FEM SEISMIC ANALYSIS OF STEEL TANKS FOR OIL STORAGE IN INDUSTRIAL FACILITIES

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ABSTRACT :
Structural and seismic engineering are involved in the design of new industrial facilities, but have certainly a primary role in the evaluation and upgrading of existing plants. Atmospheric steel tanks for oil and other hazardous material storage are commonly used in power plants, airports, and other critical plants. Their design is somehow very standardized worldwide and thus they represent a challenging topic in the context of an industrial risk assessment related to external hazards like earthquakes. In fact, their dynamic response is not trivial, since fluid/structure interactions are relevant and influence the susceptibility to seismic damage. A full stress analysis is certainly the more accurate way to design and to evaluate the risk of steel tanks under earthquake loads, but is generally demanding in terms of computational effort. This approach leads to the direct computation of the interaction between shell deformations and content motion during earthquakes. In the present paper, seismic evaluation according to Eurocode 8 is discussed and some global results of Finite Element Analyses (FEM) analyses are compared with those obtained according to simplified design procedures by Eurocode 8.

KEYWORDS:
Industrial facilities, atmospheric steel tank, seismic vulnerability, seismic design, dynamic analysis.

1. INTRODUCTION

Structural and seismic engineering are involved in the design of new industrial facilities, but have certainly a primary role in the evaluation and upgrading of existing plants. Review of typical industrial layouts (Di Carluccio 2007, Fabbrocino et. al. 2007) shows that a large number of components and systems are strongly standardized. This is a relevant aspect in the framework of seismic protection of existing plants, since simulated structural design is sometimes needed moving often from poor data. Among industrial components, atmospheric steel tanks for oil and other hazardous material storage are commonly used in power plants, airports, and other critical plants. Furthermore, their design is very standardized worldwide and thus they represent a challenging topic in the contexts of an industrial risk assessment related to external hazards like earthquakes. In fact, their dynamic response is not trivial, since fluid/structure interactions are relevant and influence the susceptibility to seismic damage. Base shear and overturning moments due to seismic actions lead to two main damage scenarios from a structural engineering point of view: large displacements at the base for unanchored tanks and elephant foot buckling of the shell, primarily in the case of anchored tanks. A full stress analysis is certainly the more accurate way to design and to evaluate the risk of steel tanks under earthquake loads, but is generally demanding in terms of computational effort. This approach leads to the direct computation of the interaction between shell deformations and content motion during earthquakes (Haroun 1999). For base constrained and rigid tanks (anchored), a complete seismic analysis requires solution of Laplace's equation for the motion of the contained liquid, in order to obtain the total pressure history on the tank shell during earthquakes (Eurocode 8). When flexible tanks are considered contribution of structural deformation cannot be neglected, this is generally the case of steel tanks. Actually the study of seismic behavior of storage steel tank is possible with two different approaches: the first based on lumped mass models and second based on the use of finite elements. In the present paper, seismic evaluation according to Eurocode 8 is discussed and some global results of FEM analyses are briefly presented and compared with those obtained according to simplified design procedures.

