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Analytical and numerical analysis of anchored flexible steel liquid storage tanks subjected to seismic loads considering soil structure interaction

M.S. Pourbehi
Cape Peninsula University of Technology, Cape Town, South Africa

JAvB. Strasheim
Stellenbosch University, Stellenbosch, South Africa

M. Georgiadis
VPE Engineering Services, Durban, South Africa

ABSTRACT: The impact of climate change on the natural and built environment is posing a significant threat to potable water supply infrastructure systems. This can result in an increase in the demand for sustainable water storage solutions to avoid service interruption to industries and municipal utilities dependent on water supply. One solution to mitigate the water supply and storage risk is to utilize modular steel water storage tanks designed and manufactured with Zincalume© steel panels. These tanks have a novel design and do not have bottom plates. They are fitted with a synthetic liner, are stiffened with vertical wind girts and are anchored to a concrete ring beam at the bottom of each wind girt using holding down bolts. Due to the high radius to wall thickness ratio (R/t) of these tanks, the structural performance, interaction with the stored fluid and the foundation soil under dynamic loads, have not been well understood. In this research, the effect of seismic loads on the global structural behavior of these tanks such as base shear, overturning moment, circumferential stress as well as the buckling behavior of the thin metal panels was studied using both analytical techniques and numerical Finite Element Method (FEM) analyses. Parametric studies using spring-mounted mass analogy techniques specified by the American Petroleum Institute code (API650) and the New Zealand Society for Earthquake Engineering recommendation (NZSEE) are conducted for three tanks with different fluid height to tank radius ratios (H_w/R). The results are compared with nonlinear time history dynamic analyses performed using the Abaqus 3D finite element software. The results indicate that, due to the flexible nature of the tanks, the method specified by API650, which does not take the flexibility of the tank and soil structure interaction into account, underestimates the global design indicators such as base shear, overturning moment and hoop stress for tanks with higher H_w/R ratios. The method specified in the NZSEE recommendation predicted the global design indicators as well as local buckling phenomena in the panels in good agreement with the FEM results.

1 INTRODUCTION

The impact of climate change on the natural and built environment is posing a significant threat to potable water supply infrastructure systems. This might lead to an increase in the demand for sustainable water storage solutions to avoid service interruption to industries and municipal utilities dependent on reliable sources of water supply. Steel storage tanks are considered a significant part of the industry and are widely used for water supply, industrial chemical storage, and nuclear power plant systems. Due to their importance, the storage tanks should, where required, be designed to withstand severe earthquakes. The seismic response of steel storage tanks which store fluids differs from that of typical buildings due to the hydrodynamic fluid-structure interaction of the tank and the stored fluid. The hydrodynamic pressure acts on the tank wall, roof, and foundation causing fluid sloshing motion and dynamic hoop stresses (Hernandez et al., 2021). This action can cause structural damage to tanks during earthquakes such as elephant's foot buckling, diamond shape buckling, damage to the tank roof and failure of the anchor bolts.

The dynamic behavior of steel storage tanks including fluid structure interaction has been studied by many researchers. The work of George Housner (Housner, 1955) is fundamental to the development of the quantification of the impulsive and convective components of the hydrodynamic pressure on the fluid container. This original paper presented an analysis of the hydrodynamic pressures developed when a fluid container is subjected to horizontal accelerations. Other follow on papers by Housner (Haroun

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& Housner, 1981; Housner, 1963) as well as a paper by Howard Epstein (Howard, 1976) also deal with the analytical solutions of this problem.

The research on seismic response of steel storage tanks has profoundly improved the regulations and standards developments since the pioneering research done by Housner. The standards developed by the American Water Works Association (AWWA) in association with the American National Standards Institution (ANSI) (American Water Work Association, 2009) and a document on Welded Steel Tanks for Water Storage - 'AWWA D100′ (American Water Work Association, 2006) use the rigid tank concept developed by Housner (Housner, 1955). In 2009 the New Zealand Society for Earthquake Engineering (NZSEE, 2009) published a revised version of their manual on the seismic design of storage tanks. It contains the recommendations of a NZSEE Study Group on this topic. This design manual contains information and a commentary on background to the recommendations, general design principles of both rigid and flexible concrete and steel storage tanks, discussion on the design loading and structural analysis, design criteria, foundation considerations as well as a set of design examples for steel and prestressed concrete tanks and reservoirs.

