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SEISMIC VULNERABILITY OF STANDARDISED INDUSTRIAL COMPONENTS: APPLICATION TO OIL STORAGE TANKS

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SUMMARY

Industrial risk requires taking into account natural hazards including earthquakes. In the Quantitative Risk Analysis (QRA) perspective seismic failure may be added to system faulty events. Fragility analysis of industrial components presents some aspects that requires the development of specific reliability tools. The present paper discusses the definition of a rational procedure for seismic vulnerability assessment of standardized industrial constructions in a probabilistic framework.

The coverage of a category of components by a single analysis process and the independence on the assumptions on the structural model, even if dynamic analyses are performed, can be addressed as the main advantages of the proposed method. Both seismic capacity and demand are considered as probabilistic.

The application example is focused on the elephant foot buckling of unanchored sliding tanks. A regression-based method is applied to relate fragility curves to parameters varying in the domain of variables for structural design.

INTRODUCTION

When industrial facilities and in particular chemical, petrochemical and process industries are concerned, earthquakes represent a natural hazard that can trigger relevant accidents resulting in release of Acutely Hazardous Materials (AHMs), fire, explosions that can produce injury to people and to near field equipments and constructions. Failure probability due to seismic hazard may be plugged into Quantitative Risk Analysis (QRA) either for consequence and loss assessment purposes (Lees 1996). Details about seismic risk in industrial risk analysis, i.e. Quantitative probabilistic seismic Risk Analysis (QpsRA), may be found elsewhere (Iervolino 2003, Fabbrocino et al. 2004). From the earthquake engineering perspective, lack of dynamic characterization of industrial critical components and their local and global performance parameters to be used for exhaustive risk analyses results mainly in vulnerability assessments which may be not suitable for QpsRA.

Traditional vulnerability assessment studies for industrial systems are based on empirical post seismic observations being focussed on the economic value of repair and replacement of components or of the whole plant. This approach may carry on inefficiency issues as: (1) inhomogeneous structural response in the existing data due to the unavailability of a reference type of the structure; (2) inhomogeneous boundary conditions among the available data (i.e. large scatter in the soil interaction); (3) different levels of maintenance and degradation at

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the time of the 'experiment', say earthquake; (4) uncertainty in the failure mode experienced by the system/component in the post-event assessment; (5) availability of a sufficient data set to plot a statistically significant curve for each considered damage state; (6) subjective damage evaluation. The goal herein pursued is development of a refined tool for structural reliability assessment which applies whether: (1) no sufficient observational data are available; (2) systems reliability is concerned by decomposing in fragility analysis of critical components; (3) standardized structures are considered, so that analysis of a number of different configurations covers actually all the real cases in the area of interest and allows to evaluate the influence of the variation of structural parameters on overall fragility.

The response fragility approach allows to define an approximate relationship between the failure probability, the seismic intensity measure and also the structural random parameters affecting the seismic response of the population of structures belonging to a given structural type. Among industrial equipments, steel tanks for oil and water storage are highly standardised all over the world (NIST GCR 97-720/730 1997); therefore they represent an interesting case study that will be discussed to show the ability of the method to give the expected results.

RELIABILITY OF INDUSTRIAL COMPONENTS

Quantitative estimation of seismic risk can be carried out according to the following equation that is an extension of the total probability theorem to earthquake engineering problems (Eq. 1):

$$Seismic\,Risk = \sum_{allIM} P\left[C \le D \mid IM\right] Pr\left[IM = im\right] \tag{1}$$

Where C is the structural capacity in terms of the limit state taken into account, D is the demand and IM is a seismic intensity measure able to characterise the response of the structure; $\Pr\left[IM = im\right]$ is the probabilistic characterization of the hazard as outcome of Probabilistic Seismic Hazard Analysis.

It is easy to recognise that conventionally the following equation can be written:

$$Fragility = P\left[C < D \mid IM = im\right] = P\left[C < D \mid D = d\right] P\left[D = d \mid IM = im\right] \tag{2}$$

where the capacity is assumed to be not dependent on the seismic intensity *IM*.

This approach has been adopted in FEMA 350 guidelines (Cornell et al. 2002), that refer to steel moment resisting frames, but have been later extended to different types of structures, i.e. concrete structures (Lupoi et al. 2002).

