Evaluation of Foundation Flexibility on Seismic Performance of Anchored and Unanchored Aboveground Cylindrical Steel Tanks

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SUMMARY:
Fluid tanks are considered among most important and essential parts in lifeline systems. In this research, effect of slab foundation flexibility on seismic performance of anchored and unanchored cylindrical steel tanks is discussed. For this aim three wide, medium, and tall tanks with height to diameter ratios of 0.343, 0.85, and 1.53, respectively, have been studied using FEM. These tanks were excited by 7 one-way records and then the average of the results has been considered. In order to model the soil and comprise interactions, as well as using direct modelling, absorbent boundaries are introduced in appropriate distances. Numerical results reveal that in the case of having a solid foundation, the value of axial and hoop stresses in tank's wall and also uplifting of the tank's bottom is less than the case of flexible foundation; however, no significant difference for free surface sloshing height has been recognized in both above-mentioned states.

Keywords: Steel tanks, Seismic performance, Foundation flexibility, FEM, Absorbent boundaries

1. INTRODUCTION

Seismic performance of tanks has attracted many researchers' interest. The first comprehensive study in this area was conducted in 1934 by [Jacobsen and Hoskins, 1934] on solid tanks. In 1957, [Housner, 1957] introduced a simple model which fluid mass divided into two solid and shaking parts. Hence, he analyzed hydrodynamic pressure inserted to the tank wall into two components. The first component was the impact pressure induced from accelerated mass of the tank while the latter was oscillatory pressure produced by surface waves. In a laboratory research conducted by [Cambra, 1983] in1983, effect of foundation flexibility on the amount of uplift on both static and dynamic states was studied. The result of that study revealed that the amount of tank's uplift and the magnitude of created stresses on tank wall once the tank is placed on a solid foundation is smaller than the case the tank sited on a flexible foundation. [Zaman and Koragappa, 1989] studied flexibility behavior of the rectangular foundations in an elastic half space. They used principle of minimum potential energy and models with symmetric geometry and loading in their study. In 2003, [Godoy and Sosa, 2003] studied the effect of local settlement of the foundation on cylindrical fluid tank with a thin wall as well as buckling and produced stresses on tank's wall. Their results showed that deformation created in the shells of thin walls (induced by local settlings of the foundation) is completely different from those developed by wind force and earthquake - which are mainly affected by nonlinear behavior of the shells. In 2008, [Bakhshi & Hassanikhah, 2008] studied seismic performance of fluid tanks in both anchored and unanchored states. They reported that with soil flexibility increases and decrease of tank's wall thickness, the axial and hoop tensional-compressive stresses inserted on the tank's wall increase considerably.[Jahangiri, 2011] studied seismic performance of steel tanks under foundation flexibility by FEM. Results show that anchored tall tanks are more sensitive about foundation flexibility than other tanks.

In the present study, the effect of flexibility of slab foundations on seismic performance of aboveground cylindrical tanks has been investigated using the finite element technic. In this study, after designing models for wide, medium, and tall tanks for both fully anchored and unanchored states, the tanks are excited by 7 one-way records and the average of the results is measured. Furthermore, vertical displacement of liquid surface (sloshing), tank's axial and hoop stresses and tank's base plate uplifting are discussed.

2. FINITE ELEMENT MODELING

2.1. Soil Modelling

To model the soils behavior, Drucker- Prager model in linear state was used.

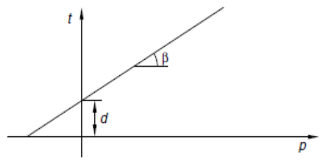

Figure 2.1. Linear Drucker - Prager

$$F = t - p\ tan\beta - d = 0 \tag{2.1}$$

In the equation (2-1), β is the linear yield surface slope in strain-stress plane and typically called fraction angle of the materials. Also, d, p, and t are cohesion of the materials, equivalent compressive stress, and triaxial compressive stress, respectively.