2. EARTHQUAKE COLLAPSE OF ATMOSPHERIC STEEL STORAGE TANKS

The seismic event is certainly one of the most critical external event to the safety of industrial plants, as demonstrated by recent earthquakes. If industrial facilities store large amount of hazardous materials, accidental scenarios as fire, explosion, or toxic dispersion may be triggered, thus possibly involving working people within the installation, the population living in the close surrounding or in the urban area where the industrial installation is located. Atmospheric steel storage tanks, anchored or unanchored, are relevant components of lifeline and industrial facilities. In fact they are very common in industrial sites where storage of water, oil, chemicals and liquefied natural gas is required. Dynamic behavior of atmospheric storage tanks when subjected to earthquake is characterized by two pre-dominant modes of vibration: the first is related to the mass that rigidly moves together with the tank structure (impulsive mass), the other corresponds to the liquid sloshing (convective mass) (Malhotra et. al., 2000). Seismic response of steel tanks depends however on complex fluid/structure interaction that result in global overturning moments and base shear induced by horizontal inertial forces. Overturning moment causes an increase of the vertical stress in the tank wall and even uplift of the base plate, while the base shear can lead to relative displacements between the base plate and the foundation. Failure modes reflect these specific aspects of the seismic demand on the structure and depend basically upon the type of interface at the tank base and the presence of mechanical devices are used to en-sure an effective connection between the base plate and the foundation (unanchored or anchored). When unanchored tanks are of concern, the friction at the base is able to ensure the needed stability of the structure under environmental actions, i.e. wind, but can be ineffective when strong ground motions take place, thus generating large relative displacements. Indeed, tank sliding reduces the maximum acceleration suffered by the equipment, however relatively small frictional factor may produce large relative displacements, hence large deformations and even failure of piping and connections can occur (Fig. 1-left).

Figure 1 (left) Example of I/O pipe failure (http://www.eqe.com); (right) Elephant Foot Buckling damage in Turkish plant. (Kocaeli, 1999, Turkey; Saatcioglu et. al. 2001)

In addition, another large-displacement mechanism is the partial uplift of the base plate. This phenomenon reduces the hydrodynamic forces in the tank, but can increases significantly the axial compressive stress in the tank wall and the possibility that a characteristic buckling of the wall (Elephant Foot Bucking – EFB) occurs. EFB (Fig. 1-right) is usually associated with large diameter tanks with height to radius (H/R) ratios in the range 2 to 3, whereas another common buckling mode, known as diamond shape buckling (DSB), is associated with taller tanks, that is H/R ratios about 4. While EFB is associated with an elastic-plastic state of stress, the DSB is a purely elastic buckling. Other structural damage are the collapse of support columns for fixed roof tanks, tank failures due to foundation collapse, splitting and leakage associated only with bolted and riveted tanks. Liquid sloshing during earthquake action produces several damages by fluid–structure interaction phenomena and can result as the main cause of collapse for full or nearly full tanks. Historical analysis and assessment of seismic damages of storage tanks have demonstrated that basically full (or near full) tanks experienced catastrophic failures. Low H/R tanks only suffered cracks in conical roof connection, or damage by floating panel sinking. A very common shell damage is the EFB. For unanchored tanks and H/R < 0.8, EFB is typically not experienced but the base plate or the shell connection can fail causing spillage (Ballantyne & Crouse 1997).

3. SEISMIC ANALYSES ACCORDING TO EUROCODE 8

3.1. Rigid Tank

The response of a rigid tank when subjected to seismic action can be described by two hydrodynamic components called rigid-impulsive component and convective component. For a tank under lateral excitations, the liquid in the lower part of the container tend to move at unison with the shell and are subjected to the same acceleration. The rigid impulsive pressure component is:

$$p_i(\xi,\zeta,\theta,t)=C_i(\xi,\zeta)\rho_l H\cos(\theta)\ddot{x}_g(t) \tag{3.1}$$

where $\xi = r/R$, $\zeta = z/H$, ρ_l is the density of liquid, $\ddot{x}_g(t)$ is the ground acceleration, finally r, θ and z are the radial, circumferential and vertical coordinates respectively, $C_i(\xi,\zeta)$ is as follows:

$$C_i(\xi,\zeta)=\sum_{n=0}^{\infty}\left[(-1)^n \Big/ I_1'\left(\frac{\upsilon_n}{\gamma}\right)\upsilon_n^2\right]\cos(\upsilon_n\zeta)I_1\left(\frac{\upsilon_n}{\gamma}\xi\right) \tag{3.2}$$

