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SEISMIC FRAGILITY ANALYSIS OF STEEL STORAGE TANKS

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Abstract. *Earthquakes can cause significant damages to industrial liquid storage tanks resulting in losses of functionality, fires or environmental contamination due to the leakage of hazardous chemicals. Typical damages of ground supported tanks during past earthquakes were in the form of cracking at the corner of the bottom plate and compression buckling of tank wall due to uplift, sliding of the base, anchorage failure, sloshing damage around the roof, failure of piping systems and plastic deformation of base plate. Liquid tanks can be also located at some elevated positions due to operational purposes. This makes them susceptible to collapse due to increased base shears and overturning moments. The seismic response of elevated tanks has been widely investigated in the past considering different materials and configurations of support structures. This paper addresses the problem of elevated tanks with particular attention focused on the steel storage tanks resting on short RC columns. The vulnerability of a real example of elevated tanks is assessed though the probabilistic analysis performed using non-linear lumped mass models. Consequently different fragility curves are built for identifying the most important damage states and calculating the corresponding probability of occurrence. The results show how the support structure, especially when composed by RC columns, is the most influencing one, whereas the remaining damage states have a limited influence.*

1 INTRODUCTION

Earthquakes can cause severe damages to the tank structures and other facilities in industrial sites resulting in losses of functionality, fires or environmental contamination due to the leakage of hazardous chemicals [1], [2], [3], [4], [5], [6], [7], [8], [9]. Above ground liquid storage tanks have suffered serious damages during past earthquakes. These typical damages were in the form of cracking at the corner of the bottom plate and compression buckling of tank wall (elephant's foot buckling) due to uplift, sliding of the base, anchorage failure, sloshing damage around the roof, failure of piping systems and plastic deformation of base plate.

Liquid tanks can also be elevated due to operational purposes. Steel storage tanks upon reinforced concrete columns represent a typical structural configuration. Moreover, in case of reinforced concrete supports, the high lateral stiffness may induce premature failure in the columns, as shown in recent seismic events. For example, during Izmit earthquake (1999) in Turkey, a group of elevated tanks for the storage of liquefied oxygen were seriously damaged or collapsed [10], [11], [12], [13].

On the basis of the above-depicted framework, this paper addresses the problem of the vulnerability assessment of elevated tanks with particular attention focused on the steel storage tanks resting on short RC columns. The seismic response of one of the liquefied oxygen storage steel tanks collapsed during the 1999 Izmit earthquake due to premature failure of columns has been analyzed under seismic excitation using a 3D model implemented in OpenSEES [14].

Consequently, after a brief review of existing methods for the evaluation of fragility curves and the definition of damage limit states for steel storage tanks, the vulnerability analysis of the selected case study has been assessed using two different methods, namely Cloud method and Increment Dynamic Analysis highlighting the most frequent damage conditions for the tank under seismic action [15].

2 DAMAGE STATES OF STORAGE TANKS

The seismic vulnerability of a structure is generally expressed as probability of exceeding one of more structural limit states. Existing data concerning post earthquake damage observations for steel storage tanks have been collected in order to propose the limit state classifications of equipment response. In fact, according to HAZUS damage state list [16], the effects of seismic actions have been related to structural damage and its reparability. Possible damage limits for steel storage tanks are reassumed in Table 1.

#	Failure mode
1	Overturning
2	Elastic buckling
3	Sliding
4	Elasto-plastic buckling
5	Tank roof damage
6	Uplift

Table 1: Possible failure modes for tanks

These failure modes are usually collected in a limited number of damage states, as reported in Table 2 [16].

In this work, the attention is paid on elevated tanks, whose typical configuration does not include any floating roof. In order to estimate the seismic vulnerability and to derive fragility curves, extensive (buckling) and complete damage states (complete collapse due to the failure of the support system) have been here considered.

#	Damage state	Description
1	No damage	---
2	Slight damage	Roof and pipes damages, sliding without leakage
3	Moderate damage	Elastic buckling, yielding of tank wall and uplift without leakage
4	Extensive damage	Elastic buckling, uplift, pipes detachment with leakage
5	Complete damage	Complete damage (boilover, injections, complete LOC, total collapse)

Table 2: Damage states for tanks

The damage state associated to instability phenomena in the tank wall can be quantified with the level of stress that activates buckling. Three distinct buckling phenomena have been observed during past seismic events: Diamond Shape Buckling (DSB), Elephant's Foot Buckling (EFB) and Secondary Buckling (SB) [17]. An example of both the buckling phenomena is illustrated in Figure 1.

