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APPLICATION OF MASSLESS FOUNDATION MODEL TO FLUID-ELEVATED TANKS-SOIL/FOUNDATION SYSTEM

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ABSTRACT

Soil-structure interaction effects play an important role on behavior of the structures especially for special types like towers, tanks, reactors etc. These effects generally take place more critically for the slender structures and structures founded relatively small area as an elevated tank than the other types. Mostly, to determine the effects of interaction are a complex phenomenon, there should be an easy way to consider it with the general purpose programs. For this purpose, the massless foundation approximation is comparatively investigated in this study with the relatively more rigorous approximation of FEM with viscous boundary. Finally it was seen that massless foundation approximation gave very close results to the other methods used in this study. Also it should be said as a result of the study for the structure types and the soil types investigated the methods can be easily used.

Keywords: Massless Foundation, Elevated Tanks, Soil-Structure- Fluid Interaction

INTRODUCTION

Tanks are the structures frequently used in order to store fluid for not only drinking but also for fire fighting. Some example of these upsetting experience occurred during the 1999 Kocaeli earthquake in Turkey. i.e. the earthquake caused significant structural damage to the Tupras 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 (Scawthorn, 1999). The Production Index dropped by 12.1% and 9%, respectively, in the 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 TUPRAŞ (Akgiray et al., 2003). 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 (Haroun and Ellaithy, 1985).

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 behavior of the fluid and/or supporting structure. The soil-structure interaction effects on the dynamic behavior of the tanks are not discussed in these studies. Furthermore, how the seismic analysis of all soil-structure-fluid systems can be practically carried out has not been investigated for this type of tank.
In the 1950 s the concept of analyzing probability of elevated water tanks as a single degree of freedom system was suggested (Chandrasekaran and Krishna, 1954). When the fluid in the vessel

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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 for the scope of this paper. Housner (1963) proposed the equivalent impulsive mass (mass moving firmly with the walls) and convective mass (sloshing mass) to represent the dynamic behavior 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 k1 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 were reported by Sonobe and Nishikawa (1969) and Shepherd (1972). Housner's two-mass model has been commonly used for seismic analysis of elevated tanks (Priestley et al., 1986).

Rai (2002) searched the seismic behaviour and retrofitting of reinforced concrete elevated tanks. Shenton and Hampton (1999) and Shrimali and Jangid (2003a, 2003b) 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 synthesis above; very few have presented the seismic behaviour of elevated tanks when compared to tens of the studies for ground-supported cylindrical tanks (Rammerstorfer et.al. 1990). But Livaoğlu and Doğangün (2005) 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 systems. Livaoğlu (2005) performed a comparative study of seismic behavior of the elevated tanks by taking both fluid and soil interaction effects on the elevated tanks into account. Finally Livaoğlu and Doğangün (2006) summarized simplified techniques simply 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 (Haroun and Abdel-Hafiz, 1986; Haroun, and Abou-Izzeddine, 1992).

It is easy to say from above-mentioned studies that practical methods are also needed on taking into account soil-structure interaction effects for the Fluid-Elevated Tanks-Soil/Foundation Systems. So, it is necessary that new studies need to be carried out in connection with fluid-structure-foundation/soil interaction for elevated tanks. Therefore, the purpose of this study was selected to investigate the influence of the massless foundation for seismic behaviour of elevated tanks with frame supporting system on different subsoils.

SOIL-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 (Wolf and Song, 1996). The near field is modeled 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 (Nofal, 1998). But sufficiently a large soil model can define the soil structure interaction as shown in Fig.3. Furthermore, reflecting and radiation effects of the propagating waves from the structure-foundation layer may be avoided by an adequately large model. It is important to note that as other used approximations it may be said; the artificial and/or transmitting boundaries with numerical method like finite element or boundary element etc. 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 boundary types in frequency or time domain with different sensitivities. Firstly, Lysmer and Kuhlmeyer (1969) developed viscous boundary 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, Doubly-Asymptotic Multi Directional Transmitting Boundary and etc.

Massless Foundation Approach

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 paper. 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*"; others 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*). It is worth to say here that Soil/foundation mass is not fully ignored in this approach, foundation part may be introduced into the introduced part "structure", but mass of soil modeled behind the foundation is ignored.

