PERFORMANCE OF CYLINDRICAL LIQUID STORAGE TANKS IN SILAKHOR, IRAN EARTHQUAKE OF MARCH 31, 2006

Sassan Eshghi[1] and Mehran S. Razzaghi[2]

ABSTRACT

Several on-ground cylindrical liquid storage tanks experienced strong ground motion during the "Silakhor" earthquake of March 31, 2006 in western Iran, and some of the tanks suffered minor to moderate damage. In this study two of the affected tanks that were located close to the station of recording the time history of the earthquake were investigated. Responses of these tanks to the earthquake were estimated using published methods and also non-linear time history analysis, for both rigid foundation and flexible foundation assumptions. Theoretical results were compared and were generally in good agreement with the observed performance of tanks during the earthquake. For the broad tank uplift displacements observed from the earthquake matched quite closely the predictions of numerical analysis and some of the published methods, although there was a significant variation in the predictions of various methods. It was also shown that axial stresses in tank shells uplifting under earthquake are very dependent on the rigidity of the foundation.

1. INTRODUCTION

Understanding of the dynamic behaviour of complex structures such as liquid storage tanks is enhanced by the combined use of theoretical approaches and experimental techniques. Natural seismic events can be considered as full scale tests on structures. If the response of a structure to a certain earthquake is recorded, it offers an opportunity to directly study the dynamic behaviour. For special structures like cylindrical liquid storage tanks, due to the operational limitations of instrumentation (e.g. shape of the structure), it can be difficult to obtain detailed measurements of structural response during earthquake shaking (Eshghi, Razzaghi, 2006). However, if the ground motion that the structure was subjected to is recorded, then observable damage and evidence of dynamic response of the structure can provide useful tools to calibrate analytical models of the behaviour.

On March 31, 2006 an earthquake of magnitude M_L=6.1 occurred in Silakhor in western Iran. Several equipment and industrial structures suffered minor to extensive damage due to the earthquake. More than 25 on-grade cylindrical steel tanks were located in the earthquake affected region and some of them suffered minor to moderate damage.

In this study the dynamic response of two cylindrical tanks that suffered damage is investigated. A strong motion ground shaking record was available from an accelerograph located very close to the selected tanks. Theoretical responses of the tanks were estimated using both numerical time history analyses and analytical expressions available from published methods and these predictions were compared with observations of actual damage suffered by tanks.

2. METHODOLOGY

The two damaged tanks selected for study were of different height-to-diameter ratios (H/D = 0.55 and 1.2) representing broad and tall tanks. The tanks were inspected carefully and all observed damage was recorded, together with measurements of tank geometry and liquid levels contained within.

The tanks were analysed using the methods published by NZSEE (1986) and API650 (API, 2001) using earthquake shaking inputs corresponding to the recorded earthquake ground motion.

The maximum sloshing wave height was estimated by Equation 1 (Myers, 1997):

$$d_{\max} = 1.124 ZIC_2 T^2 \tanh\left(4.77\sqrt{\frac{H}{D}}\right) \qquad (1)$$

where

$$C_2 = \begin{cases} \dfrac{0.75S}{T} & ,\mathrm{T} \le 4.5 \\ \dfrac{3.375S}{T^2} & ,\mathrm{T} > 4.5 \end{cases} \qquad (1a)$$

$$T = \frac{2\pi\sqrt{D}}{\sqrt{3.67 g \tanh\left(\dfrac{3.67}{D/H}\right)}} \qquad (1b)$$

d_{max} is the maximum height of sloshing wave (in feet), H is the height of the liquid (in feet), D is the diameter of the tank (in feet), g is the gravity acceleration, S is a site coefficient which is equal to 1.0 for rock sites, Z is seismic zone factor which is equal to 0.4 for high-risk zones. T is the natural period of the first sloshing mode in seconds and I is the importance factor.

