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MULTI-DIMENSIONAL FRAGILITY ANALYSIS OF A RC BUILDING WITH COMPONENTS USING RESPONSE SURFACE METHOD

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ABSTRACT

Conventional fragility curves describe the vulnerability of the main structure under external hazards. However, in complex structures such as nuclear power plants, the safety or the risk depends also on the components associated with a system. The classical fault tree analysis gives an overall view of the failure and the interaction between the components and structure, but it fails to show the cumulative distribution of the failure. As an alternative, Cimellaro et al. suggested a method, which combines the fragility of the structure and the components, by using multidimensional performance limit state functions. This approach gives the possibility of deriving the cumulative fragility taking into account the interaction of different components. Here we use this approach to evaluate seismic vulnerability of a representative electrical building infrastructure, including the components, of a nuclear power plant. A simplified model of the structure, which represents the nonlinear material behavior is modelled using Abaqus©. The input variables considered are the material parameters, boundary conditions and the seismic input. The variability of the seismic input is obtained from selected ground motion time histories of spectrum compatible synthetic accelerograms. Unlike the usual Monte Carlo methods used for the probabilistic analysis of the structure, a response surface method is used. This method reduces the computational effort of the calculations by reducing the required number of samples.

INTRODUCTION

In complex structures such as nuclear power plants, the failure of the plant depends on the structural failure as well as system and/or component failures which are critical to the safety. Seismic probabilistic safety/risk assessment (SPRA) methods are widely used for the safety analysis in nuclear power plants. The seismic fragility curves describe the cumulative distribution of probability of failure of the structure or component conditional to ground motion intensity. The various methods for the development of fragility curves of structures are found in literature by several authors, for brevity not all are listed here. A thorough compilation of development of fragility curves can be seen in Calvi, et al. (2006). The major difference in the methodology being, in the consideration of uncertain parameters, uncertainty in the earthquakes, selection of seismic indicators and numerical or statistical methods. Empirical fragility curves using existing damage data was proposed by Shinozuka et al. (2000b), for bridge columns in Japan. Kennedy, et al. (1980) developed seismic fragilities for critical structures and equipment of Nuclear power plant (NPP) using available data and engineering judgement (J. Reed, et al. 1991). However due to limited availability of damage data, analytical fragility curves were proposed from the structural response under seismic load. The analytical fragility curves derived for various structural systems are found in the literature. Shinozuka et al. (2000a) compared the analytical fragility curves obtained for the bridge using time-history analysis and capacity spectrum method. The development of seismic fragilities in NPP using different methods are well explained in Reed and Kennedy (1994), apart from the empirical methods and engineering judgement fragility curves, analytical fragility development using Monte Carlo simulation is included in it. Recently, it is seen in the literature that, fragility curves are developed using numerical simulations, however, most of the analysis are performed on simplified models. Consideration of uncertainty is also crucial in critical structures. More recently Coleman (2014), has shown the nonlinear structure interaction effects on the nuclear facilities considering uncertainty in seismic input. Zentner (2010) developed fragility curve for NPP equipment using non-linear dynamic analysis considering uncertainties in the model as well as uncertainty in the seismic load using Monte Carlo methods.

Monte-Carlo simulations are the commonly used methods for the structural simulations in probabilistic analysis procedures. In order to obtain an accurate failure probability P_f, the number of samples, n, required to be used at least in the order of 1/ P_f (Gupta and Manohar 2004). The input variables are combined randomly and deterministic analyses are performed on the structure for each sample. For complex structures, it is computationally impractical to perform Monte Carlo simulation for nonlinear time history response of the structure, even with present day computational capacity. Improved sampling methods like Latin Hypercube sampling, which divides the sampling space into sectors of equal probability of occurrence and selects one realization from each sector, has been used by several researchers for example, Hwang and Huo (1994), Song and Ellingwood (1999), Zentner (2010) etc. Use of surrogate models or meta models for the complex structures are becoming popular in every field. Towashiraporn (2004) used response surface method as a metamodel in combination with Monte Carlo simulation for the fragility analysis of masonry structure. An improved response surface method proposed by Gupta and Manohar (2004) optimizing the response surface in such a way that it includes significant response points which contribute to failure probability.