Figure 1: EFB and DSB observed after the 2012 Emilia Earthquake in Italy [8]

DSB is due to excessive meridional stresses so that to induce lateral instability of the wall in the elastic field. This form of buckling has been observed in those parts of the shell where the thickness is reduced with respect to the thickness of the base and/or the internal pressure (which has a stabilizing effect) is also reduced with respect to the maximum value attained at the base. EFB normally occurs close to the base of the tank, due to a combination of vertical compressive stresses and tensile hoop stresses inducing an inelastic biaxial state of stress.

These two buckling types are explicitly mentioned in current codes as Eurocode 8 [18] and New Zealand guidelines [19] and implicitly accounted for in the API formulation [20]. For example, Eurocode 8 and New Zealand guidelines suggest the Rotter [21] formula to compute the buckling capacity with respect to elastic–plastic buckling:

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

where $\sigma_{c1} = 0.605 E_w t_w R$ 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 $r = R/(400 t_w)$.

In addition to the well-known elephant's foot and diamond shape buckling modes, a third kind of buckling, due to external pressure and cavitation, is mentioned by Rammerstofer et al. [22] and confirmed also by observations on real tanks after earthquakes. However, this buckling mode is not covered by current codes, and until now, no empirical formula associated to it exists in literature.

Failure conditions in the support structure can be usually treated using common approaches adopted for civil structures. For example, it is frequent to find RC columns as support structure. In this case, the ultimate capacity can be calculated in accordance to the current codes for the assessment of existing RC structures, both for ductile and fragile mechanisms [23].

3 FRAGILITY ASSESSMENT METHODS

In this paper, a fragility function, $P[D_{EDP}>LS|IM]$ is developed for the liquid storage tanks to investigate the damage likely to occur during a seismic event. This function represents the probability of the response, D_{EDP}, of the selected engineering demand parameter, EDP, exceeding a selected structural limit state, LS, for a specific intensity measure, IM, of seismic excitation.

Different methods to build fragility curves have been already proposed in literature, considering or not the randomness of the structure characteristics [15]. The most common used is the Cloud method. This method implements non-linear dynamic analyses through (linear) regression-based probabilistic model [24]. When the seismic demand and the structural limit states are assumed to follow a lognormal distribution, the probability of exceeding a specific damage state can be estimated with the lognormal cumulative distribution function, given as:

$$P\left[D_{EDP} > LS \mid IM\right] = 1 - \Phi\left(\frac{\ln\left(LS_m\right) - \ln\left(D_m\right)}{\sqrt{\beta_{d|IM}^2 + \beta_{LS}^2}}\right) \tag{2}$$

where $\Phi(\cdot)$ is the standard normal cumulative distribution function, LS_m is the median estimate of the structural limit state, D_m is the median estimate of the demand, $\beta_{d|IM}$ is the dispersion of the demand conditioned on the IM, and β_{LS} is the dispersion of the structural limit state.

The estimate of the median demand can be predicted by a power function:

$$D_m = a\left(IM\right)^b \tag{3}$$

where a and b are regression coefficients based on the collection of d_i and IM_i from the time-history seismic analyses of the analyzed tank and from the selected a suite of n ground motions. The dispersion of the demand conditioned on the IM can also be estimated from the regression analysis of the seismic demands:

$$\beta_{d|IM} = \sqrt{\frac{\sum_{i=1}^{n}\left[\ln\left(d_i\right) - \ln\left(aIM_i^b\right)\right]^2}{n-2}} \tag{4}$$

The Cloud method is very attractive because it allows deriving closed-form solutions of the fragility curves. However, the power-law form of the demand is, at best, only a good estimate of the behavior of a structure under earthquake excitation in the interval of values where the locally linear fit is made. In addition, the dispersion is considered as constant, when in reality there is a clear dependency of the level of *IM* analyzed.

