Overview of Methods for Seismic PRA and Margin Analysis Including Recent Innovations

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1. Introduction

This paper will compare and contrast Seismic Probabilistic Risk Assessment (SPRA) and Seismic Margin Methods and will discuss the potential advantages gained from a hybrid of the two methods (Hybrid Method). A brief overview of both the SPRA and the Seismic Margin Methods will be presented. However, details of both methods will not be presented because these details are readily available in the referenced publication. For example, Refs. 1 through 4 provide details on the SPRA Method with the emphasis of Refs. 3 and 4 being on seismic fragility assessment. Similarly, Refs. 5 through 10 provide details on the Seismic Margin Method. Many additional references on both methods also exist. The Hybrid Method will be discussed in more detail because it is only briefly presented in Ref. 4.

2. Seismic Probabilistic Risk Assessment Method

2.1 Overview of SPRA Method

Figure 1 schematically and simplistically illustrates the critical steps of a SPRA. This figure defines the steps necessary for estimating the mean core damage frequency. However, it can be easily extended to estimating the entire probability distribution function on the core damage frequency. Estimating the mean core damage frequency will be discussed first.

2.1.1 Seismic Hazard Estimate:

In order to estimate a mean core damage frequency, a mean seismic hazard estimate is required. This estimate defines the mean annual frequency of exceedance H (called herein *hazard exceedance frequency*) versus ground motion levels for a specified ground motion quantity. The ground motion quantity can be the peak ground acceleration (PGA), 5% damped spectral acceleration (SA) at any specified natural frequency, average 5% damped spectral acceleration ($\overline{SA}$) over a specified natural frequency range, or any other ground motion quantity of interest. The only requirement is that the same ground motion quantity be used for the hazard estimate and all seismic fragility estimates. It is strongly preferable that this ground motion quantity be highly correlated with the damage of critical structures, systems, or components being considered in the SPRA.

If uncertainty is to be propagated through the SPRA in order to determine the probability distribution on the core damage frequency, then uncertainty in the hazard curve must be shown by the use of multiple possible hazard curves with a probability weighting assigned to each curve. These probability weights must add to unity. Typically about 5 to 10 probability weighted hazard curves are used.

Figure 2 shows some representative mean seismic hazard curves. The ground motion quantities shown in Figure 2 are the 5% damped spectral accelerations SA at 1 and 10 HZ natural frequencies. Ten hazard curves typical of eastern and central U.S. sites (labeled EUS sites) are shown. Also shown are hazard curves for a California and Washington site. In addition, the hazard curve for a typical site in the United Kingdom is shown. All hazard curves are normalized to their spectral acceleration value at an exceedance frequency of $1x10^{-4}$ for ease of comparison of hazard curve shapes. It can be seen that the UK hazard curve shape is similar to the typical EUS hazard curve shapes.

An important aspect of these hazard curves is their slope. Typical seismic hazard curves are close to linear when plotted on a log-log scale (for example see Fig. 2). Thus over at least any ten-fold difference in exceedance frequencies such hazard curves may be approximated by:

$$H_{(a)} = K_I a^{-K_H} \quad (1)$$

where $H_{(a)}$ is the annual frequency of exceedance of ground motion level "a", K_I is an appropriate constant, and K_H is a slope parameter defined by:

$$K_H = \frac{1}{\log(A_R)} \quad (2)$$

in which A_R is the ratio of ground motions corresponding to a ten-fold reduction in exceedance frequency. Either A_R or K_H may be used to define the slope of the hazard curve. The ground motion ratio A_R will be used herein. A large A_R represents a shallow-sloped hazard curve, whereas a small A_R represents a steep hazard curve. Within the annual frequency range of interest, A_R typically lies in the range of 2 to 4 for EUS and UK sites, and in the range of 1.75 to 2.25 for California sites.

2.1.2 Seismic Fragility Estimates:

For each seismic safety significant structure, system, or component (SSC) a seismic fragility curve must be estimated in terms of the same ground motion quantity for which the hazard curve is defined.

Figure 3 shows a representative SSC fragility curve defining the conditional probability of failure (unacceptable performance) P_F versus ground motion level for a specified ground motion quantity. The ground motion quantity shown in Figure 3 is the 5% damped spectral acceleration SA at 5 Hz natural frequency. In the most general case, fragility curves are typically described in terms of a lognormal distribution defined by the medium capacity $C_{50\%}$ and two lognormally distributed random variables with logarithmic standard deviations β_{UNC} and β_{RAND} which define the uncertainty and randomness, respectively, of the fragility curve. The uncertainty β_{UNC} defines the width of the confidence bands of the fragility curves shown in Figure 2 while the randomness β_{RAND} defines the shape of each fragility curve in Figure 2. If one does not propagate uncertainties of both the hazard curve and fragility curves in a seismic risk assessment, it is sufficient to define the fragility curve by a single mean (composite) fragility curve defined by a median capacity $C_{50\%}$ and composite logarithmic standard deviation β given by:

$$\beta = \left[\beta^2_{UNC} + \beta^2_{RAND}\right]^{1/2} \quad (3)$$

Mean seismic risk estimates can be rigorously obtained by convolving the mean hazard curve with the mean fragility curve. Similarly, median seismic risk estimates can be very closely approximated by convolving the median hazard curve with the mean fragility curve. Most recent Seismic Probabilistic Risk Assessment (SPRA) studies have concentrated on defining either mean or median seismic risk, and therefore have used mean fragility curves defined by $C_{50\%}$ and β as opposed to dividing the fragility variability into both β_{UNC} and β_{RAND}. This paper will concentrate on mean or median seismic risk estimates.

For structures and major passive mechanical components mounted on the ground or at low elevations within structures, β typically ranges from 0.3 to 0.5. For active components mounted at high elevations in structures the typical β range is 0.4 to 0.6.

Given the median seismic capacity $C_{50\%}$, for example, the seismic capacity at 1% and 10% probability of unacceptable performance are given by:

$$C_{1\%} = C_{50\%} e^{-2.326\beta} \quad (4)$$

$$C_{10\%} = C_{50\%}e^{-1.282\beta} \quad (5)$$

where -2.326 and -1.282 are the standardized normal variables associated with 1% and 10% non-exceedance-probabilities, respectively.

Typically, individual SSC fragilities are estimated using the Separation of Variables approach described in Refs. 1, 3, and 4. By this approach, the median seismic capacity $C_{50\%}$ can be estimated from:

$$C_{50\%} = \frac{S_{50\%}}{D_{50\%}} F_{N_{50\%}} SME \quad (6)$$

where $S_{50\%}$, $D_{50\%}$, $F_{N_{50\%}}$ are median estimates of the component seismic strength, seismic demand for a specified seismic input (called herein SME), and inelastic energy absorption (nonlinear) factor, respectively. Next, the composite logarithmic standard deviation β is estimated from:

$$\beta = \left[\beta_S^2 + \beta_D^2 + \beta_N^2\right]^{1/2} \quad (7)$$

where β_S, β_D, and β_N are the composite logarithmic standard deviations associated with strength, demand, and the nonlinear factor. If variabilities are divided into randomness and uncertainty variabilities, then Eqn (7) can be used with the uncertainty β_S, β_D, and β_N to define the overall β_{UNC}. Similarly, Eqn. (7) can be used to find the overall β_{RAND}.

2.1.3 Systems Analysis

In order to estimate any plant damage state (such as core damage) seismic risk, it is necessary to develop plant damage state logic trees (fault trees and event trees). From these trees, a Boolean algebra cutset of components that dominate the plant damage state seismic risk can be developed. A simple Boolean cutset for Damage state DS in terms of components A, B, C, D, E, and F might look like:

$$DS = SP13SP2 \quad (8a)$$
$$SP1 = A4B4C \quad (8b)$$
$$SP2 = D4E4F \quad (8c)$$

Eqn. (8a) states that the Damage State DS occurs only when Success Paths (SP1) "AND" (SP2) both fail. Eqn. (8b) states that Success Path SP1 fails when either Component A"OR" B "OR" C fails. Similarly for Eqn. (8c). Of course, actual plant damage state minimum cutset Boolean expressions are typically much more complex than the simplified expression shown by Eqn. (8). However, this simplified Boolean expression will subsequently be used for illustrative purposes.

2.1.4 Convolve Individual SSC Fragilities to Obtain Plant Damage State Fragility

Once all of the individual SSC fragilities are defined (Step 2), and the plant damage state Boolean algebra cutset is defined (Step 3), the plant damage state fragility can be estimated by the following process. First, the ground motion parameter is divided up into a series of ground motion levels. At any ground motion level *a*, the conditional probability of failure $P_{F(SSC)}$ is determined for each SSC using its estimated fragility curve. These individual component conditional failure probabilities are combined using Boolean algebra probability combination rules to obtain the Damage State failure probability at ground motion level *a*. For example, using the simplified Boolean expression defined by Eqn. (8) at ground motion level *a*, the mean plant damage state conditional failure probability $P_{F(DS)}$ is:

$$P_{F(DS)} = P_{F(SP1)} P_{F(SP2)} \quad (9a)$$

$$P_{F(SP1)} = P_{F(A)} + (1 - P_{F(A)})[P_{F(B)} + (1 - P_{F(B)})P_{F(C)}] \quad (9b)$$

$$P_{F(SP2)} = P_{F(D)} + (1 - P_{F(D)})[P_{F(E)} + (1 - P_{F(E)})P_{F(F)}] \quad (9c)$$

where $P_{F(SP1)}$ and $P_{F(SP2)}$ are the individual success path mean conditional failure probabilities, and $P_{F(A)}$ through $P_{F(F)}$ are the individual SSC mean conditional failure probabilities. Experience has shown that the overall seismic risk is dominated by ground motion levels at which individual SSC conditional failure probabilities are in the range of 1% to 70%. Therefore, it is important to include the *not--failure* probability terms in Eqn. (9). In other words, the small probability assumption which is commonly made in internal event PRA studies should not be made in SPRA studies.

Secondly, to obtain a reasonable estimate of the plant damage state mean fragility curve, the plant damage state mean conditional failure probability $P_{F(DS)}$ should generally be estimated for at least 10 different ground motion levels within the ground motion range of interest. Thus, even for the very simple plant damage state Boolean expression of Eqn. (8), one must solve Eqn. (9) for at least 10 different ground motion levels. The computations quickly become sufficiently numerous that the development of plant damage state mean fragility estimates must be computerized.

If the variabilities in each SSC fragility estimate are separated into randomness β_{RAND} and uncertainty β_{UNC} and the uncertainties are to be propagated through to the plant damage state fragility estimate, then Monte Carlo type simulations of the uncertainties become necessary. To reasonably propagate the uncertainty, at least 1000 trials are necessary at each ground motion level. Thus, the number of computations required quickly escalates.

2.1.5 Convolution of Plant Damage State Fragility Estimate and Seismic Hazard Estimate to Obtain Damage State Seismic Risk

Once the mean plant damage state fragility estimate has been made, then the mean seismic risk P_F can be obtained by numerical convolution of the mean seismic hazard curve and mean fragility curve by either:

$$P_F = -\int_0^{+\infty}\left(\frac{dH_{(a)}}{da}\right)P_{F/a}\,da \qquad (10a)$$

$$P_F = \int_0^{+\infty} H(a)\left(\frac{dP_{F/a}}{da}\right)da \qquad (10b)$$

where $P_{F/a}$ is the conditional probability of failure given the ground motion level *a* which is defined by the mean fragility curve, and $H_{(a)}$ is the mean hazard exceedance frequency corresponding to ground motion level *a*. At the limit of very small differential da, both Eqns. (10a) and (10b) produce identical results. Therefore, the choice of using Eqn. (10a) versus (10b) is a matter of personal preference.

In order to rigorously estimate the uncertainty in the seismic risk estimate, both the uncertainty of the seismic hazard estimate and the uncertainty of the plant damage state fragility estimate must be propagated through Eqn. (10) by Monte Carlo type simulations. Experience dictates that at least 100 trials are necessary.

