USE OF INCREMENTAL DYNAMIC ANALYSES FOR NUCLEAR FRAGILITY ASSESSMENT

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Abstract: A fragility curve expresses the conditional probability of failure of a structure for a given input motion parameter, such as the peak ground acceleration or spectral acceleration. In nuclear engineering practice, fragility curve is evaluated by using margin factors. This approach represents a very convenient method but it adopts strong simplifying hypotheses. In the last years, fragility curves have also become very popular for assessing the seismic vulnerability of civil structures and one of the best current practices is the use of non-linear dynamic analyses. In this context, incremental dynamic analyses have been used in conjunction with a pushover analysis to assess a building used as a storage facility for vehicles and equipment, within a nuclear power plant in UK. The main functional requirement of the building is to provide a facility to store and maintain back-up and emergency response equipment that may be required by emergency responders for the nuclear power station.

Introduction

Following the Tohuku earthquake in Japan on 11 March 2011, a review of the UK's nuclear industry has been carried out by the Office for Nuclear Regulation (ONR) using the lessons learnt from the Tokyo Electric Power Company (TEPCO) Fukushima-Daiichi station. This review stated that there were no fundamental safety weaknesses in the UK's nuclear industry, but also concluded that using the lessons learnt from the event the industry can be made even safer. A document titled 'Japanese earthquake and tsunami: Implementing the lessons for the UK's nuclear industry' (ONR, 2012) was produced after the ONR review. This document identifies the requirement for an emergency equipment store.

The main functional requirement of the building is to provide a facility to store and maintain back-up and emergency response equipment that may be required by emergency responders for the nuclear power station. Vehicles and equipment may be required to respond to Beyond Design Basis (BDB) events, Design Basis events, and to non-nuclear related events on site. In particular, the facility has to be able to perform its main functions following a BDB event, and the equipment stored within the facility must be protected so that it is able to function following the same event. The design life of the building is 70 years.

It is therefore necessary to establish and quantify the applicable BDB events to ensure the building design demonstrates resilience. The most common method of quantifying a BDB event is by defining the Design Basis (DB) event and then adding a suitable margin. The BDB indicates that the building design should demonstrate that there is a High Confidence (at 95th percentile) of Low Probability (less than 5%) that the building can fulfil its minimum functions, following a Peak Ground Acceleration (PGA) of 4m/sec^2. To reach this purpose a fragility curve, which expresses the conditional probability of failure of a structure given a ground motion intensity measure is required.

The most common approach adopted in the nuclear engineering practice to carry out a fragility curve is a simplified method called "response factor method" (Reed et al., 1994). It represents a very convenient method to calculate the fragility curves, however, it adopts strong simplifications.

During last years, fragility analyses have become very popular also in the civil engineering applications to evaluate the vulnerability of structures. Within this context, the growth in

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computer processing power has made possible the use of more complex and accurate analyses. Hence, the state of the art has moved from static to non-linear dynamic analyses.
This paper is structured in a way that, the response factor method, usually adopted to carry out fragility curves is described first. Then, the procedure to calculate the fragility curve adopting non-linear dynamic analyses is illustrated and numerical results are provided for the building in terms of fragility curves at different confidence levels.
Because of the confidentiality of the project, specific information about the location and geometry of the building will not be provided.

The response factor method

A fragility curve expresses the probability of failure (P_f) of a structure as a function of an intensity measure (e.g., peak ground acceleration (PGA) or Spectral acceleration ($\mathrm{S_a}$)). The failure probability conditioned on a ground motion parameter, a, is given by the cumulative distribution of the capacity, A, that is considered as a random variable. In fact, a structure fails if its capacity is equal or less than a given ground motion value.
The functional form usually adopted to describe a fragility curve is the lognormal distribution (Equation 1) which is defined by two parameters: the median $\mathrm{A_m}$ and the logarithmic standard deviation β. In the same equation, $\phi(\cdot)$ represents the standard Gaussian cumulative distribution function.

$$P_{f|a}(a) = \int_0^a \frac{1}{x\beta\sqrt{2\pi}} e^{-\frac{1}{2}\left(\frac{log(x/A_m)^2}{\beta^2}\right)} dx = \phi\left(\frac{ln(a/A_m)}{\beta}\right) \quad (1)$$

