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INVESTIGATING THE EFFECT OF THE CIRCUMFERENTIAL STIFFENERS ON THE DYNAMIC BUCKLING OF STEEL STORAGE TANKS (PGA)

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ABSTRACT

The use of ring stiffeners reinforcements in the design of steel tanks may improve their resistance in the absence of seismic excitation. However, in seismic zones is not always beneficial and could cause undesirable effects. The API650 is one of the prominent codes consisting of seismic specifications to design steel storage tanks for earthquake resistance. The API650 urges the strengthening of the tank shells with circumferential ring stiffeners to increase the resistance of tank under an external pressure loading. In this work, the conditions of increasing or decreasing of resistance of cylindrical steel tanks to buckling under hydrodynamic impulsive pressure are analyzed by taking into account the presence of these ring stiffeners. It is found that the influence of rings on the capacity of resistance against buckling are significant under seismic excitation of San Fernando.

Keywords: *Dynamic Buckling; Ring stiffeners;* San Fernando*; Tanks; API 650.*

1. INTRODUCTION

The tanks are strategically a very important structure as they have vital uses in industries, nuclear power plants and also other activities connected to public life. The earthquake is the phenomenon that generates the most damage to structures a million of all earthquakes occur each year in the world. Therefore, many studies have been undertaken to understand the seismic behavior of storage tanks under earthquake loading. Cooper and Wachholz (Cooper et al. 1999) reported damage to petroleum steel tanks due to earthquakes of Long Beach1933, Kern County, 1952, Alaska 1964 San Fernando, 1971, Imperial Valley 1979, 1983 Coalinga, Loma Prieta 1989 Landers 1992, 1994 Northridge, et Kobe 1995. A Clear damage to petroleum steel tanks have been compiled by Jain and others (Jain et al. 2001) and Suzuki (Suzuki et al. 2002) during the recent earthquakes in India and Turkey. These reports on the structural behavior of tanks during earthquakes indicate that steel tanks are more sensitive than those reinforced concrete (RC) to damage, the damages caused to these structures made them generally out of service. The failures of these structures are manifested by shell buckling mode, roof damage, anchorage failure, tank support system failure, foundation failure, hydrodynamic pressure failure, etc. Amongst these negative phenomena, dynamic buckling of tank walls remains the most common and the most dangerous one; according to the fragility report established by the American Lifelines Alliance (American Lifelines 2001).

The design of these structures is usually subject to the application of international codes. Although these codes are constantly updated especially in the part governing the behavior of steel tanks under seismic loads; they have not yet examined the effect of the wall stiffeners on the behavior of these structures after seismic excitation.

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The main objective of this study is an investigation of the effect of the wall stiffening on buckling resistance capacity of cone roof tank with H/D= 2 geometrical parameters. According to the API650-2008 standard (API Standard 650 1988), circumferential stiffeners are used to strengthen the cylindrical shell to increase the buckling resistance under external pressure loading. A circumferential ring stiffeners plated section was adopted in this work which are positioned and dimensioned according to the formulas proposed in the code API650 (API Standard 650 1988).

2. PRESENTATION OF THE NUMERICAL MODEL

We are adopted a one-dimensional model based on the finite element method, (see Figure 1), with height to diameter ratios (H/D) of 2.

2.1. Walls, stiffeners and roof tank

The experience of past earthquakes has shown that tanks that are completely filled with liquid are more prone to suffer damage (American Lifelines 2001). Thus this study considers a liquid level of 90% of the height of the tank. The varying thicknesses of the tank were designed for this study using the provisions of API650 (API Standard 650 1988), and with conical roofs.

The geometrical parameters of the tanks considered are: D=15 m; spacing stiffeners was taken according to the API650-2008 standard; thicknesses; h_1 = 0.0195 m; h_2 = 0.0195 m; h_3 = 0.0158 m; h_4 = 0.0143 m; h_5 = 0.0126 m; h_6 = 0.0111 m; h_7 = 0.0095 m; h_8 = 0.0078 m. E = 2.1E-11 Pa; ν=0.3. ρ_{acier}= 7840 kg/m^3.