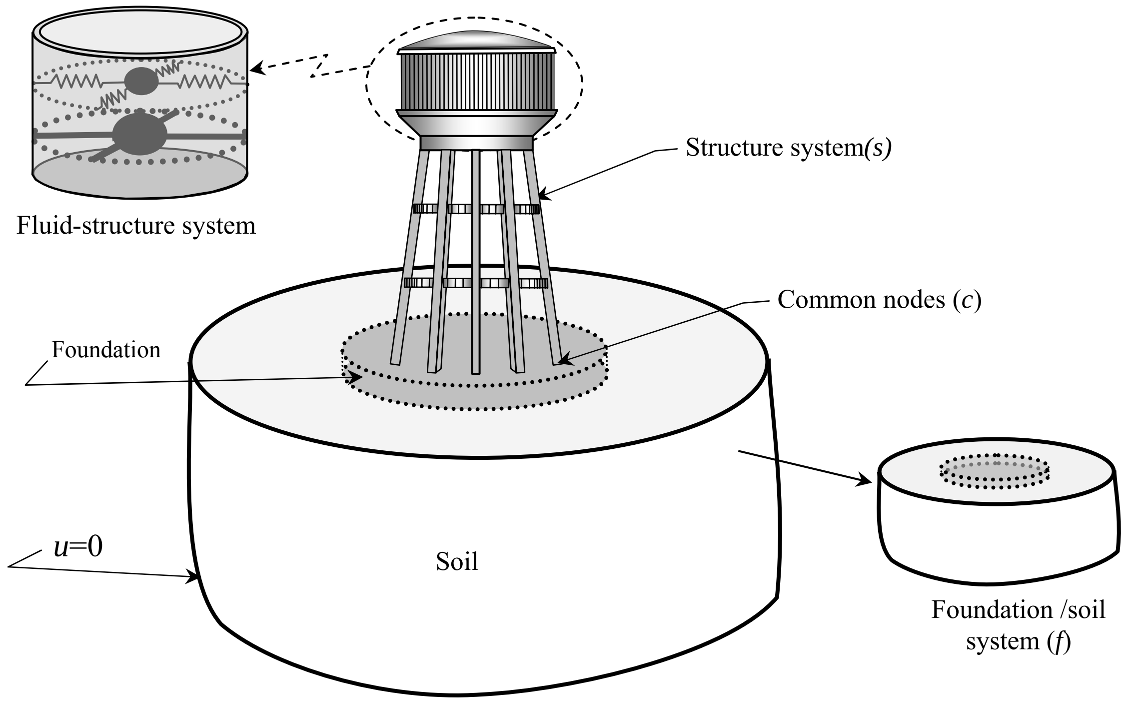


Figure 1. Considered fluid-structure-foundation/soil interaction model

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:

$$\begin{bmatrix} \mathbf{M}_{ss} & 0 & 0 \\ 0 & \mathbf{M}_{cc} & 0 \\ 0 & 0 & \mathbf{M}_{ff} \end{bmatrix} \begin{bmatrix} \ddot{U}_s \\ \ddot{U}_c \\ \ddot{U}_f \end{bmatrix} + \begin{bmatrix} \mathbf{K}_{ss} & \mathbf{K}_{sc} & 0 \\ \mathbf{K}_{cs} & \mathbf{K}_{cc} & \mathbf{K}_{cf} \\ 0 & \mathbf{K}_{fc} & \mathbf{K}_{ff} \end{bmatrix} \begin{bmatrix} U_s \\ U_c \\ U_f \end{bmatrix} = \begin{bmatrix} 0 \\ 0 \\ 0 \end{bmatrix} \tag{1}$$

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;

$$\mathbf{M}_{cc} = \mathbf{M}_{cc}^{s} + \mathbf{M}_{cc}^{f} \qquad \mathbf{K}_{cc} = \mathbf{K}_{cc}^{(s)} + \mathbf{K}_{cc}^{(f)} \tag{2}$$

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:

$$\begin{bmatrix} U_s \\ U_c \\ U_f \end{bmatrix} \equiv \begin{bmatrix} u_s \\ u_c \\ u_f \end{bmatrix} + \begin{bmatrix} v_s \\ v_c \\ v_f \end{bmatrix} \text{ and } \begin{bmatrix} \ddot{U}_s \\ \ddot{U}_c \\ \ddot{U}_f \end{bmatrix} \equiv \begin{bmatrix} \ddot{u}_s \\ \ddot{u}_c \\ \ddot{u}_f \end{bmatrix} + \begin{bmatrix} \ddot{v}_s \\ \ddot{v}_c \\ \ddot{v}_f \end{bmatrix} \tag{3}$$