Also the response factor method (Reed and Kennedy, 1994) adopts the previous formulation, but it also distinguishes between epistemic uncertainty (that accounts for the lack of knowledge about the procedure) and aleatory randomness (e.g., the variability of the PGA for an earthquake of a certain magnitude and at a certain location) by introducing the log-standard deviations β_U and β_R respectively. In this context, the capacity can be expressed as in Equation 2, where ε_u and ε_r are log-normally distributed random variables with median equal to one and respective log-standard deviations β_U and β_R.

$$A = (A_m \cdot \varepsilon_u) \cdot \varepsilon_r \quad (2)$$

Starting from the above hypotheses, it is possible to define a family of fragility curves that refer to different confidence levels, Q (Equation 3).

$$P'_{f|a}(a) = \phi\left(\frac{ln(a/A_m)+\beta_U\phi^{-1}(Q)}{\beta_R}\right) \quad (3)$$

Equation 1 (which represents the mean curve) and Equation 3 (which for $Q = 0.5$ provides the median curve) are linked by the following expression (Equation 4).

$$\beta \equiv \beta_C = \sqrt{\beta_R^2 + \beta_U^2} \quad (4)$$

In this method, in order to evaluate the fragility parameters, intermediate random variables (i.e., safety factors) are used. In particular, the capacity, A, is expressed as in Equation 5 where $\mathrm{a_{design}}$ is the PGA of design and F_i are the random margin factors, lognormally distributed (with median $\hat{F}$ and log-standard deviation β_i) that account for the conservatism and uncertainty in structural response and capacity calculations.

$$A = \left(\prod_{i=1}^{P} F_i\right) a_{design} \quad (5)$$

Thus, it results that the median capacity can be calculated as $A_m = \hat{F} \cdot a_{design}$ with $\hat{F} = \prod_i \hat{F}_i$ while the log-standard deviation is given by Equation 6.

$$\beta = \sqrt{\sum_i \beta_i^2} \; where \; \beta_i = \sqrt{\beta_R^2 + \beta_U^2} \quad (6)$$

Because of its simplicity, this approach is very very convenient. As this method is based on strong hypothesis (e.g. lognormally distributed safety factors) and expert judgment it may not provide robust results.

Fragility curve using nonlinear dynamic analysis
Currently, non-linear dynamic analyses (e.g., Incremental Dynamic Analyses, IDAs) have become very popular in the civil engineering applications to calculate fragility curves of structures. These analyses allow evaluation of the seismic response of a structure and in conjunction with a structural model, allow the calculation of the probability of failure, that is, the probability of exceedance of the seismic capacity, for different intensity levels. These probabilities (that already represent an empirical fragility curve) may also be approximated by a lognormal distribution (Equation 1) whose parameters can be calculated through a regression process.

Incremental Dynamic Analysis
An IDA (Vamvatsikos and Cornell, 2002) involves a series of non-linear dynamic analyses of the structural model under a set of ground motion records, each scaled to several intensity levels ideally selected to cover the whole range from elastic to non-linear and finally to collapse of the structure. In particular, the goal of the analysis is to record the damage state of the structure measured by an engineering demand parameter (EDP, e.g., peak roof drift ratio or inelastic displacement) for each intensity level which is measured by an intensity measure (IM, e.g., peak ground acceleration or the 5% damped first-mode spectral acceleration $S_a(T_1)$). Results can be processed to get the distribution of demand EDP given the intensity level IM. Hence, the fragility curve can be assessed by calculating for each IM the probability of exceedance of the seismic capacity.
The most important issue to conduct IDAs is selecting a suitable IM and EDP (Luco et al., 2007). There are several issues of efficiency and sufficiency associated with the IM selection. Sufficiency is defined as the independence of the distribution of EDP given the IM from any other seismological parameters that may characterize the ground motion (e.g., duration, magnitude or spectral shape). A sufficient IM accounts for all seismological information needed to determine the effect of a ground motion record on the structure being investigated and this permits a linear scaling of records to reach the intensity level considered in the analysis. An efficient IM instead, minimizes the scatter of results, therefore it is required a reduced number of ground motion records to provide good demand and capacity estimates. For first-mode-dominated structures, the 5% damped first-mode spectral acceleration, $S_a(T_1)$, is chosen as standard IM (Shome et al 1998, Shome and Cornell 1999).
On the other hand, the EDP has to be selected in order that it can well represent the damage of the structure. Hence, the peak storey accelerations are usually adopted to describe contents' damage, while the maximum peak interstorey drift ratio or the inelastic displacement are used to describe global dynamic instability and several structural performance limit-states.

