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Comparison of API650-2008 provisions with FEM analyses for seismic assessment of existing steel oil storage tanks

Naghdali Hosseinzadeh [a,*], Hamid Kazem [b], Masoud Ghahremannejad [b], Ehsan Ahmadi [b], Navid Kazem [c]

[a] Structural Engineering Research Center, International Institute of Earthquake Engineering and Seismology (IIEES), Tehran, Iran
[b] Hamid Kazem, Department of Civil and Environmental Engineering, Amirkabir University of Technology (Tehran Polytechnic), Tehran, Iran
[c] Department of Civil & Environmental Engineering, Carnegie Mellon University, Pittsburgh, PA, USA

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ABSTRACT

API650-2008 is one of the prominent codes consisting of seismic specifications to design steel storage tanks for earthquakes resistance. In spite of the code's broad application, there are some failure modes such as slide bottom, elephant-foot buckling, sloshing and uplift needing more evaluation. In this paper, 161 existing tanks in an oil refinery complex have been classified into 24 groups and investigated using both API650-2008 rules and numerical FEM models. Failure modes and dynamic characteristics of studied models have been calculated by numerical FEM analysis and compared with code requirements. The results demonstrate that, in some cases, there are some imperfections in the code requirements that require further investigation.

1. Introduction

Storage tanks in refineries and chemical plants contain large volumes of flammable and hazardous chemicals. A small accident may lead to million-dollar property loss and a few days of production interruption (Chang & Lin, 2006). Regarding the importance of these systems, specially their seismic safety for avoiding the adverse consequences such as fires, explosions and environment pollution, better understanding of their seismic behavior still seems necessary. In last decades, Trade organizations and engineering societies such as the American Petroleum Institute (API), the American Institute of Chemical Engineers (AIChE), the American Society of Mechanical Engineers (ASME), and the National Fire Protection Association (NFPA) have published strict engineering guidelines and standards for construction, material selection, seismic design and safe management of storage tanks and their accessories. Most companies follow these standards and guidelines in design, construction and operation, but tank accidents still occur (Chang & Lin, 2006).

The significance of earthquake effects on steel storage tanks compels researchers to investigate tanks seismic responses. Therefore, many studies have been undertaken to understand seismic behavior of storage tanks under earthquake loading. Examination of the seismic response of a cylindrical steel liquid storage tank using a coupling method combining finite element and the boundary element was done by Cho and Cho (2007). They developed a general numerical algorithm, which can analyze the dynamic response of cylindrical steel liquid storage tanks in a three-dimensional coordinate system. Hosseinzadeh N. (2008) categorized 181 tanks in an oil refinery complex in Iran into 30 types in order to evaluate seismic vulnerability and to retrofit. The results showed that about 60 percent of the existing tanks were vulnerable and needed retrofitting or strengthening. Goudarzi and Sabbagh-Yazdi (2009) used a simplified model known as Mass Spring Model to evaluate the seismic response of liquid storage tanks. They verified the results of these simplified models by the Finite Elements Method (FEM). For most cases, results from the time-history analysis demonstrated good agreement with the simplified models. Evaluation of the earthquake performance of Turkish industrial facilities, especially storage tanks, in terms of earthquake resistance was studied by Korkmaz, Sari, and Carhuglo (2011). Tank structures were modeled by performing 40 different earthquake data sets using nonlinear time-history analyses. In this study, the vulnerability of storage tanks in Turkey was determined

* Corresponding author. Tel.: +98 2122831116; fax: +98 2122294979.
E-mail addresses: hosseinz@iiees.ac.ir (N. Hosseinzadeh), hamidkazem@aut.ac.ir (H. Kazem), m_ghahremani@aut.ac.ir (M. Ghahremannejad), Ehsanahmadi@aut.ac.ir (E. Ahmadi), nkazem@andrew.cmu.edu (N. Kazem).

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and the probabilistic risk was defined with the results of the analysis. Hosseini, Soroor, Sardar, and Jafarieh (2011) presented a simplified method for modeling the floating roof and its interaction with the tank wall, making it possible to use FEM for calculating the seismic response of tank-floating roof system.

In spite of extensive investigations on tank seismic behavior, some researchers were focused on validation of seismic codes. Presentation of field observations during past earthquakes and then using together with finite element analyses and published experimental results to assess the accuracy of current design guidelines, with special emphasize on EUROCODE 8 (EC8), part 4 was done by Hamdan F.H. (2000). An important aim of this work was to determine the various phenomena including sloshing, required free board, base shear, overturning moment, and the buckling strength for which current design guidelines need further development. Jaiswal, Rai, and Jain (2007) reviewed and compared ten seismic tank codes such as ASCE7 (2005), IBC (2006), Eurocode 8, NZSEE, ACI (350.3&371), AWWA (D-100&D-110&D-115) and API650-2005. This study has revealed significant differences among these codes on design seismic forces for various types of tanks. Reasons for these differences were critically examined, and the need for a unified approach for seismic design of tanks was highlighted. Wieschollek, Kopp, Hoffmeister, and Feldmann (2011) described the results of a survey on existing European and American codes such as Eurocode and ASCE7 with regard to their applicability to spherical liquid storage tanks and compared design outcomes according to these codes. The studies used numerical FEM modeling and calculation, as well as simplified models for the estimation of the dynamic properties of the tank structure. The resistance of the tank was compared to the action effect determined from the European and American codes.

The literature review shows that, except for a few cases of comprehensive studies, researchers have focused on evaluation of seismic behavior of liquid storage tanks using FEM with various assumptions; in only in few studies, codes are the main subject of discussion. Therefore, it can be concluded that there is a broad area for assessing and developing code provisions especially in the field of seismic behavior of steel storage tanks to improve the ability of these codes and guidelines to satisfy seismic design requirements to prevent structural failures when earthquakes occur.

The purpose of this study is to focus on API650-2008 (API STANDARD, 2008) provisions to assess whether this code is trustable for seismic design and evaluation of steel storage tanks. In particular, this study will determine the failure modes, such as slide bottom, elephant-foot buckling, sloshing and uplift, for which current design guidelines need further development. The configuration of this study consists of three main sections. In the first section, dynamic characteristics of API650-2008 (APPENDIX E) including structural periods of vibrations, slide bottom (base shear), elephant-foot buckling, sloshing and uplift are separately reviewed with their formulations and notations. In the second section, the studied tanks, analytical approach and research assumptions in FEM modeling are explained. Also, numerical verification results are presented in this section. The last section presents a comparison of the dynamic characteristics of both methods to emphasize differences between API650-2008 assessments and FEM analyses.

