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FRAGILITY OF STANDARD INDUSTRIAL STRUCTURES BY A RESPONSE SURFACE BASED METHOD

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National and international regulatory standards require industrial risk assessment, taking into account natural hazards including earthquakes, in the framework of Quantitative Risk Analysis (QRA). Seismic fragility analysis of industrial components may be carried out similarly as what has been done for buildings, even though some peculiar aspects require the development of specific tools. In the present paper a contribution to the definition of a rational procedure for seismic vulnerability assessment of standardised industrial constructions in a probabilistic framework is given. The method covers a range of components of the same structural type. Seismic reliability formulation for structures is used. Both seismic capacity and demand are considered probabilistic with the latter assessed by dynamic analyses. The application example refers to shell 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.

Keywords: Seismic risk; structural reliability; fragility; response surface; industrial risk; process industry; tanks.

1. Introduction and Research Significance

Probabilistic approach to the structural assessment has been recognised as the more effective tool to analyse seismic risk.

When industrial facilities and in particular chemical, petrochemical and process industries are concerned, earthquakes represent a natural hazard which can trigger relevant accidents resulting in the release of Acutely Hazardous Materials (AHMs), fire, as well as explosions that can result in injury to people and equipments and constructions.

Accordingly, results of seismic risk analysis of critical components, i.e. annual failure probability due to seismic hazard, may be plugged into the Quantitative Risk Analysis (QRA) either for consequence or loss assessment purposes [Lees, 1996]. It requires the seismic vulnerability analysis with respect to those limit states which

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can trigger industrial accident and its convolution with the Probabilistic Seismic Hazard Analysis [McGuire, 1995; Cornell, 1968]. 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, the lack of dynamic characterisation of industrial critical components as well as their local and global performance parameters to be used for exhaustive risk analyses, result mainly in vulnerability assessment which may not be suitable for QpsRA.

Most of the former studies on the assessment of seismic vulnerability of 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 experience inefficiency issues such as: (1) inhomogeneous structural response in the available data due to the unavailability of a reference type of the structure; (2) non-uniformity of boundary conditions among the available data (i.e. large scatter in the soil interaction); (3) different levels of maintenance and degradation at the time of the experiment, e.g. 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 the development of a refined tool for structural reliability assessment which applies under the conditions that: (1) no sufficient observational data are available; (2) systems reliability is concerned with decomposing in the fragility analysis of critical components; (3) standardised structures are considered, so that the analysis of a number of different configurations covers all the real cases in the area of interest and allows one to evaluate the influence of the variation of structural parameters on the overall fragility.

The response fragility approach allows one 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 (i.e. tanks). 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 which shows the ability of the method in giving the expected results.

2. Fragility Functions

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

$$\text{Seismic Risk} = \sum_{all\,IM} \Pr[C \leq D|IM]\,\Pr[IM = im]\,, \tag{1}$$

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; $\Pr[IM = im]$ is the probabilistic characterisation of the hazard as an outcome of Probabilistic Seismic Hazard Analysis.

It is easy to recognise that is possible to conventionally write the following equation referring to the total probability theorem:

$$\text{Fragility} = P[C < D|IM = im] = P[C < D|D = d]P[D = d|IM = im]\,, \qquad (2)$$

where capacity is assumed to be independent on the seismic intensity IM. This approach has been followed 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 the component subjected to earthquake shaking. The goal is to express vulnerability by fragility functions relating vulnerability to structure key parameters, i.e. those affecting the seismic response of the structure, Eq. (3).

$$\begin{aligned}\text{Fragility} &= P[C(Y_1, Y_2, \ldots, Y_n) < D(X_1, X_2, \ldots, X_m)|IM] \\ &= f(X_1, X_2, \ldots, X_m)\,. \end{aligned} \qquad (3)$$

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; it is 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. As a result, the fitting of data can be carried out directly on the probability of failure, again generally according to a polynomial function.

In the present paper the latter approach is pursued in an innovative and original form. The core of the procedure is the simulation of dynamic behaviour of the structure; it can be adapted or refined for any system by changing the dynamic model. Random and epistemic uncertainties of phenomena are taken into account at two different levels. Random variables are divided into two classes, namely those that affect the capacity $\bar{Y} = (Y_1, Y_2, \ldots, Y_n)$ and those that affect the demand $\bar{X} = (X_1, X_2, \ldots X_m)$. The first vector $\bar{Y}$ consists of random variables (say mechanical and materials parameters); the second one, $\bar{X}$, is made up of variables such as structural dimensions or member shapes. It is assumed that a particular realisation of the vector $\bar{X}$ defines a particular structure.

