Assessment of Seismic Damageability by Probabilistic Seismic Demand Models and Confidence Ratios Applied to NPP Structures

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ABSTRACT

In this paper we go on with some earlier research work and papers related to the probabilistic modeling of the seismically induced damages in structures, and also to seismic fragility models for NPP structures. A performance-based earthquake engineering framework was developed by the *PEER – Pacific Earthquake Engineering Research Center*. This framework includes several blocks or models, with the PSDM – the probabilistic seismic demand model as one of its main blocks. Improvements in this PSDMs were proposed by a couple of authors in their contributions to the 13 WCEE Conf. (Vancouver, August 2004). The selection of both seismic intensity measures (IMs) and of the engineering demand parameters (EDPs) is an important task in such models. The investigation of the relationship between the IMs and the EDPs is just the essence of the PSDMs. We discuss a couple of probabilistic aspects of the performance based seismic demand models and some ways to use the confidence ratios for the PSDMs applied to nuclear facility structures.

INTRODUCTION

The estimation of the demand on a structure from a given earthquake is one of the challenges of performance-based earthquake engineering. The basic objective of the *performance based seismic design* (PBSD) is to provide engineers with the capability to design buildings that have a predictable and reliable performance in earthquakes (according to an introductory section of FEMA 349 [1]). A key to knowing how a structure will perform under a given earthquake is having the ability to estimate the damage it will sustain and the consequences of that damage. PBSD differs from the current codes in that it focuses on a building's individual performance. In its broadest sense, PBSD creates global planning opportunities for reducing economic and social (human) losses to whole communities, regions and states.

A rather widely accepted approach consists in defining and using an intensity measure (*IM*) and then computing the probability that the earthquake will cause a certain level of demand in the structure as a function of this *IM*. Traditional *IM*s include the PGA – peak ground acceleration and S_a – spectral acceleration at the first-mode period of vibration and many others. Such *IM*s consist of a single parameter. J.W. Baker & C. Allin Cornell proposed to use intensity measures consisting of <u>two</u> parameters in a contribution to the 13 WCEE [2]. The two parameters are the components of a vector intensity measure. Other ways for improving probabilistic seismic demand models through refined intensity measures were proposed by Kevin Mackie & B. Stojadinović [3] and S. Janković & B. Stojadinović [4]. Such investigations go along the standards and guidelines elaborated by FEMA and PEER in several reports released starting with 2000: FEMA 350 [5] & FEMA 356 [6], PEER 2003/08 [7], PEER 2005/02 [8] & PEER 2005/06 [9]. In the first two sections of this paper we insist on the mathematical nature and properties of the seismic demand and the notion of confidence ratio, and also on the possibilities to apply these concepts to the evaluation / prediction of seismic behavior of structures in nuclear facilities. We recall some notions and models for the evaluation of seismically induced damages and for the availability evaluation of (sub)systems in NPPs from the viewpoint of PBSD, presented in two of our earlier contributions to SMiRT 16 and SMiRT 18 [10,11].

PROBABILISTIC PERFORMANCE BASED SEISMIC DEMAND MODELS

The performance based seismic demand models (PBSDMs or PBSD models) represent an engineering concept developed in the United States since the mid eighties. A reference report on this concept was SEAOC Vision 2000 [12], but our main sources have been the reports FEMA 350 & 356, the PEER reports of 2003 & 2005 [5,6,7], and also a couple of contributions to the 13th WCEE Conference of Vancouver (August 2004) just mentioned in the Introduction. Methodologies for the evaluation or prediction of structural behavior under (strong) earthquake motions were also developed by the NUREG Commission, mainly in terms of the *seismic fragility models*, *common cause failure* (CCF) and other specific models. The seismic fragility has become a rather usual concept in the seismic analysis of civil engineering structures and not only for the structures and equipment in nuclear facilities, as they were originally developed since the early 80s. The distinction between the *uncertainty* and *randomness* is shared by both the fragility models and PBSD models. The *randomness* is conceived as a measure of our inability to precisely understand and model the factors pertaining to such phenomena as the seismic loadings and capacity of structures. The uncertainty is a measure of the error(s) introduced in the calculations or evaluations as a result of our inability to precisely characterize reality, e.g. seismic methods, structural models, etc. Both randomness and uncertainty are inherent in performance prediction for structures or systems under seismic loading. Another common feature in the fragility models and (some of the) PBSD models consists in the use of log-normal distributions. The confidence-based design provisions, developed by C. Allin Cornell, Helmut Krawinkler and other authors represent the first code-of-practice application of probabilistic PB seismic design for conventional structures.