2. API650-2008 seismic provisions

In this section we will discuss suggested dynamic characteristics and performance of API650-2008 seismic provision. API650-2008 establishes minimum requirements for material, design, fabrication, erection, and testing for vertical, cylindrical, above-ground, closed- and open-top welded storage tanks in various sizes and capacities subjected to internal pressures and seismic forces. This standard applies only to tanks for which the entire bottom is uniformly supported and to tanks in non-refrigerated service. It is designed to provide the industry with tanks of adequate safety and reasonable economy for use in the storage of petroleum, petroleum products, and other liquid products. Also, it includes criteria to provide the stability of storage tanks against seismic excitations. Vertical acceleration effects shall be considered and combined with lateral acceleration effects. The following is the summary of considered criteria and the suggested dynamic characteristics in API650-2008 (APPENDIX E), which are the main focus of this study. The criteria are expressed as the required equations in SI Units.

2.1. Structural period of vibration

The pseudo-dynamic design procedures contained in the API650-2008 code are based on response spectra analysis methods and consider two dominant response modes of the tank and its contents impulsive (T_i) and convective (T_c). The design methods in the code are independent of the impulsive period of the tank. However, the impulsive period of the tank system, in seconds, may be estimated by Eq. (1). For the convective (sloshing) period, the first mode sloshing wave period, in seconds, should be calculated by Eq. (2) where K_s is the sloshing period coefficient defined in Eq. (3).

$$T_i = \frac{1}{\sqrt{2000}} \frac{C_i H \sqrt{\rho/E}}{\sqrt{t_u/D}} \tag{1}$$

$$T_c = 1.8 K_s \sqrt{D} \tag{2}$$

$$K_S = \frac{0.578}{\sqrt{\tan h(3.68H/D)}} \tag{3}$$

Where C_i is the coefficient for determining the impulsive period of the tank system, H is the maximum design product level, ρ is the density of fluid, E is elastic modulus of tank material, t_u is the equivalent uniform thickness of tank shell, and D is the nominal tank diameter.

2.2. Slide bottom (base shear)

Ground-supported, flat-bottom, liquid storage tanks shall be designed to resist the seismic forces calculated by considering the effective mass and dynamic liquid pressures in determining the equivalent lateral forces and lateral force distribution. The seismic base shear shall be defined as the square root of the sum of the squares (SRSS) combination of the impulsive and convective vibration mode components as determined by Eqs. (4)–(6).

$$V = \sqrt{V_i^2 + V_c^2} \tag{4}$$

where

$$V_i = A_i\left(W_s + W_r + W_f + W_i\right) \tag{5}$$

$$V_c = A_c W_c \tag{6}$$

A_i and A_c are the impulsive and convective spectral accelerations, respectively. W_s, W_r, W_f, W_i and W_c are shell, roof, floor, effective impulsive and effective convective weights, respectively. It should be noted that A_i is determined from 5% response spectrum, while A_c is computed based on 0.5% response spectrum. The periods of impulsive and convective responses are generally widely separated, and the impulsive period is much shorter than the convective period. When responses are widely separated, near-simultaneous occurrence of peak values could occur. However, the convective response takes much longer to build up than the impulsive response; consequently, the impulsive component is likely to be subsiding by the time the convective component reaches its peak. It is thus recommended that the combined impulsive and convective responses be taken as the SRSS of the separate components (NZS 3106, 1998).

2.3. Elephant-foot buckling

Elephant-foot buckling is an outward bulge just above the tank base which usually occurs in tanks with a low height to radius ratio (Hamdan, 2000). The bottom of the tank is usually subjected to a biaxial stress state consisting of axial membrane compression and circumferential hoop tensile stress. Dynamic hoop tensile stresses caused by the seismic motion of the liquid shall be determined by Eqs. (7)–(11).

$$N_i = 8.48 A_i g D H \left[\frac{Y}{H} - 0.5\left(\frac{Y}{H}\right)^2\right] \tanh\left(0.866\frac{D}{H}\right) \quad \text{For}\ \frac{D}{H} \geq 1.333 \tag{7}$$

$$N_i = 5.22 A_i g D^2 \left[\frac{Y}{0.75D} - 0.5\left(\frac{Y}{0.75D}\right)^2\right] \quad \text{For}\ \frac{D}{H} < 1.333\ \text{and}\ Y < 0.7D \tag{8}$$

$$N_i = 2.6 A_i g D^2 \quad \text{For}\ \frac{D}{H} < 1.333\ \text{and}\ Y \geq 0.75D \tag{9}$$

$$N_C = \frac{1.85 A_c g D^2 \cosh\left[\frac{3.68(H-Y)}{D}\right]}{\cosh\left[\frac{3.68H}{D}\right]} \quad \text{For all proportions of}\ D/H \tag{10}$$

where N_i is the impulsive hoop membrane force in the tank shell, N_c is the convective hoop membrane force in the tank shell, g is the specific gravity, and Y is the distance from the liquid surface to the analysis point. The dynamic hoop tensile stress should be directly combined with the product hydrostatic design stress in determining the total stress.

$$\sigma_T = \sigma_h \pm \sigma_s = \frac{N_h \pm \sqrt{N_i^2 + N_c^2 + (A_v N_h)^2}}{t} \tag{11}$$

where σ_T is the total stress, while σ_h and σ_s are the product hydrostatic the hoop stress in the tank and hoop stress in the shell tank due to impulsive and convective forces of the stored liquid, respectively. N_h is the product hydrostatic membrane, A_v is the vertical spectral acceleration and t is the thickness of the shell ring under consideration.

2.4. Sloshing

Sloshing waves of high amplitude often cause damage to the roofs of tanks and render them temporarily unserviceable. As a consequence, liquid spillage over the roof may either result in fires or in the loss of water supply used in putting out fires. Sloshing of the liquid within the tank or vessel shall be considered in determining the free board required above the top capacity liquid level. A minimum free board shall be provided per Table E-7 in API650-2008. The height of the sloshing wave above the product design height can be estimated by Eq. (12).

$$\delta_s = 0.5 D A_f \tag{12}$$

where D is the diameter of the tank, and A_f is the acceleration coefficient (%g) for the sloshing wave height calculation. A_f depends on the Seismic Use Group (SUG) specified by the purchaser. For instance, A_f for SUG III can be estimated using Eqs. (13) and (14).

$$A_f = K S_{D1}\left(\frac{1}{T_C}\right) \quad \text{When}\ T_C \leq T_L \tag{13}$$

Table 1
Geometric characteristics of representative tanks.