Table 2.1. Soil Properties

$\rho\left(\frac{kg}{m^3}\right)$	$E\left(\frac{MN}{m^2}\right)$	υ	$V_s\left(\frac{m}{s}\right)$	β	d
2000	825.6	0.29	400	38	0

Where, ρ is Bulk density of the soil , E is the soil's modulus of elasticity and υ is Poisson's ratio. For modeling materials damping, Rayleigh method are applied.

$$[C] = \alpha[M] + \beta[K] \tag{2.2}$$

Where [C], [M], and [K] are matrices of damping, mass, and material's stiffness. In addition, coefficients of α and β are damping coefficients appropriate to mass and stiffness, respectively. By esteeming that damping ratio between both frequencies is constant, the following relationship can be used to estimate damping coeficient.

$$\begin{Bmatrix} \alpha \\ \beta \end{Bmatrix} = \frac{2\xi}{\omega_1 + \omega_2} \begin{bmatrix} \omega_1\omega_2 \\ 1 \end{bmatrix} \tag{2.3}$$

To determine dimensions of the soil mass, several models with different dimensions were created and analyzed until the point of reaching a dimension in which increase of dimensions has no effect on accuracy of the results. So, a rectangular cubic soil area with dimension of 150 × 150 × 40 meter was obtained. Furthermore, to prevent waves created by rebound from the soil mass boundaries, some viscose dampers were installed in these boundaries. The constant coefficients of unit surface of these dampers in directions normal and tangential to the surface can be estimated using the following equations.

$$\begin{cases} C_n = \rho V_p \\ C_t = \rho V_s \end{cases} \tag{2.4}$$

where, ρ is Bulk density of soil, V_p is P wave velocity, V_s is Shear wave velocity, C_n is Surface unit constant in direction normal to the surface boundary, and C_t is Surface unit constant in direction tangential to the surface boundary. Besides, to calculate V_p, the following equation of Lysmer is used.

$$V_p = \frac{3.4V_s}{\pi(1-\nu)} \tag{2.5}$$

2.2. Tank Modeling

To investigate seismic performance of the tanks, three steel wide, medium, and tall steel tanks were studied (table 2.1). To model the shell of the tank, four-nodes doubly curved shell element with reduced integration, hour-glass control, and finite membrane strain formulation were used. The shell and tank bottom both were made of steel with Young modulus of 210 GPa, Poisson coefficient of 0.3, yield stress of 240 MPa and ultimate stress of 360 MPa.

Table 2.1- Tanks Dimensional Details

h/D Ratio	D(m)	h(m)	h_w(m)
0.343(Wide)	35	12	10
0.85(Medium)	20	17	14.5
1.53(Tall)	15	23	20

In table (2.1), D, h, and h_w are diameter, height, and fluid height of the tank. Finally, the wide, medium and tall tanks were modeled as figure (2.2).

