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A COMPARATIVE STUDY OF SEISMIC FRAGILITY ESTIMATES USING DIFFERENT NUMERICAL METHODS

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Abstract. *Seismic fragility estimations constitute the primary part of the probabilistic seismic safety assessment of important structures, such as nuclear containments, dams, important bridges, etc. The seismic fragility of a structure is expressed through a family of 'fragility' curves, which plot the conditional probabilities of failure against varying intensities of the seismic hazard. The failure probability of the structure can be defined for multiple limit states. Over the last three decades, various numerical techniques have been developed to estimate the seismic fragility of structures. Among these, three prominent methods selected for the present study are the ones proposed by Kennedy et al. [1], Shinozuka et al. [3] and Ellingwood et al. [4]. These methods vary in terms of the numerical analysis of analytical data in estimating the seismic fragility of a structure. The present work applies these three different fragility analysis techniques to the seismic fragility estimation of a nuclear containment structure and compares the results obtained from the three different methods. The inner containment structure of an Indian PHWR is used for this case study. The results obtained show that the method proposed by Shinozuka et al. provides the most accurate fragility estimations; however, it is also the most computation intensive. The conventional method proposed by Kennedy et al. provides the least accurate results and needs to use an updated uncertainty database.*

1 INTRODUCTION

The seismic fragility estimation forms the core of probabilistic safety assessment of structures. Specifically for important structures, such as nuclear power plants or large dams failure of which is likely to result into a great amount of loss of human lives, the need for seismic fragility evaluations (and re-evaluations) is considered to be the foremost in their design or qualification. In the light of the recent seismic events (for example, the 2011 Great East-Japan Tsunami and Earthquake), there has been a greater awareness about the potential hazards related to nuclear power plants (NPP) during a high-seismic event. These events have led to the reassessment of existing methods for structural safety analysis of NPP, which is the primary motivation for the work presented here.

The seismic fragility of a structure is typically expressed through 'fragility curves'. A fragility curve plots the fragility (or, the conditional probability of failure for a given level of hazard intensity) of the structure against varying intensities of the seismic hazard. The failure probability of the structure can be defined for various types of limit states depending on how the structural failure is defined. For a selected type, a fragility plot sometimes includes multiple fragility curves corresponding to different limit states or performance levels. The seismic fragility of a NPP can be defined both at component levels and at the system level.

The pioneering works in the area of seismic fragility analysis of nuclear structures were initiated in the late 1970s and continued till the mid-1980s [1, 2]. The methodologies proposed during this period relied on sound probabilistic analyses and quite significantly on engineering judgement. However, these methods suffered due to the lack of rigorous seismic structural analysis tools, necessary computational power and available earthquake damage data. Over the last decade, seismic structural analysis have changed significantly through the use of intensive seismic structural analysis methods (such as the nonlinear response-history analysis), complexity in structural/finite element modelling techniques using damage-based nonlinear material models, probabilistic models based on real data, and the enhanced computing powers of today's computers. In this paper, we look at two different methods of seismic fragility of structures, which use detailed computation to arrive at the final fragility curves. Shinouzuka et al. [3] focussed on the statistical analysis of fragility data for bridges and Ellingwood et al. [4] focussed on the fragility analysis of building structures using incremental dynamic analysis. The primary objective of this paper is to perform a comparison of these two methods of fragility analysis, along with the 'conventional' method mentioned earlier [2], through the test case of the seismic fragility analysis of the primary containment dome of a typical Indian PHWR.

2 THREE METHODS OF SEISMIC FRAGILITY ANALYSIS

2.1 Conventional method

The work by Kennedy et al. [1] was the first to present a detailed methodology for the estimation of the median ground acceleration capacity and associated uncertainties for the estimation of fragility curves of an existing NPP. Later, a detailed procedure for the estimation of fragility curves based on the selection of components, identification of failure modes and evaluation of uncertainties using factors of safety was given by Kennedy and Ravindra [2]. The fragility of a structure in this method is estimated using a lognormal model:

$$F_r = \Phi\left[\frac{\ln(x/m_a) + \beta_U \Phi^{-1}(Q)}{\beta_R}\right] \tag{1}$$

