 | 50.6 |
| S2 | 5.09 | 2.03 | 3.35 | 3.33 | 39.9 |
| S3 | 26.63 | 10.07 | 17.52 | 16.51 | 37.8 |
| S4 | 201.77 | 60.37 | 132.74 | 98.97 | 29.9 |

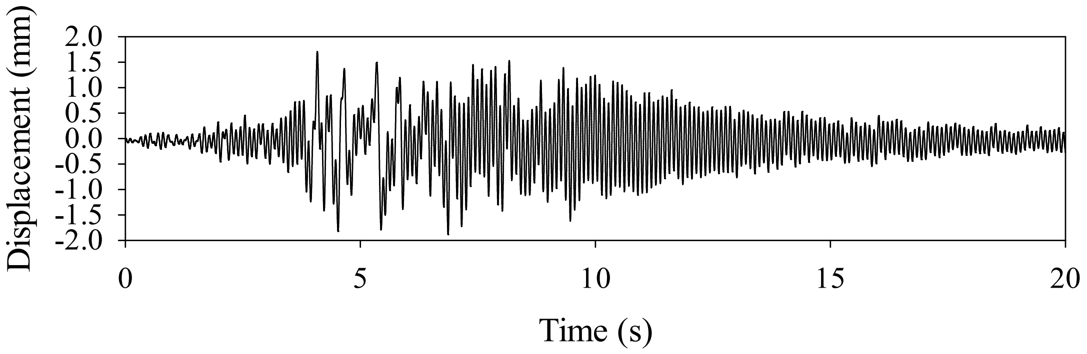
For both full fill and half fill tanks resting on soil, response displacement in radial direction increase as the soil stiffness decreases and it is highest for stiff soil (S4) and lowest for tank on hard rock (S1). The response displacement in radial direction of the tank with rigid base is lower than the tank on any soil condition, indicating the significance of including soil-structure interaction effects in the analysis.

Maximum displacement in radial direction of full fill tank in S1, S2, S3 and S4 under Northridge earthquake are 1.18, 3.35, 17.52 and 132.74 times that of tank with rigid base, demonstrating increased influence of SSI effects with reduction in soil stiffness for both full fill and half fill conditions under Northridge earthquake. Veletsos and Tang (1990) have reported that soil–structure interaction may reduce significantly the critical responses of shallow tanks, but may increase those of tall, stiff tanks that have high fundamental natural frequencies.

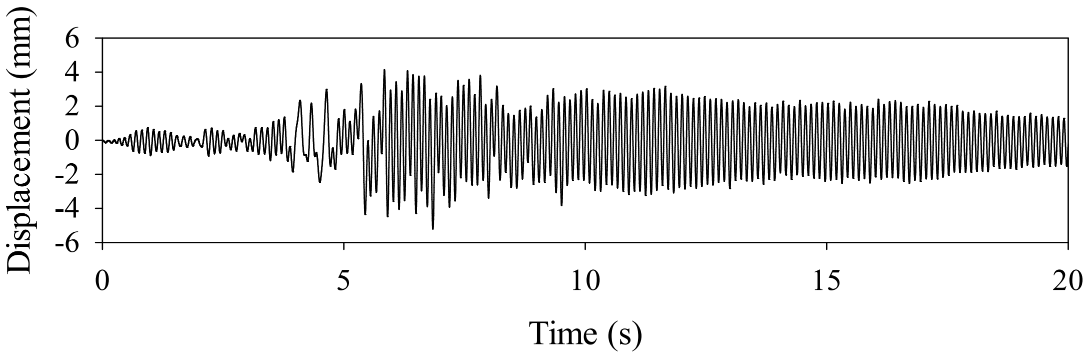
It can be seen from Table 7.3, that response displacement of the tank in full fill condition is more than that in half fill condition for all soil conditions. The percentage of maximum response in half fill conditions varies from 50.6 to 29.9% as soil property varies from that of hard rock to stiff soil. Results indicates that SSI effects have major impact on tanks in full fill condition, due to the contribution of FSI on the seismic response of tanks.

It can be noticed from the modal analysis that as the stiffness of soil decreases, the frequency decreases which is due to increased amplitude of displacement with reduced stiffness. In the hard rock (S1), responses vibrations are of small amplitude with large frequency whereas in stiff soil (S4), they are of large amplitude with low frequency.

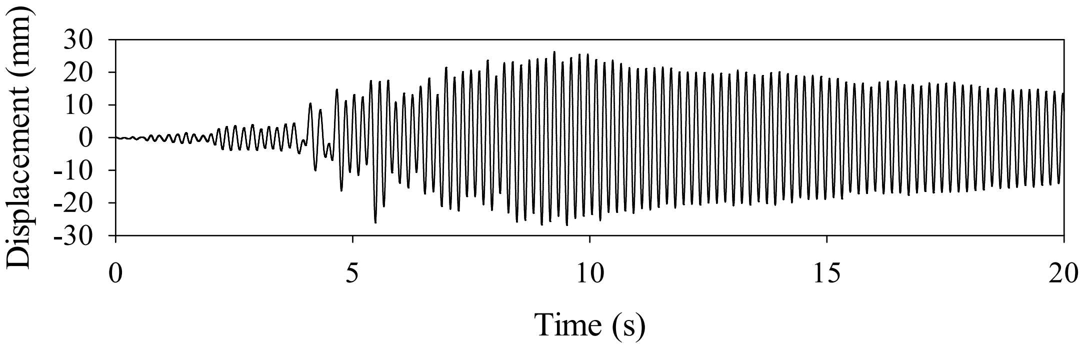
Fig.7.1 presents time history of response displacement at the top of tank wall, the location where the maximum response occurs. The increase in amplitude with reduction in soil stiffness is evident from the plots. The maximum response displacement and its time of occurrence exhibit considerable changes with soil properties, demonstrating the influence of soil properties on seismic behaviour of water tanks. Maximum response of 201.77 mm occurs in stiff soil at 9.54s where in tank on hard rock, the maximum response (1.8mm) occurs at 5.4s, against the peak input acceleration of 0.583g at 5.36s. For tanks on rock (S2) and very dense soil (S3), the maximum responses are at 6.84s and 9.52s respectively.



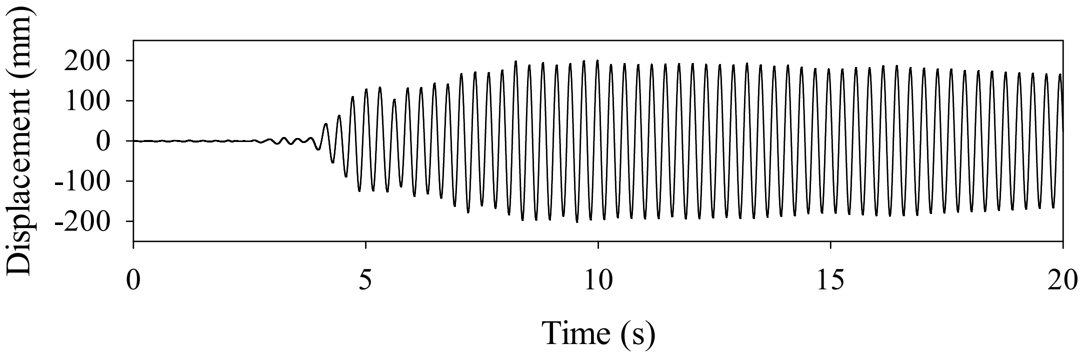

(a) Hard Rock; S1




(b) Rock; S2




(c) Very dense soil; S3




(d) Stiff soil; S4

Fig. 7.1 Time history response of radial displacement of the tank resting different soil types under Northridge earthquake

***Hoop Force***

The maximum hoop force (H) developed in the tank due to seismic loading of Northridge Earthquake low frequency content is given in Table 7.4. Maximum hoop force is observed to be increased with a decrease in soil stiffness under Northridge Earthquake in both full fill and half fill conditions. The ratio of maximum hoop force occurs on tanks of different soil to that of tank on rigid base gives the indication of role of SSI on hoop behaviour of full fill and half fill tanks on different soil conditions.

Table 7.4 Maximum Hoop force under Northridge earthquake

| Support /soil type | Maximum Hoop force (kN/m) | | $\frac{H_F}{H_{F,\text{rigid base}}}$ | $\frac{H_H}{H_{H,\text{rigid base}}}$ | Hoop force at half fill as % of full fill case $\frac{H_H}{H_F} \text{x } 100$ |
|---|---|---|---|---|---|
| | Full fill ($H_F$) | Half fill ($H_H$) | | | |
| Rigid base | 449 | 335.7 | | | 74.8 |
| S1 | 373 | 320.4 | 0.83 | 0.95 | 85.9 |
| S2 | 633.3 | 396.8 | 1.41 | 1.18 | 62.7 |
| S3 | 1251.9 | 701.9 | 2.79 | 2.09 | 56.1 |
| S4 | 3228.1 | 1341.4 | 7.19 | 4.00 | 41.6 |

Maximum hoop force of the tank on hard rock is only 83% and 95% of response value of the tank with rigid base for full fill and half fill conditions respectively. The reduction in response of tank on hard rock in comparison to tank with rigid base is due to dynamic pressure variation in the middle of tank wall due to rocking motion of foundation. This phenomenon is highly dependent on earthquake frequency content, tank configuration and soil properties (Kianoush and Ghaemmaghami, 2011). This implies that soil-structure interaction may cause a reduction in response parameter and may lead to reduction in structural demand depending upon the frequency content of earthquake and soil properties. For all soil types other than hard rock, the hoop force is more than the tank with rigid base and the maximum hoop force of full fill tank in S2, S3 and S4 are 1.41, 2.79 and 7.19 times of tank with rigid base.

It may also be noted that maximum hoop force of the tank in full fill condition is more than the response in half fill condition, as hoop force in S1, S2, S3 and S4 in half fill conditions are 0.86, 0.63, 0.56and 0.42 times of full fill tank. Influence of water fill condition is more significant in the case of hard rock and rock compared to soil with low stiffness.

The variation of hoop force along the height of the tank wall in full fill condition is plotted in Fig. 7.2 to understand the effect of soil stiffness on the hoop behaviour of the tank. For all tanks in full fill condition, except for tank on stiff soil (S4), maximum hoop tensile force occurs at a height of 8m from the bottom of the tank, whereas for tank on stiff soil, maximum is at a height of 9 m from the tank bottom. Results indicates that the tank experiences compressive force at the bottom 1m height of the tank.