[1] *Assistant Professor, International Institute of Earthquake Engineering and Seismology*

[2] *Ph.D. Candidate, International Institute of Earthquake Engineering and Seismology (mehran@iiees.ac.ir)*

Based on the Iranian guideline for designing water storage tanks (MPO, 1995), the maximum sloshing wave height d_{max} is given by Equation 2.

$$d_{\max} = \left[\frac{0.375(ABI/R)}{1-K_d(ABI/R)}\right]D \qquad (2)$$

where

$$K_d = 1.84\tanh(3.68\frac{h}{D}) \qquad (2a)$$

$$T = 1.81K_t\sqrt{h} \qquad (2b)$$

A is a seismic zone factor, I is an importance factor which is equal to 1.2 for storage tanks, R is a reduction factor which is 3.0 for underground and on-grade storage tanks and B is behaviour factor, which is a function of the sloshing wave period (T). D is the tank diameter in metres, h is the depth of liquid in metres and K_t is a period constant of sloshing liquid which can be calculated by using equation (2c).

$$K_t = \frac{0.785}{\sqrt{K_d \frac{h}{D}}} \qquad (2c)$$

It is worth mentioning that the equations (2), (2a), (2b) and (2c) are adopted from NAVFAC (1992) and modified based on the Iranian Code for Seismic Resistant Design of Buildings (1988).

The NZSEE (1986) method provides the expression in Equation 3 to estimate the maximum convective sloshing wave height.

$$d_{\max} = R\sqrt{[0.84\,C_h(T_1)]^2 + [0.07\,C_h(T_2)]^2} \qquad (3)$$

R is the tank radius in metres and $C_h(T_1)$, $C_h(T_2)$ are seismic horizontal force coefficients corresponding to the first and second convective modes respectively. The above expression is based on combining the first two convective mode wave heights using the square root of sum of squares (SRSS) method.

NZSEE (1986) also provides an expression for the maximum shell uplift by using modified Cambra's expression recommended in Equation 4:

$$v = \frac{1}{c}\left[\frac{f_{yb}t_b^2}{6N_x} + \frac{p_0L_b}{N_x}\left[\frac{L_b}{2} - \left(\frac{\overline{E}\ t_b^3}{12N_x}\right)^{1/2}\right]\right] \qquad (4)$$

where $N_x = f_{rb}t_b$ and $L_b = 2R(1-\mu)$, $\mu = \frac{r}{R}$ in which f_{yb} is the yield stress in the base plate material, c is a foundation stiffness factor equal to 1.0 for stiff foundations and 0.5 for flexible foundations, R is the radius of the tank and r is the radius of the area that does not uplift. t_b is the thickness of base plate, p_0 is the hydrostatic pressure over the base and f_{rb} is the membrane stress in base plate. In the study two foundation conditions were considered, either rigid foundations or flexible foundations.

The above analytical expressions are based on a quasi-static approach. To investigate the dynamic behaviour of the tanks due to the earthquake, non-linear time-history analyses were carried out as part of this study. The acceleration time-history of the horizontal component of the main earthquake event recorded in "Chalanchulan" station (BHRC, 2006) was used as input ground motion to the numerical models of the tanks.

For each of the cases considered the maximum axial compression in the shell was estimated based on the published methods and from time-history analysis, and the results were compared with the elastic-plastic buckling capacity recommended by Rotter (1985) in Equation 5:

$$N = 0.6E\frac{t}{R}\left[1-\frac{(pR)^2}{tf_y}\right]\left(1-\frac{1}{1.12+s^{1.5}}\right)\left[\frac{s+f_y/250}{s+1}\right] \qquad (5)$$

where $s = \frac{R}{400t}$.

In the above relation *N* is the maximum allowable axial compression per unit length of the shell, *p* is the maximum internal pressure and *t* is the shell thickness.

The various estimates of shell axial stress were also compared to the data obtained from field observations and benefits and/or shortcomings of these approaches were discussed. Parametric analysis was also carried out to investigate the effect of thickness of the base plate, shell thickness and earthquake shaking intensity on the response of inspected tanks to the earthquake.

3. DESCRIPTION OF STRONG GROUND MOTION

The Silakhor earthquake of March 31, 2006 occurred in Lorestan Province in western Iran. It had a magnitude of M_L=6.1 and the maximum ground acceleration was recorded in Chalanchulan station with the peak values of about 4.32 and 5.24 m/s^2 in the horizontal and vertical components respectively (BHRC, 2006). The acceleration time-histories of these components are shown in Figure 1.