From this standpoint, a series of meaningful values of the vector $\bar{X}$ are needed to perform the Design of Experiment (DoE) which is the base of RSM, hence a series of particular structures are analysed to observe the response 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 $\bar{X}$) and the probabilistic distribution of the capacity (depending on $\bar{Y}$) leads to a fragility curve.

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

The levels of factors $\bar{X}$ in DoE (i.e. extension of a $2k$ complete factorial design) are selected to capture their variation in the category of structures of interest; for instance, if a relevant factor is a narrowly normally distributed function around its mean in the population of structures to be investigated, the value assuming the two level in experiments is the median plus or minus the standard deviation.

2.1. *The procedure*

In the present section the procedure for the evaluation of the seismic risk of standardised structural types and components is briefly reported. For the sake of clarity, schematic description of the steps of the process is given in autonomous subsections.

Vulnerability assessment 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) (i.e. by Monte Carlo simulation); (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 in DoE. (5) Finally the fragility parameters (i.e. median and dispersion) are fitted by a polynomial in the space of the parameters of epistemic uncertainty nature; in this way a fragility surface is obtained.

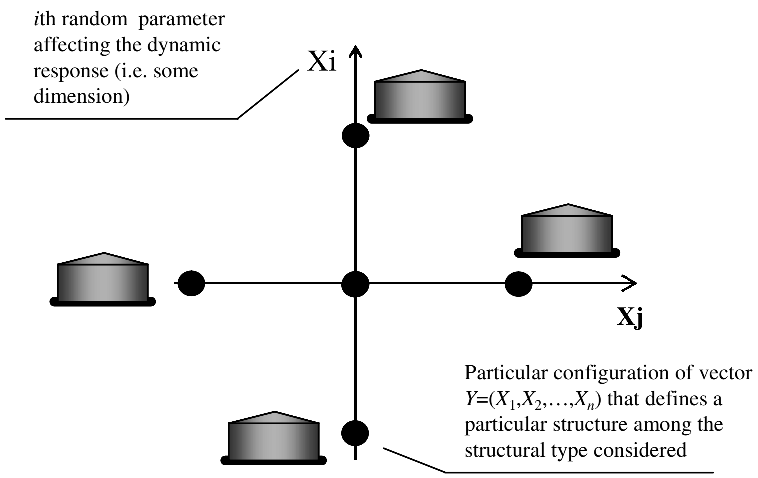

Fig. 1. Experimental design.

2.1.1. *Step 1: Design of experiments*

The variation range of random structural parameters affecting the demand (vector $\bar{X}$) is defined according to 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 $\bar{X}$ variables. In the present application, 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 shown in Fig. 1.

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.

2.1.2. *Step 2: Capacity*

A probabilistic characterisation of capacity function is needed for considered failure modes. Once a capacity function is defined, if the distribution of parameters influencing capacity are available, the probabilistic distribution can be carried out by simplified methods i.e. the simulation method such as Monte Carlo (MC).

Performing as many simulations for each $d_i \in [a, b]$, gives the failure probability for each level of the demand d_i. The points $P[C < D|D = d_i]\ \forall\, d_i \in [a, b]$ give an approximation of the capacity Cumulative Probability Function (CDF) which is the first term of the right hand side of Eq. (2).

$$C = C(Y_1, Y_2, \ldots, Y_n) \rightarrow P[C < D|D = d]\,. \tag{4}$$

In Eq. (4) C is the capacity associated with a given limit state, $Y_1, Y_2, \ldots, Y_m$ are the random variables (local parameters) included in the capacity.

It is worth noting that $\bar{Y}$ are those variables affect the capacity (materials and other local parameters) and it is assumed that they are not affecting the demand that is only governed by $\bar{X}$ variables used in the design of experiments.

2.1.3. *Step 3: Demand*

For the demand estimation, dynamic numerical simulations have to be carried out. Seismic input can be either recorded or saved in spectrum-compatible accelerograms, even if the latter can over excite higher modes or be too benign.

In the case of real records, the 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-storey 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 to avoid "spectral shape" amplification effects. If information on the site where the structure is located is available, one or more of these constraints can be relaxed [Iervolino, 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 the dynamic time-history analysis is run for each IM level in the chosen range. In this way a regression model can estimate the median demand and its variance as function of intensity measure.

$$P[D = d|IM = im] = lognormalPDF(IM)\,. \tag{5}$$

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 the Kolmogorof-Smirnov test).

