

## Research Article

# Some Remarks on the Seismic Demand Estimation in the Context of Vulnerability Assessment of Large Steel Storage Tank Facilities

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The seismic behavior of steel tanks is relevant in industrial risk assessment because collapse of these structures may trigger other catastrophic phenomena due to loss of containment. Therefore, seismic assessment should be focused on for leakage-based limit states. From a seismic structural perspective, damages suffered by tanks are generally related to large axial compressive stresses, which can induce shell buckling near the base and large displacements of unanchored structures resulting in the detachment of piping. This paper approaches the analysis of seismic response of sliding, nonuplifting, unanchored tanks subject to seismic actions. Simplified methods for dynamic analysis and seismic demand estimation in terms of base displacement and compressive shell stress are analyzed. In particular, attention is focussed on some computational issues related to the solution of the dynamic problem and on the extension of the incremental dynamic analysis (IDA) technique to storage tanks.

## 1. Introduction

Earthquakes represent an external hazard for industrial installations and may trigger accidents, that is, fire and explosions resulting in injury to people and to near field equipments or constructions, whenever structural failures result in the release of hazardous material. In recent years the quantitative risk analysis (QRA) procedures have been extended to include seismic analysis for industrial installation, for example, petrochemical plants, and chemical plants, storage facility [1–3]. From the structural perspective, steel tanks for oil storage are standardized structures both in terms of design and construction [4–6]. A review of the international standards points out that the design procedures evolved slowly. Nevertheless, a large number of postearthquake damage observations [7] are available and empirical vulnerability functions have been developed [8]. This is a privileged case with respect to other civil engineering structures that basically can be seen as single prototypes; however, empirical fragility typically suffers some shortcomings. In fact, site effects can influence vulnerability data, so that a disaggregation is hard to perform. Therefore,

the development of analytical models able to predict the response of the structural components and systems under seismic loading is worth exploring.

The present work is aimed at discussing a numerical procedure able to analyze the response of anchored and unanchored tanks under two-dimensional ground motion accounting for the structure sliding. The main objective of the study is, in particular, the optimization of the computational effort related to seismic reliability assessments [9, 10] of industrial components and systems. A lumped mass representation of the fluid-tank system motion can be adopted [11]; it is a simplified but reliable model that is also recommended by relevant international codes, that is, Eurocode 8 [12].

Based on the available theoretical approaches to the analysis of storage tanks, the numerical model adopted here combines and extends them in order to include large-displacement limit states related to sliding that leads to piping failures and consequently to the release of content.

The numerical algorithm associated to the model can be used to perform the estimation of seismic demand in terms of base plate-ground relative displacement and

shell compressive stress. The above-mentioned parameters are assumed as engineering demand measures related to the failure of connection piping and shell elephant foot buckling (EFB). In particular, reference is made to the incremental dynamic analysis (IDA) method [13] which was originally developed for buildings and then extended to tanks. Particular attention is paid to the selection criteria for earthquake data sets and to the relationship between the required accuracy of the average demand and the total number of records to be used. It is worth noting that issues related to seismic capacity are out of the scope of the present paper, so that relevant aspects on this specific subject are not included here.

## 2. Atmospheric Steel Storage Tanks

The earthquake is certainly one of the most critical external events to the safety of industrial plants, as recently showed, for instance, by extended damage scenarios in China (2008), where some 80 tons of ammonia leaked from the plants, forcing the evacuation of 6000 residents in Shifang [14, 15] or in Chile (2010), where oil leakage and collapses of elevated tanks occurred [16]. Severe damages and postearthquake fires took place later also in Japan (2011) [17] due to both ground shaking and tsunami that is outside the scope of the discussion here. 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 and/or population living close to the industrial installation.

Among industrial constructions, atmospheric steel tanks, anchored or unanchored, are relevant components of lifeline and industrial facilities. In fact they are very frequent in industrial sites where there is storage of water, oil, chemicals, and liquefied natural gas as well as grain-like materials [18]. Analysis of typical industrial layouts shows that a large number of components and systems are strongly standardized [19]. This is a relevant aspect in the framework of seismic protection of existing plants, since simulated structural design is sometimes needed starting from poor data. The dynamic response of storage tank 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 states: 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. 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 [12]. When flexible tanks are considered, contribution of structural deformation cannot be neglected; this is generally the case of steel tanks. Actually the study of the seismic behavior of storage steel tank is possible with two

different approaches: the first based on lumped mass models and the second based on the use of finite elements [20].

It is assumed that the dynamic behavior of atmospheric storage tanks is mainly affected by two predominant modes of vibration: the first is related to the mass that rigidly moves together with the tank structure (impulsive mass) and the other corresponds to the liquid sloshing (convective mass) [21]. Seismic response of steel tanks depends, however, on complex fluid/structure interaction that may result in global overturning moments and base shear induced by horizontal inertial forces. The 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 produce 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 used to ensure an effective connection between the base plate and the foundation (unanchored or anchored). When unanchored tanks are of concern, the friction at the base provides the needed stability of the structure under environmental actions, that is, wind, but can be ineffective when strong ground motions take place, resulting in 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.

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 increase significantly the axial compressive stress in the tank wall and the possibility that a characteristic buckling of the wall (elephant foot buckling—EFB) occurs. EFB 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 generally related to taller tanks ( $H/R$  ratios about 4).

While EFB is associated with an elastic-plastic state of stress, the DSB is a purely elastic buckling. Other structural damages are the collapse of support columns for fixed roof tanks and tank failures due to foundation collapse, splitting, and leakage associated only with bolted and riveted tanks.

Liquid sloshing during an earthquake action produces several damages by a fluid-structure interaction phenomena and can result as the main cause of hazardous consequences for full or nearly full tanks.

## 3. One-Dimensional Lumped Mass Modeling

Early studies by Housner approached the dynamic response of tanks under seismic loading and led to a proposal of a simplified model for seismic design and an evaluation of anchored tanks with rigid walls [22]. For tanks with a free liquid surface subjected to horizontal ground accelerations, it is assumed that a given fraction of the liquid is forced to participate in this motion as rigid mass, while the motion of the tank walls excites the liquid into oscillations which result

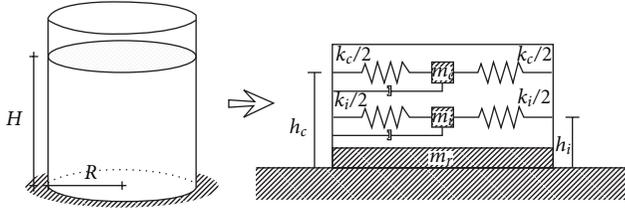


FIGURE 1: One-dimensional dynamic model of tank.