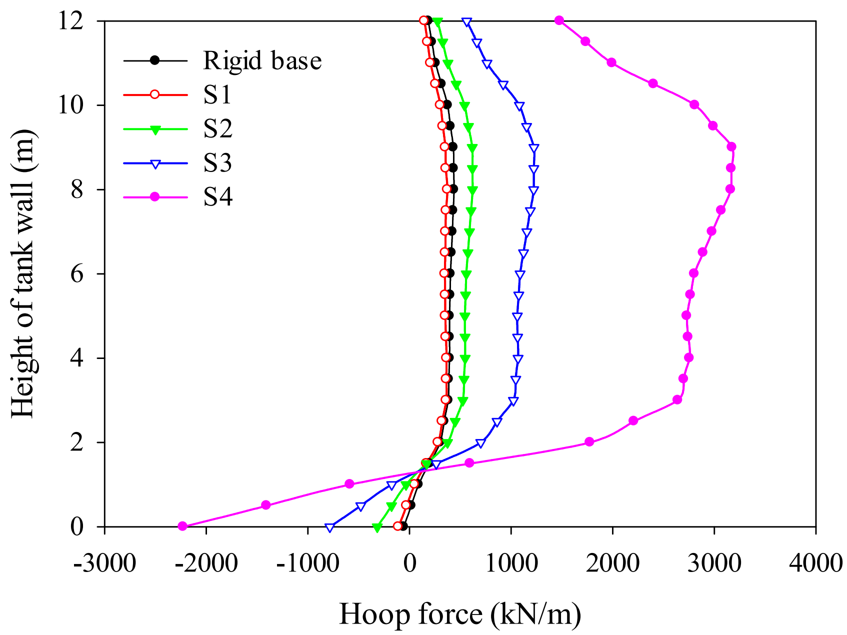



Fig. 7.2 Hoop force along the height of tank wall under Northridge earthquake

***Bending Moment***

Maximum bending moment on the tank due to seismic loading of Northridge earthquake is given in Table 7.5. The maximum bending moment of tank with rigid base for both full fill and half fill conditions are lower than the response of tank resting on soil with varying elastic properties. This indicates that that the soil-structure interaction is to be considered for seismic response and design of the tanks. The ratio of maximum bending moment of the tank on different soil conditions in full fill ($M_F$) and half fill ($M_H$) conditions to corresponding value of tank with rigid base is evaluated to study the influence of soil stiffness on bending behaviour of the tanks under seismic loading.

Table 7.5 Maximum Bending moment under Northridge earthquake

| Support /soil type | Maximum bending moment (kNm/m) | | $\frac{M_F}{M_{F,\text{rigid base}}}$ | $\frac{M_H}{M_{H,\text{rigid base}}}$ | Bending moment at half fill as % of full fill case $\frac{M_H}{M_F} \text{x } 100$ |
|---|---|---|---|---|---|
| | Full fill ($M_F$) | Halffill ($M_H$) | | | |
| Rigid base | 25.83 | 18.95 | | | 73.4 |
| S1 | 30.55 | 22.28 | 1.18 | 1.18 | 72.9 |
| S2 | 50.35 | 29.61 | 1.95 | 1.56 | 58.8 |
| S3 | 107.73 | 53.39 | 4.17 | 2.82 | 49.6 |
| S4 | 282.25 | 102.96 | 10.93 | 5.43 | 36.5 |

The bending moment is observed to be increased with reduction in soil stiffness under Northridge earthquake as observed in hoop force and response displacement. The maximum bending moment of full fill tank on hard rock, rock, very dense soil and stiff soil are 1.18, 1.95, 4.17 and 10.93 times respectively of corresponding value of tank with rigid base under low frequency content earthquake loading.

For tanks in all soil conditions, maximum bending moment of full fill tank is more than half fill condition as in case of hoop force and radial displacement. Bending moment in S1, S2, S3 and S4 in half fill conditions are 72.9, 58.8, 49.6 and 36.5% respectively of full fill tank. By comparing the values of $\frac{M_F}{M_{F,\text{rigid base}}}$ and $\frac{M_H}{M_{H,\text{rigid base}}}$, it can be inferred that SSI has major role in full fill tank in comparison to half fill tank on seismic response behaviour of water tanks.

Variation of bending moment along the tank wall at the instant of maximum bending moment for the full fill tank on each soil type is depicted in Fig. 7.3. Bending moment is the maximum at a height of 1m and is the least at about 5 m from the base of the tank wall, for all tanks. Bending behaviour of tank wall along height indicates a change in curvature around the mid height of tank wall. It is due to the rocking motion of the foundation. From the modal analysis, it can be seen that the participation factor associated with rotational mode of vibration is also significant for tanks founded on soil. Such bending behaviour can also be noticed by Priestley et al. (1986).

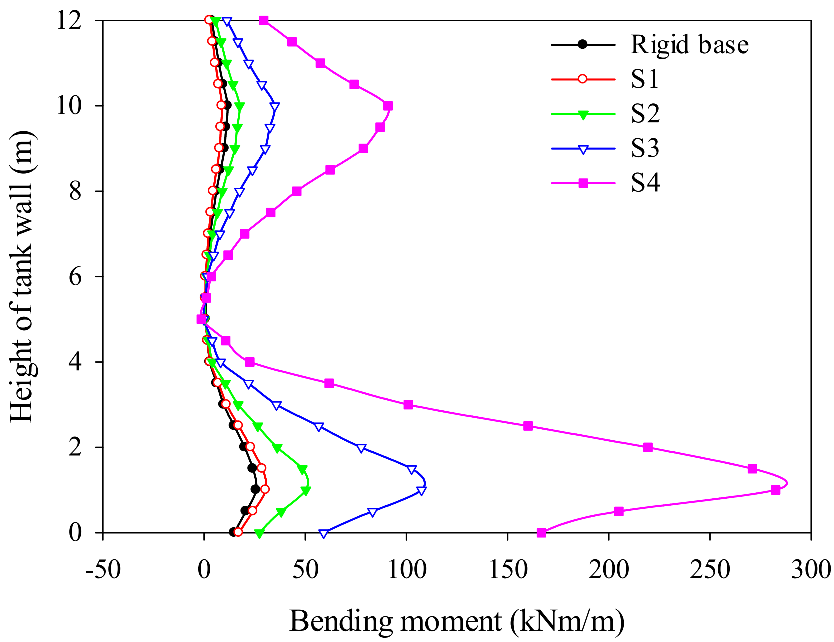



Fig. 7.3 Bending moment along the height of tank wall under Northridge Earthquake

***Base Shear***

Analysis indicated increase in base shear with reduction in soil stiffness as expected. The maximum values of base shear in tanks on different soil conditions are presented in Table 7.6. Maximum base shear of tank on any soil condition is more than the response of tank with rigid base. The influence is more pronounced in tanks of full fill condition than that of tank with water at mid height of tank. The maximum base shear is increased to 9.3 times in response to Northridge earthquake as the modulus of elasticity of soil is reduced from 7000 MPa (hard rock, S1) to 150 MPa (stiff soil, S4) in tanks with maximum water fill condition.

Table 7.6 Maximum base shear under Northridge earthquake

| Support /soil type | Maximum Base Shear (kN/m) | | $\frac{S_F}{S_{F,\text{rigid base}}}$ | $\frac{S_H}{S_{H,\text{rigid base}}}$ | Base shear at half fill as % of full fill case $\frac{S_H}{S_F}$ x 100 |
|---|---|---|---|---|---|
| | Full fill ($S_F$) | Halffill ($S_H$) | | | |
| Rigid base | 31.76 | 22.58 | | | 71.1 |
| S1 | 37.28 | 26.57 | 1.17 | 1.18 | 71.3 |
| S2 | 62.0 | 35.54 | 1.95 | 1.57 | 57.3 |
| S3 | 132.65 | 64.27 | 4.18 | 2.85 | 48.5 |
| S4 | 348.08 | 124.4 | 10.96 | 5.51 | 35.7 |

Similar to other response parameters, base shear also decreases with decrease in water height. The hoop force of half fill tank is 71.3 %, 57.3%, 48.5 % and 35.7% of the full fill tank in hard rock, rock, very dense soil and stiff soil respectively.

## 7.5 FSI – SSI EFFECTS ON RESPONSE UNDER IMPERIAL VALLEY EARTHQUAKE OF MEDIUM FREQUENCY CONTENT

Responses parameters of the tank due to hydrodynamic pressure exerted by medium frequency content record of Imperial Valley earthquake have been noted and the influence of soil stiffness and water fill conditions of tank on seismic behaviour is analysed.

### 7.5.1 Influence of Soil Properties on Responses under Imperial Valley Earthquake

***Radial Displacement***

Analysis has indicated that the reduction in soil stiffness amplifies the response displacement of the tank considerably under medium frequency content earthquake. The maximum values of radial displacement at the top of the tank wall are given in Table 7.7.

Table 7.7 Maximum radial displacement under Imperial Valley earthquake

| Support /soil type | Maximum displacement (mm) | | $\frac{U_F}{U_{F,\text{rigid base}}}$ | $\frac{U_H}{U_{H,\text{rigid base}}}$ | Displacement at half fill as % of full fill case $\frac{U_H}{U_F} \times 100$ |
|---|---|---|---|---|---|
| | Full fill ($U_F$) | Half fill ($U_H$) | | | |
| Rigid base | 1.11 | 0.49 | | | 44.1 |
| S1 | 1.88 | 0.94 | 1.69 | 1.92 | 50.0 |
| S2 | 3.58 | 1.6 | 3.23 | 3.27 | 44.7 |
| S3 | 10.63 | 8.55 | 9.58 | 19.54 | 80.4 |
| S4 | 40.35 | 37.11 | 36.35 | 75.73 | 91.9 |

Maximum displacement in radial direction of full fill tank in S1, S2, S3 and S4 under Imperial Valley earthquake are 1.69, 3.23, 9.06 and 36.35 times that of tank with rigid base whereas in half fill condition, maximum displacements at different soil conditions varies from 1.9 to 75.7 times of tank with rigid base, indicating increased impact of SSI on tanks with low stiffness. The displacement along the height of the tank wall in full fill condition is presented in Fig.7.4.

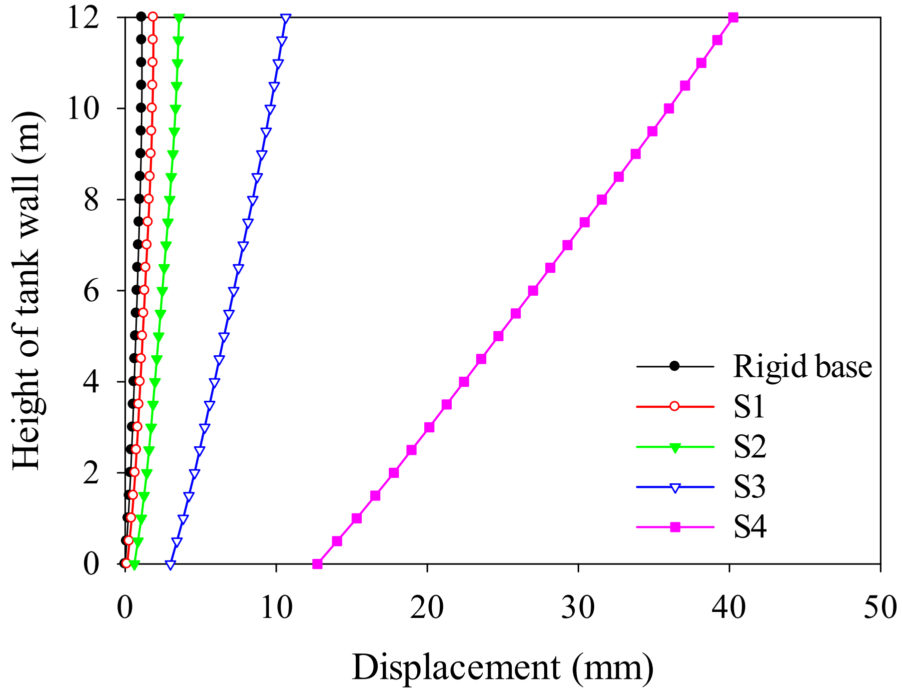



Fig. 7.4 Displacement along height of tank wall under Imperial Valley earthquake

For all tanks, the response radial displacement of full fill tank is more than half fill condition. Displacements of half fill tank is 50%, 55.3%, 19.6% and 8.1% less than the corresponding value of full fill tank in S1, S2, S3 and S4 respectively.