As a part of the SPRA, the probability of failure of the whole power plant is then determined by the Fault tree analysis which is generally used for the safety and reliability analysis (Paoazoglou 1985). The classical fault tree analysis represents the failure and combination of failure which leads to an undesirable event, assuming that the events are binary and are statistically independent, however complex interactions are not very well represented. As an alternative Rao, et al. (2009) suggested a dynamic fault tree analysis for nuclear power plant incorporating complex interaction by introducing dynamic gates in the fault tree. Other variation of dynamic fault tree was suggested by Čepin and Mavko (2001) which included the time dependent behavior of the nuclear power plant. However, the complex interactions make it difficult to understand and the fault tree analysis is not able to represent the cumulative distribution of failure, like a fragility curve.

The fragility curves developed for the systems should contain, the fragility of the components and the structure and their interaction. A multi-dimensional fragility analysis was suggested by Cimellaro, Reinhorn, et al. (2006), which combines the different failure modes of the components and structures using multidimensional performance limit state function. The method considers multiple limit state parameters, which can be the limit state of the response of component and of structure. They method was applied to a hospital building in California, and the influence of the uncertainty in the performance limit state on the fragility analysis is also illustrated in Cimellaro and Reinhorn (2010). The same approach was utilized by Wang et al for the seismic fragility of highway bridges considering two damage parametrs, column ductitlity and transverse deformation of the abutments.

In this paper, the multi-dimensional fragility curves are developed for a reinforced concrete test structure used in smart 2013 project, with fictitious components, with a computationally effective response surface method. The nonlinear material models are considered for the structure. The multidimensional performace limit state is evaluated using the generalized formula suggested by using Cimellaro et al 2006. The multidimensional performance limit state considered for the fragility curve development based on the assumtions on the interdependencies of the responses. For the response surface method , 40 samples are used for 30 set of synthetic accelerograms. The method is recommended for the system related fragility curve development.

MULTIDIMENSIONAL FRAGILITY

The seismic fragility describes the conditional probability that the response of the structure or the component exceeds a given limit state under various intensity of seismic excitation. Mathematically, the fragility $P_f(\Theta)$ for a particular earthquake intensity Θ can be described as given in Equation 1,

$$P_f(\Theta) = P\{R \geq s|\Theta\} \tag{1}$$

Where R is the random variable of the response, for example deformation or acceleration, s is the threshold limit state according to a given degree of damage and Θ is the intensity of seismic excitation such as return period, peak ground acceleration (PGA) or peak ground velocity (PGV). This can be extended for N number of parameters suggested by Cimellaro et al.(2006) and is given in Equation 2,

$$P_f(\Theta) = P\{R_1 \geq s_1 \cup R_2 \geq s_2 \cup R_3 \geq s_3 .... \cup R_N \geq s_N|\Theta\} = P\left\{\bigcup_{i=1}^{N} R_i \geq s_i|\Theta\right\} \tag{2}$$

Where R_i is the response parameter related to deformation acceleration, s_i is the threshold parameter for a given damage corresponding to the response. In this paper the performance limit state considered are the inter story drift of the reinforced concrete structure and the acceleration of the generator. Therefore, this can be simplified to as shown in Equation 3

$$P_f(\Theta) = P\{\Delta \geq \delta_s \cup A \geq A_s|\Theta\} \tag{3}$$

Where Δ is the inter story drift response variable, *A* is the acceleration response of the generator, δs, A_s are the given threshold for inter story drift and acceleration respectively. Therefore, the response in this curve can be represented using a bell surface (Cimellaro and Reinhorn, et al. 2006) in the acceleration – inter story drift plane. The surface is created with the two variables inter story drift and acceleration responses with a joint probability density function. The maximum responses are assumed to be lognormal distributed

Multidimensional performance limit state

A generalized formulation was suggested by Cimellaro and Reinhorn, et al. (2006), as a tool to consider multiple limit states associated with different response quantities in the same formulation. Different response paremeters like stresses, displacements, velocities, acceleration can be considered to form a combined performance limit state, and hence a unique fragility curve can be plotted for the given system with strcuture and components or the entire facility. The generalized formula (Cimellaro and Reinhorn, et al. 2006) for the multidimensional performance limit state (MPLS) is given as in Equation 4

$$L(R_1, ... R_n) = \left(\frac{R_1}{s_1}\right)^{N_1} + \left(\frac{R_2}{s_2}\right)^{N_2} + ... + \left(\frac{R_n}{s_n}\right)^{N_n} - 1 = 0 \tag{4}$$

Where R_i is the response parameter, s_i response threshold parameter corresponding to damage and N_i is the interaction factor which depicts the dependency of the random variable describing the performance limit state and the shape of the limit state curve. We consider only two response parameters for this study, therefore the equation 4 can be simplified to the following Equation 5

$$\left(\frac{\delta_{LS}}{\delta_{LS0}}\right)^{N_\delta} + \left(\frac{A_{LS}}{A_{LS0}}\right)^{N_a} - 1 = 0 \tag{5}$$