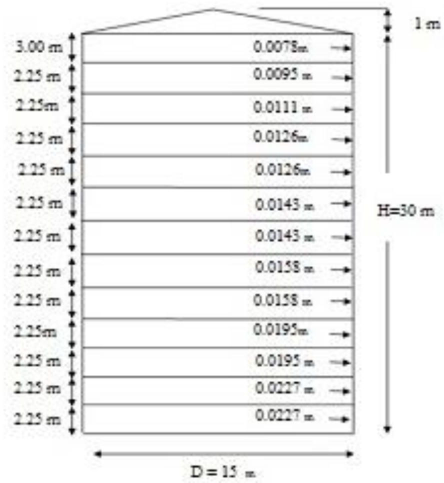


Figure 1. Geometry of tall tank model.

2.2. Liquid storage tank

Only the impulsive component of the hydrodynamic pressure is considered according to Housner (Housner 1963), the impulsive and convective components should be separated to characterize the hydrodynamic response of tank–liquid systems excited horizontally because of the noticeable difference between the frequencies of vibration. This assumption is adopted in research in this area, including that of Virella et al (Virella et al. 2006) and Djermane et al. (Djermane et al. 2014).

The fluid is modeled using the equivalent densities technique (Virella et al. 2006) (Barton et al. 1987). The values of the added liquid density are evaluated from the hydrodynamic pressure diagram of the impulsive mode of a liquid tank system developed by Velestsos and Shivakumar (Veletsos et al. 1997) distribution in regular or irregular wall height subdivisions. Impulsive pressure is given by the expression:

$$P_i = (\eta, \theta, t) = c_i(\eta)\rho R\, \ddot{x}_g\,(t)\cos\theta \tag{1}$$

Where η is a non-dimensional vertical coordinate = z/H; z is the vertical coordinate measured from the tank bottom; R is the tank radius; $\ddot{x}_g(t)$ is the ground acceleration, and t is the time. The function $c_i(\eta)$ defines the impulsive pressure distribution along the cylinder height and is computed as:

$$c_i(\eta) = 1 - \sum_{n=1}^{\infty} c_{cn}(\eta) \tag{2}$$

Where

$$c_{nc}(\eta) = \frac{2}{\lambda_n^2 - 1}\left(\frac{\cosh[\lambda_n (H/R)\eta]}{\cosh[\lambda_n (H/R)]}\right) \tag{3}$$

λ_n is the nth root of the first derivative of the Bessel function, the first three roots λ_1 = 1.841, λ_2 = 5.311, and λ_3 = 8.536. Pressure distributions defined in Eq. (1) for tank-liquid systems considered in this paper and for θ = 0 are shown in Figure 2.

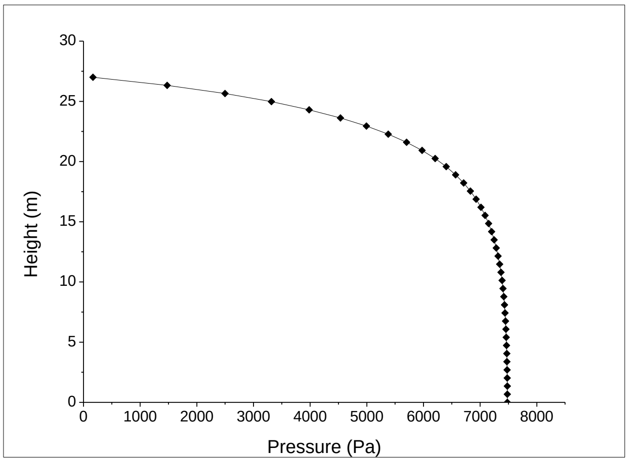


Figure. 2. Impulsive hydrodynamic pressure of tall tank model.

The time of CPU consumption depends on the type (non-linear dynamic analysis) and the mesh of structure, wherefore and to decrease the time of resolution, a short recording with maximum amplitudes occur during the first seconds and contain the essential frequency content would be interesting. For this reason, the earthquake of San Fernando 1971 (Pacoima Dam) Figure 3. is used in this work.

