On Some Special Studies Performed for Seismic Fragility Evaluation

Robert P. Kennedy
RPK Structural Mechanics Consulting, Inc., Yorba Linda, CA USA
Bimal E. Sarkar
Bechtel Power Corporation, San Francisco, CA USA
David Ovadia
Pacific Gas and Electric Company, San Francisco, CA USA

INTRODUCTION

As part of the probabilistic risk assessment (PRA) studies, seismic fragility evaluation of the key safety related structures and equipment is generally performed using somewhat conservative fragility information. The fragility estimates thus developed are based on a standard fragility evaluation method that has been applied in the seismic probability risk assessment of over thirty nuclear power plants and is well-documented (Kennedy, 1980, Kennedy, 1984a). The objective of this paper is to describe some of the analyses that were performed to improve the fragility estimates for plant structures and equipment over those using standard methods.

The following two studies were specifically performed to accomplish this objective:

- Structural Response Variability Study which provided probabilistic in-structure response spectra, including the median and associated randomness and uncertainty variabilities, and
- A nonlinear probabilistic study which, in addition to providing an improved fragility estimate of a building structure, also provided a comparison with the fragility estimate extrapolated from a single median-centered elastic response spectrum analysis obtained using the standard separation-of-variables fragility evaluation method.

STRUCTURAL RESPONSE VARIABILITY STUDY

The evaluation of the Structural Response factor used in developing fragility descriptions for structures and equipment in most previous PRA studies has employed simplified methods using the separation-of-variables approach. Because of the significant variables associated with each of the factors and the uncertainties associated with the simplified approach (how the individual variabilities combine), a more rigorous approach was undertaken through this study to establish structural response variability.

The Structural Response factor is a measure of the conservatism introduced in the development of the reference in-structure floor response spectra. The important variables used in the development of equipment fragilitites, which affect the generation of in-structure floor spectra include:

1) Ground-motion spectral shape
2) Structural damping
3) Structural frequency
4) Structural mode combination
5) Earthquake directional combination
6) Soil-structure interaction

These six variables were included in the structural response variability study for which a simplified rock-structure interaction model of a building structure was used. Acceleration time histories used in the structural response variability study were developed to represent ground motions that might be expected at a rock site within 10 kilometers of the fault rupture surface due to shallow crustal earthquakes having magnitudes in the range of 6.5 to 7.5 and having strike-slip, oblique, or reverse faulting mechanisms. A total of 38 pairs of horizontal ground-motion time-history records were used in this study. Each time-history pair was scaled such that the average 5 percent damped spectral acceleration over the frequency range of 4.8 to 14.7 Hz was 2.0 g for the average of the two horizontal components.

Variability in structural response due to variation in structural damping, structural frequency and rock modulus were included in the study. A Latin Hypercube simulation was used to select the random variables (model parameter values) used in the analysis. Since the earthquake time histories selected were assumed to be equally likely, the sample size was set equal to the number of earthquake records provided. The damping ratios, frequencies, and rock modulus values were assumed to be lognormally distributed with medians and variabilities as shown:

Parameter	Median	β
Structure Frequency Ratio	1.0	0.25
Structure Damping	0.07	0.35
Rock Modulus Ratio	1.00	0.45

The time-history output from each of the 38 analyses was obtained for both horizontal directions for several locations in the building. From the floor response time histories, floor response spectra were generated. The spectral accelerations were arranged in descending order at each of the selected frequency points, and the median and 84th percentile values were extracted. The resulting median and 84th percentile floor spectra were then plotted and digitized for use in the fragility evaluations. The median and 84th percentile spectra at one of the building locations are depicted in Figure 1.

The combined variability associated with variation of the six parameters included in the variability study was determined by comparing the 5 percent damped median and 84th percentile floor spectra.

$$\beta_C = \ln (S_a84/S_a50)$$

It was found that the variabilities tend to be consistent over certain frequency bands. The structural variabilities (β_C) at the basemat and high in the structure were:

Elevation	Frequency Range		
	Low <0.5 f_n	Mid 0.6 f_n to 1.4 f_n	High >1.4 f_n
Basemat	0.24	0.26	0.24
High in structure	0.26	0.41	0.26

where f_n is the median frequency of the structure. It can be seen that the combined variability is relatively insensitive to changes in floor level in low and high frequency ranges, and in these frequency ranges, the combined variability is virtually all due to randomness. However, in the frequency band near the fundamental frequency of the building, it can be seen that at higher elevations, the combined variability increases substantially. The majority of the increase in the combined variability is due to the uncertainty associated with the structural property values and is assigned to β_U.