Structural model
In order to evaluate the capacity of the structure, a pushover analysis can be adopted (e.g., D'Ayala et al. 2014). The pushover analysis of a structure is a static non-linear analysis under permanent vertical loads and gradually increasing lateral loads (incremental loads) up to failure. The equivalent static lateral loads approximately represent inertial forces induced by the earthquake. The output of this analysis is therefore the capacity curve (force-displacement relationship) of a Multi Degree of Freedom (MDoF) system with the estimated damage states.
In order to conduct the fragility assessment, this curve has to be converted into a capacity curve for the equivalent Single Degree of Freedom (SDoF) system. The definition of the equivalent SDoF from a pushover curve can be obtained by using the approach followed in the classical N2 method (Fajfar, 2000) and implemented in Eurocode 8 (CEN, 2004).

This transformation is made by dividing the base shear and displacement of the MDoF system with a transformation factor (the first mode participation factor, Γ) that can be calculated according to Equation 7, where m^* is the mass of the equivalent SDoF (defined as $m^* = \sum m_i \phi_i$) while ϕ is the fundamental mode shape.

$$\Gamma = \frac{m^*}{\sum m_i \phi_i^2} \tag{7}$$

For the calculation of the equivalent mass m^* and the factor Γ, the assumed displacement shape ϕ is normalized, (i.e., the value at the top is equal to 1).
Then, in order to perform non-linear dynamic analyses, the pushover curve for the equivalent SDoF has to be idealized. In particular, depending on the shape of the pushover curve, this can be idealized by a bilinear (an elastic-plastic) or multilinear (elastic-plastic with residual strength) model. To derive the idealized capacity diagram of the SDoF system, the equal energy principle can be used, that is, the idealized curve is determined by imposing that the areas under the actual curve of SDoF and the idealized curve are equal.

Regression process
Once the non-linear dynamic analyses have been performed, the conditional distribution of the EDP given the intensity level, IM, can be calculated. Hence, the probability of failure, that is, the probability of exceedance of the seismic capacity for each IM can be assessed.
These probabilities already represent an empirical fragility curve. However, results may also be fitted by a cumulative lognormal distribution (Equation 1) whose parameters can be assessed by adopting a regression process (Porter, 2007).
In fact, it is possible to convert Equations 1 to a linear regression problem by taking the inverse Gaussian cumulative distribution function of each side and fitting a line (Equation 8) to the data.

$$y = sx + c \tag{8}$$

In the above equation, s is the slope of the trend line and c is the value of y where the line has a x-value of 0 (the intercept). Parameters of fragility curve are related to the fitting line; in fact, the β value correspond to $1/s$, while $A_m = -c/s$. Obviously, parameters can be defined also for different confidence levels.

Results for the case study
Incremental Dynamic Analyses and results of the pushover analyses have been used to perform the fragility assessment of a storage facility building within a nuclear power plant in UK. The structure is a regular one floor steel building and has been modeled through the SAP2000 software. The collapse limit state has been defined according to ASCE/SEI 43-05, (i.e., it is assumed that the structure reaches the failure when the inelastic energy absorption factor F_μ reaches a value equal to 2).
The elastic modal analysis indicates that the first mode participating mass (m_p) is equal to 82% for the longitudinal direction and equal to 95% for the transversal direction, that is, the building can be considered as a first-mode-dominated structure. Hence, the 5% damped first-mode spectral acceleration $S_a(T_1)$ has been selected as intensity measure (sufficient and efficient given the characteristic of the structure) for conducting the non-linear dynamic analyses, whereas the inelastic displacement (Δ_{in}) has been chosen as EDP.
In order to conduct the fragility assessment, the first mode participation factor (Γ) has been evaluated for each direction (according to Equation 7) to transform the pushover curve of the MDoF system to the capacity curve of the equivalent SDoF (both displacement and base shear have been divided by Γ). According to the shape of the pushover curve, a bilinear approximation, considering an hardening behaviour after the achievement of the yielding force, F_y (De Luca et al. 2013) has been used. Figure 1 and Figure 2 show the pushover curves (idealized) for the equivalent SDOF systems, for the longitudinal and transversal directions respectively.