Where δ_{LS0} represents the independent peak inter story drift limit state, A_{LS0} is the independent peak acceleration limit state, δ_{LS} represents the dependent peak inter story drift limit state, A_{LS} is the dependent peak acceleration limit state, and N_δ and N_a are the interaction factors determining the shape of the limit state. A further simplified equation is obtained when the interaction factor for inter story drift is assumed to be equal to 1 as given in Equation 6

$$\frac{A_{LS}}{A_{LS0}} + \left(\frac{\delta_{LS}}{\delta_{LS0}}\right)^{N} - 1 = 0 \tag{6}$$

Where δ_{LS0} A_{LS0} are independent quantities and are calculated from field data collected or experimental laboratory test. It can be considered as random or deterministic. Limit states can either be linear or nonlinear and is dependent on the value of *N*. *N* is determined using the comparison with experimental data and judgement. *N* takes the real number greater than or equal to 1. When *N* takes the value 1, the limit state is a line between acceleration and drift which represents the velocity. When *N* is greater than 1 there is a nonlinear relationship. When acceleration and interstory drift are considered as unrelated, then N tends to infinity and the two parallel lines as shown in the Figure 1 a.

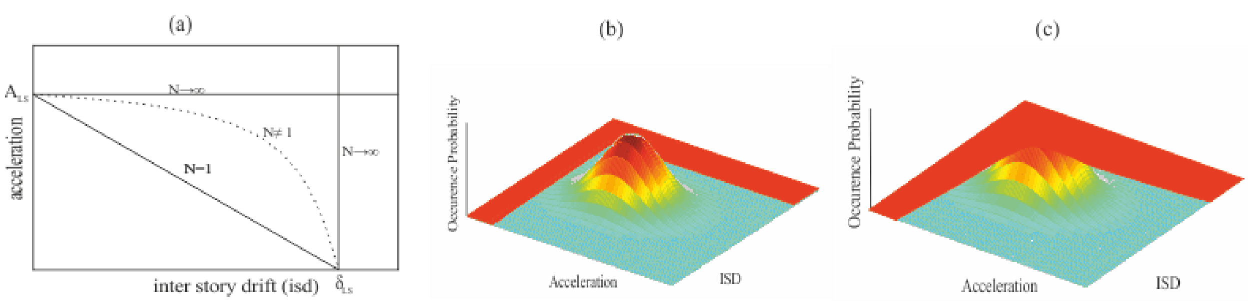

Figure 1 (a) influence of N on the limitstate. (b) Simplified nonlinear model (c)Position of generator

Multidimensional fragility curve development

The maximum response of the acceleration of the component and interstory drift of the structure are computed for the given set of time histories. The bi-dimensional performance limit state is the limit state for the system. The responses obtained from several samples for one intensity are assumed to be lognormal distributed (Cimellaro, Reinhorn, et al. 2006), with a mean μ and standard deviation σ of the associated normal distribution are estimated using regression analysis. The fragility curves are calculated, based on the assumption whether the response variables are stochastically independent or dependent. The lognormal probability density function (pdf) of different variables are combined to get the pdf of the multidimensional responses. In inelastic range, it is assumed that there is no relationship between the interstory drift and acceleration (Cimellaro, Reinhorn et al. 2006), hence the random response can be considered independent. In this paper the inelastic material behavior is considered for the structure. However, two cases are considered here, for the fragility analysis, first as independent response and then with dependent response and the fragility results are compares. Therefore, the probability of exceeding the performance limit state is calculated by integrating the area under the limit state curve as given by Equation 7

$$P\{\delta \geq \delta_s \cup A \geq A_s\} = \iint_s f(\delta) f(A) d\delta dA \tag{7}$$

The domain s is the shaded area in the Figure 1b for the dependent case as shown in Figure 1c and is mathematically represented as in the Equation 8. When the response variables are assumed to be independent, then the probability of exceeding the limit state can be evaluated from the cumulative distribution functions of both the acceleration and the interstory drift responses. In the case when the variables are dependent, then the number of times N_f, that the maximum acceleration and the maximum ISD exceed a given performance threshold and divide by the total number of trials n related to the respective intensity, so the probability P_f of reaching or exceeding the performance limit state is obtained as N_f/n.

$$\frac{A}{A_{LS0}} + \left(\frac{\delta}{\delta_{LS0}}\right)^N - 1 \geq 0 \qquad (\delta \geq 0, A \geq 0) \tag{8}$$

The procedure is repeated for each intensity and finally the lognormal distribution fragility is plotted using maximum likelihood method or regression analysis.