However, as previously noted, Eqn. (10) only needs to be solved once using the mean hazard curve and the mean fragility curve in order to rigorously define the mean risk. Furthermore, experience has shown that the median risk (50% exceedance probability) or any other point estimate within the range of the 10% to 60% exceedance probability can be closely approximated by using the mean (composite) fragility estimate coupled with the hazard estimate $H_{(a)}$ corresponding to the exceedance frequency of interest. This approximation is substantially more precise than the accuracy with which either the seismic hazard or seismic risk estimate is defined. Therefore, unless there is a need to estimate the 0% to 10% or 60% to 100% exceedance probability tails of the seismic risk estimate, there is no need to separate the fragility variability β into its uncertainty β_{UNC} and randomness β_{RAND} components. Furthermore, since only a point estimate plant damage state fragility is needed, there is no need for Monte Carlo simulations to propagate individual SSC fragility uncertainties to plant damage state fragility uncertainties or to propagate plant damage state fragility uncertainties to seismic risk uncertainties. This simplification typically reduces the computational effort by more than a factor of 10,000.

2.2 Methodological Observations From Past SPRA Studies

2.2.1 Observations Concerning Accuracy of Seismic Risk Estimates

At ground motion levels corresponding to about the 10^{-5} annual frequency of exceedance, Ref. 11 shows that the 15% to 85% non-exceedance probability (NEP) range on this annual exceedance frequency is about two orders of magnitude wide. In other words, the 15% to 85% NEP range on the hazard exceedance frequency is about 10^{-6} to 10^{-4} or a factor of 100 wide at least for the Central and Eastern U.S. At lower hazard exceedance frequencies, this range is even wider. Therefore, even if no uncertainty exists in the fragility estimate, the 15% to 85% NEP range on the estimated seismic risk would be a factor of 100 wide. A typical β_{UNC} estimate of about 0.3 or less on the plant damage state fragility estimate contributes less than about a factor 4 range on the 15% to 85% NEP seismic risk for a typical EUS hazard curve with an A_R ratio of about 2.5. Therefore, the seismic risk uncertainty is nearly totally dominated by the seismic hazard uncertainty. Thus, over the central region of the seismic risk estimate from about the 40% to 90% NEP range, it is unnecessary to consider the fragility estimate uncertainty β_{UNC}, but to simply define the fragility by its mean (composite) β variability.

Prior to about 1990, most high quality SPRA studies separated the total fragility variability β into its randomness β_{RAND} and uncertainty components β_{UNC}. In fact, at one stage it was considered absolutely essential for a high quality SPRA to make such a separation and to rigorously propagate the individual uncertainty estimates in order to estimate the uncertainty range on the estimated seismic risk. Unfortunately the division of the overall fragility β into β_{UNC} and β_{RAND} has always been partially subjective and a source of controversy between practitioners. Furthermore, the rigorous propagation of this uncertainty from the individual SSC fragility estimates to the plant damage fragility estimate and then to the seismic risk estimate resulted in an extremely substantial increase in the numerical computations involved.

However, based on the above observation gleaned from these SPRA results, none of this increased complexity and controversy associated with subdividing the fragility variability β into β_{UNC} and β_{RAND} is either necessary or even desirable. The practice should be abandoned. Point estimates of the seismic risk such as the mean risk, median risk, and 85% NEP risk can be made working only with the mean (composite) SSC fragility estimates defined by $C_{50\%}$ and β with a precision substantially better than the accuracy of the underlying data. The added complexity of separating the fragility variability β estimates into β_{UNC} and β_{RAND} and then rigorously propagating this uncertainty through to seismic risk uncertainty can lead to the mistaken impression that the complex process produces an accuracy in estimated seismic risk that simply doesn't exist.

Furthermore, the fact that the median and 85% NEP seismic risk, typically differ by about a factor of 10 should be sufficient to demonstrate that none of the point estimates (mean, median, or 85% NEP) are highly accurate. In my judgement, even the highest quality SPRA cannot produce point estimate seismic risk estimates accurate to within better than a factor of 5 because of inaccuracies of the underlying data.

2.2.2 Observations Concerning Apparent Lack of Sensitivity of Reported Seismic Risk to Significant Seismic Design Changes

Certainly, for a deterministic engineer, changing the seismic capacity of a structure, system or component (SSC) by a factor of 1.5 represents a substantial change in the seismic vulnerability of that SSC. However, based on the typical slope of the seismic hazard curves shown in Figure 2 a factor F_S change in the seismic capacity will only result in a factor R_P change in the resulting seismic risk where based on the approximation of Eqn. (1):

$$R_P = (F_S)^{-K_H} \quad (11)$$

For F_S = 1.5 and the typical EUS and UK hazard curves shown in Figure 2:

A_R	K_H	$(R_P)^{-1}$
2.0	3.32	3.8
4.0	1.66	2.0

Thus, a substantial change in the seismic capacity of a factor of 1.5 typically results in a change in seismic risk of only a factor of 2 to 4 because of the shallow slope on typical seismic hazard curves.

Considering my closing comment of the previous section that the seismic risk cannot be estimated to within an accuracy of better than a factor of 5, a change in seismic risk of only a factor of 2 to 4 by a substantial design change might be considered by some to be inconsequential. The problem is that one must be careful not to confuse the inaccuracy of absolute seismic risk estimates with the importance of changes in the relative seismic risk. Even though the absolute seismic risk cannot be estimated within a factor of 5, a change in the relative seismic risk of a factor of 2 to 4 represents a significant seismic design change that should not be ignored.

My conclusion is that fairly small differences (less than a factor of 5) in absolute seismic risk estimates should be de-emphasized whereas changes in the relative seismic risk as small as a factor of 2 to 4 are important and should be emphasized. Unfortunately, in order to be comparable, two relative seismic risk estimates must both be computed by the exact same prescriptive approach to estimating the seismic hazard and SSC fragilities. Otherwise, the comparisons of small relative differences become meaningless. This dichotomy between the level of accuracy of seismic risk estimates and the importance to design of relatively small changes in the estimated seismic risk remains a continuing problem in the use of SPRA results for making design or other safety related decisions. In the mid-1980's to 1990 time frame as a result of this dichotomy considerably increased emphasis was placed on defining the Seismic Margin of a plant as opposed to defining the Seismic Risk.

3Seismic Margin Method

3.1 Definition of Seismic Margin

Nuclear power plants have been designed for a Design Basis Earthquake (DBE) called the Safe Shutdown Earthquake (SSE) in the U.S. The design criteria is conservatively established. Therefore, these plants have additional seismic margin to withstand earthquake ground motion larger than their DBE ground motion. Ref. 7 suggested that one way to consistently describe this margin over a wide range of plants and their SSC's was to define this seismic margin in terms of the High-Confidence-Low-Probability-of-Failure (HCLPF) Capacity of each critical SSC and Plant Damage State. It was recommended that this HCLPF capacity should represent approximately the 95% confidence of less than about 5% probability of failure point on the fragility curve (see Figure 3). Thus:

$$C_{HCLPF} \approx C_{50\%} e^{f_H} \quad (12)$$

$$f_H = -1.645(\beta_{UNC} + \beta_{RAND}) \quad (12a)$$

where C_{HCLPF} is the HCLPF seismic capacity and -1.645 is the standardized normal variable associated with 5% NEP. This recommendation was adopted by an *Expert Panel on the Quantification of Seismic Margins* (Ref. 5) and the U.S. Nuclear Regulatory Commission
(Ref. 6).

However, as noted in Section 2.2.1, it was decided in about 1990 to no longer recommend that the fragility variability β be subdivided into β_{UNC} and β_{RAND}. Based on Eqn. (3), so long as $0.5 \leq (\beta_{UNC}/\beta_{RAND}) \leq 2.0$:

$$f_H \approx -2.326\,(\beta) \quad (12b)$$

It is unlikely that $(\beta_{UNC}/\beta_{RAND})$ falls outside of the above range. If $(\beta_{UNC}/\beta_{RAND})$ does fall outside of this range, then Eqn. (12b) becomes an increasingly conservative approximation of Eqn. (12a).

Based on Eqn. (12b), an alternate definition of the HCLPF capacity is that the HCLPF capacity corresponds to approximately the 1% probability of failure point on the mean (composite) fragility curve. This alternate definition of the HCLPF capacity was adopted by the U.S. Nuclear Regulatory Commission (Ref. 6) when β is not subdivided into β_{UNC} and β_{RAND}. Thus:

$$C_{HCLPF} \approx C_{1\%} \quad (13)$$

where $C_{1\%}$ is the 1% probability of unacceptable performance capacity as defined by Eqn. (4).

3.2 Methods for Computing HCLPF Capacity of SSC

Ref. 5 recommends two methods for computing the HCLPF capacity of a SSC. These methods are the Fragility Method, and the Conservative Deterministic Failure Margin (CDFM) Method.

In the Fragility Method, the fragility curve is computed as described in Section 2.1.2. Next, the $C_{1\%}$ is computed from Eqn. (4) and the HCLPF capacity C_{HCLPF} is approximated by Eqn. (13).

Ref. 7 recommended the CDFM Method as an approximate method for estimating the $C_{1\%}$ point on the fragility curve and thus C_{HCLPF}. This method is deterministic, but has been extensively benchmarked against the Fragility Method. This method is most extensively described in Ref. 10. A summary of the CDFM Method together with a description of the conservatism introduced into the method is given in Appendix A. This appendix demonstrates that:

$$C_{1\%} \approx C_{CDFM} \quad (14)$$

where C_{CDFM} is the CDFM Method computed capacity. Combining Eqns. (13) and (14):

$$C_{1\%} \approx C_{CDFM} \approx C_{HCLPF} \quad (15)$$

Thus, the CDFM capacity, HCLPF capacity, and the 1% probability of unacceptable performance capacity are essentially interchangeable terms for the same Seismic Margin Capacity and are often used interchangeably in the literature.

Within my experience most highly experienced deterministic design engineers are not comfortable with estimating the seismic fragility curve for a SSC. Their training has not been directed toward estimating median seismic capacities $C_{50\%}$ or variabilities β. These engineers are much more confident in estimating a High-Confidence-Low Probability-of-Failure (HCLPF) capacity C_{HCLPF} by the CDFM Method rather than estimating the fragility parameters $C_{50\%}$ and β, and then computing C_{HCLPF} from Eqn. (4). Furthermore, the CDFM Method requires significantly less effort than does the Fragility Method. Therefore, in the majority of seismic margin reviews the CDFM Method has been used to estimate the HCLPF seismic margin capacity of individual SSCs.

In my own practice, for very critical SSCs I estimate the HCLPF capacity by both the Fragility Method using Ref. 4 and by the CDFM Method using Ref. 10. If both methods produce C_{HCLPF} estimates that agree within 20%, I tend to consider the Fragility Method result to be the more accurate. Nearly always such agreement occurs. When the results from the two methods differ by more than 20%, I search for the cause of the difference and I generally discover that I have made some error in the Fragility Method. The CDFM Method is much less error prone and is therefore more reliable, but somewhat less accurate. For less critical SSCs, I exclusively use the CDFM Method.

3.3 Plant Damage State HCLPF Capacity

In a seismic margin review, the plant damage state Boolean cutsets can be developed using either the plant damage state logic trees (fault trees and event trees) mentioned in Section 2.1.3 or using the Success Path approach described in Ref. 10. The Success Path approach results in simple Boolean cutsets of the type shown in Eqn. (8). The Success Path approach is simpler to use, potentially more error prone, and sometimes introduces excessive conservatism by losing useful information on the benefits provided by alternate redundant paths embedded within the overall Success Path.

Once the plant damage state Boolean cutset is established, the HCLPF Max/Min Method is used to estimate the HCLPF capacity of a Damage State given the HCLPF capacities of every component in the Boolean cutset describing that Damage State. This method consists of the following two rules:

1. The HCLPF capacity of a combination of components combined by "OR" gates is equal to the

minimum HCLPF capacity of the components being combined.

2. The HCLPF capacity of a combination of components combined by "AND" gates is equal to the maximum HCLPF capacity of the components being combined.

With these two simple rules, the Damage State HCLPF capacity can be immediately estimated from the individual component HCLPF capacities and the Damage State Boolean cutset.

Component lognormal fragility curves should be truncated at some lower bound since the lognormal tail extends to zero. Ref. 5 suggests that the HCLPF capacity represents a reasonable lower bound on the seismic capacity. Rule #1 for "OR" combinations is rigorous when individual fragility curves are truncated at their HCLPF capacity. It is slightly unconservative when individual fragility curves are not truncated at their HCLPF capacity. This unconservatism is negligible except when more than two components have essentially the same minimum HCLPF capacities. From experience, the unconservatism is never more than 20% which is probably about the accuracy with which the HCLPF capacities can be estimated in the first place.