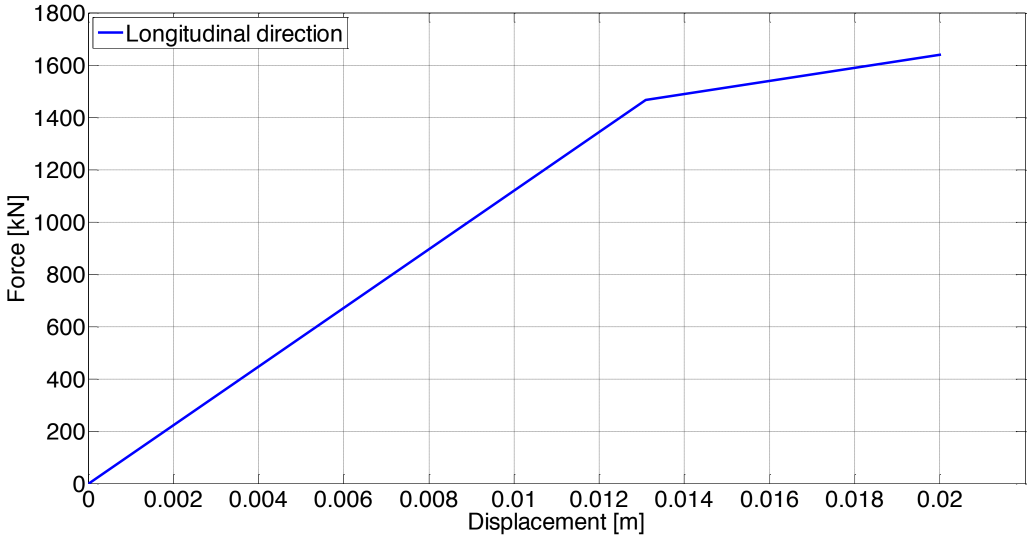

Figure 1. Linearized pushover curve for the equivalent SDOF (longitudinal direction)

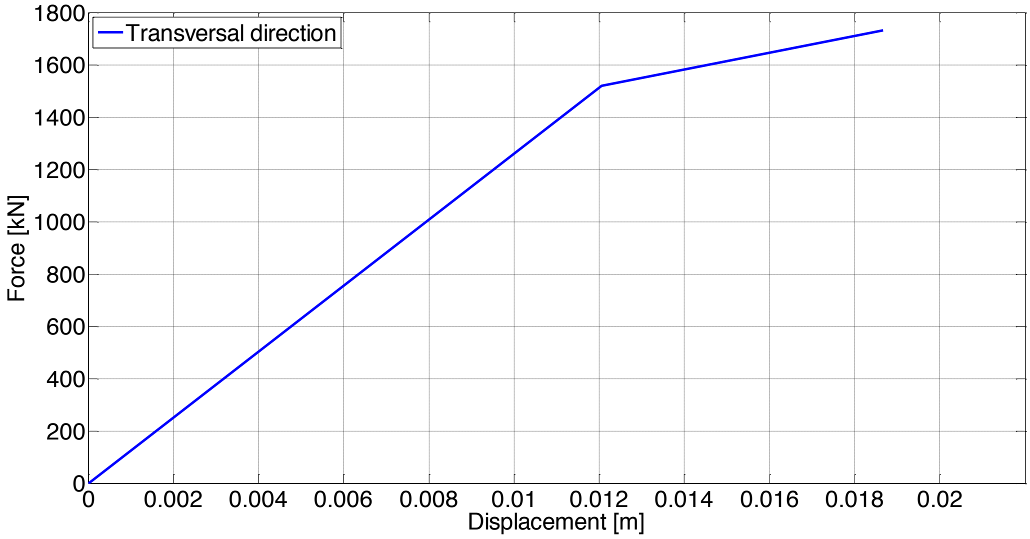

Figure 2. Linearized pushover curve for the equivalent SDOF (transversal direction)

Table 1 summarizes for each direction, the characteristics of the equivalent SDOF systems, that is: the first mode participation factor (Γ), the yielding force (F_y), the capacity force at collapse (F_u), the yielding and ultimate displacements (d_y and d_u respectively), the hardening value (h), the first mode participating mass (m_p) and finally the stiffness (k^*), mass (m^*) and period (T^*) of the equivalent SDoF.

