ALE Formulation for the Evaluation of Seismic Behaviour of Anchored and Unanchored Tanks

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Summary:

Estimation of the potential degree of risk for tank failure during an earthquake is very difficult to quantify since the liquid-tank system possesses many different nonlinear behaviour mechanisms which may be triggered simultaneously or separately depending on the characteristics of earthquake, contained liquid properties, fluid depth, dimensions of the tank, roof type, material properties, supporting conditions and stiffness of underlying soil medium. These nonlinear behaviour mechanisms can emerge in the form of elephant foot and diamond shape buckling at the tank wall, rupture at the junction between tank wall and base, buckling at the top of tank and roof, settlement at tank support system and foundation and large amplitude deformations at the base plate. For the case of unanchored tank, in addition to these mechanisms, uplift of tank base, sliding of the tank and successive contact and separation between base plate and foundation can be observed when tank subjects to seismic loadings. The analysis tool used to quantify the tank behaviour has to take into account the effects of all aforementioned factors. Since LS-DYNA is capable of handling complexities associated with the nonlinear transient seismic response of unanchored tanks it is utilized in this study. ALE technique and contact algorithms of LS-DYNA are used to model the coupling of tank and fluid and the interaction between tank base and soil, respectively. The results are compared with the provisions given in tank seismic design codes used in the current practice.

Keywords:

Liquid containment tanks, fluid-structure interaction, earthquake, sloshing, buckling

1 Introduction

Liquid containment tanks under seismic loads are generally analyzed using simplified methods in which several assumptions are made not only for tank response itself but also for fluid behaviour. In these methods, fluid is assumed to be incompressible and inviscid; fluid motion is irrorational and the wave amplitudes at the free surface are very small in comparison with the wavelengths and depths. It is common to treat the shell as rigid with constant thickness and fixed to its base. Even if tank shell flexibility is taken into account, it is assumed that it deforms in an analogous manner to a vertical cantilever beam in a prescribed shape without cross-sectional distortions. For the case of unanchored tank, structural response mechanism is considered as rigid body rocking motion of the shell. Tank base plate is idealized prismatic beam that rests on a rigid base. Furthermore, the hydrodynamic pressures exerted by an earthquake on rigid tank are assumed to be developed by two components. First component is caused by the fluid portion which moves in unison with the tank as a rigidly attached mass. The other portion moves independently, experiencing sloshing at the free surface. The fluid mass associated with the former component is called impulsive mass, whereas sloshing is generated by convective fluid mass. If the tank is considered as flexible, a third pressure component which is called the impulsive effect due to tank deformation is taken into account. In these simplified methods, the simultaneous interactions of these components are ignored and the results of the each component are combined with different techniques such as square root of sum of squares (SRSS) or direct sum.

The provisions given in current tank seismic design codes are based on these simplified methods since they are practical and easy for application. However, real behavior of storage tanks includes many complexities which are caused by material yielding, large amplitude free-surface sloshing, non-linear fluid structure interaction, high deformations of tank base and shell, soil-tank interaction, successive separation and contact between the base plate and foundation and plastic rotations of tank base plate. These nonlinear behavior mechanisms result in different failure modes such as buckling at the tank shell (elephant foot buckling or diamond shape buckling), separation of the junction between the base plate and tank wall, uneven settlements at the tank base and rupture of the anchors. When the tank is unrestraint at its base, the response modes of the tank are controlled by the uplift behavior of the base plate. Although uplift can not cause directly any damage on tanks, it triggers buckling at the tank wall which remains with contact. The large vertical displacement due to uplift may cause breaking of pipes which are connected to the tank due to lack of enough flexibility. Moreover, uplift induces large out-of-round distortions of the tank cross section especially in tanks without a roof.

Experiences from recent earthquakes revealed that code provisions may not be adequate to accurately predict the complicated seismic response of cylindrical tanks. Therefore, more enhanced methods are required to evaluate tank behavior comprehensively in order to prevent future earthquake tank failures. Finite Element Method permits to take into account all complexities associated with tank-fluid-soil system simultaneously. Therefore, in this study, analysis capabilities of general purpose finite element code LS-DYNA are utilized to investigate the seismic behavior of anchored and unanchored liquid storage tanks. An explicit integration procedure for the fluid tank system is employed using Arbitrary Lagrangian Eulerian (ALE) formulation to model the coupling effects between fluid and structure. The interaction forces between unanchored tank and foundation soil including the successive contact and separation between them are captured by contact modeling algorithm of finite element method.