No.	Group name	Number of tanks in the group	Dimension of representative tank				Liquid density (kg/m^3)	Base support
			Tank diameter (m)	Tank height (m)	Liquid height (m)	Tank volume (m^3)		
1	RA	6	87.5	14.63	13.23	79,514	871	Unanchored
2	RB	7	45.6	14.63	14.13	23,064	754	Unanchored
3	RC	7	39.2	14.63	13.23	15,958	820	Unanchored
4	RD	23	37.2	14.63	14.13	15,349	1084	Unanchored
5	RE	2	37.2	14.63	14.13	15,349	814	Unanchored
6	RF	10	35.1	14.63	13.23	12,795	820	Unanchored
7	RG	5	33.3	14.63	14.13	12,299	835	Unanchored
8	RH	10	27.7	14.63	13.23	7968	804	Unanchored
9	RI	2	23.71	12.2	10.8	4766	1025	Unanchored
10	RJ	7	22.31	12.2	11.7	4571	1060	Unanchored
11	RK	2	21.66	12.2	10.8	3977	743	Unanchored
12	RL	3	19.36	12.2	10.8	3177	802	Unanchored
13	RM	2	17.64	9.78	9.28	2266	887	Unanchored
14	RN	7	13.7	11.2	10.8	1591	1020	Anchored
15	RO	2	13.69	8.3	7.8	1147	1840	Unanchored
16	RP	2	10.9	6.56	6.06	565	1800	Unanchored
17	RQ	15	10.67	9.11	5.58	498	994	Anchored
18	DA	16	24.39	12.8	11.98	5594	1060	Unanchored
19	DB	1	34.14	12.8	12.1	11,070	820	Unanchored
20	DC	2	43.9	14.63	13.87	20,983	720	Unanchored
21	DD	9	53.65	18.3	17	38,411	1020	Unanchored
22	DE	18	53.65	18.28	17.08	38,591	840	Unanchored
23	PA	2	14.62	12.81	12	2013	851	Anchored
24	PB	1	12.2	9.2	9.1	1063	851	Anchored

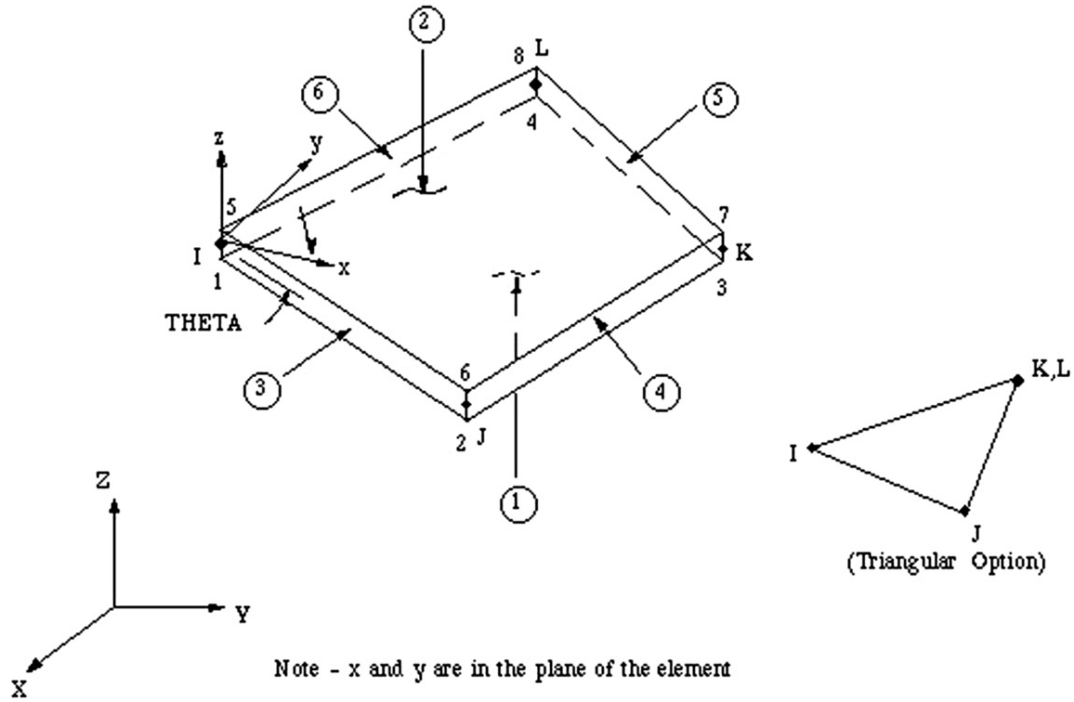

Fig. 1. Shell-63 element, elastic shell.

$$A_f = KS_{D1}\left(\frac{T_L}{T_C^2}\right) \quad \text{When} T_C > T_L \tag{14}$$

where K is the coefficient to adjust the spectral acceleration from 5% to 0.5% damping (1.5 unless otherwise specified), S_{D1} is the design (5% damped) spectral response acceleration parameter at 1 s based on the ASCE7 methods in %g, and T_L is the regional-dependent transition period for longer period ground motion.

2.5. Uplift

Both anchored and unanchored tanks may undergo local uplift when the magnitude of the overturning moment exceeds a critical value. As a result, a strip of the base plate is also lifted from the foundation. Although uplift does not necessarily result in the collapse of the tank, its consequences include serious damage to any piping at the connection to the tank and an increase in the axial stress acting on the tank wall which remains in contact with the ground. The maximum uplift at the base of the tank shell for a self-anchored tank constructed to the criteria for annular plates may be approximated by Eq. (15).

$$y_u = \frac{12.1 F_y L^2}{t_b} \tag{15}$$

where y_u is the estimated uplift displacement for the self-anchored tank, F_y is the minimum specified yield strength of bottom annulus,