Table 1. Parameters of the equivalent SDOF

Parameters of the equivalent SDoF	**Longitudinal direction**	**Transversal direction**
Γ	1.368	1.2
$F_y\ [kN]$	1465	1518
$F_u\ [kN]$	1640	1732
$d_y\ [m]$	0.013	0.012
$d_u\ [m]$	0.020	0.018
h	0.227	26750
$m_p\ [\%]$	82	95
$m^*\ [kN \cdot sec^2/m]$	130.583	100
$k^*\ [kN/m]$	111832	126500
$T^*\ [sec]$	0.215	0.177

The set of ground motions used to perform the non-linear dynamic analyses is based on the FEMA P695 far-field ground motion set, which includes 22 record pairs, each with two horizontal components for a total of 44 ground motions. Those ground motions are recorded at sites located greater than or equal to 10 km from fault rupture; event magnitudes range from M 6.5 to M 7.6 with an average magnitude of M 7.0.

The scaling factor to be applied at each record for each IM has been calculated through the evaluation of the exact spectral response at the period of the SDoF by using OpenSees and Matlab software.

Since the structure can be defined regular, IDAs have been performed independently in the two directions. Figure 3 and Figure 4 show the IDAs results for the longitudinal and transversal directions respectively.

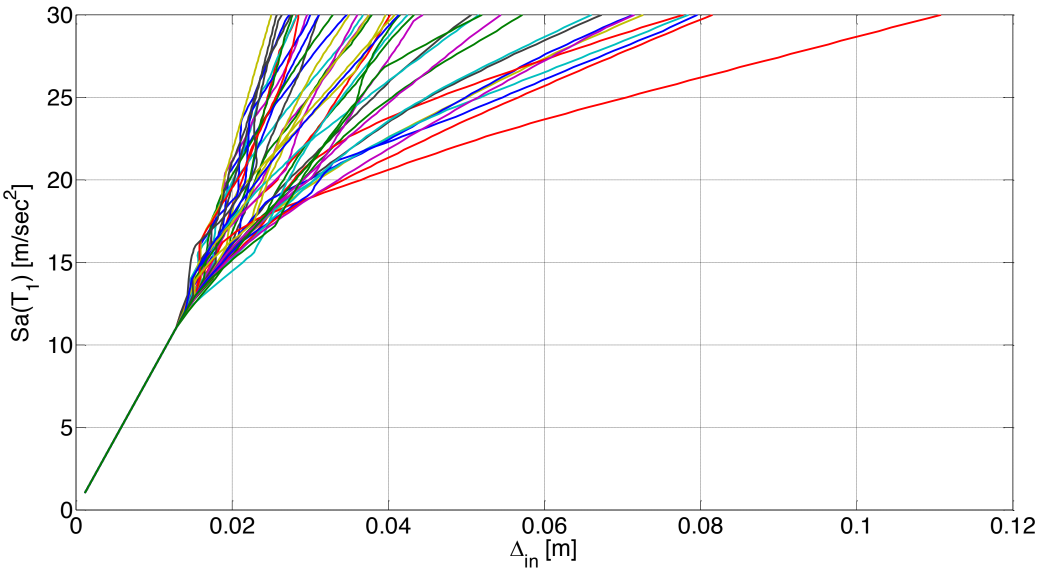

Figure 3. Incremental Dynamic Analysis for the longitudinal direction

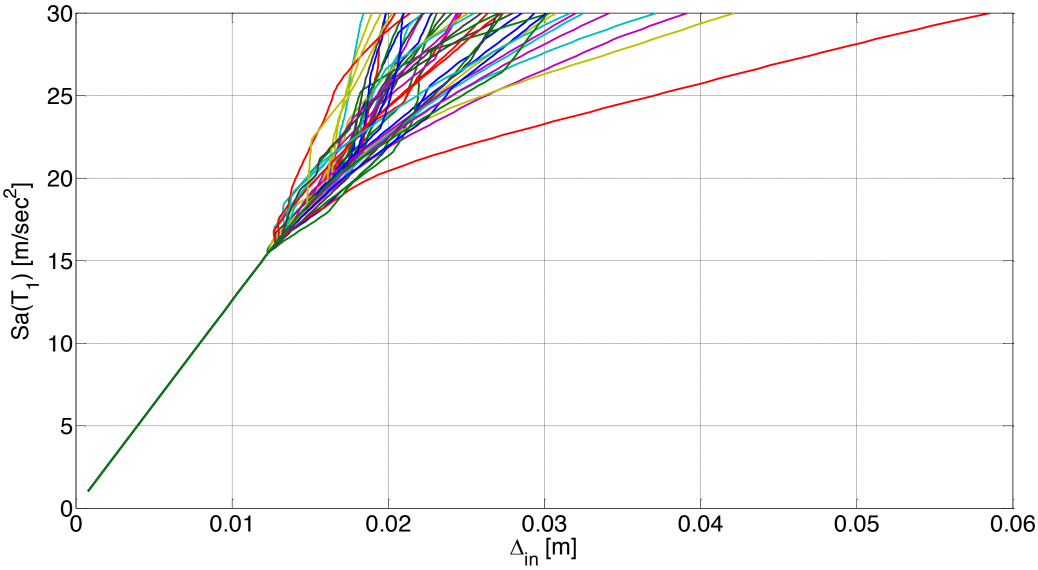

Figure 4. Incremental Dynamic Analysis for the transversal direction

Once the IDAs have been performed, the conditional probability density function (assumed as lognormal) of the inelastic displacement, Δ_{in}, given each intensity level has been evaluated. Hence, the probability of failure, that is, the probability of exceedance of the seismic capacity has been assessed.

The empirical fragility curve is then fitted with a lognormal distribution (Equation 1) whose parameters have been defined by using the regression procedure described in the previous section.

Curves have been calculated for different confidence levels (i.e., for the 50th 5th and 95th percentile). Figure 5 and Figure 6 show the family of fragility curves obtained for the longitudinal and transversal directions respectively.

