RAI Volume 2, Chapter 2.1.1.4, Ninth Set, Number 2:

Subject: Generation of seismic fragility curves for ITS buildings.

Seismic fragilities for ITS buildings are developed from an approximation of the capacity at one percent conditional probability of failure, $C_{1\%}$, and a composite logarithmic standard deviation, β, estimated by judgment (DOE 2007ab, Section 4.4.2).

2.1 Provide technical basis for generating the fragility curves of ITS buildings based on an approximation of $C_{1\%}$ and an assumed β value, instead of using the separation of variables method (SAR Section 1.7.2.4). Compare the probability of failure of ITS buildings using both methods to justify the approach used by DOE to generate the fragility curves.

The seismic risk analysis of the ITS buildings include unique characteristics that may not be captured with a simplified methodology in which seismic fragilities are obtained from an approximation of $C_{1\%}$ and an assumed β. For instance, some ITS buildings include structural characteristics not considered in the calibration of the method, such as a concrete pool or structural irregularities. Also, the difference between the mean annual frequency of exceedance for the seismic design level and the performance level limit is more than two orders of magnitude.

2.2 For ITS buildings that may experience nonlinear behavior, provide technical bases for not performing nonlinear structural analyses to support the generation of fragility curves. Alternatively, provide nonlinear structural analyses of the structures when subjected to seismic levels associated to the expected range of conditional probabilities of failure.

The building capacity at $C_{1\%}$ is obtained from a simplified linear elastic analysis (BSC 2007ba, Section B.4.2). The structural response at this seismic level, however, is expected to be inelastic or close to the inelastic threshold. Therefore, a fragility curve based on linear analyses at $C_{1\%}$ is unlikely to represent the expected nonlinear response of reinforced concrete, steel, and soil materials at higher seismic solicitations, especially for buildings exhibiting horizontal and vertical structural irregularities.[1]

2.3 Demonstrate that the selected β range (from 0.3 to 0.5) is consistent with the structural systems, seismic hazard, and refinement of the numerical models used in the evaluation.

For ITS buildings, DOE (2007, Section 4.4.2) selected a β range from 0.3 to 0.5 based on ASCE (2005, Section C2.2.1.2). This range, however, should be

[1] Structural irregularities as defined in ASCE/SEI (2005b; Section 12.3).

justified, given that different studies suggest β values larger than 0.5 for reinforced concrete systems (Campbell et al. 1988; Kennedy and Ravindra, 1984; and Shinozuka et al. 2003). Also, epistemic uncertainty could increase the β values because ITS buildings are analyzed using simplified linear elastic methods to predict potential nonlinear response under low-probability seismic events.

2.4 For the generation of fragility curves, BSC (2007ba, Section B4.3) recommends to use the lower bound value, β = 0.3, to obtain a higher probability of unacceptable behavior. Provide technical bases to demonstrate that for all the systems evaluated with this method, the probability of unacceptable behavior increases as the fragility parameter β decreases.

2.5 For the generation of fragility curves, provide technical basis for using a value of 0.4 for the composite logarithmic standard deviation, β (Table 6.2-1 of BSC 2008bg), instead of the lower bound value, β = 0.3. This lower bound is recommended to obtain a higher probability of unacceptable behavior (BSC 2007ba, Section B4.3, Step 7).

1. RESPONSE

1.1 TECHNICAL BASIS FOR THE METHOD USED TO GENERATE FRAGILITY CURVES OF THE IMPORTANT TO SAFETY FACILITIES

NUREG/CR-4334, *An Approach to the Quantification of Seismic Margins in Nuclear Power Plants* (Budnitz et al. 1985) documents the recommended guidance of an expert panel on quantification of seismic margins. The panel expressed a preference for establishing high confidence of low probability of failure seismic capacity directly by conservative deterministic computations as opposed to back-computing this capacity from median capacity and variability estimates, as is done in the fragility analysis method approximated by the separation of variables approach (Budnitz et al. 1985, p. 3).