2 Performance of Liquid Storage Tanks during Earthquakes

The dynamic behaviour of liquid storage tanks is different than conventional buildings, since tanks are exposed to hydrodynamic forces under earthquake motion and these forces have to be taken into account in their seismic design. In addition to their hydrodynamic aspects, energy dissipating capacity of these structures is very low therefore they have low ductility and redundancy. Furthermore, the natural periods of these structures occupy two widely separated ranges. The typical periods of sloshing are very long, up to 6-10 seconds for very large tanks whereas the coupled vibration modes of elastic shell and the contained liquid have periods less than 1 second. So, tanks respond to earthquake motions as two separated system. An earthquake near the tank site containing high frequencies can excite coupled system, but relatively small first mode sloshing. On the other hand, ground motion resulting from a large earthquake at far distance with low amplitude and long period can generate large amplitude first sloshing mode but not considerable coupled vibration modes. These aspects make these structures more vulnerable than the conventional structures against earthquakes.

Consequences of tank damages are not generally limited to the economic value of tanks and their contents and financial losses due to disruption of production. In most cases, failure of such structures has been threatened human lives and caused long-term environmental damages. For example, during the 1906 San Francisco earthquake several water supply tanks damaged and the fires could not be controlled due to lack of water. Following the Nigata and Alaska earthquakes of 1964, failure of tanks containing petroleum products led to spillage of toxic chemicals and liquefied gases from damaged tanks. Released dangerous liquids and gas clouds caused disastrous effects in populated areas.

Greenville-Mt Diablo earthquake of 1980 damaged various wine tanks settled close to epicentral area [1]. The common feature of damaged tanks was that they were unanchored to their foundation and completely full. The elephant foot buckling were dominant failure mode for broad tanks, while tall tanks suffered a diamond shaped buckling spreading around the circumference.

Several unanchored cylindrical ground supported tanks located at six sites within the oil producing area near to epicentral region damaged in the form of elephant foot buckling, joint rupture, top shell buckling, bottom plate rupture and floating roof failure during the 1993 Coalinga Earthquake [2]. The earthquake resulted in large oil spillage over the top of many tanks and secondary damages occurred in pipe connections, ladders, etc.

A few incidents of damage to tanks of old and modern design were induced by the Loma Prieta earthquake of 1989. Uplift of large unanchored tanks led to the failure of rigidly attached appurtenance such as piping and conduits, and in turn, led to tank rupture and the loss of contents. The damaged caused by the uplifting of tank walls was most likely associated with sloshing [3].

The Northridge earthquake of 1994 caused severe damage to a number of cylindrical liquid storage tanks, and even resulted in the collapse of a tank. Typical modes of tank damage, primarily buckling of the shell, failure at the roof-shell connection, base uplift, anchor failure and elephant-foot buckling near its base, were observed through the affected area. Some of them suffered damage at the base (tearing and buckling) and at the roof (collapse of the wood trusses). Several others emptied due to inlet-outlet pipe damage from rocking [4].

The Kocaeli earthquake of 1999, Turkey, occurred in a region with densely populated heavy industrial facilities and most of the tanks in this area were heavily damage. Sloshing lead to break of seal of tanks with floating roof and metal to metal contact between floating roof and tank wall ignited the fire which could not put out several days. Buckling of the tank shell caused the separation of the piping connections. Rupture of the connection between tank shell and roof caused leakage of petroleum products which led to environmental pollution.

3 General Seismic Behavior of Anchored Tanks

An anchored tank which is rigidly fixed to a substantial foundation has the ability to resist seismic overturning moment and responds this moment in a manner like a circular cylinder shell moving in its modal forms (shell buckling modes) with out-of-round cross-sectional distortions (circumferential cos $n\theta$-type modes, $n>1$). However, tank design codes assume that anchored tanks experience cantilever beam type motion without cross-sectional distortions (cos θ type mode) considering earthquake motions tend to strongly excite this type of mode only. Yet, experimental studies verified that earthquake motion excites these higher order out-of-round distortions (cos $n\theta$-type modes) from circular form of cross section of the tank due to probably geometric imperfections. These higher order distortions increase the axial compressive stress on the wall which, in turn, accelerates the buckling. Since these higher order distortions of the tank cross section are of considerable importance for the overall structural response, the simple cantilever type model adopted in the design codes may be revised.