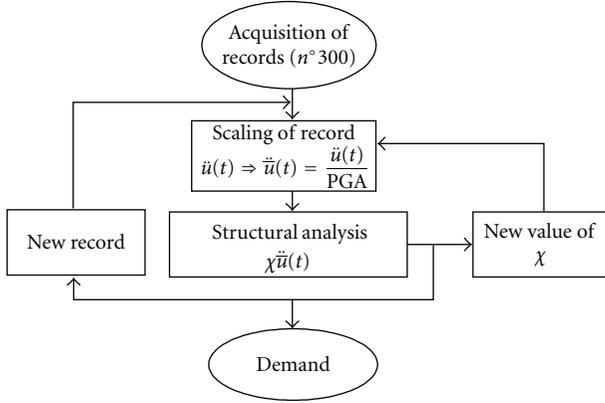


FIGURE 2: Flow chart of incremental dynamic analysis.

in a dynamic force on the tank. This force is assumed to be the same of a lumped mass, known as a convective mass, which can vibrate horizontally restrained by a spring [23, 24].

Rosenblueth and Newmark [25] reviewed the relationships suggested by Housner to estimate the convective and rigid masses and gave updated formulations for the evaluation of the seismic design forces of liquid storage tanks. In 1983 Haroun developed a model to evaluate the seismic response of storage tanks including wall deformation [26]. According to such an idealization, a part of the liquid moves independently with respect to the tank shell, again convective motion, while another part of the liquid oscillates at unison with the tank. If the flexibility of the tank wall is considered, a part of this mass moves independently (impulsive mass) while the remaining accelerates back and forth with the tank (rigid mass). Figure 1 shows the idealized structural model of liquid storage tank. The contained continuous liquid mass is lumped as convective, impulsive, and rigid masses and referred as  $m_c$ ,  $m_i$ , and  $m_r$ , respectively.

The convective and impulsive masses are connected to the tank wall by different equivalent springs having stiffness  $k_c$  and  $k_i$ , respectively. In addition, each spring can be associated to an equivalent damping ratio  $\xi_c$  and  $\xi_i$ . Damping for impulsive mode of vibration can be assumed to be about 2% of critical for steel tanks, while the damping for convective mode can be assumed as 0.5% of critical. However, liquid damping effects are herein neglected without any loss of generality and relevance of results. Under such assumptions, this model for anchored storage tank has been extended to analyze unanchored base-isolated liquid storage tanks [27–29].

Effective masses are given in (1)–(4) as a fraction of the total mass  $m$  (5). Coefficients  $Y_c$ ,  $Y_i$ , and  $Y_r$  depend upon the filling ratio  $S = H/R$ , where  $H$  is the liquid height and  $R$  is the tank radius, as clearly shown in Figure 1:

$$m_c = mY_c, \quad (1)$$

$$m_i = mY_i, \quad (2)$$

$$m_r = mY_r, \quad (3)$$

$$m_{\text{tot}} = m_c + m_i + m_r, \quad (4)$$

$$m = \pi R^2 H \rho_w. \quad (5)$$

Similarly, natural frequencies of convective mass  $\omega_c$  and impulsive mass  $\omega_i$  (6) can be retrieved from [27]:

$$\omega_c = \sqrt{1.84 \left( \frac{g}{R} \right) \tanh \left( 1.84 \frac{H}{R} \right)}, \quad (6)$$

$$\omega_i = \frac{P}{H} \sqrt{\frac{E}{\rho_s}},$$

where  $E$  and  $\rho_s$  are the modulus of elasticity and density of tank wall, respectively,  $g$  is the acceleration due to gravity; and  $P$  is a dimensionless parameter depending on the ratio  $H/R$  as well.

#### 4. Dynamic Response of Unanchored Tanks

Motion of unanchored tanks can be affected by large-displacement phenomena: during the ground motion, the tank can slide with respect to the foundation and the base plate may uplift due to overturning moment.

The sliding depends on the base shear; once it reaches the limit value corresponding to the frictional resistance (7), a relative motion between the tank and the foundation starts. Sliding, however, reduces the maximum acceleration suffered by the tank; this reduction is dependent upon to the frictional factor ( $\mu$ ), but relatively small values of the latter may produce large relative displacements.

Different models can be used in a sliding system to describe the frictional force and belong basically to two classes: conventional and hysteretic [30]. The conventional model is discontinuous and a number of stick-slide conditions lead to solve different equations and to repeated check at every stage; on the other hand, the hysteretic model is continuous and the required continuity is maintained by the hysteretic displacement components. In the following analyses the conventional model is used for the frictional force, but this assumption actually does not represent a limitation of the approach.

In detail, the friction force is evaluated by considering the equilibrium of the base: the system remains in the nonsliding phase if the frictional force in time  $t$  is lower than the limit frictional force expressed by (7), where  $g$  represents the gravitational acceleration:

$$F_{\text{lim}} = m\mu g. \quad (7)$$

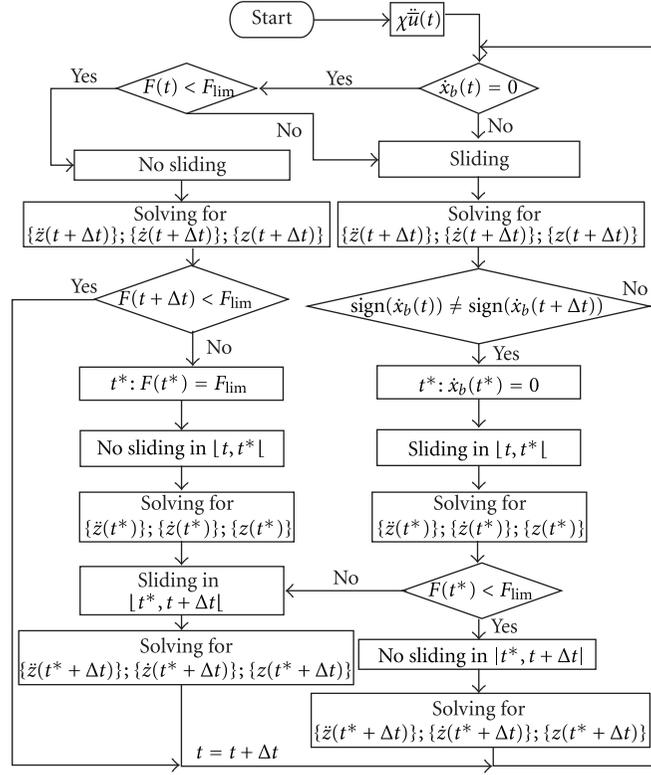
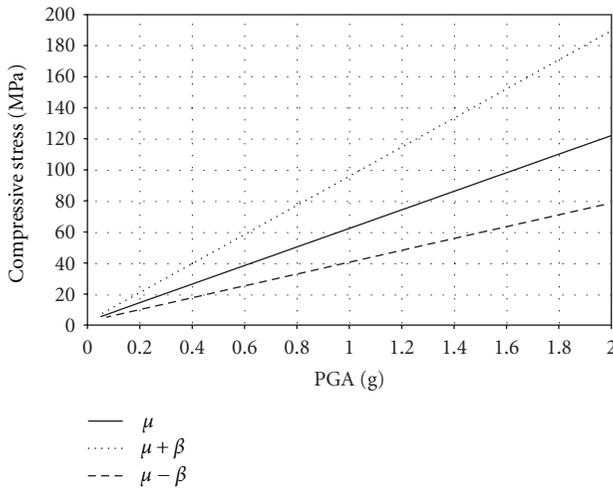


FIGURE 3: Algorithm flow chart.