Alternative procedures to develop fragility analysis, as the one herein proposed, are based on the numerical simulation of the system and of the component subjected to earthquake shaking. The goal is to express vulnerability by fragility functions that relates the vulnerability to structure key parameters, i.e. those affecting the seismic response of the structure, Eq. 3.

$$Fragility = P\left[C(Y_1, Y_2, .., Y_n) < D(X_1, X_2, .., X_m) \mid IM\right] = f(X_1, X_2, .., X_m) \tag{3}$$

This fragility relationships can be used for a wide range of constructions of the same structural type since the dependence of the factors that seismically define the response of the structure is somehow incorporated into the failure probability. Assuming a particular structure means degenerating the fragility function into a fragility curve.

Response Surface Method (Khuri and Cornell, 1996) may be helpful in the matter; it was originally developed for the statistical evaluation of the relation between variables that are presumed to affect an experimental outcome and the outcome itself. This is not a new concept in structural reliability analysis and may be applied according to different purposes.

For example RSM can be used to fit a set of data by an approximate polynomial function depending on a set of selected parameters (Yao and Wen 1996). In that case, the goal of the procedure is the estimation of the weight of each variable on the structural response of the structure and the definition of an approximation of limit state function that can be subsequently managed according to traditional failure probability computation methods (i.e. First Order Reliability (FORM) Method or Monte Carlo Simulation) to get failure probability (Guen and Melchers 2000).

An alternative approach consists of a preliminary planning of a number of experiments; they are performed in order to define a region of interest in terms of the chosen random variables. Then fragility curves are evaluated for each selected point in the experimental space. Thus the data fitting can be carried out directly on probability of failure, again according to a polynomial function.

In the present paper the latter approach is formulated in an innovative and original form. The core of the procedure is the simulation of the dynamic behaviour of the structure; it can be adapted or refined for any system by changing the dynamic deterministic model. Random and epistemic uncertainties of phenomena are taken into account at two different levels. Random variables are divided in those affect the capacity $\overline{Y} = (Y_1, Y_2, ..., Y_n)$ and those affecting the demand $\overline{X} = (X_1, X_2, ..., X_m)$. The first vector $\overline{Y}$ consists of random variables (i.e. mechanical and materials parameters); the second one $\overline{X}$ is made of variables such as structural dimensions or member shapes. A particular realization of the vector $\overline{X}$ defines a particular structure. From this standpoint, a series of meaningful values of the vector $\overline{X}$ are needed to perform Design of Experiment (DoE) which is the base of RSM, hence a series of particular structures are analysed to observe the response that is finally interpolated for further predictions. A reliability evaluation is performed at each point of the DoE: combination of estimated probabilistic distribution of the demand (depending on $\overline{X}$) and the probabilistic distribution of the capacity (depending on $\overline{Y}$) leads to a fragility curve.

Response Surfaces is used to fit the parameters of fragility curves in the space of $\overline{X}$ vector, i.e. median and dispersion of a known distribution model fragility curve. Obviously any kind of parameter defining the fragility can be regressed.

The levels of factors $\overline{X}$ in DoE (i.e. extension of a 2k complete factorial design) are selected to capture their variation in the category of structures of interest.

THE PROCEDURE

The procedure for evaluation of the seismic vulnerability of standardized structural types and components is briefly reported; it is based on the following steps:

1. Preliminary definition of the capacity formulation for each failure mode taken into consideration. Capacity functions are probabilistically treated to get their Cumulative Distribution Function (CDF);
2. Selection and probabilistic characterisation of random parameters to be considered in the Design of Experiments;
3. Incremental time-history analyses performed according to the experimental design (Franchin et al. 2003).
4. Combination of the random capacity with the random demand function to get the fragility curve for each configuration of DoE.
5. Fitting of the fragility parameters (median and dispersion) by a polynomial in the space of the parameters of epistemic uncertainty nature; to obtain a fragility surface.

The variation range of random structural parameters affecting the demand (vector $\overline{X}$) is defined according to the available information and specific data. For example, the percentage of the population considered in the study and the location of the region of interest of the fitted approximated function or the variation in the structural type of interest may be assumed to input distributions of $\overline{X}$ variables. An orthogonal *2k* experimental plan with in addition $\mu \pm 3\sigma$ points has been designed. Each point corresponds to a particular structural configuration defining a particular structure. It is intended to associate a fragility curve plan to the DoE of Fig. (1); then parameters of fragility may be used to obtain their approximation as function of $\overline{X}$ vector by Response Surface.

Optimisation of RS and plan of experiments are not herein discussed for sake of brevity, since the attention is mainly focussed on the development of the procedure and on its ability to give effective answers suitable for

QpsRA purposes.