4 General Seismic Response of Unanchored Tanks

The anchoring of a tank is not practical to construct and considerably expensive because it needs a large number of bolts and suitable attachments onto the tank wall. Improperly designed bolts can tear tank wall. Also, a massive foundation is required especially for large tanks. In practice, it is more common to construct tank shell on a simple ringwall foundation or directly on the compacted soil due to these disadvantages of anchoring.

Yet, the dynamic behavior of an unanchored tank is quite different than that of anchored. The partial uplift of the tank bottom plate caused by the overturning moments controls the highly nonlinear dynamic behavior of such structures. Since uplift represents the stiffness loss of the whole system, the frequencies of the coupled fluid-tank system decreases and the axial compression forces in the tank wall increases. This increment in axial compression stress makes an unanchored tank more prone to buckling than that of anchored. The corresponding mode shapes, the damping values and the dynamically activated pressures are changed after uplift is observed.

5 Advantages of ALE formulation for Tank Problems

In order to solve complex tank-fluid interaction problems, an appropriate numerical simulation method, which can cope with large deformations of free surface of the fluid and the structure and accurately predicts the hydrodynamic forces due to the high-speed impacts of sloshing liquid on a tank wall and roof, is required. The nonlinear finite element techniques with either Lagrangian and/or Eulerian formulations may be employed as a numerical method to model tank problems. But, most of the Lagrangian formulations used to solve such problems fails due to high mesh distortion of the fluid. The Arbitrary Lagrangian Eulerian techniques with or without multi-material formulations are capable of keeping mesh integrity during the motion of the tank.

6 Tank Model

Metal cylindrical ground-supported tanks are widely constructed in Turkey especially to store petrochemical products. However, there is no Turkish design code for the seismic analysis of tanks and liquid storage tanks are designed as per minimum requirements of international codes. API 650 [5], Eurocode 8 [6] and NZSEE [7] standarts are the most commonly referred ones among the others in current practice. In this study, a real tank model with typical proportions and material properties constructed in Turkey are analysed under the Turkish Seismic Code [8] design spectra compatible earthquake record including all three components.

The tank under consideration has a radius of 24 m and a total height of 18 m. The flat tank roof is constructed on a set of radial beams and rafters which are supported by columns. Tank shell consists of 9 courses which are tapered from bottom to top. The thicknesses of the bottom plate and the first shell course nearest to the bottom are 0.007 m and 0.020 m, respectively. The thickness of the tank shell decreases 0.002 mm at each two courses and it reaches 0.012 m at the top course. The steel of the cylindrical shell, the roof, base plate, columns and roof rafters has a modulus of elasticity E = 200 GPa, poison's ratio of 0.30, and mass density of 7800 kg/m^3. Steel material is assumed elastic-perfectly plastic with a yield stress 3.55 10^8 and the reserve strength due to strain hardening is ignored. Since, earthquake experiences revealed that almost full tanks are more vulnerable against to damage, the liquid level of approximately 90% of the height of the tank is assumed for the tank to be analyzed and water (ρ = 1000 kg/m^3) is filled up to a height of 16 m. The analyses are performed on the same tank model under two support conditions. In one case, the base of the shell is rigidly clamped to the rigid foundation; in the other case, no restraint against translational and rotational degree displacements is provided and tank is assumed directly resting on a rigid base.

7 Selection of Records for Seismic Analysis

Seismic design codes generally define ground shaking in the form of a response spectrum of acceleration and allow using response spectrum compatible earthquake records for linear and nonlinear time history analyses. One of the methods to obtain response spectrum compatible record is based on scaling of the selected real earthquake records in time domain by simply multiplying the record up or down in a uniform manner. But, the real earthquake records which are selected for scaling to match elastic response spectrum specified in the code have to possess similar characteristics (magnitude, distance, site condition and faulting type) with the site under consideration. Also, in order to preserve non-stationary characteristics of the initial time history, it is essential to start with an acceleration time history whose spectrum is as close to the target spectrum as possible in the period range of interest. A close initial fit also ensures a speedy convergence to the design values.