where, $\Phi(.)$ is the standard normal CDF operator, x is the seismic intensity (typically, PGA) at which the fragility is evaluated, m_a is the median ground acceleration capacity, β_R and β_U respectively measure the randomness (aleatory) and uncertainty (epistemic) associated with the estimation of the ground acceleration capacity, and Q is the non-exceedance probability level. The ground acceleration capacity (a) is expressed in terms of its median capacity (m_a) and associated uncertainties:

$$\begin{aligned} a &= m_a \epsilon_R \epsilon_U \\ &= a_{\text{DBE}} \bar{F} \epsilon_R \epsilon_U \\ &= a_{\text{DBE}} (F_S F_\mu F_R) \epsilon_R \epsilon_U \end{aligned} \tag{2}$$

where ϵ_R and ϵ_U follow lognormal distributions with a median equal to one and lognormal standard deviations β_R and β_U, respectively. a_{DBE} is the intensity of the design basis earthquake, $\bar{F}$ is the median factor of safety, which is composed of three different factors of safety. Details of these factors of safety and the associated uncertainties were discussed in depth in a recent paper by Pisharady and Basu [5]. As mentioned earlier, due to the lack of necessary tools and data, the quantification of these uncertainties depended significantly on engineering judgement. Although this method originally used response spectrum based linear elastic analyses, Pisharady and Basu [5] showed that modern analysis techniques, such as the nonlinear static pushover analysis can easily be incorporated in this framework. The two-parameter lognormal model of Equation 1, in its various forms, has now been widely accepted for seismic fragility analysis of structures. The two other fragility methods discussed in this paper also conform to this.

2.2 Maximum likelihood method

The method proposed by Shinouzuka et al. [3] uses a maximum likelihood estimation method to determine the parameters m_a and β_R to evaluate fragilities as a function of the intensity measure x:

$$F_r = \Phi\left[\frac{\ln(x/m_a)}{\beta_R}\right] \tag{3}$$

The likelihood function takes the form of a Bernoulli distribution:

$$L = \prod_{i=1}^{N} [F_r(x_i)]^{p_i} [1 - F_r(x_i)]^{q_i} \tag{4}$$

where $F_r(x_i)$ represents the fragility at PGA = x_i based on a specific limit state, and is calculated using Equation 3. N represents the number of sample response points at the selected PGA. p is 1 or 0 depending on whether the limit state is exceeded or not, respectively, and $q = 1 - p$. This method can incorporate data from random samples of earthquake response, either observed or obtained through analysis. The two parameters m_a and β_R are evaluated by maximizing the likelihood function:

$$\frac{d \ln L}{d m_a} = \frac{d \ln L}{d \beta_R} = 0 \tag{5}$$

These values are substituted in Equation 3 to obtain the final fragility curve.

2.3 Method based on IDA and regression

Incremental dynamic analysis (IDA) [6], which have been used very commonly in the last 10 years in probabilistic seismic demand analysis, was used by Ellingwood et al. [4] for the fragility evaluation of steel and concrete building frames. Using multi-IDA response data for a set of acceleration records, they generated sample responses for a scaled intensity measure, which was the first-mode pseudo spectral acceleration (S_a). The variation in the response quantity (maximum interstorey drift ratio, $\theta_{\max}$) at any given intensity was then modelled with a lognormal distribution. The response was expressed as an exponential function of the intensity adopting a regression approach:

$$\theta_{\max} = c(S_a)^d \epsilon \tag{6}$$

where, c and d are regression parameters and ϵ quantifies the dispersion in the estimation of the response parameter. ϵ follows a lognormal distribution with a median equal to one and lognormal standard deviation β_R. Finally, fragility values are calculated using a displacement-based approach, where the failure is defined by the exceedance of the limit state response θ_{LS}:

$$F_r = \Phi\left[\frac{\ln\{c(S_a)^d/\theta_{\text{LS}}\}}{\beta_R}\right] \tag{7}$$

3 STUDY MODEL

The primary containment (PC) structure of a 700 MWe Indian pressurised heavy water reactor (PHWR) is considered for a comparison of the three selected methods of fragility analysis. It consists of a prestressed concrete cylindrical wall capped by a segmental prestressed concrete dome through a massive ring beam. The containment shell is supported on a circular raft. The PC structure is idealised using a 2D ‘stick model’ with lumped masses connected by 2D beam-column elements [7]. A simple illustration of the containment structure along with its stick idealisation is shown in Figure 1.