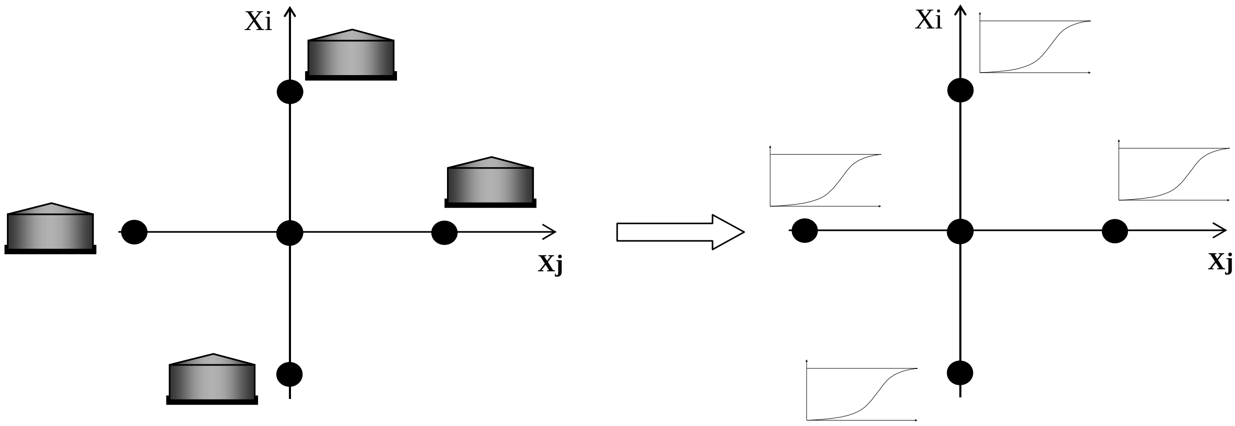


Figure 1. Experimental design meaning

A probabilistic characterisation of capacity function is needed for considered failure modes. Once a capacity function is defined, if distribution of parameters influencing capacity are available, probabilistic distribution can be carried out by simplified methods i.e. simulation method as Monte Carlo (MC).
A number of simulations for each $d_i \in [a,b]$ are carried out, so that the failure probability for each level of the demand d_i can be defined. The points $P[C < D \mid D = d_i] \quad \forall d_i \in [a,b]$ give an approximation of the capacity Cumulative Probability Function (CDF) which is the first term of the right hand side of Eq. 2.

$$C = C(Y_1, Y_2, ..., Y_n) \rightarrow P[C < D \mid D = d] \qquad (4)$$

where C is the capacity associated to a given limit state, and $Y_1, Y_2, ..., Y_m$ are the random variables (local parameters) included in the capacity.
It is worth noting that $\overline{Y}$ are those variables affecting the capacity (materials and other local parameters) and it is assumed that they do not affect the demand that is only governed by $\overline{X}$ variables used in the design of experiments.
For the demand estimation, dynamic numerical simulations have to be carried out. Seismic input can be either recorded or spectrum-compatible accelerograms, even if the latter can over excite higher modes or be too benign.
In the case of real records, selection of accelerograms should fit the following criteria: (1) Far field records: distance over 15 km, in order to avoid possible directivity and pulse effects; (2) soil type C – D according to USGS classification to avoid site effects; (3) Free field or one story instrument housing; (4) Limited number of records coming from the same event to avoid event biasing of the demand estimation. (5) Elastic spectra should be checked avoiding "spectral shape" amplification effects. If information on the site where the structure is located are available, one or more of these constraints can be relaxed (Iervolino and Cornell 2004).
A simple regression of the seismic demand for each configuration can be carried out according to the Incremental Dynamic Analysis of Structures (Vamvakistos and Cornell 2002) by scaling accelerograms by the intensity measure of interest (i.e. spectral acceleration at the fundamental period of the structure).
In such a way a relationship between the demand and a ground motion parameter can be defined in a given range.
The next step is the definition of the probabilistic distribution of the demand. More in detail, for each of the points of the seismic intensity measure the set is scaled to the given *IM* in order to estimate the demand at that level with the minimum possible variance. Then dynamic time-history analyses are performed for each *IM* level in the chosen range. In this way a regression model can estimate the median demand and its variance as a function of the intensity measure.

$$P[D = d \mid IM = im] = log\ normalPDF(IM) \tag{5}$$

A lognormal or other kind of distribution for the demand can be assumed at each IM level, as in the Eq. (5) example; this assumption may be validated (i.e. by Kolmogorof-Smirnov test).
The above steps are to be repeated for each of the structural configuration included in the Design of Experiments (DOE); then for each realization of the vector $\overline{X}$ that defines a particular structural configuration, a fragility curve can be generated, comparing distribution of the demand and the capacity at each IM level, Eq. (6).

