Seismic behaviour of cylindrical elevated tanks with a frame supporting system on various subsoil

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The seismic responses of an elevated tank with a frame supporting system on various subsoils are investigated. The finite element method is used for modelling the elevated tank and subsoil system. Fixed-base and elastic media assumptions are made for subsoil. The tank fluid has been modelled as lumped masses such as impulsive and convective masses proposed by Housner. An added mass approach is used for fluid-structure interaction by representing both the impulsive mass and the convective mass. For estimating the seismic response of elevated tanks, response spectrum analysis with mode superposition is used. The results obtained from modelling the elevated tanks on a fixed base and on an elastic medium are compared. It has been observed that the seismic response of the elevated tank is altered significantly depending on the properties of the subsoil. The soil-structure interaction especially affects the impulsive modes and lateral displacement of elevated tanks rather than the torsional modes. It is proposed that the coupling effect for perpendicular directions in the axisymmetrical plan geometry should be considered in the seismic design of the elevated tanks.

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Elevated tanks are the structures frequently used in order to store fluid for not only drinking but also for fire fighting. There has been intensive focus on the seismic safety of lifeline structures; i.e., tanks in seismic regions; particularly those critical structures that may fail during earthquakes and can potentially endanger drinking water, fail to stop large fires and result in substantial economical losses. The 1999 Kocaeli earthquake in Turkey caused significant structural damage to the Tubras refinery itself and an associated tank farm with crude oil and product jetties. Of 112 tanks on the farm six of varying sizes were damaged due to ground shaking and fire. The consequent fire in the refinery and on the tank farm caused extensive additional damage. Fire started in one of the naphtha tanks lasting for three days and endangered the safety of the whole region. During the earthquake this fire initiated as a result of sparks created by bouncing of the floating roof in one of the tanks. The sparks ignited the naphtha. The Production Index dropped by 12.1% and 9%, respectively, during August and September of 1999, resulting in an annual drop of 5% from 1998 level. This is largely attributed to the slowdown in production at the TUBRAS.

All the reasons mentioned above show that this type of structure and its reliability against failure under seismic load are of critical concern.

Upsetting circumstances were experienced through damage to the staging of elevated tanks in some earthquakes which occurred in different regions of the world.

Numerous studies have been done for dynamic behaviour of liquid storage tanks, most of which are concerned with ground level cylindrical tanks. Contrary to this, very few studies exist related to underground and elevated tanks. It is generally assumed that the elevated tanks are fixed to the ground. So, attention is focused on the dynamic behaviour of the fluid and/or supporting structure.

In the 1950s the concept of analyzing probability of elevated water tanks as a single degree of freedom system was suggested. Some codes still recommend this approach. If the tank is completely full of water, vertical motion of the sloshing is prevented, and supporting structure has a uniform rigidity along the height. In this regards, the elevated tank may be treated as a single degree of freedom system or in other words as a normal inverted pendulum. When the fluid in the vessel oscillates, this changes dynamic behaviour of the elevated tanks. It is indicated that with observations of real elevated tanks large errors are involved in using a single-degree-of-freedom system model. Therefore, the methods which consider sloshing in the elevated fluid storage tanks are preferred and used in present study.

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Housner\textsuperscript{8} proposed the equivalent impulsive mass (mass moving firmly with the walls) and convective mass (sloshing mass) to represent the dynamic behaviour of fluid. The impulsive mass is connected to tank walls by rigid links, whereas the convective one by springs. A two-mass model is developed by using this equivalent masses and springs. In this model, walls were assumed as rigid and the rigidity of supporting structure characterized by $k_1$ rigidity which equals to that of the supporting structure for a horizontal force applied at the same height as the mass. Further, applications of two-mass model for elevated tanks have been reported by Sonobe and Nishikawa\textsuperscript{9} and Shepherd\textsuperscript{10}. Housner’s two-mass model has been commonly used for seismic analysis of elevated tanks\textsuperscript{11}.

Haroun and Ellaithy\textsuperscript{3} developed a model which included an analysis of a variety of elevated rigid tanks undergoing translation and rotation; the model includes sloshing modes; and it assesses the effect of tank wall flexibility on the earthquake response of elevated tanks. Resheidat and Sunna\textsuperscript{12} investigated the behaviour of rectangular elevated tank during earthquakes considering soil-foundation-structure interaction. They neglected the sloshing effects on the seismic behaviour of the elevated tanks. Haroun and Temraz\textsuperscript{13} analyzed models of two-dimensional X-braced elevated tanks supported on isolated footings to investigate the effects of dynamic interaction between the tower and the supporting soil-foundation system. While doing this, they also neglected the sloshing effects. Marashi and Shakib\textsuperscript{14} carried out an ambient vibration test in order to evaluate the dynamic characteristics of elevated tanks. Dutta \textit{et al.}\textsuperscript{15,16} studied on the comparisons of the supporting system of elevated tank with reduced torsional vulnerability and they suggested approximate empirical equations to determine the values of lateral, horizontal and torsional stiffness for different frame supporting systems. They also investigated how the inelastic torsional behaviour of the tank system with accidental eccentricity varied in accordance with the increasing number of panel and column\textsuperscript{17}.