Rule #2 for "AND" combinations is always conservative. This conservatism can become substantial when two or more components have essentially the same maximum capacity. As a result, Damage State seismic risk will be overestimated when the Damage State Boolean cutset is dominated by components combined by "AND" gates.

3.3.1 Example Application of HCLPF Maximum Method

For this example application, it is assumed that Eqn. (8) defines the Damage State Boolean cutset. Furthermore, the individual HCLPF capacities and variability are assumed to be:

Component A, B, C

$$C_{HCLPF} = 0.35g$$
$$\beta = 0.4 \tag{16}$$

Component D, E, F

$$C_{HCLPF} = 0.50g$$
$$\beta = 0.4$$

Table 1 shows the C_{HCLPF} capacities obtained for Success Paths SP1, and SP2, and Damage State DS using the HCLPF Max/Min Method. This table also shows the HCLPF capacities obtained by the rigorous convolution of the component fragility curves performed as described in Section 2.1.4. Each fragility curve is assumed to have the Eqn. (16) HCLPF capacities and $\beta = 0.4$ with no truncation of the tails of the fragility curves.

This example shows that about 14% unconservatism is introduced by the HCLPF Max/Min Method when three identical component fragilities without truncation are combined in an *OR* combination. It also illustrates the conservatism introduced when two components are combined in an *AND* combination, For more complex Damage State Boolean cutsets, the HCLPF Max/Min Method generally introduces a conservative bias to the computed Damage State HCLPF capacities, as is the case in this simple example.

3.4 Use of Screening Tables To Screen Out SSCs From HCLPF Capacity Computations

Ref. 10 provides screening tables which can be used as part of a seismic walkdown to screen out many SSC from having to have HCLPF capacity calculations performed. The Ref. 10 screening tables are set at 0.8g, 5% damped peak spectral acceleration (approximately 0.33g PGA) and 1.2g 5% damped peak spectral acceleration (approximately 0.5g PGA). The advantage of screening out components from further review is that a great amount of unnecessary HCLPF capacity computations are eliminated for components whose HCLPF capacities clearly exceed the screening level. However, if the screening level is set too low and all computed HCLPF capacities for the non-screened out components either nearly equal or exceed the screening level, then no weaker link components which govern the seismic risk will be determined from the Seismic Margin Review. Therefore, the screening level needs to be set sufficiently high that it is unnecessary to determine which components are the weaker link components when all components have HCLPF capacities which exceed this screening level.

In recent years, the Ref. 10 screening tables have also often been used in SPRA reviews. When screening tables are used in a SPRA review, a *surrogate* element must be added to the Damage State Boolean cutsets to replace all of the components which have been screened out from review. Based on Ref. 4, this *surrogate* element should have the following median capacity $C_{50\%}$ and variability β:

<u>*Surrogate* Element</u>

$$C_{50\%} = 2(SL)$$

$$\beta = 0.3 \qquad (17)$$

where SL is the screening level. This *surrogate* element produces a HCLPF capacity $C_{HCLPF} = C_{1\%} = SL$ as can be seen from Eqn. (4). If no *surrogate* element is added, then the Damage State seismic risk coming from the screened out components will have been ignored.

The advantages of using screening tables in a SPRA is that computing fragilities for a large number of uninteresting components can be eliminated. The disadvantage is that the replacement *surrogate* element may be a significant contribution to the computed seismic risk which can mask the actual risk contributions. This disadvantage can be overcome by following the below steps when selecting a screening level:

1. Establish a permissible level of seismic risk P_{FS} which can be contributed by the *surrogate* element.

2. Enter the seismic hazard curve at an exceedance frequency $H_{S(a)}$ given by:

$$H_{S(a)} = 2P_{FS} \qquad (18)$$

and determine the corresponding ground motion level a_s

3. Set the screening level SL at:

$$SL \geq 0.8\, a_s \qquad (19)$$

This approach will assure that the *surrogate* element will not contribute a seismic risk greater than P_{FS}. The basis for this approach is presented in Appendix B.

Table 2 presents a representation seismic hazard curve for a low seismic EUS or UK site. For this hazard curve, if the permissible *surrogate* element seismic risk was set at 0.5 x 10^{-5}, then $H_{S(a)} = 1\text{x}10^{-5}$, $a_s = 0.66$g and SL $\geq$ 0.53g. Thus, a screening level of 0.8g is more than sufficient. However, if the permissible *surrogate* element risk was only 0.5x10^{-6}, then by the same approach SL $\geq$ 1.0g and a screening level of 0.8g would not be sufficient.

4. Issues Addressed by SPRA and Seismic Margin Reviews

The SPRA Method addresses each of the following questions:

1. What is the seismic risk to the plant?
2. What range of ground motion levels dominate the seismic risk?
3. What plant components are dominant contributors to seismic risk?
4. What is the median (or mean) seismic capability of the plant?
5. What is the ground motion level at which a HCLPF of seismic-induced core damage exists?
6. Are there any *weaker link* components which reduce the HCLPF capacity of the plant below some desired SME level? If so, what are these components?

Because of large uncertainties in seismic hazard estimates of the annual probability of exceedance of ground motion levels which dominate the seismic risk (i.e., ground motion levels well beyond the SSE level), Question 1 can only be addressed with considerable uncertainty. Questions 2 and 3 are only mildly dependent on the seismic hazard estimate and, as such, avoid the concerns about the large uncertainty in the seismic hazard estimate. Questions 4 through 6 are independent of the seismic hazard.

In order to address Questions 2 through 4, a full expression of the seismic capability of every important component is needed in terms of conditional probability of failure versus ground motion level, or, in other words, a fragility curve. Such fragility curves also contain uncertainty and controversy because of a limited data base. Thus, Questions 2 through 4 can also only be addressed with uncertainty, although less than for Question 1.

In many cases, Questions 5 and 6 are of the greatest interest. These questions only require the HCLPF capacity of seismic safety significant SSCs to be determined. Thus, these questions can be addressed equally well by a Seismic Margin review. However, a Seismic Margin review cannot directly address Questions 1 through 4.

5. Hybrid Method

5.1 Introductory Comments on Hybrid Method

A need existed to establish an intermediate method midway between the SPRA Method and the Seismic Margin Method. This intermediate method (called herein Hybrid Method) has to be capable of addressing all 6 questions listed in the previous section with only a small and tolerable loss of precision relative to the SPRA Method. In addition this Hybrid Method needed to retain the fundamental simplicity of the Seismic Margin Method in only requiring HCLPF capacities to be computed by the CDFM method as opposed to the SPRA Method of developing seismic fragilities. Thus the advantage of the SPRA Method of addressing all 6 questions, and the simplification of the Seismic Margin Method are both retained.

5.2 Hybrid Method Steps

The following steps of the SPRA Method are retained:

- ☐ Seismic Hazard Estimate (Section 2.1.1)
- ☐ Systems Analysis (Section 2.1.3)
- ☐ Convolve Individual SSC Fragilities to Obtain Plant Damage State Fragility (Section 2.1.4)
- ☐ Convolution of Plant Damage State Fragility Estimate and Seismic Hazard Estimate to

Obtain Damage State Seismic Risk (Section 2.1.5)

However, the Seismic Fragility Estimate Step (Section 2.1.2) is revised and simplified as follows.

First, estimate the HCLPF Capacity by the CDFM Method. Next, approximately estimate the fragility logarithmic standard deviation β by judgement and the following guidance. For structures and major passive mechanical components mounted on the ground or at low elevations within structures, β typically ranges from 0.3 to 0.5. For active components mounted at high elevations in structures the typical β range is 0.4 to 0.6. When in doubt, use $\beta = 0.4$. Lastly, note that $C_{1\%} \approx C_{CDFM}$ and use Eqn. (4) to define the median capacity $C_{50\%}$. Table 3 presents the ratio $C_{50\%}/C_{CDFM}$ for typical β values. Note that overestimating β is unconservative because it increases $C_{50\%}$.

5.3 Basis for Hybrid Method

The Hybrid Method is based on the observation that the annual probability of unacceptable performance P_F for any SSC is relatively insensitive to β. This annual probability (seismic risk) can be computed with adequate precision from the CDFM Capacity C_{CDFM} and a crude estimate of β. This point is illustrated in Table 4 using the representative seismic hazard estimate given in Table 2 for two different CDFM capacities. Table 4 was developed by numerical integration of Eqn. (10a). Over the range of β from 0.3 to 0.6, the computed seismic risk differs by a factor of approximately 2.6. The computed seismic risk at $\beta = 0.3$ is approximately 1.5 times that at $\beta = 0.4$, while at $\beta = 0.6$ the computed seismic risk is approximately 60% of that at $\beta = 0.4$. A very crude estimate of β is sufficient to estimate the seismic risk P_F within a factor of 1.6. There is no need to obtain an improved estimate of β.

6. Simplified Hybrid Method

6.1 Introductory Comments on Simplified Hybrid Method

Even though the Hybrid Method represents a considerable simplification of the SPRA Method, it still requires substantial numerical calculations in order to convolve individual SSC fragilities to obtain plant damage state fragilities (Section 2.1.4) and to convolve plant damage state fragilities with the seismic hazard estimate to obtain the estimated damage state seismic risk (Section 2.1.5). Therefore, access to a computer program and computer to perform these steps is required. The Simplified Hybrid Method was developed to avoid the need for these numerical calculations and to enable the Damage State seismic risk to be reasonably estimated almost instantly by simple manual analysis given:

- Seismic hazard estimate (Section 2.1.1)
- CDFM based HCLPF capacity for individual SSC (Section 3.2 and Appendix A)
- Damage State Boolean cutsets (Section 2.1.3)

This method is intended for quick estimation of the seismic risk significance of design changes or modifications to existing plants and to provide a sanity check on seismic risk results obtained by either the SPRA or Hybrid Methods. The basis for this method is given in Appendix B.

6.2 Estimation of Seismic Risk P_F for an Individual SSC By Simplified Hybrid Method

Step 1: Determine the component HCLPF capacity by the CDFM Method and estimate β as per the Hybrid Method (Section 5.2)

Step 2: Estimate the 10% conditional probability of failure capacity $C_{10\%}$ from:

$$C_{10\%} = F_\beta C_{HCLPF}$$
$$F_\beta = e^{1.044\beta} \quad (20)$$

where 1.044 is the difference between the 10% NEP standard normal variable (-1.282) and the 1% NEP standardized normal variable (-2.326). F_β is tabulated in Table 3 for various β values.

Step 3: Determine hazard exceedance frequency $H_{10\%}$ that corresponds to $C_{10\%}$ from hazard curve.

Step 4: Determine seismic risk P_F from:

$$P_F = 0.5\, H_{10\%} \quad (21)$$

6.2.1 Example Application of Simplified Hybrid Method for Individual SSC

The following example computes the seismic risk P_F for a SSC with a CDFM based HCLPF capacity C_{HCLPF} of 0.35g and the seismic hazard estimates tabulated in Table 2.

Step 1: Estimate $\beta = 0.40$

Step 2: Estimate $C_{10\%}$

$C_{10\%} = 1.52\ (0.35) = 0.532g$

Step 3: Determine $H_{10\%}$ from hazard estimate:

$H_{10\%} \approx 2.04 \times 10^{-5}$

Step 4: Estimate seismic risk P_R from Eqn. (21):

$P_F \approx 1.02 \times 10^{-5}$

Table 4 presents in parentheses the seismic risk computed by the Simplified Hybrid Method for all 8 cases considered. Table 4 shows that for the hazard curve considered, the Simplified Hybrid Method introduces a 0% to 25% conservative bias. The cause of this conservative bias is discussed in Appendix B. This bias is of about the same size as the uncertainty in seismic risk resulting from even a moderate level of uncertainty in β, and is very small in comparison to the uncertainty in estimating the mean seismic hazard exceedance frequency.