Under the earthquake of medium frequency content, SSI effects are found to be more substantial for tanks in half fill conditions as the ratio $\frac{U_H}{U_{H,\text{rigid base}}}$ is higher than $\frac{U_F}{U_{F,\text{rigid base}}}$ for all soil types. Hence it can be inferred that, water fill condition is also a governing factor that decides the seismic behaviour of tanks under medium frequency content earthquake.

***Hoop Force***

Maximum hoop force of the tank increases with decrease in soil stiffness for both full fill and half fill conditions under seismic loading of Imperial Valley earthquake (Table 7.8). Also the hoop force of tank on any soil condition is more than the response without SSI effects.

Table 7.8 Maximum hoop force under Imperial Valley earthquake

| Support /soil type | Maximum Hoop force (kN/m) | | $\frac{H_F}{H_{F,\text{rigid base}}}$ | $\frac{H_H}{H_{H,\text{rigid base}}}$ | Hoop force at half fill as % of full fill case $\frac{H_H}{H_F}$ x 100 |
|---|---|---|---|---|---|
| | Full fill ($H_F$) | Halffill ($H_H$) | | | |
| Rigid base | 340.65 | 225.95 | | | 66.3 |
| S1 | 411.79 | 313.15 | 1.21 | 1.39 | 76.0 |
| S2 | 458.23 | 313.19 | 1.35 | 1.39 | 31.7 |
| S3 | 509.89 | 556.06 | 1.50 | 2.46 | 109.1 |
| S4 | 658.37 | 830.2 | 1.93 | 3.67 | 126.1 |

Hoop force on tanks resting on soils S3 and S4 indicates that the reduction in modulus of elasticity from 500MPa (very dense soil) to 150MPa (stiff soil) causes an increase of 29% of hoop force for full fill and 49% for half fill conditions.

Under the Imperial Valley earthquake, the hoop force of the half fill tank is found to be more effected by SSI since $\frac{H_H}{H_{H,\text{rigid base}}}$ is higher than $\frac{H_F}{H_{F,\text{rigid base}}}$ for all soil types, indicating the significance of water fill condition on seismic behaviour of tanks.

The hoop force of half fill tank is less than the full fill tank with rigid base and on hard rock (S1) and rock (S2). But the maximum hoop force of half fill tank in very dense soil (S3) and stiff soil (S4) are 9.1% and 26.1% more than the full fill tank on same soil condition. The variation of hoop force along the height of tank wall of full fill and half fill tanks in very dense soil and stiff soil at the instant of maximum hoop force are given in Fig.7.5. For tanks in half fill condition, maximum response is at 3m and in full fill condition, maximum is at 9m from the bottom of the tank. The higher outward pressure at the low height leads to higher value of hoop force in the half fill condition for tanks soil with low stiffness (S3 and S4). The maximum hoop force of 556.06 kN/m in half fill tank on S3 occurs at 2.66s and maximum response of full fill tank on same soil type is at 2.24s. For the soil with least stiffness (S4), hoop force is maximum at 11.18s and 12.08s for half fill and full fill conditions respectively.

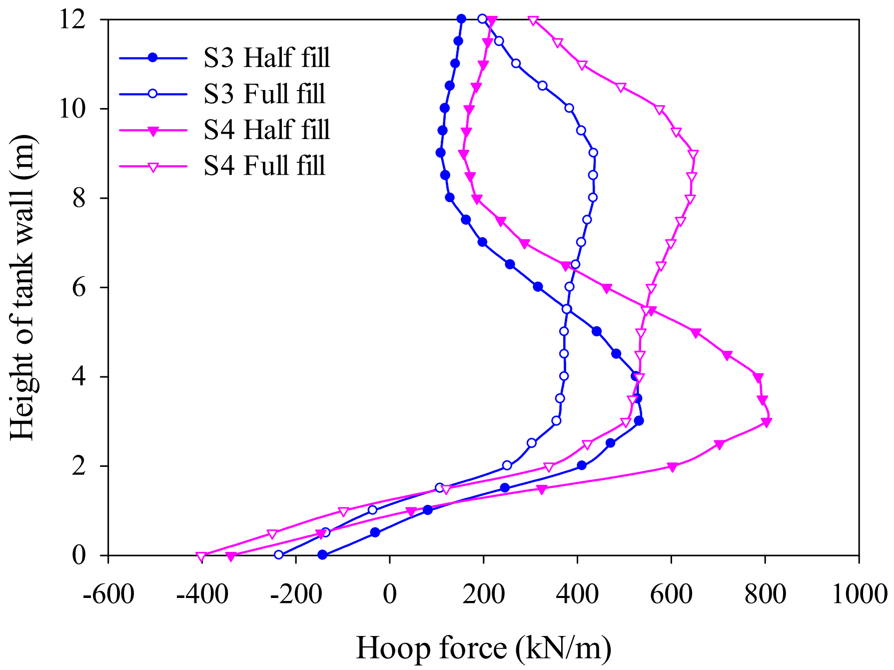



Fig. 7.5 Hoop force of full fill and half fill tanks on very dense soil and stiff soil

Variation of hoop force along the height of tank wall for the full fill tank on different soil conditions at the event of maximum hoop force are given in Fig.7.6. The maximum hoop force is at a height of about 8m from base of tank for tank with rigid base, tank on hard rock (S1) whereas for tanks on S2 (rock), S3 (very dense soil) and S4 (stiff soil), the maximum response is at about 9m. From the plot, it can be observed that the variation in hoop force from 2m to 7.5m is less. As in Northridge earthquake, here also tank experiences compressive force at the bottom 0.5 to 1.5 m of the tank wall; depending on the soil characteristics.

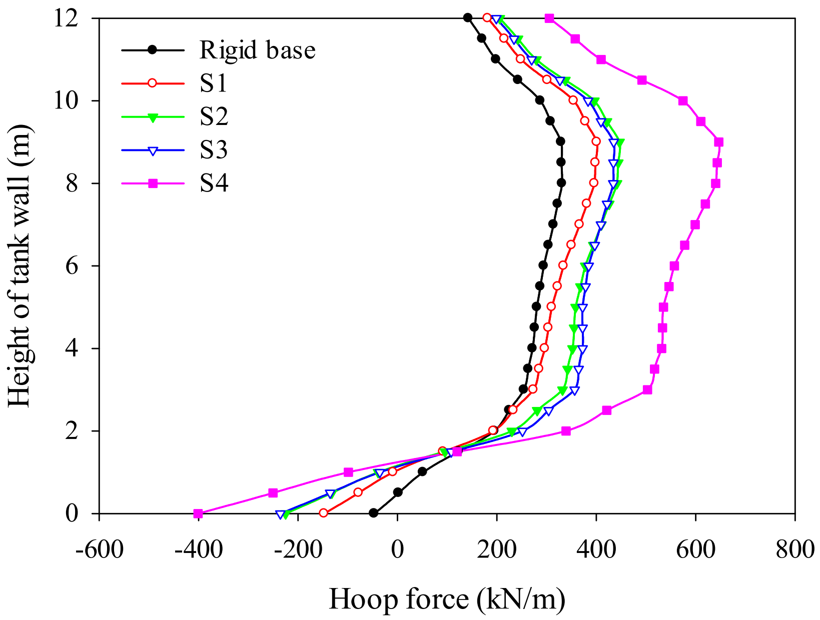



Fig. 7.6 Hoop force along height of tank wall on different soil conditions under Imperial Valley earthquake

***Bending Moment***

The maximum bending moment of the tanks under Imperial Valley earthquake of medium frequency content are given in Table 7.9. Bending moment increases with decrease in soil stiffness for both full fill and half fill conditions and the response of tank on soil is higher than that of rigid base.

Table 7.9 Maximum Bending moment under Imperial Valley earthquake

| Support /soil type | Maximum bending moment (kNm/m) | | $\frac{M_F}{M_{F,\text{rigid base}}}$ | $\frac{M_H}{M_{H,\text{rigid base}}}$ | Bending moment at half fill as % of full fill case |
|---|---|---|---|---|---|
| | Full fill ($M_F$) | Half fill ($M_H$) | | | $\frac{M_H}{M_F}$ x 100 |
| Rigid base | 16.59 | 11.82 | | | 71.2 |
| S1 | 27.77 | 20.11 | 1.67 | 1.70 | 72.4 |
| S2 | 33.52 | 22.57 | 2.02 | 1.91 | 67.3 |
| S3 | 44.71 | 43.71 | 2.69 | 3.70 | 97.7 |
| S4 | 62.62 | 65.09 | 3.77 | 5.51 | 103.9 |

The maximum bending moment of the full fill tank on S1, S2, S3 and S4 are 1.67, 2.02, 2.69 and 3.77 times of tank with rigid base and for the half fill condition, corresponding values are 1.7, 1.9, 3.7 and 5.51. SSI effects are predominated in tanks in half fill condition, especially for tanks on soil with low stiffness. As in case of response displacement and hoop force, here also higher impact of water fill condition on seismic response of tank on soil with low stiffness under medium frequency content earthquake is evident. Maximum bending moment on tank with half fill condition on stiff soil is even more than the response of full fill tank.

Bending moment of the half fill tank on stiff soil (S4) is more than the full fill tank (3.9%) , whereas for all other soil conditions, full fill response is higher than the half fill condition. From Fig.7.7, the difference in bending behaviour of the half fill tank wall can be seen which leads to the higher value of bending moment in half fill condition of tank founded on stiff soil, S4.

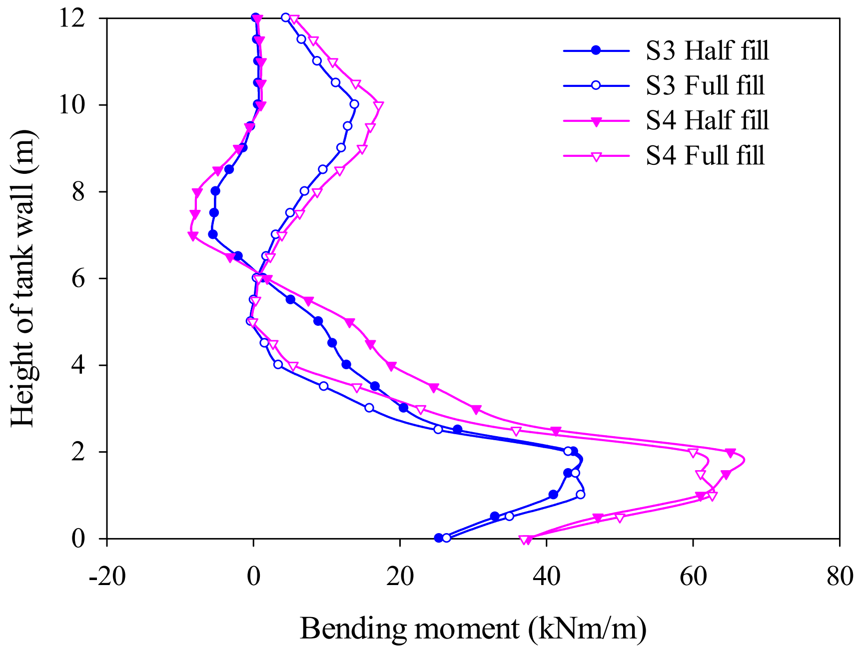



Fig. 7.7 Bending moment of full fill and half fill tanks on very dense soil and stiff soil

***Base Shear***

SSI effects on base shear can be clearly revealed from Table 7.10. Base shear response of the tank on any soil condition is more than the tank with rigid base and base shear increases with reduction in soil stiffness for both full fill and half fill conditions. For full fill condition, the maximum base shear of full fill tank on S1, S2, S3 and S4 are 1.64, 2.02, 2.68 and 3.74 times of maximum base shear of tank with rigid base. For tanks in soil with low stiffness, SSI effects dominates for half fill condition.