Asthana and Sridhar\textsuperscript{18} added fluid mass to the wall to investigate seismic behaviour of elevated tank. Dogangün \textit{et al.}\textsuperscript{19} studied the efficiency of added mass approach for fluid-structure interaction in elevated tanks neglecting subsoil effects. Rai\textsuperscript{5} searched the seismic behaviour and retrofitting of reinforced concrete elevated tanks. Shenton & Hampton\textsuperscript{20} and Shrimali & Jangid\textsuperscript{21,22} investigated seismic response of isolated elevated tanks. These researchers concluded that the base shear of elevated liquid storage tank was significantly reduced due to isolation and proposed approximate methods which accurately predict the peak response of the isolated elevated steel tank with significantly less computational efforts. It is clear from the literature; very few researchers have presented the seismic behaviour of elevated tanks when compared to tens of the studies for ground-supported cylindrical tanks\textsuperscript{23}. But Livaoğlu and Doğangün\textsuperscript{24} proposed a simple analytical procedure for the seismic analysis of fluid-elevated tank-foundation/soil systems and they used this approximation in selected tanks considering fluid-elevated tank-soil/foundation system. Livaoğlu\textsuperscript{25} performed a comparative study of seismic behaviour of the elevated tanks by taking both fluid and soil interaction effects on the elevated tanks into account. Livaoğlu and Doğangün\textsuperscript{26} summarized simplified techniques which are able to determine seismic response of the fluid-elevated tanks-soil/foundation system.

Soil-structure interaction may be more important in elevated tanks due to the fact that most of the masses lumped above the ground and foundation supported on relatively small area. The problem of soil-structure interaction for ground supported cylindrical tanks was addressed by many investigators\textsuperscript{27,28}. But the same condition is not relevant to the elevated tanks. So, there is a need to study the fluid-structure-foundation/soil interaction for elevated tanks. Therefore, the purpose of this study is to investigate the seismic behaviour of elevated tanks with frame supporting system on different subsoils.

**Fluid-Structure-Soil/Foundation Interaction**

The seismic analysis and design of elevated tanks are complicated by fluid, structure, foundation and soil interaction. The model shown in Fig. 1 is used to investigate these interaction problems by using general purpose structural analysis programs.

**Housner’s equivalent spring-mass model for fluid**

The equivalent spring-mass model proposed by Housner regarding dynamic behaviour of fluid inside a vessel is shown in Fig. 2. It is important to note that the factors in Housner’s equations were modified by Epstein\textsuperscript{29}. The fluid is replaced by an impulsive mass $m_i$ and this mass is rigidly attached to the tank wall and a convective mass $m_e$ to the tank wall by linear springs whose total stiffness is $k_2$. This fluid-structure
model is used to account for fluid-structure interaction effects for elevated water tanks in this study.

The impulsive and convective masses for a cylindrical tank are determined from:

\[ m_i = m_t \frac{\tanh \left( \frac{1.74 R}{h} \right)}{\left(1.74 \frac{R}{h}\right)} \];

\[ m_o = m_t \cdot 0.318 \frac{R}{h} \tanh \left( \frac{1.84 h}{R} \right) \] \quad ... (1)

where \( m_t \) is the total mass of the fluid, \( R \) is the inner radius of the vessel and \( h \) is the depth of the fluid. Convective masses of additional higher-mode may also be included if desired. A single convective mass is generally used for practical design of elevated tanks and higher modes of sloshing have negligible influence on the forces exerted on the tank even if the fundamental frequency of the structure is in the vicinity of one of the natural frequencies of sloshing.

Sloshing frequency of \( \omega \) and the stiffness of \( k_2 \) for a cylindrical tank are given by:

\[ \omega^2 = \frac{g}{R} \frac{1.84 \tanh \frac{1.84 h}{R}}{R}; \quad k_2 = m_o \omega^2 \] \quad ... (2)

where \( g \) is the ground acceleration. The impulsive and convective masses are located from the bottom of the vessel at the distances \( h_i \) and \( h_o \) as shown in Fig. 2, respectively; the heights are given by,

\[ h_i = \frac{3}{8} h; \quad h_o = \left[ 1 - \frac{\cosh \left( \frac{1.84 h}{R} \right) - 1}{1.84 \frac{h}{R} \cdot \sinh \left( 1.84 \frac{h}{R} \right)} \right] h \] \quad ... (3)
Soil/foundation-structure interaction