FIGURE 4: IDA curve for the compressive axial stress for the anchored storage tank with  $V = 30000 \text{ m}^3$  and filling level equal to 50%.

Therefore, the motion can be divided into non-sliding and sliding phases. Whenever the tank does not slide, the dynamic equilibrium of forces in (8) applies in the case of one horizontal component, while (9) fits the case of both horizontal components acting together:

$$F_x = -\left(m_c \ddot{x}_c + m_i \ddot{x}_i + m_{\text{tot}} \ddot{x}_b + m_{\text{tot}} \ddot{u}_{gx}\right), \quad (8)$$

$$F_x = -\left(m_c \ddot{x}_c + m_i \ddot{x}_i + m_{\text{tot}} \ddot{x}_b + m_{\text{tot}} \ddot{u}_{gx}\right), \quad (9)$$

$$F_y = -\left(m_c \ddot{y}_c + m_i \ddot{y}_i + m_{\text{tot}} \ddot{y}_b + m_{\text{tot}} \ddot{u}_{gy}\right).$$

In addition, another large-displacement mechanism can be addressed according to field observations of the seismic response of unanchored liquid storage tanks; it is represented by the partial uplift of the base plate [31]. This phenomenon reduces the hydrodynamic forces in the tank but increases significantly the axial compressive stress in the tank wall. In fact, base uplifting in tanks supported directly on flexible soil foundations does not lead to a significant increase in the axial compressive stress in the tank wall, but may lead to large foundation penetrations and several cycles of large plastic rotations at the plate boundary [11, 32, 33]. Flexibly supported unanchored tanks are therefore less prone to elephant foot buckling damage, but more sensitive to uneven settlement of the foundation and fatigue rupture at the plate-shell connection. A particularly interesting aspect is represented by the force-displacement relationship for the plate boundary. The definition of this relationship is complicated by the nonlinearities arising from (1) the continuous variation of the contact area of the interface between the base plate and the foundation, (2) the plastic yielding of the base plate, and (3) the effect of the membrane forces induced by the large deflections of the plate. In the following, partial uplift of the base plate is not considered in compliance with the primary objective of the paper. However, the numerical procedure herein discussed can be

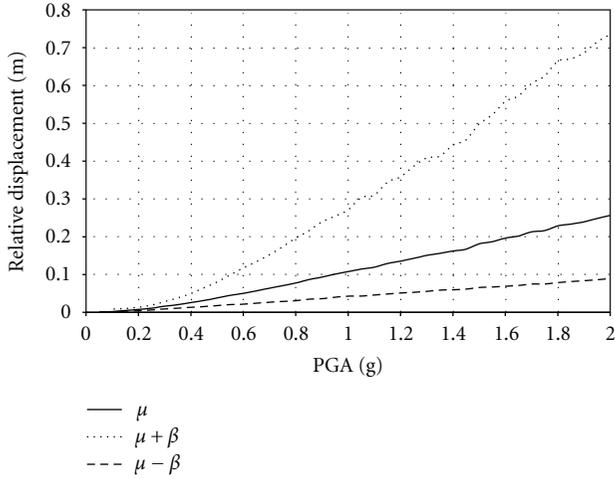


FIGURE 5: IDA for the sliding induced displacement for the bidirectional analysis for the unanchored storage tank with  $V = 5000 \text{ m}^3$ , filling level equal to 80% and friction factor equal to 0.3.

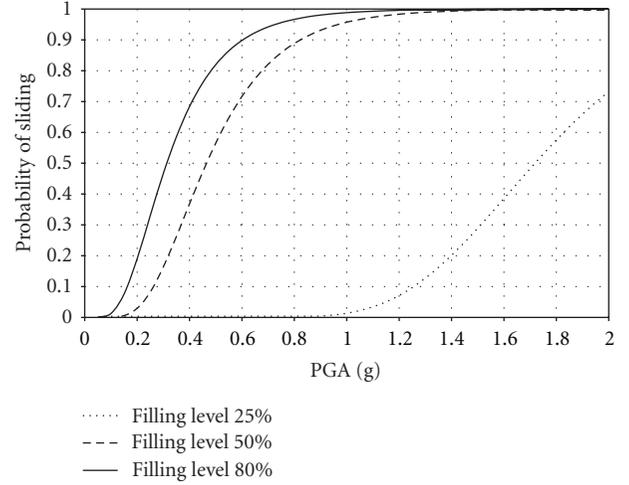


FIGURE 7: Probability of sliding for unanchored storage tank with  $V = 30000 \text{ m}^3$ , friction factor equal to 0.5 and filling level equal to 25%, 50% and 80%.

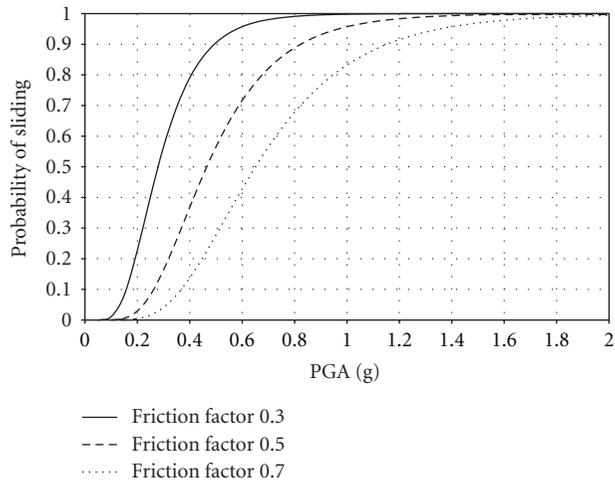


FIGURE 6: Probability of sliding for unanchored storage tank with  $V = 30000 \text{ m}^3$ , filling level equal to 50%, and friction factor equal to 0.7, 0.5 and 0.3.

enhanced to take account of the phenomenon without any relevant increase of the computational effort.

## 5. Equations of Motion

The equations of motion for the unanchored tank under a two-dimensional input ground motion, for nonsliding and sliding phase, can be expressed in the following matrix format:

$$[M]\{\ddot{z}\} + [B]\{\dot{z}\} + [K]\{z\} + \{F\} = -[M][r]\{\ddot{u}_g\}, \quad (10)$$

where  $[M]$ ,  $[B]$ , and  $[K]$  are the mass, damping, and stiffness matrices, respectively,  $[r]$  is the influence coefficient matrix,  $\{z\}$  is the displacement vector,  $\{F\}$  is the frictional force vector, and  $\{\ddot{u}_g\}$  is the earthquake acceleration vector.