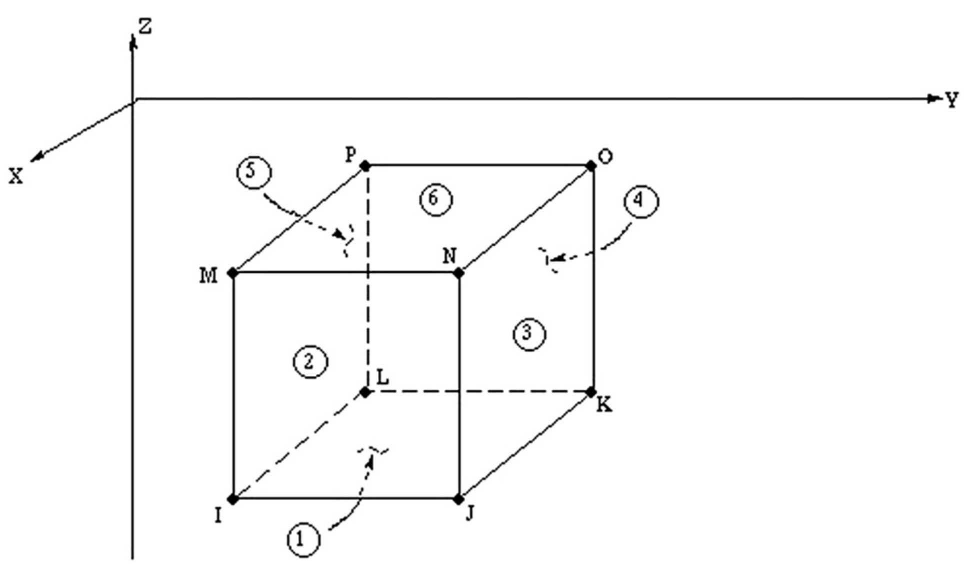

Fig. 2. Fluid-80, 3-D contained fluid element.

L is the required minimum width of thickened bottom annular ring measured from the inside of the shell, and t_b is the thickness of tank bottom less corrosion allowance.

3. FEM modeling

In previous section we discussed the suggested dynamic characteristics from the API650-2008 seismic provision. In Section 3, we present the FEM modeling and verification results.

3.1. Geometric features of studied tanks

In this research, 161 tanks in an oil refinery complex have been classified into 24 groups for detailed seismic analyses. The dimensions and mechanical properties for each type of tank were similar. All required information was obtained from the document center of oil complex. For this purpose, all structural drawings and documents, geotechnical reports, material specifications, and construction details have been studied. Based on the collected data, geometric characteristics of all 24 tank groups such as tank diameter, tank height, liquid height and tank volume are listed in Table 1. These characteristics are key factors in seismic behavior of tanks.

3.2. Analytical approach & methodology

In this part of the research, steel storage tanks were analyzed using nonlinear FEM analysis using ANSYS software (2007). The tank roof system was modeled as shell and beam elements placed in the radial and circular directions. The tank wall was modeled assigning Shell-63 element with bending and membrane behavior. Both in-plane and normal loads were permitted. The Shell-63 element had six degrees of freedom at each node: translations in the nodal x, y, and z directions and rotations about the nodal x, y, and z axes, as shown in Fig. (1). Stress stiffening and large deflection capabilities were included. A consistent tangent stiffness matrix option was available for use in large deflection (finite rotation) analyses (ANSYS website, 2012). Tank contents were modeled by fluid element Fluid-80 with ability to consider fluid-structure interaction and applied acceleration. Fluid-80, described in Fig. (2) is a modification of the 3-D structural Solid-45 element. The fluid element was used to model fluids contained within vessels having no net flow rate. Another fluid element, Fluid-66, was available to model fluids flowing in pipes and channels. The Fluid-80 element was particularly well suited for calculating hydrostatic pressures and fluid/solid interactions. Acceleration effects, such as those occurring in sloshing, as well as temperature effects, may be included. The Fluid-80 element was defined by eight nodes having three degrees of freedom at each node: translation in the nodal x, y, and z directions (ANSYS website, 2012). Static, modal, spectral and nonlinear time-history analyses were done in order to determine maximum responses and failure modes as follows:

Static analyses: The behavior of the selected tanks affected by gravity loads and hydrostatic pressure was determined by static

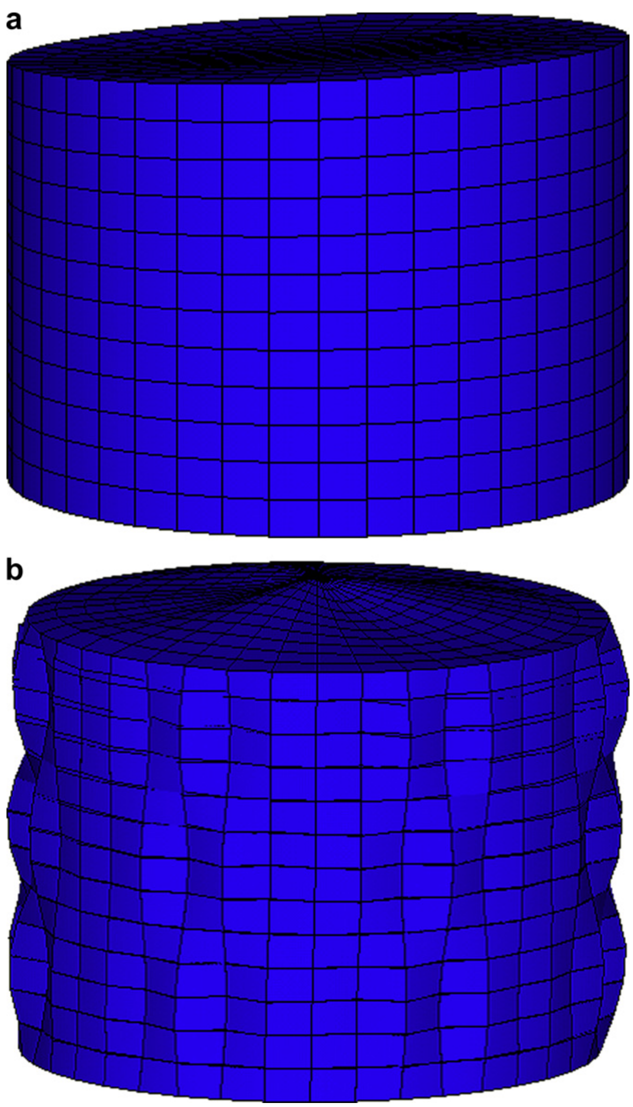

Fig. 3. Storage tank modal shape: a) convective mode, b) impulsive mode.