The simulation of the infinite medium in the numerical method is a very important topic in dynamic soil-structure interaction problems. The general method for treating this problem is to divide the infinite medium into the near field (truncated layer), which includes the geometric irregularity as well as the non-homogeneity of the foundation, and the far field, which is simplified as an isotropic homogeneous elastic medium. The near field is modelled using finite elements and the far field is treated by adding some special artificial boundaries or connecting some special elements. The soil is in most cases a semi-infinite medium, an unbounded domain, or so large in extent that the simultaneous modelling together with the structure may be impractical. In a dynamic problem, it may be insufficient to prescribe a zero displacement at a large distance from the structure, as is routinely done in static analysis. But sufficiently a large soil model can define the soil structure interaction as was performed in this study (Fig. 3). Furthermore, reflecting and radiation effects of the propagating waves from the structure-foundation layer may be avoided by a sufficiently large model. It is important to note that as others used approximations, it may be said; the artificial and/or transmitting boundaries with numerical method like finite element or boundary element. Furthermore, reflecting and radiation effects of the propagating waves from the structure-foundation layer may be avoided by means of these types of boundaries. There are different types in frequency or time domain with different sensitivities. Firstly, Lysmer and Kuhlmeyer developed viscous boundaries using one-dimensional beam theory and this theory has been commonly used with the FEM. Then more complex boundary types are used and developed like damping-solvent extraction method and doubly-asymptotic multi directional transmitting boundary.

The most common soil structure interaction (SSI) approach used for three-dimensional soil structure systems is based on the added motion formulation. This formulation is mathematically simple, theoretically correct, and is easy to automate and use within a general linear structural analysis program. Soil/foundation-structure interaction model given in Fig. 1 is considered in this study. The model is divided into three sets of node points. The common nodes at the interface of the structure and foundation are identified as “c”; some within the structure as “s”; and the others within the foundation as “f” nodes. In this figure, the absolute displacement \( U \) is estimated out of the sum of the free field displacement \( v \) and the added displacement \( u \).

From the direct stiffness approach in structural analysis, the dynamic force equilibrium of the system is given in terms of the absolute displacements, \( U \), by the following sub-matrix equation:

\[
\begin{bmatrix}
M_{ss} & 0 & 0 & U_s \\
0 & M_{cc} & 0 & U_c \\
0 & 0 & M_{ff} & U_f
\end{bmatrix}
+ \begin{bmatrix}
K_{ss} & K_{sc} & 0 & U_s \\
K_{cs} & K_{cc} & K_{cf} & U_c \\
0 & K_{fc} & K_{ff} & U_f
\end{bmatrix}
= \begin{bmatrix}
0 \\
0 \\
0
\end{bmatrix}
\]

where the mass and the stiffness at the contact nodes are the sum of the contribution from the structure \((s)\) and foundation \((f)\), and are given by:

\[
M_{ec} = M_{ec}^{(s)} + M_{ec}^{(f)} \quad K_{ec} = K_{ec}^{(s)} + K_{ec}^{(f)}
\]

The three-dimensional free-field solutions are designated by the free field displacements \( v \) and accelerations \( \ddot{v} \). By a simple change of variables it is now possible to express the absolute displacements \( U \) and accelerations \( \ddot{U} \) in terms of displacements \( u \) relative to the free-field displacements \( v \) as given below:

\[
\begin{bmatrix}
U_s \\
U_c \\
U_f
\end{bmatrix}
= \begin{bmatrix}
u_s \\
v_c \\
v_f
\end{bmatrix}
+ \begin{bmatrix}\ddot{u}_s \\
\ddot{u}_c \\
\ddot{u}_f
\end{bmatrix}
\quad \text{and} \quad \begin{bmatrix}
\ddot{U}_s \\
\ddot{U}_c \\
\ddot{U}_f
\end{bmatrix}
= \begin{bmatrix}\ddot{v}_s \\
\ddot{v}_c \\
\ddot{v}_f
\end{bmatrix}
+ \begin{bmatrix}\dddot{u}_s \\
\dddot{u}_c \\
\dddot{u}_f
\end{bmatrix}
\]

Fig. 3—Vertical cross-section of the elevated tank.
Eq. (4) can now be rewritten as

\[
\begin{bmatrix}
M_{ss} & 0 & 0 & \xi_S \\
0 & M_{cc} & 0 & \xi_C \\
0 & 0 & M_{ff} & \xi_F \\
\end{bmatrix}
\begin{bmatrix}
K_{ss} & K_{sc} & 0 \\
K_{cs} & K_{cc} & K_{cf} \\
0 & K_{fc} & K_{ff} \\
\end{bmatrix}
\begin{bmatrix}
\xi_S \\
\xi_C \\
\xi_F \\
\end{bmatrix}
+ \begin{bmatrix}
K_{sc} & 0 \\
K_{cf} & K_{ff} \\
\end{bmatrix}
\begin{bmatrix}
u_s \\
\nu_f \\
\end{bmatrix}
= \begin{bmatrix}
u_s \\
\nu_c \\
\nu_f \\
\end{bmatrix}
\]

\[
\{R\} = \begin{bmatrix}
M_{ss} & 0 & 0 & \xi_S \\
0 & M_{cc} & 0 & \xi_C \\
0 & 0 & M_{ff} & \xi_F \\
\end{bmatrix}
\begin{bmatrix}
K_{ss} & K_{sc} & 0 \\
K_{cs} & K_{cc} & K_{cf} \\
0 & K_{fc} & K_{ff} \\
\end{bmatrix}
\begin{bmatrix}
\xi_S \\
\xi_C \\
\xi_F \\
\end{bmatrix}
= \begin{bmatrix}
u_s \\
\nu_c \\
\nu_f \\
\end{bmatrix}
\]