Miguel Ormeño et al (Ormeño et al., 2012) conducted a comparative study of the loads obtained using the API 650 (API, 2012) and the NZSEE design code guideline documents for a range of flexible tanks. According to this research the difference in stiffness between the rigid tank, rigid foundation and the flexible soil, has an important effect on the seismic response when considering soil-foundation-structure interaction (SFSI). This often causes an elongation of the period of the impulsive mode of vibration. A further issue not well understood, for unanchored tanks, is the uplift of the tank base that usually occurs under more intense dynamic loading. The seismic response computed using these codes is presented for a range of tanks. The analysis results show that both guideline standards provide similar base shear and overturning moment values. The results for the anchorage requirement and uplift are not in good agreement. Hosseinzadeh et al. (Hosseinzadeh et al., 2013) describes a comparison of the API650-2008 code provisions with numerical FEM analyses for the seismic assessment of existing steel oil storage tanks. They concluded that the API650 code cannot accurately predict the key variables of some cases of steel storage tanks and further investigation needs to be carried out.

Estekanchi and Alembagheri (Estekanchi & Alembagheri, 2012) proposed a time history analysis procedure using the endurance time method (ET) technique to predict the seismic response of the steel storage tanks. The ET method uses an artificial gradually intensifying dynamic excitation as the loading function. The variations of relevant response parameters and damage indexes are monitored as the intensity is increased. The structural performance is assessed based on its response at various dynamic excitation levels.

Buckling plays a fundamental role in the design of steel tanks because of the thin walled nature of this class of structures. In a paper by Buratti et al. (Burati & Tavano, 2014) the dynamic buckling and seismic fragility of anchored steel storage tanks is dealt with. In this case the added mass method is used. The research considers the different buckling modes and especially the secondary buckling occurring in the top part of the tank for circular cylindrical steel tanks that are fully anchored at the base. A case study based on a cylindrical tank is also discussed. The peak ground displacement was found to be the most efficient and sufficient intensity measure so far as the maximum relative displacement of the tank walls is concerned.

In this paper a set of flexible profiled plate metal storage tanks which do not have bottom plates but a concrete ring beam support and anchoring as well as a polymer bladder lining to provide a leak proof storage vessel is studied under earthquake loading. The flexible steel storage tanks with three different water level to tank radius (H_w/R) aspect ratio subjected to seismic loading are investigated using the analytical methods which are adapted from the flexible tank design concept developed by NZSEE (NZSEE, 2009) and the rigid tank design method suggested by the API650 standard. For the same set of tanks, the numerical models using Abaqus FE software are developed and the results are compared with the findings of the analytical methods. The global and local (buckling) behavior of a typical tank are also studied.

2 ANALYTICAL CODE BASED STRUCTURAL MODELS AND ANALYSIS METHODS

The mass-spring tank structural models on which the API 650 and NZSEE approaches are based are shown in Figure 1. The API method models the convective mass m_c and the rigid impulsive mass m_i. The NZSEE method models the convective mass m_c and the flexible impulsive mass m_f as well as the rigid body mass m_r. Based on this code, a tank with H_w/R (aspect ratio) larger than 1 can be defined as a flexible tank. The higher aspect ratio leads to the higher contribution of the flexible impulsive mass m_f in the analytical modelling of the tank. The deformation of the soil foundation is included in the calculation of the impulsive mode periods of vibration for both the horizontal and vertical directions. For flexible tanks this effect is included in the estimation of the periods associated with both the impulsive mode masses i.e. the rigid body mode mass m_r and the impulsive mode mass m_f. The mass of the shell, base and any support system or structural foundation is included with the impulsive masses, when calculating the periods or frequencies of vibration. The effect of soil structure foundation interaction and tank wall flexibility is neglected when calculating the periods of vibration of the convective modes of vibration of the tank.