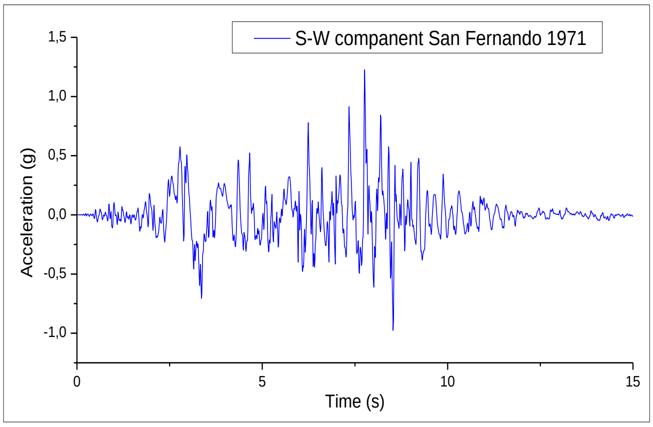


Figure 3. Strong motion during the first duration of the accelerogram.
of San Fernando 1971 (Pacoima Dam).

The development of computers and numerical methods has enabled the development of software dedicated to solving the problem of equilibrium and equilibrium bifurcation structures. It is useful to recall that the software used (ABAQUS (ABAQUS Explicit 2002)) is a code finite element created in 1978.This study was treated using the ABAQUS finite element, the finite element mesh of tank using shell elements for the cylinder and the roof was adopted. The tank bottom has not been modeled. The cylinder tank shell is meshed by S4R quadrilateral shell elements and the roof is meshed by shell S3R triangular elements and beam elements B31 for the roof rafters. The characteristics of these elements are described in ABAQUS (ABAQUS Theory 2002).

3. RESULTS AND DISCUSSION

3.1 Natural frequency analysis

The natural frequencies of free vibration analysis of the impulsive period of the tank system given by international regulations most used can be summarized by the following formulas:

- Code EC8 (Eurocode 8 2002)

$$f_i = 1/\left(C_i \frac{H\sqrt{\rho}}{\sqrt{\frac{t}{R}}\sqrt{E}} \right) \tag{4}$$

- Code API (API Standard 650 1988)

$$f_i = 1/\left(\frac{1}{\sqrt{2000}} \frac{C_i H^{\sqrt{\frac{\rho}{E}}}}{\sqrt{\frac{t}{D}}} \right) \tag{5}$$

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 the fluid, E is the elastic modulus of tank material, t is the equivalent uniform thickness of tank shell, and D is the nominal tank diameter. Table 1 compares the results of the model used with and without a roof (roofless), and stiffeners compared to those given by regulations.

Table1. Comparison of the fundamental frequencies of tanks (Hz).

Model	Code (1) EC 8	Code (1) API 650	Numerical models with roof: without stiffeners (2)	Numerical models with roof: with stiffeners (3)	error between (1) and (2)	error between (2) and (3)
***H/D* = 2**	4,200	4,200	4.760	4.759	3.07%	14.45%

3.2 Dynamic analysis

The buckling behavior of steel tanks under seismic excitation identified by experimental tests and numerical analyses is usually classified as an elastic buckling (Djermane et al. 2014). Researchers in the domain of dynamic instability use mainly two criteria in the investigation of the critical dynamic load of structures. These criteria are: Budiansky & Roth criterion 1962 (Budiansky et al. 1962), called the resolution of the equations of motion and the phase plan of the total potential energy of Hoff & Bruce criterion 1954 (Ari-Gur et al. 1997). In this study, elastic buckling is observed in the top of the cylindrical shell wall of the tank. After several analyses of the structure subjected to different ground motion records at different points, the critical value of the PGA was determined according to the above-cited criteria.

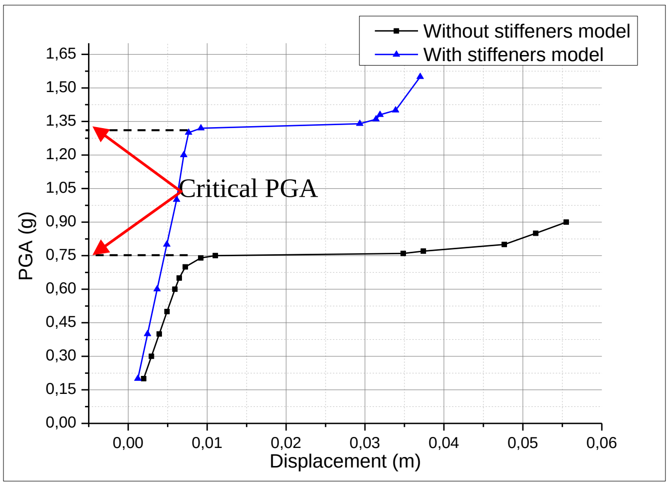


Figure 4. Displacement response dynamic curve under San Fernando.