If the free-field displacement \(v_c\) is constant over the base of the structure, the term \(v_c\) is the rigid body motion of the structure. Therefore, Eq. (7) can be further simplified by the fact that the static rigid body motion of the structure is:

\[
\begin{bmatrix}
K_{ss} & K_{sc} \\
K_{cs} & K_{cc} \\
\end{bmatrix}
\begin{bmatrix}
v_s \\
v_c \\
\end{bmatrix}
= \begin{bmatrix}0 \\
0 \\
\end{bmatrix}
\]

\[
\text{... (8)}
\]

Also, the dynamic free-field motion of the foundation requires that:

\[
\begin{bmatrix}
M_{cc} & 0 & 0 & \xi_c \\
0 & M_{ff} & 0 & \xi_f \\
\end{bmatrix}
\begin{bmatrix}
K_{cc} & K_{cf} \\
K_{fc} & K_{ff} \\
\end{bmatrix}
\begin{bmatrix}
v_c \\
v_f \\
\end{bmatrix}
= \begin{bmatrix}0 \\
0 \\
\end{bmatrix}
\]

\[
\text{... (9)}
\]

Therefore, the right-hand side of the Eq. (7) can be written as

\[
R = \begin{bmatrix}
M_{ss} & 0 & 0 & \xi_S \\
0 & M_{cc} & 0 & \xi_C \\
0 & 0 & M_{ff} & \xi_F \\
\end{bmatrix}
\begin{bmatrix}
\xi_S \\
\xi_C \\
\xi_F \\
\end{bmatrix}
\]

\[
\text{... (10)}
\]

Hence, the right-hand side of the Eq. (7) does not contain the mass of the foundation. Therefore, three-dimensional dynamic equilibrium equations for the complete soil-structure system with damping added, are of the following form for a lumped mass system:

\[
M \ddot{\xi} + C \dot{\xi} + K\xi = -m_x \ddot{x} - m_y \ddot{y} - m_z \ddot{z} \text{ ... (11)}
\]

where \(M, C\) and \(K\) are the mass, damping and stiffness matrices of the soil/foundation-structure model, respectively, \(\xi\) are the added, relative displacements, \(\xi_s\) exist for the soil-structure system and must be set to zero at the sides and bottom of the foundation. The terms of \(\ddot{x}, \ddot{y}, \text{ and } \ddot{z}\) are the free-field components of the acceleration if the structure is not present. The column matrices, \(m_x\), are the directional masses for the added structure only.

In addition, the formulation is valid for free-field motions caused by earthquake waves generated from all sources. The method requires that the free-field motions at the base of the structure be calculated before the soil structure interaction analysis. Most structural analysis computer programs automatically apply the seismic loading to all mass degrees-of-freedom within the computer model and cannot solve the soil-structure interaction problem. This lack of capability has motivated the development of the massless foundation model. This allows the correct seismic forces to be applied to the structure; however, the inertia forces within the foundation material are neglected. To activate the soil-structure interaction within a general purpose structural analysis program it is only necessary to identify the foundation mass in order that the loading is not applied to that part of the structure. The SAP2000 general purpose structural analysis program has been selected to consider not only soil/foundation-structure interaction but also fluid-structure interaction for the sample elevated tanks. In the model that takes into soil structure and fluid structure interaction effect account, the soil beneath the structure can be modelled with massless-solid elements. For fluids, it can be modelled by using the mass approximation of Housner that was modified by Epstein. To model fluid structure-foundation soil model with general purpose structural analysis programs.

**Finite Element Models And System Data**

R/C elevated tank that has frame supporting system with a vessel capacity of approximately 900 m³ in Fig. 4 is considered. This tank has been accepted as a standard project by Ministry of Public Works and Settlement of Turkey. The tank vessel is intez type, (i.e., below the cylindrical vessel is a conical shell with a dome shaped tank floor that provides an economical advantage if not thick floor slabs in elevated tanks have to be used). Since it has an optimal load balancing shape the intez-type vessels are widely preferred.