In order to correctly incorporate the above-mentioned variability of mean and dispersion with *IM*, many alternative methods have been proposed. One of them is the Incremental Dynamic Analysis (IDA) method that was initially derived only for collapse condition [25] but can be adopted for different damage levels. This method produces a set of *IM* values associated with the onset of collapse for each ground motion. Fragility function parameters can be estimated by taking logarithms of each ground motion's intensity measure value associated with onset of collapse or damage, and computing their mean and standard deviation:

$$\ln \hat{\mu} = \frac{1}{n}\sum_{i=1}^{n} \ln IM_i \tag{5}$$

$$\hat{\beta} = \sqrt{\frac{1}{n-1}\sum_{i=1}^{n}\left(\ln\left(IM_i / \mu\right)\right)^2} \tag{6}$$

where n is the number of ground motions considered, IM_i is the *IM* value associated with onset of collapse or damage for the i_{th} ground motion.

Consequently, given the limit state, the *IM* is now a random variable, whose probability of being lower than given *IM* is the following that also represents the analytical expression of the fragility curve:

$$P[D_{EDP} > LS \mid IM] = \Phi\left(\frac{\ln\left(IM / \hat{\mu}\right)}{\hat{\beta}}\right) \tag{7}$$

Other possible methods can be derived from the reliability theory, as for example the Effective Fragility Analysis, already proposed for the vulnerability assessment civil and industrial structures [26], [27], [28]. This method makes use of the Response Surface technique.

In this work, Cloud method and IDA will be used to derive fragility curves of the elevated tank described in section 4.

4 PROBABILISTIC SEISMIC RESPONSE ANALYSIS

4.1 Numerical model of elevated tanks

In this paper, the probabilistic seismic analysis of elevated liquefied gas storage tanks at the Habas facility in Izmit, Turkey is presented [10], [12]. The two tanks containing 80% full liquid oxygen were collapsed during the Kocaeli Earthquake in 1999 while the tank and supporting structure containing 25% full liquid nitrogen was undamaged except for some hairline cracks in the columns, as shown in Figure 2.

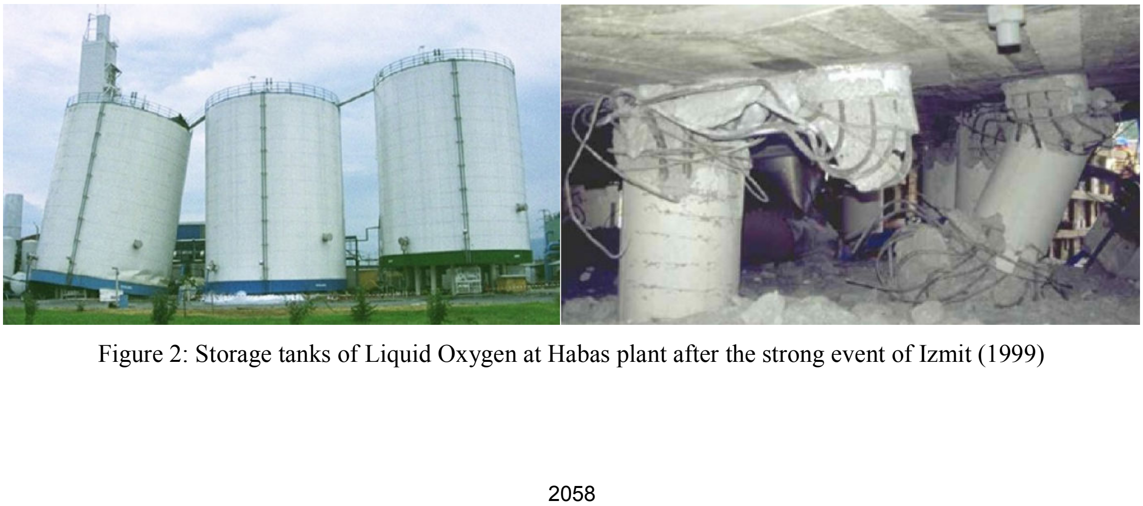

Figure 2: Storage tanks of Liquid Oxygen at Habas plant after the strong event of Izmit (1999)

These tanks were built in 1995 and each tank consisted of two concentric stainless steel shells, one with an outside diameter of 14.6 m and the other with an outside diameter of 12.8 m. The gap between the shells is filled with insulation. Both shells were supported on a 14.6 m-diameter and 1.07 m-thick reinforced concrete slab that was in turn supported by sixteen 500 mm-diameter reinforced concrete columns. Each column was 2.5 m in height and reinforced with sixteen 16 mm-diameter longitudinal bars and 8 mm-diameter ties at 100 mm on the center. According to [10], the concrete used for the columns and the bottom slab was of class C30/37, whereas the steel bars had a yielding strength of 430 MPa. The density of the oxygen was 11.50 kN/m^3.