Table 7.10 Maximum base shear under Imperial Valley earthquake

| Support /soil type | Maximum Base Shear (kN/m) | | $\frac{S_F}{S_{F,\text{rigid base}}}$ | $\frac{S_H}{S_{H,\text{rigid base}}}$ | Base shear at half fill as % of full fill case $\frac{S_H}{S_F}$ x 100 |
|---|---|---|---|---|---|
| | Full fill ($S_F$) | Half fill ($S_H$) | | | |
| Rigid base | 20.52 | 13.9 | | | 67.7 |
| S1 | 33.66 | 23.95 | 1.64 | 1.72 | 71.1 |
| S2 | 41.41 | 26.97 | 2.02 | 1.94 | 65.1 |
| S3 | 55.0 | 52.77 | 2.68 | 3.80 | 95.5 |
| S4 | 76.68 | 78.57 | 3.74 | 5.65 | 102.5 |

Except for tank on S4, base shear of full fill tank is more than half fill condition. For tank on stiff soil, maximum base shear of half fill tank is 1.02 times of full fill tank, showing the necessity to consider the effect of water fill condition on the seismic analysis of water tanks.

Seismic response analysis of water tanks subjected to medium frequency content earthquake leads to the importance of consideration of water fill condition on seismic analysis of water tanks. The fundamental step in the seismic analysis by various codes includes the determination of time period/ frequency which is suggested only for tank in maximum depth of liquid / design depth. But from these studies, it is inferred that analysis should include the water fill condition in the seismic analysis of tanks. The water fill condition significantly influences the seismic behaviour of tanks founded on soil with low stiffness if pile foundations are not provided to transfer the load to a hard strata.

## 7.6 FSI – SSI EFFECTS ON RESPONSE UNDER KOYNA EARTHQUAKE OF HIGH FREQUENCY CONTENT

This section includes the response parameters of tank resting on different soil conditions under the action of high frequency content record of Koyna earthquake. Comparisons are also made with tank without SSI effects to assess the influence of soil properties on dynamic response.

### 7.6.1 Influence of Soil Properties on Responses under Koyna earthquake

***Radial Displacement***

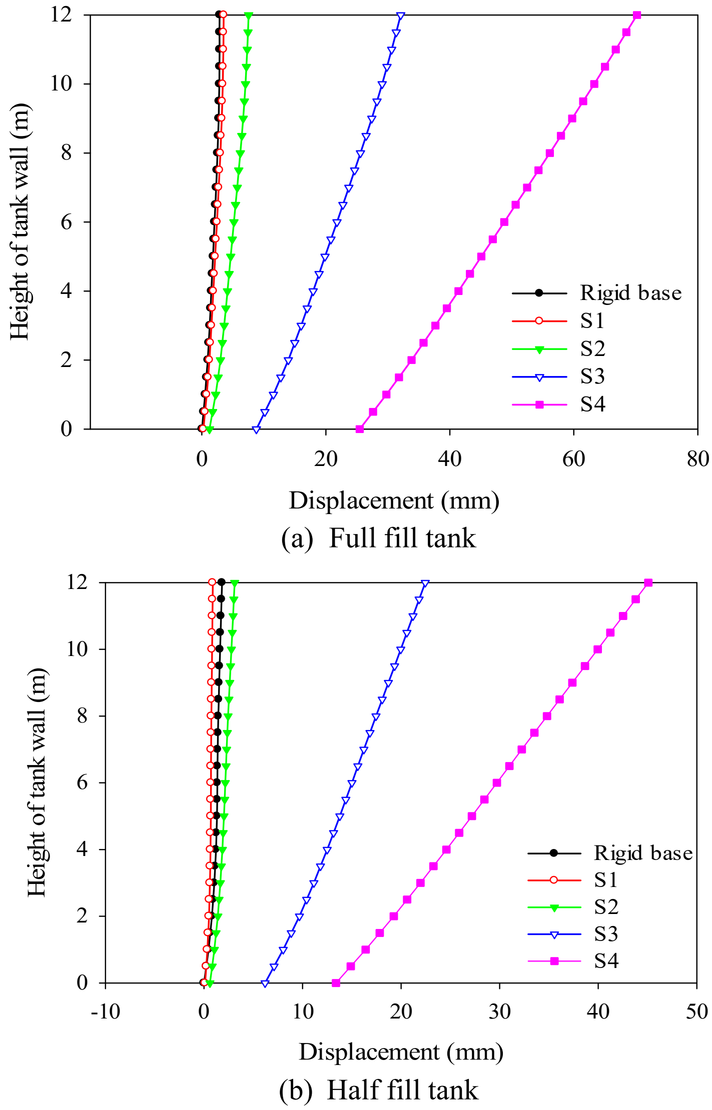
Maximum value of radial displacement (Table 7.11) on top of tank wall indicates considerable increase in response displacement with reduction in soil stiffness under Koyna earthquake. Response displacement of tank with rigid base is less than tank resting on any soil type considered, for full fill condition. But for tanks in half fill condition, response of tank on hard rock is less than that of tank with rigid base. Maximum displacement in radial direction of full fill tank on S1, S2, S3 and S4 under Koyna earthquake are 1.23, 2.64, 11.4 and 24.54 times of maximum response of tank with rigid base.

Table 7.11 Maximum radial displacement under Koyna earthquake

| Support /soil type | Maximum displacement (mm) | | $\frac{U_F}{U_{F,\text{rigid base}}}$ | $\frac{U_H}{U_{H,\text{rigid base}}}$ | Displacement at half fill as % of full fill case $\frac{U_H}{U_F}$ x 100 |
|---|---|---|---|---|---|
| | Full fill ($U_F$) | Halffill ($U_H$) | | | |
| Rigid base | 2.86 | 1.83 | | | 64.0 |
| S1 | 3.51 | 0.96 | 1.23 | 0.53 | 27.4 |
| S2 | 7.55 | 3.12 | 2.64 | 1.70 | 41.3 |
| S3 | 32.6 | 22.45 | 11.4 | 12.27 | 68.9 |
| S4 | 70.19 | 45.08 | 24.54 | 24.63 | 64.2 |

Response displacement of half fill tank is less than full fill condition for all soil conditions. As the soil stiffness decreases, the effect of water fill condition on seismic response of tank is observed to varies from 27.4 to 68.9%.

The displacement along the height of the tank wall for full fill and half fill conditions at the instant of maximum radial displacement are plotted in Fig. 7.8. Radial displacement of tank founded on hard rock and rock is similar to that with rigid base. However higher displacement even at the bottom of tank can be noticed in soils of low stiffness (very dense soil and stiff soil).




(a) Full fill tank



(b) Half fill tank

Fig. 7.8 Displacement along height of tank wall due to Koyna earthquake

The time history of response displacement of tank on hard rock is given in Fig. 7.9, where the maximum response of 3.51mm occurs at 4.2s against the peak input acceleration of 0.489g at 4.12s. For tanks on all soil conditions, maximum response is at the top of tank wall, but the instant of occurrence of maximum response depends on the soil properties.

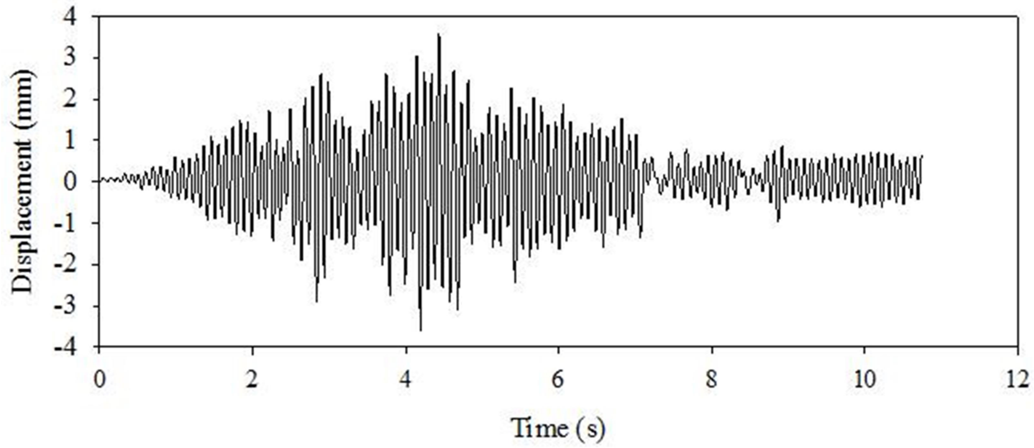



Fig. 7.9 Displacement time history of the full fill tank on hard rock under Koyna earthquake

***Hoop Force***

Table 7.12 Maximum hoop force under Koyna earthquake

| Support /soil type | Maximum Hoop force (kN/m) | | $\frac{H_F}{H_{F,\text{rigid base}}}$ | $\frac{H_H}{H_{H,\text{rigid base}}}$ | Hoop force at half fill as % of full fill case $\frac{H_H}{H_F} \times 100$ |
|---|---|---|---|---|---|
| | Full fill ($H_F$) | Half fill ($H_H$) | | | |
| Rigid base | 864.5 | 812.04 | | | 93.9 |
| S1 | 761.18 | 351.34 | 0.88 | 0.43 | 46.2 |
| S2 | 960.06 | 615.54 | 1.11 | 0.76 | 64.1 |
| S3 | 1525.4 | 1443.3 | 1.76 | 1.78 | 94.6 |
| S4 | 1184.8 | 1172.5 | 1.37 | 1.44 | 98.9 |

Hoop force of full fill tank on hard rock is less than the tank with rigid base for full fill and half fill conditions. Under Koyna earthquake, half fill tank rests on very dense soil and stiff soil has indicated maximum hoop force, more than 90% of that noticed in full fill condition.

Variation of hoop force along the height of tank wall under Koyna earthquake (at the event of maximum hoop force in each case) for full fill and half fill conditions are given in Fig. 7.10.