Finite element method is used for modelling the elevated tanks as shown in Fig. 3. Degrees of freedom were fixed at the base nodes and left free at the others for the one called fixed-base system. Finite element meshes were used to model the subsoil for the other
In the finite element models, massless foundation approach (Fig. 1) was applied to represent the soil-structure interaction. 3D finite element meshes (Fig. 3) are generated and intended to model the influence of fluid-structure and soil-structure effects on the seismic behaviour of elevated tank. A parametric study is carried out to determine the distance of the boundary from the tank. For this purpose three different models with three different mesh type of soil medium are used. These models and mesh types are illustrated in Fig. 5. A parametric study is performed by using transient analysis with direct integration technique to decide the dimension of the soil medium of the model and to select the mesh type of the soil medium. Firstly, the models were analyzed with 25 m diameter of the soil medium and then this value was increased as 5 m for each test model. Finally, it is seen that the subsoil beyond this boundary does not affect the seismic behaviour of the tank. In other words, displacements at the nodes on the lateral boundaries are almost zero. The common nodes on the interface of the structure and foundation are free but the soil nodes on the bottom boundaries are fixed. It is important to note here that all test model illustrated in Fig. 5 give almost same result. So, there is not a disadvantage of them and can be easily used for the analyses carried out in this study.

Columns and beams were modelled as frame elements and vessel walls and truncated cone as quadrilateral shell elements. Because of lack of rotational freedom capability of brick element on the soil-structure interaction surface, foundation is modelled using the shell element and soil with the isoparametric 8 node-brick element that has three translational degrees of freedom per node. The rotational DOF of the shell elements at the common nodes with the brick elements are neglected. It should be noted that plate element also can be used easily.

Fluid-structure interaction problems can be investigated by using added mass, Lagrangian, Eulerian and Lagrangian-Eulerian approaches in the finite element method. Added mass approach was selected for fluid-tank interaction. Equivalent impulsive and convective masses proposed by Housner were used for the determination of added masses. These masses for fluid were calculated as \(m_i = 591.870 \text{ kg} \) and \(m_o = 210.300 \text{ kg} \) via Eq. (1). By using Eq. (3), their heights from vessel ground level \(h_i\) and \(h_o\) were determined as 3.0 m and 5.26 m, respectively. These parameters were obtained for intze-type tank vessel by considering it as an equivalent cylindrical tank. It was shown that errors associated with such an approximation were small and model parameters corresponding to equivalent cylindrical tank for intze tanks could be used for the design purposes. Impulsive mass is added to the unit mass of finite elements for vessel wall and truncated invert cone suitable for the height level of impulsive mass. Convective mass was connected to the finite element...
joints at the $h_o$ level by the stiffness of $k_2 = 623.44$ kN/m determined by Eq. (2).

Young’s modulus and unit weight of concrete were taken to be 32,000 MPa and 25 kN/m$^3$, respectively. The vessel was filled with the water density of 1.000 kg/m$^3$. Soil conditions, Young’s modulus and Poisson’s ratios accounted for this study were given in Table 1 32. Types of subsoil defined in EC8-Part 1 33 were considered and the values of Young’s modules for soil are selected to be suitable for these classes.

The analyses were carried out taking four different subsoil classes as subgrade medium. Subsoil classes regarded in this study can be classified as subsoil class of A, B, C, and D defined in the Eurocode-8. Soil properties accounted for this paper are given in Table 1. The soil characteristics in Table 1 can be calculated with well-known equations

$$G = E / (1 + \nu) \quad \text{and} \quad v_s = \sqrt{G / \rho} \quad \text{where, } \rho \text{ is the mass density and } G \text{ is the shear modulus.}$$

<table>
<thead>
<tr>
<th>Subsoil</th>
<th>Young’s modulus $E$, [MPa]</th>
<th>Poisson ratio $\nu$</th>
<th>Shear wave velocity $v_s$, [m/s]</th>
<th>Subsoil class (from EC 8)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Limestone</td>
<td>30,000</td>
<td>0.2</td>
<td>2476</td>
<td>A</td>
</tr>
<tr>
<td>Granite partially decomposed</td>
<td>7,000</td>
<td>0.2</td>
<td>1196</td>
<td>A</td>
</tr>
<tr>
<td>Very dense glacial till</td>
<td>2,000</td>
<td>0.2</td>
<td>639</td>
<td>B</td>
</tr>
<tr>
<td>Sandy clay</td>
<td>500</td>
<td>0.3</td>
<td>307</td>
<td>C</td>
</tr>
<tr>
<td>Hard clay</td>
<td>150</td>
<td>0.3</td>
<td>177</td>
<td>D</td>
</tr>
<tr>
<td>Medium clay</td>
<td>100</td>
<td>0.3</td>
<td>145</td>
<td>D</td>
</tr>
</tbody>
</table>

Response modification factor was selected as 2. Commonly building systems have an average value of 6, which is reduced to one-third for elevated tank systems. This means that the design forces determined for these systems are about three times larger than those of a building system having similar dynamic properties.

The damping values for the reinforced concrete tanks are taken as 5% for impulsive mode and 0.5% for convective mode as recommended in most literature 29,38. The 0.5% damped spectra for convective mode is available in the EC8-Part 1 39. In this study, 5% of damping ratio is considered for the models. The design response spectrums are drawn in Figs 6a and 6b for 5% and 0.5% damping, for subsoil classes defined in EC-8 Part 1 that is A, B, C and D, with structure performance factor of 2. It should be noted for convective mode that the elastic spectrum may be used as the convective mass which mostly does not excite the structure into inelastic range. It should be explained that a performance factor of 2 is considered in the study as it is recommended for elevated tanks by EC-8 Part-1 39.