It is assumed $\upsilon_n = \dfrac{2n+1}{2}\pi$, while γ is the filling level (*H/R*), I_1 and I_1' denote the first order modified Bessel function and its derivate. The part of mass that move as rigid impulsive mass is:

$$m_i = m2\gamma\sum_{n=0}^{\infty}\left[I_1\left(\frac{\upsilon_n}{\gamma}\right)\Big/\upsilon_n^3 I_1'\left(\frac{\upsilon_n}{\gamma}\right)\right] \tag{3.3}$$

where *m* represent the total liquid mass. The liquid located in the upper part of the tank moves independently respect to the tank wall, configuring the so-called sloshing. It leads to pressures on the tank walls and base. Spatial-temporal variation of this component is given by:

$$p_c(\xi,\zeta,\theta,t)=\rho_l\sum_{n=1}^{\infty}\frac{2R\cosh(\lambda_n\gamma\zeta)J_1(\lambda_n\xi)}{(\lambda_n^2-1)J_1(\lambda_n)\cosh(\lambda_n\gamma)}\cos(\theta)\ddot{x}_n(t) \tag{3.4}$$

where J_1 is Bessel function of first order and $\ddot{x}_n(t)$ is the response acceleration of a single degree of freedom oscillator having a frequency f_n. Frequencies associated with the sloshing modes are usually low and for a given mode depend on tank dimensions but are basically independent on liquid height. The n^{th} sloshing frequency is given by:

$$f_n = \frac{1}{2\pi}\sqrt{g\frac{\lambda_n}{R}\tanh(\lambda_n\gamma)} \quad (Hz) \tag{3.5}$$

where g is the acceleration due to gravity and λ_n are the zeros of the first derivative of the first order Bessel function and values of λ_n for the first four modes are given by:

$$\lambda_1 = 1.811 \quad \lambda_2 = 5.331 \quad \lambda_3 = 8.536 \quad \lambda_4 = 11.706 \tag{3.6}$$

The convective pressure varies as a cosine function in the circumferential direction. Generally, only the fundamental sloshing mode is accounted for in the computation of convective pressures as the higher sloshing modes have low participating mass and consequently low associated pressures. The mass associated to the n^{th} sloshing mode is expressed by equation 3.7.

$$m_{cn} = m \frac{2 \tanh(\lambda_n \gamma)}{\gamma \lambda_n (\lambda_n^2 - 1)} \tag{3.7}$$

3.2. Flexible tanks

When flexibility of the walls cannot be neglected, an additional flexible-impulsive pressure component is activated. This is dependent upon the flexibility of the tank and is generated by an interaction between the fluid motion and the deformation of the wall. The pressure due to the vibration of fluid tank system results from the contribution of an infinite number of fluid tank vibration modes. Different formulations for the flexible impulsive pressure component have been proposed by a number researchers (Haroun & Housner 1981, Veletsos & Yang 1977, Tedesco et. al. 1989, Fischer et. al. 1991) and are reported in international design codes (Eurocode 8, ASCE 1984). Assuming that the vibration modes of system are known the flexible pressure distribution is given in equation 3.8 :

$$p_f(\zeta, \theta, t) = \rho H \psi \sum_{n=0}^{\infty} d_n \cos(\upsilon_n \zeta) \cos(\theta) \ddot{x}_f(t) \tag{3.8}$$

The above expression is, in general, based on the assumption that only the first circumferential (n= 1) mode significantly contributes to the pressure. Fischer et. al. 1991 report that this assumption is valid since: modes with n≠1 do not contribute to the overturning moment which leads to the most common failure mode for tanks and higher order n= 1 modes have low participating mode factors. Expressions for the fundamental natural frequency of the flexible impulsive mode of vibration are available; an approximated formula is provided by Eurocode 8 and has the following form:

$$f_s = \frac{1}{2Rg(\gamma)} \sqrt{\frac{Es_{1/3}}{\rho_l H}} \tag{3.9}$$

where $s_{1/3}$ is the thickness at 1/3 height and:

$$g(\gamma) = 0.01675\gamma^2 - 0.15\gamma + 0.46 \tag{3.10}$$

3.3. Simplified design procedure

Eurocode 8 provides also a very simplified procedure (Malhotra 1997). In particular, the fluid-tank system is analyzed as two single degree freedom systems, one corresponding to the impulsive and the other corresponding to the convective action. The natural periods of the impulsive and the convective responses, in seconds, are:

$$T_{imp} = C_i \frac{H\sqrt{\rho_l}}{\sqrt{s/R}\sqrt{E}} \quad (a) \qquad T_{con} = C_c \sqrt{R} \quad (b) \tag{3.11}$$

where s is the equivalent uniform thickness of the tank wall. Coefficients C_i and C_c are provided depending on the filling level. The total base shear and overturning moment are expressed as follows:

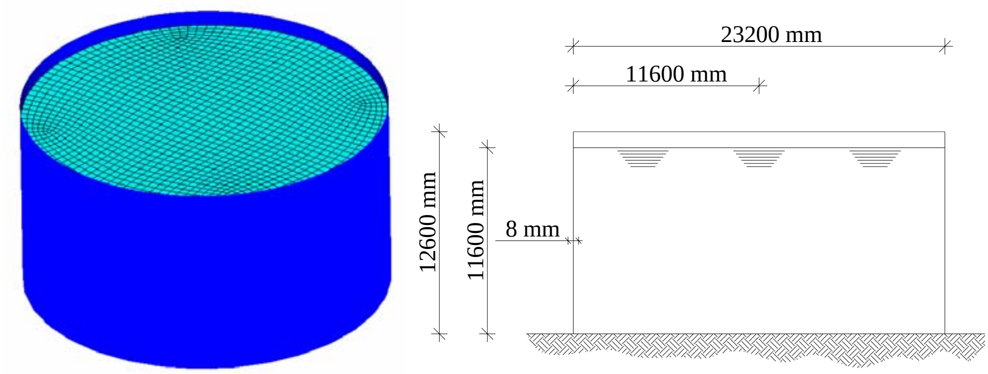
$$Q = (m_i + m_w + m_{rof})S_e(T_{imp}) + m_c S_e(T_{con}) \tag{3.12}$$

$$M = (m_i h_i + m_w h_w + m_{rof} h_{rof})S_e(T_{imp}) + m_c h_c S_e(T_{con}) \tag{3.13}$$

where, m_w is the mass of tank wall, m_{rof} is the mass of tank roof, h_i and h_c are the heights of the centroid of the impulsive and convective hydrodynamic pressure, h_w and h_{rof} are heights of the centres of gravity of the tank wall and roof, respectively; $S_e(T_{imp})$ is the impulsive spectral acceleration and $S_e(T_{con})$ is the convective spectral acceleration.

4. NUMERICAL ANALYSIS

The structure analyzed in the present study, shown in Fig. 2, is a typical atmospheric storage steel tank with a volume of $5000\ m^3$. The tank is filled with liquid gasoline with a density of $680\ kg/m^3$. The cylinder has an inner diameter of *23200 mm*, a thickness of *8 mm* and is made of a steel plate with $E=210\ GPa$, $\nu=0.3$ and $\rho=7850\ kg/m^3$. The cylinder is 12600 *mm* high with a liquid height of *11600 mm*. Additional details on geometry dimension can be found in Figure 2.



Shell Elements	3420
Solid Elements	21000
Total Joints	26880

Figure 2 LsDyna Finite Element models and geometry details.

Table 4.1 Set of European earthquakes used in the time history analysis.