6.3 Estimation of Damage State Seismic Risk P_F By Simplified Hybrid Method

Step 1: Combine the individual SSC HCLPF capacities by the HCLPF Max/Min Method (Section 3.3) to estimate the Damage State HCLPF capacities

Step 2: Estimate the Damage State variability β. Because of the convolution described in Section 2.1.4, the Damage State fragility curve has a lower β than the individual component fragility curves, it is recommended that $\beta = 0.3$ be used for the Damage State variability.

Step 3: Estimate the Damage State seismic risk P_F by Steps 2 through 4 of the previous Section 6.2.

For an example application, the simple Damage State Boolean cutset given in Eqn. (8) will be used. The individual component fragilities are defined in Eqn. (16). The seismic hazard estimate given in Table 2 will be used. First, the rigorous SPRA Method described in Sections 2.1.4 and 2.1.5 is applied to convolve the individual fragility estimates to Damage State fragility estimates and then to convolve these Damage State fragility estimates with the seismic hazard estimate to obtain the Damage State seismic risk. Table 5 presents the seismic risk P_F computed for Success Paths SP1 and SP2, and Damage State DS by a rigorous application of the SPRA Method.

Table 5 also presents the seismic risks P_F computed for these same cases by the Simplified Hybrid Method described above. The HCLPF capacities computed in Step 1 by the HCLPF Max/Min Method are shown in Table 1. These HCLPF capacities together with $\beta = 0.3$ are used with Steps 2 through 4 of Section 6.2 to compute the seismic risks.

The Simplified Hybrid Method underestimates the seismic risk for Success Paths 1 and 2 by about 20% to 25%. This underestimation is primarily due to the HCLPF Max/Min Method overestimating the HCLPF capacity for these *OR* combination cases (see Table 1). The Success Path SP1 and SP2 seismic risk estimates represent a very severe test for the HCLPF Max/Min method because all three components have identical SA_{CDFM} capacities. From actual SPRA experience it is unlikely that as many as three components in an *OR* combination will all have the same minimum capacity.

The Simplified Hybrid Method estimates the same seismic risk for Success Path SP2 and Damage State DS because the HCLPF Max/Min Method produces the same HCLPF capacity for these two cases. Although not shown, for Damage State Boolean cutsets dominated by *AND* combinations, the Simplified Hybrid Method can overestimate the seismic risk by as much as a factor of two. Within my experience this very simple procedure provides a sufficiently precise estimation of the seismic risk for most applications.

6.4 Treatment of Non-Seismic Failures and Human Errors In Cutsets

For simplicity, non-seismic failures and human errors (called herein *random failures)* were not included in the Damage State DS and Success Paths SP1 and SP2 Boolean cutset Eqn. (8). Both the SPRA Method and the Rigorous Hybrid Method automatically include the effects of these random failures when they are included in the Boolean cutset equation because both methods perform the rigorous numerical convolution described in Section 2.1.4 to determine the Damage State fragility which then includes the effect of these random failures.

However, the Simplified Hybrid Method does not directly consider the effect of random failures on seismic risk because it uses the HCLPF Max/Min Method to approximate the Damage State fragility. Even so. the effect of random failures can be approximated as described for the following example in which the Damage State Boolean cutset of Eqn. (8) is revised to:

DS = (SP14R1)3(SP24R2)3R3(22)

where R1, R2, and R3 are random failures with failure probabilities $P_{F(R1)}$, $P_{F(R2)}$, $P_{F(R3)}$, respectively.

First, reduce the HCLPF capacity of any Success Path with large random failure probabilities combined in an *OR* combination using the following empirical guidance:

$P_{F(R)}$	HCLPF Reduction Factor
≤ 2%	No Reduction
5%	0.9
10%	0.8
30%	0.7

Next, use the HCLPF Max/Min Method with these revised (if necessary) success path HCLPF capacities to define the Damage State seismic HCLPF ignoring the *AND* combined random failure R3. Using this Damage State seismic HCLPF, compute the seismic risk P_{FS} by Eqns. (20) and (21) of Section 6.2. Lastly, incorporate the *AND* combined random failure R3 by:

$$P_F = P_{FS} \& P_{F(R3)} \quad (23)$$

7. Summary and Conclusions

The necessary steps in the Seismic Probabilistic Risk Assessment (SPRA) Method to estimate Damage State seismic risk are summarized in Section 2.1. Some important observations concerning the SPRA Method are presented in Sections 2.2.

Section 3 summarizes the simpler Seismic Margin Method. The concept of defining seismic margin in terms of the High Confidence Low Probability at Failure (HCLPF) capacity is discussed in Section 3.1. Some problem areas associated with the use of Screening Limits to reduce the number of HCLPF or fragility computations required in either Seismic Margin or SPRA reviews are discussed in Section 3.4 and an approach to avoid these problems is presented.

Section 4 summarizes and compares the seismic issues which can be addressed by a SPRA review versus a Seismic Margin review. The Seismic Margin review addresses some of the most important issues, but not all of the issues addressed by a SPRA review.

Section 5 presents a Hybrid Method which addresses all of the issues, addressed by a SPRA review while retaining much of the simplicity of the Seismic Margin Method. It is recommended that this Hybrid Method should be used in lieu of either the SPRA Method or the Seismic Margin Method for most future seismic risk studies. The simplifications gained over the SPRA Method are worth the slight loss in precision.

Lastly, Section 6 presents an even much simpler Simplified Hybrid Method for estimating Damage State seismic risk. This method is intended for quick estimation of the seismic risk significance of design changes or modifications to existing plants and to provide a sanity check on seismic risk results obtained by either the SPRA or Hybrid Methods. This very simple procedure provides a sufficiently precise estimation of the seismic risk for most applications. Given a seismic hazard estimate, with this Simplified Hybrid Method, all Seismic Margin study HCLPF results can be essentially instantly converted into Damage State seismic risk estimates. Therefore, for any plant for which a high quality Seismic Margin review has been performed, Damage State seismic risks can easily be estimated using this Simplified Method.

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Table 1 Success Path and Damage State
HCLPF Capacities for Example Problem

HCLPF Capacit	es $C_{HCLPF}(g)$	
Failure Mode	Max/Min Method	Rigorous Convolution
SP1	0.35	0.30
SP2	0.50	0.43
DS	0.50	0.52

Table 2 Representative Mean Seismic Hazard Estimate

Hazard Exceedance FrequSAcy H(g)	Spectral Acceleration

$1x10^{-2}$	0.039
$3x10^{-3}$	0.072
$1x10^{-3}$	0.117
$3x10^{-4}$	0.200
$1x10^{-4}$	0.302
$3x10^{-5}$	0.473
$1x10^{-5}$	0.661
$3x10^{-6}$	0.953
$1x10^{-6}$	1.254
$3x10^{-7}$	1.695
$1x10^{-7}$	2.161

Table 3 Hybrid and Simplified Hybrid Method Parameters

β	Median/CDFM Capacity ($C_{50\%}/C_{CDFM}$)	β ($C_{10\%}/C_{HCLPF}$)
0.3	2.01	1.37
0.4	2.54	1.52
0.5	3.20	1.69
0.6	4.04	1.87

Table 4 Component Seismic Risk Results
(all risk results are to be multiplied by 10^{-6})

CDFM Capacity C_{CDFM}	Seismic Risk P_F *			
	$\beta = 0.3$	$\beta = 0.4$	$\beta = 0.5$	$\beta = 0.6$
0.35	12.7 (14.3)	8.4 (10.2)	6.1 (7.2)	4.8 (5.2)
0.50	4.0 (4.4)	2.6 (3.2)	1.9 (2.2)	1.6 (1.6)

* Results shown in parentheses ( ) are by the Simplified Hybrid Method of Section 6.2

Table 5: Seismic Risk P_F Computed for Success Paths SP1 and SP2, and Damage State DS
(all risk results are to be multiplied by 10^{-6})

Failure Mode	SPRA Method	Simplified Hybrid Method
SP1	17.8	14.3
SP2	5.8	4.4
DS	4.3	4.4

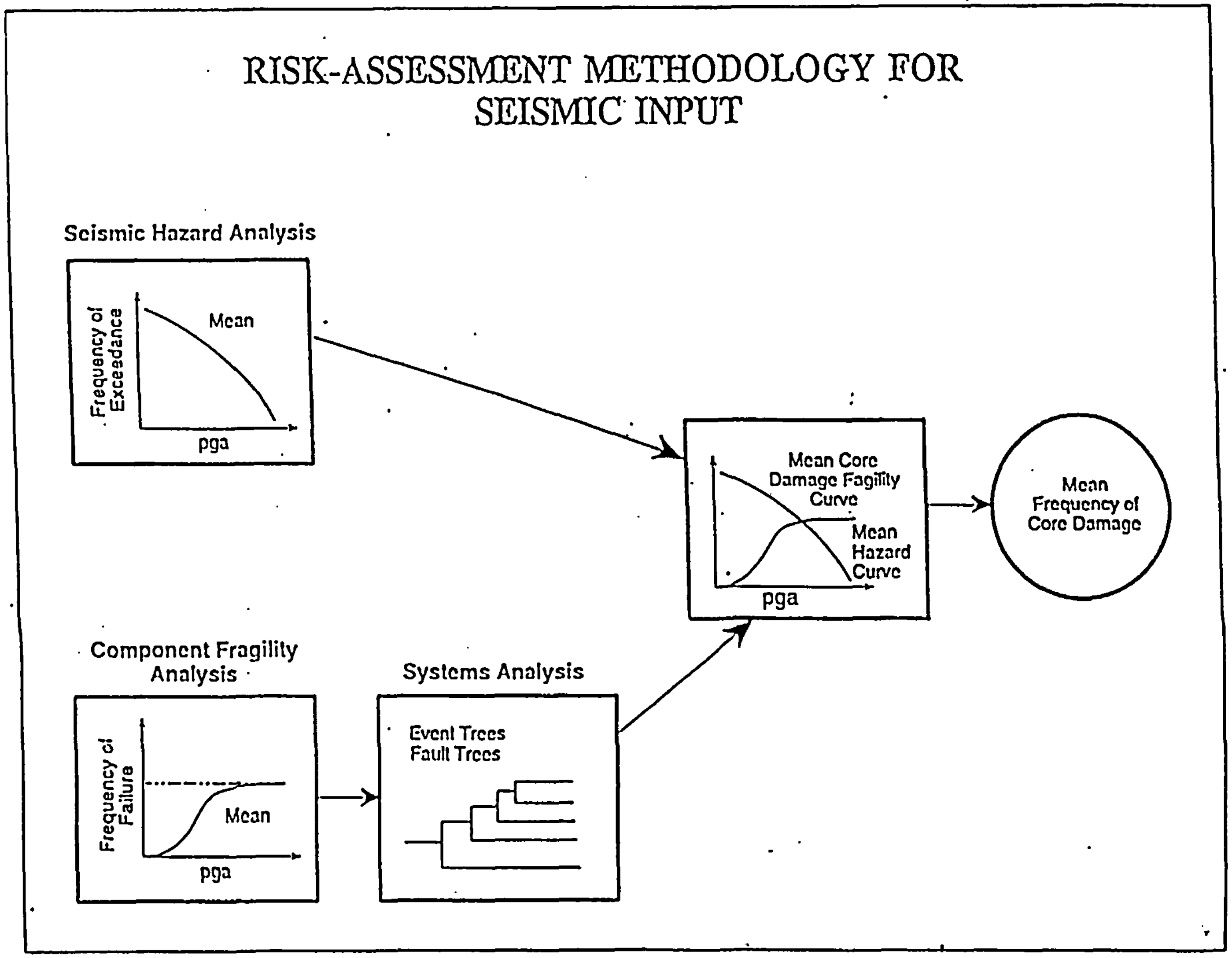
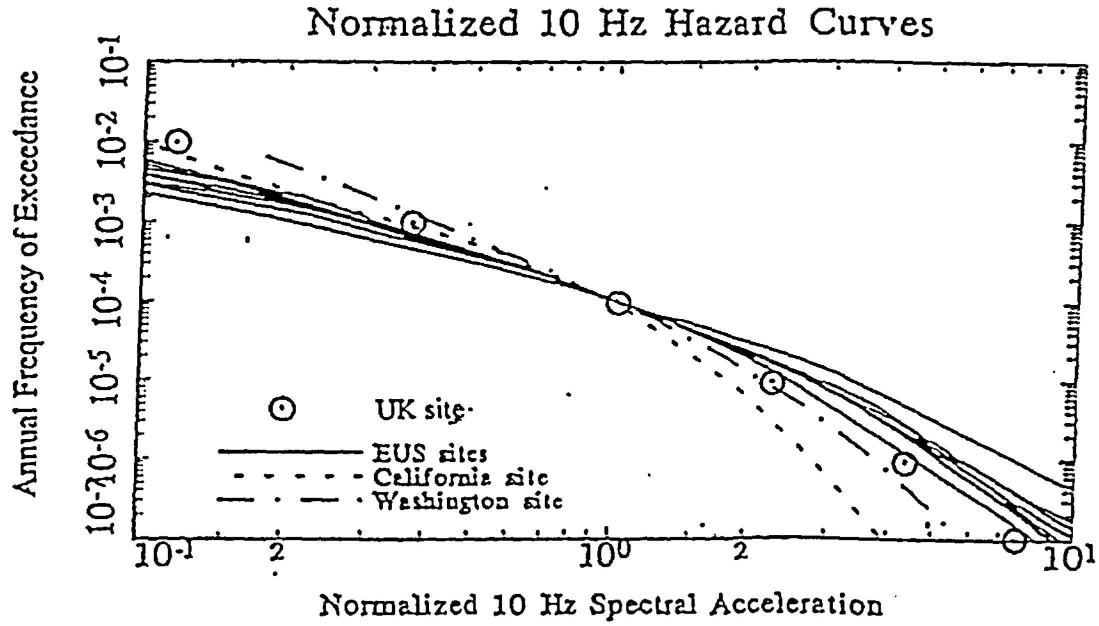