2.1.4. *Step 4: Fragility*

Steps 1 and 2 are repeated for each of the structural configuration included in the Design of Experiments (DoE); then for each realisation of the vector $\bar{X}$ that defines a particular structural configuration, a fragility curve can be generated to compare distribution of the demand and the capacity at each IM level, Eq. (6).

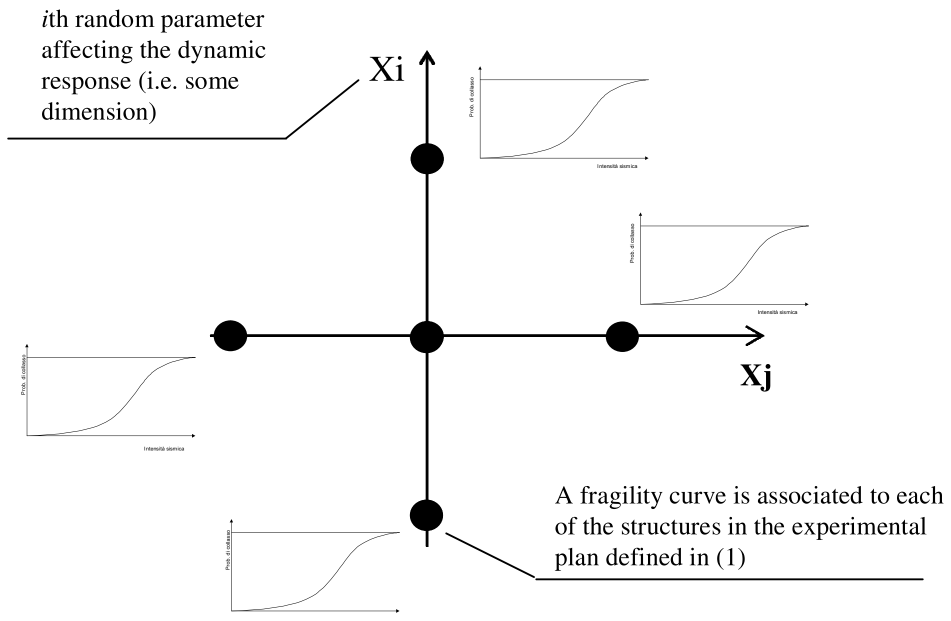

Fig. 2. Evaluation of reference fragilities for RS development.

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

In Eq. (6) fragility is intended as a series of points $P_f|IM$; $F_{D|IM}(u)$ is the estimated CDF of demand at a given IM; $f_C(u)$ is the PDF of capacity and u is a dummy variable. So far, a fragility curves plan (Fig. 2) is associated to the DoE of Fig. 1.

Now parameters of fragility may be used to obtain their approximation as function of $\bar{X}$ vector by RS.

2.1.5. *Step 5: Fragility RS's*

Parameters that define fragility curves (i.e. median and dispersion) in the DoE plan are experimental-observed responses, their regression can be carried out referring, for instance, to a polynomial function such as a second order model:

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

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

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

3. Application — Steel Tanks Shell Buckling Fragility

According to the definition of Guidelines for Seismic Evaluation and Design of Petrochemical Facilities [ASCE, 1997], 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 structures.

Building-like structures are pipeways 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 a loss of content during earthquakes, as demonstrated by post-event surveys. Sometimes failure of storage tanks resulted in fires causing disastrous consequences in the 1964 Niigata and 1991 Costa Rica Earthquakes. It has caused 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 is common. Hazardous tanks systems are also present in the main airport infrastructures.

Welding 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 the base uplifting or sliding occurs. All those failure modes can trigger a loss of content [Salzano *et al.*, 2003]. However post-earthquake damage observations show a very common type of failure that is known as elephant foot buckling. This is caused by the large overturning base moments resulting from the impulsive and convective liquid loading on the tank wall during an earthquake (e.g. sloshing).

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 the 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 in the capacity.

3.1. *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 often been 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 with 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 ($\bar{X}$ 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; 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].

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. 3). 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's base.