Eq. (1) can now be rewritten as

$$\begin{aligned} &\begin{bmatrix} \mathbf{M}_{ss} & 0 & 0 \\ 0 & \mathbf{M}_{cc} & 0 \\ 0 & 0 & \mathbf{M}_{ff} \end{bmatrix} \begin{bmatrix} \ddot{u}_s \\ \ddot{u}_c \\ \ddot{u}_f \end{bmatrix} + \begin{bmatrix} \mathbf{K}_{ss} & \mathbf{K}_{sc} & 0 \\ \mathbf{K}_{cs} & \mathbf{K}_{cc} & \mathbf{K}_{cf} \\ 0 & \mathbf{K}_{fc} & \mathbf{K}_{ff} \end{bmatrix} \begin{bmatrix} u_s \\ u_c \\ u_f \end{bmatrix} = \\ &-\begin{bmatrix} \mathbf{M}_{ss} & 0 & 0 \\ 0 & \mathbf{M}_{cc} & 0 \\ 0 & 0 & \mathbf{M}_{ff} \end{bmatrix} \begin{bmatrix} \ddot{v}_s \\ \ddot{v}_c \\ \ddot{v}_f \end{bmatrix} - \begin{bmatrix} \mathbf{K}_{ss} & \mathbf{K}_{sc} & 0 \\ \mathbf{K}_{cs} & \mathbf{K}_{cc} & \mathbf{K}_{cf} \\ 0 & \mathbf{K}_{fc} & \mathbf{K}_{ff} \end{bmatrix} \begin{bmatrix} v_s \\ v_c \\ v_f \end{bmatrix} = \{\mathbf{R}\} \end{aligned} \tag{4}$$

If the free-field displacement v_c is constant over the base of the structure, the term v_s is the rigid body motion of the structure. Therefore, Eq. (4) can be further simplified by the fact that the static rigid body motion of the structure is:

$$\begin{bmatrix} \mathbf{K}_{ss} & \mathbf{K}_{sc} \\ \mathbf{K}_{cs} & \mathbf{K}_{cc}^{s} \end{bmatrix} \begin{bmatrix} v_s \\ v_c \end{bmatrix} = \begin{bmatrix} 0 \\ 0 \end{bmatrix} \tag{5}$$

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

$$\begin{bmatrix} \mathbf{M}_{cc}^{f} & 0 \\ 0 & \mathbf{M}_{ff} \end{bmatrix} \begin{bmatrix} \ddot{v}_c \\ \ddot{v}_f \end{bmatrix} + \begin{bmatrix} \mathbf{K}_{cc}^{f} & \mathbf{K}_{cf} \\ \mathbf{K}_{fc} & \mathbf{K}_{ff} \end{bmatrix} \begin{bmatrix} v_c \\ v_f \end{bmatrix} = \begin{bmatrix} 0 \\ 0 \end{bmatrix} \tag{6}$$

Therefore, the right-hand side of the Eq.4 can be written as

$$\mathbf{R} = \begin{bmatrix} \mathbf{M}_{ss} & 0 & 0 \\ 0 & \mathbf{M}_{cc}^{s} & 0 \\ 0 & 0 & 0 \end{bmatrix} \begin{bmatrix} \ddot{v}_s \\ \ddot{v}_c \\ 0 \end{bmatrix} \tag{7}$$

Hence, the right-hand side of the Eq. 4 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 (Clough and Penzien, 1993)

$$\mathbf{M}\,\ddot{\boldsymbol{u}} + \mathbf{C}\,\dot{\boldsymbol{u}} + \mathbf{K}\,\boldsymbol{u} = -m_x\ddot{v}_x - m_y\ddot{v}_y - m_z\ddot{v}_z \tag{8}$$

Where **M, C** and **K** are the mass, damping and stiffness matrices of the soil/foundation-structure model, respectively, the added, relative displacements, $\boldsymbol{u}$, exist for the soil-structure system and must be set to zero at the sides and bottom of the foundation. The terms of $\ddot{v}_x$, $\ddot{v}_y$ and $\ddot{v}_z$ are the free-field components of the acceleration if the structure is not present. The column matrices, m_i , are the directional masses for the added structure only.

Soil/Foundation-Structure Interaction with Viscous Boundary

The general method treating of soil-structure interaction problem is to divide the infinite medium into the near field (truncated layer), which includes the irregularity as well as the non-homogeneity of the foundation, and the far field, which is simplified as an isotropic homogeneous elastic medium as before mentioned. As mentioned before, the near field is modeled using finite elements and the far field is treated by adding some special artificial boundaries or connecting some special elements. For this problem, more appropriate approximations are the artificial and/or transmitting boundaries. 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 (1969) developed viscous boundary 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 (Song and Wolf, 1994), Doubly-Asymptotic Multi Directional Transmitting Boundary (Wolf and Song, 1995) and etc. In this study, viscous boundaries are used for three dimensions.