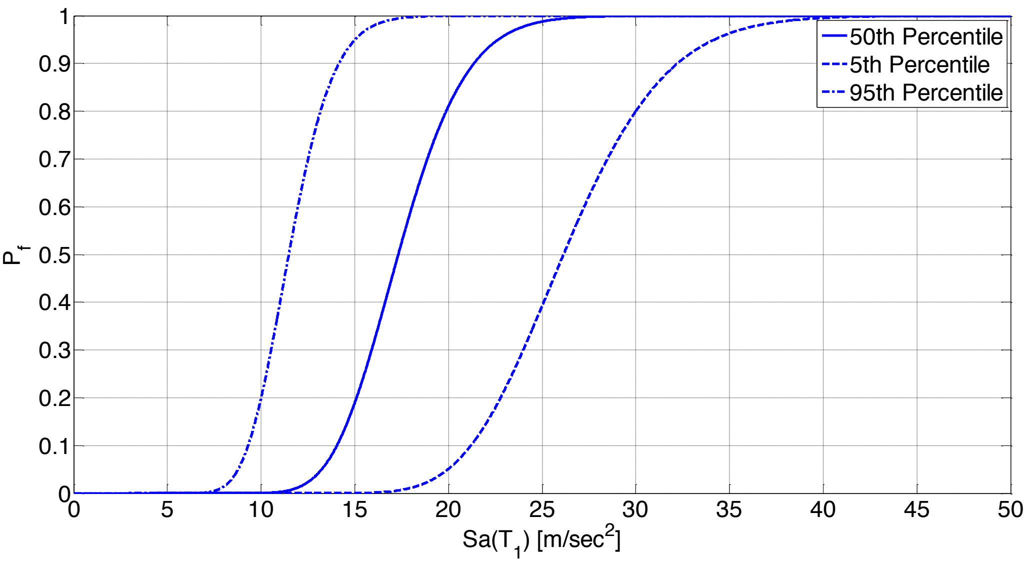

Figure 5. Fragility curves for the longitudinal direction

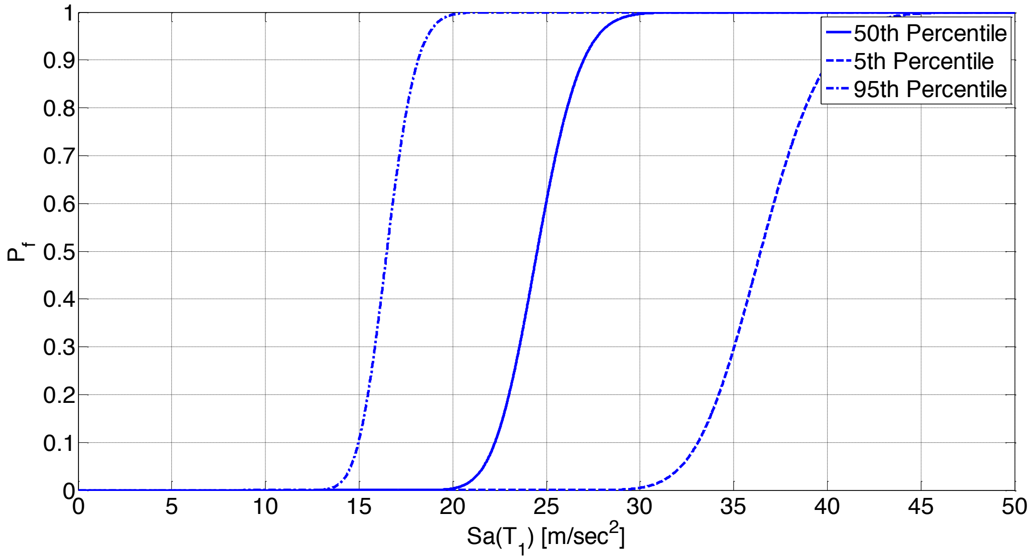

Figure 6. Fragility curves for the transversal direction

In order to perform the fragility assessment of the building, the seismic demand, that is, the design $S_a(T_1)$ value of the reference code spectrum (with a $PGA = 4\, m/sec^2$) has been evaluated for each direction. Values are equal to 8.2 m/sec^2 for the longitudinal direction and 10.2 m/sec^2 for the transversal direction.
For these values, the probabilities of failure have been calculated considering the fragility curves at 95th percentile. The worst condition (reached for the longitudinal direction) has provided a probability of failure equal to $\mathrm{P_f} = 2\%$. This result shows that the design criteria of the building respect the High Confidence of Low Probability (less than 5%).

CONCLUSIONS
A fragility curve expresses the conditional probability of failure of a structure for a given input motion parameter, such as the peak ground acceleration or spectral acceleration. In nuclear engineering practice, fragility curve is usually evaluated by using margin factors. This approach represents a very convenient method but it adopts strong simplifying hypotheses. In the last years, fragility curves have also become very popular for assessing the seismic vulnerability of civil structures and one of the best current practices is the use of non-linear dynamic analyses.
In this context, incremental dynamic analyses (IDAs) have been used to perform the fragility assessment of a building used as a storage facility for vehicles and equipment within a nuclear power plant.
As the structure is regular and first mode dominated, IDAs have been performed in the two directions independently, considering the first mode spectral acceleration value as intensity measure (IM) while the inelastic displacement has been selected as engineering demand parameter (EDP). IDAs results have allowed the calculation of the conditional distribution of the EDP given the IM (assumed as lognormal). Hence, the probability of exceedance of the seismic capacity has been assessed for each intensity level.
These results, that already provide an empirical fragility curve, have been fitted by a cumulative lognormal distribution whose parameters have been defined by using a regression process. Fragility curves have been evaluated for each direction and for three different confidence levels (50th, 5th and 95th percentile).