In case of the nonsliding phase (rest) of the system the equations of the motion are reported as follows:

$$\begin{aligned} m_c \ddot{x}_c + b_c \dot{x}_c + k_c x_c &= -m_c \ddot{u}_{gx}, \\ m_i \ddot{x}_i + b_i \dot{x}_i + k_i x_i &= -m_i \ddot{u}_{gx}, \\ m_c \ddot{y}_c + b_c \dot{y}_c + k_c y_c &= -m_c \ddot{u}_{gy}, \\ m_i \ddot{y}_i + b_i \dot{y}_i + k_i y_i &= -m_i \ddot{u}_{gy}, \end{aligned} \quad (11)$$

where  $x_c$  and  $y_c$  are the components of displacements of convective masses relative to the base,  $x_i$  and  $y_i$  are the components of displacements of impulsive masses relative to the base,  $x_b$  and  $y_b$  are the component of displacement of the base relative to the ground, and  $\ddot{u}_{gx}$  and  $\ddot{u}_{gy}$  are the two horizontal components of ground acceleration. In this case  $m_c$  and  $m_i$  represent two simple oscillators, there is no coupling between two directions of motion, and the dimension of matrices is four, as reported in the following:

$$\begin{aligned} [M] &= \begin{bmatrix} m_c & 0 & 0 & 0 \\ 0 & m_i & 0 & 0 \\ 0 & 0 & m_c & 0 \\ 0 & 0 & 0 & m_i \end{bmatrix}, \\ [B] &= \begin{bmatrix} b_c & 0 & 0 & 0 \\ 0 & b_i & 0 & 0 \\ 0 & 0 & b_c & 0 \\ 0 & 0 & 0 & b_i \end{bmatrix}, \\ [K] &= \begin{bmatrix} k_c & 0 & 0 & 0 \\ 0 & k_i & 0 & 0 \\ 0 & 0 & k_c & 0 \\ 0 & 0 & 0 & k_i \end{bmatrix}, \end{aligned}$$

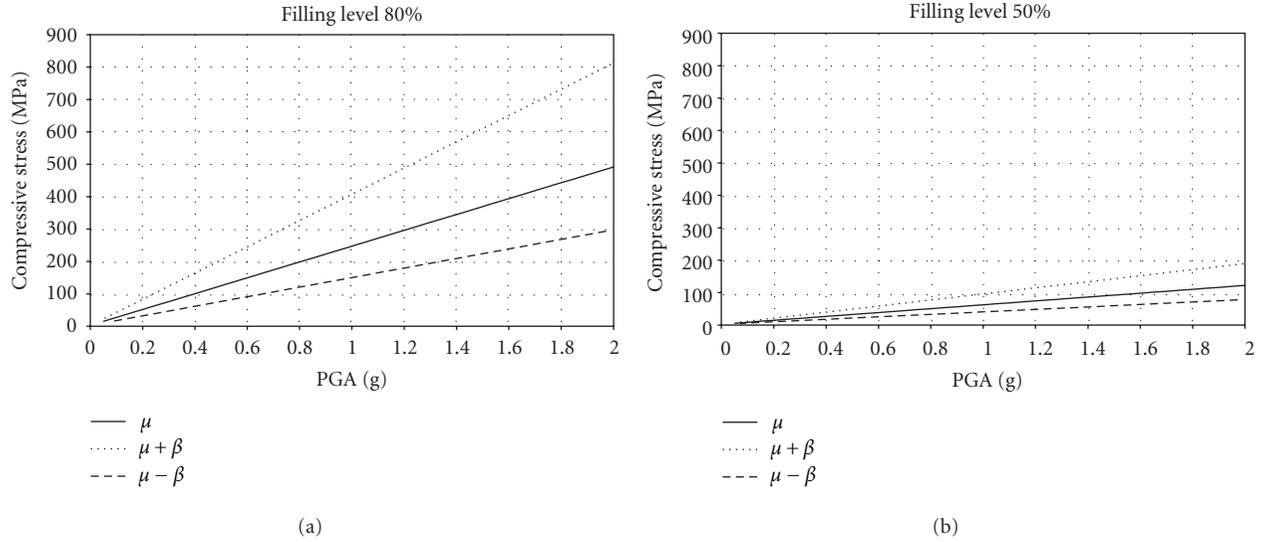


FIGURE 8: IDA curve of the compressive axial stress for the anchored storage tank with  $V = 30000 \text{ m}^3$  and filling level equal to 80% (a) and 50% (b).

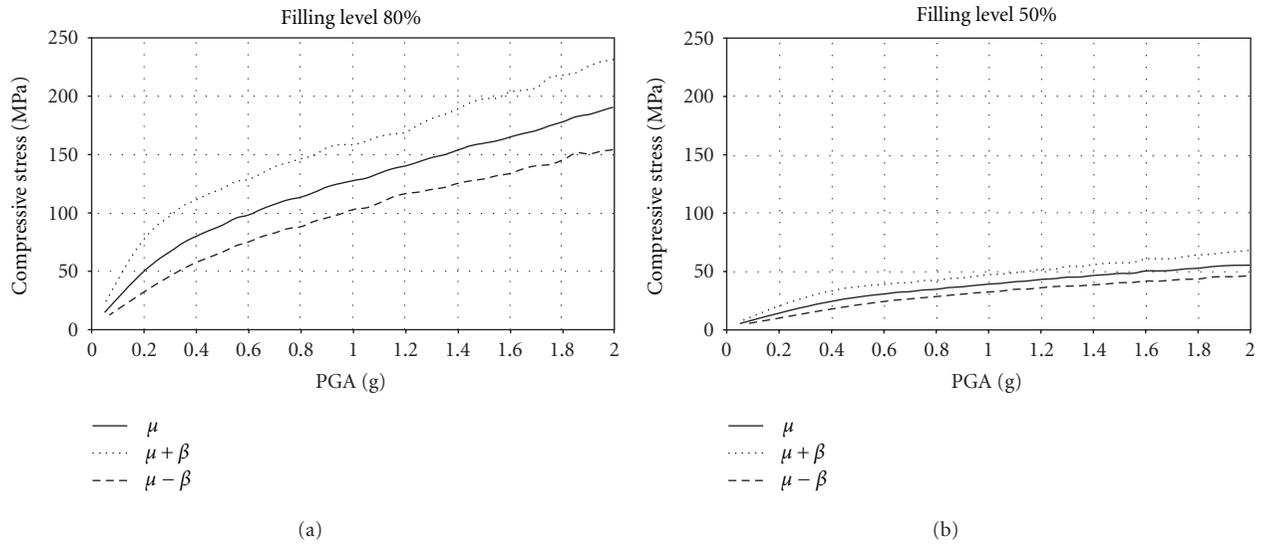


FIGURE 9: IDA curve of the compressive axial stress for the unanchored storage tank with  $V = 30000 \text{ m}^3$ , filling level equal to 80% (a) and 50% (b), and friction factor equal to 0.5.