To calculate the properties of this boundary condition, it is considered a plane wave propagating in the *x*-direction. The one dimensional equilibrium equation in the x-direction is:

$$\rho \frac{\partial^2 u}{\partial t^2} - \frac{\partial \sigma_x}{\partial x} = 0 \tag{9}$$

One-dimensional partial differential equation is written in the classical wave propagation form:

$$\frac{\partial^2 u}{\partial t^2} - v_p^2 \frac{\partial^2 u_x}{\partial x^2} = 0 \tag{10}$$

where v_p is the wave propagation velocity of the material and is given by $v_p = \sqrt{E_c/\rho}$ in which ρ is the mass density and E_c is the bulk modulus. The solution of the equation for the harmonic wave propagation in the positive x-direction is a displacement $u(t,x)$ and velocity $\dot{u}(t,x)$ of the following form:

$$u(t,x) = U\left[\sin\left(\omega t - \frac{\omega x}{v_p}\right) + \cos\left(\omega t - \frac{\omega x}{v_p}\right)\right] \quad \dot{u}(t,x) = U\omega\left[\cos\left(\omega t - \frac{\omega x}{v_p}\right) - \sin\left(\omega t - \frac{\omega x}{v_p}\right)\right] \tag{11}$$

The strain in the same direction and the corresponding stress can be expressed in the following simplified forms (Wilson, 2002) as can be seen same results in the study carried out by Lysmer and Kuhlmeyer (1969) :

$$\varepsilon(t,x) = \frac{\partial u}{\partial x} = -\frac{\dot{u}(x,t)}{v_p} \qquad \sigma_x = E_c \varepsilon(t,x) = -\rho v_p \dot{u}(x,t) \tag{12}$$

Where ρ, v_p and v_s are mass density, dilatational and shear wave velocities of the considered medium, respectively. Finally these viscous boundaries can be used with the FE mesh as shown in Fig. 1. In this figure A_n, A_{t1} and A_{t2} are the fields that controlled viscous dampers, σ and τ are the normal and shear stresses occurred in the boundaries of the medium and *n* and *t* are the subscripts represent normal and tangent directions in the boundary.

When the viscous boundaries are taken into consideration, well-known equation of motion can be written as below

$$[\mathbf{M}_{ss}]\{\ddot{u}(t)\} + [\mathbf{C}_{ss}]\{\dot{u}(t)\} + [\mathbf{C}_i^*]\{\dot{u}(t)\} + [\mathbf{K}_{ss}]\{u(t)\} = \{R(t)\} \tag{13}$$

where $\mathbf{C}_i^*$ is the special damping matrix and that is:

$$[\mathbf{C}_i^*] = \begin{bmatrix} A_n \rho v_p & 0 & 0 \\ 0 & A_{t1} \rho v_s & 0 \\ 0 & 0 & A_{t2} \rho v_s \end{bmatrix} \tag{14}$$

Finally equation of the motion concerning the fluid-elevated tank-soil/foundations system is

$$([\mathbf{M}_{ss}] + [\mathbf{M}_f])\{\ddot{u}(t)\} + ([\mathbf{C}_f] + [\mathbf{C}_{ss}] + [\mathbf{C}_i^*])\{\dot{u}(t)\} + ([\mathbf{K}_f] + [\mathbf{K}_s] + [\mathbf{K}_{ss}])\{u(t)\} = \{R(t)\} \tag{15}$$

Where $\mathbf{M}$, $\mathbf{K}$ and $\mathbf{C}$ are the mass, stiffness and damping matrix, subscript of *ss*, f , i and s indicate the soil-structure, fluid, boundary surface and fluid surface of the fluid-structure –soil/foundation system, respectively.

FLUID-STRUCTURE INTERACTION

Fluid-structure interaction problems can be investigated by using different approaches such as added mass, Lagrangian, Eulerian, and Lagrangian-Eulerian in FEM and Smoothed Particle Hydrodynamic (SPH) methods (Anghileri et al., 2005) or by using the analytical methods like Housner's two mass representations (Housner, 1963), multi mass presentations of Bauer (1964) etc. Among these, displacement based Lagrangian approach is selected to model fluid-elevated tank interaction. The fluid elements are defined by eight nodes having three degree-of-freedom at each node; translation in the nodal x, y, and z directions. Brick fluid element also includes special surface effects, which may be thought as gravity springs used to hold the surface in place. This is performed by adding springs to each node, with the spring constants being positive on the top of the element. Gravity effects must be included if a free surface exists. For an interior node, the positive and negative effects cancel out (ANSYS, 2006).