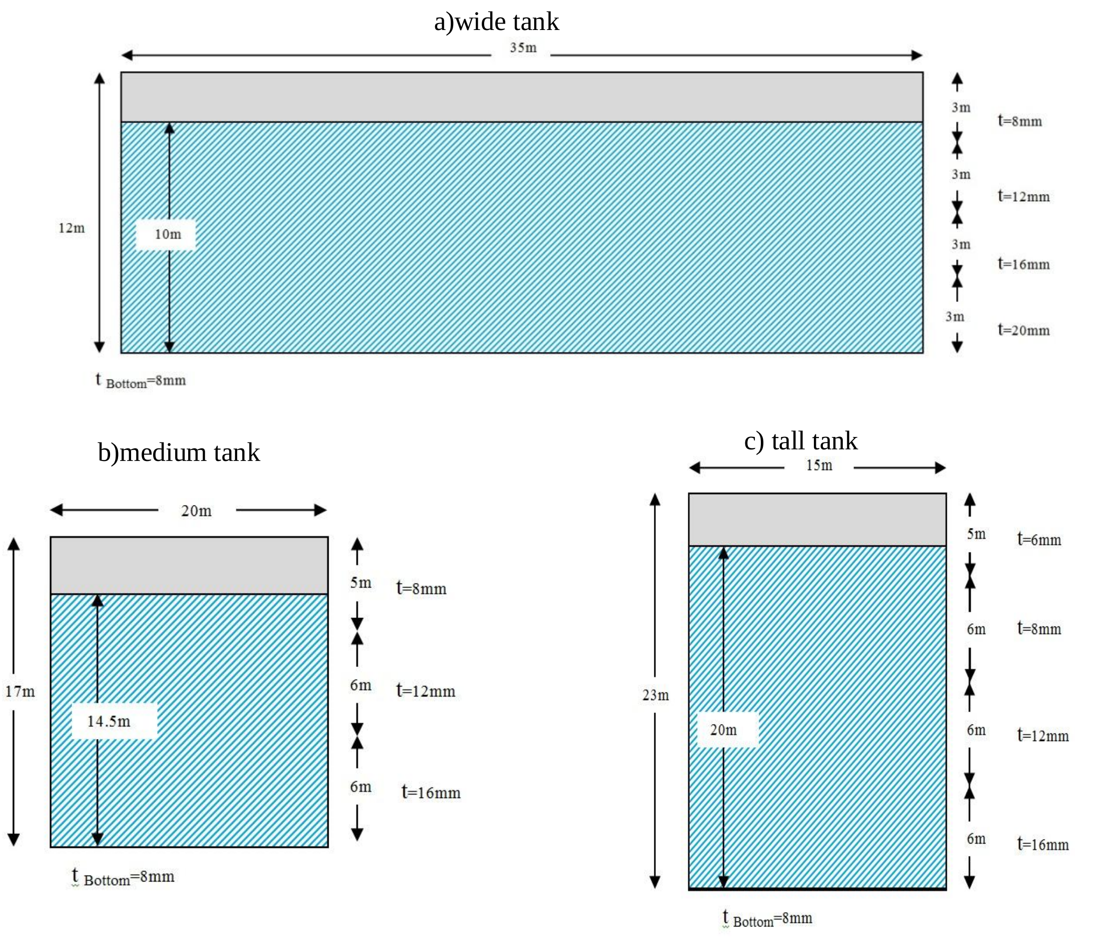

Fig. 2.2. Dimensions and wall's thicknesses of tanks, i.e a)wide tank b)medium tank c)tall tank

2.3. Fluid Modeling

In hydrodynamic problems response of excited liquid storage tank systems appears the impulsive and the convective actions. Thus, for accurate modeling of the fluid behavior it is necessary to consider sloshing phenomenon. For this regard, linear equation of state was used. Since the fluid modeled in this study was water, fluid density was 1000 kg/m^3 and the sound velocity in the fluid volume was taken 1460 m/s. In this study, the fluid was considered as non-compressible and non-viscose.

2.4. Foundation Modeling

The behavior model assumed for the foundation materials is elastic model. In term of materials, two materials were studied for the foundation. The first material was concrete which nearly model the behavior of the solid foundation, while the second one was soil mass which was considered for the flexible state. Regarding to this choice, as well as discussing the effect of foundation flexibility on seismic behavior of the tanks, it is also possible to investigate the effect of lack or presence of concrete foundation.

In solid state, where the concrete materials were used, due to presence of reinforcement bars in the foundation (which were not modeled in this study) to enhance the tensional strength, It is possible to use elastic modulus [Park and Paulay, 1975]:

$$E_c = 4730\sqrt{f'_c} \quad (2.6)$$

Where, f_c' is the 28-day compressive cylinder strength of concrete (MPa) Based on these assumptions, elastic modulus of the concrete is calculated in MPa. Since f'_c was taken 36.8 MPa, the elastic modulus of the concrete is as follows:

$$E_c = 4730\sqrt{f'_c} = 4730.\sqrt{36.8} = 28.69\ GPa$$

Table 2.3. Foundation Materials Properties

Foundation Stiffness	Material	$\rho\ (\frac{kg}{m^2})$	ν	$E\ (GPa)$
Flexible	Soil	*2000*	*0.29*	*0.8256*
Solid	Concrete	*2400*	*0.2*	28.69

To design the slab foundation, the manual API650 (Appendix B) has been used. Finally, based on recommendations of these manual a foundation with the following dimensions is designed.