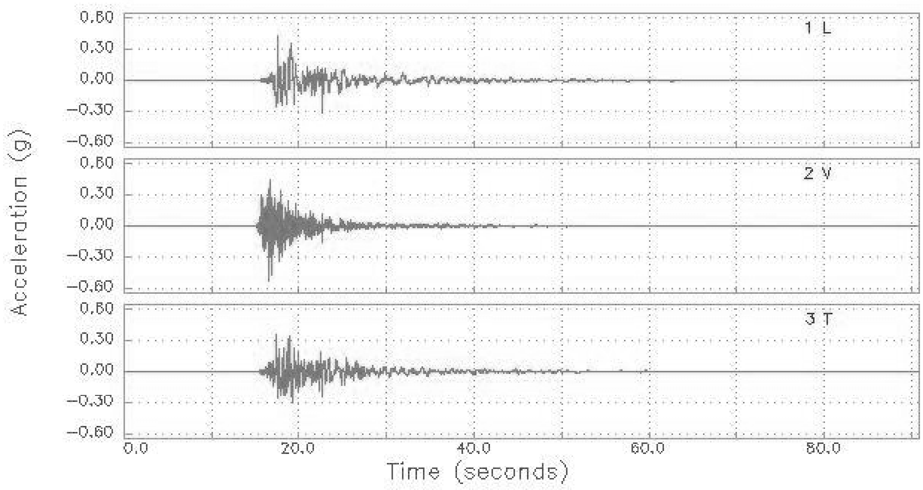


Figure 1. Accelerograms from Chalanchulan station (BHRC, 2006).

4. DESCRIPTION OF TANKS

During the Silakhor earthquake many un-anchored cylindrical steel tanks were subjected to strong ground motion. Some of the affected tanks were ground supported and others were elevated. The elevated tanks would have been subjected to modified earthquake response of the structures supporting them. This paper focuses on ground-supported tanks, which were directly subjected to the ground motion.

Most of the tanks affected by the earthquake were located in the "Lorestan" sugar factory in Chalanchulan, which is situated close to the Chalanchulan accelerograph station. For this study two un-anchored storage tanks were selected that were damaged by the earthquake and with different height-to-diameter ratios (H/D).

Tank "T1", was a fixed roof cylindrical tank of H/D = 0.55 representing a broad tank aspect ratio. Tank "T2", was a fixed roof tank of H/D = 1.2 representing a tall tank aspect ratio. Both of the tanks were made of steel and contained fuel oil. During the earthquake tanks T1 and T2 were approximately 90% and 85% full respectively. The tank dimensions are summarized in Table 1. In this table "h" denotes the height of the contained liquid.

Table 1. Specifications of considered tanks

Tank Name	H (m)	D (m)	h (m)	Anchorage	Roof Type
T1	8.6	15.3	7.97	Un-anchored	Cone Roof
T2	4.5	3.8	3.2	Un-anchored	Cone Roof

Tank T1 was constructed from 6 shell courses of 1.43 m in height. The thickness of wall shell of this tank varied from 12 mm at the base to 6 mm at the top. Tank T2 was constructed from three shell courses of 6 mm thickness.

5. ANALYTICAL ESTIMATION OF TANK RESPONSE

The maximum sloshing wave heights calculated by Equations (1) to (3) based on the various published methods for both tanks are summarized in Table 2. As can be seen, there is a large variation in predicted wave heights. It is noted that the maximum sloshing wave height predicted by the NZSEE 1986 method (Equation 3) is much greater than the estimations of other analytical relations.

Table 2. Maximum sloshing wave height predicted by various published methods

Tank Name	d_{max} (m)	Reference
T1	0.29	Eqn 1 Myers 1997
	0.69	Eqn 2 MPO 1995
	1.60	Eqn 3 NZSEE 1986
T2	0.15	Eqn 1 Myers 1997
	0.35	Eqn 2 MPO 1995
	0.95	Eqn 3 NZSEE 1986