DETAILS OF MODELS

A reinforced concrete elevated tanks on six different soil types with a container capacity of 900 m3 are considered in seismic analyses (Fig.2). One of them has frame supporting system whereas the others have the shaft supporting system. The elevated tanks with a frame supporting system in which columns are connected by the circumferential beams at regular interval at 7 m and 14 m elevations. . Since the intze type tank container has an optimal load balancing shape, it is widely preferred (Rai 2002). It is also used in the tanks modeled in this study. The elevated tanks with frame supporting structure have been used as a typical project in Turkey up to recent years. Young's modulus and the weight of concrete per unit volume are selected as 32,000 MPa and 25 kN/m3, respectively. The container is also filled with the water density of 1,000 kg/m3 and as seen from Fig.2.

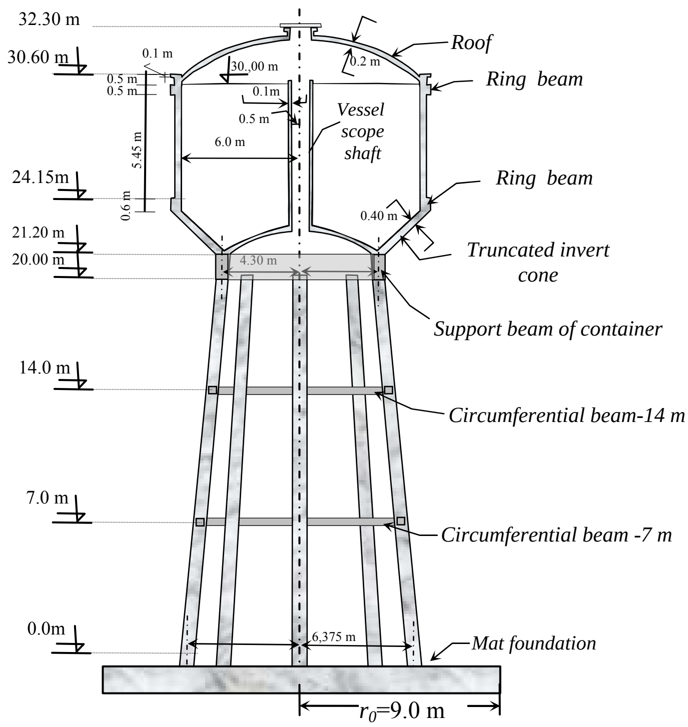


Figure 2.Vertical cross section of the reinforced concrete elevated tanks considered for the seismic analysis

In the seismic analysis, it is assumed that tanks are subjected to North-South component of the August 17, 1999 Kocaeli Earthquake in Turkey. Approximately first twenty seconds of ground acceleration of North-South component of this earthquake was taken into consideration. To evaluate variations of the dynamic parameters in the elevated tanks depending on different soil conditions, six soil types as shown in Table 1 were considered. Soil conditions recommended in the literature are taken into account in the selection of the soil types and their properties (Bardet, 1997; Coduto, 2001). For six different soil types, seismic analysis of the elevated tank and soil systems were carried out. For these analyses the models as seen from Fig.3 are used. As can be seen from the Fig.3a a model was performed for massless foundation approximation and from Fig3b a model carried out by considering viscous boundaries. In all models frame supporting system with are modeled beam element, similarly for foundation and vessel, shell elements are used. Fluid within the vessel was added to the FEM by using Lagrange fluid elements. Finally, soils are simulated with solid elements in the models.

Table 1. Properties of subsoil considered.

Soil types	ζ_g	Young's modulus E (kN/m^2)	Shear modulus G (kN/m^2)	Bulk modulus E_c (kN/m^3)	Unit weight γ (kg/m^3)	Poisson ratio, υ	Shear wave velocity v_s (m/s)	P wave velocity v_p (m/s)
S1	5.00	7000000	2692310	9423077	2000	0.30	1149.1	2149.89
S2	5.00	2000000	769230	2692308	2000	0.30	614.25	1149.16
S3	5.00	500000	192310	673077	1900	0.35	309.22	643.68
S4	5.00	150000	57690	201923	1900	0.35	169.36	352.56
S5	5.00	75000	26790	160714	1800	0.40	120.82	295.95
S6	5.00	35000	12500	75000	1800	0.40	82.54	202.18