RESPONSE SURFACE METHOD

Monte Carlo simulations are computationally expensive to get accurate results if complex structures are involved in the calculations with time history analysis. The use of metamodels reduces the computational effort considerably without losing much of the accuracy (Towashiraporn 2004). Response surface methodology (RSM) is one of the most commonly used metamodel, it involves the process of developing functional relationship between random input variables (RIV) and random output variable(ROV), which is the random response of the structure. The main steps involved in the generation of response surface are, design of experiments, choosing a function for the surface representation, fitting the model. The design of experiments is the initial step and several types of experimental designs are available the most commonly used one is the central composite design. Spacing optimized Latin hypercube sampling is used in this paper for the design. Various combinations of input variables are formulated using this method. At these points, structural responses are computed. The relation between the obtained random response and the random input variable are represented using a surface, fitted using polynomial regression. For the polynomial regression model, the function of the response surface is approximated by a polynomial function. The higher the polynomial degree, the more computations are required. To decrease computational effort, the polynomial function is commonly assumed to be of first or second order. Here, second order polynomial is used and is given by the Equation 9,

$$y = \beta_0 + \sum_{i=1}^{k} \beta_i x_i + \sum_{i=1}^{k} \beta_{ii} x_i^2 + \sum_{i \prec} \sum_{j} \beta_{ij} x_i x_j + \varepsilon \tag{9}$$

Where y is the ROV, x_i; x_j are the RIV, β_0, β_i are the regression coefficients, i represents the expected change in response y per unit change in x_i when all remaining independent variables x_j (j i) are held constant (Myers, et al. 2016). k denotes the number of input variables, and ε the experimental error which describes the drifference of functional values y from the response values of the analysis. It is inferred that the response of the structure lies within this surface and therefore the surfaces are further evaluated using Monte Carlo method for each PGA, giving the random response corresponding to each peak ground acceleration.

NUMERICAL MODEL AND UNCERTAINTY

The test structure of the international benchmark project Smart 2013 project (Richard and Chaudat 2014) is considered for the analysis of multidimensional fragility in this study. The test structure is three-storied reinforced concrete structure, and is a 1/4th scaled model of a typical electrical facility of a nuclear power plant. The technical specification of the strcuture including the general description, seismic inputs, material parameters and fragility analysis recommendations are given in Richard and Chaudat (2014). A simplified model of the test structure is modelled using Abaqus©, representing the material non linearity of the test structure. Figure 2a and 2b shows the geometry and the numerical model of the test structure. The reinforced concrete walls are modelled using the closed section beam elements of Abaqus© with a stringer reinforcement modelled as a box section. The nonlinear material parameters are considered in the reinforced concrete walls and foundation. Concrete damaged palsticity model of Abaqus©, which is a modified Drucker-Prager plasticity is used for the concrete and elastic-perfectly plastic material model for steel. The model is validated for linear properties using the natural frequency and linear time histoy analysis. The nonlinear behavior is validated from the shaking table test results of the benchmark. The model is simplified for the analysis from the previous model presented in Rajan, et al. (2017).

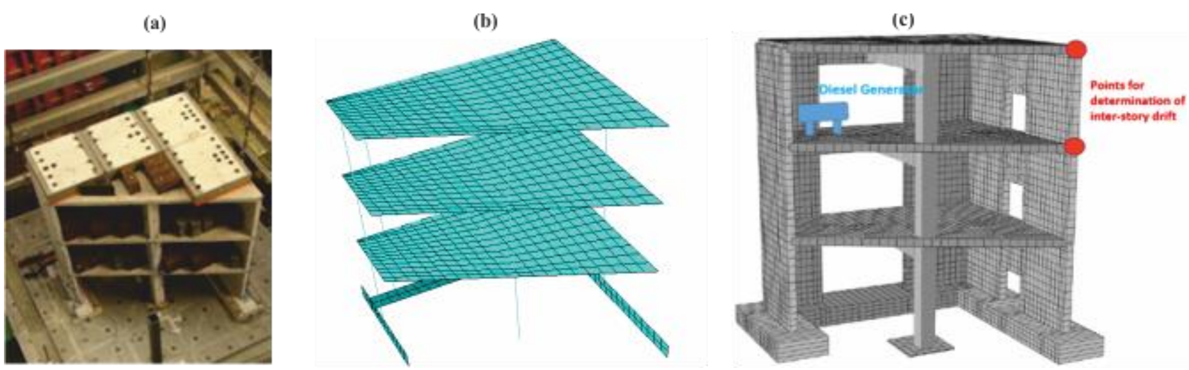

Figure 2 (a) Test structure photo. (b) Simplified nonlinear model (c)Position of generator