Figure 1: Critical Steps of a Seismic Probabilistic Risk Assessment



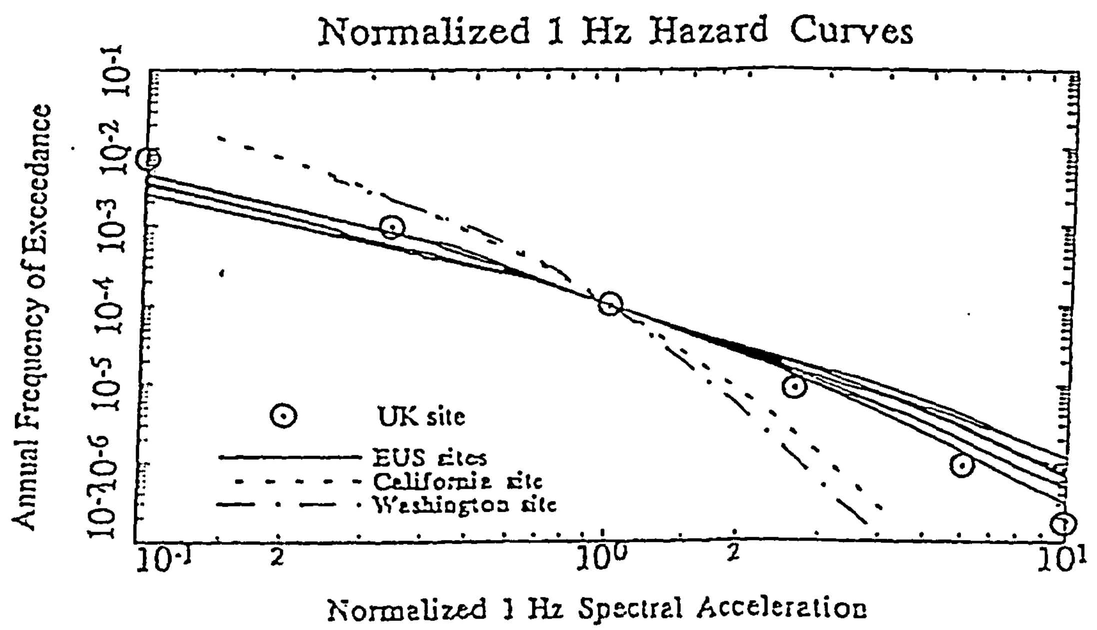
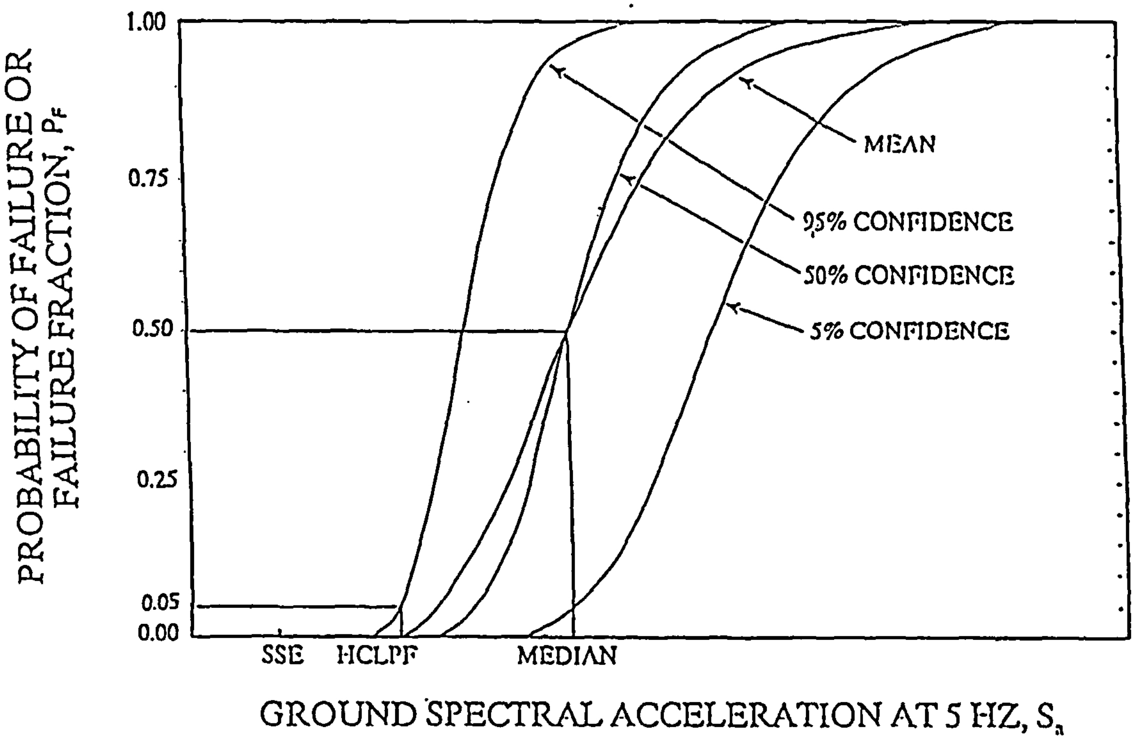


Figure 2: Seismic Hazard Curves Normalized By the Spectral Acceleration Value Corresponding to a 10^{-4} Annual Probability

Figure 3: ILLUSTRATION OF FRAGILITY CURVES



Appendix A
Summary of Conservatism Introduced
Into the CDFM Method for Computing
Seismic Capacity

A.1 General Philosophy of CDFM Method

As noted in Section 3.2, the CDFM Method is a deterministic method for estimating seismic capacity and is aimed at achieving a seismic capacity corresponding to about the 1% non-exceedance probability (NEP). The general criteria (from Ref. 7 through 10) for this approach are outlined in Table A.1 and are briefly summarized below.

Essentially, the approach intends to achieve the following:

1. For the specified seismic margin earthquake ground motion level SME, the elastic computed response (SME Demand) of structures and components mounted thereon should be defined at the 84% nonexcedance probability (NEP).

2. Strengths for most components should be defined at about the 98% exceedance probability so that even if the SME demand slightly exceeds this CDFM strength by more than a permissible conservatively specified inelastic energy absorption capability, there will result a very low probability of failure. However, for the CDFM strength of very brittle failure modes (weld failure, relay chatter, etc.) which have no inelastic energy absorption capability, so that this capability cannot be conservatively underestimated, the conservatism at which the strength is defined should be increased to about the 99% exceedance probability.

3. Inelastic distortion associated with a Demand/Strength ratio greater than unity is permissible. The permissible level of inelastic distortion should be specified at about the 5% failure probability level. The inelastic energy absorption capability, F_N should be slightly conservatively estimated at about the 84% NEP for this permissible level of inelastic distortion.

4. Finally,

$$\text{Seismic Demand/Strength} \leq F_N \quad \text{(A.1)}$$

where F_N is the inelastic energy absorption factor.

Because of the conservatism introduced at the various steps, the result is a high-confidence-of-a-low-probability-of-failure (HCLPF) when Eqn. (A.1) is satisfied. Any seismic evaluation which introduces approximately the level of conservatism as defined in the above four steps meets the intent of the CDFM approach and would be expected to achieve a HCLPF.

A.2 Estimation of the Conservatism Introduced By the CDFM Method as Generally Applied

A.2.1 Basic Approach

The median seismic capacity $C_{50\%}$ can be estimated from:

$$C_{50\%} = \frac{S_{50\%}}{D_{50\%}} F_{N_{50\%}} SME \quad \text{(A.2)}$$

where $S_{50\%}$, $D_{50\%}$, $F_{N_{50\%}}$ are median estimates of the component seismic strength, seismic demand for a specified SME input, and inelastic energy absorption (nonlinear) factor, respectively. In turn, the CDFM seismic capacity C_{CDFM} is given by:

$$C_{CDFM} = \frac{S_{CDFM}}{D_{CDFM}} F_{N_{CDFM}} SME \quad (A.3)$$

where S_{CDFM} D_{CDFM}, and $F_{N_{CDFM}}$ are the deterministic strength, demand, and nonlinear factors defined in accordance with the CDFM method. Defining R_S, R_D, and R_N as the median conservatism ratios associated with the CDFM method, then:

$$\begin{aligned} S_{50\%} &= R_S S_{CDFM} \\ D_{50\%} &= D_{CDFM}/R_D \\ F_{N_{50\%}} &= R_N F_{N_{CDFM}} \end{aligned} \quad (A.4)$$

and

$$\begin{aligned} C_{50\%} &= R_C C_{CDFM} \\ R_C &= R_S R_D R_N \end{aligned} \quad (A.5)$$

where R_C is the overall median conservatism ratio associated with the CDFM method. The ratios R_S, R_D, and R_N will be estimated in the following three subsections.

A.2.2 Median Strength Conservatism Ratio

The CDFM strength is normally computed using code specified allowable ultimate (maximum) strengths.

Based upon a review of median capacities from past seismic probabilistic risk assessment studies versus US code specified ultimate strengths for a number of failure modes, it is judged that for ductile failure modes when the conservatism of material strengths, code strength equations, and seismic strain-rate effects are considered, the code ultimate strengths have at least a 98% probability of exceedance. For a low ductility failure modes, an additional factor of conservatism of about 1.33 is typically introduced. Thus:

$$\begin{aligned} &\text{(Ductile)} && R_S = e^{2.054\beta_S} \\ &\text{(Low Ductility)} && R_S = 1.33e^{2.054\beta_S} \end{aligned} \quad (A.6)$$

where β_S is the strength logarithmic standard deviation (typically in the 0.2 to 0.4 range), and 2.054 is the standardized normal variable for 2% NEP.

A.2.3 Median Demand Conservatism Ratio

Within the US, seismic demands for CDFM evaluations are typically computed in accordance with the requirements of ASCE 4-86 (Ref. 12), except that median input spectral amplifications are used instead of median-plus-one-standard deviation amplification factors. When both are anchored to the same average spectral acceleration computed over a broad frequency range of interest such as 3 to 8 Hz, the ratio of median-plus-one-standard-deviation to median spectral acceleration amplification factor averages about

1.22. In addition, as noted in its foreword, ASCE 4-86 is aimed at achieving about a 10% probability of the actual seismic response exceeding the computed response, given the occurrence of the SME. Thus the median demand ratio R_D can be estimated from:

$$R_D = \frac{e^{1.282\beta_D}}{1.22} \quad \text{(A.7)}$$

where β_D is the seismic demand logarithmic standard deviation for a specified seismic input (typically in the 0.2 to 0.4 range).