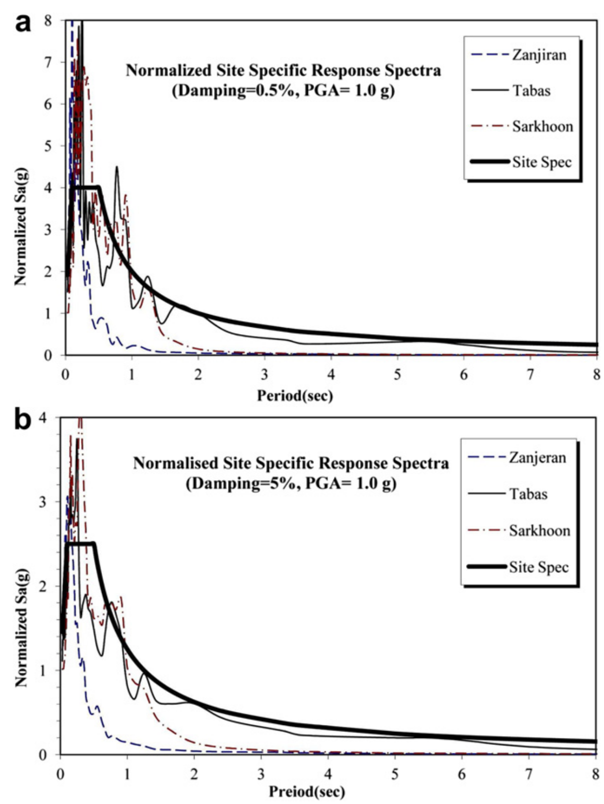

Fig. 4. Normalized site-specific response spectra: a) damping = 0.5%, b) damping = 5%.

Table 2
Characteristics of selected ground motions.

Earthquake	Occurrence	Direction	PGA (g)	PGV (cm/s)	PGD (cm)
Zanjiran-Iran	June 1994	NS	1.06	34	5.5
		UP	0.983	37	4
Tabas-Iran	Sep 1987	NS	0.897	85	38
		UP	0.717	83	9.4
Sarkhoon-Iran	Mar 1975	NS	0.09	5.5	0.88
		UP	0.042	6.1	0.49

analysis. This analysis can be used as a criterion to validate analytical models of static behaviors. Distribution of hydrostatic pressure creates hoop tensile stresses in tank shell and produces static displacements.

Modal analyses: Modal parameters including natural frequency, mode shapes, participation factors, modal coefficients, and mass distribution were obtained from modal analyses. A sample modal analysis as shown in Fig. (3) depicts dominant impulsive and convective vibration modes. This analysis also can be a starting point

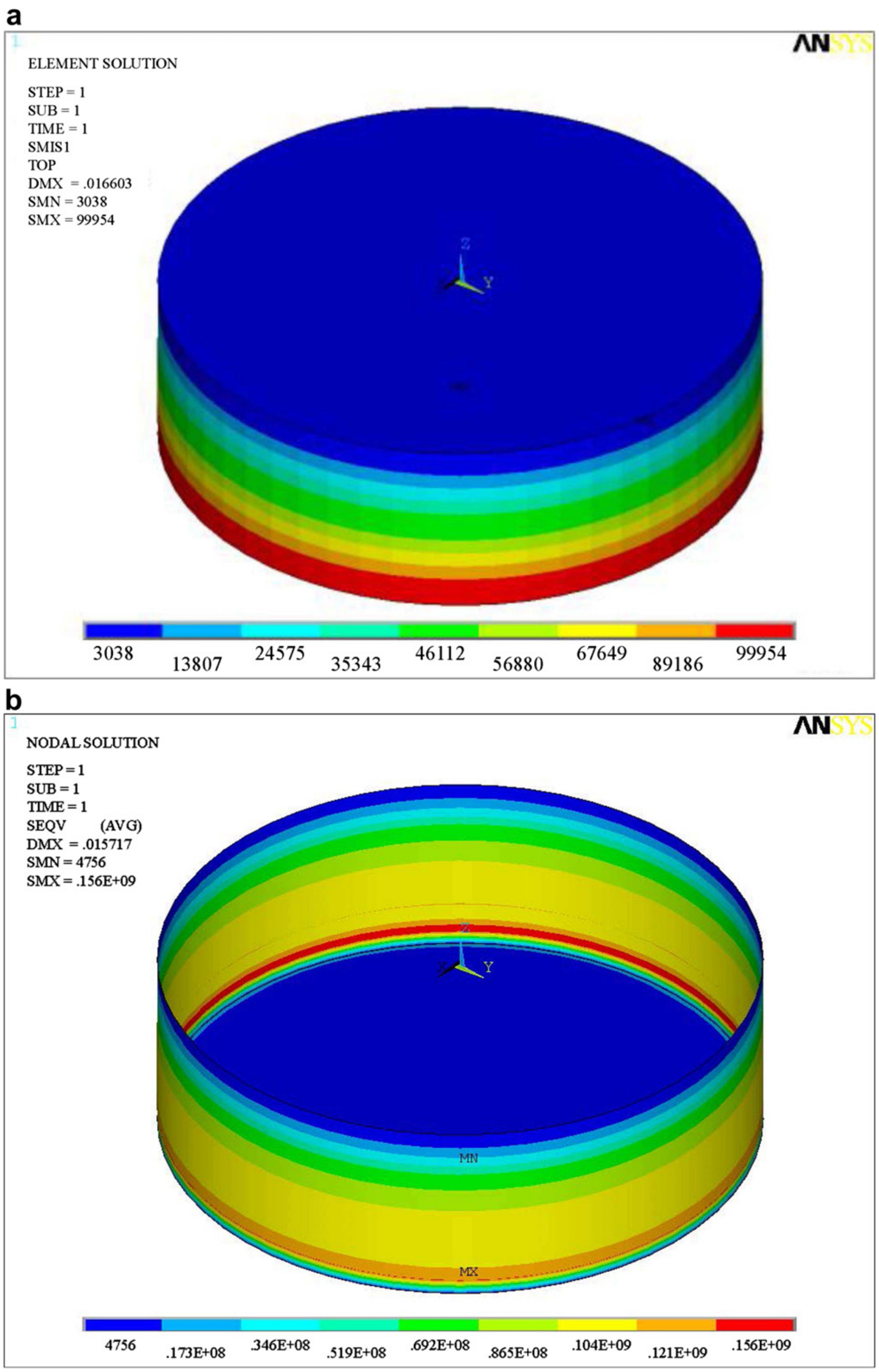

Fig. 5. A) hydrostatic pressure distribution at fluid element (mean value at the center of element), b) Von Mises stresses at the shell of tank due to static loading.

for spectral and time-history analyses. In this study, the reduced method is selected for extracting mode shapes and respective periods due to high-rate convergence. In problems for which the mass and stiffness matrices are endless because of fine mesh and large number of degrees-of-freedom (DOFs), the reduced method was preferred. This method saves time and gives sufficient accuracy.