The conservative deterministic failure margin method was refined and published with numerous examples in *A Methodology for Assessment of Nuclear Power Plant Seismic Margin (Revision 1)* (EPRI 1991). The conservative deterministic failure margin method aims directly at estimating the 1% failure probability capacity ($C_{1\%}$) point on the fragility curve. NUREG-1407, *Procedural and Submittal Guidance for the Individual Plant Examination of External Events (IPEEE) for Severe Accident Vulnerabilities, Final Report* (Chen et al. 1991), endorsed both the fragility analysis method and the conservative deterministic failure margin method for estimating the high confidence of low probability of failure capacity of any component. More recently, the conservative deterministic failure margin method has been further updated in ASCE/SEI 43-05, *Seismic Design Criteria for Structures, Systems, and Components in Nuclear Facilities*.

"Overview of Methods for Seismic PRA and Margin Analysis Including Recent Innovations" (Kennedy 2001) discusses the development of seismic fragility curves by computing the $C_{1\%}$ point on the fragility curve by the conservative deterministic failure margin method and then estimating the composite variability natural logarithmic standard deviation β_c. For structures, it is recommended that β_c should lie in the range of 0.3 to 0.5. The annual probability of unacceptable performance (P_F) is then obtained by numerically convolving the seismic hazard and fragility curves. It is shown in both the methodology overview (Kennedy 2001) and "Performance-Goal Based (Risk Informed) Approach for Establishing the SSE Site Specific Response Spectrum for Future Nuclear Power Plants" (Kennedy 2007) that for β_c in the range of 0.3 to 0.6, the resulting P_F is not particularly sensitive to β_c so long as the $C_{1\%}$ capacity point has been defined. The P_F computed for $\beta_c = 0.3$ is approximately 1.5 times that computed for $\beta_c = 0.4$, and the P_F computed for $\beta_c = 0.6$ is approximately 0.7 times that computed for $\beta_c = 0.4$. Thus, a conservative estimate of P_F for structures can be obtained using $\beta_c = 0.3$, and a more median-centered estimate can be obtained using $\beta_c = 0.4$.

The conservative deterministic failure margin method provides the most accurate estimate of the $C_{1\%}$ capacity point on the fragility curve, but a less accurate determination of the median capacity ($C_{50\%}$) because β_c is estimated as opposed to being computed. The fragility analysis method separation of variables approach defined in *Methodology for Developing Seismic Fragilities* (EPRI 1994) computes both $C_{50\%}$ and β_c and back estimates $C_{1\%}$. Because of uncertainties in computing $C_{50\%}$ and β_c, the $C_{1\%}$ capacity estimated by the fragility analysis method is considered to be less reliable than the $C_{1\%}$ capacity directly computed by the conservative deterministic failure margin method. For typical hazard curve slopes, the computed P_F is dominated by ground motions spread between the $C_{1\%}$ and $C_{50\%}$ capacity points. Thus, computing $C_{1\%}$ by the conservative deterministic failure margin method and estimating β_c or computing $C_{50\%}$ and β_c by the fragility analysis method separation of variables approach provides equally realistic P_F estimates.

The conservative deterministic failure margin method has been chosen for estimating the fragility of the important to safety (ITS) facilities. The $C_{1\%}$ capacity can be more reliably estimated by the conservative deterministic failure margin method than by the fragility analysis method separation of variables approach for any building, including the ITS facilities with structural irregularities or that contain a concrete pool. The $C_{1\%}$ capacity estimate is independent of the mean annual frequency of exceedance (H_e) initially established for the seismic design level. It is influenced by the conservatism included in the design. Once $C_{1\%}$ is defined, the annual failure probability P_F can be computed. The magnitude range between H_e and the Category 2 event sequence lower threshold for P_F (stated as the performance level limit in the RAI) is irrelevant to the method.

1.2 TECHNICAL BASIS FOR NOT PERFORMING NONLINEAR ANALYSES

In accordance with guidance presented in *A Methodology for Assessment of Nuclear Power Plant Seismic Margin (Revision 1)* (EPRI 1991) and ASCE/SEI 43-05, the $C_{1\%}$ capacity is computed for the ITS facilities by multiplying the linear elastic computed capacity to demand ratio $(C/D)_e$ (stated as the strength margin factor F_s in the fragility evaluations) by an appropriate inelastic factor F_μ and by the ground motion parameter for which the structure has been evaluated (e.g., the peak horizontal ground acceleration of the beyond design basis ground motion PGA_{BDBGM}):

$$C_{1\%} = (C/D)_e \times F_\mu \times PGA_{BDBGM} \quad \text{(Eq. 1).}$$

This same basic approach is recommended in *Methodology for Developing Seismic Fragilities* (EPRI 1994) for fragility analyses conducted by the fragility analysis method separation of variables approach. The only difference for this approach is that median and variability estimates are made for $(C/D)_e$ and F_μ instead of conservatively biased estimates.