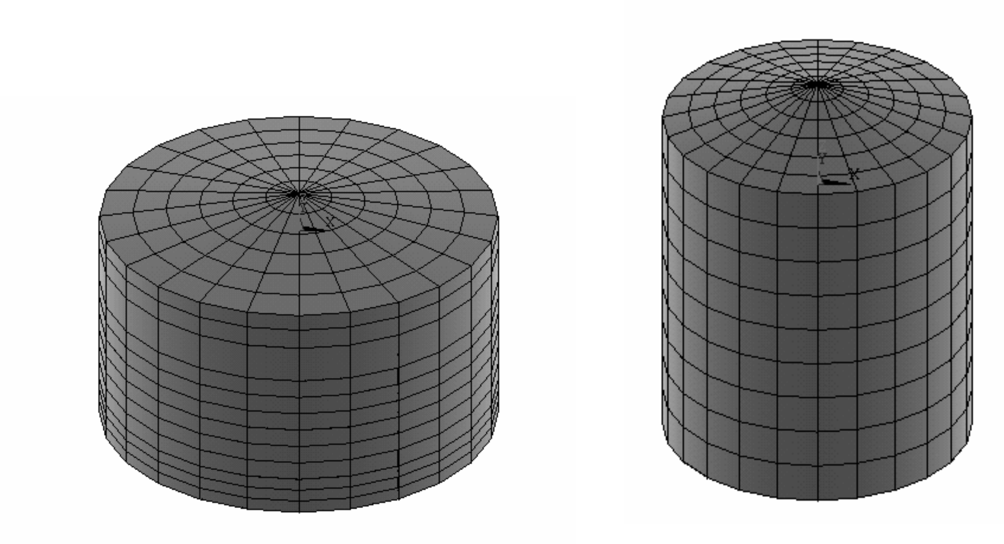
The maximum shell uplift for the tanks assuming either rigid or flexible foundations based on the method included in NZSEE 1986 (Equation 4) are summarized in Table 3. The NZSEE method predicts that both of the tanks would experience shell uplift under the peak ground acceleration of 0.44g from the Silakhor earthquake. As indicated in Table 3, the shell uplift of tank T2 is about ten times greater than shell uplift in tank T1. Moreover, for both tanks the maximum shell uplifts expected on flexible foundations (i.e. soft soils) are two times greater than the uplifts predicted on rigid foundations (i.e. rock).

Table 3. Summary of the Shell Uplift using NZSEE 1986 Equation.

Model Name	Type of Foundation	*Shell Uplift* (mm)
T1	Rigid	4.45
	Flexible	8.9
T2	Rigid	10
	Flexible	19.9

6. TIME-HISTORY ANALYSIS

Time-history analysis was carried out on a non-linear finite element model to investigate the dynamic behaviour of the tanks to the Silakhor earthquake (Eshghi, Razzaghi, 2006) using the strongest (longitudinal) component of the accelerogram in Figure 1. The finite element models are shown in Figure 2.



(a) **(b)**

Figure 2. Finite element models of Tanks. a) Tank T1 b) Tank T2.

The following mechanical properties were considered for steel in the tank shell, cone roof:

Modulus of elasticity = 210 GPa
Poisson's ratio = 0.3
Density = 7850 kg/m^3
Yield Stress = 240 MPa

A bilinear stress-strain model with strain hardening of 10% was defined for the steel material. The density of contained liquid was assumed to be 920 kg/m^3.

For the analysis of the tank T1 model it was found that an integration time step of 0.5 milliseconds was sufficiently small to characterize the response. It was found necessary to reduce integration time step to 0.25 milliseconds for analysis of the tank T2 model to prevent the solution becoming unstable due to the large changes in stiffness occurring as the shell uplifts during the analysis.

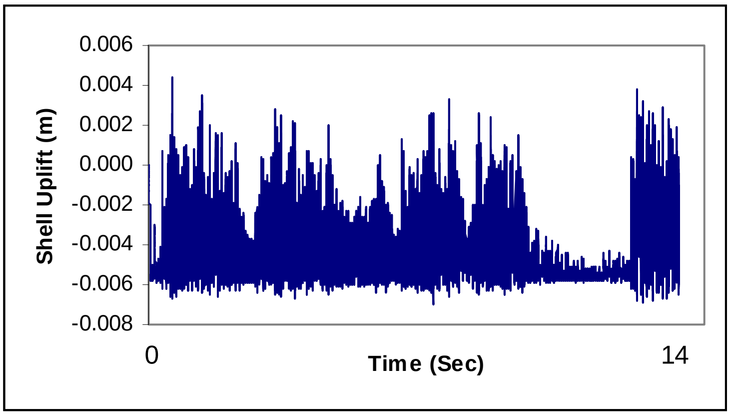
Results:

Key results from the finite element time-history analysis, including sloshing wave height, shell uplift displacements and maximum shell axial stress, are summarized in Table 4. Although both tanks displayed shell uplift response, the ratio of uplift-to-tank diameter in model T2 (Tall tank) is much greater than in model T1.