In Fig. 3 m_c, m_i and m_r 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

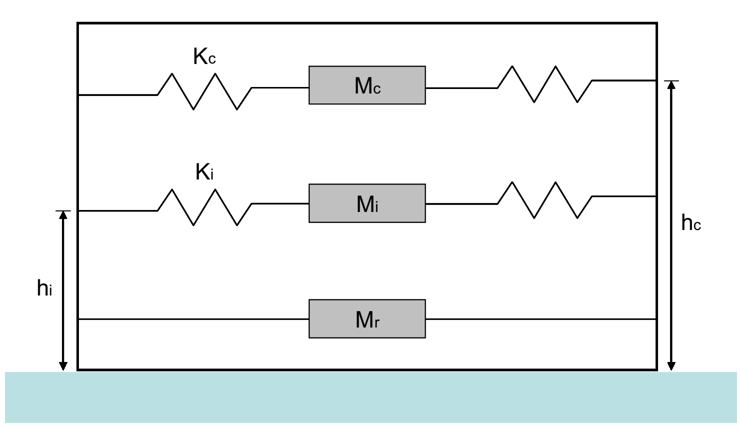

Fig. 3. 1D dynamic model for tanks.

and k_i are the equivalent stiffness which can be associated to the convective and impulsive motions of the fluid.

In the following, the attention is focused 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 tanks velocity. The system is made of three equations expressing convective, impulsive and tank as a whole equilibrium [Eq. (9)].

$$\begin{cases} m_c(\ddot{x}_c(t)+\ddot{x}_b(t))+k_c x_c(t)=-m_c a_g(t) \\ m_i(\ddot{x}_i(t)+\ddot{x}_b(t))+k_i x_i(t)=-m_i a_g(t) \\ m_c\ddot{x}_c(t)+m_c\ddot{x}_c(t)+M\ddot{x}_b(t)=-Ma_g(t)-\mathrm{sgn}(\dot{u}_b(t))fMa_g(t) \end{cases}, \tag{9}$$

where M is the total mass of the tank; x_i and x_c are the relative displacement of impulsive and convective masses respectively in respect of the tank's base; x_b is the displacement of the base with respect to the ground, a_g is the ground acceleration and f is the friction factor.

According to the proposed model in Eq. (10), the 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, that are related to the motion by Eq. (10).

$$OTM(t)=m_i\,\ddot{u}_i(t)h_i+m_c\ddot{u}_c(t)h_c\,. \tag{10}$$

In this equation the position of masses fractions centroid $(h_i; h_c)$ appears; the buckling critical axial stress is increased by content pressure both for perfect and imperfect shells. Hence, the demand can be evaluated by the static formula that relates the overturning moment to the compression stress as shown in Eq. (11).

$$\sigma_{dem}=\frac{W_t+1.273|OTM/4r^2}{t}, \tag{11}$$

where W_t is the weight of the tank's structure per unit of length.

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]:

$$\sigma_{cap}=1.19\left(\frac{H}{2r}\right)^{-0.0256}\frac{t}{2r}E\,. \tag{12}$$

In Eq. (12) r is the nominal radius of the tank, E is the Young's modulus of the steel of the tank; t is the thickness of the shell and H is the filling height of the tank. Based on the above set of equations it is possible to evaluate the demand of the compression stress by monitoring the time history of unanchored tanks subjected to sliding and then compare it with the capacity.

3.2. *Uncertainties in steel tanks analysis*

Random variables related to epistemic uncertainty are the 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 which then define the vector $\bar{X}$.

On the capacity side, looking at Eq. (12), the thickness of the shell and the Young's modulus of the steel (E) can be considered as random variables composing vector $\bar{Y}$.

Thickness is varying according to the corrosion of the shell during the service life of the structure which has to be properly taken into account during design. 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 may only 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. Other factors should be taken into account if a probabilistic characterisation is available. Any other random factor that does not strictly concern the structure can be processed, i.e. workmanship, suitability of mathematical model or degradation, without affecting the proposed procedure.

The capacity function in Eq. (12) is non-linear with respect to the random variables. Simplified or simulation methods can be used in approximate computation of the cumulative distribution function of the capacity. Probabilistic distribution of factors is reported in Table 1.

3.3. *Reliability analysis*

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.

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

Table 1. Random variables characterisation.

Random Variables	PDF	Mean	C.o.V.
Young Modulus (E)	Normal	210 000 [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
Configuration 1	μ	μ
Configuration 2	$\mu - 1\sigma$	$\mu - 1\sigma$
Configuration 3	$\mu + 1\sigma$	$\mu + 1\sigma$
Configuration 4	$\mu - 1\sigma$	$\mu + 1\sigma$
Configuration 5	$\mu + 1\sigma$	$\mu - 1\sigma$
Configuration 6	$\mu + 3\sigma$	μ
Configuration 7	$\mu - 3\sigma$	μ
Configuration 8	μ	$\mu + 3\sigma$
Configuration 9	μ	$\mu - 3\sigma$

Table 3. Configurations table.