Finally, considering the fragility curve at 95th percentile and the design spectral acceleration value of the reference code spectrum, the probability of failure for each direction has been calculated. The worst result, reached for the longitudinal direction, has provided a probability of failure equal to 2%. This has led to the conclusion that the building design, following a

Beyond Design Basis event, demonstrates a High Confidence (at 95th percentile) of Low Probability (less than 5%) of failure.

FURTHER THOUGHTS

Resilience of critical facilities has always been one of the major challenges for Facility Owners of structures exposed to extreme events, as the sole reliance on National Codes is insufficient. Eurocode, itself, recommends the use of a risk analysis approach together with hazard identification for assessment of extreme events. The most common method of qualifying the resilience of critical facilities in extreme events is by using simplified approaches that adopts safety factors to define parameters of the fragility curves. However, recent research has highlighted the potential use of non-linear dynamic analyses for the following reasons:

- Extra rigour and thought is required to highlight failure modes and weaknesses in the structure.
- It allows a comparison of different design solutions which leads to a more transparent decision making and robust design.
- The approach permits the main uncertainties (and “weak links”) to be identified for the specific structure, which can be dealt with directly by practical risk reduction measures.

One of the recommended probabilistic approaches to assess a structure is the calculation of fragility curves by using non-linear dynamic analysis. This method is a robust and elegant procedure compared to the current practice as it accounts for the uncertainty related to the seismic ground motion, and eventually, epistemic uncertainties related to the geometry or the characteristics of the materials of the specific structure. This approach has been extensively used in the insurance sector for financial modelling (especially for civil structures) and can be used for other high risk industry.

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