A detailed three-dimensional rock-structure interactions deterministic analysis was performed separately with median model parameters and a reference input motion which is consistent with the median shape derived from the time-histories used in the variability study. The 5 percent damped median reference floor spectra developed for selected locations in the building from the rock-structure interaction deterministic study were compared with those developed in structural response variability study. It was found from the comparison that the spectra showed good agreement. Thus, it was judged that the floor spectra obtained using a median input spectrum and medium structure properties could be used to approximate probabilistic median floor spectra obtained from multiple time-history analyses.

PROBABILISTIC NONLINEAR ANALYSIS OF A BUILDING STRUCTURE

Preliminary fragility (probabilistic seismic capacity) evaluation of one of the building structure obtained by using the standard separation-of-variables approach identified that this structure could possibly be a significant contributor to the seismically induced risk of core damage. Thus, it was determined that a probabilistically based, nonlinear evaluation of the building would be extremely valuable for the purposes of:

- Improving the probabilistic seismic capacity (fragility) estimates for severe overall distress of the building for use in the seismic probabilistic risk assessment.

- Comparing the fragility estimate based upon multiple nonlinear analyses with the estimates extrapolated from a single median-centered elastic response spectrum analysis obtained using the standard separation-of-variables fragility evaluation method.

It was concluded from the initial studies that the most probable cause of overall severe distress was substantial inelastic drift and strength degradation of two major east/west load-carrying shear walls which span from the foundation level to the operating floor (55 feet high, composed of three stories). Thus, the nonlinear analyses consisted of an assessment of the east/west response of the building, with emphasis on the two major east/west load-carrying shear walls.

The behavior of reinforced concrete walls under lateral loading can be typically illustrated by a shear force-shear distortion diagram which show reverse-cycle loading behavior characterized by stiffness degradation and pinching of the hysteresis loops. This behavior which is depicted by a typical shear force-shear distortion diagram (Wang, 1975) in Figure 2, was approximated by the 10 Rule hysteretic model shown on Figure 3 (Kennedy, 1984b).

To study the dispersion in response due to uncertainty in structure properties, a Monte Carlo technique was used in the nonlinear analysis. Important structure variables affecting structure response (damping, stiffness, and strength), were assumed to be lognormally distributed with median and logarithmic standard deviations as shown below:

		Logarithmic Standard Deviation		
Variable	Median Value	Random	Uncertainty	Composite
Damping	7%	0	.35	.25
Stiffness Ratio	1.0	0	.50	.50
Strength Ratio	1.0	0	.25	.25

Note that the stiffness and strength ratios were used to scale the median stiffness and median strengths of each of the structural elements of the nonlinear model. For each nonlinear analysis, the median stiffness and strengths of the shear walls and the operating floor were multiplied by a probabilistically defined stiffness and strength ratio. Stiffness and strength ratios were independently defined for each element (shear walls and operating floor). Thus, a given element could simultaneously have a high stiffness ratio and a low strength ratio. Similarly, shear walls could have a low strength ratio and the operating floor have a high strength ratio. 50 sets of stiffness ratios, strength ratios and damping were randomly selected.

Twenty-five earthquakes time history records were selected. Each of these records were scaled by constant-amplitude (frequency-independent) scaling factors to obtain the same average spectral acceleration in the frequency range of 3 to 8.5 Hz. Figure 4 depicts the mean, median, 84 percent probability of non-exceedance, and upper-bound spectra for the ensemble of 25 records scaled to an average spectral acceleration of 2.25 g.

Each of the 25 time histories, as modified above, scaled to an average spectral acceleration of 3.0 and 6.0 g (50 trials), were applied to the nonlinear structure model with median strength, stiffness and damping properties. In a similar manner, 50 nonlinear analyses each were conducted at an average spectral acceleration values of 3.0 g, 4.0 g, and 6.0 g (150 total trials), incorporating the randomly selected structure damping, stiffness and strength ratios. These analyses include both input motion randomness variability and structural property uncertainty. Thus, a total of 200 nonlinear analyses were performed and from each analysis, the maximum shear wall drift was determined.