	H/r	mc/M	mi/M	hc/H	hi/H	Tc (sec)	f
Configuration 1	0.840	0.52	0.48	0.59	0.41	4.95	0.5
Configuration 2	0.588	0.65	0.35	0.56	0.40	5.47	0.35
Configuration 3	1.092	0.42	0.58	0.63	0.42	4.65	0.65
Configuration 4	0.588	0.65	0.35	0.56	0.40	5.47	0.65
Configuration 5	1.092	0.42	0.58	0.63	0.42	4.65	0.35
Configuration 6	1.596	0.29	0.71	0.70	0.44	4.53	0.5
Configuration 7	0.084	0.99	0.01	0.49	0.39	7.39	0.5
Configuration 8	0.840	0.52	0.48	0.59	0.41	4.95	0.95
Configuration 9	0.840	0.52	0.48	0.59	0.41	4.95	0.05

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; mc/M is the convective fraction of the content, Fig. 4; mi/M is the impulsive mass fraction, Fig. 4; hc/H and hi/H are the positions of the centres of gravity of the convective and impulsive masses, Fig. 5; Tc is the oscillation period of the convective mass and f is the friction factor.

For each configuration an incremental time history analysis has 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). They match the criteria previously stated on the subject; in addition spectra have been checked to avoid "peak-valleys" effects in the spectral shape at the period of interest. This check helps avoiding bias in the demand estimation thought scaling of records. 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.

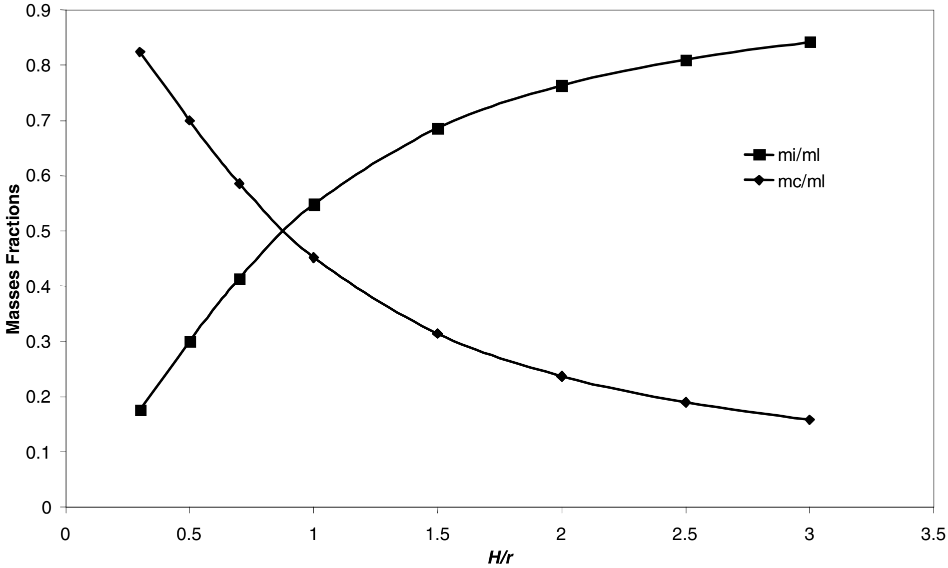

Fig. 4. Convective and impulsive mass fractions as function of H/r.

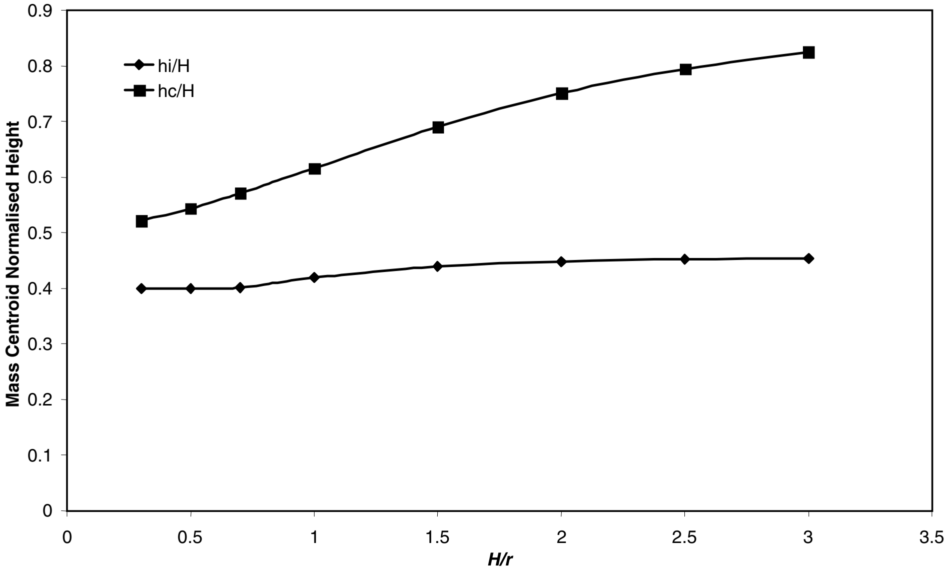

Fig. 5. Convective and impulsive mass fractions positions as function of H/r.