A.2.4 Median Nonlinear Conservatism Ratio

In the CDFM method, the nonlinear factor is expected to be specified at about the 5% NEP level. Thus for ductile failure modes, the median nonlinear factor ratio R_N should be:

$$\text{Ductile } R_N = e^{1.645\beta_N} \quad \text{(A.8a)}$$

where β_N is the logarithmic standard deviation for the nonlinear factor (typically in the 0.2 to 0.4 range for ductile failure modes) and 1.645 is the standardized normal variable for 5% NEP.

However, for low ductility (brittle) failure modes, no credit is taken for a nonlinear factor, i.e.:

$$F_{N_{50\%}} \approx F_{N_{CDFM}} \approx 1.0$$
$$R_N \approx 1.0 \quad \text{(A.8b)}$$

A.2.5 Resulting CDFM Capacity Conservatism

Combining Eqns. (A.5) through (A.8) the median CDFM capacity ratio R_C is estimated to be:

$$\text{(Ductile Failures)} \quad R_C = 0.82e^{2.054\beta_S+1.282\beta_D+1.645\beta_N}$$
$$\text{(Low Ductility)} \quad R_C = 1.09e^{2.054\beta_S+1.282\beta_D} \quad \text{(A.9)}$$

and from Eqns. (A.5) and (4):

$$C_{1\%} = R_C C_{CDFM} e^{-2.326\beta} \text{(A.10)}$$

$$\beta = \left[\beta_S^2 + \beta_D^2 + \beta_N^2\right]^{1/2} \quad \text{(A.11)}$$

Table A.2 presents the ratio of ($C_{1\%}/C_{CDFM}$) for typical values of β_S, β_D, and β_N as computed from Eqns. (A.9) through (A.11). It can be seen that over this entire range of β values:

$$C_{1\%} \approx C_{CDFM} \text{(A.12)}$$

with the ratio ($C_{1\%}/C_{CDFM}$) ranging from 0.93 to 1.20 with a median value of 1.07.

The CDFM capacity can also be used to estimate the 10% probability of unacceptable performance capacity $C_{10\%}$. From Eqns. (A.5) and (5):

$$C_{10\%} = R_C \, C_{CDFM} e^{-1.282\beta} \quad (A.13)$$

Table A.3 presents the ratios of $(C_{10\%}/C_{CDFM})$ for typical values of β_S, β_D, and β_N. It can be seen that:

$$C_{10\%} \quad 1.5\, C_{CDFM} \quad (A.14)$$

However, the approximation of Eqn. (A.14) is not as good as that of Eqn. (A.12).

Table A-1

SUMMARY OF CONSERVATIVE DETERMINISTIC FAILURE MARGIN APPROACH

Load Combination:	Normal + SME
Ground Response Spectrum:	Conservatively specified (84% Non-Exceedance Probability).
Damping	Conservative estimate of median damping
Structural Model:	Best Estimate (Median) + Uncertainty Variation in Frequency
Soil-Structure-Interaction:	Best Estimate (Median) + Parameter Variation
Material Strength:	Code specified minimum strength or 95% exceedance actual strength if test data are available.
Static Strength Equations:	Code ultimate strength (ACI), maximum strength (AISC), Service Level D (ASME), or functional limits. If test data are available to demonstrate excessive conservatism of code equation then use 84% exceedance of test data for strength equation.
Inelastic Energy Absorption:	For non-brittle failure modes and linear analysis, use 80% of computed seismic stress in capacity evaluation to account for ductility benefits, or perform nonlinear analysis and go to 95% exceedance ductility levels.
In-Structure (Floor) Spectra Generation:	Use frequency shifting rather than peak broadening to account for uncertainty plus use median damping.

Table A.2 Ratio of ($C_{1\%}/C_{CDFM}$)

Strength Variability β_S	Demand Variability β_D	Low Ductility Failure Modes	Ductile Failure Modes β_N=0.2	Ductile Failure Modes β_N=0.4
0.2	0.2	1.10	0.99	0.99
	0.3	1.08	0.97	1.00
	0.4	0.97	0.93	0.99
0.3	0.2	1.13	1.04	1.08
	0.3	1.11	1.04	1.11
	0.4	1.05	1.01	1.10
0.4	0.2	1.13	1.07	1.15
	0.3	1.14	1.09	1.19
	0.4	1.11	1.07	1.20

Table A.3 Ratio of ($C_{10\%}/C_{CDFM}$)

Strength Variability β_S	Demand Variability β_D	Low Ductility Failure Modes	Ductile Failure Modes β_N=0.2	Ductile Failure Modes β_N=0.4
0.2	0.2	1.48	1.42	1.65
	0.3	1.57	1.49	1.76
	0.4	1.55	1.56	1.85
0.3	0.2	1.64	1.61	1.90
	0.3	1.72	1.70	2.04
	0.4	1.78	1.77	2.15
0.4	0.2	1.81	1.79	2.16
	0.3	1.92	1.91	2.33
	0.4	2.00	2.00	2.47

Appendix B

Derivation of Simplified Hybrid Method for Estimating Seismic Risk

Typical seismic hazard curves are close to linear when plotted on a log-log scale (for example see Fig. 2). Thus over at least any ten-fold difference in exceedance frequencies such hazard curves may be approximated by:

$$H_{(a)} = K_I a^{-K_H} \quad \text{(B.1)}$$

where $H_{(a)}$ is the annual frequency of exceedance of ground motion level "a", K_I is an appropriate constant, and K_H is a slope parameter defined by:

$$K_H = \frac{1}{\log(A_R)} \quad \text{(B.2)}$$

in which A_R is the ratio of ground motions corresponding to a ten-fold reduction in exceedance frequency.

So long as the fragility curve $P_{F/a}$ is lognormally distributed and the hazard curve is defined by Eqn. (B.1), a rigorous closed-form solution exists for the seismic risk Eqn. (10). This closed-form solution is derived in Ref. 13 as:

$$P_F = H F_{50\%}^{-K_H} e^{\alpha} \quad \text{(B.3a)}$$

$$F_{50\%} = \frac{C_{50\%}}{C_H} \quad \text{(B.3b)}$$

$$\alpha = \tfrac{1}{2}(K_H \beta)^2 \quad \text{(B.3c)}$$

where H is any reference exceedance frequency, C_H is the ground motion level that corresponds to this reference exceedance frequency H from the seismic hazard curve, $C_{50\%}$ is the median fragility, and β is the logarithmic standard deviation of the fragility. This derivation is reproduced herein in Appendix C.

Next, a specific hazard exceedance frequency $H_{10\%}$ is substituted for H in Eqn. (B.3a) where $H_{10\%}$ is defined at the ground motion $SA_{10\%}$ corresponding to a 10% conditional probability of failure. Thus:

$$F_{50\%} = \frac{C_{50\%}}{C_{10\%}} = e^{1.282\beta} \quad \text{(B.4)}$$

from which:

$$(P_F/H_{10\%}) = e^{-h_\beta} \quad \text{(B.5a)}$$

$$h_\beta = 1.282(K_H\beta) - 0.5(K_H\beta)^2 \quad \text{(B.5b)}$$

Table B.1 tabulates the ratio $(P_F/H_{10\%})$ over a range of A_R and β values.

Over the most common A_R range:

$$P_F \approx 0.5\, H_{10\%} \quad \text{(B.6)}$$

The use of Eqn. (B.6) should be limited to the range of A_R values for which $(P_F/H_{10\%})$ is less than about 0.6 in Table B.1 in order to avoid significant error. Thus, for $\beta = 0.4$, Eqn. (B.6) should only be used in the range of A_R from 1.6 to 5. However, this range should cover essentially any hazard curve of interest.

At $\beta = 0.4$, Eqn. (B.6) produces slightly conservatively biased estimates in the typical range of A_R from 1.75 to 3.0. Additional conservative bias occurs because when plotted on a log-log scale the hazard curves are not linear, but are slightly convex (see Fig. 2). The combination of these two factors typically leads to about 0% to 25% conservatism over the range of A_R values from 1.75 to 3.0.

Table B.1 Ratio $(P_F/H_{10\%})$ as a Function of A_R and β

A_R	K_H	β			
		0.3	0.4	0.5	0.6
5.0	1.43	0.63	0.57	0.52	0.48
4.5	1.53	0.62	0.55	0.50	0.47
4.0	1.66	0.60	0.53	0.49	0.46
3.5	1.84	0.57	0.51	0.47	0.45
3.0	2.10	0.54	0.49	0.45	0.44
2.75	2.28	0.53	0.47	0.44	0.44
2.5	2.51	0.51	0.46	0.44	0.45
2.25	2.84	0.48	0.44	0.44	0.48
2.0	3.32	0.46	0.44	0.47	0.57
1.75	4.11	0.44	0.47	0.59	0.89
1.60	4.90	0.45	0.55	0.87	1.74
1.5	5.68	0.48	0.72	1.48	4.21

Appendix C

Derivation of Closed Form Solution to Risk Equation

Assuming a lognormally distributed fragility curve with median capacity, C_{50}, and logarithmic standard deviation β, and defining the hazard exceedance probability $H_{(a)}$ by Equation (B.1), from Equation (10b) one obtains:

$$P_F = \int_0^{+\infty} \left\{K_I a^{-K_H}\right\} \left[\left(a\beta\sqrt{2\pi}\right) \exp\left\{\frac{(\lambda n\, a - M)^2}{2\beta^2}\right\}\right]^{-1} da \quad \text{(C.1)}$$

$$M = \lambda n \quad C_{50}$$

Defining $X = \lambda n\, a$, Equation (C.1 becomes:

$$P_F = \frac{K_I}{\beta\sqrt{2\pi}} \int_{-\infty}^{+\infty} \left[\exp\left\{K_H x - \left(\frac{(x - M)^2}{2\beta^2}\right)\right\}\right] dx \quad \text{(C.2)}$$

Many statistical textbooks (for example Appendix A of Ref. C.1) provide the solution to the definite integral shown in Eqn. (C.2). Thus:

$$P_F = K_I \exp\left\{-K_H M + \tfrac{1}{2}(K_H\beta)^2\right\} \quad \text{(C.3)}$$

or from the previous definition of M:

$$P_F = K_I C_{50}^{-K_H} e^{\frac{1}{2}(K_H\beta)^2} \quad \text{(C.4)}$$

Defining H as any reference exceedance frequency, C_H is the ground motion level that corresponds to this reference exceedance frequency H, then from Eqn. (B.1):

$$K_I = H[C_H]^{K_H} \quad \text{(C.5)}$$

from which:

$$P_F = HF_{50\%}^{-K_H} e^{\alpha} \quad \text{(C.6a)}$$

$$F_{50\%} = \frac{C_{50\%}}{C_H} \quad \text{(C.6b)}$$

$$\alpha = \tfrac{1}{2}(K_H\beta)^2 \quad \text{(C.6c)}$$

Reference:

C.1: Elishakoff, I., *Probabilistic Methods in the Theory of Structures*, John Wiley & Sons, 1983

* RPK Structural Mechanics Consulting, 28625 Mountain Meadow Road, Escondido, CA 92026 USA

Loess
SUMMARY STATISTICS

Statistic	Plastic Limit (%)	Plasticity Index (%)	Liquid Limit (%)	Su - UU Test (psf)	Su - UC Test (psf)	Moisture Content (%)	Dry Density (pcf)
CPS USAR Data - Station Site							
Count:	6	6	6	1	2	9	9
Average:	17.5	19.1	36.6	2229	975	19.3	103.9
St. Dev:	4.1	15.2	18.0	N/A	636	4.2	8.8
Min:	12.3	3.4	16.9	2229	525	13.6	95
Max:	22.8	45.5	64.8	2229	1425	24.6	120
EGC ESP Data							
Count:	2	2	2	0	2	2	2
Average:	14	18.0	32.0	N/A	3130	17.3	107.8
St. Dev:	0.0	14.1	14.1	N/A	1457	3.7	5.2
Min:	14	8	22	N/A	2100	14.7	104
Max:	14	28	42	N/A	4160	19.9	111