A typical performance based seismic demand (PBSD) model involves an equation of the form

$$\lambda_{EDP}(z) = \sum_{\text{all}\, x_i} P[EDP > z \mid IM = x_i] \cdot \Delta IM(x_i) \tag{1}$$

where *EDP* denotes an *engineering demand parameter* compared with the level z, *IM* is an intensity measure with x_i as one of its possible values and the annual frequency of exceeding it given by $\lambda_{IM}(x_i)$. The conditional probability that occurs in Eq. (1) is the essential issue in predicting the structural response to seismic demands of intensity x_i. Other variants of equations similar to Eq. (1) may be met in the literature, depending on the nature of the parameters that occur in the model. As just mentioned above, <u>two</u> conditioning IMs are considered by J. Baker & C. A. Cornell [2]. In fact, a PSDM (probabilistic seismic demand model) basically consists in evaluating conditional probabilities of the form $P[EDP \mid IM]$ but it also involves conditional probabilities like $P[LS \mid EDP]$ where LS is a limit state. According to the PEER probabilistic PB (performance-based) evaluation methodology, the annual frequency of exceeding the limit state LS can be mathematically expressed as

$$\lambda[LS] = \int_{[EDP]} \int_{[IM]} P[LS \mid EDP]\, dP\big[EDP \mid IM\big]\, d\lambda[IM] \tag{2}$$

where $P[LS \mid EDP]$ is the conditional probability that the seismic response is the conditional probability of exceeding LS given the value of the engineering demand parameter EDP ; $P[EDP \mid IM]$ is the conditional probability of exceeding each value of EDP given the value of the ground motion intensity measure, IM ; $\lambda[IM]$ is the mean annual frequency of exceeding each value of IM , hence it quantifies the ground motion (or seismic) hazard. We have denoted by $[EDP]$ and $[IM]$ the domains (or intervals) of the possible values for the two parameters. The conditional probabilities $P[EDP \mid IM]$ are obtained, in the PSDMs (*probabilistic seismic demand models*), by statically analyzing the results of nonlinear time-history analyses of structure responses under earthquakes of different intensities. Several PSDMs are investigated in [4] from the points of view of their “sufficiency” and “efficiency”. The selection of ground motion intensity measure IM is a very important issue in the probabilistic evaluation of the seismic response of structures. Seven such intensity measures were analyzed in [4], namely : *PGA* – the peak ground acceleration, *PGV* – the peak ground velocity, *EPA* – the effective peak acceleration, *CAV* – the cumulative absolute velocity, S_a – the acceleration response spectrum, S_v – the velocity response spectrum, and *Cord* – the Cordova measure. We do not give details on the *EDP*s – engineering demand parameters but only mention that they are usual in the seismic analysis of frame structures : the interstory drift ratio $IDR_{\max}$ and the maximum frame drift ratio DR_F. However, let us mention that a comprehensive (and comparative) study of almost 50 intensity measures was reported in [3] although the authors took into account several thousands of existing and new IMs.