Table 2.4. Foundation dimensions

h/D Ratio	Radius(m)	Depth(m)
0.343(wide)	18.5	1
0.85(medium)	11.2	1.2
1.53(tall)	8.7	1.2

The mesh generation properties is available in table (2.5) and the designed models are shown in figures (2.3), (2.4), and (2.5).

Table 2.5. Mesh generation properties

h/D Ratio	Part	Mesh peroperties	Number of elements
0.343(Wide)	Soil	8-node linear brick, reduced integration, hourglass control	67308
	Foundation	8-node linear brick, reduced integration, hourglass control	3000
	Tank	4-node doubly curved thin shell, reduced integration, hourglass control, finite membrane strains	3180
	Water	8-node linear brick, reduced integration, hourglass control	13800
0.85(Medium)	Soil	8-node linear brick, reduced integration, hourglass control	43040
	Foundation	8-node linear brick, reduced integration, hourglass control	1760
	Tank	4-node doubly curved thin shell, reduced integration, hourglass control, finite membrane strains	3440
	Water	8-node linear brick, reduced integration, hourglass control	20880
1.53(Tall)	Soil	8-node linear brick, reduced integration, hourglass control	47840
	Foundation	8-node linear brick, reduced integration, hourglass control	1760
	Tank	4-node doubly curved thin shell, reduced integration, hourglass control, finite membrane strains	4400
	Water	8-node linear brick, reduced integration, hourglass control	28800

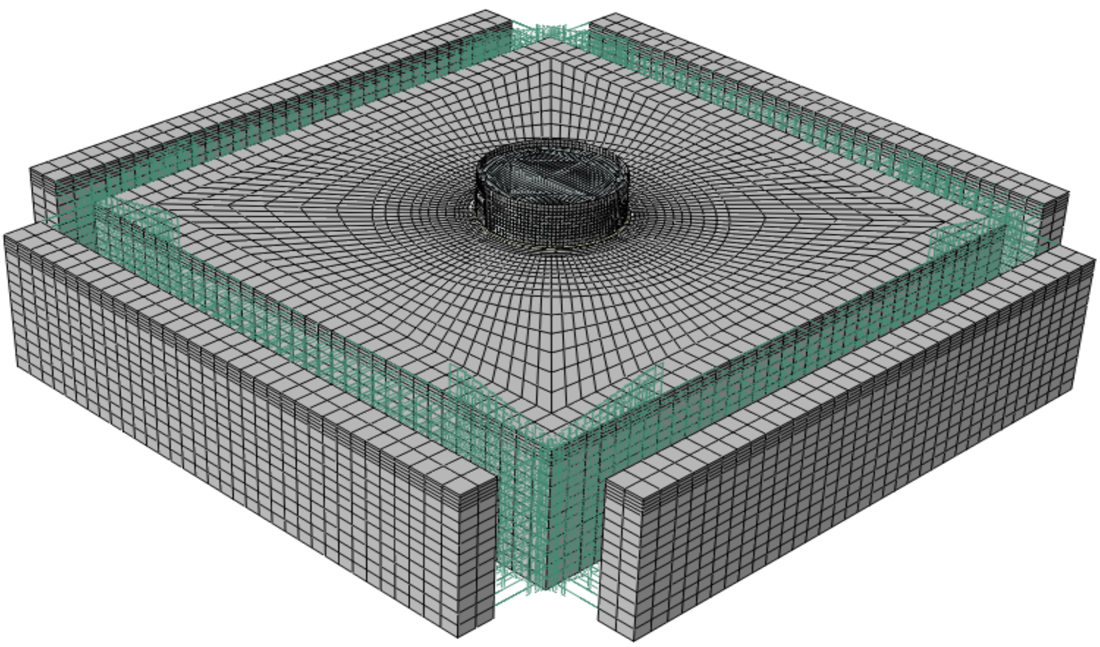

Figure 2.3. Finite element mesh for the wide tank system (H/D=0.343)