Wisconsinan Till
SUMMARY STATISTICS

Statistic	Plastic Limit (%)	Plasticity Index (%)	Liquid Limit (%)	Su - UU Test (psf)	Su - UC Test (psf)	Moisture Content (%)	Dry Density (pcf)
CPS USAR Data - Station Site							
Count:	37	37	37	16	17	10	9
Average:	12.9	10.2	23.1	3349	3137	12.1	123.1
St. Dev:	1.4	3.6	3.9	1590	1831	2.7	4.3
Min:	10.4	2.1	13.1	880	1320	7.5	116
Max:	16.8	17.1	30.7	6600	7900	15.6	128
EGC ESP Data							
Count:	2	2	2	0	2	2	2
Average:	13	10	23	N/A	3040	15.5	116.7
St. Dev:	0.0	0.0	0.0	N/A	735	0.6	1.4
Min:	13	10	23	N/A	2520	15	116
Max:	13	10	23	N/A	3560	15.9	118

Interglacial Soil
SUMMARY STATISTICS

Statistic	Plastic Limit (%)	Plasticity Index (%)	Liquid Limit (%)	Su - UU Test (psf)	Su - UC Test (psf)	Moisture Content (%)	Dry Density (pcf)
CPS USAR Data - Station Site							
Count:	25	25	25	6	10	12	12
Average:	12.6	12.9	25.5	4160.5	2094.5	17.0	113.4
St. Dev:	1.9	7.5	7.2	3657.1	1257.6	8.0	15.6
Min:	9.8	4.4	16.9	1530	800	7.1	74
Max:	16.6	30.4	42.5	10437	4000	36.8	133
EGC ESP Data							
Count:	3	3	3	0	2	3	2
Average:	25.7	14.3	40.0	N/A	5120	31.0	86.5
St. Dev:	21.9	4.9	19.5	N/A	5600	24.1	30.2
Min:	13	11	25	N/A	1160	15.4	65
Max:	51	20	62	N/A	9080	58.8	108

Illinoian Till
SUMMARY STATISTICS

Statistic	Plastic Limit (%)	Plasticity Index (%)	Liquid Limit (%)	Su - UU Test (psf)	Su - UC Test (psf)	Moisture Content (%)	Dry Density (pcf)	Initial Void Ratio	Compression Index	Re-Compression Index	Pre-Consolidation Pressure (psf)
CPS USAR Data - Station Site											
Count:	53	53	53	33	6	21	21	18	17	18	16
Average:	10.8	7.0	17.8	15323	6865	8.7	135.3	0.22	0.10	0.01	19063
St. Dev:	1.9	5.0	6.1	8529	5195	2.4	6.6	0.11	0.04	0.01	3610
Min:	8	1.7	13.1	962	1720	6.4	110	0.15	0.053	0.007	10500
Max:	22.3	35	46.7	36000	13410	17.7	141	0.56	0.19	0.03	25000
EGC ESP Data											
Count:	7	7	7	2	4	8	6	2	2	2	2
Average:	9.4	9.0	18.4	11016	15175	9.1	137.8	0.22	0.08	0.01	12000
St. Dev:	1.7	1.9	1.1	8859	10498	3.2	2.9	0.02	0.01	0.00	2828
Min:	8	6	17	4752	3360	5.4	134	0.199	0.079	0.0055	10000
Max:	13	11	20	17280	28800	14.9	141	0.234	0.089	0.0075	14000

Lacustrine
SUMMARY STATISTICS

Statistic	Plastic Limit (%)	Plasticity Index (%)	Liquid Limit (%)	Su - UU Test (psf)	Su - UC Test (psf)	Moisture Content (%)	Dry Density (pcf)
CPS USAR Data - Station Site							
Count:	2	2	2	5	0	2	2
Average:	11.5	7.4	18.9	5056	N/A	11.2	126.0
St. Dev:	1.8	0.1	1.7	2379	N/A	0.8	0.0
Min:	10.2	7.3	17.7	2502	N/A	10.6	126
Max:	12.8	7.5	20.1	7415	N/A	11.7	126
EGC ESP Data							
Count:	1	1	1	0	0	1	1
Average:	11	17	28	N/A	N/A	12.7	117.9
St. Dev:	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Min:	11	17	28	N/A	N/A	12.7	118
Max:	11	17	28	N/A	N/A	12.7	118

Pre-Illinoian Till
SUMMARY STATISTICS

Statistic	Plastic Limit (%)	Plasticity Index (%)	Liquid Limit (%)	Su - UU Test (psf)	Su - UC Test (psf)	Moisture Content (%)	Dry Density (pcf)
CPS USAR Data - Station Site							
Count:	11	11	11	9	1	5	5
Average:	13.9	13.4	27.3	10022	5400	14.2	120.2
St. Dev:	2.8	6.8	9.0	6944	N/A	4.2	9.4
Min:	9.6	4.4	14.9	2124	5400	6.8	116
Max:	17.8	25.5	43.3	20600	5400	16.9	137
TSC Data							
Count:	3	3	3	0	3	3	3
Average:	13.7	15.0	28.7	N/A	10080	13.70	121.7
St. Dev:	5.1	6.6	10.4	N/A	1215	4.62	11.98
Min:	8	9	17	N/A	9100	8.6	111
Max:	18	22	37	N/A	11440	17.6	134

TABLE 5-2
Summary of Shear and Compression Wave Velocity Test Data

		EGC ESP Site Results								CPS Site Results			
		Suspension Logging Test at B-2 Receiver to Receiver Measurements				Seismic Cone Test at CPT-2		Seismic Cone Test at CPT-4		Uphole Survey at P-14		Downhole Survey at P-14	
		Compression Wave Velocity (fps)		Shear Wave Velocity (fps)		Shear Wave Velocity (fps)		Shear Wave Velocity (fps)		Compression Wave Velocity (fps)		Shear Wave Velocity (fps)	
Depth Interval at B-2 (ft bgs)	Stratigraphic Unit	Range	Average	Range	Average	Range	Average	Range	Average	Range	Typical	Range	Typical
0 to 42	Loess & Wisconsinan Till	1680 to 6030	4788	820 to 1340	975	703 to 1354	1034	641 to 1077	838	NA	4800	900 to 1100	900 to 1100
42 to 59	Interglacial Zone (Weathered Illinoian Till)	5720 to 7500	6465	860 to 1970	1343	1022 to 1231	1132	1006 to 1602	1256	NA	4800	NA	1100
59 to 162	Illinoian Till	5720 to 8880	7552	1100 to 3250	2188	NA	NA	NA	NA	NA	7400	NA	2100
162 to 190	Lacustrine	6080 to 8040	6971	1390 to 2670	1829	NA	NA	NA	NA	NA	7400	NA	2100
190 to 269	Pre-Illinoian Till	5270 to 8230	6925	1560 to 2800	2068	NA	NA	NA	NA	NA	7400	NA	2100
269 to 292	Pre-Illinoian Alluvial / Lacustrine	5270 to 7940	6579	1190 to 3310	2045	NA	NA	NA	NA	NA	7400	NA	2100
292 to 307	Weathered Bedrock	7850 to 8440	8096	3250 to 3880	3420	NA	NA	NA	NA	NA	12000	NA	5700