A relatively small number of nonlinear time history analyses have been performed for structures in seismic probabilistic risk assessments submitted to the NRC. The majority of structure fragility or margin analysis has been performed using the basic approach as in Equation 1. Therefore, the approach followed for the structural fragility evaluation is consistent with the current state of practice.

One case where nonlinear time history analyses were performed was the fragility and margin analyses of the concrete shear wall structure of the Diablo Canyon Turbine Building below the operating deck. This building exhibited structural irregularities. Two hundred nonlinear time history analyses of a simplified model of this structure with probabilistically defined properties were performed. Results were summarized in *Final Report of the Diablo Canyon Long Term Seismic Program* (PG&E 1988). The approach for estimating F_μ ultimately recommended in *A Methodology for Assessment of Nuclear Power Plant Seismic Margin (Revision 1)* (EPRI 1991) was benchmarked against these nonlinear analyses. Details of these analyses, and a comparison with F_μ results obtained using the recommended approach (EPRI 1991), are presented in *Probabilistic Evaluation of the Diablo Canyon Turbine Building Seismic Capacity Using Nonlinear Time History Analyses* (Kennedy et al. 1988). This comparison showed excellent agreement between F_μ values derived from nonlinear analysis and those obtained from the approach. Therefore, the recommended approach (EPRI 1991) for estimating F_μ is representative for structures with structural irregularities.

1.3 JUSTIFICATION FOR SELECTED RANGE OF COMPOSITE STANDARD DEVIATION

The selected composite β_c range from 0.3 to 0.5 for reinforced concrete structures was based on the recommendation from "Overview of Methods for Seismic PRA and Margin Analysis Including Recent Innovations" (Kennedy 2001), which in turn was based on both the structural fragility evaluations summarized in *Compilation of Fragility Information from Available Probabilistic Risk Assessments* (Campbell et al. 1988) and the Diablo Canyon fragility evaluation (PG&E 1988), which was not summarized in the compilation.

Compilation of Fragility Information from Available Probabilistic Risk Assessments (Campbell et al. 1988) reports random variability β_r and uncertainty β_u estimates for 58 concrete shear wall and concrete diaphragm failure modes for structures similar to the ITS facilities. These 58 sets of results are presented in Table 1, which also shows the composite β_c computed by Equation 2:

$$\beta_c = \left[\beta_r^2 + \beta_u^2\right]^{0.5} \quad \text{(Eq. 2).}$$

The column in Table 1 labeled REC# presents the record number in the compilation's tabulations (Campbell et al. 1988). Based on these 58 records, the median value of β_c is 0.42, and the full range of the estimated β_c is 0.26 to 0.72, with the middle 67% of the results lying in the range of 0.35 to 0.54.

The fragility estimates summarized in *Compilation of Fragility Information from Available Probabilistic Risk Assessments* (Campbell et al. 1988) were performed prior to publication of *A Methodology for Assessment of Nuclear Power Plant Seismic Margin (Revision 1)* (EPRI 1991) and *Methodology for Developing Seismic Fragilities* (EPRI 1994). In general, the fragility estimates in the compilation were not performed consistent with all of the recommendations in the later methodologies. In particular, the structural inelastic factor F_μ was estimated in a simpler manner than the more rigorous approach recommended in *A Methodology for Assessment of Nuclear Power Plant Seismic Margin (Revision 1)* (EPRI 1991) and *Methodology for Developing Seismic Fragilities* (EPRI 1994). To compensate for the simplicity with which some of these earlier estimates were made, the estimates of β_u and β_r were increased. As a result, some of the β_u and β_r values reported in Table 1 are too large to be considered consistent with fragility evaluations conducted in accordance with the more rigorous recommendations of the two methodologies (EPRI 1991; EPRI 1994).