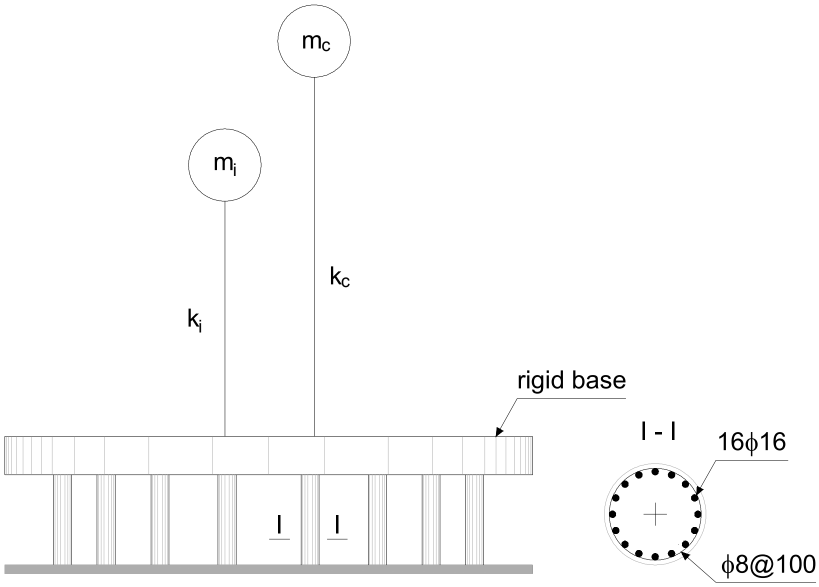

Figure 3: Numerical model of 3D elevated tanks

Based on the typical details taken from the examined elevated tank, 3D numerical model is generated using the finite element platform OpenSEES [14]. The hydrodynamic pressure acting on the tank wall during earthquake loading can be easily performed using the simplified model shown in Figure 3, in which the liquid mass is lumped and subdivided in two components: impulsive and convective masses named m_i and m_c, respectively [29], [30], [31]. The impulsive and convective masses are connected to the tank wall by springs of stiffness k_i and k_c. The RC base is assumed to be rigid and the RC supporting columns are modeled using 3D nonlinear beam–column elements with fiber-defined cross-sections. Concrete is modeled according to Kent-Park model, which include the confinement effect of stirrups whereas the steel bars are modeled using the Menegotto-Pinto model.

The seismic response of the elevated tank is evaluated by using a set of time-history analyses using the nonlinear 3D model of Figure 3. In this respect, the system is subjected to 30 natural records selected from PEER Strong Ground Motion Database. They have been identified according to the following hazard criteria:

- Magnitude: $5 < M < 7$
- Distance from the fault: $0 < d < 20$ km
- S-waves velocity between 360 m/s and 760 m/s

The response spectra of all un-scaled records are shown in Figure 4 together with the elastic spectrum imposed by the EN1998:1 [32]. The design ground acceleration of 0.4g is selected for a soil condition A.