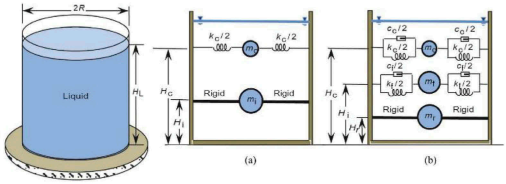

Figure 1. Spring-Mass model for horizontal earthquake, a) rigid tank model based on API 650, b) flexible tank model according to NZSEE (NZSEE, 2009).

As far as local elastic and/or plastic shell buckling is concerned the NZSEE document provides a figure which compares elastic diamond buckling and elastic-plastic (elephant's foot) buckling collapse stresses for steel with fy=250 MPa and a=1.0 (NZSEE, 2009). It shows the critical flexural compression stress induced in the wall of a pressurized cylindrical tank and it can be seen that for a membrane circumferential stress parameter, pR/t (p - the pressure at the bottom of the tank) less than 100 MPa for a tank with aspect ratio R/t larger than 2000, elastic diamond buckling (elastic plastic collapse) governs as a failure mode. When the internal pressure parameter is more than 100 MPa, the stress in the wall is dominated by elephant's foot buckling (Teng & Rotter, 2004). Equation (1) was developed by Rotter to compute the elastic-plastic buckling capacity f_{pb} as follows:

$$f_{pb} = \sigma_{c1} \cdot \left[1 - \left(\frac{p.R}{t_w.f_y}\right)^2\right] \cdot \left(1 - \frac{1}{1.12 + r^{1.5}}\right) \cdot \left(\frac{r + f_y/250}{r+1}\right) \quad (1)$$

In Equation (1) $\sigma_{c1}=605(E_w.t_w)/R$ is the Euler's critical axial compressive stress, R is the tank radius, p is the total internal pressure, E_w and t_w are the elastic modulus and the thickness of the tank walls, f_y is the steel yielding stress, and r is a coefficient defined as $r=R/\ (400.t)$. For this study MATLAB code implementing the NZSEE method and spread sheets implementing the API650 code, considering all of the key variables and parameters were developed and the results of the analytical tanks for selected tanks are interpreted and reported in the following sections.

3 FEM MODELLING OF TANKS WITH THREE H_W/R RATIOS

In this section the finite element models of the steel storage tanks developed are described. The layout of the tanks and the material properties for steel, concrete, water and soil are presented.

3.1 *Tank layout, geometry and material parameters*

Three steel storage tanks with aspect ratios H_w/R of 0.87, 1.07 and 1.77 were considered for this research. The tanks geometrical parameters are supplied in Table 1. Only one soil type is considered in the analytical code-based method.

The material properties for the steel, concrete and water used in both analytical and numerical models are presented in Table 2. The results of the analytical methods according to the NZSEE and API650 and sensitivity analyses are presented in the following sections.

Table 1. Typical steel tank dimensions and aspect ratios included in this study.

Tank	Diameter	H_Tank	H_w/R	R/t_w
	(m)	(m)		
T1	21.16	9.39	0.8790	4167
T2	7.51	4.2	1.0786	5000

Table 2. Material parameters for the steel, concrete, soil and water.

Material	Mass density	Young Modulus	Shear wave velocity	Bulk Modulus
	(kg/m^3)	(GPa)	(m/s)	(GPa)
Steel	7850	200	-	-
Concrete	2500	30	-	-
Soil	1870	-	250	-
Water	1000	-	-	2.2

3.2 *Finite element models of tanks*

The tank geometry and finite element mesh for both the tank panels and the contained water are shown in Figure 2. The mesh size for the panel shell elements is 0.8 × 0.8 m. The Abaqus S4R shell element type is used in this model. The acoustic element type AC8DR with 8 nodes and one degree of freedom of pressure is utilized in modelling the water behavior due to the seismic excitation. The size of acoustic elements in this model is 0.5 × 0.5 m. To check the dependency of the analysis on the mesh size, a sensitivity analysis was carried out with a very fine mesh and the results obtained show that the results using the selected mesh size differs only by 5 % from that of the model with the fine mesh size. In this model, the water is considered as an incompressible, inviscid and irrotational fluid. The Helmholtz equation for a hydrodynamic pressure domain is used with the speed of the sound set at 1450 m/s in the water domain. The density of the water used is 1000 kg/m^3. The nonlinearity of the steel material is considered by defining a plastic stress-strain material relationship in the Abaqus FE software.