In Table 1, two cases are reported with β_c less than 0.30. These cases are REC# 1235, for which β_r = 0.01, and REC# 1299, for which β_r = 0.11. Both of these estimates of β_r are not credible. One source of β_r, the horizontal ground motion directional variability β_{dir}, is recommended in *Methodology for Developing Seismic Fragilities* (EPRI 1994) to be 0.13. Using this value, and considering the other contributing sources, a credible estimate of β_r cannot be less than 0.15, and even that estimate is low. Every β_r estimate in Table 1 less than 0.15 is not credible and should be increased. With such an increase, none of the β_c values reported in Table 1 will be less than 0.30.

On the opposite extreme, the β_u estimates for REC# 1277, 1308, 1309, 1310, and 1311 range between 0.52 and 0.64, which are very high. A β_u of 0.52 corresponds to a 90% confidence bandwidth on the median capacity of 5.5, while β_u of 0.64 corresponds to a 90% confidence bandwidth on the median capacity of 8.2. These β_u estimates are not credible. For structures, β_u estimates in excess of about 0.4 are indicative that simplistic capacity C and inelastic factor F_μ evaluations were performed, and that the fragility analyst compensated for this simplicity by increasing the estimate of β_u.

Similarly, for REC# 1231, 1232, and 1277, high estimates for both β_r and β_u were made. These estimates were not consistent with the degree of rigor recommended in the two methodologies (EPRI 1991; EPRI 1994) for computing demand D, capacity C, and inelastic factor F_μ.

Deleting the outlier cases discussed above (i.e., REC# 1231, 1232, 1235, 1277, 1299, and 1308 through 1311), a total of 49 cases remain in Table 1. For these remaining 49 cases, the median β_c = 0.40 and the range β_c = 0.30 to 0.55.

Significant advancements in the seismic fragility and seismic margin methodologies were made as a result of the multiyear studies conducted as part of the Diablo Canyon seismic probabilistic risk assessment and seismic margin review (PG&E 1988). Several large probabilistic studies were conducted as part of that review, including the extensive benchmarking of the conservative deterministic failure margin method against the fragility analysis method separation of variables approach and improvements in the methodology used for estimating F_{μ}. The primary authors of *A Methodology for Assessment of Nuclear Power Plant Seismic Margin (Revision 1)* (EPRI 1991) and *Methodology for Developing Seismic Fragilities* (EPRI 1994) were directly involved in these studies. Many of the seismic margin and seismic fragility methodology recommendations for structures presented in the Electric Power Research Institute (EPRI) methodologies (EPRI 1991; EPRI 1994) resulted from these Diablo Canyon studies.

The fragility analyses summarized in *Compilation of Fragility Information from Available Probabilistic Risk Assessments* (Campbell et al. 1988) do not reflect these methodology improvements. These improvements should lead to a reduction in the higher β_u estimates commonly made in many of the earlier fragility evaluations. As a result, for margin and fragility evaluations of structures conducted in accordance with the recommendations presented in the EPRI methodologies (EPRI 1991; EPRI 1994), the upper range on β_c as shown above should be slightly reduced. Guidance in this regard can be obtained from the β_r, β_u, and β_c values reported in *Final Report of the Diablo Canyon Long Term Seismic Program* (PG&E 1988) for structures. These values are summarized in Table 2. From Table 2, the median $\beta_c = 0.40$ and the range $\beta_c = 0.33$ to 0.42.

The $C_{1\%}$ capacity evaluations for the ITS facilities have been conducted in accordance with the recommendations of *A Methodology for Assessment of Nuclear Power Plant Seismic Margin (Revision 1)* (EPRI 1991). Therefore, for the ITS facilities, the median $\beta_c = 0.40$ and the range $\beta_c = 0.30$ to 0.50 are appropriate.

The RAI also refers to "Seismic Fragilities for Nuclear Power Plant Risk Studies" (Kennedy and Ravindra 1984). Although this reference was written prior to the EPRI methodology (EPRI 1991) and does not explicitly provide recommendations for β_c, it supports the statements made above.