Coming back to Eq. (1), let us present a more detailed version of such an equation, with the spectral acceleration S_a as the intensity parameter and the limit state frequency defined as the product of the mean rate of occurrence v of events with seismic intensity larger than a certain “minimum” level and the probability that the seismic demand D exceeds the capacity C when such an event occurs :

$$H_{LS} = v \cdot P[D > C] = v \cdot \sum_{\text{all}\, x} P[D > C \mid S_a = x] \cdot P[S_a = x]. \tag{3}$$

This equation is proposed in the PEER Report [7] and it illustrates a seismic format for the DCFD – *demand and capacity factor design*. In a slightly more general approach, the conditioning event $[S_a = x]$ in Eq. (3) can be replaced by $[D = d]$ what results in

$$H_{LS} = v \cdot P[D > C] = v \cdot \sum_{\text{all}\, x} P[D > C \mid D = d] \cdot P[D = d]. \quad (4)$$

with its probability factorized, in its turn, as a product involving a conditional probability :

$$P[D = d] = P[D = d \mid S_a = x] \cdot P[S_a = x]. \quad (5)$$

Certainly, another intensity parameter or intensity measure (*IM*) can be taken in the conditioning event that occurs in Eq. (5). In this equation, the spectral acceleration appears as the second interface variable while the first one is the displacement-based demand.

The *spectral acceleration hazard* is analytically and probabilistically described in [7] as the mean annual frequency that the intensity of future ground motions will be greater that or equal to a specific value *x*. It is denoted as $H_{S_a}(x)$ and its analytical expression involves the coefficient (or rate parameter) v which occurs in Eqs. (3) & (4), times the probability of exceeding the spectral acceleration, $G_{S_a}(x) = P[S_a \geq x]$**:**

$$H_{S_a}(x) = v \cdot G_{S_a}(x). \quad (6)$$

Spectral acceleration hazard curves are normally available from the seismological databases like those provided by USGS for a given site. Luco and Cornell proposed (in 1998) an approximation of the power-law relationship $H_{S_a}(x)$ that does not imply the rate parameter v, namely

$$H_{S_a}(x) = P[S_a \geq x] = k_0 \cdot x^{-k} \quad (7)$$

where k_0 and k are parameters defining the shape of the hazard curve. Expression (7) appears as Eq. (1.3) in [7] but the argument of H_{S_a} in the leftmost side appears as S_a instead of x. The analytical formulation of the PSDM in [7] uses complete databases for *IM*s and *EDP*s with a single variable selected from each group. The functional form for the EDP distribution was taken as

$$\ln(EDP) = \ln(E\hat{D}P) + \sigma\varepsilon \quad (8)$$

In the right-hand side, $E\hat{D}P$ is the median of the EDP, ε is an error term assumed to be a standard-normally distributed random variable, hence it is a standard Gaussian variate in N(0,1), with its standard deviation σ which is assumed to be independent of the intensity variable IM.

Ground Motion Hazard, Demand, Capacity and Log-Normal Models

A general probabilistic performance-based framework, based upon the PEER Center codes and standards, is recalled in by Orbović, Stojadinović, Bouchon, and Wiley in their contribution [13] to 13 WCEE. Their total probability integral equation, similar to Eq. (2), involves a *decision variable DV* used to quantify the performance limit state and a *damage measure DM* that quantifies the ability of a structure to sustain a limit state in engineering terms. The *EDP* and *IM* parameters have the same significance as earlier. This approach sets the ground for the confidence-based design, for the risk reduction design and acceptance criteria, and it is better suited for the seismic PB evaluation of structures in nuclear facilities. A probability of failure is defined as the probability that a performance limit state is not met by a direct comparison of demand and capacity :

$$P_F = P[D \geq C] = \int_{[D]} \int_{[S_a]} P[D \geq C] P[D \geq d \mid IM \geq s_a] \mid dH(s_a) \mid . \quad (9)$$