The peak response is investigated in Figure 4 through the plot of the maximum radial displacement for several values of the PGA according to Ari Gur criterion & Simonetta (Ari-Gur et al. 1997). Buckling critical value for without stiffeners model under San Fernando is observed around 0.75g. In the sudden drop of the curve slope in Figure 4 indicates buckling of the shell after PGA has been exceeded the critical value load. A similar behavior was obtained for with stiffeners model and the dynamic buckling critical value is near 1.30g.

The chronological curves (Budiansky& Roth criterion) for critical PGA values (0.76 g and 1.32 g) and their corresponding deformed shape are shown in Figures 5 and 6. These figures confirm what has been explored in Figure 4. A clear jump in displacements is observed for critical PGA value for a small increase of excitation amplitude, which corresponds to the critical conditions.

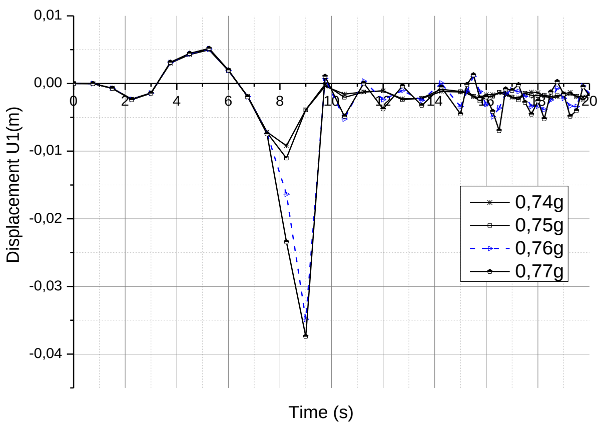


Figure 5. Time history response under San Fernando, before and after PGAs (without

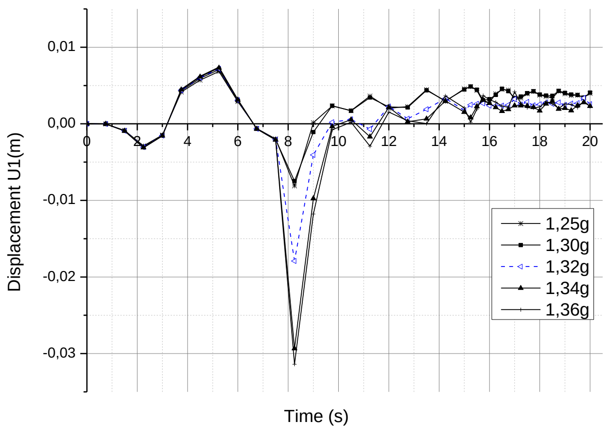
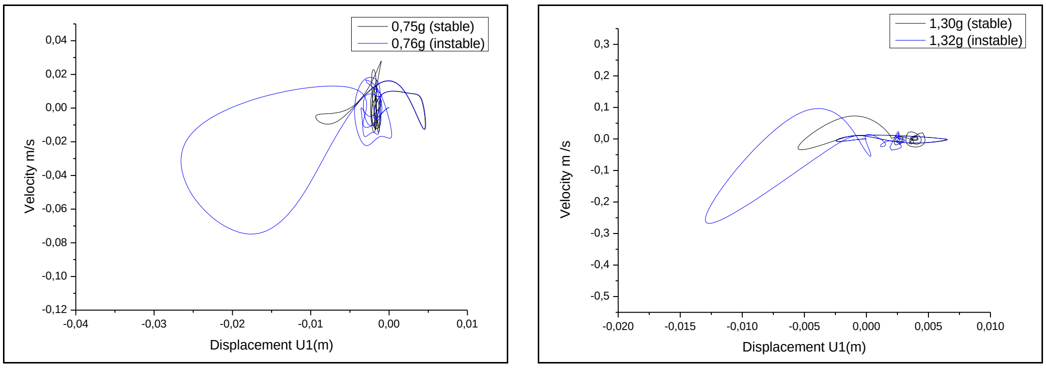


Figure 6. Time history response under San Fernando, before and after PGAs (with

Figure 7 shows the phase planes before and after buckling according to the Hoff & Bruce criterion. Stable and unstable movements are shown for both models.