For the probabilistic analysis of the model, to consider a realistic RC structural behavior, the numerical model includes equivalent foundation to represent the soil-structure interaction (SSI) effect. This is done introducing springs and dashpots, so that features of SSI, such as swaying, pumping and rocking motion can be represented. The stiffness and damping to be considered for spring and dashpots were given in the SMART 2013 technical specifications (Richard and Chaudat 2014). Apart from the given benchmark strcutrue, a fictitious model of a generator is also added to the structure inorder to obtain the fragility of the system as shown in Figure 2c. The generator is modelled as a rigid body with a mass equivalent to the scaled mass of a typical generator in an NPP (3 tonnes) and is assumed to be rigdly connected to the structure. The generator is placed away from the points of interstory drift damage definition, so that the failure of the generator is is not affeted by the failure mode of the structure. The uncertain parameters considered are those suggested by the Smart 2013 project, which include the foundation stiffness and damping. A lognormal distribution is chosen for the material parameters. Rayleigh numerical damping is introduced in the nonlinear material model with a value of 3.0% on the first and the third natural frequency. A given set of 30 ground motions are considered in both x and y direction. The accelerograms are code generated synthetic accelerograms compatible with median and +/-1σ spectra for a magnitude M=6.5, at a distance of 9 km.

Probabilistic Analysis

Probabilistic analysis of the system (structure+generator) is carried out using response surface method. A software program Nessus© is used in combination with Abaqus© to obtain the probabilistic response. The design of experiments, is done using spacing-otpimized Latin hypercube sampling. A minimum number of samples recommended in the users manual is chosen, for 4 variables,40 samples are chosen for the response surface method, for each of 30 set of accelerograms. The structural responses are computed at these points in order to obtain the training data to develop a functional relationship between input and output variables. A second degree polynomial function is used to fit the curve to form the response surface. Fitted response surface with two RIV and 1 ROV corresponding to a PGA of 2.5g is shown in Figure 3. It is assumed that the probable response of the structure lies on this surface This function is then evaluated using Monte Carlo for 10000 samples to obtain the optimum response. The random response thus obtained is further used for the multidimensional fragility analysis

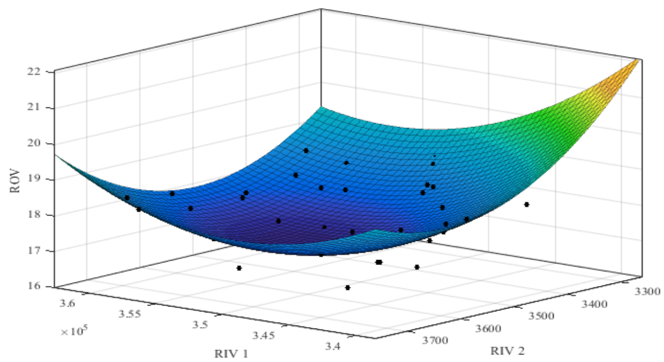

Figure 3. Second degree polynomial fitted response surface corresponding to PGA of 2.5g.

DEVELOPMENT OF FRAGILITY CURVES

A reliable assessment of seismic fragility of structures depends mainly on the definition of the level of damage, the failure threshold and seismic intensity measure. The seismic indicator used in this study is the peak ground acceleration. The damage indicator for the structure is defined by maximum interstory drift at point as shown in Figure 2c. Three levels of damage are considered: Light damage (drift = h/400), controlled damage (drift = h/200), and extended damage (drift = h/100), where h is the story height. The generator is assumed to be acceleration sensitive. The acceleration limit state of the generator is a fictitious value. It is calculated based on the natural frequency range of the typical diesel generator of a nuclear power plant. The fragility curve is generally modeled using a lognormal cumulative distribution function, a choice supported by studies in the past in different fields for example Shinozuka et al (2000b). Therefore, the fragility curve is mathematically described as given in Equation 10,