The vertical displacement time-history for tank T1 base perimeter is shown in Figure 3a. It is apparent that the assumption of rigidity of the foundation may lead to reduction of the shell uplift but the greater than axial membrane compression of the shell, compared with tanks supported on flexible foundations. Figure 3b shows the vertical displacement time-history of a node on the liquid free surface adjacent to fluid-shell interface of model T1 (ie. the sloshing behaviour).

Table 4. Summary of the results of time history analysis

Tank Model	Type of Foundation	Sloshing wave height d_{max} (mm)	Shell uplift v (mm)	Maximum Shell Compression (MPa)
T1	Rigid	80	4.5	102
	Flexible	81	4.1	20
T2	Rigid	70	36.8	170
	Flexible	70	10	17



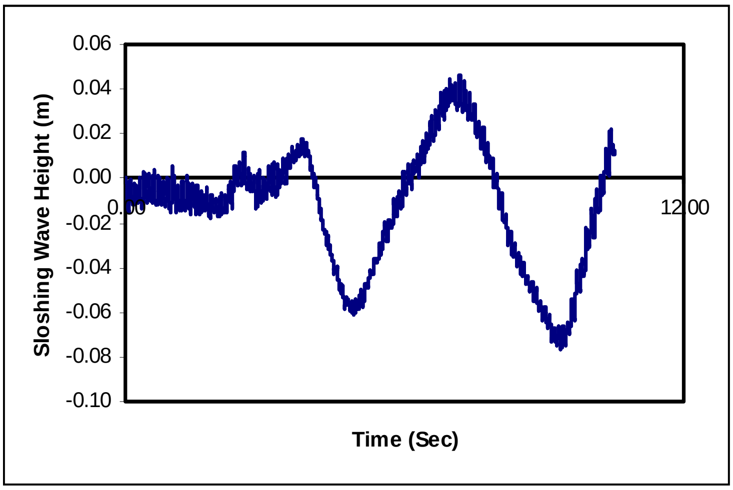


Figure 3. Typical time history response (a) Shell uplift (b) Liquid sloshing.

7. OBSERVED BEHAVIOUR OF THE REAL TANKS TO THE ACTUAL EARTHQUAKE

Tanks T1 and T2 suffered minor and moderate damage respectively due to the Silakhor earthquake. In the following sections the behaviour during the earthquake and observed damage of the two tanks is described.

7.1. Behaviour of tank T1 during Earthquake

There were two tanks with the specification of T1 at the Lorestan Sugar Factory. The general behaviour of the tanks during the earthquake was the same, although there were some differences in localized damage in the roof and base plates of the tanks. During the earthquake both of the tanks experienced shell uplift within the range 5-10 mm, which is consistent with the amount predicted by the time-history analysis in Figure 3. A slight downward settlement was observed in one of the tanks. Insulation on a heating pipe was cracked due to the shell uplift, as shown in Figure 4, but no observable damage occurred in the pipe itself. None of the rigid piping connections were damaged during the earthquake. There was visible distortion of roof plate due to the roof buckling in one of tanks, as shown in Figure 5. The location for this distortion is generally directly above the location where the most shell uplift was found to occur. No damage was visible to the roof lap joints or the roof-to-wall junctions. Since only one of the two identical tanks suffered distortion and damage to the roof, it seems that geometric imperfections may have been the main cause. Leakage of the liquid contents due to sloshing motion and overflowing did not occur in any of the tanks.

7.2. The Behaviour of tank "T2" during Earthquake

During the earthquake tank T2 uplifted and the concrete foundation was cracked. Figure 6 shows buckling that occurred in the tank shell in a location close to the damaged portion of the foundation due to the shell uplift. No over-topping or loss of liquid contents occurred.

(a)

(b)

Figure 3 (a, b): The evidences of shell uplift on tank foundation next to the base plate.

Figure 4. Cracks observed on tank piping system

Figure 5. Damage to tank roof

8. DISCUSSION

It is useful to compare the various predictions of published methods against the finite element time-history analyses results and actual observed behaviour of the tanks due to the Silakhor earthquake.

Shell buckling:
By comparing the finite element time-history results with elastic-plastic buckling capacity (Eqn. 6), it can be seen that tank T1 would not be expected to suffer shell buckling due to the Silakhor earthquake. However, the analyses for tank T2 indicated that the maximum axial compression at the bottom quarter of shell would exceed the buckling capacity of the wall. As indicated in Figure 6, the predicted buckling is consistent with the observed actual buckling behaviour of tank during the earthquake.