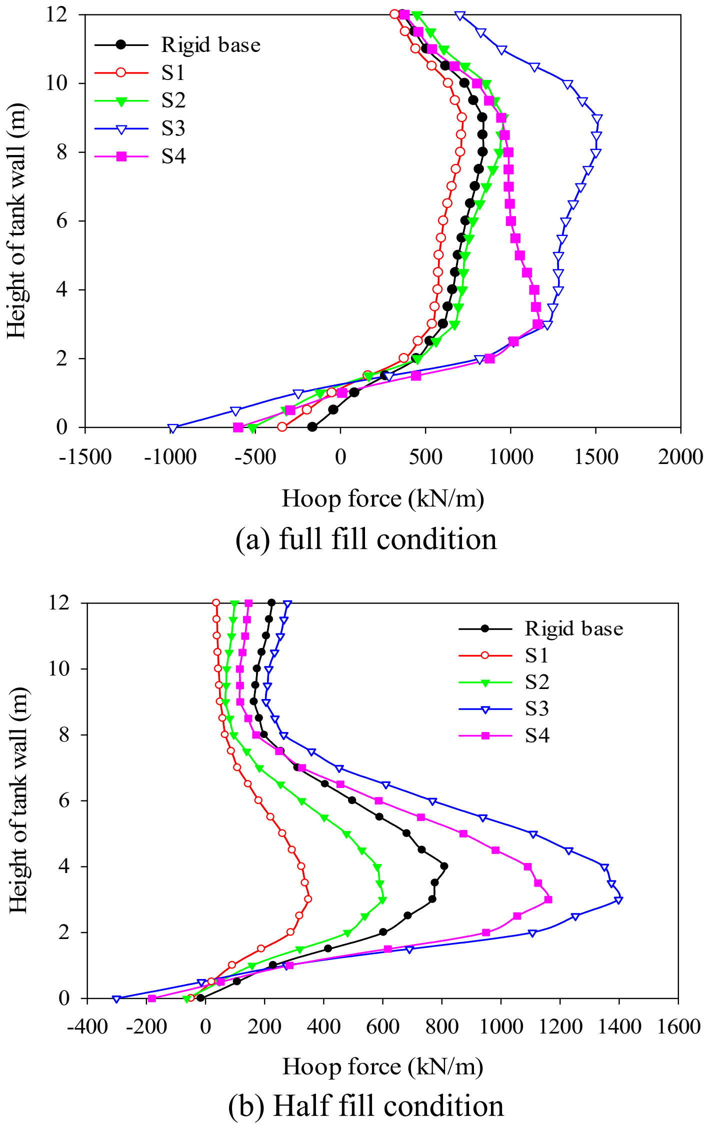



Fig. 7.10 Hoop force along the height of tank wall under Koyna earthquake

For the tanks in full fill condition with rigid base, hoop force is maximum at a height of 9m from tank bottom whereas for tank on S1, S2 and S3, maximum response is at about 8m from bottom of tank. But for tank in stiff soil (S4), maximum hoop force is at 3m from bottom of tank, and after which a sudden change in response can be observed which may be due to the rocking motion of the tank. This kind of response is observed only for full fill tank on stiff soil under high frequency content earthquake. For the tanks in half fill condition, the maximum hoop force occurs at a height of 3m from the base for soil types but for tank with rigid base, the hoop force is maximum at a height of 4m from the base.

Variation of hoop force with time at the point of maximum response of tank on hard rock (S1) is given in Fig. 7.11. Maximum hoop force of 761.18 kN/m occurs at 4.2 s at a height of 8 m from bottom of tank.

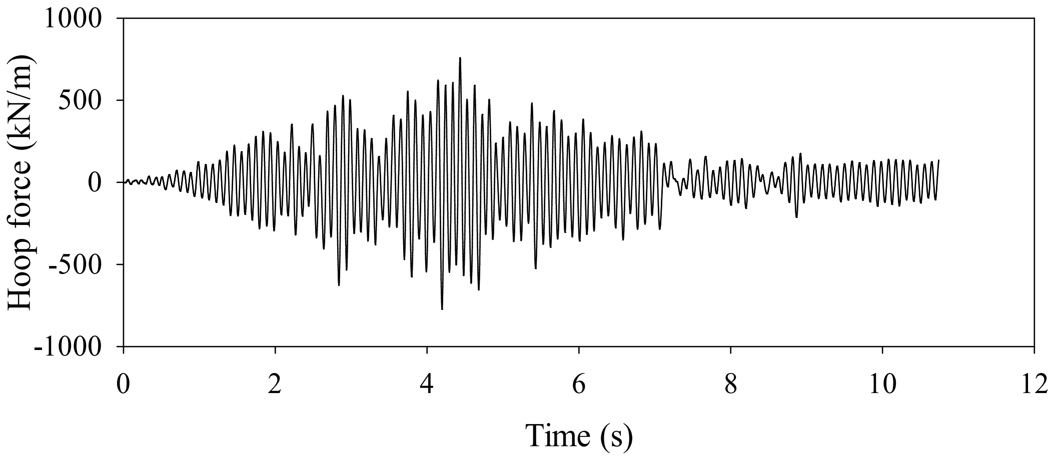



Fig. 7.11 Time history response of hoop force of full fill tank on hard rock under Koyna earthquake

***Bending Moment***

The maximum values of bending moment on tank wall under Koyna earthquake loading are given in Table 7.13 for different soil conditions. Bending moment is the highest for tank resting on very dense soil (S3) as observed in the case of maximum hoop force. The decrease in bending moment in soil S4 may be attributed to higher yielding of tank on stiff soil as indicated by high radial displacement (Fig. 7.8) leading to low structural demand.

Table 7.13 Maximum Bending moment under Koyna earthquake

| Support /soil type | Maximum bending moment (kNm/m) | | $\frac{M_F}{M_{F,\text{rigid base}}}$ | $\frac{M_H}{M_{H,\text{rigid base}}}$ | Bending moment at half fill as % of full fill case $\frac{M_H}{M_F}\text{x } 100$ |
|---|---|---|---|---|---|
| | Full fill ($M_F$) | Half fill ($M_H$) | | | |
| Rigid base | 38.06 | 40.33 | | | 105.9 |
| S1 | 46.78 | 25.89 | 1.23 | 0.64 | 55.3 |
| S2 | 76.44 | 41.27 | 2.01 | 1.02 | 54.0 |
| S3 | 127.85 | 101.75 | 3.36 | 2.52 | 79.6 |
| S4 | 112.65 | 85.66 | 2.96 | 2.12 | 76.0 |

$\frac{M_F}{M_{F,\text{rigid base}}}$ and $\frac{M_H}{M_{H,\text{rigid base}}}$ ratios indicate that bending behaviour of the tank in full fill condition is different from that of half fill tanks. For the tank with rigid base, bending moment in half fill condition is higher than in full fill condition;variation of bending moment along height of tank wall for both half fill and full fill conditions at the instant of maximum bending moment are given in Fig. 7.12. The high value of bending moment in half fill condition under the Koyna earthquake is due to difference in bending behaviour of the tank under half fill condition as can be envisaged from Fig.7.12.

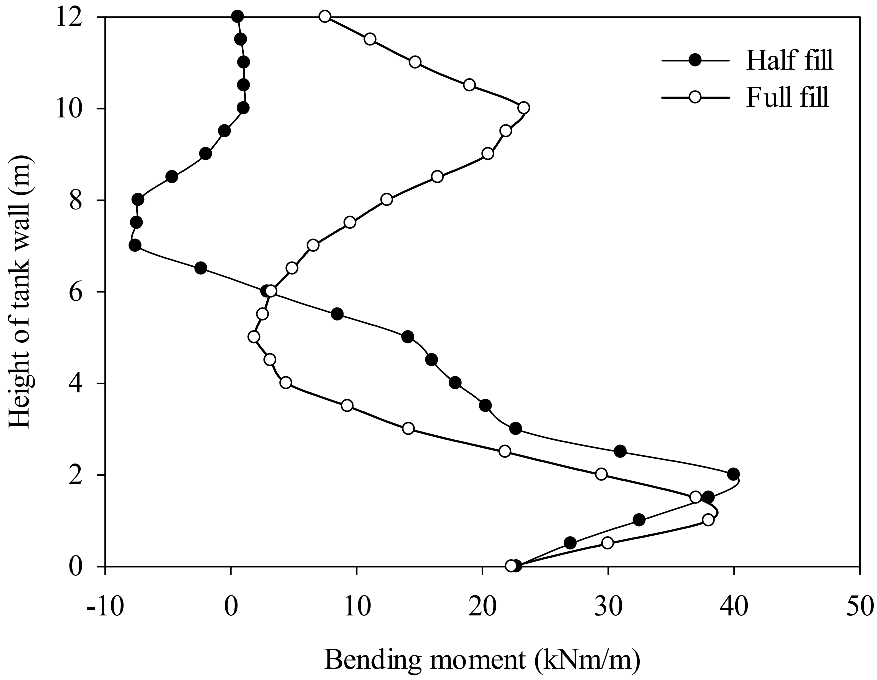



Fig. 7.12 Bending moment along height of tank wall with rigid base in full fill and half fill conditions under Koyna earthquake

Under Koyna earthquake of high frequency content, bending moment in half fill tank is less than full fill tank for all soil conditions. As the soil stiffness decreases, the effect of water fill condition on seismic response of tank is observed to varies from 54 to 79.6%.

Variation of bending moment along height of tank wall at the instant of maximum bending moment under Koyna earthquake is plotted in Fig.7.13 for the tanks in full fill condition. For all tanks, maximum bending moment is at a height of 1m from the base of tank wall and all tanks follow the similar bending pattern. Time history response of bending moment of full fill tank on hard rock is also depicted in Fig. 7.14.