$$P_f(\theta) = \Phi\left[\frac{\ln(\theta / A_m)}{\beta}\right] \tag{10}$$

Where Φ is the standard normal probability distribution function, θ is the seismic intensity (PGA in this study), A_m is the median capacity expressed in terms of θ, β is the lognormal standard deviation. The median capacity and lognormal standard deviation can be determined either by regression analysis or maximum likelihood method (Shinozuka, et al. 2000b) (Zentner 2010). Maximum likelihood method is used for determining A_m and β and is given by the Equation 11

$$\left\{\hat{\theta},\hat{\beta}\right\} = \underset{\theta,\beta}{\arg\max} \sum_{i=1}^{m} \left\{ \ln\begin{pmatrix} n_i \\ z_i \end{pmatrix} + \ln \Phi\left(\frac{\ln(\theta / A_m)}{\beta}\right)(n_i - z_i)\ln\left[1 - \Phi\left(\frac{\ln(\theta / A_m)}{\beta}\right)\right]\right\} \tag{11}$$

Where z_i is the observed probability of collapse out of n_i ground motions (samples in the analysis) with θ_i as seismic intensity, m is the ground motion causing collapse. The fragility curves are plotted using the maximum likelihood method.

For the multidimensional fragility analysis, in the inelastic behavior the acceleration and the interstory drift are considered as independent of each other and are assumed to be log normally distributed. Therefore, the probability of exceeding the given performance limit state is deducing to equation 12.

$$P\{\delta \geq \delta_s \cup A \geq A_s\} = 1 + F_A(A_s)F_\delta(\delta_s) - F_A(A_s) - F_\delta(\delta_s) \tag{12}$$

The fragility curves for the system are plotted using maximum likelihood method and is as shown in the Figure 4. The curve depicts the likelihood of damage corresponding to specific damage states with peak ground acceleration. The probability of failure increases with increase in the PGA. The median capacity for light damage is 0.3g and for controlled and extended damage is 0.61g and 1.01g. The maximum deviation is seen in the light damage behavior, equal to 0.4. the total collapse of the structure in the extended damage occurs in all cases occurred at a PGA of 1.63g. The presented fragility curve represents the overall failure of the structure and the component. Therefore, it is capable of representing likelihood of seismic damage is for the system, however, the fragility of each component are not visible individually. The represented fragility curve is only applicable to this test structure with generator.

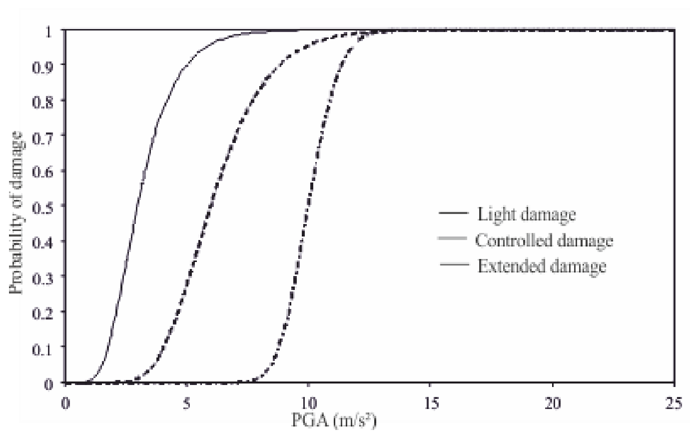

Figure 4. Fitted fragility curve for the system.

Influence of the interaction factor on the fragility curves

The assessment of the interaction factors is the disadvantage of the multidimensional limit. There are no field measurement data available for each structure after the earthquake, in order to be able to get the realistic interaction factors. Laboratory experiments are very expensive to carry out and is rarely done. The other option is to go with the engineering judgement for selecting the interaction factor. Generally assumed N value is between 1 and 15. The influence of interaction factor is not shown in this paper to limit the scope.

CONCLUSION

As a part of the SPRA, the fragility is a key component of the risk analysis. Generally, in a NPP the fragility curves of various structures and components are used in the fault tree or event tree analysis, which analyses the overall risk of the facility. As an alternative approach to the fault tree, system related fragility curves are proposed to get the fragility curves of the overall system. A method using multidimensional performance limit state (Cimellaro et al. 2006), which can represent different failure types in one formulation, which intern gives a unique fragility curves representing the overall failure of the system including the structure and components is presented here. As an example a test structure with fictitious component is analyzed in the paper. Computationally efficient, Response surface methodology is used instead of the expensive Monte Carlo approach for the probabilistic analysis. The work presented here includes only one component with the structure, however it can be extended to complex systems or components. Expert engineering judgement is required to get the interaction factor N, in the absence of collected data.

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