Spectral analyses: The results of modal analysis and design response spectra were used to perform spectral analysis. The Complete Quadratic Combination (CQC) method was implemented for modal combination of spectral analyses. The number of modes considered in spectral analysis was based on the achievement of 90% seismic structural mass. The response spectra were constructed with 5% damping ratio for impulsive vibration modes and 0.5% damping for convective vibration modes, corresponding to API650-2008. The probabilistic Earthquake Hazard Level [ASCE/SEI7-05] used in this study and its corresponding mean return periods (the average number of years between events of similar severity) was 10% over 50 years. Site-specific acceleration response spectra shown in Fig. (4) have been determined from site compatible ground records.

Time-history analyses: Nonlinear time-history analyses of the studied tanks were conducted using β Newmark method (Newmark, 1959). Since the time-history analysis was time-consuming and, in some cases, even impossible due to the large number of elements, the reduced method was utilized to overcome this problem. A suite of three ground motions were selected and scaled based on the site specific PGA of 0.4 g. Table 2 presents characteristics of selected ground motions including designations, year and respective PGAs, PGVs, and PGDs. In addition, postulated damping in time-history analyses was based on Rayleigh damping assumptions (Liu & Gorman, 1995). A further assumption was incorporated into the model for unanchored tanks, i.e., the Gap element (ANSYS website, 2012) was adopted to model the uplift.

3.3. Verification of modeling

Numerical verification is necessary to show that numerical models are able to predict responses with reasonable accuracy and precision. Therefore, static, linear modal and nonlinear time-history analysis results of the sample tank model have been investigated.

It is vital for the analytical model to have correct behavior in its static condition without any seismic excitation. Therefore, for the first verification, static analysis of a studied sample tank was performed, and the distribution of hydrostatic pressure is displayed in Fig. (5). It can be inferred from Fig. (5-a) that the maximum hydrostatic pressure occurs at the tank bottom level. Similarly, Von Mises stress at the tank shell due to the hydrostatic pressure is presented in Fig. (5-b). As shown in this figure, the maximum stresses occur in tank bottom level where elephant-foot buckling is more likely to happen. Also, Fig. (6) shows differences between theoretical hydrostatic pressures calculated by the $P = \rho g H$ formula and obtained from the FEM analysis of the studied tanks. Good agreement between theoretical and analytical results, with about 6% difference, verifies the accuracy of numerical modeling for static analysis.

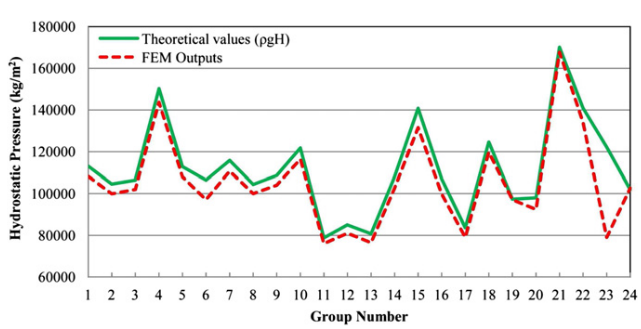

Fig. 6. Comparison of theoretical hydrostatic pressures values and FEM outputs.

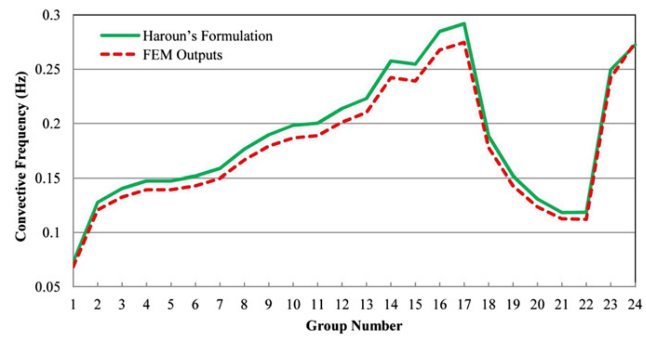

Fig. 7. Comparison of convective vibration frequencies of Haroun's formulation and FEM outputs.

The second verification of analytical results was related to modal analysis. To verify dynamic properties of the model, linear modal analysis of the studied tanks was performed, and dominant frequency of convective vibration modes was obtained. Also, the exact formulation of Haroun, M.A. (1983) was used to calculate the dominant frequency of the convective vibration mode using Eq. (16). The predicted and exact frequencies are compared in Fig. (7). It is obvious from Fig. 7 that the frequencies of convective vibration modes of the exact solution and numerical modeling are approximately the same; the maximum difference is 6.63%, except for group name PA which has 17.04% difference between exact and predicted frequencies.

$$\omega_s^2 = \frac{3.68g}{D}\tanh\left(\frac{3.68H}{D}\right) \tag{16}$$

The final verification of analytical results was for sloshing time-history of the liquid. For this purpose, some nodes have been defined in order to show time-history analysis results. According to Fig. (8), symmetric nodes of 1–2 and 3–4 were considered at the top level of the liquid. The sloshing displacement of these couple nodes subjected to earthquake ground motion must be unsymmetrical with the same absolute values. Fig. (9) presents sloshing displacements of specified nodes of sample tank subjected to Tabas ground motion. As shown in this figure, the unsymmetrical response of mentioned double nodes indicates an appropriate compatibility and acceptable accuracy of numerical model and time-history analyses results.

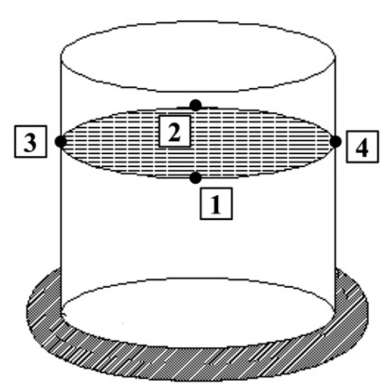

Fig. 8. Number of key nodes for time-history analysis results.