Moreover, scaling factor should not exceed certain limits depending on the type of problem to which the resulting motion will be applied. For analysis of linear elastic structures an upper limit of 4 could be accepted [9 and 10], for nonlinear analyses scaling factors in the range of 0.5 to 2.0 are advised.

In this study, the earthquake records, which are used for the nonlinear dynamic time history analyses for tank under consideration, are obtained by the time domain scaling procedure which employs the least square method [11]. The response spectrum specified in Turkish Seismic Code [8] for the local site class Z1, which represents rock, and 1^{nd} degree earthquake zone (A_o = 0.4), which corresponds to very high seismic risk zone, is selected as target spectrum. The earthquake records available in the Pacific Earthquake Engineering Research (PEER) Center, NGA strong motion data base [12] are utilized to find the best matched real earthquake records to the target spectrum within the period range of interest. The importance factor of tank is assumed as 1.5 and response modification factor is considered as unity. Within the 4062 records from 92 shallow crustal earthquakes in active tectonic regions around the world, the H-E05140 component of Imperial Valley earthquake of 15 October 1979 recorded at the El Centro Array #5 station is obtained as the best matched earthquake record. Since transient analysis of tank is carried out for all three components of the earthquake, the other horizontal and the vertical components of the same earthquake record are also scaled to match the corresponding spectrum. Due to the fact that the vertical response spectrum is not specified in the Turkish Seismic Code, the two thirds of the horizontal spectrum coefficients are used for those of vertical. The scaling factors are obtained as 1.29, 1.03 and 1.15 for two horizontal and vertical components of the selected earthquake record, respectively. The scaled time acceleration records of Imperial Valley earthquake and corresponding response acceleration spectra along with the Turkish Seismic Code [8] design spectrum defined for (Z1, Ao=0.4, I=1.5) are given in Figures 1 to 3.

8 Finite Element Analysis of Anchored and Unanchored Flexible Liquid Storage Tanks

The tank model whose dimensions and properties are given in the previous section are analyzed under the three components of the selected earthquake record for two different support condition: unanchored and rigidly fixed. For the unanchored tank case, a rigid shell is used to represent the ground underlying the tank and the interaction between ground and the tank base is modeled with the surface to surface contact algorithm. Static friction and dynamic friction are taken into account with coefficients of 0.50 and 0.45, respectively. The support boundary conditions of the anchored tank model are supplied restraining all rotational degree of freedom of the tank base nodes. Arbitrary Lagrangian Eulerian (ALE) description of the liquid-structure interface is employed in order to enforce compatibility between structure and liquid elements and the nodes at the interface of fluid and structure are merged.

Four noded fully integrated shell elements with 3 integration points through the thickness are used for the discretization of tank. The resulting tank model has 5668 shell elements. Total number of beam elements which construct the radial beams, rafters and columns is 649. The liquid inside the tanks is discretized to 20736 ALE single material fluid elements with a total of 24508 nodes. *MAT_NULL material model with *EOS_LINEAR_POLYNOMIAL equation of state is used for fluid. In the numerical simulations, both material and geometric nonlinearities are considered in order to accurately determine stress, strain and strain rate distributions throughout the tank and fluid. The command *ALE_REFERENCE_SYSTEM_GROUP is utilized to allow mesh motion for the Eulerian elements. The model is loaded with gravity (g = 9.81 m/sec2) with *LOAD_BODY_Z with a ramp function.

9 Analysis Results

The seismic behavior of the unanchored tank is controlled by the uplift mechanism of the base plate. Since the loading is three-dimensional, uplift is observed all the nodes around the circumference of the base plate. But, the highest uplift displacement (0.0761 m) occurs at a point on the Y axis (Y=24 m) when the acceleration amplitude of the X direction component of the earthquake motion reaches its highest value at 6 sec.