$$P_{f|IM} = \int_0^\infty \left[1 - F_{D|IM}(u)\right] f_C(u)\,du \quad \forall IM \tag{6}$$

In Eq. (6) fragility is intended as a series of points $P_f \mid IM$; $F_{D|IM}(u)$ is the estimated CDF of demand at given *IM*; $f_C(u)$ is PDF of capacity and *u* is a dummy variable.
Parameters that define fragility curves (i.e. median and dispersion) in the DoE plan are experiments observed response, their regression can be carried out referring for instance to a polynomial function such as a second order model:

$$\mu(\overline{X}) = \beta_0 + \sum_{i=1}^{m} \beta_i X_i + \sum_{i=1}^{m} \beta_{ii} X_i^2 + \left[\sum_{i=1}^{m-1} \sum_{j=2}^{m} \beta_{ij} X_i X_j\right]_{i<j} \tag{7}$$

$$\sigma(\overline{X}) = \gamma_0 + \sum_{i=1}^{m} \gamma_i X_i + \sum_{i=1}^{m} \gamma_{ii} X_i^2 + \left[\sum_{i=1}^{m-1} \sum_{j=2}^{m} \gamma_{ij} X_i X_j\right]_{i<j} \tag{8}$$

Where $\sigma(\overline{X}), \mu(\overline{X})$ are the fragility parameters of interest to be approximated; $X_1, X_2, ..., X_m$ are the variables, which are supposed to influence the response, and $\beta_1, \beta_2, ..., \beta_k$; $\gamma_1, \gamma_2, ..., \gamma_k$ are the estimated coefficients.

STEEL TANKS SHELL BUCKLING FRAGILITY

According to the definition of Guidelines for Seismic Evaluation and Design of Petrochemical Facilities (ASCE, 1997), industrial components are to be divided in building-like and non-building-like structures. The majority of structures found in petrochemical facilities are non-building-like.
Building-like structures are pipe ways and support systems while non-building-like are basically vessels and tanks. The latter are generally made of steel and are highly standardised components. They can experience failure with loss of content during earthquakes, as demonstrated by post-event surveys.
Sometimes failure of storage tanks caused disastrous consequences as fires causing extended damages in the 1964 Niigata and 1991 Costa Rica Earthquakes and polluted waterways in the 1978 earthquake in Sendai Japan. Often refineries and deposits are located near seaports due to the mainstream oil transportation practice that is by sea, thus pollution of marine environment can occur. Hazardous tanks systems are also present in the main airport infrastructures.
Welded on grade steel tanks for oil storage can experience anchor failures or large differential settlements due to foundation failure, excessive tensile hoop tension in the shell or connecting pipes failure due to large vertical or horizontal displacements if base uplifting or sliding occurs. All those failure modes can trigger loss of content (Salzano et al. 2003). However post-earthquake damage observations show that a very common type of failure is represented by the so-called elephant foot buckling. It is caused by the large overturning base moment due to the impulsive and convective liquid loading on the tank wall during the ground shaking.

The high vertical compressive stresses, which develop in the shell, may cause buckling of the shell with a typical shape (NIST GCR 97-720/730 1997). Severity of buckling mainly depends on filling level and ground/anchoring conditions at the time of earthquake as the intensity of structural actions during the dynamic motion depends on hydrostatic and hydrodynamic pressures. However, buckling of the shell and large displacements due to sliding can be assumed as main damage mechanisms. According to the probabilistic framework, filling level and tank-ground friction factor are assumed as epistemic variables in the response while materials/local parameters are considered as randomness sources for the capacity.

Dynamic behaviour and capacity formulation

International standards (API 620-650 1998; Bandyopadhyay 1995) provide simplified procedures to design the thickness of the shell under static loads while the other component dimensions are just chosen in tabled ranges. Other codes (Eurocode 8 2000) suggest more refined seismic analysis, but existing structures have been often designed without any consideration of lateral seismic loads or according to simplified relationships between seismic actions and axial stress acting on the shell.

Dynamic behaviour of this type of structure is governed by fluid/structure interaction. Liquid mass can be divided into two oscillating fractions corresponding to the *sloshing motion* (i.e. liquid below the free surface) with a different period respect to the *impulsive motion* which is corresponding to deeper liquid (Malhotra, 2000). Convective and impulsive masses and their centre of gravity positions are dependent on the geometry of the tank. Since the height to be considered is the effective liquid filling level the structural parameters affecting the structural dynamic response ($\overline{X}$ vector) can be listed as follows: (1) content depth over radius; (2) friction coefficient between bottom plate and grade of unanchored tanks.