Seismic analysis of the elevated tanks

A total of twelve soil-elevated tank-fluid systems were taken into account for seismic analysis. These systems were given a name from System I through System XII to symbolize the models. Influence of the subsoil is ignored for the first system, System-I and a fixed support is assumed. And for the rest from System II to System XII, the influence of the subsoil...
is taken into account, and subsoil is also divided into finite elements having eleven different Young’s modulus, and three different Poisson ratios given in Table 1.

In this study, to judge the validity of the applied model which is named as massless foundation methods, result obtained analysis of 3D finite element model with artificial boundary or viscous boundary are used. For comparison, transient analyses are performed by using the North-South component of 1999 Kocaeli Earthquake (Yarımca record) for the considered elevated tank models. As can be seen from Figs 7 and 8, both approximations gave almost same result for roof displacements and base shear deviations, i.e., while the maximum displacement obtained from the massless foundation model in A subsoil class are calculated as 0.087 m, from the other more rigorous approximation is evaluated as 0.089, similarly for D subsoil class this results are obtained 0.111 and 0.118 respectively. When the time history of the base shear is evaluated, same comparison can be made easily, almost coincided deviation is seen in the illustrations for both A and D subsoil classes. These illustrations show that using the massless foundation approximation cannot cause the large errors or misleading result for the scope of investigated model in this research.

Response spectrum analyses were carried out using SAP2000 package program. First twelve modes are taken into account in the modal analysis. SRSS and CQC methods were used for mode superposition and the practically same results were determined from both methods as similarly mentioned in the study by Shrimali and Jangid. From these results estimated period values, order of the modes and contribution of the modal mass for selected systems are presented in Table 2. Because it was aimed for using the response spectrum to investigate how soil-structure and fluid-structure interactions change the dynamic behaviour of structure with known modal characteristics, it was assumed free field motion at the bottom of foundation level. All the models were subjected to the same seismic effect. But it is important to note that for design process of such structures considering interaction effects researcher or designer must consider wave propagation effects by comparison with other free field analysis results.

Periods for two modes given in Table 2 have the same value for the 1st and the 2nd sloshing modes and for the 3rd and the 4th impulsive modes with perpendicular directions in the axisymmetrical plan geometry of the elevated tank. As can be seen from Table 2 although the values of the period for sloshing modes increase from 3.68 s to 3.79 s, the values of the

![Fig. 7](image1) ![Fig. 8](image2)

**Fig. 7**—For two different approximation, variations of the roof displacement in time for two different subsoil classes

**Fig. 8**—For two different approximations, variations of the base shear force in time for two different subsoil classes
### Table 2—Modal properties such as periods, mode orders and % contributions of the model masses

<table>
<thead>
<tr>
<th>Systems</th>
<th>Mod properties</th>
<th>Sloshing Mode</th>
<th>Horizontal mode for structure</th>
<th>Torsional mode</th>
<th>HM for CSS</th>
<th>Vertical mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>System-I:</td>
<td>Period, (s)</td>
<td>3.68</td>
<td>1.03</td>
<td>0.19</td>
<td>1.01</td>
<td>0.14</td>
</tr>
<tr>
<td>Fixed-base</td>
<td>Mode order</td>
<td>1st &amp; 2nd</td>
<td>3rd &amp; 4th</td>
<td>6th &amp; 7th</td>
<td>5th</td>
<td>10th</td>
</tr>
<tr>
<td>System-IV:</td>
<td>Period, (s)</td>
<td>3.68</td>
<td>1.04</td>
<td>0.19</td>
<td>1.01</td>
<td>0.14</td>
</tr>
<tr>
<td>System-V:</td>
<td>Period, (s)</td>
<td>3.68</td>
<td>1.04</td>
<td>0.19</td>
<td>1.01</td>
<td>0.14</td>
</tr>
<tr>
<td>System-VI:</td>
<td>Period, (s)</td>
<td>3.69</td>
<td>1.13</td>
<td>0.22</td>
<td>0.15</td>
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</tr>
<tr>
<td>Sandy clay-</td>
<td>Mode order</td>
<td>1st &amp; 2nd</td>
<td>3rd &amp; 4th</td>
<td>6th &amp; 7th</td>
<td>5th</td>
<td>10th</td>
</tr>
<tr>
<td>System-VII:</td>
<td>Period, (s)</td>
<td>3.69</td>
<td>1.19</td>
<td>0.23</td>
<td>0.15</td>
<td>---</td>
</tr>
<tr>
<td>Sandy clay-</td>
<td>Mode order</td>
<td>1st &amp; 2nd</td>
<td>3rd &amp; 4th</td>
<td>8th</td>
<td>5th</td>
<td>---</td>
</tr>
<tr>
<td>System-VIII:</td>
<td>Period, (sec.)</td>
<td>3.70</td>
<td>1.26</td>
<td>0.25</td>
<td>0.16</td>
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<tr>
<td>Hard clay-</td>
<td>Mode order</td>
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<td>3rd &amp; 4th</td>
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<td>9th &amp; 10th</td>
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<tr>
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<td>Period, (sec.)</td>
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<td>0.16</td>
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<td>3rd &amp; 4th</td>
<td>6th &amp; 7th</td>
<td>9th &amp; 10th</td>
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<tr>
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<td>Period, (sec.)</td>
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<td>0.17</td>
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<td>1st &amp; 2nd</td>
<td>3rd &amp; 4th</td>
<td>6th &amp; 7th</td>
<td>9th &amp; 10th</td>
<td>5th</td>
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<td>System-XI:</td>
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<td>0.29</td>
<td>0.17</td>
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<tr>
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<td>Mode order</td>
<td>1st &amp; 2nd</td>
<td>3rd &amp; 4th</td>
<td>6th &amp; 7th</td>
<td>9th &amp; 10th</td>
<td>5th</td>
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<tr>
<td>System-XII:</td>
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<td>1.84</td>
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<td>Medium clay-</td>
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<td>3rd &amp; 4th</td>
<td>7th &amp; 8th</td>
<td>9th &amp; 10th</td>
<td>5th</td>
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<tr>
<td>System-XIII:</td>
<td>Period, (sec.)</td>
<td>3.80</td>
<td>1.96</td>
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<td>0.19</td>
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When the contribution of modal masses for the analyzed systems are investigated, it can be seen from Table 2 that the contribution of the convective mass for excited direction of the systems reaches 20%, and the contributions for impulsive modes range 44-76%. As seen from Table 2, one of the most important result is that the coupling effects for the 3rd and the 4th impulsive modes of the systems in two directions (excitation direction and perpendicular to the excitation direction) in the axisymmetrical plan geometry can be more effective with increasing softness of the soil (subsoil classes like C and D). In other words, while contribution of the 3rd and 4th impulsive modal masses for fixed-base system are 76% for excitation direction and 1% for perpendicular direction to the excitation, these values for System VIII were 50% and 26%, respectively. So, the coupling effects must be taken into consideration in dynamic analysis of soil-elevated tank-fluid systems, especially, for soft soils.