$$\{z\} = \begin{Bmatrix} x_c \\ x_i \\ y_c \\ y_i \end{Bmatrix}, \quad \{F\} = \begin{Bmatrix} 0 \\ 0 \\ 0 \\ 0 \end{Bmatrix},$$

$$\{r\} = \begin{Bmatrix} 1 & 0 \\ 1 & 0 \\ 0 & 1 \\ 0 & 1 \end{Bmatrix}, \quad \{\ddot{u}\} = \begin{Bmatrix} \ddot{u}_{gx} \\ \ddot{u}_{gy} \end{Bmatrix}. \quad (12)$$

If the system slides, the motion depends on the dynamic equilibrium for convective and impulsive masses, and for the

system as a whole according to (13), the matrices are given in (14) and (15):

$$\begin{aligned} m_c \ddot{x}_c + m_c \ddot{x}_b + b_c \dot{x}_c + k_c x_c &= -m_c \ddot{u}_{gx}, \\ m_i \ddot{x}_i + m_i \ddot{x}_b + b_i \dot{x}_i + k_i x_i &= -m_i \ddot{u}_{gx}, \\ m_c \ddot{x}_c + m_i \ddot{x}_i + m_{\text{tot}} \ddot{x}_b - m_{\text{tot}} \mu g \cos(\alpha) &= -m_{\text{tot}} \ddot{u}_{gx}, \\ m_c \dot{y}_c + m_c \dot{y}_b + b_c \dot{y}_c + k_c y_c &= -m_c \ddot{u}_{gy}, \\ m_i \dot{y}_i + m_i \dot{y}_b + b_i \dot{y}_i + k_i y_i &= -m_i \ddot{u}_{gy}, \\ m_c \dot{y}_c + m_i \dot{y}_i + m_{\text{tot}} \dot{y}_b - m_{\text{tot}} \mu g \sin(\alpha) &= -m_{\text{tot}} \ddot{u}_{gy}. \end{aligned} \quad (13)$$

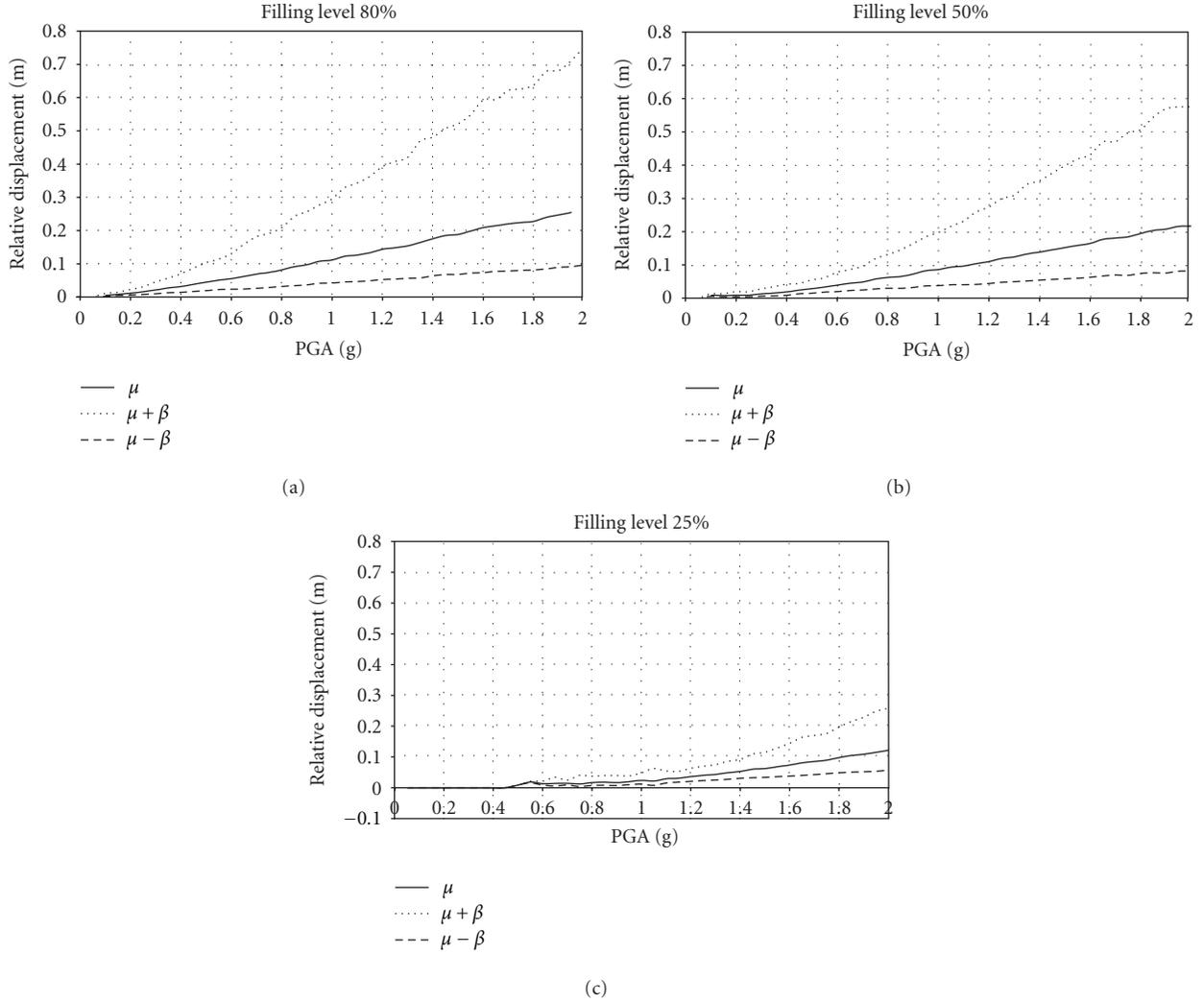


FIGURE 10: IDA for the sliding induced displacement for the bidirectional analysis of the unanchored storage tank with  $V = 30000 \text{ m}^3$ , filling level equal to 80% (a), 50% (b), and 25% (c), and friction factor equal to 0.3.