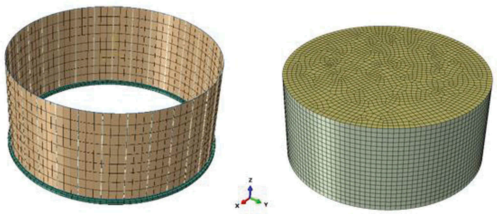

Figure 2. Shell and water elements and concrete ring beam.

3.3 *Dynamic seismic loading and analysis procedure*

The Endurance Time (ET) technique, developed by Estekanchi (Estekanchi & Alembagheri, 2012), is used to overcome the difficulties with regard to non-linear time history dynamic analysis in typical engineering problems, such as water storage tanks. These engineering fluid structure interaction problems require several dynamic analyses using a variety of seismic signals, which covers most of the range of frequencies in the system. This is computationally intensive and time consuming. Figure 3 shows the first 3 seconds of the Endurance Time (ET) signal adopted from Estekanchi et al (Estekanchi & Alembagheri, 2012) . To be compatible with the peak ground acceleration (PGA) of 0.2 g used in the analytical analysis, only the part of the signal up to 3 s is used.

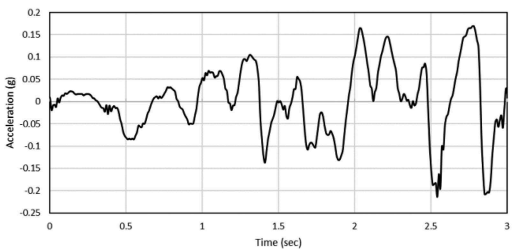

Figure 3. Endurance Time signal with duration of 3 seconds adopted from ETA20e03 (Estekanchi & Alembagheri, 2012).

4 ANALYSIS RESULTS AND DISCUSSION

In the following paragraphs, the results of the analytical and FEM analysis for the selected tanks are presented. The effect of the seismic loading on three different tanks ranging from $H_w/R=0.87$, $H_w/R=1.0786$ and $H_w/R=1.7713$ are summarized. The results obtained by using both NZSEE and API650 standards are compared with the FEM analyses.

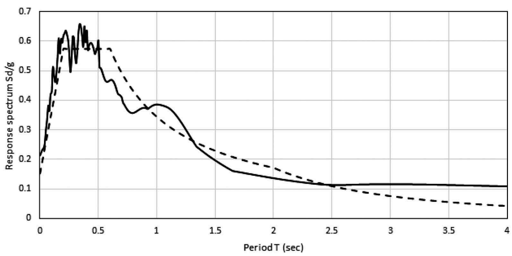

Figure 4. Comparison between the design elastic response spectrum for ground type 3 suggested by SANS10160-4 (dashed line) and the response spectrum calculated using SeismoSignal software for ET signal (solid line).

4.1 *Comparison of the results of the analytical methods versus the FEM approach*

Table 3 presents the comparison of hydrodynamic pressure, base shear, overturning moment, hoop stress and tensile force in the anchors for the T1, T2 and T3 tanks based on the results calculated in section 2 and 3 for the analytical and FEM methods respectively.

4.2 *Discussion of the FEM modelling procedure and results*

The non-linear geometrical behavior of structure is considered in the dynamic FEM analysis. The analysis steps include a static analysis for gravity and hydrostatic loading which provides a pre-stress condition before the dynamic analysis of 3.0 seconds commences. The earthquake is applied in both the horizontal and vertical directions. A coefficient of 0.7 is applied to the ET signal in vertical direction to be compatible with the analytical method. The horizontal seismic load is defined in global x-direction. The hydrodynamic pressure (POR) is due to the seismic loads in excess to the hydrostatic pressure. The effect of sloshing wave at the surface of water is modelled by defining impedance properties using the interaction module in Abaqus FE software at the surface of the water. In order to simulate the fluid-tank interaction without considering the convective mode, one can apply a pressure boundary condition equal to zero at the top surface of the water.