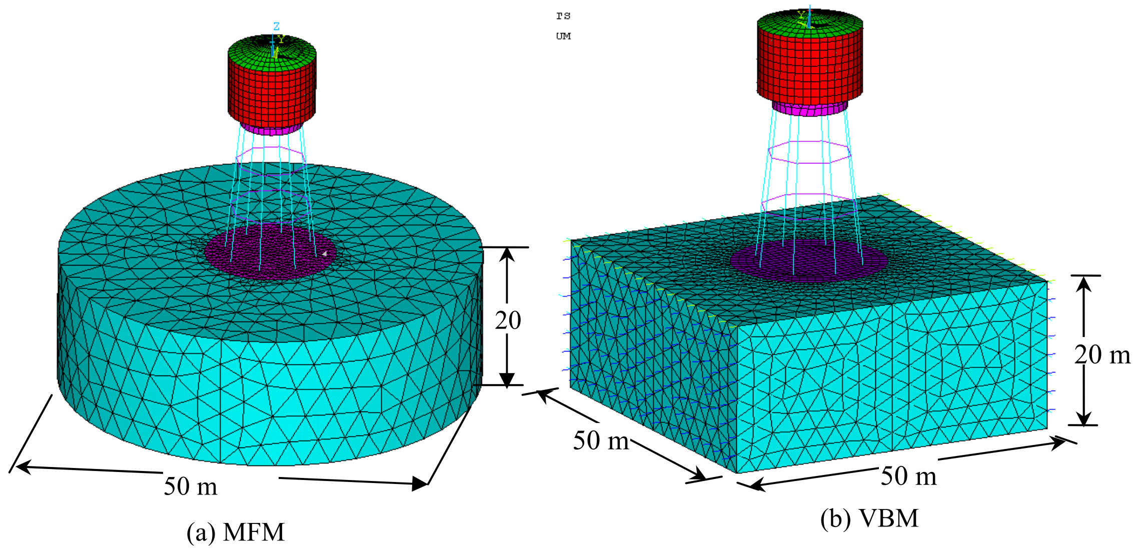


Figure 3. Considered FEM and mesh type for a) massless foundation approximation (MFM) and b) viscous boundary approximation (VBM)

DISCUSSION OF THE ANALYSIS RESULTS

The obtained peak values and their times of the maximum sloshing displacements (u_{smax}), according to the soil condition, from the different 12 models (six for MFM and six for VBM) are given in Table 2 respectively. As can be seen from the table, these maximum responses of the systems obtained about 10.1 to 10.35 seconds and maximum responses are calculated for the systems in S6 soil type as can be expected. Similarly the maximum roof displacement are calculated for S6 soil type as 0.25 m and maximum displacement for all soil types are obtained about 9,4~9.8 s. All obtained values and their deviations are discussed and some of them and their deviations in time are illustrated under following titles.

Table 2. Results of sloshing displacement and roof displacements of the models obtained from all seismic analysis

	Sloshing displacement											
Soil Type	S1		S2		S3		S4		S5		S6	
	t (s)	u_s(m)	*t* (s)	u_s(m)	*t* (s)	u_s(m)	*t* (s)	u_s(m)	*t* (s)	u_s(m)	*t* (s)	u_s(m)
For MFM	10.10	1.97	10.10	1.97	10.15	-1.99	10.15	2.05	10.15	2.10	10.25	2.18
For VBM	10.10	-1.96	10.10	-1.98	10.15	-2.02	10.15	-2.14	10.20	-2.26	10.35	-2.42
	Roof displacement											
For MFM	9.45	0.10	9.45	0.10	9.45	0.11	9.55	0.12	9.60	0.14	9.75	0.18
For VBM	9.45	0.10	9.45	0.11	9.50	0.12	9.55	0.14	9.60	0.17	9.80	0.25

Roof Displacements

Maximum displacement along the height of the elevated tank and roof displacement are illustrated in Fig.4. From the results realized can be seen that for relatively stiff soil maximum displacement is obtained in the level of the support beam of the container, but in the systems in relatively softer soil as S5 and S6 it is the roof level. Because of the rocking response of the soil/foundation system for softer soil type, one can easily observe this occurred behavior for both models of MFM and VBM.