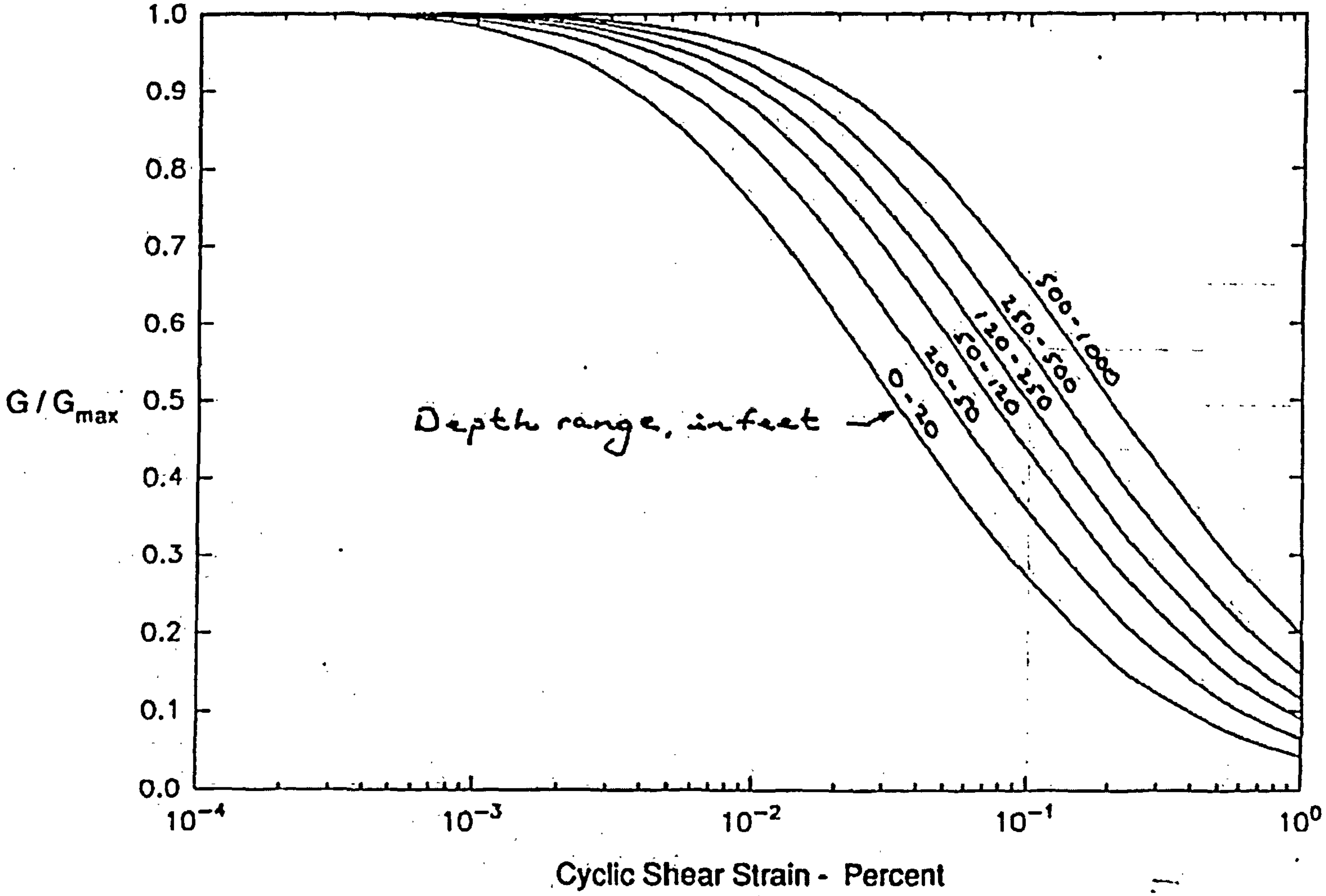


Figure 7.A-18
Modulus Reduction Curves for Generic ENA Sites

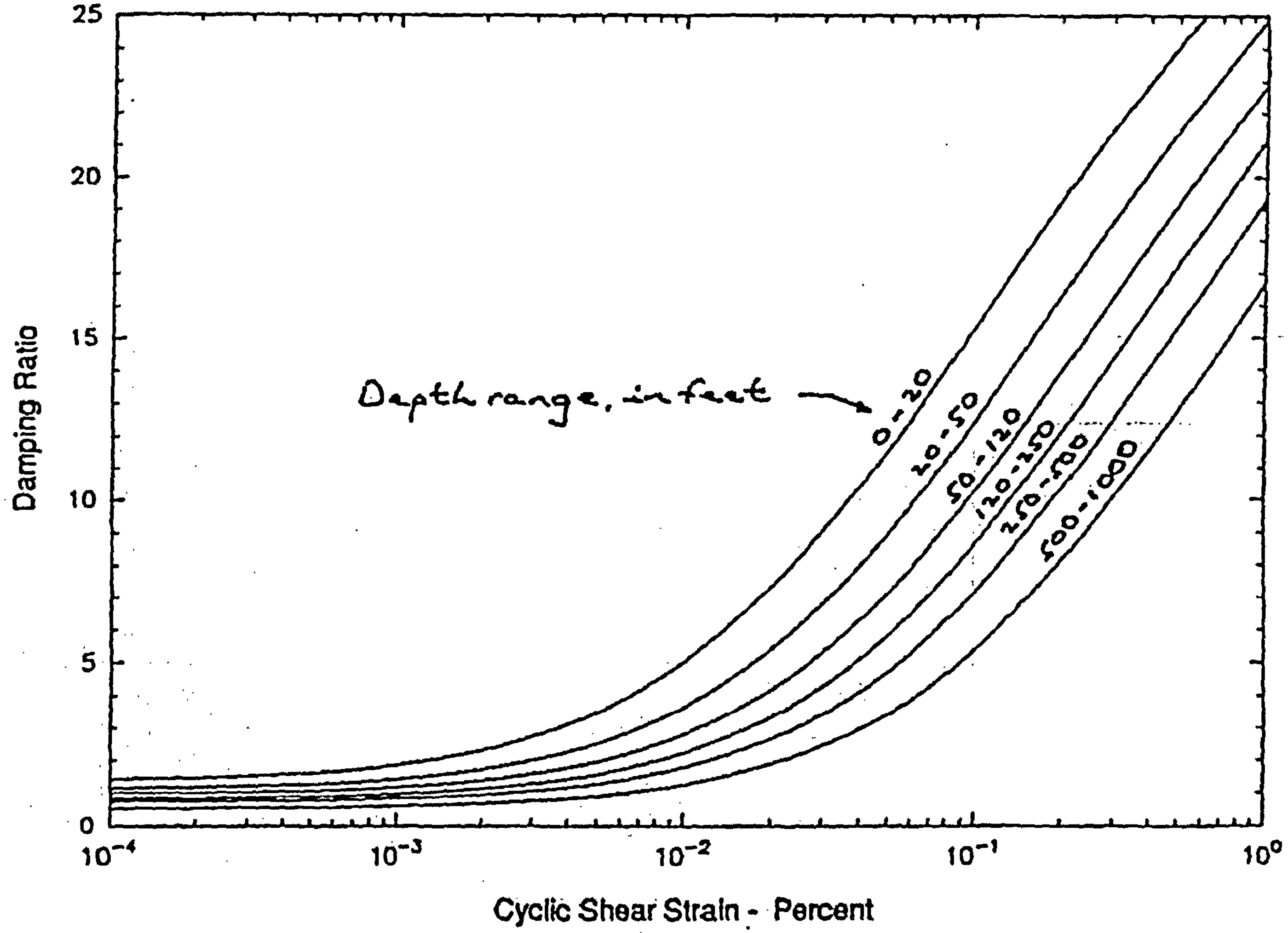


Figure 7.A-19
Damping Curves for Generic ENA Sites

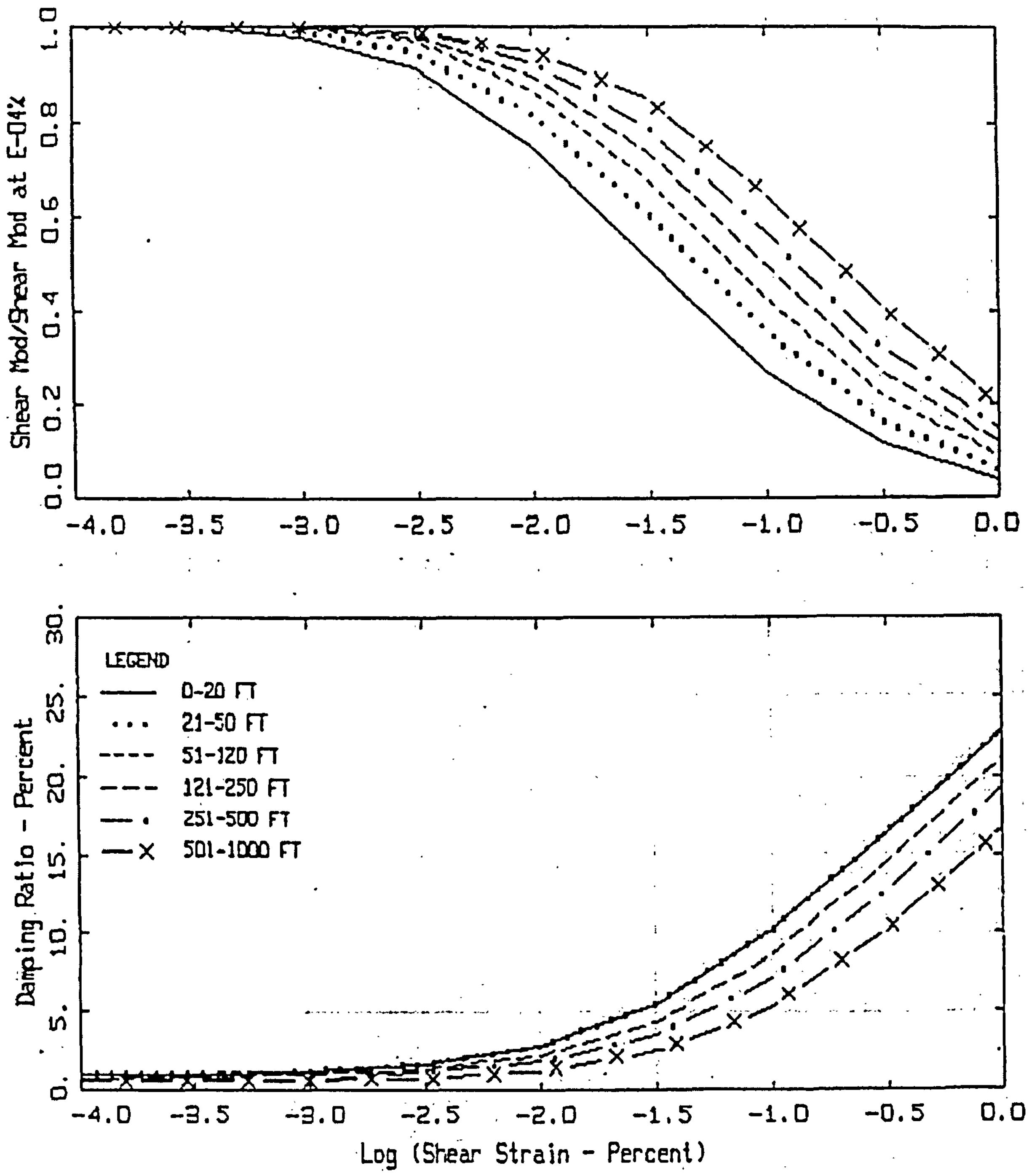


Figure 6-9. Modulus reduction and damping curves (median) used in the site response analyses. Damping curves are the same for depths ranging from 0 to 120 ft.

Description

This attachment presents a detailed example calculation of the factor of safety against liquefaction for the EGC ESP Site, as summarized in Section 6.1.1 of Appendix A to the SSAR. The calculation follows the method outlined in Youd et al. (2001). The example presented below corresponds to subsurface conditions at Borehole B-1, at a depth of 38.5 feet.

1) Design parameters

$GW_{field} = 6\ ft$	Measured Depth to Groundwater from ground surface
$GW_{design} = 5\ ft$	Design Depth to Groundwater
$ER = 52$	SPT Hammer Energy Transfer Ratio, percent
$PGA = 0.3$	Design Peak Ground Acceleration, percent of g
$M = 8$	Design Earthquake Magnitude

2) Input for Computations at Depth of 38.5 Feet

$Depth = 38.5\ ft$	Soil Depth Considered in Calculation
$N_{field} = 13$	SPT Blowcount @ 38.5 feet
$Fines = 13$	Fines Content @ 38.5 feet, percent

3) Computation of Overburden Stresses

The total overburden pressure at the 38.5-foot depth is the sum of the products of unit weight and thickness of each overlying soil unit. Effective overburden pressures are calculated based on the groundwater depth, as shown below.

Total Overburden Pressure @ 38.5 feet

$\sigma_{tot} = 2.561\ tsf$ (Sum of products of unit weight and thickness of overlying soils)

Field Effective Overburden Pressure @ 38.5-feet, tsf

$$\sigma_{eff.field} = \sigma_{tot} - (Depth - GW_{field}) \frac{62.4\ pcf}{2000} \qquad \sigma_{eff.field} = 1.547\, tsf$$

Design Effective Overburden Pressure @ 38.5 feet, tsf

$$\sigma_{eff.des} = \sigma_{tot} - (Depth - GW_{design}) \frac{62.4\ pcf}{2000} \qquad \sigma_{eff.des} = 1.516\, tsf$$

4) Computation of Liquefaction Strength (CCRR)

The soil resistance against liquefaction is calculated by the Cyclic Resistance Ratio (CRR). The CRR is calculated as a function of the clean sand blowcount, and the Corrected CRR (CCRR) adjusts the CRR for the overburden stress.

A) Blowcount Corrections: Four correction factors are applied to adjust the SPT blowcount to a standardized value ($N_{1.60}$). A clean sand blowcount is also calculated ($N_{1.60.cs}$), based on the fines content.

$$C_{over} = \frac{2.2}{\left[1.2 + \left(\frac{\sigma_{eff.field}}{1.058\ tsf}\right)\right]}$$

Overburden Correction Factor

$C_{over} = 0.826$

$$C_{rod} = 1.00$$

Rod Length Correction Factor
(1.0 for rod length of 30 to 100 feet)

$$C_{hammer} = \frac{ER}{60}$$

Hammer Energy Correction Factor

$C_{hammer} = 0.867$

$$C_{samp.meth} = 1.1$$

Sampling Method Correction Factor
(1.1 if a sample liner is not used)

Corrected Blowcount, without fines ($N_{1.60}$)

$$N_{1.60} = N_{field}\, C_{over}\, C_{rod}\, C_{hammer}\, C_{samp.meth}$$

$N_{1.60} = 10.242$

Clean Sand Blowcount ($N_{1.60.cs}$)
(Equation for fines between 5 and 30 percent)

$$N_{1.60.cs} = \exp\left[1.76 - \left(\frac{190}{Fines^2}\right)\right] + \left[0.99 + \left(\frac{Fines^{1.5}}{1000}\right)\right] N_{1.60}$$

$N_{1.60.cs} = 12.508$

B) Calculation of the CRR and the CCRR

Cyclic Resistance Ratio (CRR)
(Equation for $N_{(1.60.cs)}$ between 5 and 30)

$$CRR = \frac{\left[0.048 - 0.004721\left(N_{1.60.cs}\right) + 0.0006136\left(N_{1.60.cs}\right)^2 - 0.00001673\left(N_{1.60.cs}\right)^3\right]}{\left[1 - 0.1248\left(N_{1.60.cs}\right) + 0.009578\left(N_{1.60.cs}\right)^2 - 0.0003285\left(N_{1.60.cs}\right)^3 + 0.000003714\left(N_{1.60.cs}\right)^4\right]}$$

$CRR = 0.135$

CRR Overburden Correction Factor (K_σ)
(Equation for $\sigma_{eff.des}$ between 1 and 3 tsf)

$$K_\sigma = 1.1 - \left(\frac{\sigma_{eff.des}}{tsf}\right) 0.1 \qquad K_\sigma = 0.948$$

CRR Static Shear Correction Factor (K_α)
(1.0 for level to gently sloping ground)

$$K_\alpha = 1$$

Corrected Cyclic Resistance Ratio (CCRR)

$$CCRR = CRR\ K_\sigma\ K_\alpha \qquad CCRR = 0.128$$

5) Computation of Earthquake Induced Stress (CSR)
The Cyclic Stress Ratio (CSR) is calculated as a function of depth (via the stress reduction coefficient), ground acceleration, and groundwater depth.

Stress Reduction Coefficient (r_d)
(Equation for depth between 30 and 75 feet)

$$r_d = 1.174 - 0.00814 \frac{Depth}{1 ft} \qquad r_d = 0.861$$

Cyclic Stress Ratio (CSR)

$$CSR = 0.65\ PGA \left(\frac{\sigma_{tot}}{\sigma_{eff.des}}\right) r_d \qquad CSR = 0.284$$

6) Computation of Factor of Safety (FS)
The Factor of Safety (FS) is a function of the ratio of CCRR to CSR, as well as the Magnitude Scaling Factor (MSF).

Magnitude Scaling Factor (MSF)

$$MSF = \frac{10^{2.24}}{M^{2.56}} \qquad MSF = 0.847$$

Factor of Safety (FS)

$$FS = \left(\frac{CCRR}{CSR}\right) MSF \qquad FS = 0.384$$