Code	Event Name	Country	Date	Station Name	PGA [m/sec^2]	Duration [sec]
000196xa	Montenegro	Yugoslavia	15/04/1979	Petrovac-Hotel Oliva	4.4530	48.230
006334xa	South Iceland (aftershock)	Iceland	21/06/2000	Solheimar	4.3543	84.995
000199ya	Montenegro	Yugoslavia	15/04/1979	Bar-Skupstina Opstine	3.5573	47.820
000535ya	Erzincan	Turkey	13/03/1992	Erzincan-Meteorologij Mundurlugu	5.0275	20.750
006263ya	South Iceland	Iceland	17/06/2000	Kaldarholt	5.0180	72.480
006328ya	South Iceland (aftershock)	Iceland	21/06/2000	Kaldarholt	3.8393	51.380
006334ya	South Iceland (aftershock)	Iceland	21/06/2000	Solheimar	7.0614	84.995

Time history analyses using LsDyna finite element program have been carried out. A set of 7 European strong motion records was considered; relevant data concerning seismic input are reported in Table 4.1.

The selected earthquake ground motions records are all stiff soil records and are compatible with Eurocode 8 design spectra for stiff soil and high hazard level (Zone 1). Selection of earthquakes has been carried out according to criteria given in (Iervolino et. al. 2008).
The finite element analyses presented have been performed with LsDyna code using a Lagrangian approach. The Finite Element program used in the analysis is LsDyna (LSTC 2003). LsDyna uses an explicit Lagrangian numerical method to solve nonlinear, three dimensional, dynamic, large displacement problems. Implicit, arbitrary Lagrangian-Eulerian, Smoothed Particle Hydrodynamics (also known as SPH) are also available; Lagrangian, ALE and SPH numerical method can be used for liquid storage tank (Vesenjak et. al. 2004). For the modeling of tank wall four joints shell elements has been used; the liquid has been modeled with solid elements. Details on the analyzed models are shown in Figure 3. The material models used for steel is MAT_1. In LsDyna it is an isotropic elastic material and is available for beam, shell, and solid elements. Liquid component of the structural system has been modelled using MAT_9 material type, that is the NULL material.