To calculate the corresponding probability of severe distress, the onset of severe shear wall damage (significant strength degradation) was defined in terms of shear wall drift limits. The median estimate of shear wall drift (expressed as a percentage of story height) corresponding to the onset of significant strength degradation and the associated logarithmic standard deviations were taken as:

$\check{D} = 0.7\%$ (median drift limit)
$\beta_R = 0.15$
$\beta_U = 0.30$
$\beta_C = 0.335$

When treated on a composite basis (using β_C), there is about a 16 percent probability of severe distress at 0.5 percent drift and about an 84 percent probability of severe distress at 1.0 percent drift. These estimates might be more conservative than necessary. The probability of severe shear wall distress is estimated for each of the 200 trials using this random shear wall distress criteria. Defining P_{Fi} as the probability of P_F of severe distress of Trial i, the median estimate of the probability of each average spectral acceleration value is obtained from:

$$P_F = \frac{\sum_{i=1}^{N} P_{Fi}}{N}$$

Where N is the number of trials. Thus, the overall probability estimates for each case studies (randomness only at an average spectral acceleration of 3.0 g and 6.0 g and randomness plus uncertainty at an average spectral acceleration of 3.0 g, 4.0 g, and 6.0 g) were obtained. These results were then fit by a "best fit" lognormally distributed fragility estimate defined in terms of median, $\check{S}_a$, and logarithmic standard deviations for randomness variability, β_R, composite variability, β_C, and uncertainty variability, β_U. The high-confidence-low-probability-of-failure (HCLPF) capacity, defined as 95 percent confidence of less than 5 percent probability of failure, is calculated from:

$$\text{HCPLF } \bar{S}_a = \check{S}_a e^{-1.65(\beta_R + \beta_U)}$$

Thus, the building fragility estimate becomes:

$\check{S}_a$ = 4.59 g
β_C = 0.37 (from randomness and uncertainty runs)
β_R = 0.23 (from randomness only runs)
$\beta_U = (0.37^2 - 0.23^2)^{1/2} = 0.29$

$$\text{HCPLF } \bar{S}_a = 4.59\ e^{-1.65(.23 + .29)} = 1.95 \text{ g}$$

FRAGILITY ESTIMATE FROM A SINGLE MEDIAN CENTERED ELASTIC ANALYSIS

In the standard separation-of-variables approach the median fragility is estimated from an elastic analysis which has used the median input spectrum shape. The adequacy of this approach to approximate the fragility estimates was studied in relation to the multiple nonlinear analyses.

It was found that so long as the inelastic energy absorption capability factor F_μ is estimated by the procedures defined in NUREG/CR3805 (Kennedy, 1984b), the fragility estimated from multiple nonlinear analyses can be slightly conservatively approximated by a separation-of-variables extrapolation of a single median centered elastic response spectrum analysis using the input spectrum shape. The comparison of the fragility parameters, $\check{S}_a$, β_R, β_U, and HCLPF $\bar{S}_a$ estimated from extrapolating a single elastic analysis versus those obtained from the multiple nonlinear time history analyses is shown below.

Parameter	Multiple Nonlinear Analyses	Extrapolated Elastic Analyses
Median $\check{S}_a$	4.59	4.39
Randomness β_R	0.23	0.23
Uncertainty β_U	0.29	0.32
HCPLF $\bar{S}_a$	1.95	1.77

The extrapolated elastic analysis median $\check{S}_a$, is about 5% too low, the uncertainty β_U is about 10% too large, and the HCLPF $\bar{S}_a$ is about 10% too low. This slight conservatism is a minor penalty when compared to the great simplification introduced by being able to extrapolate the results of a single median centered elastic analysis.

It should be noted that a realistic estimate of the inelastic energy absorption capability, F_μ, should be made in order that a good inelastic fragility estimate can be derived from a median-centered elastic analysis. It was found that the Spectral Averaging Method (Kennedy, 1984b) provided an excellent estimate of F_μ over a broad range of maximum story drifts. It was also observed that the more commonly used Riddell-Newmark method provided too liberal an estimate for this nonlinear building model. However, when modified slightly into an effective Riddell-Newark method, the method provided equally good estimates of F_μ (Kennedy, 1988).

CONCLUSIONS

The special studies performed in the seismic fragility evaluation helped to provide better estimates of fragilities of structures and equipment by allowing a more rigorous determination of certain individual factors and their variabilities which contribute to the overall fragilities. The non-linear study showed that the fragility estimated from multiple nonlinear analyses can be slightly conservatively approximated by a separation-of variables extrapolation of a single median-centered elastic response spectrum analysis.