Lastly, the RAI also refers to *Statistical Analysis of Fragility Curves* (Shinozuka et al. 2003). This report provides both empirical and analysis-based fragilities for bridges. Two sets of empirical data are presented. One data set is for Caltrans' bridges subjected to the Northridge earthquake. The second set is for Hanshin Expressway Public Corporation's report on damage to reinforced concrete bridge columns from the Kobe earthquake. Table 3 shows the log-standard deviation β_c reported for these two sets of bridge data. Both data sets cover a wide variety of generic reinforced concrete highway bridges. Generic data should have a substantially higher β_c than is appropriate for an individual structure with well-defined design properties. Because the β_c values shown in Table 3 are for a generic class of bridges, these values are inappropriately high for an individual, well-defined structure at a specific site.

Statistical Analysis of Fragility Curves (Shinozuka et al. 2003) also provides analysis based fragilities for two bridges in Memphis, Tennessee. The reported log-standard deviations β_{CR} are shown in Table 4. These β_{CR} values are too low because many of the sources of variability required to be considered in Table 3-1 of *Methodology for Developing Seismic Fragilities* (EPRI 1994) were not included in these two fragility analyses. The only sources of variability considered were concrete and steel strength variability and input ground motion variability.

The demand sources of variability not included in the analyses (Shinozuka et al. 2003) were as follows:

- Damping
- Modeling
  - Structure frequency
  - Mode shape
- Soil-structure interaction.

The variability for these demand sources β_{DI} is approximately 0.20 or higher.

In the strength evaluation, a deterministic nominal strength equation was used, and only material properties were varied. The strength equation uncertainty β_{EQN} also needs to be included. Assuming that the computed failure mode was flexure, β_{EQN} is approximately 0.10. For other failure modes, β_{EQN} will be somewhat higher.

The uncertainty in ductility levels associated with damage was also not included. A reasonable estimate of β_{μ} is approximately 0.20 or higher. An adjusted β_c, which includes the sources of variability, can be computed using Equation 3:

$$\beta_c = \left[\beta_{CR}^2 + \beta_{DI}^2 + \beta_{EQN}^2 + \beta_{\mu}^2\right]^{0.5} \quad \text{(Eq. 3).}$$

These adjusted β_c values are also shown in Table 4, and are likely to be too low for the ITS facilities for two reasons:

1. The β_{DI}, β_{EQN}, and β_{μ} estimates used in Equation 3 are likely to be low.

2. The bridge structures considered are simpler structures than the ITS facilities.

Nothing reported in *Statistical Analysis of Fragility Curves* (Shinozuka et al. 2003) invalidates the median value and range on β_c values for the ITS facilities (i.e., 0.40, and 0.30 to 0.50, respectively).

1.4 TECHNICAL BASIS TO DEMONSTRATE THE PROBABILITY OF UNACCEPTABLE PERFORMANCE INCREASES AS THE COMPOSITE STANDARD DEVIATION DECREASES

Starting with a specified $C_{1\%}$ capacity, both "Overview of Methods for Seismic PRA and Margin Analysis Including Recent Innovations" (Kennedy 2001) and "Performance-Goal Based (Risk Informed) Approach for Establishing the SSE Site Specific Response Spectrum for Future Nuclear Power Plants" (Kennedy 2007) have demonstrated through numerous examples that the computed annual probability of failure P_F is higher for $\beta_c = 0.30$ than it is for higher values of β_c. Numerous convolutions of hazard curves and fragility curves have shown that the predominant fractile of the total computed P_F comes from ground motions between the $C_{1\%}$ and $C_{50\%}$ capacities. With increasing β_c, the ratio $C_{50\%}/C_{1\%}$ becomes larger. For example:

β_c	$C_{50\%}/C_{1\%}$
0.30	2.01
0.40	2.54
0.50	3.20
0.60	4.04

So long as the predominant fractile of the total P_F comes from ground motion in excess of $C_{1\%}$, the use of $\beta_c = 0.30$ must always produce a higher computed P_F than would be obtained if a higher β_c estimate were used.

One of the primary advantages of the conservative deterministic failure margin method over the fragility analysis method approximated by the separation of variables approach is that the conservative deterministic failure margin method is aimed at estimating the $C_{1\%}$ capacity instead of the median $C_{50\%}$ capacity. Starting with the $C_{1\%}$ capacity, a conservative P_F estimate can be obtained by underestimating β_c, and $\beta_c = 0.30$ represents a practical lower bound on β_c, as discussed in Section 1.3 of this response. Therefore, a rigorous estimate of β_c is unnecessary if the fragility is based on the conservative deterministic failure margin method.