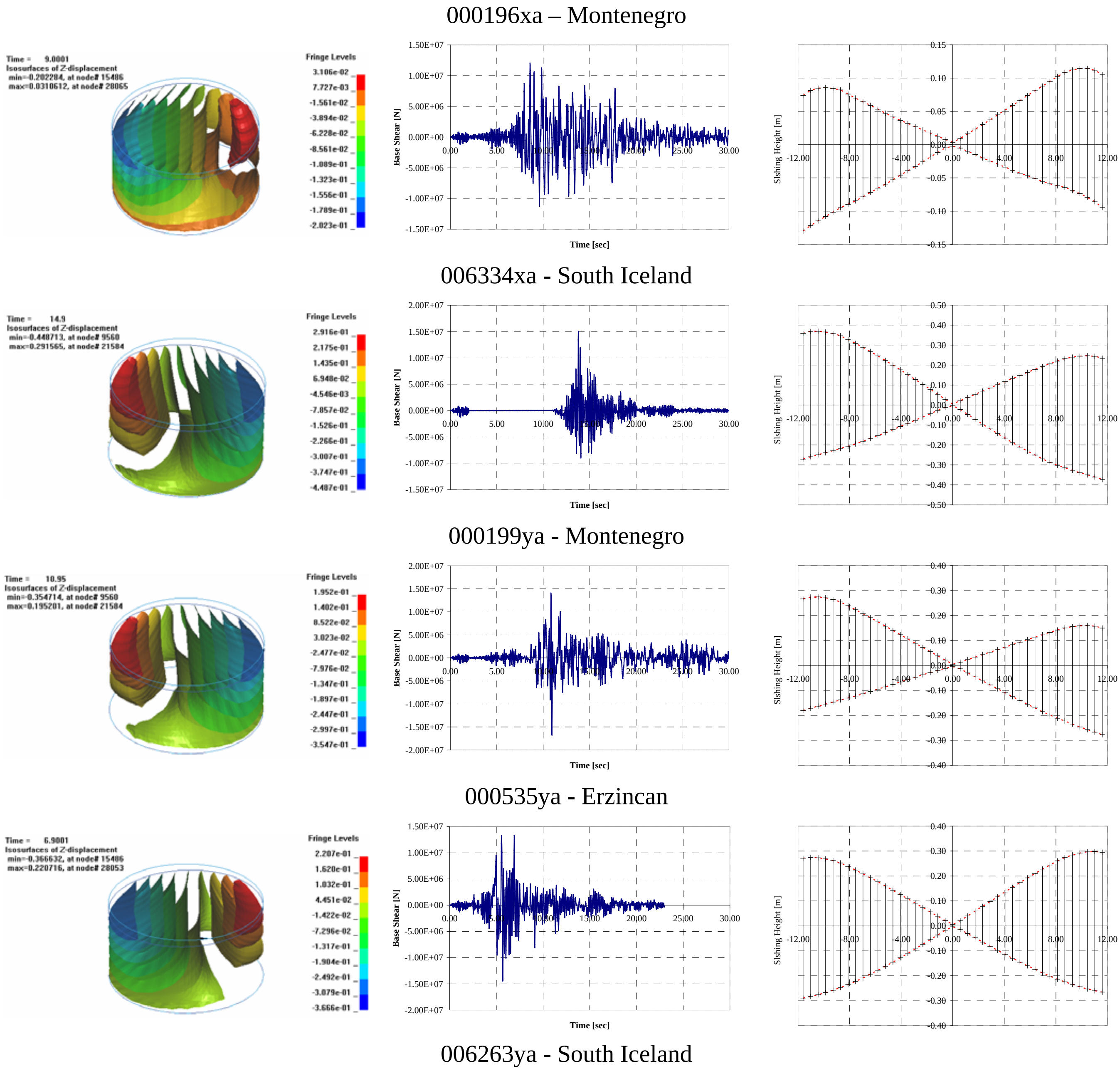


Figure 3.a LsDyna Finite Element results.