Subsoil classes were generally defined in earthquake codes with similar subsoil class definition. But as can be seen from Table 1, different Young’s modulus between the intervals from 35 MPa to 100 MPa were proposed for hard clay. Furthermore, massive volcanic rocks and hard rock are in the same soil group of A in the Turkish Earthquake Code. When the soil is considered as an
elastic media, changes in properties of the soil like Young’s modules and Poisson ratio affect not only the values of periods but also deterioration on the row regularities for vibration modes except for torsional mode. For example, period value of the 8th mode for Systems of VIII, IX, X and XI in which soil may be assumed soft is for vertical vibration mode. But this period value for the other systems is of horizontal mode of the vessel scope shaft. When first seven systems are investigated (Table 2.) it can be seen that the 8th and the 9th modes relating to the vessel scope shaft do not efficiently affect the dynamic behaviour of systems whereas the 10th mode is more important than the 8th and 9th modes for design. Furthermore, vertical mode only occurs at the 8th and 6th vibration modes of the last five systems (defined as soft soil). So, this indicates that choosing the actual values for the properties of the subsoil is very important for seismic design of elevated tanks. Besides, the results may include some important deficiencies, if only first few modes are included in the design.

The relationship between periods in the first five modes and Young’s modulus of soil for the systems can be seen from Fig. 9 in the logarithmic scale. The periods for convective and impulsive modes decrease with increasing Young’s modulus for soil. This decrease was dramatic in the interval of 35 MPa-500 MPa values of Young’s modulus for impulsive mode. When the Young’s modulus increase on the interval of 500 MPa to 7 GPa, the same decrease occurs gradually. For the larger values of Young’s modulus the periods of all modes have practically the same value as the fixed-base system. But the decrease of the sloshing periods (1st & 2nd) is relatively small compared to the period for impulsive mode as in Fig. 9. Here, the period value for torsional modes of the elevated tank is not considerably affected by the properties of the subsoil.

Lateral displacements of the systems I, III, VI, VII, XI and XII are shown in Figs 10 and 11 to avoid confusion. Fig. 10a presents the lateral displacements of the selected soil-elevated tank-fluid systems in excited direction. Here, the lateral displacement of the fixed-base system has the smallest value of 59 mm and System-XI yields the largest value of 173 mm. Similarly, Fig. 10b presents the lateral displacement occurred in the perpendicular direction to the excitation direction. Again the smallest value is obtained for the fixed-base system as 11 mm, System-XII gives the largest value of 142 mm in this direction. Thus, misleading results can be determined as seen from Figs 10a and 10b, if one rules out the coupling effects.