The opposite holds true with the fragility analysis method approximated by the separation of variables approach, which is aimed at estimating the $C_{50\%}$ capacity. In this case, an underestimate of β_c will result in an unconservative estimate of P_F. As a result, a rigorous estimate of β_c is more important with the fragility analysis method.

An example is presented to illustrate the variation in the computed P_F as a function of $\beta_c = 0.3$, 0.4, 0.5, and 0.6. For this example, the surface facilities area horizontal mean hazard curve for peak ground acceleration shown in Figure 1 and tabulated in Table 5 is used. The $C_{1\%}$ capacity for the Canister Receipt and Closure Facility (CRCF) is used. For the CRCF, $C_{1\%} = 1.82$ g. For $\beta_c = 0.3$, 0.4, 0.5, and 0.6, Table 6 presents the computed median capacity $C_{50\%}$, the total annual failure probability P_F, the failure probability $\Delta P_{F1\%}$ associated with ground motion less than $C_{1\%}$, and the failure probability $\Delta P_{F50\%}$ associated with ground motion greater than $C_{50\%}$. The following observations can be made from the results shown in Table 6:

1. The total computed P_F is a factor of approximately 1.5 greater for $\beta_c = 0.3$ than for a median centered $\beta_c = 0.4$, whereas P_F for $\beta_c = 0.6$ is a factor of approximately 0.7 times that for $\beta_c = 0.4$.
2. The fraction of P_F resulting from ground motions less than $C_{1\%} = 1.82$ g is small, ranging from approximately 0.05 for $\beta_c = 0.3$ to approximately 0.25 for $\beta_c = 0.6$.
3. The fraction of P_F resulting from ground motions exceeding $C_{50\%}$ is even smaller, ranging from approximately 0.15 for $\beta_c = 0.3$ to negligible for $\beta_c = 0.6$.
4. The fraction of P_F resulting from ground motions between $C_{1\%}$ to $C_{50\%}$ is approximately 0.8 for $\beta_c = 0.3$ to 0.5, and approximately 0.75 for $\beta_c = 0.6$.

Although these specific reported fractile ΔP_F results are specific to the hazard curve shape shown in Figure 1 and to the location of $C_{1\%} = 1.82$ g on this hazard curve, the trend of results is consistent with results obtained from many other convolutions of hazard curves and fragility curves.

The $P_F = 8.07 \times 10^{-7}$ shown in Table 6 is approximately 3% higher than the $P_F = 7.8 \times 10^{-7}$ reported in Table 6.2.1 of *Seismic Event Sequence Quantification and Categorization Analysis* (BSC 2009) for the CRCF with $\beta = 0.40$. The difference is due to using a finer integration interval and extending the integration to a higher ground motion in order to be able to define $\Delta P_{F1\%}$ and $\Delta P_{F50\%}$. This difference is negligible.

So long as the fragility curve is defined by $C_{1\%}$ capacity, it is extremely unlikely for a higher β_c to result in an increase in the computed P_F. This situation can only arise when more than 50% of the computed P_F comes from ground motion below $C_{1\%}$ (i.e., the extreme lower tail of the lognormal fragility curve).

1.5 TECHNICAL BASIS FOR USING A COMPOSITE STANDARD DEVIATION VALUE OF 0.4

The technical basis for using a median estimate of β_c is presented in Section 1.3 of this response. Use of $\beta_c = 0.4$ in Table 6.2-1 of *Seismic Event Sequence Quantification and Categorization Analysis* (BSC 2009) provides a median estimate of P_F. However, as shown in Table 6 of this response, using a lower bound $\beta_c = 0.3$ results in a conservative $P_F = 1.21 \times 10^{-6}$ per year, which is still less than the Category 2 event sequence lower threshold of 2×10^{-6} per year.

2. COMMITMENTS TO NRC

None.

3. DESCRIPTION OF PROPOSED LA CHANGE

None.