The plastic strain initiates to appear at the tank shell of the unanchored tank at 5.2 sec when hoop stress reaches its yield value before base plate experience uplift and axial compressive stress level is at the order of magnitude of $1\ 10^7$ at that moment. At 5.48 sec, west side of the tank starts experience uplift which causes plastic deformations to propagate larger area at the opposite side which is in contact with the ground. The axial stress at this region reaches a value which is higher than the critical theoretical buckling stress (Table 1) and concentrates over the relatively narrow zone of contact, its peak value is 1 10^8 Pa. Also, when the tank hits the ground axial compressive stress increases higher values and extends very large area along the tank height. After 8.5 seconds, base uplift subsides and axial compressive stress never reaches again higher values and remains under 5 10^7 Pa whereas the axial compressive stress in the anchored tank wall does not exceed 5 10^7 Pa. Therefore, uplift mechanism of the base plate is the only responsible factor for the increase in the axial compressive stresses in the tank shell.

For the anchored tank, plasticity at the tank wall appears at the same time with the unanchored tank when hoop stress exceeds yield stress but it extends larger region around the tank circumference. In contrast to unanchored tank, almost entire shell of the anchored tank behaves beyond the plastic limit (Figures 4 and 5). The maximum plastic strain is 0.004 and 0.001 for the anchored and unanchored tanks, respectively.

Although very large cross sectional distortions are not observed due to restrictive action of the complex roof configuration which consists of rafter and beam elements, the cross sections of the both tanks do not remain circular and tanks do not vibrate as a single degree of freedom system as considered in the codes. Especially for the anchored tank case, the circular cylinder tank shell moves in its modal forms and shell buckling modes govern the response of the system. Low amplitude outward bulging of the lower part of the tank wall is observed for both tank cases. The unanchored tank experiences smaller radial and roof deformations than the anchored tank. Due to high joint stresses, the junction of the tank shell and the roof experiences plasticity for both support conditions.

Simulation results confirm that sloshing behavior of the tank is very similar for both support conditions. Sloshing initiates to be effective on the system response after 5.7 sec. At 7.4 sec, hydrodynamic pressure generated by sloshing causes increase in the joint stresses at the junction of the tank wall and roof and activates the plasticity at this region. High stress concentration at the junction propagates all around the circumference and plasticity accompanies it. Sloshing waves causes roof deformations at the region where they rise.

10 Code Comparisons

The response parameters of the anchored and unanchored tanks obtained from numerical analysis are compared with the predictions of API 650 [5], Eurocode 8 [6] and NZSEE [7] standarts in order to verify consistency of results (Table 1). These codes generally use different design methodologies. For example, API 650 is based on the allowable (working) stress design (ASD) methods, whereas NZSEE (1986), and Eurocode-8 specify seismic design forces at the strength design level. In strength design, factored loads are used and they correspond to ultimate level. In order to compare the seismic response parameters of tanks obtained by API 650 (2006), NZSEE (1986), and Eurocode-8, a scaling factor of 1.1 (Whittaker and Saunders, 2008) is used to convert seismic design forces from strength design level to allowable stress design level.

Tank seismic design codes consider elastic and elasto-plastic buckling mechanisms as the most relevant failure mode for tanks. Elephant foot (elasto-plastic) buckling is caused by the combined action of vertical compressive stresses exceeding the critical stress, hoop tension close to the yield limit and local bending stresses due to the restraints at the tank base. Diamond shape buckling is an elastic buckling phenomenon due to the presence of high axial compressive stresses. Therefore, the evaluation of shell stress level is crucial for the assessment of the buckling.

The hoop stresses at the tank wall predicted by code equations and obtained by numerical simulations are very consistent for both support conditions, but these values are higher than the allowable stress which is equal to yield strength of the tank material. For the anchored tank case, axial compressive stresses obtained from code provisions and numerical results are match well and they are lower than the axial stress required to cause elastic (diamond shape) buckling. However, in the numerical simulations, the axial stress acting on the unanchored tank wall which remains in contact with the ground reaches a maximum value of 1.00 10^8 Pa which is higher than the critical theoretical buckling stress (Table 1). Although NZSEE (1986) predicts this stress lower than the critical stress, it is higher than the allowable value defined in this code and high axial compressive stresses may cause elastic buckling of the unanchored tank shell according to NZSEE (1986) code provision. However, simulation results show that these stresses do not cause elastic buckling of the tank shell. Instead, they affect the development of the small amplitude outward bulging of the lower part of the unanchored tank shell which is mainly caused by the excessive hoop stresses for this case. Also, the small amplitude elephant-foot buckling of the anchored tank wall observed from numerical simulations is primarily generated by hoop stresses which exceed the yield limit of the tank material.