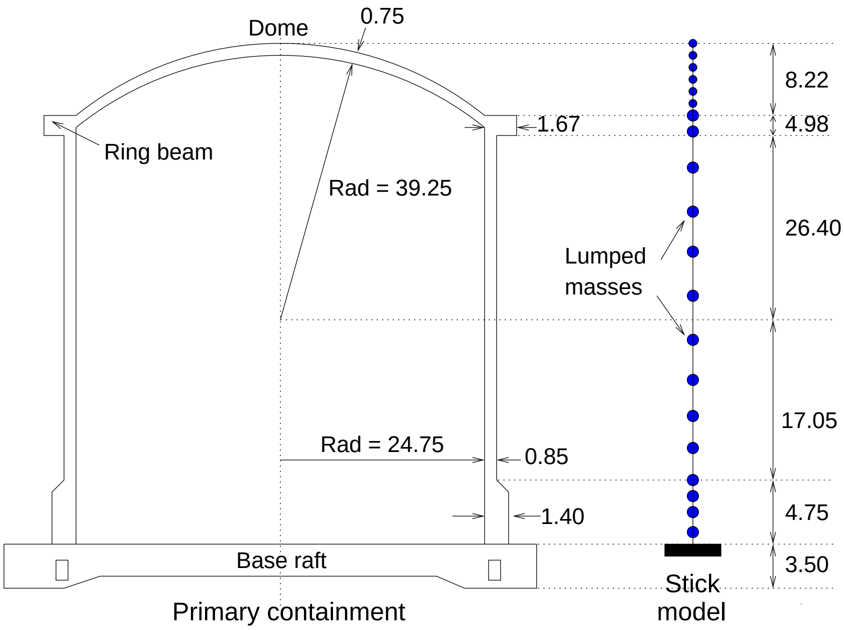

Figure 1: Schematic of the primary containment structure and its stick model (dimensions in m).

The containment structure responds to horizontal base excitation like a cantilever beam with an annular cross-section. This cantilever is modelled using the *nonlinearBeamColumn* element in the OpenSees platform [8]. This element is a force-based element and it considers the spread of plasticity along the length of the member and five integration points are considered along the length of an element for this purpose. The cross-section is modelled with a *FiberSection*, where concrete is modelled as an annular patch of the *concrete02* material and reinforcing steel as a circular layer of *steel01* material. A damaged plasticity model (for both compression and tension behaviour) is considered for concrete, while the steel has a elastic-1% strain hardening plasticity behaviour. The shear deformation behaviour is modelled using the *sectionAggregator* approach. For simplicity, an elastic-perfectly plastic (EPP) type of shear force-deformation model is considered. This model requires two parameters: slope of the elastic curve and yield strength of the material. The slope of this curve is GA_s, where, G is the shear modulus of concrete and A_s is the shear area of the particular annular section. The yield shear strength of each section is calculated as per ACI-318 [9]. Further details of the 2D stick model can be found in the dissertation by Mandal [10].

4 FRAGILITY ANALYSIS

Seismic fragility analysis of the study structure is carried out using the 'Conventional', 'Likelihood' and 'Regression' methods discussed in Section 2. For the 'Conventional' method, a nonlinear static pushover analysis is conducted to obtain the response of the structure. Using the method suggested by Pisharady and Basu [5], the 50 percentile fragility curve is obtained based on the results of this pushover analysis. The median ground acceleration capacity is based on the Collapse Prevention (CP) limit state for maximum interstorey drift ratio defined in FEMA-356 [11]: $\theta_{\text{LS}} = 0.75\%$. The associated uncertainties are obtained from the tables compiled in their paper.