For this reason coupling effect was considered in this paper and the results calculated for the two directions is
Fig. 11—Final lateral displacements of the elevated tank systems; System I ( ), System III ( - - - - ), System VI ( - - - - ), System VII ( - - - - ), System XI ( - - - - ), System XII ( - - - - ) superposed with the both SRSS and CQC techniques. Fig. 11 presents one of the final lateral displacements because SRSS and CQC techniques give the same results in the studied models. As would be expected; the smallest value of 60 mm is recounted for the fixed-base system and System-XII gives the largest value of 210 mm. The deviation between analysis including the coupling effects and ignoring the coupling effect is 1.7% for the fixed base system, while it is 35% for System XII.

Following comments may be drawn from Figs 10 and 11. Maximum lateral displacement occurs at the top of the elevated tank for System-XII whereas it occurs at the intersection of truncated invert cone and vessel wall for fixed-base system. Therefore, deformed shapes of these systems are significantly different from each other. Maximum displacement for System-XII is 3.5 times larger than that of fixed-base system and allowable displacement was exceeded for System-IV to XII. In some cases these large displacements may cause instability of the elevated tanks, although internal forces have small values. The first three systems (I to III) have practically the same lateral displacement.

Maximum roof displacements for excitation (x) and perpendicular to the excitation (y) directions are shown in Fig. 12a. Final roof displacement of the elevated tank is shown in Fig. 12b. As can be seen from Fig. 12 the coupling effect is more pronounced with the value of the Young's modulus less than 500 MPa. Contrary, the base shear and overturning moment decreased and larger for the values than 7000 MPa, they practically had the same value as was obtained for fixed-base system (Figs 13 and 14).

Maximum base shear forces for excitation (x) and perpendicular to the excitation (y) direction at the supporting structure of the elevated tank are presented on normalized form according to the fluid weight in

![Fig. 12](image1.png)  
**Fig. 12**—Roof displacements of the elevated tank (a) for excitation (x) ( ◊ ) and perpendicular to the excitation (y) directions ( □ ) directions, (b) final roof displacement of the elevated tank.

![Fig. 13](image2.png)  
**Fig. 13**—Normalized base shears for the elevated tank (a) for excitation (x) ( ◊ ) and perpendicular to the excitation (y) directions ( □ ), (b) normalized superposed base shears of the elevated tank.
As can be seen from Figs 13 and 14, the values of the base shear and the overturning moment are the highest of all when value of Young’s modulus is equal to 500 MPa. Coupling effect is also important for this type of structures. For example the base shear of the system VII increases from 3068 kN to 4135 kN in case of the coupling effects are considered and the deviation is almost 35%.

Conclusions
The following conclusions are drawn from the study:
(i) Variations of the displacements along the height of the elevated tank are rather different from each other for the fixed base system and for the systems with low Young’s modulus. In fact, it is observed that the elevated tanks supported on an elastic medium having relatively low value of Young’s modulus may have displacement larger than the allowable limits. In some cases, the displacements are so large that the elevated tanks can loose stability even though the internal forces are small.
(ii) The base shear and bending moment at the base of supporting structure increases as the Young’s modulus increases and Poisson’s ratio decreases.
(iii) The periods for convective (sloshing) modes were typically long and were less influenced by the foundation displacement. But this does not mean that sloshing effects can be ignored for seismic design of elevated tanks. The contribution of the convective modes may reach to the significant values. It may be considered for the roof design and details.
(iv) The periods for impulsive modes were significantly influenced by the rigidity of the soil. But effect was getting smaller as Young’s modulus increases. The periods of impulsive modes were not influenced by the Young’s modulus if it was higher than 2000 MPa. Additionally, another mode that could be called as torsional mode was not considerably affected by the properties of the subsoil in all cases.
(v) Any change in Young’s modulus and Poisson ratios of the soil affects not only the values of the periods but also the mode orders. This indicates that considering the first few modes only may cause important deficiencies in the design. In fact, vertical modes tend to be included within the first ten modes if the value of the Young’s modulus is low.
(vi) The seismic behaviour of elevated tanks supported on soft soil, especially if the Young’s modulus is less than 50 MPa, was sensitive to the contribution of modal masses and mode order of the systems. In this interval truncation of the soil domain may affect the behaviour of the elevated tanks. Effects of horizontal, rocking and vertical motion increase as the stiffness decreases. It is observed that the horizontal and rocking motion are the most significant factors for the elevated tanks. For this reason, further studies should be performed in the inelastic range to reveal actual seismic response of elevated tanks supported on such soils.
(vii) When the perpendicular modes are evaluated, it is true that the modes at perpendicular directions are
not taking place in the same time. But earthquake motion has 3D-components, so if perpendicular modes a system have ~15% mass participation as being in this study, the perpendicular modes have critical importance and during the design process this must be considered. This may not be needed for a regular building system. But for the system investigated in this study. The results show that the coupling effect may be critical.

The coupling effects for excitation and perpendicular to excitation direction should be considered in the seismic design of the soil/foundation-erated tank-fluid systems. Otherwise the design may include significant deficiencies.

References
7 Guide to the analysis design and construction of oncrete-pedestal water tower, American Concrete Institute ACI 371R, 1995.