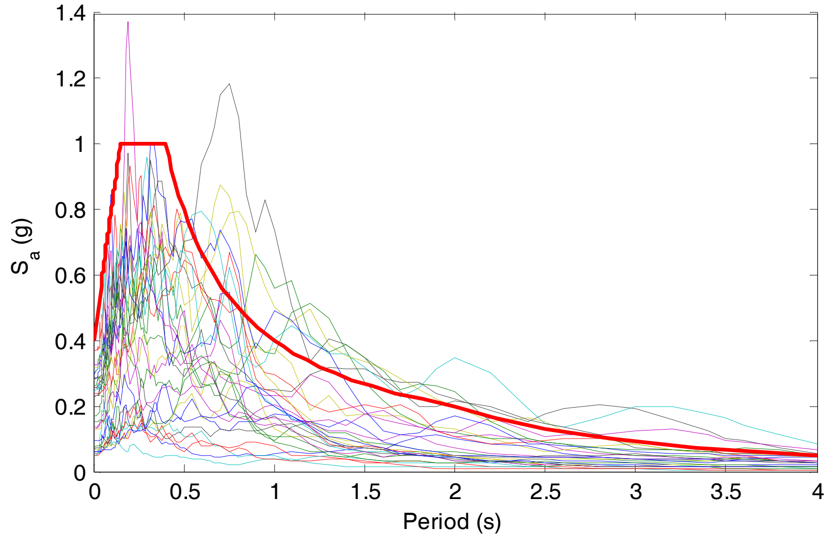

Figure 4: Response spectra of the 30 un-scaled accelerograms

4.2 Probabilistic seismic response analysis of the case study

Concerning the Cloud method, for each ground motion a series of non-linear time-history analyses are performed, where the peak value of seismic response for each accelerogram is recorded. For example, Figure 5 shows the maximum lateral displacement, δ, of the tank base for each of the 30 accelerograms. In this work, the selected demand parameters (EDPs) include the drift ratio of supporting columns, δ/H, and the meridional stress in the tank wall, σ_z.

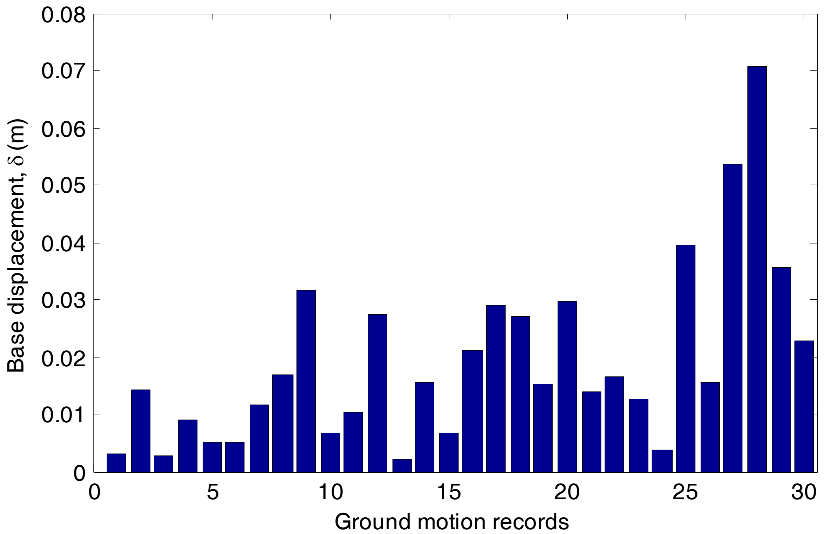

Figure 5: Peak displacement response of tank base for each selected ground motion

Given the high axial stiffness of columns, the drift well approximates the chord rotation that is $\theta \approx \delta/H$. This latter is usually adopted as a parameter for the seismic assessment of ductile mechanism of columns [23]. The meridional stress has been calculated according to the approximated formula provided by the standard API 650 [20].

A linear regression analysis has been performed to determine the regression coefficients of the probabilistic seismic demand model. The coefficient a and b have been identified using a log-log representation of the EDP-PGA relationship (Figure 6). The fragility curves can be developed using Eq. (2).

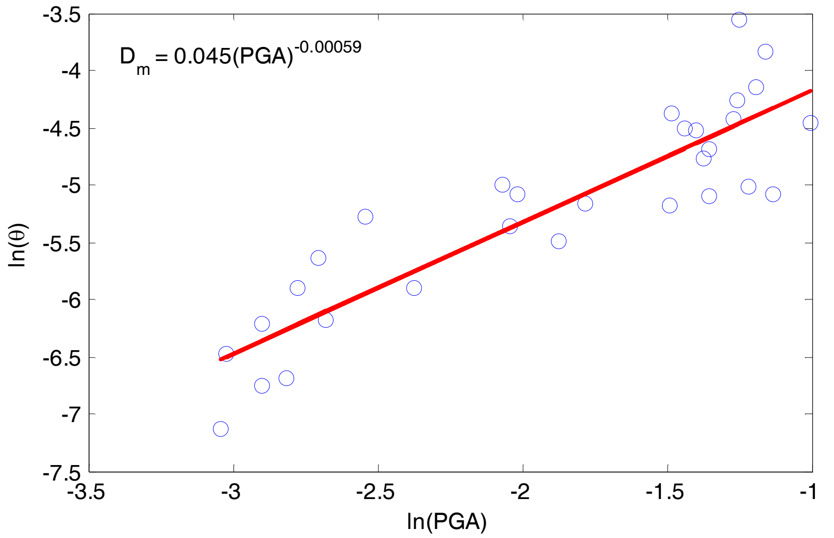

Figure 6: Linear regression analysis for peak drift ratio of supporting columns

With IDA, the ground motions are scaled to a given LS. This process produces a set of PGA values associated with the onset of the LS. Fragility function parameters for each EDP are then estimated from these values by using Eq. (5-6). Figure 7 shows the IDA results for supporting columns in terms of maximum column drift ratio.