Results of time histories allow for the estimation of mean and coefficient of variation as a function of the spectral acceleration in the regression. In Figs. 6 and 7 results are plotted directly in terms of compressive axial stress in the shell median and standard deviation.

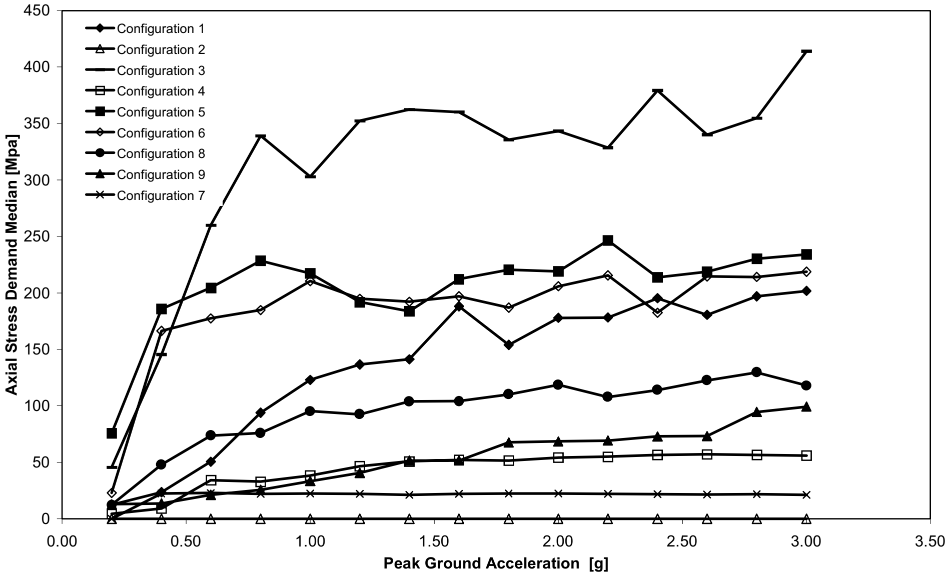

Fig. 6. Seismic demand analysis medians.

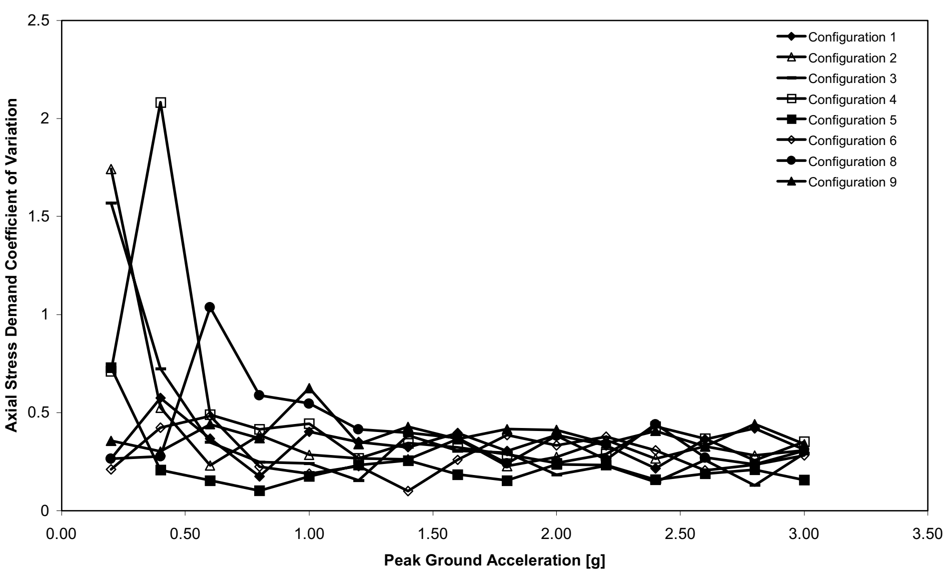

Fig. 7. Seismic demand analysis results coefficients of variation.