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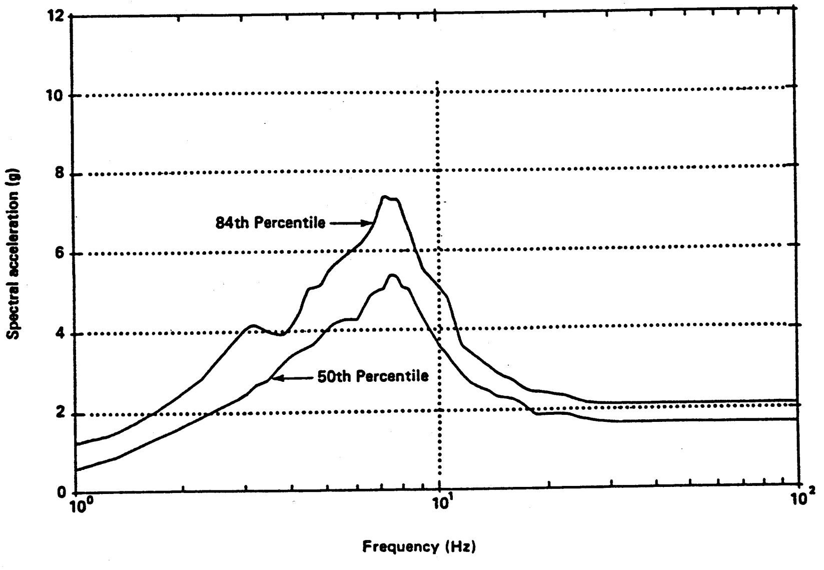

Figure 1

50th and 84th percentile response spectra (5% damping) at upper elevation.

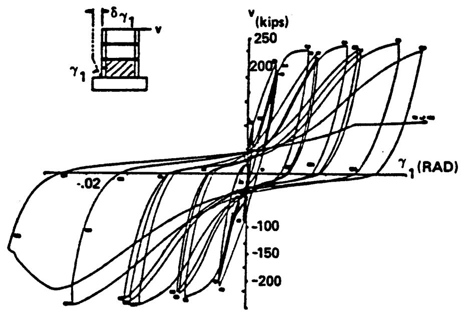

Figure 2

Cyclic load-deflection behavior of concrete shear walls (Wang, 1975).

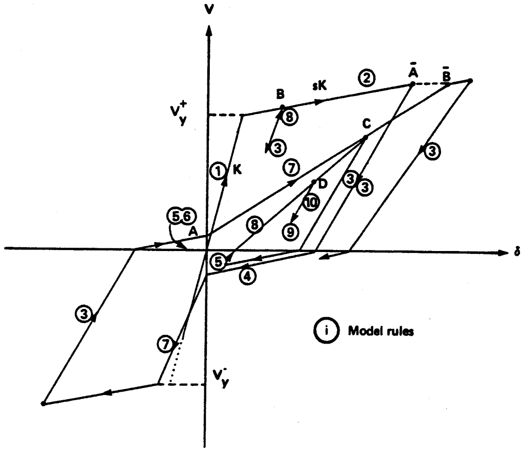

Figure 3

Shear deformation hysteretic behavior.

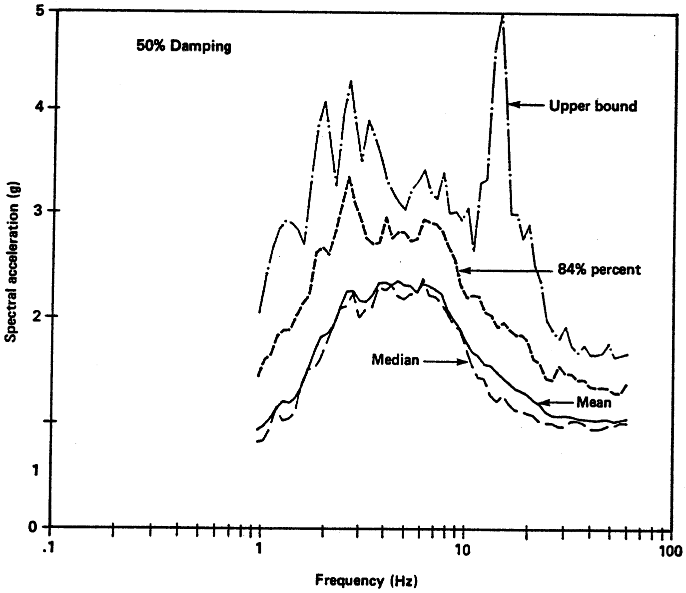

Figure 4

Mean, median, 84 percent probability of nonexceedance, and upper-bound spectra for 25 records scaled to an average spectral acceleration of 2.25 g over the frequency range of 3.0 to 8.5 hertz.