4. REFERENCES

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Table 1. Variability Reported in Campbell et al. 1988 for Concrete Shear Wall Structures and Concrete Diaphrams

REC#	β_r	β_u	β_c	
1204	0.2	0.31	0.37	
1205	0.11	0.33	0.35	
1206	0.13	0.27	0.30	
1208	0.24	0.26	0.35	
1211	0.23	0.29	0.37	
1212	0.31	0.26	0.40	
1213	0.29	0.27	0.40	
1214	0.31	0.26	0.40	
1215	0.31	0.26	0.40	
1216	0.3	0.29	0.42	
1217	0.22	0.27	0.35	
1218	0.20	0.22	0.30	
1219	0.22	0.24	0.33	
1220	0.29	0.26	0.39	
1222	0.31	0.24	0.39	
1227	0.38	0.36	0.52	
1230	0.39	0.39	0.55	
1231	0.43	0.44	0.62	*
1232	0.43	0.44	0.62	*
1234	0.34	0.37	0.50	
1235	0.01	0.26	0.26	*
1238	0.32	0.40	0.51	
1241	0.24	0.35	0.42	
1242	0.27	0.36	0.45	
1244	0.12	0.29	0.31	
1245	0.21	0.33	0.39	
1251	0.31	0.26	0.40	
1258	0.31	0.26	0.40	
1261	0.25	0.35	0.43	
1264	0.36	0.40	0.54	
1267	0.31	0.41	0.51	
1269	0.33	0.41	0.53	
1273	0.33	0.39	0.51	
1274	0.34	0.43	0.55	
1275	0.29	0.39	0.49	
1276	0.33	0.43	0.54	
1277	0.50	0.52	0.72	*
1278	0.36	0.40	0.54	
1281	0.22	0.24	0.33	
1283	0.25	0.34	0.42	
1284	0.29	0.30	0.42	

REC#	β_r	β_u	β_c	
1285	0.27	0.38	0.47	
1287	0.21	0.34	0.40	
1288	0.24	0.33	0.41	
1292	0.24	0.27	0.36	
1294	0.28	0.30	0.41	
1298	0.29	0.33	0.44	
1299	0.11	0.26	0.28	*
1300	0.24	0.32	0.40	
1301	0.30	0.33	0.45	
1302	0.30	0.33	0.45	
1303	0.17	0.29	0.34	
1304	0.30	0.28	0.41	
1306	0.24	0.27	0.36	
1308	0.17	0.63	0.65	*
1309	0.19	0.61	0.64	*
1310	0.24	0.56	0.61	*
1311	0.16	0.64	0.66	*

NOTE: *Outlier cases discussed in text

Median β_c = 0.42
Full Range β_c = 0.26 – 0.72
Mid 2/3 Range β_c = 0.35 – 0.54

Table 2. Variability Reported in PG&E, 1988 for Diablo Canyon Structure Fragility Evaluations

Structure	β_r	β_u	β_c
Containment Building	0.26	0.30	0.40
Concrete Internal Biostructure	0.20	0.31	0.37
Intake Structure	0.28	0.31	0.42
Auxiliary Building	0.21	0.26	0.33
Turbine Building Shear Wall	0.26	0.33	0.42

NOTE: Median β_c = 0.40
Range β_c = 0.33 – 0.42

Table 3. Variability β_c from Shinozuka et al. 2003 for Generic Bridge Damage

Damage Level	β_c	
	Caltrans	HEPC
Minor	0.84	0.59
Moderate	0.72	0.45
Major	0.65	0.43
Collapse	0.67	—

NOTE: HEPC = Hanshin Expressway Public Corporation

Table 4. Variability β_c Estimates from Shinozuka et al. 2003 for Bridge Damage Based on Analysis

	Bridge 1		Bridge 2	
	Minor Damage	**Major Damage**	**Minor Damage**	**Major Damage**
Reported β_{CR}	0.18	0.13	0.20	0.31
Adjusted β_c	0.35	0.33	0.36	0.43

Table 5. Surface Facilities Area Horizontal Mean Hazard Data for Peak Ground Acceleration