In this Eq. (9) we have used our notation $[\,\ldots\,]$ for the range of possible values of the (random) variable inside, as in Eq. (2), and also the brackets around an event whose probability is considered. Obviously, $H(s_a)$ is just the spectral acceleration hazard $H_{S_a}(x)$ that occurs in Eqs. (7) & (8). Like in Eq. (3), C is the capacity and D is the demand. All the three variables, capacity *C*, demand *D* and the ground motion hazard are assumed to be realistically modeled by log-normal distributions. Thus, the ground motion hazard is described in terms of its median $\hat{H}(s_a)$ and its standard deviation β_H :

$$\overline{H}(s_a) = \hat{H}(s_a)\exp\left(\frac{1}{2}\beta^2\right). \tag{10}$$

The spectral acceleration hazard corresponding to a probability P_H of exceeding a value $H(s_a)$ can be approximated in a similar way to the power-law relationship (7) :

$$P_H = P[S_a \geq s_a{}^{P_H}] = k_0\left(s_a{}^{P_H}\right)^{-k}. \tag{11}$$

Both log-normal distributions for the capacity and demand are similarly defined in terms of their respective medians and standard deviations. These two variables are described by functional expressions involving powers, namely

$$\hat{D} = a\left(S_a{}^{P_H}\right)^b,\ \hat{C} = a\left(S_a{}^{\hat{C}}\right)^b. \tag{12}$$

In this Eq. (12), *a* and *b* are shape parameters. A value $b = 1$ corresponds to the equal displacement rule which applies to a majority of regular structures [13]. The variances of the two lognormal distributions followed by both demand and capacity are quantified – like in most of the fragility models – as the geometrical means of the respective (pairs of) standard deviations describing the dispersions entailed by the uncertainty in determining the demand and to randomness pertaining to the ground motion. These “total” or combined standard deviations can be also regarded as the Euclidean norms of two vectors in $\mathbf{R}^2$:

$$\beta_{DT} = \sqrt{\beta_{DU}^2 + \beta_{DR}^2} = \|(\beta_{DU}, \beta_{DR})\|,\ \beta_{CT} = \sqrt{\beta_{CU}^2 + \beta_{CR}^2} = \|(\beta_{CU}, \beta_{CR})\|. \tag{13}$$

The mean value of the failure probability in Eq. (9) was evaluated in closed form by Cornell *et al*. and it is recalled in [13] as

$$\overline{P}_F = \overline{H}(s_a^{\hat{C}})\exp\left(\frac{1}{2}\frac{k^2}{b^2}\left(\beta_{DU}^2 + \beta_{DR}^2 + \beta_{CU}^2 + \beta_{CR}^2\right)\right). \tag{14}$$

The failure probability follows itself a Log-N distribution that is entirely characterized by its median value and standard deviation :

$$\hat{P}_F = \overline{H}(s_a^{\hat{C}})\exp\left(\frac{1}{2}\frac{k^2}{b^2}\left(\beta_{DU}^2 + \beta_{DR}^2\right)\right),\ \beta_{P_F} = \frac{k}{b}\sqrt{\beta_{DU}^2 + \beta_{DR}^2} \underset{(13)}{=} \frac{k}{b}\beta_{DT}. \tag{15}$$

If K_x denotes the standard Gaussian variable associated with the probability x of not being exceeded, then the failure probability corresponding to an x% confidence level is given by

$$P_F^x = \hat{P}_F(s_a^{\hat{C}})\exp\left(K_x\beta_{P_F}\right) = \exp\left(K_x\frac{k}{b}\beta_{DT}\right). \tag{16}$$

If the mean value of the failure probability is set equal to the probability of exceeding a hazard level, that is $\hat{P}_F = P_H$, Eq. (14) leads to an inequality which is specific to the load-and-resistance factor design format, namely

$$\phi\hat{C} \geq \gamma\hat{D}^{P_H} \text{ with } \phi = \exp\left(-\frac{1}{2}\frac{k}{b}\beta_{CT}^2\right)\ \&\ \gamma = \exp\left(\frac{1}{2}\frac{k}{b}\beta_{DT}^2\right). \tag{17}$$