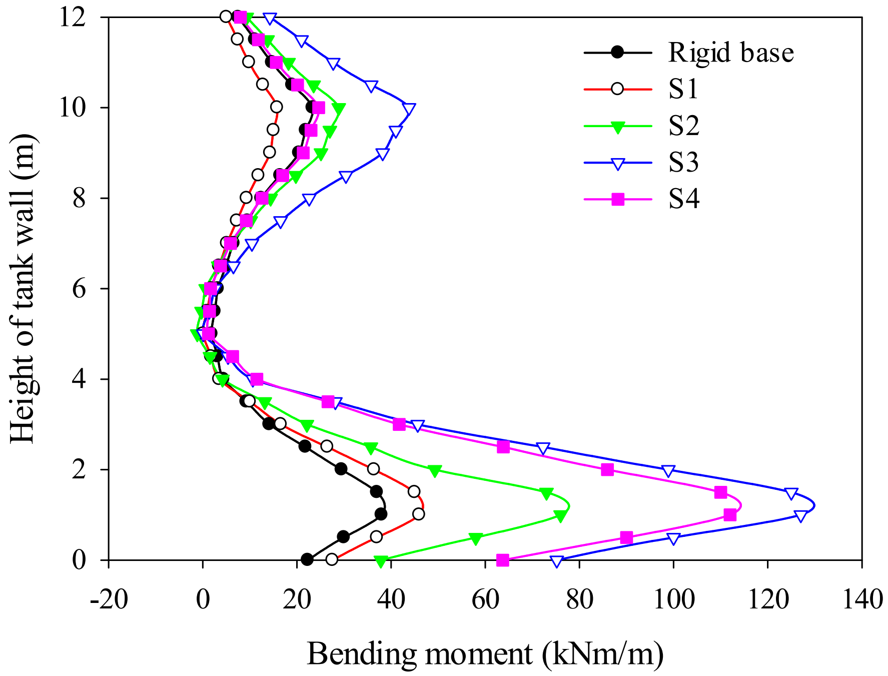



Fig. 7.13 Bending moment along height of tank wall due to Koyna earthquake

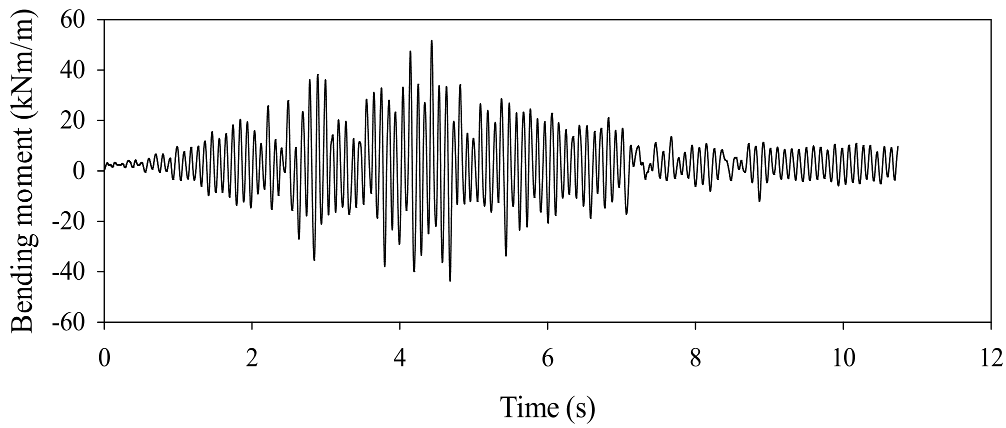



Fig. 7.14 Time history response of bending moment of full fill tank on hard rock under Koyna earthquake

***Base Shear***

Maximum values of base shear of the tank on different soil conditions are tabulated in Table 7.14. Base shear is maximum for tank resting on very dense soil (S3) for tanks in full fill and half fill conditions as observed in bending moment.

Table 7.14 Maximum base shear under Koyna earthquake

| Support /soil type | Maximum Base Shear (kN/m) Full fill ($S_F$) | Halffill ($S_H$) | $\frac{S_F}{S_{F,\text{rigid base}}}$ | $\frac{S_H}{S_{H,\text{rigid base}}}$ | Base shear at half fill as % of full fill case $\frac{S_H}{S_F}$ x 100 |
|---|---|---|---|---|---|
| Rigid base | 47.43 | 47.82 | | | 100.8 |
| S1 | 57.45 | 30.63 | 1.21 | 0.64 | 53.32 |
| S2 | 93.85 | 48.89 | 1.98 | 1.02 | 52.09 |
| S3 | 158.1 | 128.16 | 3.33 | 2.68 | 81.06 |
| S4 | 137 | 102.12 | 2.89 | 2.14 | 74.54 |

Tanks in full fill condition is found to be more influenced by soil stiffness than in half fill condition as $\frac{S_F}{S_{F,\text{rigid base}}}$ is higher than $\frac{S_H}{S_{H,\text{rigid base}}}$ for all types. Also, water fill condition is observed to have more influence on tank resting on soil with low stiffness where base shear at half fill condition is more than 70% of that of full fill condition.

## 7.7 INFLUENCE OF CHARACTERISTICS OF EARTHQUAKE ON SEISMIC RESPONSE

To study the influence of characteristics of earthquake on tank having different support conditions, the response of tank having wall fixed at base, tank with rigid base slab fixed at bottom and tank resting on soil having four different properties are compared. Conclusions are drawn from already reported results of time history analysis of the tanks in full fill and half fill conditions, under the action of three earthquakes having different characteristics, acting one at a time.

***Radial Displacement***

The response of the tanks to different seismic loading are compared to assess the influence of characteristics of earthquake on the seismic response of the tanks. The response displacement of the tanks having (i) idealised fixed boundary condition, (ii) having rigid base and (iii) base slab resting on soil having four different properties are tabulated in Table 7.15 for both full fill and half fill conditions. From the table, and from the discussions on sections 6.6.1, 7.4, 7.5 and 7.6, it is observed that that the influence of soil properties on response displacement is profound for all earthquake.

Table 7.15 Maximum displacement of the tank on different support conditions

| Earthquake loading | Tank wall fixed at bottom | Tank wall with rigid Base | Tank wall with base slab and soil S1 | Tank wall with base slab and soil S2 | Tank wall with base slab and soil S3 | Tank wall with base slab and soil S4 |
|---|---|---|---|---|---|---|
| Displacement (mm) – full fill tank | | | | | | |
| NEQ | 0.58 | 1.52 | 1.8 | 5.09 | 26.63 | 201.77 |
| IEQ | 0.52 | 1.11 | 1.88 | 3.58 | 10.63 | 40.35 |
| KEQ | 1.69 | 2.86 | 3.51 | 7.55 | 32.6 | 70.19 |
| Displacement (mm) – half fill tank | | | | | | |
| NEQ | 0.28 | 0.61 | 0.914 | 2.03 | 10.07 | 60.37 |
| IEQ | 0.19 | 0.49 | 0.939 | 1.6 | 8.55 | 37.11 |
| KEQ | 0.53 | 1.83 | 0.961 | 3.12 | 22.45 | 45.08 |

Of the three earthquakes, for tanks with fixed base, having rigid base slab and for tanks on hard rock (S1), rock (S2) and very dense soil (S3), the highest response radial displacement is for Koyna earthquake of high frequency content, whereas for tank on stiff soil (S4), the highest response is for low frequency content Northridge earthquake for which the PGA is high. The tank wall fixed at bottom is subjected to lowest displacement for all earthquakes, both in full fill and half fill conditions. It can be concluded that reduction in soil stiffness amplifies the response displacement.

***Hoop Force***

Fig. 7.15 and 7.16 infers that for low and medium frequency content earthquakes, Northridge and Imperial Valley, a direct relation of maximum hoop force developed to the stiffness of soil on which the tank rests can be noticed. As in case of radial displacement, hoop force of full fill and half fill tanks with fixed base, rigid base slab, and on hard rock, rock, and very dense soil due to Koyna earthquake of high frequency content is more than other earthquakes. For tanks on stiff soil (S4), hoop force in response to low frequency content earthquake is the highest, attributable to high PGA of Northridge earthquake. Response due to seismic loading of imperial valley earthquake of medium frequency content is the least because of its low PGA 0.349g compared to other two earthquakes. Maximum hoop force of tank with wall fixed at bottom is the least under all three earthquakes in both full fill and half fill conditions indicating the necessity of including soil-structure interaction in the analysis.

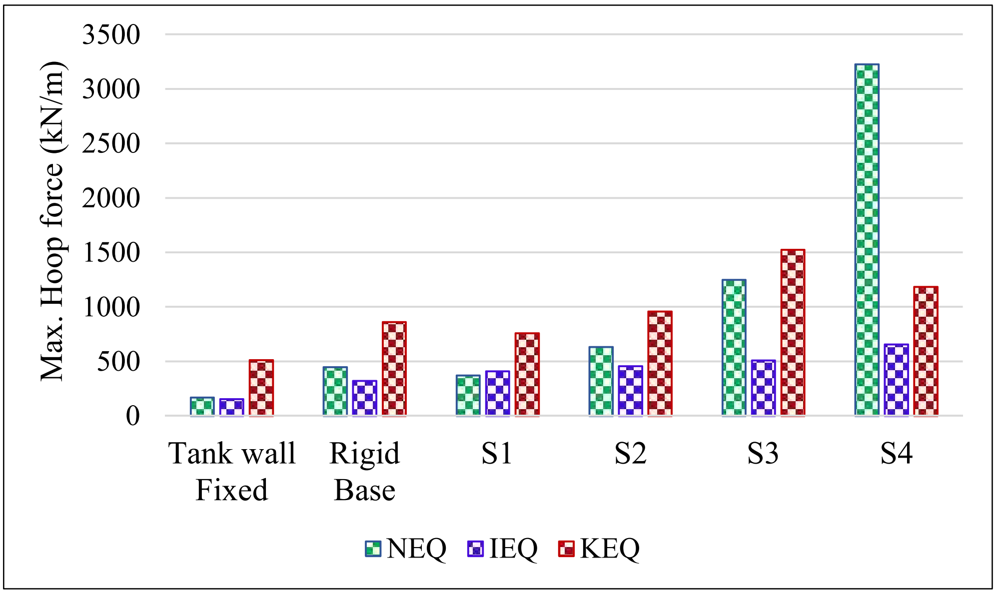



Fig. 7.15 Maximum hoop force of full fill tank on different support conditions

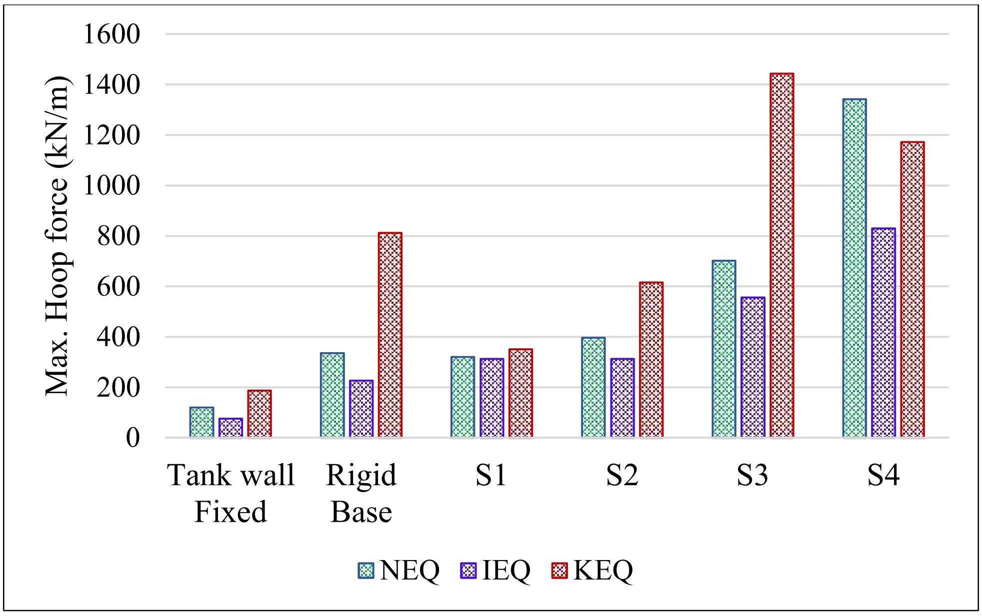



Fig. 7.16 Maximum hoop force of half fill tank on different support conditions

***Bending Moment***

Fig. 7.17 and 7.18 compares maximum bending moment occurring on the tank under the action of the three different earthquakes.