Spectral Acceleration (g)	Mean Annual Probability of Exceedance (/year)
0.01449	3.18×10^{-2}
0.01879	3.18×10^{-2}
0.02437	3.14×10^{-2}
0.03161	2.78×10^{-2}
0.04100	2.17×10^{-2}
0.05317	1.62×10^{-2}
0.06896	1.17×10^{-2}
0.08944	8.43×10^{-3}
0.11599	5.86×10^{-3}
0.15044	3.90×10^{-3}
0.19511	2.53×10^{-3}
0.25305	1.60×10^{-3}
0.32819	9.82×10^{-4}
0.42564	5.73×10^{-4}
0.55203	3.24×10^{-4}
0.71595	1.80×10^{-4}
0.92855	9.51×10^{-5}
1.20428	4.71×10^{-5}
1.56188	2.17×10^{-5}
2.02568	8.61×10^{-6}
2.62719	2.44×10^{-6}
3.40732	4.89×10^{-7}
4.41910	8.95×10^{-8}
5.73132	1.50×10^{-8}
7.43320	1.45×10^{-9}

Table 6. Annual Failure Probabilities P_F Corresponding to $C_{1\%}$ = 1.82 g for Several β_c Estimates

β_c	$C_{50\%}$ (g)	P_F $\times 10^{-7}$	$\Delta P_{F1\%}$ $\times 10^{-7}$	$\Delta P_{F50\%}$ $\times 10^{-7}$
0.3	3.657	12.08	0.65	1.82
0.4	4.615	8.07	0.92	0.43
0.5	5.823	6.40	1.28	0.07
0.6	7.348	5.76	1.73	0.01

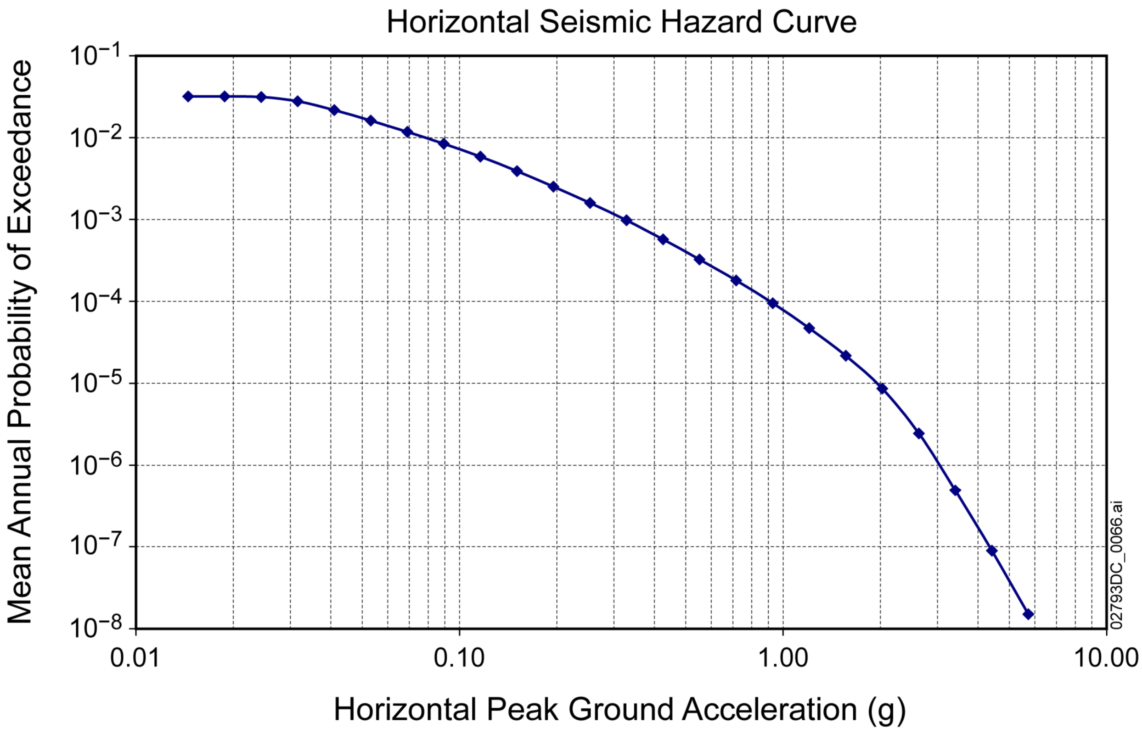

Figure 1. Surface Facilities Area Horizontal Mean Hazard Curve for Peak Ground Acceleration