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

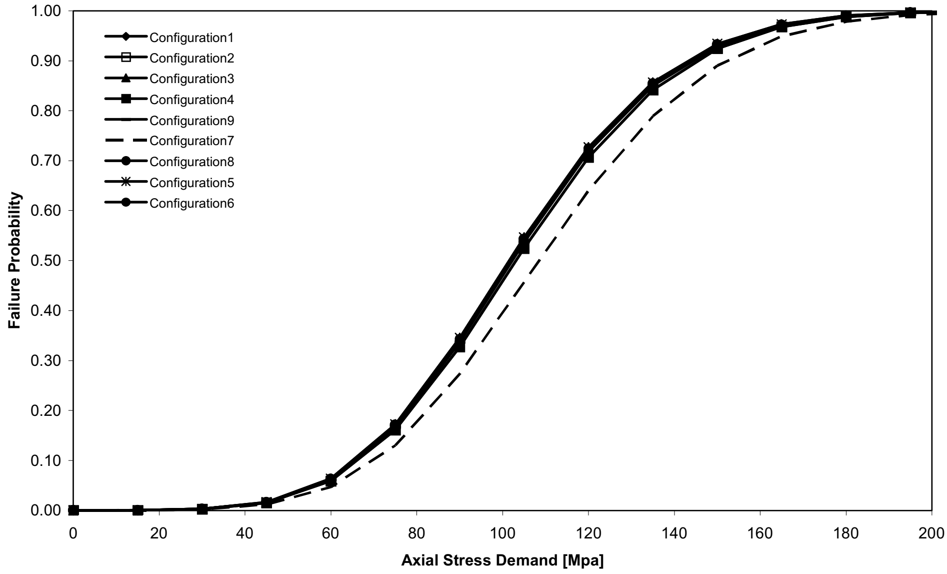

Fig. 8. Buckling capacity.

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} which is given in Fig. 8.

3.4. *Fragility*

Fragility curves of each considered configuration have been approximated by lognormal distribution and are reported in Fig. 9.

The median values strongly depend on the filling level; in particular, the lower 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 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:

$$\text{Fragility} = LN\left(\mu\left(\frac{H}{r}, f\right), \beta\left(\frac{H}{r}, f\right)\right), \tag{13}$$

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

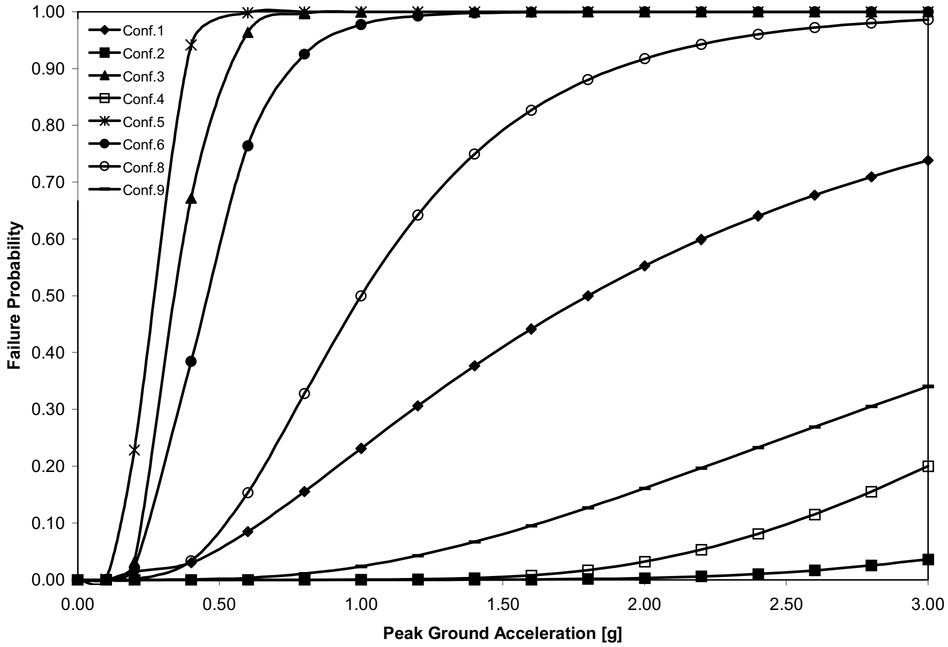

Fig. 9. Fragility.