The allowable stress provided by current codes is mono-directional whereas the stress at the bottom of the shell is bi-directional including hoop tensions also; this condition leads to an additional eccentricity resulting in the elephant foot shape of elastic-plastic buckling. It can be assessed reducing the classical buckling stress by a factor depending on the yielding stress, radius over thickness ratio and internal pressure (Shih 1981).

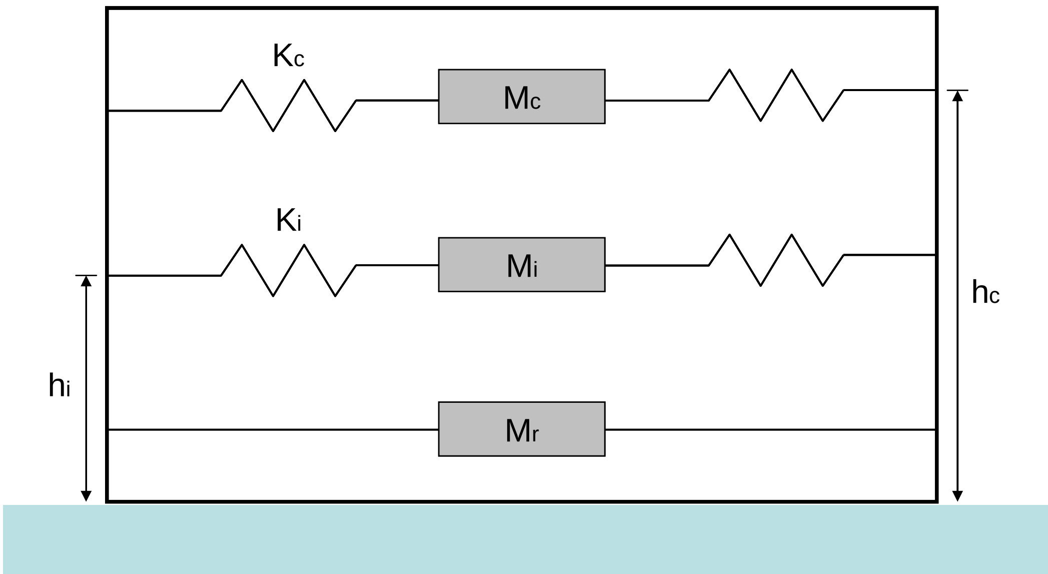


Figure 2. 1D dynamic model for tanks

When unanchored tanks are considered, sliding motion can occur, as observed during post-earthquake damage assessment. A simple but effective dynamic model of sliding tanks has been proposed by Shrimali and. Jangid (2002) Fig. (2), the seismic behaviour of the tank is assumed as a one-dimensional problem ruled by convective and impulsive liquid fraction masses within the tank plus a rigid part moving together with the tank base.

In Fig. (2) m_c and m_i are the convective and impulsive liquid masses respectively; h_c and h_i are the positions of the centroids of the mentioned masses; k_c and k_i are the equivalent stiffness which can be associated to the convective and impulsive motions of the fluid; m_r is the rigid mass. All this parameters may be retrieved as function of tank geometry (Haroun, 1983).

In the following, the attention is focussed on flat tanks that due to the specific aspect ratio can experience basically sliding phenomena. During earthquake shaking, the tank can rest or slide depending on base acceleration and tank velocity (Iervolino et al., 2003).

According to proposed model, response measures related to the considered failure modes can be monitored; in fact, absolute displacements is a direct outcome of the analysis as the overturning moment. Hence, the demand can be evaluated by the static formula that relates the overturning moment to the compression stress as shown in Eq. (9).

$$\sigma_{dem} = \frac{W_t + 1.273\left|OTM / 4r^2\right|}{t} \qquad (9)$$

Where W_t is the weight of the tank structure per unit of length; *OTM* is the overturning moment and *t* is the shell's thickness.

On the capacity side, numerical analyses showed the effectiveness of a one-dimension (simplified) design equation for buckling capacity in terms of critical stress (Kim and Kim 2002):

$$\sigma_{cap} = 1.19\left(\frac{H}{2r}\right)^{-0.0256}\frac{t}{2r}E \qquad (10)$$

where *r* is the nominal radius of the tank, *E* is the Young's modulus of the steel of the tank. Based on the above set of equations it is possible to evaluate the demand of the compressive stress monitoring time histories of unanchored tanks subjected to sliding and compare it with the capacity.