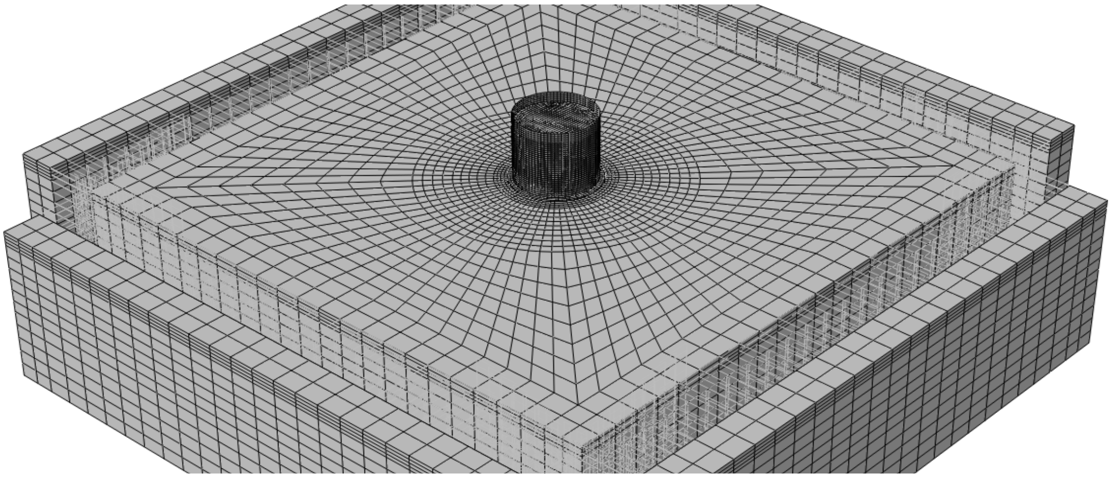

Figure 2.4. Finite element mesh for the medium tank system (H/D=0.85)

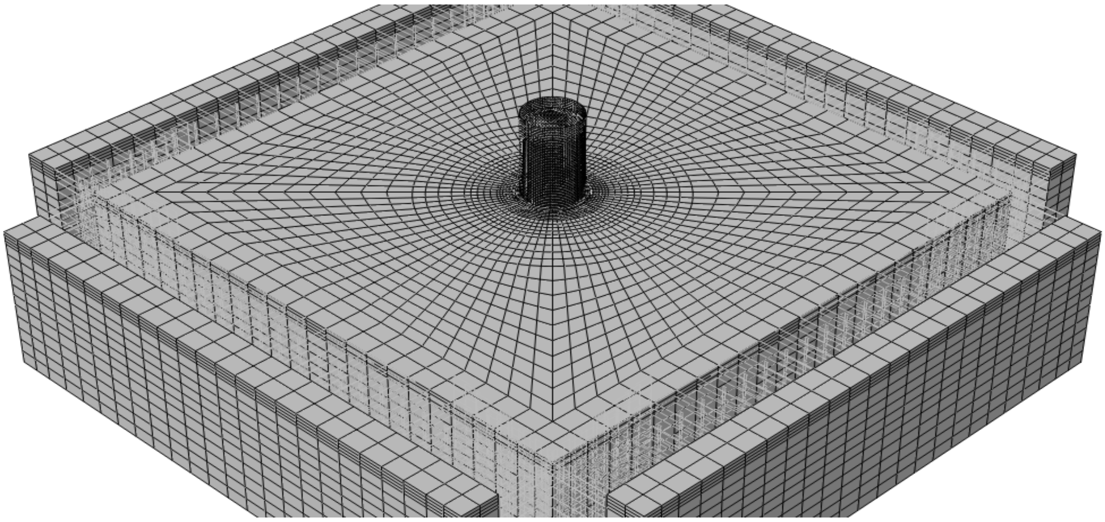

Figure 2.5. Finite element mesh for the tall tank system (H/D=1.53)

3. INPUT GROUND MOUTIONS

All designed models were subjected to 7 one-way ground motions whose characteristics are shown in table (3.1). In this process, at first the weight of group is inserted to the system for 1 second in uniform manner. In the next step, the system is released for 1 second to remove its movements and then for 10 seconds strong excitation of the each record is inserted to the system. After doing all analyses, the average of the obtained results is introduced as final result. It must be noticed that all mentioned records were scaled to nonlinear acceleration spectrum.