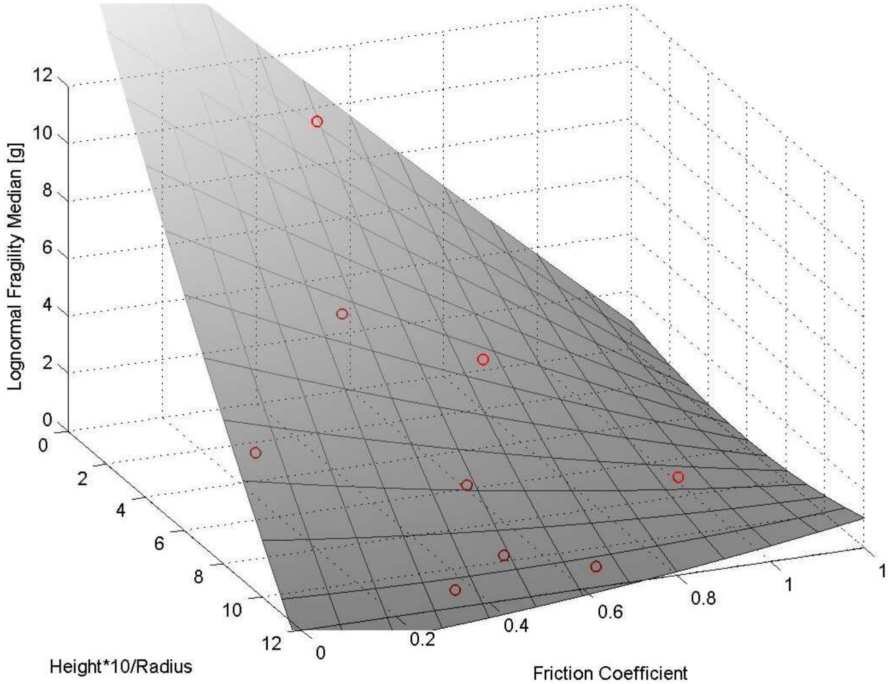

Fig. 10. Second order fitting for fragility median.

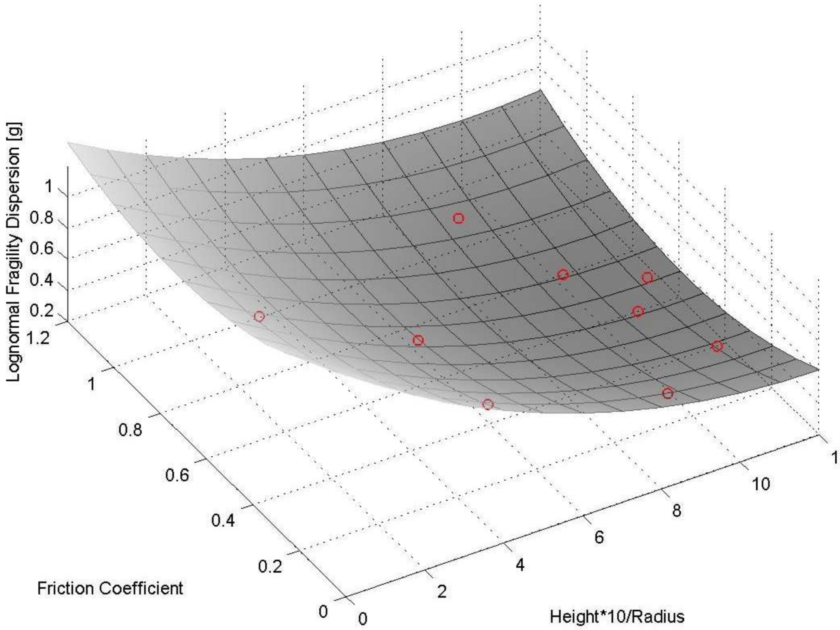

Fig. 11. Second order fitting for fragility standard deviation.

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 seismic behaviour.

A complete second order model was optimised by least squares method to fit their variability. Results are plotted in Figs. 10 and 11.

Dotted points in Figs. 10 and 11 are 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 standard deviation ($\mathrm{R}^2 \sim 70\%$) but still good enough to fully capture the influence of the structural parameters on the fragility and then predict it.

Results show how the reduction of filling level correspond 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 for recognition that the higher the failure median intensity, the higher the dispersion is. This result confirms that an increase intensity levels in Dynamic Incremental Analyses of nonlinear systems leads to a large scatter of the output around the medians.

4. Final Remarks

The procedure proposed in the present paper enables effective seismic risk estimation for a large number of 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 (i.e. tanks) 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. Furthermore, 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.

Applications to steel tanks for oil storage show that the procedure is simple to apply even though it requires dynamic modelling. With regards to the elephant foot buckling limit state the capability of the method to capture functional relationship among fragility and response affecting parameters such as filling level has been shown.

However, the results cover at the present stage a single class of the unanchored steel tanks and have been generated according to a structural model that has been optimised in terms of structural input, computational time, and response validation.

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