Fragility surfaces

Random variables related to epistemic uncertainty are filling level in terms of filling height over radius ratio (*H/R*) and friction factor (*f*), these two parameters are those affecting the seismic response then defining the vector $\overline{X}$.

On the capacity side, looking at Eq. 10, the thickness of the shell and the Young's modulus of the steel (*E*) can be considered as random variables are collected in the vector $\overline{Y}$.

Thickness can vary according to the corrosion of the shell during the service life of the structure. Corrosion allowance may be assumed as the one-standard deviation value for the thickness distribution. Young's modulus is less uncertain and its distribution is given by tests

It is worth noting that the first set of variables is taken as deterministic in the capacity evaluation and they only may vary in the demand analysis.

In fact, the proposed method solves a reliability problem for each point of the experimental plan than for any given structure represented by a point of the design space where the epistemic values (as dimensions) are set to a certain value (deterministically known if a given structure is concerned).

The other random parameters are meaningful in limit states monitoring but do not affect the response. Others factors should be taken into account if a probabilistic characterization is available.

Any other random factor that does not concern strictly the structure can be processed, i.e. workmanship, suitability of mathematical model or degradation, without affecting the proposed procedure. Probabilistic distribution of factors is reported in Table 1.

The number of experiments needed to develop the response surface depends on the chosen polynomial fitting the surface. The considered experimental plan is summarised in Table 2.

Table 1. Random Variables Characterisation

Random Variables	*PDF*	*Mean*	*C.o.V.*
Young Modulus (E)	*Normal*	*210000 [MPa]*	*0.15*
Shell's Thickness	*Normal*	*8 [mm]*	*0.2*
Filling level (H/R)	*Normal*	*0.7*	*0.3*
Friction factor (f)	*Normal*	*0.5*	*0.3*

Table 2. Design of experiments

Configuration	*Filling level over radius H/r*	*Friction factor f*
Reference	μ	μ
Configuration 2	μ-1σ	μ-1σ
Configuration 3	μ+1σ	μ +1 σ
Configuration 4	μ -1 σ	μ +1 σ
Configuration 5	μ +1 σ	μ -1 σ
Configuration 6	μ +3 σ	μ
Configuration 7	μ -3 σ	μ
Configuration 8	μ	μ +3 σ
Configuration 9	μ	μ -3 σ

The dynamic properties of each configuration are, according to the proposed model only dependent on the dimensional parameters and the assumed filling level.

The DoE shown in Table 2 results in a design matrix for a series of different tanks configurations listed in Table 3, where: *H/R* is the filling height over the radius of the tank; m_c/M is the convective fraction of the content; m_i/M is the impulsive mass fraction; h_c/H and h_i/H are the positions of the centres of gravity of the convective and impulsive masses; T_c is the oscillation period of the convective mass; *f* is the friction factor.

For each configuration an incremental time history analysis have been performed. A set of six recorded accelerograms has been scaled due to the structure spectral acceleration. The chosen records have been selected from the Pacific Earthquake Engineering Center Database (http://peer.berkeley.edu).

The scaling range goes from 0.1 [g] to 3 [g] in terms of Peak Ground Acceleration. The interval is divided in 20 points; each record has been scaled to match the considered PGA level.

Results of time histories allow estimating mean and coefficient of variation as a function of the spectral acceleration in the regression. Given the characterisation of the demand (i.e. lognormally distributed around the median values) at each PGA level a demand distribution is associated to the particular structural configuration considered.

Then the fragility for each structure can be evaluated by FORM analysis at each PGA level by comparing the estimated distribution of the demand with the distribution of the capacity which is fixed given the configuration.

A series of FORM analysis has been performed to get the Cumulative Distribution Function of the capacity compression stress in the shell σ_{cap}.

Fragility curves of each considered configuration have been approximated by lognormal distribution and are reported in Fig. (3)

The median values strongly depend on the filling level; in particular, the lower is the filling level, the lower is the vulnerability of the tank related to the considered limit states. This circumstance is confirmed by results of Configuration 7, which is characterised by a very low filling level. In this case, the probability of failure is almost negligible all over the investigated range of spectral acceleration and the median of the fragility curve is by far higher than the remaining and of the values expected according to observational data (Salzano et al. 2003).
Since numerical checks confirm that Lognormal CDF can approximate the fragilities, results in terms of seismic fragility of the system can be defined as follows:

$$Fragility = LN\left(\mu\left(\frac{H}{r}, f\right), \beta\left(\frac{H}{r}, f\right)\right) \tag{11}$$

where $\mu(H/R,f)$ and $\sigma(H/R,f)$ are mean and dispersion as a function of structural parameters. Generated data can be fitted in a fragility surface based on a second order model. Fitting $\mu(H/R,f)$ and $\beta(H/R,f)$ instead of the failure probability enables the estimation of the influence of the structural parameters independently for the median and the dispersion, improving the knowledge and the consciousness of the structure.