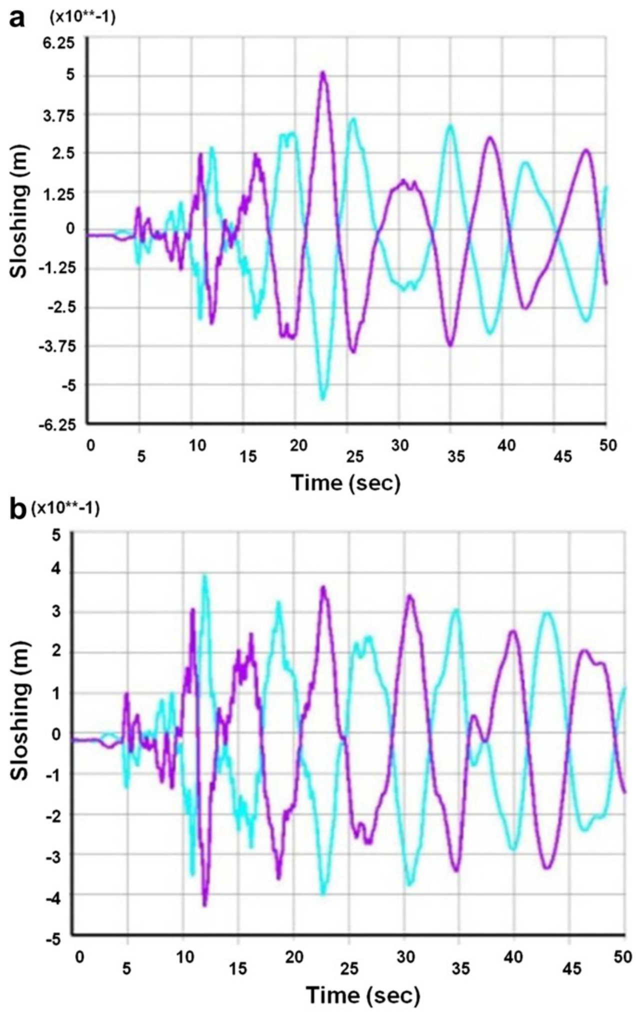

Fig. 9. Sloshing displacement time-history of tank subjected to Tabas ground motion for: a) Nodes 1 and 2, b) Nodes 3 and 4.

4. Results and discussion

The previous section discussed geometry features, methodology and verification of FEM modeling and in Section 4 we discuss the results. In this research, all of the 24 tank groups were analyzed and investigated using API650-2008 requirements and FEM analyses results. Dominant frequency content and important failure modes such as slide bottom (base shear), elephant-foot buckling, sloshing (free board) and uplift have been determined and compared in the following sections:

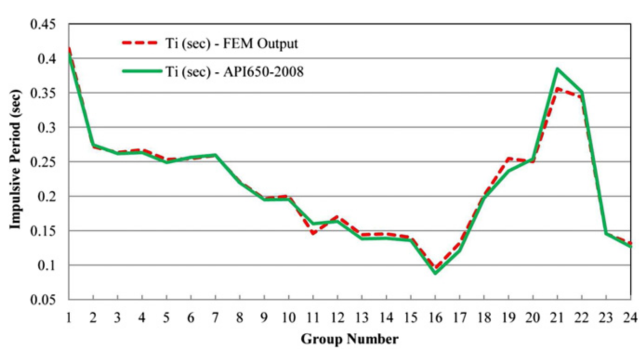

Fig. 10. Comparison of impulsive period of vibration (T_i) obtained from API650-2008 and FEM analyses.

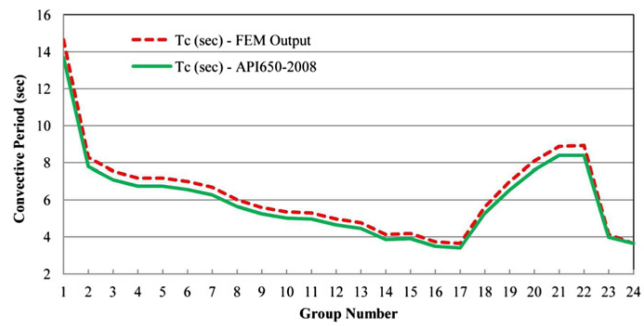

Fig. 11. Comparison of convective period of vibration (T_c) obtained from API650-2008 and FEM analyses.

4.1. Dominant periods of vibration

Both structural periods of vibration (Impulsive and Convective periods) were calculated using FEM analyses and the code formulations. Figs. (10) and (11) compare obtained values of dominant impulsive and convective periods for each tank group, respectively. These figures indicate that estimated code values for structural periods of vibration are reasonable. Although the results are accurate, one modification to Eq. (1) is necessary. Values obtained from this equation for impulsive period (T_i) are nearly 30 times greater than the analytical values. Therefore, the constant coefficient ($1/\sqrt{2000}$) in this equation should be changed to ($1/\sqrt{2{,}000{,}000}$).

4.2. Seismic base shear (slide bottom)

Generally, seismic base shear depends on the frequency content and the total weight of the storage tank including both liquid and structure. Therefore, variation of base shear coefficient (V/W) to total weight (W) obtained from Eqs. (4)–(6) and input ground motion from Fig. (4) are depicted in Fig. (12). The general trend of this figure indicates that, by increasing the total weight of the tank, the base shear coefficient (V/W) decreases. This coefficient for small tanks (less than 100,000 kN) is more than 0.15. However, for large tanks (more than 700,000 kN), this coefficient is less than 0.06. The trend is an important issue in seismic assessment of studied tanks.

4.3. Elephant-foot buckling

This criterion is usually a concern for large diameter tanks. In this study, the variation of elephant-foot buckling stress to yield stress ratio (σ_e/σ_y) as a function of tank diameter (D) is investigated. Fig. (13) presents the variation in the yield stress ratio obtained

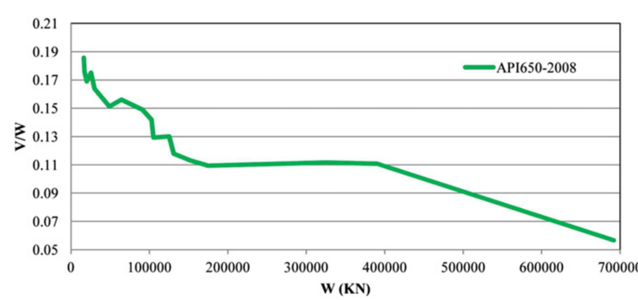

Fig. 12. Variation of normalized base shear versus tank total weight.