Figure 5 illustrates the pressure distribution in the water elements. It is mentioned that this variable (POR) shows the hydrodynamic pressure in the water domain. In this case of a full tank condition, this maximum hydrodynamic pressure is about 39 kPa. To compare this value with the result from the analytical method, one can use the SRSS method to combine the pressure due to the impulsive, convective and vertical modes of vibration. This leads to a resultant pressure equal to 37 kPa for the analytical method.

Table 3. Comparison between, NZSEE, API650 and FEM for different flexible tanks.

	NZSEE			API650			FEM		
Variable	T1	T2	T3	T1	T2	T3	T1	T2	T3
Hydro-dynamic pressure (kPa)	37	15.8	12.5	21.6	9.1	5.9	39	18.5	13.2
Base shear (MN)	4.96	0.34	0.08	4.1	0.27	0.053	4.56	0.356	0.11
Over-turning moment (MN.m)	19.1	0.606	0.117	15.46	0.48	0.069	19	0.67	0.15
Maximum hoop stress (MPa)	345	307	106	259	244.3	80.7	300	318	116
Tension force in anchors (kN)	111	28.4	18	94	23.2	16.1	103	32.2	19.5

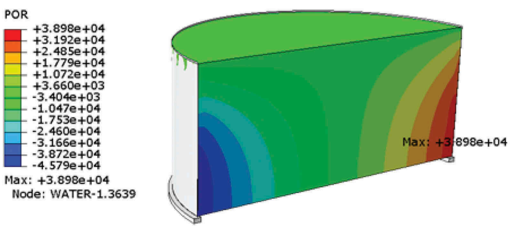

Figure 5. Typical hydrodynamic pressure distribution in the water at the time of maximum hoop stress in Pascal.

The maximum hoop stress at the bottom for the full tank condition is shown in Figure 6. It can be observed that the stress is equal to 300 MPa which leads to the yielding of the panel at the bottom of the tank. This finding is in good agreement with the analytical results.

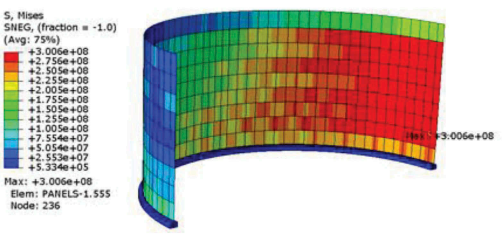

Figure 6. Time history values of typical hoop stress values at the bottom of the shell elements t=2.9 sec in Pascal.

In Figure 7 the red and blue colored vectors represent the base shear and overturning moment respectively. These values are also in a good agreement with the base shear and moments obtained using analytical calculations. See Table 3 for more detail.

Figure 8 shows the vertical force in the anchors at the bottom of the tank. The result presented shows the vertical force in the anchors per meter length of the tank. To compare the forces in the anchors obtained using the analytical methodology with the FEM results presented in this figure, one must multiply the values calculated by analytical method by 2.14 m (the distance between the wind girts of the tank). The maximum force obtained with the FEM model equals 103 kN which compares well with the results of the analytical method i.e. 110 kN.

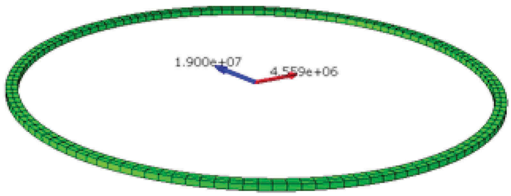

Figure 7. Typical base shear and moment at the bottom of the tank.

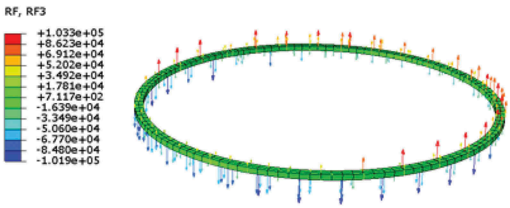

Figure 8. Typical vertical force in the anchors at t=2.8 seconds.