Table 3.1- Input ground motions

One-Way Records	Station	PGA(g)
Kobe 1995, N-S	KJMA	0.599
Loma Prieta 1989, N-S	57007 Corralitos	0.644
Northridge 1994, N-S	24278 Castaic – Old Ridge Route	0.568
San Fernando 1971, N-S	128 Lake Hughes #12	0.366
Cape Mendocino 1992, N-S	89324 Rio Dell Overpass – FF	0.385
Manjil 1990, N-S	BHRC 99999 Abbar	0.505
Chi Chi 1999, N-S	CHY080	0.902

4. UPLIFTING RESULTS UNDER FOUNDATION FEXIBILITY

Tank uplifting phenomenon in unanchored tanks is important in the sense that it can lead to creation of axial and hoop stresses in the tank shell and result in development of diamond shape and elephant foot buckling. In table (3.2) the maximum amounts of tank uplifting produced from averaging from the values obtained from all records analysis are introduced.

Table 4.1-Maximum Uplifting of tanks

	Base Plate Uplifting	
h/D Ratio	On Solid Foundation (m)	On Flexible Foundation (m)
0.343 (Wide)	0.034	0.05
0.85 (Medium)	0.082	0.091
1.53 (Tall)	0.081	0.087

The results indicate that tanks endure smaller uplifting in the presence of solid concrete foundation than flexible foundation. Figure 4.1 shows the uplift time history on wide tank under Kobe excitation.

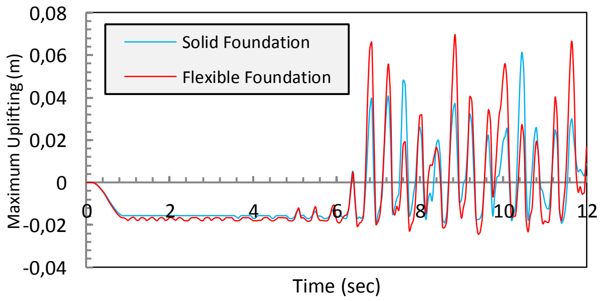

Figure 4.1. Wide tank's uplifting subjected to Kobe ground motion

5. HOOP AND AXIAL STRESSES VALUES UNDER FOUNDATION FLEXIBILITY

The magnitude of hoop and axial stresses on the wall shell of tank are main factors in health monitoring of the tanks. It is worth to mention that control of maximum amount of these factors can guarantee the safety of the tank. In tables (5.1), (5.2), (5.3), and (5.4) the results of maximum value of these stresses in control of anchored and unanchored tanks on flexibility of the slab foundation are presented.

Table 5.1. Maximum hoop compressive stress in anchored and unanchored tanks

	Maximum hoop compressive stress (MPa)			
h/D Ratio	Unanchored Tanks		Anchored Tanks	
	Solid Foundation	Flexible	Solid	Flexible
0.343(Wide)	-215	-310	-237	-256
0.85(Medium)	-292	-298	-111	-160
1.53(Tall)	-52	-54	-171	-307

Table 5.2. Maximum hoop tensional stress in anchored and unanchored tanks

	Maximum hoop tensional stress (MPa)			
h/D Ratio	Unanchored Tanks		Anchored Tanks	
	Solid Foundation	Flexible Foundation	Solid Foundation	Flexible Foundation
0.343(Wide)	300	348	239	295
0.85(Medium)	314	331	194	244
1.53(Tall)	239	245	276	336

Table 5.3. Maximum axial compressive stress in anchored and unanchored tanks

h/D Ratio	Maximum axial compressive stress (MPa)			
	Unanchored Tanks		Anchored Tanks	
	Solid Foundation	Flexible Foundation	Solid Foundation	Flexible Foundation
0.343(Wide)	-301	-331	-297	-324
0.85(Medium)	-287	-294	-159	-241
1.53(Tall)	-148	-159	-175	-345