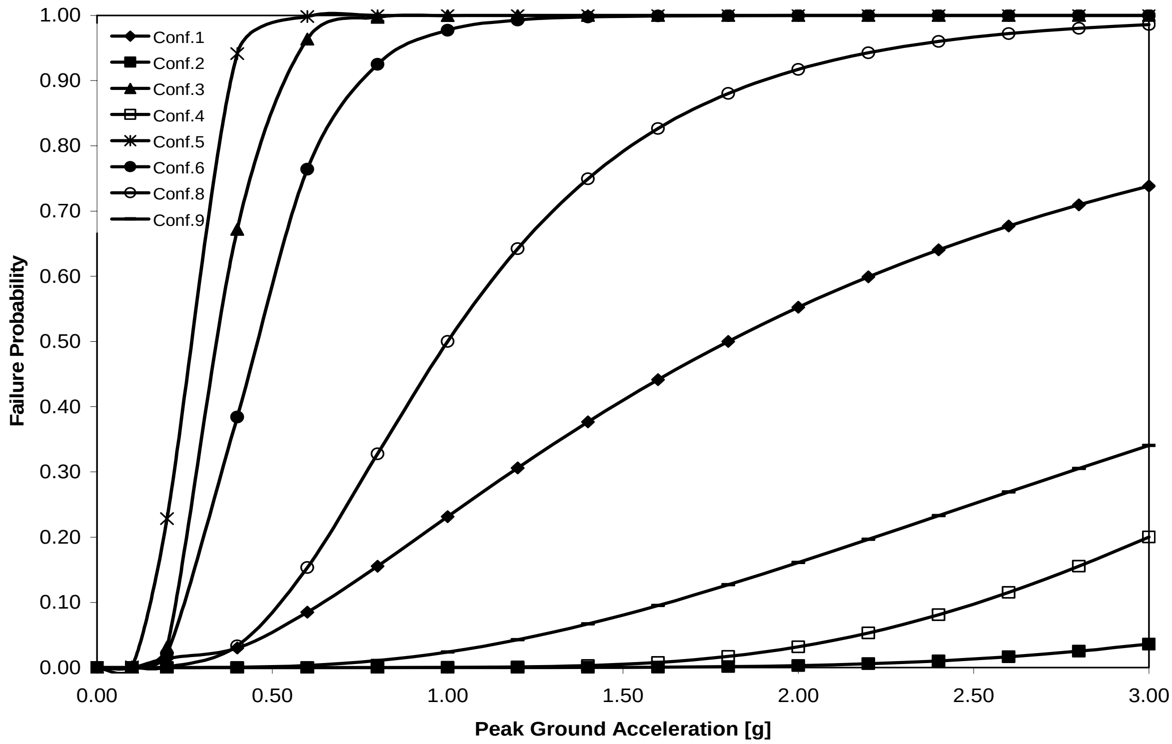
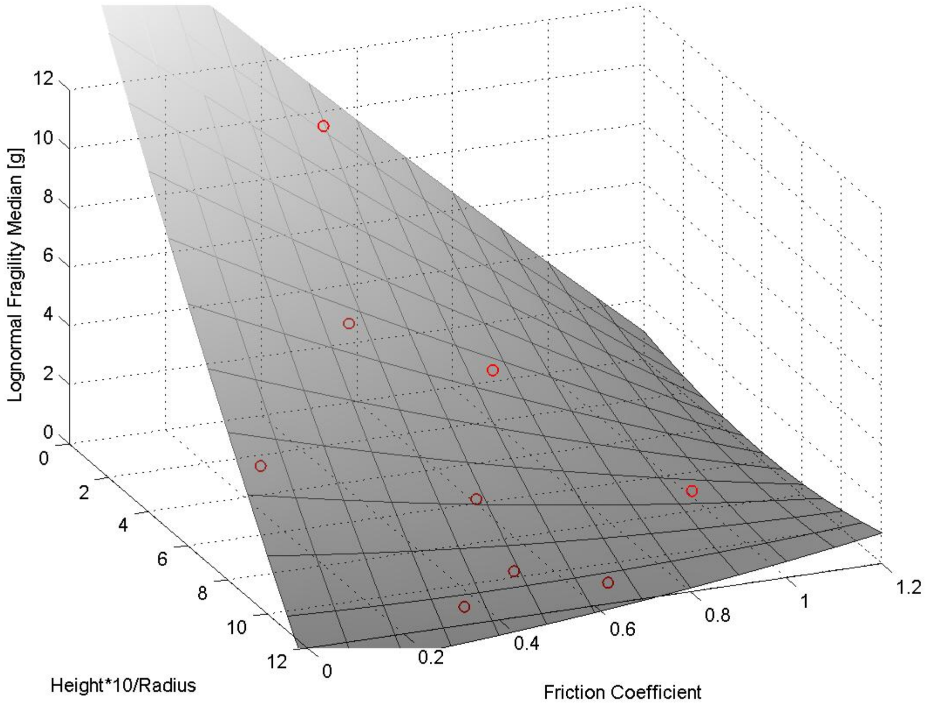
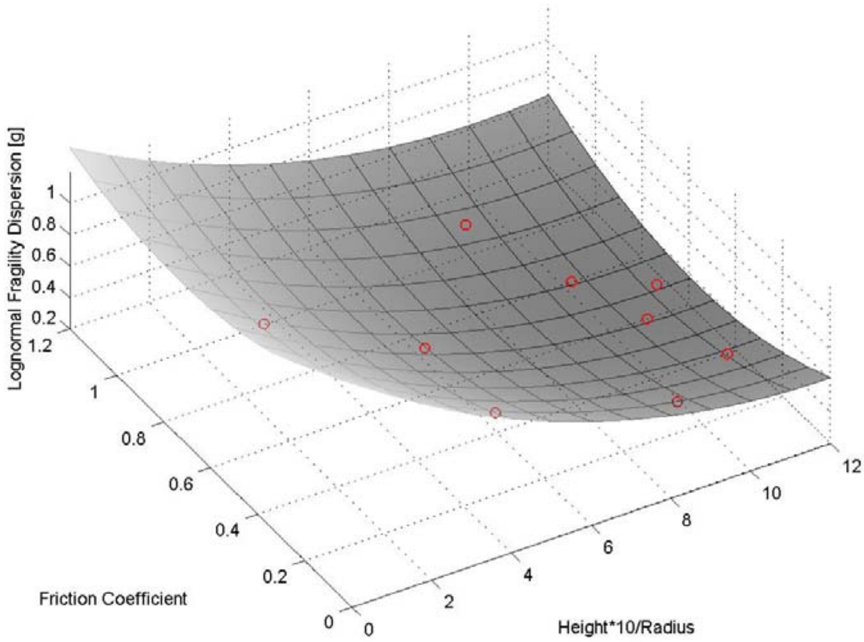