Sloshing wave height observed from simulations is considerably lower than that of specified in the codes. All code provisions overestimate displacement of the free surface and recommend very conservative value to provide necessary freeboard in order to prevent spilling of liquid and possible damage to the tank roof due to sloshing. Base uplift of the unanchored tank predicted by API 650 and NZSEE guidelines is significantly greater than that of obtained by the numerical analysis.

11 Conclusions

Fluid-structure interaction response of a liquid containment tank with two different types of support conditions are evaluated under a three component of a selected earthquake ground motion record using the explicit time-integration capabilities of the LS-DYNA code. A comparison between the earthquake response of anchored and unanchored tanks are presented. It is observed that, the dynamic behavior of the unanchored tank is quite different from that of anchored tank. The partial uplift of the tank bottom caused by the overturning moment leads to increase in axial compressive forces in the tank wall. For both tank types the predominant failure mode is small amplitude outward buckling of the tank wall and plasticity at the connection of the tank roof and the wall due to high joint stresses. Plasticity is observed most parts of the wall of the anchored tank whereas only the lower bottom part of the unanchored tank represents plastic deformation. The axial membrane compression stress in the unanchored tank walls reaches higher values than allowable stresses specified in the codes and it exceeds even the theoretical buckling stress. Code provisions give very conservative results for both maximum free surface wave height and base uplift displacement.

12 Literature

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13 Acknowledgement

This work is supported by the Turkish Scientific and Research Organization Engineering Research Group (TUBITAK-MAG) through contract numbers 108M607.

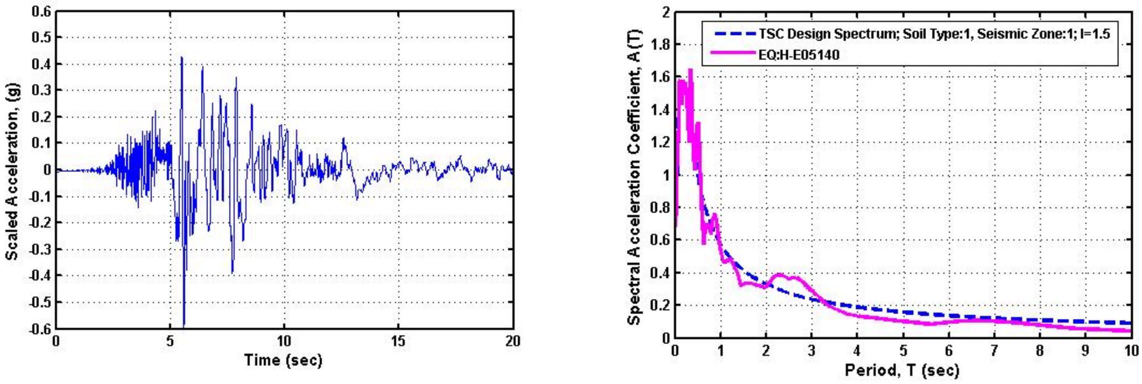

Figure 1: The scaled time acceleration record (H-E05140) of Imperial Valley earthquake and corresponding spectrum along with the Turkish Seismic Code [8] design spectrum (scale factor: 1.29)

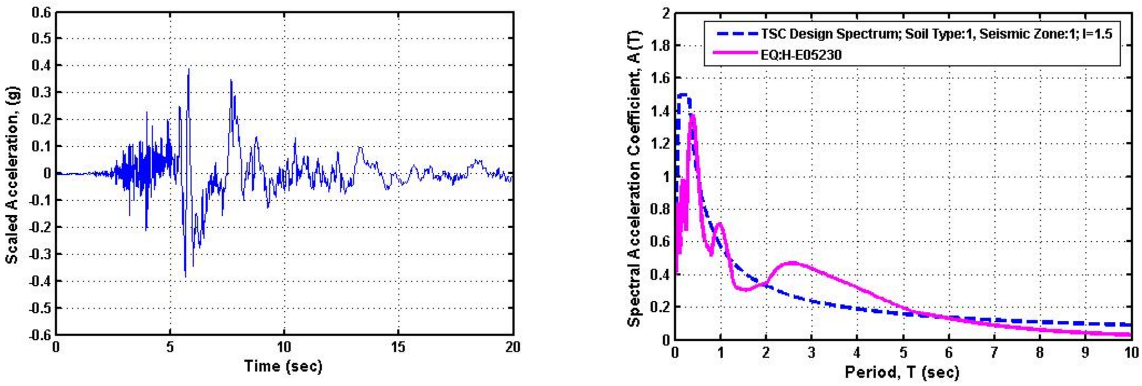

Figure 2: The scaled time acceleration record (H-E05230) of Imperial Valley earthquake and corresponding spectrum along with the Turkish Seismic Code [8] design spectrum (scale factor: 1.03)