For the 'Likelihood' and 'Regression' methods, sample responses are generated using a multi-IDA study. The multi-IDA involves nonlinear response-history analysis (NLRHA) subjected to scaled intensities of 25 recorded accelerations. For this, intra-plate records suitable for the NPP site (in peninsular India) are selected, details of which are provided in Table 1. PGA is selected as the intensity measure (IM) and $\theta_{\max}$ as the damage measure (DM) in this IDA study. The IDA plots for the 25 records are shown in Figure 2. This plot also contains the pushover curve in PGA vs. $\theta_{\max}$ format which is used in the 'Conventional' fragility analysis. The vertical line labelled 'LS' represents the limit state of performance mentioned earlier. The probability of exceedance of this limit state is calculated at every intensity level to get the direct IDA-based fragility values. For the 'Likelihood' method, where the NLRHA-based responses are considered as random samples, this exceedance gives the p (and q) values for different intensity levels. The 'Likelihood' fragilities are obtained using Equation 3 after solving for m_a and β_R for maximizing the likelihood function. For the 'Regression' method, the multi-IDA data is modelled with a lognormal distribution at each intensity level and the median value is expressed as a function of PGA. c and d are obtained from the nonlinear regression and β_R from the lognormal distributions. The final fragility curve is obtained using Equation 7, while replacing S_a with PGA. It should be noted that for both of these methods, only the randomness due to the variation in possible earthquakes is included and the (epistemic) uncertainties are ignored, based on the consideration that the primary source of uncertainty in this case is the inherent randomness in earthquakes [12].

5 COMPARISON OF RESULTS

The fragility curves obtained using the three selected methods are presented in Figure 3. This figure also shows the discrete fragility data points obtained directly from the multi-IDA

Record	Event	Station	Component	R (km)	PGA (g)
GM-1	Bhuj, 2001	Ahmedabad	Radial	239.00	0.106
GM-2	Bhuj, 2001	Ahmedabad	Transverse	239.0	0.080
GM-3	Koyna, 1967	Koyna Dam	Radial	35.30	0.474
GM-4	Saguenay, 1988	St.-Ferreol	Radial	117.23	0.121
GM-5	Saguenay, 1988	St.-Ferreol	Transverse	117.23	0.097
GM-6	Saguenay, 1988	Quebec	Radial	149.40	0.051
GM-7	Saguenay, 1988	Quebec	Transverse	149.40	0.051
GM-8	Saguenay, 1988	Tadoussac	Radial	163.03	0.027
GM-9	Saguenay, 1988	Tadoussac	Transverse	163.03	0.002
GM-10	Saguenay, 1988	Baie-St-Paul	Radial	106.34	0.125
GM-11	Saguenay, 1988	Baie-St-Paul	Transverse	106.34	0.174
GM-12	Saguenay, 1988	La Malbaie	Radial	125.70	0.124
GM-13	Saguenay, 1988	La Malbaie	Transverse	125.70	0.060
GM-14	Saguenay, 1988	St.-Pascal	Radial	167.00	0.046
GM-15	Saguenay, 1988	St.-Pascal	Transverse	167.00	0.056
GM-16	Saguenay, 1988	Riviere-Ouelle	Radial	150.20	0.040
GM-17	Saguenay, 1988	Riviere-Ouelle	Transverse	150.20	0.057
GM-18	Saguenay, 1988	Ste.-Lucie-de-Beauregard	Radial	136.36	0.014
GM-19	Saguenay, 1988	Ste.-Lucie-de-Beauregard	Transverse	136.36	0.023
GM-20	Saguenay, 1988	Chicoutimi-Nord	Radial	45.69	0.107
GM-21	Saguenay, 1988	Chicoutimi-Nord	Transverse	45.69	0.131
GM-22	Saguenay, 1988	St-Andre-du-Lac-St-Jean	Radial	92.96	0.156
GM-23	Saguenay, 1988	St-Andre-du-Lac-St-Jean	Transverse	92.96	0.091
GM-24	Saguenay, 1988	Les Eboulements	Radial	114.31	0.125
GM-25	Saguenay, 1988	Les Eboulements	Transverse	114.31	0.102

Table 1: Details of the ground motion records considered.

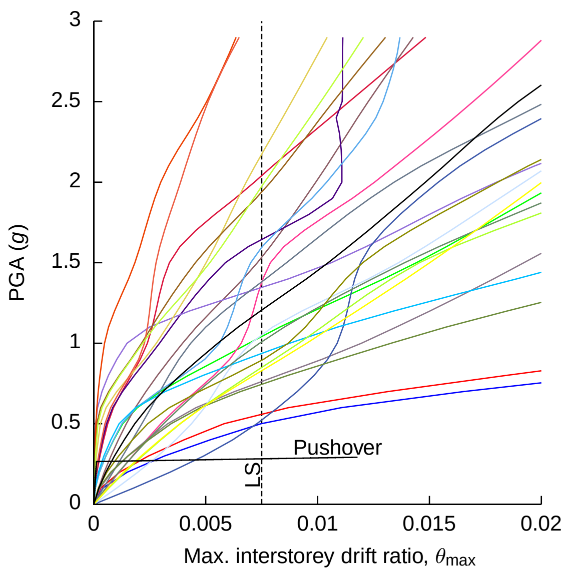

Figure 2: IDA plots for 25 acceleration records, along with the normalised pushover curve.