The Equation (14) shows that convective and impulsive masses are no more two simple oscillators, but there is an inertial coupling. As shown in (16), the sliding motion leads to an inertial coupling and to the coupling between two directions of motion ( $\alpha$  is the slope of the tangent to trajectory of motion, as shown in Figure 14):

$$[M] = \begin{bmatrix} m_c & 0 & m_c & 0 & 0 & 0 \\ 0 & m_i & m_i & 0 & 0 & 0 \\ m_c & m_i & m_{\text{tot}} & 0 & 0 & 0 \\ 0 & 0 & 0 & m_c & 0 & m_c \\ 0 & 0 & 0 & 0 & m_i & m_i \\ 0 & 0 & 0 & m_c & m_i & m_{\text{tot}} \end{bmatrix}, \quad (14)$$

$$\{z\} = \begin{Bmatrix} x_c \\ x_i \\ x_b \\ y_c \\ y_i \\ y_b \end{Bmatrix}, \quad \{\ddot{u}\} = \begin{Bmatrix} \ddot{u}_{gx} \\ \ddot{u}_{gy} \end{Bmatrix},$$

$$[B] = \text{diag}[b_c, b_i, 0, b_c, b_i, 0],$$

$$[K] = \text{diag}[k_c, k_i, 0, k_c, k_i, 0],$$

$$[r] = \begin{bmatrix} 0 & 0 & 1 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 1 \end{bmatrix}^T,$$

(15)

$$\{F\} = -m_{\text{tot}}\mu g \begin{Bmatrix} 0 \\ 0 \\ \cos(\alpha) \\ 0 \\ 0 \\ \sin(\alpha) \end{Bmatrix}. \quad (16)$$

## 6. Numerical Study

In the present section, some aspects related to the implementation of dynamic analyses solving the equations of motion

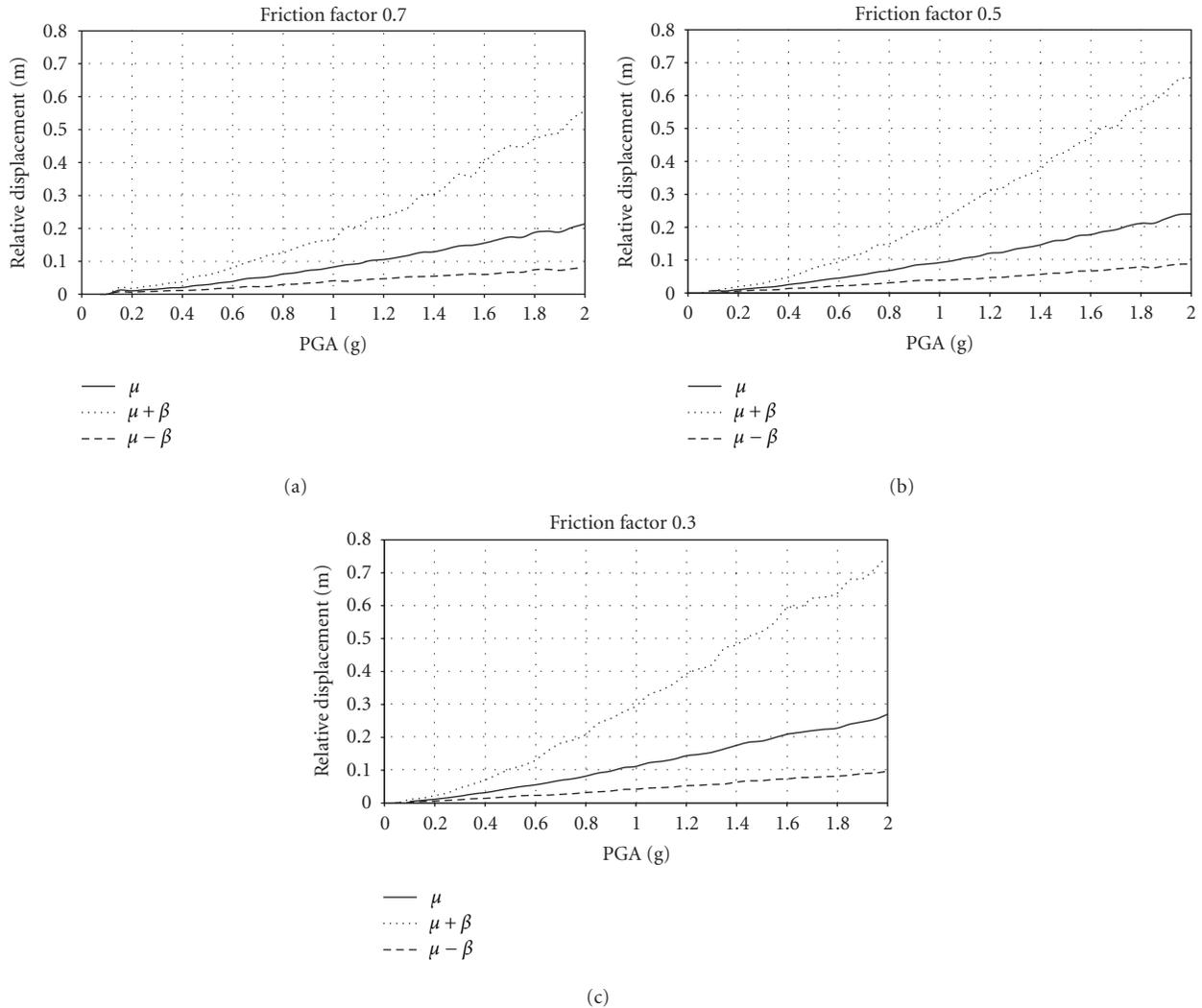


FIGURE 11: IDA for the sliding induced displacement for the bidirectional analysis of the unanchored storage tank with  $V = 30000 \text{ m}^3$ , filling level equal to 80%, and friction factor equal to 0.7 (a), 0.5 (b), and 0.3 (c).

previously reported are presented. In addition, some issues related to the application of incremental dynamic analyses are addressed. It is also tested the ability of the procedure to point out the influence of the structural and geometrical relevant parameters on the seismic demand of the tanks. Comparative analyses with full stress calculations made using refined FEM models able to take into account in direct manner the fluid-structure interaction have shown that the lumped mass models are able to give good estimation of global parameters [34, 35]. This is certainly of interest in the framework of seismic fragility numerical calculation. The incremental dynamic analysis (IDA) is adopted to analyze the seismic response of tanks under seismic loads [13] and characterize the related structural demand. IDA requires to analyze a model under a suite of ground motion records, all scaled to several levels of intensity. IDA is a procedure developed for frame structures where a sample of 20 ground

motions is considered sufficient to get unbiased estimation of the seismic demand if spectral acceleration is considered as the ground motion intensity measure. In order to extend the methodology to tank structures, the record sample size has to be computed again considering the coefficient of variation of the demand to the peak ground acceleration which is the IM herein. Assuming that a standard error affecting the estimate of the seismic demand equal to 0.1 is satisfactory, the size of the records sample is given by the squared ratio of the coefficient of variation of the structural response divided by target standard error [10]. In the case of unanchored storage tank, the minimum number of records to obtain a seismic demand with a standard error of 0.1 is estimated in 300 records. This number of records has been then assumed as a reference for all analyses. Their output is provided in terms of response curves that relate the selected demand measure (DM) to the ground motion intensity, generally given by a