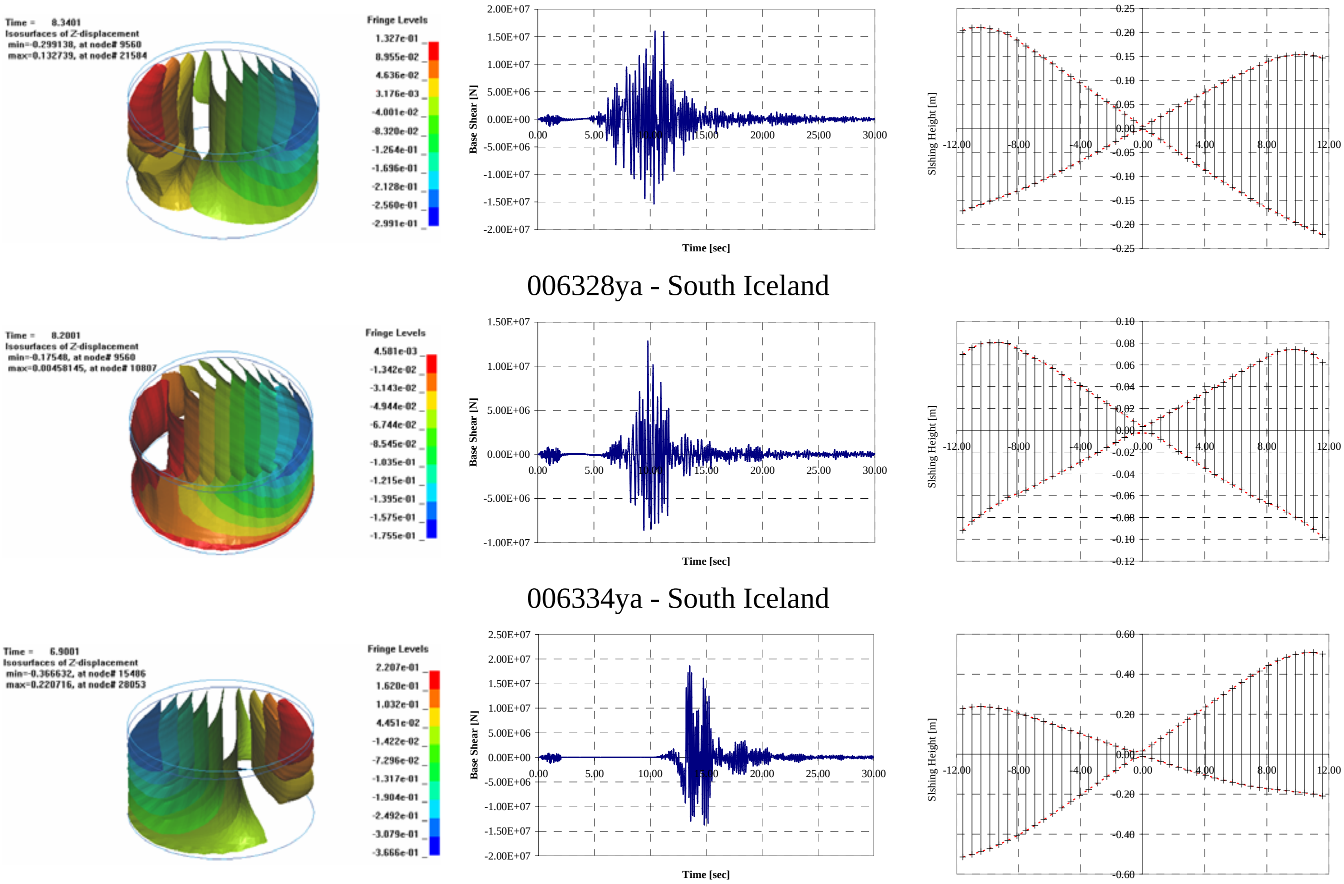


Figure 3.b LsDyna Finite Element results.

This kind of material takes account of the equation of state without computing the deviatoric stress and has zero shear stiffness. It has no yield strength and behaves in fluid-like manner. For analyses presented in the paper the Grunesein's equation was used. As contact type a Contact Node to Surface is used. All the analyses need a preliminary dynamic relaxation; duration of such phase has been calibrated and it has been found that 2 seconds can give good results and optimise computational time. Figure 3 reports selected results for each time history analyses. On the left hand side of the figure, FEM results are given in terms of liquid displacements along vertical direction. In particular, the surfaces characterised by the same displacement are shown. On the right hand the vertical displacement of free surface is showed, while the central plot shows a part of base shear time history.

Table 4.2 Numerical results in terms of base shear and sloshing height.

Earthquake	Peak of Base Shear [kN]	Peak of Sloshing Height [m]	EC8 Base Shear [kN]	EC8 Sloshing Height [m]
000196xa	12100	0.13		
006334xa	15100	0.37		
000199ya	16800	0.28		
000535ya	14500	0.30		
006263ya	16000	0.22		
006328ya	12900	0.10		
006334ya	18600	0.51		
Mean	15143	0.27	20200	0.51
Standard deviation	2241	0.14		

Table 4.2 reports global results of FEM analyses in terms of peak base shear and peak of sloshing height. Comparison with corresponding values obtained from design code formulation demonstrate that the latter give by

fare more conservative results compared with those obtained from full stress dynamic analyses. This is a useful result in view of assessment of existing facilities, but it cannot be addressed as a definitive result. Additional calculations are needed to give reliable suggestions for risk assessment procedures.

5. CONCLUSIONS

This paper reports an evaluation of the seismic response of typical steel components used in the process industry. In particular, attention has been focussed on the seismic design and analysis of tanks for storage of oil and other hazardous materials. They are very common worldwide and can help to develop methods of seismic analysis able to take account of fluid/structure interactions. A satisfactory capacity of simplified models to fit the overall response of tanks has been shown. This circumstance is by far more relevant, since computational efforts for full stress analyses are huge compared to those required by simplified methods.

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