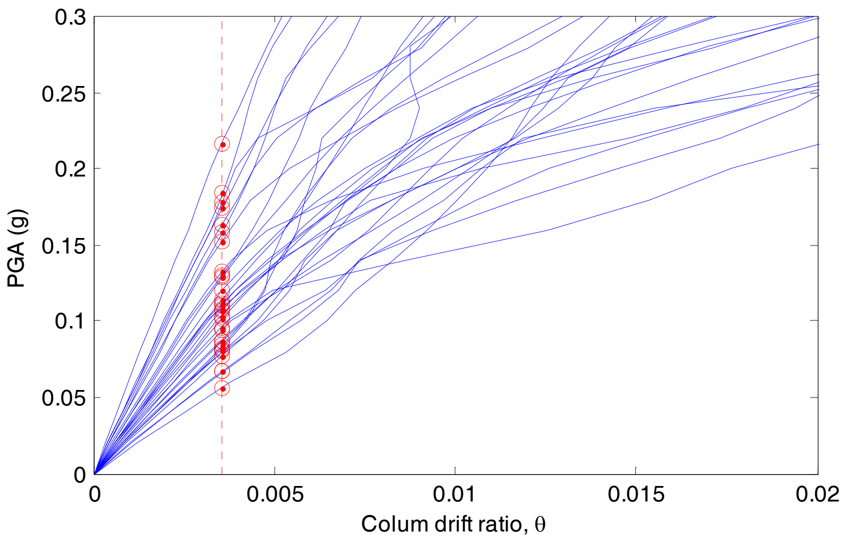

Figure 7: IDA results for peak drift ratio of supporting columns

4.3 Fragility curves estimation

According to the methods presented in section 3, fragility curves of the tank as shown in Figure 8 have been constructed for the several above-described damage states. However, given that the prevalent damage condition is represented by the damage in the RC columns for excessive lateral displacement of the tank base, the fragilities have been calculated under the hypothesis that the other failure modes due to the buckling phenomenon would be excluded. This has also been demonstrated in the reality where the collapse was uniquely caused by the failure of the columns. In this respect, the ultimate capacity of the columns is evaluated in terms of chord rotation θ_u under the hypothesis of ductile mechanism. The ultimate chord rotation can be evaluated according to EN1998:3 [23] as follows:

$$\theta_u = \frac{1}{\gamma_{el}}\left(\theta_y + \left(\phi_u - \phi_y\right)L_{pl}\left(1 - \frac{0.5L_{pl}}{L_V}\right)\right) \tag{8}$$

where γ_{el} = 1.5 is the safety coefficient, $\theta_y = \phi_y L_v/3$ is the chord rotation at yielding, ϕ_u and ϕ_y are the ultimate and yielding section curvatures, L_{pl} is the plastic hinge height and L_v is the distance between the zero moment point and one of the two edges.

Given that the model of Figure 3 includes the overturning effect of the tank, and thus the variation of axial force in the columns during the earthquake, θ_u = 0.0036 has been calculated as the minimum value obtained during the time-history analyses.