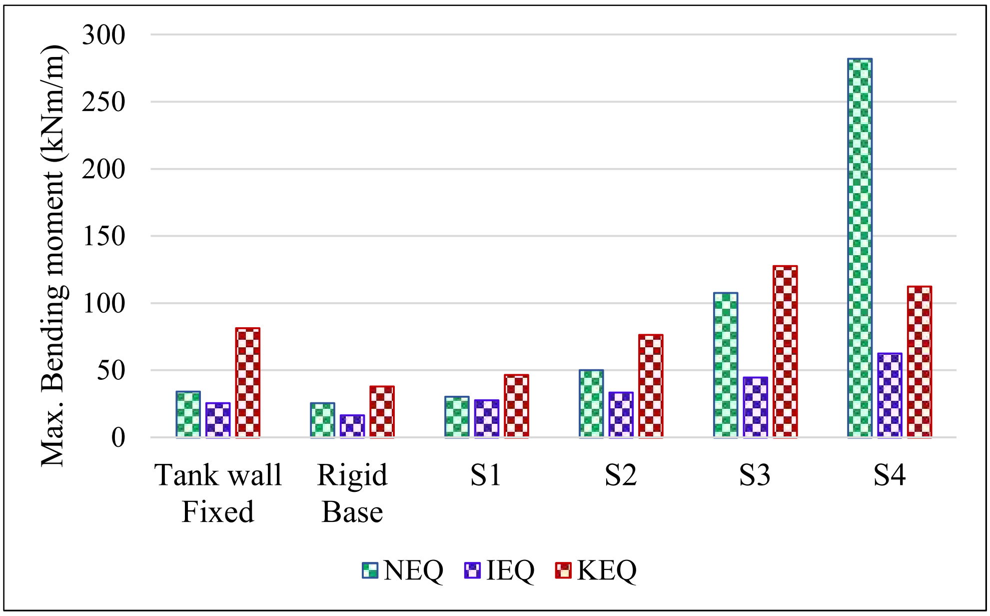



Fig. 7.17 Maximum bending moment of full fill tank on different soil conditions

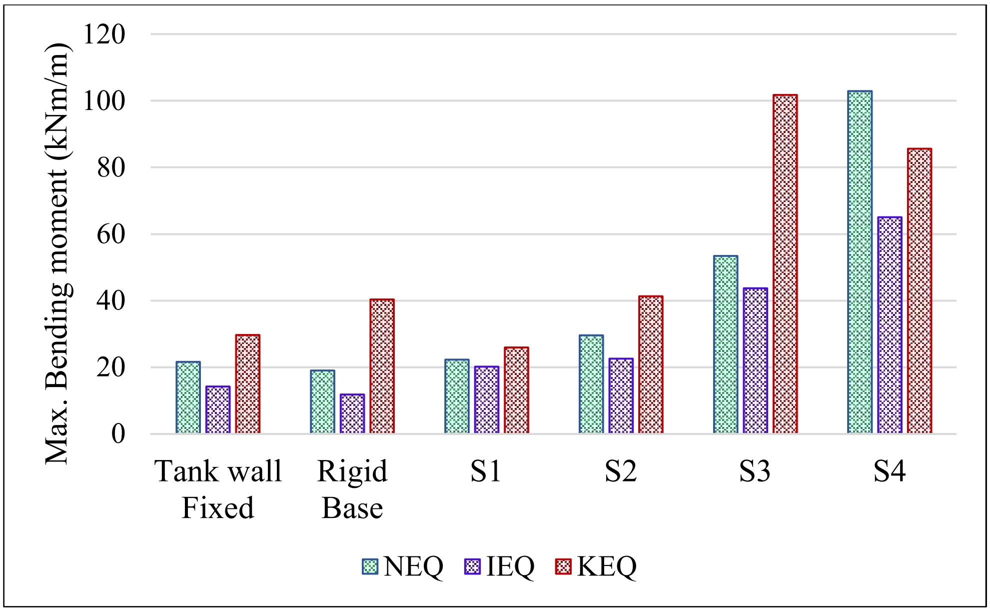



Fig. 7.18 Maximum bending moment of half fill tank on different soil conditions

It can be noticed that bending moment in response to seismic loading of high frequency content Koyna earthquake is more than (16.2 to 241.2 %) that of low and medium frequency content earthquakes for both full fill and half fill tanks with fixed base, rigid base slab and on hard rock (S1), rock (S2) and very dense soil (S3), as reported in the case of hoop force. For full fill tanks on stiff soil (S4), the responses are high for seismic loading having high PGA, Northridge earthquake. As in the case of hoop force, direct relation of maximum bending moment developed to the stiffness of soil on which the tank rests can be observed in response to low and medium frequency content earthquakes. The maximum bending moment of the tank with tank wall fixed at bottom is more than the tank with rigid base slab and tank on hard rock for full fill and half fill conditions except the response of half fill tank under seismic loading of Koyna earthquake.  However, very dense soil and stiff soil has produced maximum bending moment significantly higher than that of idealized fixed condition. In all cases response due to seismic loading of Imperial Valley earthquake is the least.

**Base shear**

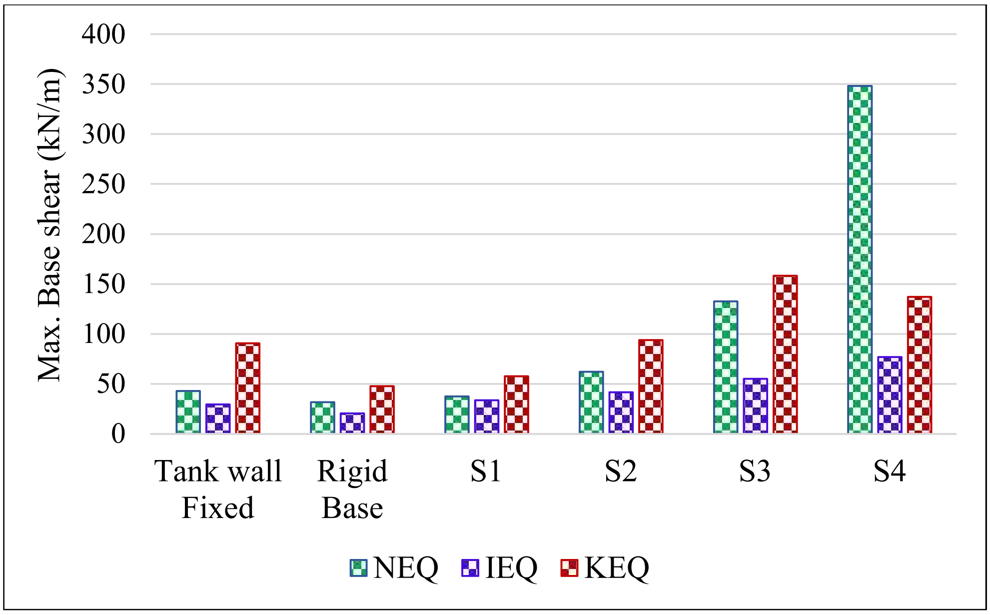



Fig. 7.19 Maximum base shear of full fill tank on different soil conditions

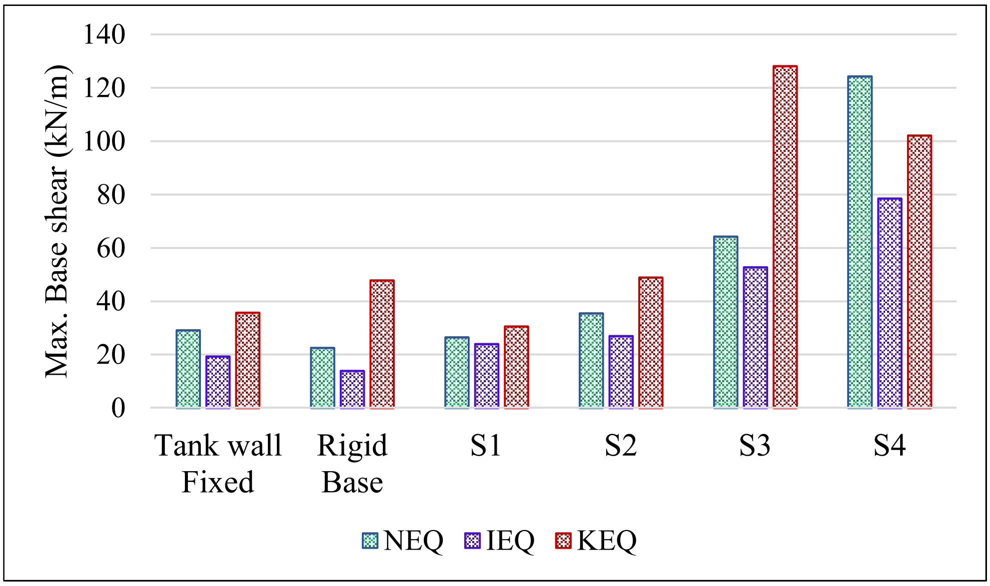



Fig. 7.20 Maximum base shear of half fill tank on different soil conditions

Similar to the hoop and bending moment response, base shear (Fig. 7.19 and Fig. 7.2) of the tank with fixity at bottom, having rigid base slab and tank resting on hard rock, rock and very dense soil, the response to high frequency content Koyna earthquake is more than (15%to 224%) other two earthquakes. For tank on stiff soil, response to low frequency Northridge earthquake of high PGA, is the highest for both full fill and half fill conditions. Here also, maximum base shear is the least in the case of medium frequency content earthquake of low PGA. Direct relation of maximum base shear developed to the stiffness of soil on which the tank rests can be observed in response to low and medium frequency earthquakes, as the response hoop force and bending moment. Base shear of tank with wall fixed at bottom is observed to be higher than the tank with rigid base and tank on hard rock for full fill condition.

The base shear values of the tanks of same dimension for storing drinking water (importance factor =1.5) in Zone V (Zone factor = 0.36) are computed using the parameters suggested in IS 1893 Part2:2014 and IS 1893 Part 1 :2002 for tanks on different soil conditions. The obtained base shear of 3543.45 kN, indicates that average base shear/m is near to the finite element analysis results for tank on hard strata (hard rock, S1 and rock, S2). But for tank on soil with low stiffness, the code underestimates the base shear. Hence it is necessary to incorporate soil-structure interaction effects in seismic response analysis of tanks, especially for tanks resting on soil having low stiffness such as very dense soil (S3) and soft soil (S4).

### 7.8 SUMMARY

Seismic analysis of ground supported tank that includes soil-structure interaction has been carried out. As the stiffness of soil decreases, the fundamental impulsive frequency decreases and amplitude of displacement increases. The influence of water fill condition on fundamental frequency is marginal in the case of tanks founded on soils with low stiffness.. The seismic behaviour of water tank depends on the characteristics of earthquake, soil properties and water fill condition of the tank. Frequency content of earthquake is the governing factor in the transient response of tank on soil with high stiffness, whereas for tanks resting on soil with low stiffness, ground acceleration of the earthquake is the dominating factor. Accordingly, it is essential to include SSI effects in the coupled FSI analysis for evaluation of seismic performance of water tanks.

# CHAPTER 8

# CONCLUSIONS AND SCOPE FOR FURTHER WORK

## 8.1 GENERAL

The dynamic behaviour of ground supported concrete cylindrical water tanks have been studied by performing the free vibration analyses and time history analyses. The necessity to include the water height in the expressions for natural frequency of water tank is established from the free vibration analysis of tanks of aspect ratio varying from 0.2 to 2.0. The effect of aspect ratio, water fill condition and characteristics of earthquake on seismic behaviour of water tanks have been investigated. The role of soil-structure interaction on seismic response of tank has been identified by performing the time history analysis with consideration of fluid – structure and soil – structure interaction effects. The conclusions from the analyses which are applicable to the range of parameters of the study and characteristics of earthquakes are presented in the following sections.