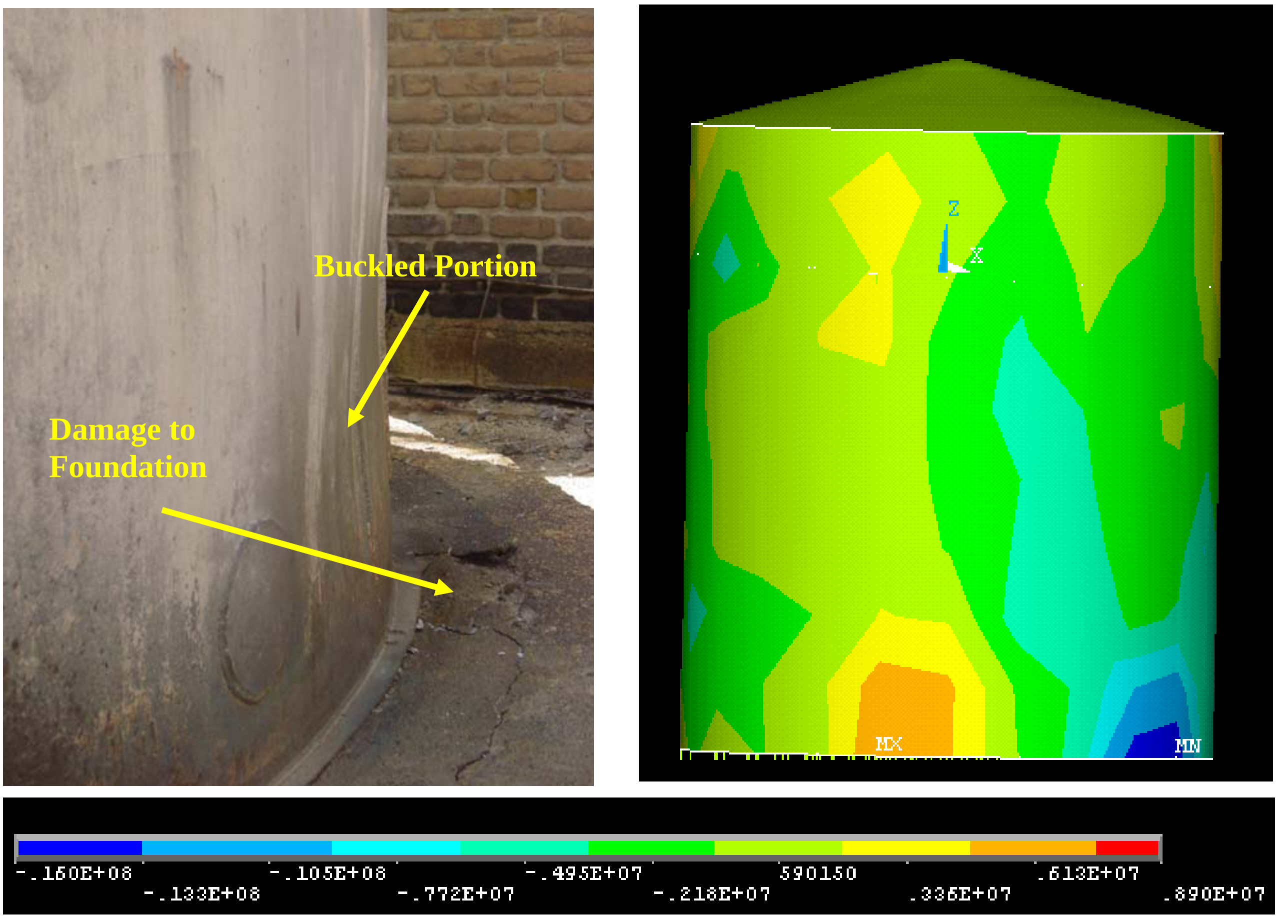


Figure 6. Tank T2 Shell buckling; actual behaviour compared with finite element analysis.