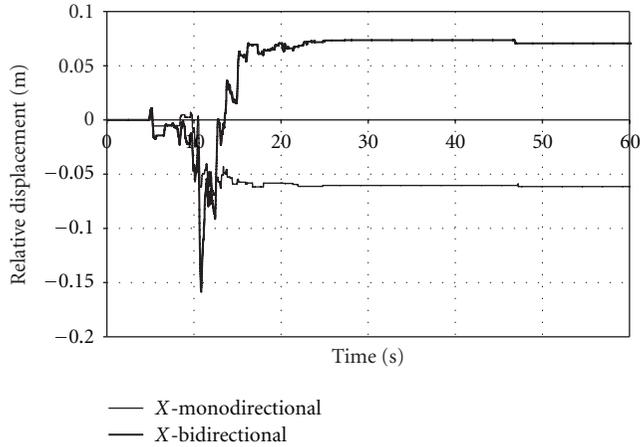


FIGURE 12: Comparison of unidirectional and bidirectional analyses, along  $x$  axis, for the unanchored storage tank with  $V = 5000 \text{ m}^3$ , filling level equal to 80%, and friction factor equal to 0.3 subjected to the 000187-Iran earthquake.

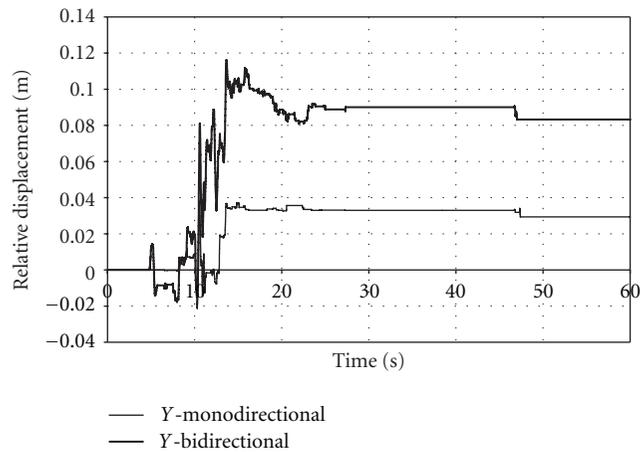


FIGURE 13: Comparison of unidirectional and bidirectional analyses, along  $y$  axis, for the unanchored storage tank with  $V = 5000 \text{ m}^3$ , filling level equal to 80%, and friction factor equal to 0.3 subjected to the 000187-Iran earthquake.

scalar intensity measure (IM). In the following, IDA analyses taking into account generalized, two-dimensional ground motion records are presented.

Figure 2 reports the main steps of the procedure. It is worth noting that the seismic demand at a selected ground motion intensity level is obtained scaling the peak ground acceleration (PGA) of the real record. The scaling factor  $\chi$  varies to let PGA ranges between 0.05g and 2.00g. In order to assess the influence on seismic demand of geometrical and functional parameters, different volume capacities were considered ( $5000 \text{ m}^3$  and  $30000 \text{ m}^3$ ).

Furthermore, three different filling conditions were analyzed (25%–50%–80%), and anchored and unanchored configuration of the tanks were used. In the last case of unanchored tanks, a set of friction coefficients,  $\mu$ , were used.

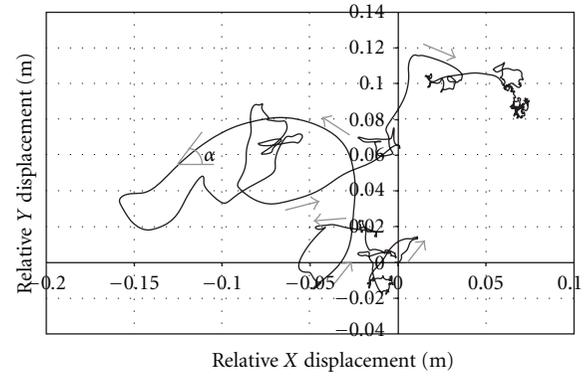


FIGURE 14: Trajectory of unanchored storage tank with  $V = 5000 \text{ m}^3$ , filling level equal to 80%, and friction factor equal to 0.3 subjected to the 000187-Iran earthquake.

The solution of the equations presented in the previous sections was achieved by a computer code using the Wilson theta method [36]. The flow chart of the procedure able to perform the time history analysis for storage tanks is shown in Figure 3.

Firstly the check of the base velocity at time  $t$  has to be performed: whenever it is zero, the full knowledge of the phase of motion requires an additional check of the value of the base shear. If the base shear at time  $t$  is lower than the limit frictional value, the motion is rest (nosliding) type in  $[t, t + \Delta t]$ ; otherwise the system, in the same time interval, slides.

If the system does not slide, checking the value of the base shear at  $t + \Delta t$  is also needed. Whenever it is lower than  $F_{lim}$ , the integration of the equation of dynamic equilibrium for the next  $\Delta t$  is possible; otherwise the computation of the time  $t^*$ , intermediate between  $t$  and  $t + \Delta t$  and corresponding to base shear that reaches the limit value, is required. In other words, between  $t$  and  $t^*$ , the tank does not slide while between  $t^*$  and  $t + \Delta t$  slides. In order to compute the intermediate time  $t^*$ , a linear variation of base shear is assumed in the time interval  $[t, t + \Delta t]$ .

If at time  $t$  the velocity is not equal to zero, the system is in sliding phase and another check at  $t + \Delta t$ , is necessary; when the velocity has changed its signum between  $t$  and  $t + \Delta t$  the computation of  $t^*$  corresponding to the point zero velocity is performed. Otherwise the integration of the equation of dynamic equilibrium for the next  $\Delta t$  can take place. The time  $t^*$  is calculated again via a linear variation of velocity. Between  $t$  and  $t^*$ , the system slides but the type of motion between  $t^*$  and  $t + \Delta t$  depends on the base shear at  $t^*$ : if it is lower than the limit, then the system is in no sliding (rest) between  $t^*$  and  $t + \Delta t$ ; otherwise it is sliding. The number of equations and the number of unknowns depend on the type of motion at time  $t$  (sliding or rest).

**6.1. Results and Discussion.** The parameters of the investigated tanks are described in Table 1,  $t_b$  is the base plate thickness and  $t_s$  is the shell thickness,  $H$  is the height of liquid in the tank,  $\rho_s$  and  $\rho_l$  are the specific weights of steel and

TABLE 1: Tank parameters.

Volume (m <sup>3</sup> )	$R$ (m)	$h$ (tank) (m)	Filling (%)	$H$ liquid (m)	$\rho_s$ (kgm <sup>-3</sup> )	$\rho_l$ (kgm <sup>-3</sup> )	$E$ (GPa)	$t_b$ (m)	$t_s$ (m)
5000	12.25	10.8	25%	2.700	7850	1000	210	0.008	0.007
			50%	5.400	7850	1000	210	0.008	0.007
			80%	8.640	7850	1000	210	0.008	0.007
30000	22.75	18.5	25%	4.625	7850	1000	210	0.008	0.007
			50%	9.250	7850	1000	210	0.008	0.007
			80%	14.800	7850	1000	210	0.008	0.007

TABLE 2: Subset of ground motion records used for demand analysis.