Due to the higher H_w/R for the T3 tank, the FEM results are slightly higher than the analytical values. This is due to the flexibility of the tank. The NZSEE method is based on a flexible tank model, it however uses the impulsive formulae to calculate the impulsive pressure for the tank. The NZSEE approach, using the impulsive pressure formulae for tanks with H_w/R less than 1.5, yields good results, but this approach underestimates the pressure for tank T3 with H_w/R=1.7713. The FEM results for the other tanks such as base shear, overturning moment, hoop stress and forces in the anchors are summarized in Table 3.

4.3 *Buckling stress analysis of a T1 tank*

In order to study the buckling phenomena in flexible storage tanks, a squat steel tank is selected to display typical results for a parametric study. Three water level conditions (H_w/R) namely, full tank, half-full and three-quarter full tank are considered. For this tank for the full case the compressive stress in the tank wall at the bottom of the tank is more than the plastic limit failure stress. Figure 9 illustrates the buckling stress, elastic (diamond) and plastic (elephant's foot) buckling stress values for the three H_w/R parameter values. The elastic stress limit does not apply here seeing that the internal pressure parameter pr/t > 100 MPa. The failure of the tank is governed by the plastic and not the elastic limit. Figure 10 shows the history of the horizontal displacement at the bottom of the tank with H_w/R=0.87 as predicted using Abaqus FE modelling. This result indicates that the tank experiences failure due to buckling when the seismic loading is higher than a PGA of 0.2g when about 3 seconds of the analysis time has elapsed. This finding confirms the results of the NZSEE based analytical method shown in Figure 9. The parametric analysis does not show any buckling failure for half-full and three-quarter full tanks respectively.

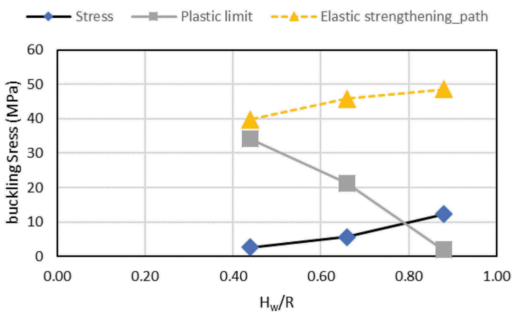

Figure 9. Compression stress, elastic (diamond) buckling and plastic (elephant's foot) buckling for three H_w/R ratios based on NZSEE analytical method for squat tank case.

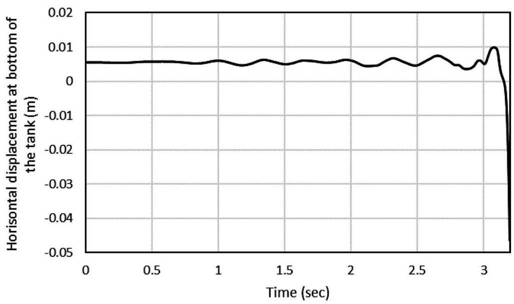

Figure 10. Horizontal displacement at the bottom of the tank with H_w/R=0.87 from time history analysis using Abaqus - buckling behavior is observed at 3 seconds.

5 CONCLUSION

Parametric studies using spring-mounted mass analogy techniques specified by the American Petroleum Institute code (API650) and the New Zealand Society for Earthquake Engineering recommendation (NZSEE) using South African seismic standards were conducted for three tanks with different fluid height to tank radius ratios (H_w/R). The results were compared with nonlinear time history dynamic analyses performed using Abaqus 3D finite element software.

The results indicate that, due to the flexible nature of the tanks, the method specified by API650, which does not take the flexibility of the tank and soil structure interaction into account, underestimates the global design indicators such as base shear, overturning moment and hoop stress for tanks with higher H_w/R aspect ratios. The global design force, moment and stress indicators as well as local buckling phenomena in the tank wall panels obtained by the method specified in the NZSEE recommendation (using SANS seismic parameters) are in good agreement with the FEM results obtained. The effect of the flexibility of a tank is more evident for the tanks with higher aspect ratios. For high tank aspect ratios, the flexible impulsive mass of the tank plays an important role in the total mass of the system.

In future research the effect of the wind girts on the panel buckling results obtained by the analytical methods can be investigated and used to predict the actual buckling behavior of a steel tank stiffened with the vertical wind girts as was done in the FEM models. In addition, the failure mechanisms of the wind girt anchor bolts should be investigated using numerical modelling.

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