Sloshing of Liquid:
The maximum sloshing wave heights of liquid in the tanks were estimated using finite element analysis and different published formulae. As indicated in Figure 7, there is a large variation of predicted sloshing heights. For both tanks the maximum sloshing wave height estimated by the NZSEE 1986 method (Eqn. 3), at 1.6 m and 0.95 m for tanks T1 and T2 respectively, is obviously much greater than the results of either numerical analysis and/or other published methods. Since both of tanks were about 90% full it would appear that adequate freeboard was available to prevent roof damage or loss of contents. Therefore it is considered that the maximum sloshing wave height given by the NZSEE1986 method is over-estimated. As indicated in Figure 7, the variation in sloshing wave heights for tank T2 is still large, but less than the variation for tank T1.

The sloshing wave heights obtained by the finite element analyses were substantially less than predicted from published expressions for both tanks. Myers 1997 Method (Equation 1) gave predictions closest to the finite element analysis values. No measurements of the actual liquid sloshing heights in the real tanks was possible.

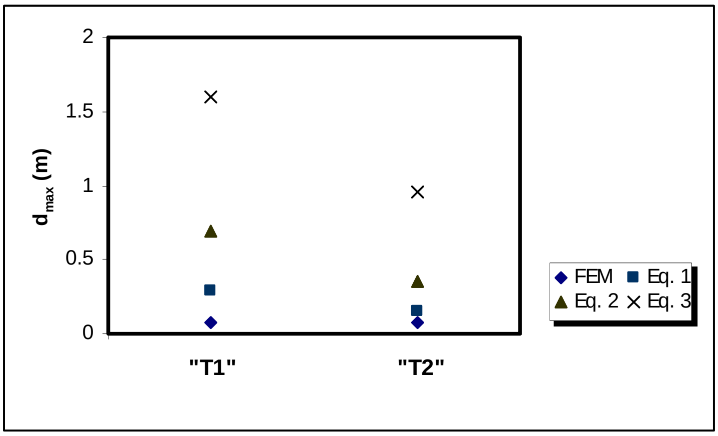
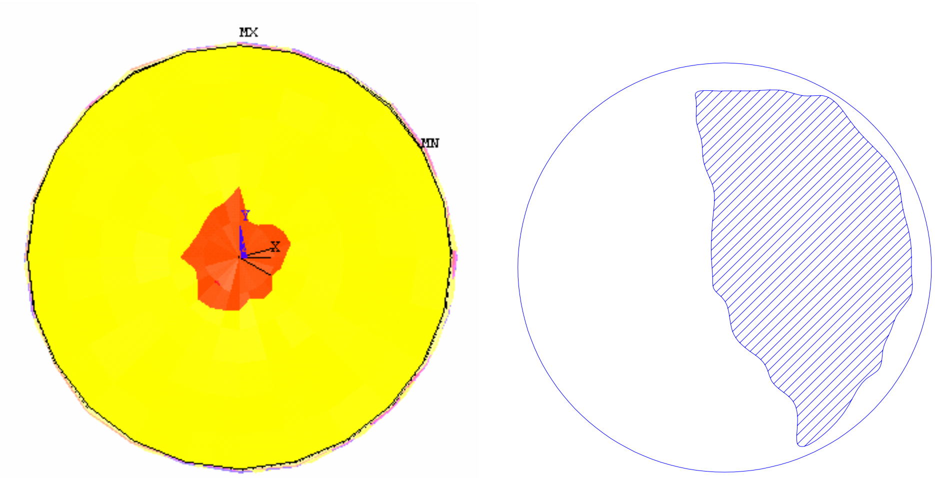


Figure 7. Predictions of Maximum sloshing wave heights by various methods.

According to the results of numerical analysis, flexibility of foundation does not have a significant effect on the maximum sloshing wave heights.

Damage to Tank Roofs:

For tank T1 the finite element analyses did predict localized plastic deformations around the top of the cone. In one of the real tanks, as indicated in Figure 5, extensive distortions occurred during the earthquake. This may be due to geometric imperfections of the cone-type roofs which were neglected in numerical models. Comparison of the finite element analysis predictions of roof distortions and the actual distortions in one of the tanks are shown in Figure 8.