Station	Earthquake	Data	Nation	Mw	Epicentral distance (km)	PGA*	Type of soil
000113	Friuli (aftershock)	11/09/1976	Italy	5.3	21	0.17g	Stiff
000120	Friuli (aftershock)	11/09/1976	Italy	5.5	15	0.09g	Stiff
000123	Friuli (aftershock)	11/09/1976	Italy	5.5	15	0.23g	Stiff
000159	Friuli (aftershock)	16/09/1977	Italy	5.4	7	0.24g	Stiff
000196	Montenegro	15/04/1979	Yugoslavia	6.9	25	0.45g	Stiff
000197	Montenegro	15/04/1979	Yugoslavia	6.9	24	0.29g	Stiff
000202	Montenegro	15/04/1979	Yugoslavia	6.9	56	0.06g	Stiff
000239	Dursunbey	18/07/1979	Turkey	5.3	6	0.29g	Stiff
000244	Valnerina	19/09/1979	Italy	5.8	39	0.04g	Stiff
000291	Campano Lucano	23/11/1980	Italy	6.9	16	0.18g	Stiff
000295	Campano Lucano	23/11/1980	Italy	6.9	58	0.05g	Stiff
000336	Preveza	10/03/1981	Greece	5.4	28	0.14g	Stiff
000376	Lazio Abruzzo	07/05/1984	Italy	5.9	69	0.02g	Stiff
000536	Erzincan	13/03/1992	Turkey	6.6	65	0.03g	Stiff
000584	South Aegean	23/05/1994	Greece	6.1	64	0.06g	Stiff
000595	Umbria Marche	26/09/1997	Italy	5.7	25	0.05g	Stiff
000947	Potenza	05/05/1990	Italy	5.8	28	0.10g	Stiff
001862	Aetolia	18/05/1988	Greece	5.3	20	0.06g	Stiff
001863	Aetolia	22/05/1988	Greece	5.4	21	0.04g	Stiff
006264	South Iceland	17/6/2000	Iceland	6.5	52	0.07g	Stiff

\* $g = 9.81 \text{ m/sec}^2$ .

liquid, respectively, and  $E$  is the modulus of elasticity of the tank structure. In the parametric analysis,  $\mu$  (friction factor) varies from 0.3 to 0.7. For the analysis presented in the paper, a suitable set of 300 ground motion records are used. Selected earthquake ground motion records are all stiff soil records to avoid specific problems related to site effects.

Table 2 reports some data concerning a subset of the selected ground motion records and gives an overview of values of the magnitude and distance. All the records herein employed come from the European Strong Motion Database (<http://www.isesd.hi.is/>) and can be easily retrieved from there. All the recorded components are used for tanks under generalized excitation. As one-dimensional analyses are concerned, the component affected by the peak ground acceleration is assumed as reference.

Figure 4 shows the demand curve in terms of axial compressive stress [MPa] obtained from the bidirectional

model and computed according to the following equation after AWWA D100-96:

$$\sigma_c = \left( w_t + \frac{1.273M}{D^2} \right) \frac{1}{1000t_s} [\text{MPa}], \quad (17)$$

where  $t_s$  is the thickness of the tank wall,  $D$  is the tank diameter,  $M$  is the overturning moment,  $w_s$  is the weight of the tank shell and portion of the roof reacting on the shell, determined as  $w_t = W_s/\pi D + w_{rs}$ , where  $w_{rs}$  is the roof load acting on the shell and  $W_s$  is total weight of tank shell.

Curves include media and  $1\beta$  bounds ( $\beta$  is the standard deviation of the logarithms of the demand).

The base-displacement demand curve depending on the  $\text{PGA}_{g.m.}$  for the set of ground motions is shown in Figure 5; again the median and  $\pm 1\beta$  curves are given. This curve represents, point to point, the median of probability distribution of the parameter which is investigated. In the

case of the rigid displacement, the probability distribution is conditioned not only to  $PGA_{g,m}$  but also to sliding motion, because not all records with an assigned value of  $PGA_{g,m}$  cause the sliding motion.

Figures 6 and 7 summarize the probability of sliding motion at the different  $PGA_{g,m}$  levels depending on the filling level and the friction factor  $\mu$ . As could be expected, for a given value of filling level and  $\mu$ , the higher the  $PGA_{g,m}$ , the higher the probability of sliding motion. At same level of  $PGA_{g,m}$ , the probability of sliding motion increases as the filling level increases; similarly it is larger when the friction factor decreases.

Figures 8 and 9 show the influence of filling level on the compressive stress demand, for anchored and unanchored tanks respectively.

Figures 10 and 11 show the influence of filling level and friction factor on the rigid displacement, respectively. For the same value of friction factor, the rigid displacement increases when the filling level increases. The variability of filling level has influence both on the mean value and on the standard deviation.

The variability of friction factor, as shown in Figure 11, has the same influence; in fact for the same value of filling level the rigid displacement increases when friction factor decreases.

Figures 12 and 13 report some numerical results able to demonstrate the effects of contemporary presence of the two horizontal components; thus a comparison between unidirectional and bidirectional results in terms of base displacement can be done. These two analyses are not equivalent; in fact, the maximum base displacement in  $X$  direction for the one-dimensional analysis is 0.069 m, while when the second component is taken into consideration, the displacement goes to 0.158 m.

The same effect can be recognized in terms of displacements along  $Y$  direction, which are characterized by a maximum equal to 0.038 m when one-dimensional analysis is performed and equal to 0.116 m when the two components are introduced. In Figure 14, as an example, the trajectory of the tank is plotted. It represents the displacement of the geometric centroid of the tank subjected to the 000187-Iran earthquake.

## 7. Conclusions

The paper deals with the estimation of the seismic demand of steel tanks under generalized ground motion. It is a complex topic, since structural response of such systems is strongly influenced by fluid-structure interactions.

Furthermore, full stress analysis can be certainly addressed as an effective and accurate tool but is very demanding from the computational point of view. In fact, storage facilities are generally characterized by large number of tanks with different serviceability conditions. This is the reason why simplified methods can be addressed as interesting for seismic vulnerability assessment and plant protection from natural hazards.

Available simplified models for steel tanks dynamic analysis have been reviewed and discussed in detail from

a computational point of view. An extensive parametric analysis has been carried out in order to assess the ability of the procedure to point out the main mechanical phenomena involved in the seismic response. In this way, helpful suggestions for the extension of IDA to such structural components and systems have been provided. The relevance of generalized ground motion and the dimension of the strong motion data set are certainly helpful results in the context of fragility assessment study and risk management of hazardous materials storage facilities.

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