CONFIDENCE RATIOS AND RISK REDUCTION RATIOS

A *confidence ratio* can be used in order to formulate an acceptance criterion associated with a probability of exceedance P_H. This ratio is expressed in terms of the two sides of the inequality in (17):

$$\lambda_{\text{con}} = \frac{\gamma \hat{D}^{P_H}}{\phi \hat{C}} = \exp\left(-K_x \beta_{UT} + \frac{k}{2b} \beta_{UT}^2 \right) \tag{18}$$

where P_H is the probability of exceeding a spectral acceleration hazard, D is the demand, C is the capacity (with their medians occurring in the mid-side of Eq. (18)), k is a slope of the log-log plot of P_H, β_{UT} is the total standard deviation due to uncertainty,

$$\beta_{UT} = \sqrt{\beta_{DU}^2 + \beta_{CU}^2} = \|(\beta_{DU}, \beta_{CU})\| \tag{19}$$

and K_x is the Gaussian variable considered in the previous section. The confidence ratio is involved in an analytical expression of a risk reduction ratio ; ϕ is the capacity reduction factor and γ is the load factor. The familiar LRFD involves the inequality in (17) and this implies

$$0 < \lambda_{\text{con}} \leq 1. \tag{20}$$

This confidence ratio enables direct computation of the confidence level associated with the probability of failure P_H using the tabulated values in [14].

The *risk reduction factor design* (RRFD) has been employed in some codes of US Department of Energy, where the acceptance criteria are based on evaluating the probability of unacceptable performance using a truncated form of the general probabilistic PB design framework equation. The evaluation of the capacity-demand comparison is formulated in terms of a fragility function (the conditional probability of failure given a ground motion intensity level) that evaluates the probability that a performance limit state is crossed:

$$P_F = P[C \leq D] = \int_{[S_a]} P[F \mid s_a] | dH(s_a) |. \tag{21}$$

In the design and seismic risk evaluation of NPP structures the probability of failure is required to be reduced with respect to the probability of the seismic hazard. The amount of risk reduction can be quantified either in terms of the RR ratio $R_R = P_H / P_F$ as in [13] or by the inverse of this ratio,

$$\rho_R = P_F / P_H. \tag{22}$$

Confidence Ratios and HCLPF values

Next we try to establish connections between the confidence ratios, the HCLPF (high confidence low probability of failure) values involved in the fragility models and the confidence intervals that occur in estimation theory. The so-called double lognormal format that is essential in the fragility models is based on the assumption that both the ground acceleration *A* corresponding to failure and its median $\hat{A}$ follow log-normal distributions. If a denotes the current value of *A* and *c* is the current value of the median, then $A = \hat{A} \varepsilon_R \varepsilon_U$ are random variables with unit medians and respective logarithmic standard deviations β_R & β_U (as measures of dispersion due to randomness and uncertainty, respectively) then the failure probability at a level a of the ground acceleration is given by

$$P_f(a) = \Phi\left[\frac{\ln(a / \hat{A})}{\beta_R} \right] \tag{23}$$

and the probability density of the median

$$f_{\hat{A}}(c) = \frac{1}{\sqrt{2\pi}\, c\beta_U} \exp\left[-\frac{1}{2}\left(\frac{\ln(a/\hat{A})}{\beta_U}\right)\right]. \tag{24}$$

The model consisting of the function in (23) and the density in (24) makes possible to evaluate the seismic fragility, that is the failure frequency of a structure, conditional on a seismic intensity a, as

$$F(a, p_0) = \Phi\left[\frac{1}{\beta_R} \ln\left(\frac{a}{\hat{A}\exp(-\beta_U\,\Phi^{-1}(p_0)}\right)\right] \tag{25}$$

where Φ is the standard normal (or Gaussian) *cdf* and p_0 is a probability that determines a corresponding fragility curve. For instance, $p_0 = 0.5$ determines the median curve.