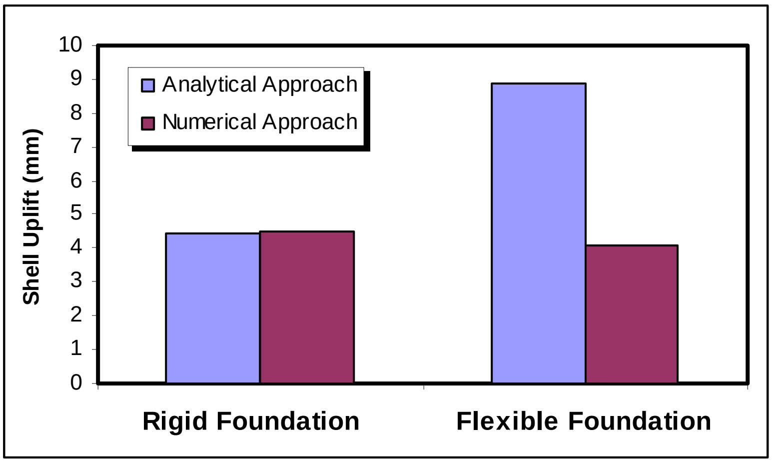


(a) **(b)**

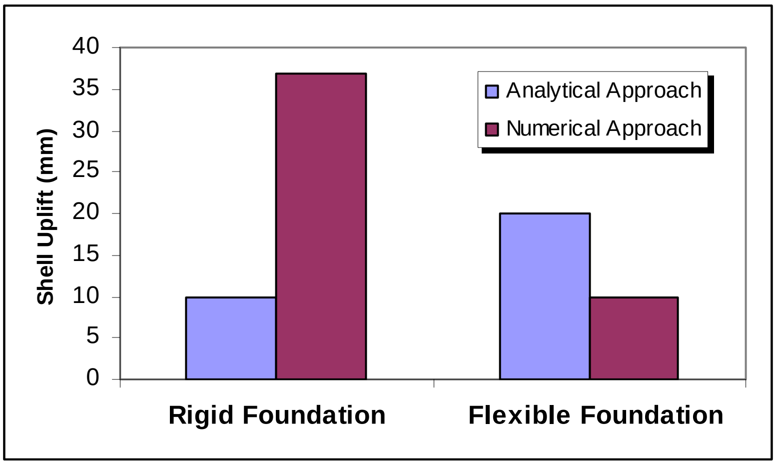
Figure 8. Comparison of roof distortions from finite element model with actual performance of tank T1 (a) FEM model (b) Schematic of observed damage.

The finite element analysis of tank T2 predicted no yielding or damage to the roof of the tank due to the earthquake. These results are consistent with the observed actual performance of the tank during the earthquake.

Shell uplift:
Figure 9 shows the comparison of maximum shell uplifts obtained from the finite element analyses and the NZSEE 1986 method. As shown in this figure, by increasing the H/D ratio, the difference between numerical and analytical results increased.



(a)



(b)

Figure 9. Comparison of shell uplift predict by finite element analysis and NZSEE method (a) Tank T1 (b) Tank T2.

9. CONCLUSIONS

During the "Silakhor" earthquake of March 31, 2006 in western Iran, several cylindrical steel tanks experienced strong ground motion and some damage. In this study, seismic responses of two affected unanchored cylindrical tanks were estimated by using various published analytical expressions and also by finite element analysis. Results of these analyses generally showed good agreement with the performance observed during the earthquake.

The results of this study showed that:

a) For the estimation of the maximum sloshing wave height, there is a large scatter among the different theoretical approaches. Variation of the predicted wave heights is less in tall tanks than in broad tanks. The wave heights from the NZSEE 1986 expressions are greater than the predictions of other methods. Numerical wave height estimates were found to be smaller than all the predictions obtained from published solutions. Observed sloshing wave heights from the actual earthquake were much less than predicted by the NZSEE equations, suggesting the the NZSEE predictions may be vary conservative.

b) For the broad tank numerical results supported the value of tank uplift estimated by using the NZSEE method. Observed shell uplift to the tank shell caused by the earthquake also closely matched the theoretical predictions. For the tall tank the maximum shell uplift obtained from finite element time-history analysis was greater than the analytical prediction using the NZSEE method. For this tank the maximum axial membrane stresses obtained from time-history analysis was much greater than predicted by the NZSEE method and, may exceed the theoretical buckling stress. This result is consistent with the observed behaviour of the real tank during the actual earthquake.

c) Results of time-history analyses showed no significant difference in shell uplift displacements for tanks on rigid foundations compared with tanks on flexible foundations. This finding is not predicted by the NZSEE method. By contrast the axial membrane stresses are much higher for tanks on rigid foundations compared to tanks on flexible foundations. Foundation flexibility does not appear to have significant influence on maximum sloshing wave height.

10. ACKNOWLEDGEMENTS

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