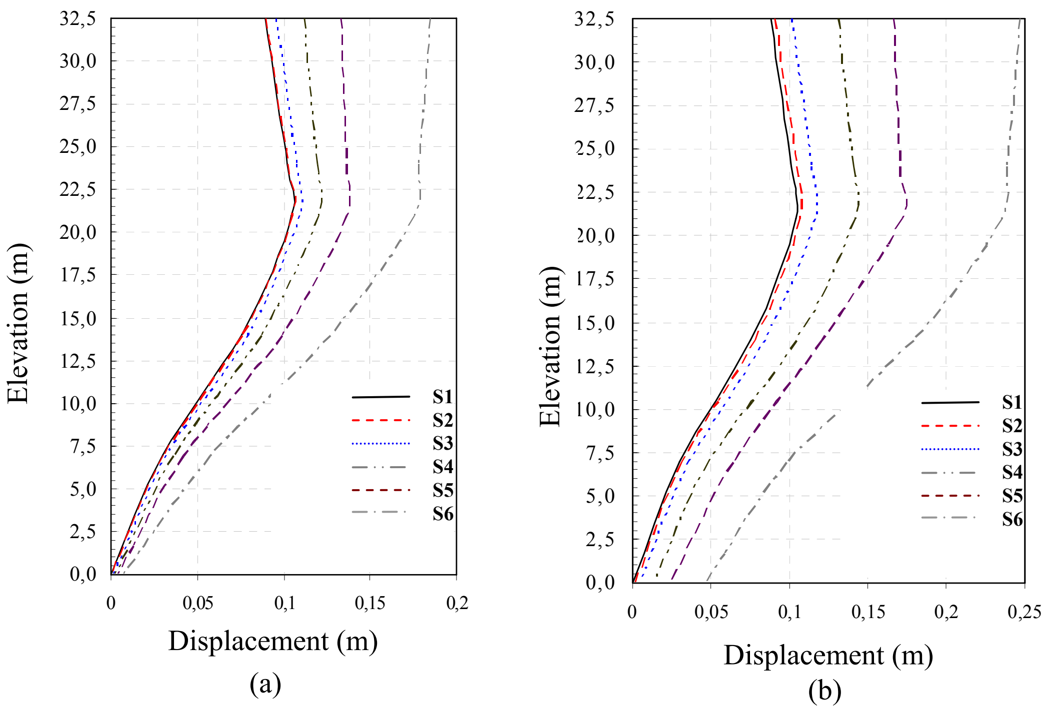


Figure 4. Maximum displacements along the height of the elevated tanks for six soil types, (a) For MFM and (b) for VBM,

When systems are evaluated for almost all soil types, it is assigned that soil interaction affects the roof displacement, and since it cause the decrease both horizontal and rocking stiffnesses of soil/foundation system and the displacement values are significantly increased especially for softer soil like S5 and S6, If this circumstance is studied in case of stiff soil, it is not effective so much. Therefore it must be expressed that after S1 and more stiff soil than this type, differences on roof displacement would be disappeared.

If the behavior of the systems in softer soil which is not considered here investigates, it will be most probably seen that this tendency will increase. Also it is worth to note that VBM includes soil deformation arise from inertial effects of soil medium. Whereas, it is not possible to consider the deformations in MFM model. Therefore the soil medium displacements at the base level of elevated tanks are ignored. If one takes into this consideration, it is clearly seen that the result obtained from different approximation are coincided for almost all soil types investigated in this study. Also it should be noted that from both results given Table 2 and illustrated Fig.4 when the soil gets softer the result obtained from the different approximation tends to go away from each other a bit.

As can be seen from Fig.5a,b and c, roof displacement of systems in S6 soil type gave maximum displacement and affected more than other type of soil. It must be noted that displacements in such an elevated tank must be controlled in permission limit because for this type of structure considered in this study, 0.25 m is not allowable. For the other soil type this is observed with same tendency but the decrease is getting smaller with increasing stiffness of soil and finally decrease is almost equal to zero for S1 soil type.