## 8.2 CONCLUSIONS

Conclusions arising out of this research work is discussed under five headings viz; (i) free vibration characteristics of ground supported concrete cylindrical water tanks (ii) effect of water fill condition on seismic response of ground supported water tanks (iii) effect of aspect ratio on seismic response of water tanks (iv) influence of earthquake characteristics on seismic response of tank and (v) role of soil-structure interaction on seismic behaviour of water tanks.

### 8.2.1 Free Vibration Characteristics of Ground Supported Concrete Cylindrical Water Tanks

(i) The fundamental impulsive frequency of tanks filled with design water height increases with increase in aspect ratio from 0.2 to 1.0. whereas the influence of aspect ratio is marginal for tanks of aspect ratio 1.0 to 1.2.

(ii) As the water height decreases from full fill condition to half fill condition, the impulsive frequency increases and reaches a maximum value, but further reduction of water in the tank does not cause significant variation in impulsive frequency. The expressions for frequency suggested by design codes excluding the water fill condition with respect to tank wall height is not sufficient for determination of frequency of partially filled tanks.

(iii) The coefficient proposed and designated as '$C_{i,\ Proposed}$', instead of $C_i$ in the expression for time period of impulsive mode of vibration specified in IS 1893: Part 2 (2014), can be applied to determine the time period of impulsive mode of vibration of ground supported cylindrical water tank for any water fill condition with sufficient accuracy.

### 8.2.2 Effect of Water Fill Condition on Seismic Response of Tanks with Tank Wall Fixed at Bottom

Time history analyses were performed on tanks having height of 12 m with aspect ratio 0.6, 0.8, 1.0, 1.2 and 1.4 to determine the seismic behaviour of the tanks. All tanks have been subjected to horizontal components of three earthquakes (Northridge earthquake, Imperial Valley earthquake and Koyna earthquake) having different characteristics, acting one at a time. Conclusions drawn from the analysis of tanks in three water fill conditions i.e. full fill, half fill and quarter fill conditions, subjected to seismic loadings are given below:

i) Under Northridge earthquake of low frequency content and Imperial Valley earthquake of medium frequency content, all response parameters such as longitudinal displacement, hoop force, bending moment and base shear decreases with decrease in water height.

(ii) The maximum hoop force in half fill tank under Northridge earthquake is 0.63 to 0.81 times of maximum value in full fill condition and for Imperial valley earthquake, the maximum hoop force in half fill condition is 0.49 to 0.64 times of full fill condition, as aspect ratio changes from 0.6 to 1.4.

(iii) Bending moment and base shear decrease with decrease in water height for low and medium frequency content earthquakes. Maximum bending moment in half fill condition is 0.62 to 0.71 of full fill condition, under Northridge earthquake and 0.52 to 0.68 times upon Imperial valley earthquake, depending upon the aspect ratio.

(iv) Under Koyna earthquake of high frequency content, response parameters such as peak hoop force, bending moment and base shear of wider tank with aspect ratio 0.6 in half fill condition is 24%, 32% and 18% more than that in full fill condition.

(v) It is not possible to expect the maximum seismic response always in full fill condition. Depending upon the aspect ratio and frequency content of earthquake, the tanks with water level other than full fill may undergo adverse impact during a seismic event. Hence the water fill condition should take into consideration in the seismic analysis of water tanks.

### 8.2.3 Effect of Aspect Ratio on Seismic Response of Tanks with Tank Wall Fixed at Bottom

(i) Under earthquakes of low and medium frequency contents, peak hoop force decreases with increase in aspect ratio, but such a dependence on aspect ratio cannot be observed for tanks under earthquake of high frequency content.

(ii) For tanks of same height, the maximum hoop force of tank with aspect ratio 0.6 in full fill condition is 3.6 times of tank with aspect ratio 1.4, under Northridge earthquake. Maximum bending moment in aspect ratio 0.6 is 1.33 times of that in aspect ratio of 1.4. Variation in base shear with aspect ratio is insignificant for tanks of all aspect ratios in any water fill conditions.

(iii) Peak hoop force in full fill tank with aspect ratio 0.6 is 3.9 times of tank with aspect ratio 1.4 under Imperial Valley earthquake. Aspect ratio of the tank cannot be related directly to the peak responses of displacement, bending moment and base shear of the water tank subjected to earthquake of medium frequency content. However, the highest response is noticed in tank with the lowest aspect ratio.

(iv) There is no direct relation of longitudinal displacement, hoop force, bending moment and base shear with aspect ratio under Koyna earthquake having high frequency content. For the full fill tank, maximum hoop force is for tank with aspect ratio 1.0 while for half fill and quarter fill condition, maximum occurs for aspect ratio 0.6.

(v) Ratio of peak hoop force due to seismic loading to hydrostatic loading decreases with increase in aspect ratio under earthquakes of low and medium frequency contents.

(vi) As aspect ratio increases, the ratio maximum bending moment due to seismic loading to hydrostatic loading increases under all the three earthquakes considered.

### 8.2.4 Influence of Earthquake Characteristics on Seismic Response of Tanks with Tank Wall Fixed at Bottom

(i) The Koyna earthquake produced the highest response of radial displacement, hoop force, bending moment and base shear for all tanks with aspect ratio greater than 0.8, compared to Northridge earthquakes and Imperial Valley earthquakes, attributable to its highest frequency content.

(ii) Maximum hoop force of tank with aspect ratio 1.0 in full fill condition due to Koyna earthquake is 3.02 and 3.31 times earthquakes of low and medium frequency contents respectively.

(iii) Under high frequency content earthquake, seismic behaviour of the tank is dominated by characteristics of earthquake whereas for earthquake of low frequency content, the characteristics of the tank determines the seismic behaviour.

(iv) Tanks with low aspect ratio in full fill condition, responses are less influenced by the characteristics of earthquake.

### 8.2.5 Role of Soil-structure interaction on Seismic Response

Free vibration analyses and time history analyses were carried out on circular water tank of aspect ratio 1.0 resting on four different soil conditions and compared with that of tank with rigid base. The major conclusions drawn from dynamic analyses of the tanks are:

(i) Fundamental impulsive frequency of the SSI system decreases with decrease in stiffness of soil on which the tank rests.

(ii) Fundamental impulsive frequency of the half fill tank on any soil type is greater than the frequency of full fill tank.

(iii) Under the low and medium frequency content earthquakes, the response parameters such as displacement, hoop force, bending moment and base shear is increased with reduction in soil stiffness.

(iv) Peak hoop force of the full fill tank in hard rock, rock, very dense soil and stiff soil due to low frequency content Northridge earthquake are 0.83, 1.41, 2.79 and 7.19 times of hoop force of tank with rigid base, whereas the bending moment in tanks on respective soil types are 1.18, 1.95, 4.17 and 10.93 times of tank with rigid base.

(v) The reduction in modulus of elasticity from 7000 MPa (hard rock) to 150 MPa (stiff soil) causes an increase of 60% of hoop force for full fill and 165% for half fill conditions, upon medium frequency content Imperial Valley earthquake. The peak bending moment of the full fill tank varies from 1.7 to 3.8 times of tank with rigid base as soil property varies from that of hard rock to stiff soil.

(vi) Under Imperial Valley earthquake, SSI effects dominates for half fill condition, especially for tank on soil with low stiffness.

(vii) Under seismic loading of high frequency content Koyna earthquake, the influence of water height on the responses are marginal for tank rests on soil with low stiffness.

(viii) Of the three earthquakes, for tanks with rigid base and for tanks on hard rock, rock and very dense soil, the highest response parameters such as displacement, hoop force, bending moment are for high frequency content Koyna earthquake, whereas for tank on soil with lowest stiffness (stiff soil), the highest response is for low frequency content Northridge earthquake for which the PGA is high.

(ix) Frequency content of earthquake is the governing factor in the transient response of tank resting on soil with high stiffness, whereas for tanks on low stiff soil, peak ground acceleration predominates.

## 8.3 SCOPE FOR FUTURE WORK

This study forms a part of the investigation on water tanks subjected to seismic loading. The areas on which continued research can be undertaken to provide better understanding of the behaviour of the tank and thus be of more use to the designers are:

(i) Analysis of tanks with different tank wall height for varying aspect ratio

(ii) Studies on convective response and influence of vertical ground acceleration

(iii) Response of water tank supported by piles on soil with low stiffness.

(iv) Studies on steel tanks and tanks with composite materials.

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# APPENDIX

Structural design of tank wall based on IS:3370 (Part 1-2009, Part 2-2009) for tank with outer diameter of 20 m is presented in Table A1. Design has been carried out for resisting maximum hoop tension, bending moment and base shear obtained from static finite element analysis.

Table A1 Design details of tank wall based on static analysis

| | |
|---|---|
| Inner Diameter (D) of tank (m) | 20<br>(aspect ratio 0.6) |
| Maximum Hoop Tension<br>(kN/m) | 697.6 |
| Maximum +ve Bending Moment<br>(kN-m/m) | 33.9 |
| Maximum -ve Bending Moment<br>(kN-m/m) | 68.4 |
| Base Shear<br>(kN/m) | 118.6 |
| Characteristic strength of concrete – 30 MPa<br>Characteristic strength of steel – 415 MPa | |
| Hoop Reinforcement on both faces | 20 mm Φ @ 110 mm c/c |
| Minimum thickness of tank wall for resisting hoop tension (mm) | 418 |
| Vertical reinforcement in inner face | 20 mm Φ @ 200 mm c/c |
| Vertical reinforcement on outer face | 16 mm Φ 250 mm c/c |

# LIST OF PAPERS BASED ON THESIS

**Refereed Journals**

1. Asha, J., and Glory, J. (2018) Dynamic Behaviour and Seismic Response of Ground Supported Cylindrical Water Tanks*, Journal of The Institution of Engineers (India) Ser. A,* 99(3):453–461.

2. Asha, J., and Glory, J. (2019) Fluid - Structure- Soil Interaction Effect on Dynamic Behaviour of Circular Water Tanks, *International Journal of Structural Engineering* (Inderscience), 10(1), 25-39.

3. Asha, J., and Glory, J. (2019) Influence of water fill condition on the seismic response of ground supported circular water tanks, *Journal of Structural Engineering, SERC* (Accepted for publication in Vol 46, No.4).

4. Asha, J., and Glory, J. (2018) Seismic behaviour of circular water tanks under combined Fluid – Structure and Soil – Structure Interaction effects, *Journal of Emerging Technologies and Innovative Research*, September 2018, 5 (9).

5. Asha, J., and Glory, J. (2016) Response of ground supported cylindrical tanks to harmonic loading", *Civil Engineering and Urban Planning: An International Journal,* 3(2).

**Conference Proceedings**

1. Asha, J., and Glory, J. (2016), "Parametric study on dynamic behaviour of ground supported cylindrical tanks", *Proceedings of International conference on Recent Advances in Civil Engineering (ICRACE 2016)*, Kochi, 2016.

# CURRICULUM VITAE

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