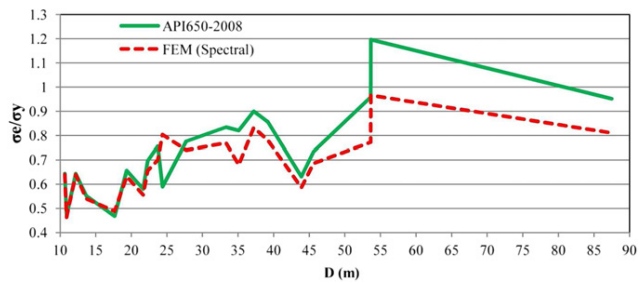

Fig. 13. Variation of normalized elephant-foot buckling stress versus tank diameter.

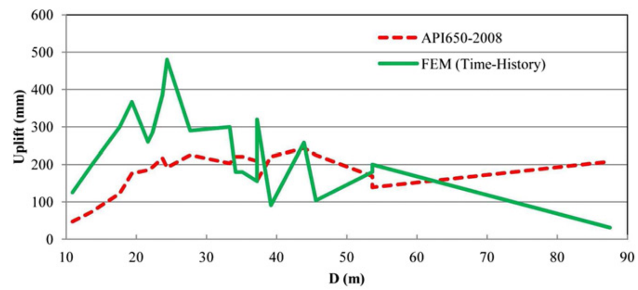

Fig. 15. Variation of uplift versus tank diameter (for unanchored tanks).

from code requirements and from FEM analyses results. General trend of this Fig. (13) shows increasing stress ratio with increasing tank diameters, up to 55 m, after which the stress ratio decreases. Also, according to this figure, API650-2008 and FEM analyses results are similar for small diameter tanks (less than 25 m); in contrary, the code predicts higher values than FEM analyses for larger diameter tanks. This observation indicates that the elephant-foot buckling stresses obtained from Eq. (11) are overestimated compared to FEM analyses results for large diameter tanks (more than 55 m).

4.4. Sloshing

Tank diameter (D) and the fundamental period of convective mode (T_c) are dominant factors for evaluation of fluid sloshing. The variation of liquid sloshing versus tank diameter (D) obtained from code requirements and FEM analyses is presented in Fig. (14). Two different trends are indicated in this figure for sloshing. Based on the code requirement, the sloshing amplitude reduces linearly with increasing of tank diameter, while, the maximum sloshing amplitude obtained from FEM analyses occurs for tank diameters between 20 m and 35 m. This observation suggests that the sloshing amplitude obtained from code requirements is not sensitive to the frequency content of input ground motion in low frequency ranges. Generally, sloshing amplitudes obtained from Eq. (12) are greater than FEM analyses results. However, the sloshing amplitudes obtained from code requirements are three times greater than FEM analyses results for small diameter tanks (less than 15 m). The higher sloshing amplitudes required by the code, affects free board which is an important economic issue. In the case of large diameter tanks (about 80 m), the code and analyses results were in agreement.

4.5. Uplift

Usually a tank's movement during strong earthquakes does not lead to total destruction but it may cause tearing in the shell at the connections with external piping. Therefore, all steel tanks should be designed for uplift, overturning or sliding. The variation in uplift versus tank diameter determined from code requirements and FEM analyses (nonlinear time-history analysis) for the studied tanks is presented in Fig. (15). Uplift values based on the code requirements are approximately constant for tanks with diameters greater than 20 m. The variation in uplift obtained from FEM analyses is an indicator of tank dynamic characteristic and input motion (see Fig. 12) which is not appeared in code requirement (see Eq. (15)). One important issue regarding large diameter tanks (more than 80 m) is that code-specified uplift is about three times higher than FEM analyses results. However, FEM analytical results predict more realistic values for large diameter tanks because of relatively small seismic base shear values.

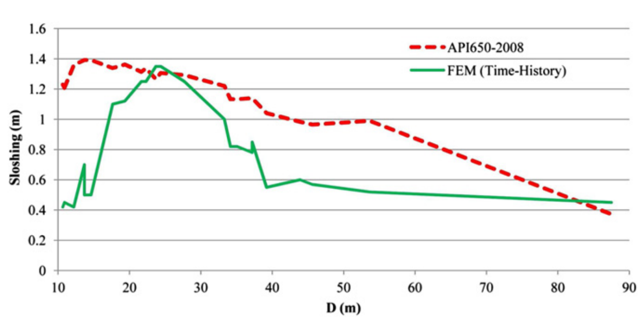

Fig. 14. Variation of sloshing versus tank diameter.

5. Conclusion

In this paper, a comprehensive study was conducted to highlight the shortcomings of the API650-2008 code. For this purpose 161 existing tanks in an oil complex were classified into 24 groups for seismic assessments. The numerical finite element models of tanks were constructed using ANSYS software (2007). Modal periods, base shear, elephant-foot buckling, sloshing and uplift predicted using the code and from analytical approaches were compared. Concluding remarks obtained from this research can be summarized as follows:

- Consequently, impulsive and convective modal periods obtained from API650-2008 code requirements and FEM analyses result are very similar.
- The overall trend of API650-2008 code-estimated base shear indicates that seismic base shear coefficient decreases with increasing total tank weight.
- Comparison of elephant-foot buckling stresses obtained using API650-2008 and FEM analyses indicates that these stresses are similar for small diameter tanks; conversely, the code predicts higher stresses in larger diameter tanks.
- Sloshing amplitude based on the API650-2008 code requirements, reduces almost linearly with increasing tank diameter. This observation suggests that the code-estimated sloshing is not sensitive to low frequency content of input ground motions. In addition, code-estimated sloshing is greater than FEM analyses predictions for small diameter tanks. However, in the case of large diameter tanks, code and analyses results are almost similar.
- The variation in uplift obtained from FEM analyses is an indicator of tank dynamic characteristic and input motion which is not appeared in API650-2008 code requirement. An important issue regarding large diameter tanks is that code-specified uplift is higher than the FEM analyses results. Small seismic base shear values in large diameter tanks indicate that FEM analytical results are more real and more reasonable than code evaluations.

Although the API650-2008 code regulation agrees with numerical models in some cases, this investigation highlights the major shortcomings of the code requirements for key parameters in the design of liquid storage tanks. Therefore, more efforts including various experimental studies and different field observations are required in this field to calibrate the API650-2008 regulations.

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