(a) (b)

Figure 7. Phase plane diagram under San Fernando, before and after PGAs
(a) without stiffeners and (b) with stiffeners models.

The iso-stresses and the distribution of the stresses, for several increasing values of the PGA, are shown in Figures 8 and 9 where it is noted that the plasticity is produced after the advent of dynamic buckling, which indicates that the buckling produced is the elastic buckling since this stress values do not exceed the elastic limit.

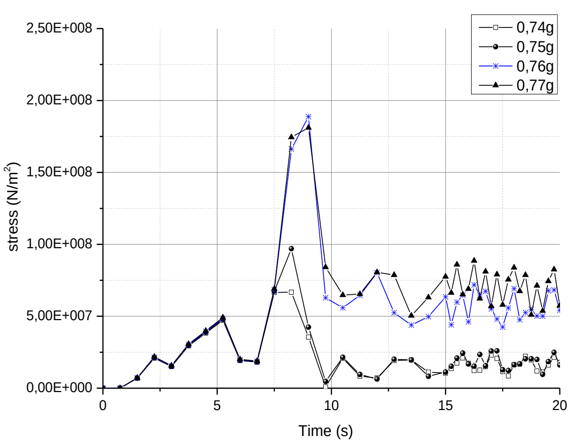


Figure 8. Von Mises stress under San Fernando (without stiffeners model)

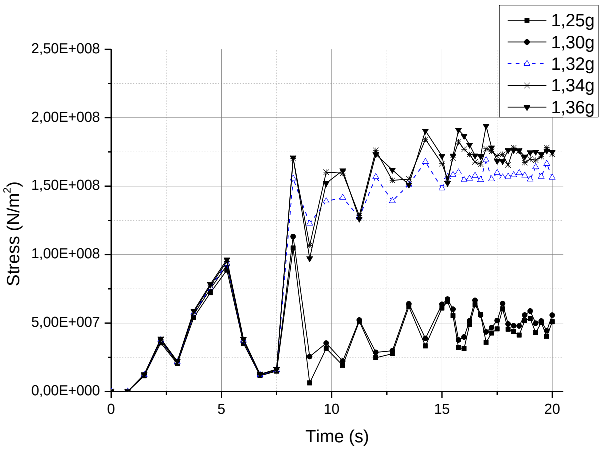


Figure 9. Von Mises stress under San Fernando (with stiffeners model)

The critical buckling stress observed in the numerical analyses and the values assumed in design codes Eurocode 8 (Eurocode 8 2002), API 650 (API Standard 650 1988) are listed in Table 2. The value derived from design code Eurocode 8 reveals that the observed critical buckling load is close to the value predicted by the numerical model. As for design codes API, the observed critical load is about 0.21. The substantial differences between design code values and numerical values may be stemmed from the type of analysis and loads. The elastic and plastic critical buckling stresses assumed in design are based on static buckling and for shell under uniform axial compression or combination with the circumferential shear stress, which is different from the actual loading conditions in this study.

Table 2. Comparison between critical stress and critical buckling correspondent.

Standards	EC8	API	without stiffeners	With stiffeners
			Under San Fernando	
Critical stress	1.384E8	1.330E7	1.875E8	1.741E8
Critical buckling	0.91g	0.21g	0.76g	1.32g

The comparison between the results of with stiffeners model and those of without stiffeners model revealed that the presence of rings required by the API design code has increased the values of the PGA under earthquake of San Fernando (PGAs = 1.32g).

4. CONCLUSION

The dynamic response behavior of a cylindrical thin storage tank is a complex operation that requires a certain expertise. Indeed, it is necessary to properly target the nodes that actually indicate that the tank has undergone buckling as well as the choice of a judicious time step. In our study, several instability criteria are used to predict buckling, after several checks of the points in the deformed region to ensure the reliability of the results. Our numerical model gave a local buckling elastic diamond type, located near the top cover for a PGA value equal to 0.76g under San Fernando. The results of with stiffeners model compared with the results of without stiffeners model have shown good consequences of resistance of the tank against dynamic load with San Fernando excitation.

Further numerical and experimental investigations are needed to better understand the behavior of tanks reinforced by stiffeners. Further, the effect of the position and the cross section required to reinforce tanks against dynamic buckling. This step is therefore necessary in order to make the appropriate proposals for international regulatory requirements not yet available for this type of study.

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