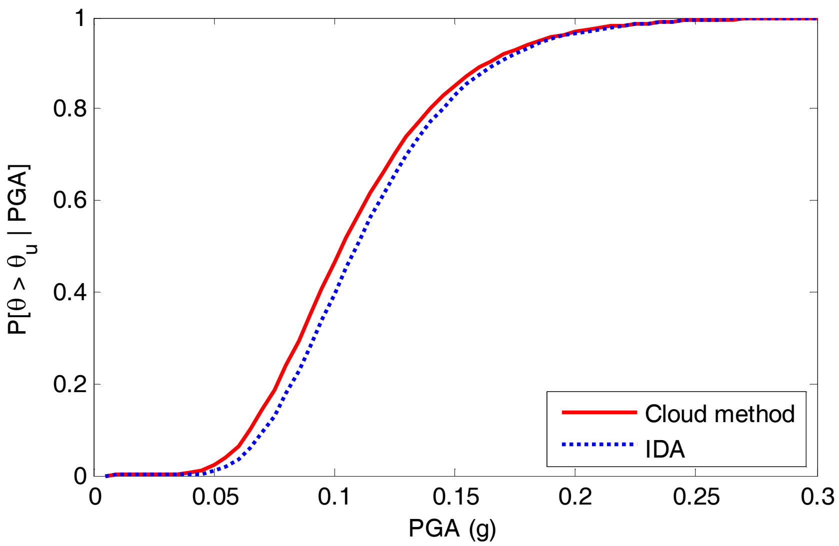

Figure 8: Fragility functions obtained by Cloud method and IDA for ductility of columns

The fragility curves obtained based on the Cloud and the IDA methods are presented in Figure 8. For a given PGA, the probability of exceedance of the ductile mechanism in the support columns is similarly predicted by the two methods. However, IDA method appears to be slightly more conservative. It can be noticed that for a PGA = 0.2g, the probability of failure is approximately equal to 95%. This is a clear demonstration of the high seismic vulnerability of the tank, as dramatically demonstrated during the 1999 Izmit earthquake [10].

For completeness, the fragility curve relative to the failure mode of the tank due to elephant's foot buckling has been calculated. As previously mentioned, because an eventual collapse due to the failure of the columns avoids any buckling phenomena in the tank wall, the fragility curve has been built assuming for the columns a linear elastic behavior whose stiffness has been calculated according to the secant modulus of the concrete. The curve is illustrated in Figure 9 compared with the fragility curve for columns collapse condition.

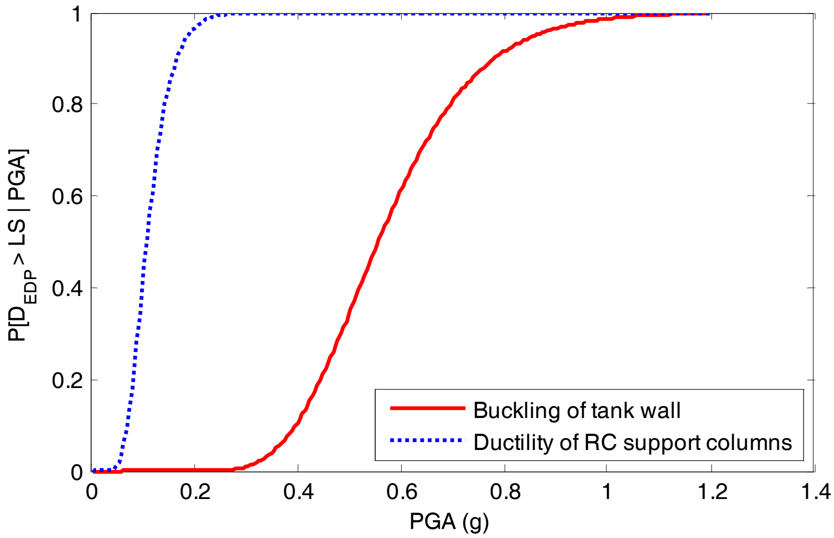

Figure 9: Fragility functions obtained by IDA for ductility of columns and buckling of tank wall

5 CONCLUSIONS

Typical damages of above-ground and elevated tanks during past earthquakes were in the form of cracking at the corner of the bottom plate and compression buckling of tank wall due to uplift, sliding of the base, anchorage failure, sloshing damage around the roof, failure of piping systems and plastic deformation of base plate. The high vulnerability of this typical industrial component is thus evident. A typical tool for a quantitative evaluation of the vulnerability of a structure is the fragility analysis, whose outcome is represented by evaluation of the probability of exceeding a certain damage state for a given intensity measure (e.g. the PGA).

In this respect, the present paper addressed the problem of steel storage tanks. The seismic vulnerability has been assessed though the probabilistic response analysis (Cloud method and IDA) performed using non-linear lumped mass models. As case study, an emblematic example of elevated tank collapsed during the Kocaeli Earthquake in 1999 at Habas pharmaceutics plant in Turkey was considered. Different fragility curves were built identifying the most important damage states and calculating the corresponding probability of occurrence. The results have shown that the use of different methods provides similar results. In particular, it was evidenced that failure in support structure columns is the most influencing one. This is fully in accordance with the real collapse mode.

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