response at each PGA level. For the two methods using responses from the multi-IDA, these data points are treated as representing the 'actual' fragility values. The 'Conventional' fragility curve is very 'steep' compared to the others. It attains the 100% fragility by PGA = $1.5g$. The very high slope of the curve indicates to relatively smaller uncertainties (small β_R). Although the other two methods consider uncertainties only in the earthquake's randomness, that amounts to a greater uncertainty compared to the 'Conventional" fragility analysis.

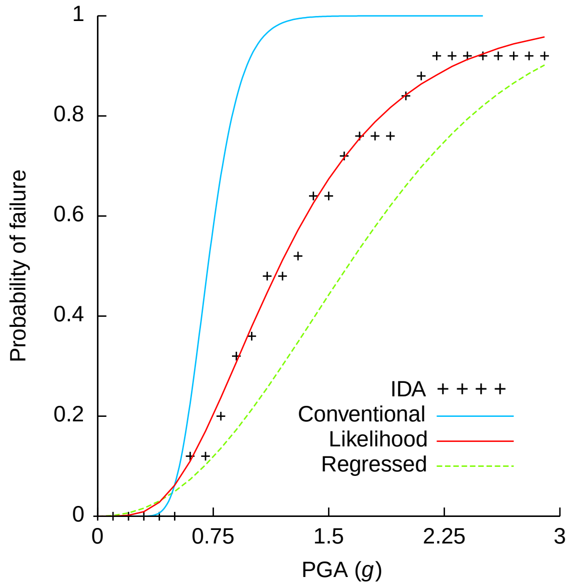

Figure 3: Fragility curves obtained using different methods.

A quick look at the 'Likelihood' and 'Regression' curves shows that the former is a better approximation of the discrete IDA-based fragility data points. This is because of the levels of approximations involved in the 'Regression' method, whereas the maximum likelihood based method is a more direct approach at the idealisation. However, there remains a need for a quantitative comparison of fragility estimates based on these two methods with respect to the IDA-based fragility data. The comparison is performed using three quantities: sum squared error (SSE) averaged over the total number of intensity levels (E), coefficient of determination (R^2), and Pearson's product-moment correlation coefficient (ρ_P). This comparison is summarised in Table 2. All of the three measures show very clearly that the 'Likelihood' method provides more accurate results than the 'Regression' method.

	Likelihood	Regression
E	0.827×10^{-3}	0.0194
R^2	0.995	0.948
ρ_P	0.997	0.974

Table 2: Accuracy/error in IDA-based fragility curves.

6 CONCLUSIONS

A comparison of three selected methods of fragility analysis is performed in this paper through the case study of the seismic fragility analysis of a primary containment structure of a typical Indian NPP. The methods are selected based on their popularity, and differences in the basic approach focussed primarily on how data and computational tools are used. Of the three methods selected here, the 'Conventional' method gives very different results as compared to the other two. Although this method is easy to use, the database of uncertainty quantification used in this method does not seem realistic and it needs to be updated/calibrated based on thorough probabilistic and seismic structural analysis.

The 'Likelihood' and the 'Regression' methods are computation intensive because sample earthquake responses need to be generated through a large set of NLRHA. However, looking at the level of accuracy these methods can achieve and the computational prowess available with today's engineers (at least for important structures, such as NPP), these are definitely recommended over the 'Conventional' method. Between these two methods, the 'Likelihood' fragility analysis proves to be the more accurate one, but the user needs to note that solving Equation 5 is quite computation heavy. The fragility analyses performed here are based on several important assumptions (such as, no epistemic uncertainty, no soil-structure interaction, etc.); however, we do not expect the conclusions to change when more rigourous analyses are performed incorporating all reasonable sources of uncertainty.

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