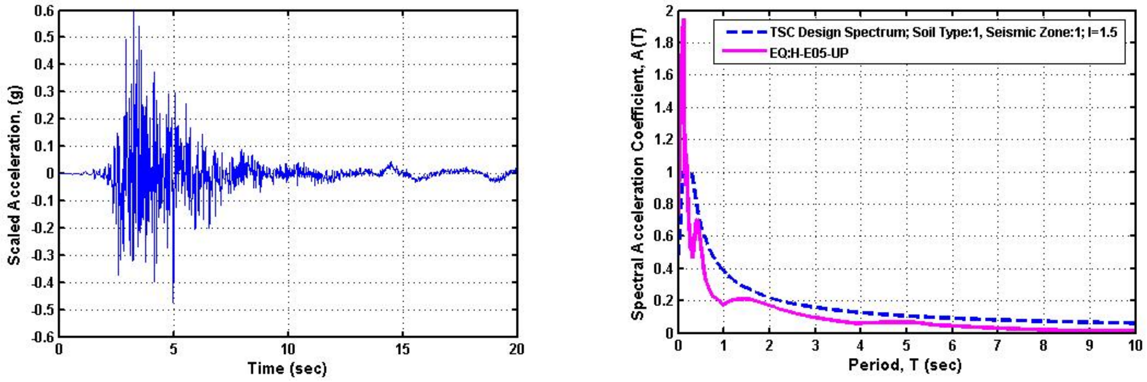

Figure 3: The scaled time acceleration record (H-E05-UP) of Imperial Valley earthquake and corresponding spectrum along with the Turkish Seismic Code [8] design spectrum (scale factor: 1.15)

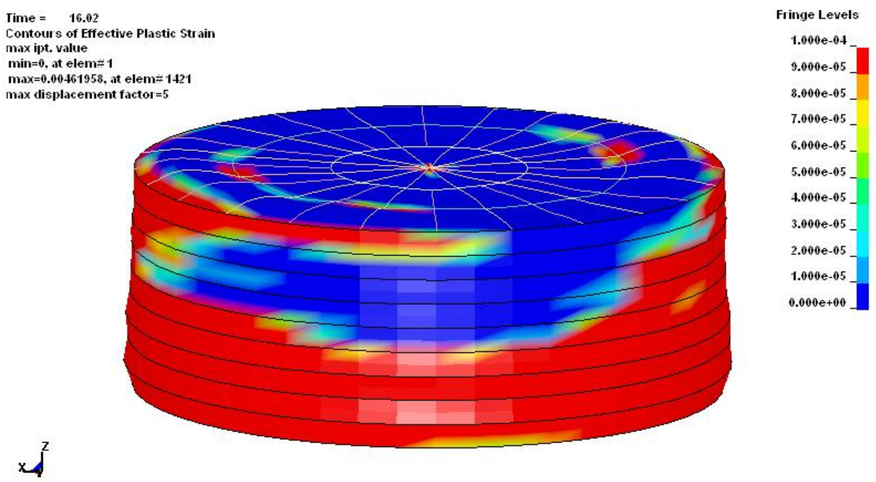

Figure 4: Plastic deformations and deflections of the roof of the anchored tank (displacements magnified 5 times)