Figure 3. Fragility of the DoE configurations.

A complete second order model was optimised by least squares method to fit their variability. Results are plotted in Fig. (4).



a)



b)

Figure 4. Second order fitting for fragility median (a) and standard deviation (b)

Points reported in Fig. 4 are the results obtained by the procedure for the points of experimental plan. Second

order response surfaces are good in fitting data, in particular the median RS has a R^2 of about 99% while it is less efficient for the standard deviation (R^2 ~ 70%) but still good to fully capture the influence of the structural parameters on the fragility and predict it.
Results show how the reduction of filling level corresponds to larger vulnerability of the tank, as confirmed by empirical data. Same influence trend is observed for friction factor.
Evaluation of the results in terms of dispersion of fragility allows recognising that the higher is the failure median intensity, the higher is dispersion. This result confirms that increasing of intensity levels in Dynamic Incremental Analyses of nonlinear systems leads to a large scatter of the output around the medians.

CONCLUSIONS

The procedure proposed in the present paper enables effective seismic risk estimation for a large number of standardised structural components and constructions as a whole.
The main advantages of the proposed method can be summarised as follows: (1) seismic vulnerability assessment is applicable to all the structures belonging to the same population type with no lack of accuracy; (2) capacity and demand are probabilistically characterised; for what concerns the demand it is evaluated by means of non-linear dynamic analyses with explicit propagation of errors; (3) regression in terms of fragility shows the influence of DOE factors directly on the probability of failure. The generality of the procedure and its portability are not dependent on the dynamic model assumed for estimation of the demand or chosen limit states.
Application to steel tanks for oil storage show how the procedure is simple to apply even though requires dynamic modelling. Regarding to the elephant foot buckling limit state the capability of the method to capture functional relationship between fragility and response affecting parameters such as filling level has been shown.

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