Table 5.4. Maximum axial tensional stress in anchored and unanchored tanks

h/D Ratio	Maximum Axial Tensional Stress (MPa)			
	Unanchored Tanks		Anchored Tanks	
	Solid Foundation	Flexible Foundation	Solid Foundation	Flexible Foundation
0.343(Wide)	197	253	262	305
0.85(Medium)	285	298	168	211
1.53(Tall)	219	237	206	311

According to the obtained results, it is obvious that the value of hoop and axial stresses on the tank's wall in the presence of flexible foundation is larger than corresponding solid foundation. Moreover, it is clear that the effect of foundation flexibility parameter on hoop and axial stresses of the tank's shell in anchored tanks is extremely higher than corresponding parameters in unanchored tanks, particularly for the tall anchored tanks where magnitude of this effect reaches to 170 MPa.

In figure (5.1), the values of hoop stresses on high and medium tank's wall using Kobe ground motion is shown.

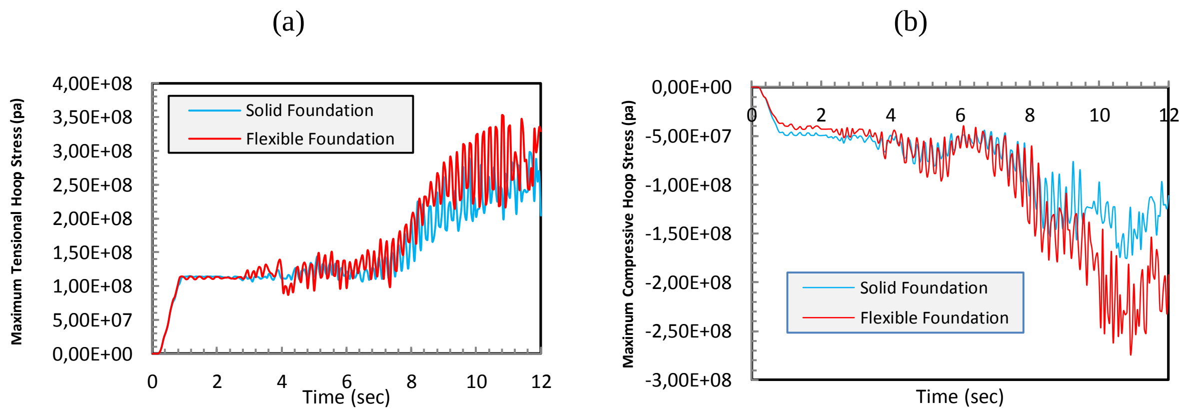

Figure 5.1. a) Maximum tensional hoop stress in tall tank b) Maximum compressive hoop stress in medium tank

6. SLOSHING RESULTS UNDER FOUNDATION FEXIBILITY

The height of created waves is regarded in this sense that it can bring harms to the equipment installed on the tank. Besides, in the case of tank's overflow, this parameter can be troublesome, particularly when tank's fluid is toxic or flammable. Also, the sloshing in the tanks having floating roof can cause to serious harms; so control of these parameters is of great importance. In Tables (6.1) and (6.2) the maximum amounts of vertical displacement of liquid surface from average of the values obtained from all records analysis are introduced. In figure (6.1), the values of sloshing height on all tanks under Kobe ground motion in anchored tanks are shown.

Table 6.1 Maximum vertical displacement of liquid surface (anchored tanks)

	Water Sloshing	
h/D Ratio	On Solid Foundation (m)	On Flexible Foundation (m)
0.343(Wide)	0.58	0.61
0.85(Medium)	0.35	0.32
1.53(Tall)	0.34	0.33

Table 6.1 Maximum vertical displacement of liquid surface (unanchored tanks)

	Water Sloshing	
h/D Ratio	On Solid Foundation (m)	On Flexible Foundation (m)
0.343(Wide)	0.62	0.67
0.85(Medium)	0.4	0.42
1.53(Tall)	0.37	0.36