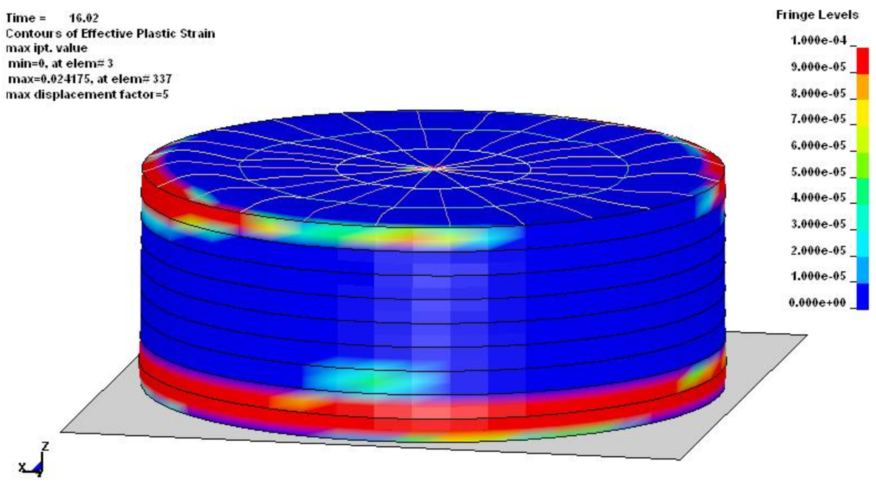

Figure 5: Plastic deformations and deflections of the roof of the unanchored tank (displacements magnified 5 times)

Table 1: Response parameters of the tank models as per code requirements (units: N, Pa, m and sec)

	Numerical	API 650	Eurocode 8		NZSEE (1986)
		Housner (1954)	Veletsos and Yang (1977)	Malhotra et al. (2000)	Veletsos (1984)
Impulsive Hor. Mode Period	-	0.29	-	0.29	0.31
Impulsive Ver. Mode Period	-	-	-	-	0.28
Sloshing Mode Period	-	7.91	7.93 / 4.26	7.95	7.93 / 4.26
Sloshing Wave Height	1.31 (anchored) 1.24 (unanchored)	3.94	3.31	3.94	3.34
Base Shear	$1.5\ 10^{8}$ (anchored) $6.9\ 10^{7}$ (unanchored)	$1.64\ 10^{8}$	$1.95\ 10^{8}$	$2.06\ 10^{8}$	$1.81\ 10^{8}$
Overturning Moment (excluding base pressure)	$8.28\ 10^{8}$ (anchored) $4.40\ 10^{8}$ (unanchored)	$1.00\ 10^{9}$	$1.32\ 10^{9}$	$1.44\ 10^{9}$	$1.15\ 10^{9}$
Overturning Moment (including base pressure)	-	$3.12\ 10^{9}$	$3.33\ 10^{9}$	$3.52\ 10^{9}$	$2.90\ 10^{9}$
Shell Axial Membrane Stress for Anchored Tank	$5.0\ 10^{7}$	$3.34\ 10^{7}$	$4.20\ 10^{7}$	$4.57\ 10^{7}$	$3.69\ 10^{7}$
Shell Axial Membrane Stress for Unanchored Tank	$1.00\ 10^{8}$	*	**	**	$8.61\ 10^{7}$
Allowable Axial Membrane Stress					
Elastic Buckling	-	$3.48\ 10^{7}$	$7.10\ 10^{7}$	$8.59\ 10^{7}$	$8.23\ 10^{7}$
Elasto-Plastic Buckling	-	-	***	***	***
Hoop Stress	$4.0\ 10^{8}$ (anchored) $4.0\ 10^{8}$ (unanchored)	$4.92\ 10^{8}$	-	-	$4.68\ 10^{8}$
Allowable Hoop Stress	-	$3.52\ 10^{8}$	$3.55\ 10^{8}$	$3.55\ 10^{8}$	$3.55\ 10^{8}$
Uplift for Unanchored Tank	0.076	0.33	**	**	0.174
Plastic Rotation	-	-	****	****	0.115
Allowable Plastic Rotation	-	-	0.20 rad (11˚)	0.20 rad (11˚)	0.20 rad (11˚)
Radial Membrane Stresses in the Base Plate	-	-	****	****	$5.41\ 10^{8}$

According to classical buckling theory $\sigma_{cl} = \dfrac{1}{\sqrt{\left[3(1-\nu^2)\right]}} \dfrac{E\,t}{R} = 9.20\ 10^{7}\ Pa$

* Tank should be anchored as per minimum API 650 requirements.
** The parameter is the out of range of the graphs given in the corresponding code.
*** Elasto-plastic buckling check formulation gives negative value.
**** For the estimation of this quantity, the uplift height is necessary.