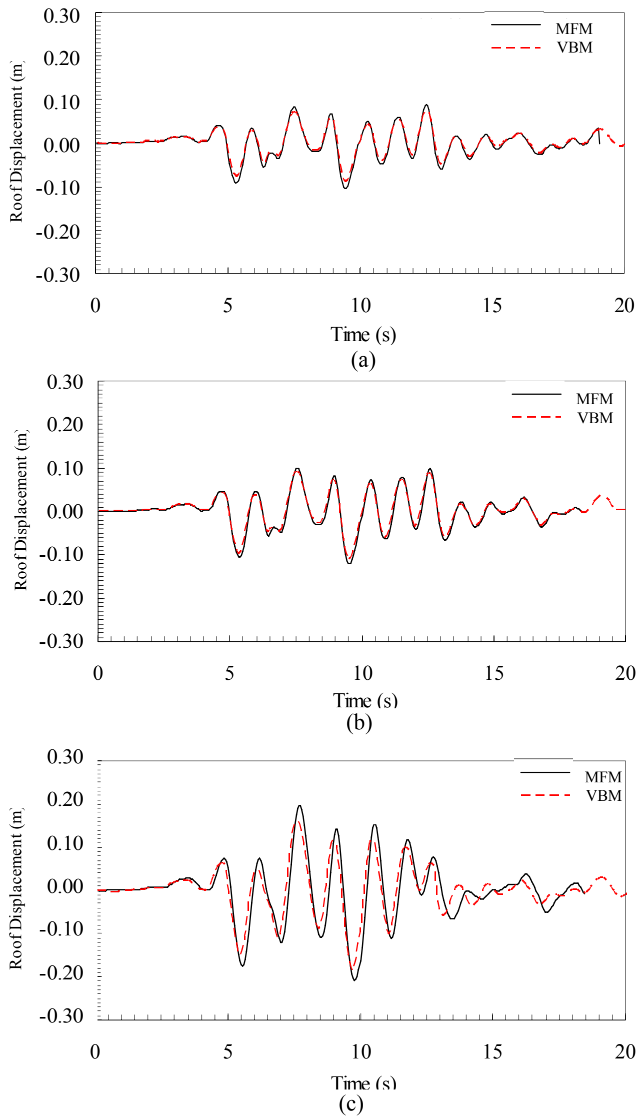
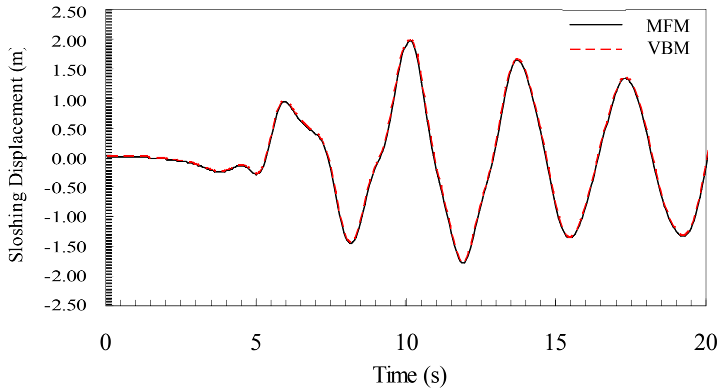


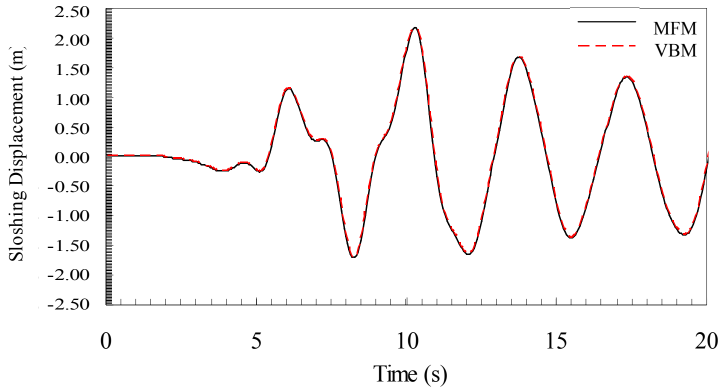
Figure 5. Time history of the roof displacement results obtained from the two different models for (a) S1 soil types (b) S4 soil type (c) S6 soil type

Sloshing Displacement

The estimated sloshing displacements varying in time for soils of S1 and S6 were illustrated in Fig.6. As seen from this figure sloshing responses obtained for different approximation are almost same. Maximum displacement reaches 2.42 m in 10.20 s for the system in S6. It is seen that approximately the maximum displacement practically occurs at the same time (t =10 s~10.2s) for all systems. When the deviation is investigated for the other systems deviations are less and for this reason it is seen that model assumptions are not effective on sloshing displacement.



(a)



(b)

Figure 6. Time history of the sloshing displacement results obtained from two different approximations for (a) S1 soil types (b) S6 soil type

CONCLUSIONS

It was seen the considered models with massless foundation approximation can be used this type of structures for interval of soil type investigated in this study.

Considering the fluid and soil interaction effect, a procedure is presented to determinate soil-structure interaction effects on elevated tank for seismic analysis of fluid-elevated tank-foundation/soil systems. The procedure provides to determine not only structural response of the system but also the sloshing responses.

Variations of the displacements along the height of the elevated tank are rather different from each other for the elevated tank in different soil types. In fact, it is observed that the elevated tanks supported on an elastic medium having relatively soft soil 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.
It is recommended that more numerical examples should be analyzed for different soil types and foundation conditions. So, using the procedure presented here results can be generalized.

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