A *high confidence low probability of failure* (HCLPF) is defined by

$$HCLPF = \hat{A}\exp\left[-1.645\,(\beta_R + \beta_U)\right]. \tag{26}$$

The failure probability of a component or structure whose fragility is characterized by

$$P_f = \Phi\left[\frac{\ln HCLPF - \ln \hat{A}}{\beta_C}\right] \tag{27}$$

where $\beta_C = [\beta_U + \beta_R]^{1/2}$ is the "composite" variability due to both uncertainty and randomness, as in the previous section. The HCLPF concept was introduced in a contribution by Howard Hwang to the SMiRT 10 Conference of Anaheim (1989) and is was considered in many subsequent studies on the (seismic) safety assessment for NPPs. The formulations of Eqs. (26) and (27) were adapted from the extended version or our earlier paper [15] communicated to SMiRT 12.

A confidence ratio as defined by Eq. (18) compares the demand with the capacity. A lower value of this ratio would result in a higher confidence that the failure would not occur. A possibility to combine the confidence ratio concept with the HCLPF would consist in taking this value instead of K_x that occurs in Eq. (18). This results in

$$\lambda_{\text{HCLPF}} = \frac{\gamma \hat{D}^{P_H}}{\phi\,\hat{C}} = \exp\left(-HCLPF\,\beta_{UT} + \frac{k}{2b}\beta_{UT}^2\right). \tag{28}$$

We do not offer numerical examples to evaluate a confidence ratio as this one of Eq. (28) here. We only remark that other three contributions due to Korean participants in the 13 WCEE Conference propose methods for seismic damage assessment, evaluation of seismic capacities of Korean NPP structures and improvement of seismic safety. Two of them involve fragility models.

SUMMARY AND CONCLUSIONS

Several formal relationships between the structural and seismic hazard parameters met in the performance-based seismic design have been considered and discussed in our paper. Most of them have been developed within the provisions and guidelines worked out by FEMA and PEER Center, and also in a couple of contributions to the 13th World Conference of Earthquake Engineering (Vancouver, August 2004). These concepts of performance based seismic design have found some use in fields like the seismic risk assessment of structures in NPPs. We thus continue some earlier investigations reported in several papers, including the two contributions to SMiRT 16 & SMiRT 18 listed below. We propose a couple of ways to go further with the employment of the models involving seismic fragility, the confidence ratios and and the HCLPF concept in the evaluation of the seismic capacity of structures and equipment in NPPs. Our approach suggests some ways to take advantage of certain procedures of PBSD in the probabilistic seismic risk assessment of structures in NP facilities, in connection with the fragility taking into account that both some PBSD models and some earlier approaches to the assessment of seismic safety in NPP structures involve double log-normal formats.

NOMENCLATURE

PSDM	= probabilistic seismic demand model
PBSD	= performance-based seismic design
EDP	= engineering demand parameter
IM	= (seismic) intensity measure with $\lambda[IM]$ = mean annual frequency of exceeding each value of *IM*
$P[EDP \mid IM]$	= conditional probability of exceeding each value of *EDP* given the value of the ground motion intensity measure, *IM*
LS	= limit state with $\lambda[LS]$ = mean annual frequency of exceeding each value of *LS*
$P[LS \mid EDP]$	= conditional probability that the seismic response is the conditional probability of exceeding *LS* given the value of the engineering demand parameter *EDP*
S_a	= spectral acceleration ; $[S_a = x]$ = the event that spectral acceleration takes value x
$H(s_a)$	= spectral acceleration hazard also denoted $H_{S_a}(x)$, with median $\hat{H}(s_a)$ and st. deviation β_H
A	= acceleration corresponding to failure with median $\hat{A}$
P_F or P_f	= probability of failure
β_U, β_R	= standard deviations describing varibilities from uncertainty, rendomness (in fragility models)
β_C	= the “compound” standard deviation, $\beta_C = [\beta_U + \beta_R]^{1/2}$
HCLPF	= high confidence low probability of failure value (in fragility models)
Φ	= the standard normal (or Gaussian) cumulative distribution function
ϕ	= capacity reduction factor
γ	= load factor
λ_{con}	= the confidence ratio

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