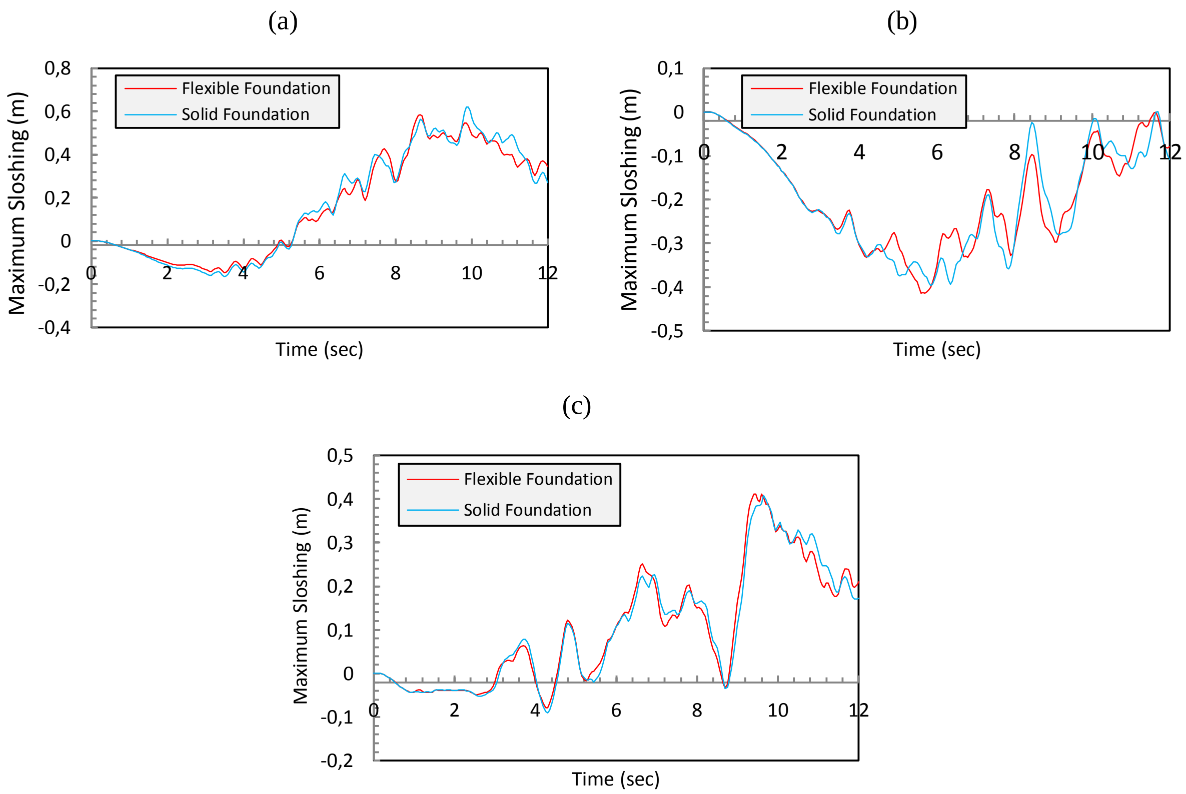

Figure 6.1. Maximum sloshing of anchored tanks using Kobe ground motion
a)wide tank b)medium tank c)tall tank

On the basis of these results, it is obvious that the value of sloshing height on the tanks located on flexible and solid foundations tolerates no significant difference.

7. CONCLUDING REMARKS

Tank uplifting in the anchored tanks was a considerable parameter which could lead to formation of extremely high stresses in wall shell of tanks. The results revealed that values of tank's uplifting in the presence of solid foundation is less than corresponding values of flexible foundation, so it could be concluded that it was an effective tool to control tank's uplifting.

Hoop and axial stresses in medium and tall unanchored tanks in the presence of solid foundation showed no significant difference with the case of using flexible foundation; however for the wide tank, one might notice that the stresses in the case of solid foundations show smaller values than the case of using flexible foundation. It was also worth to mention that in anchored tanks effect of foundations flexibility on the stresses was significant: In all three anchored wide, medium, and tall tanks the value of hoop and axial stresses in the presence of solid foundation is less than the state of using flexible foundations – particularly in tall tanks where this difference reaches to amount of 170 MPa. It was obvious that the value of sloshing height on the tanks in the presence of flexible foundation has no significant difference with the condition of solid foundation.

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