PERFORMANCE-GOAL BASED (RISK INFORMED) APPROACH FOR ESTABLISHING THE SSE SITE SPECIFIC RESPONSE SPECTRUM FOR FUTURE NUCLEAR POWER PLANTS[1]

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1. Introduction

The Safe Shutdown Earthquake (SSE) Site-Specific Response Spectrum (SSRS) for future nuclear power plants can be established following the Performance-Goal Based (Risk Informed) Approach defined in the ASCE (2005) Standard 43-05. The standard is a professional consensus committee developed standard. This standard is formally constructed to produce designs aimed at achieving a target acceptable seismic risk goal, defined as the annual probability of seismic induced unacceptable performance. The first step in this process is to develop a risk-consistent or Uniform Risk Response Spectrum (URRS) which will be used as the SSRS. When these URRSs are used as the SSRSs, plants at different sites (all designed to the same design criteria, such as NUREG 0800, for their particular SSRSs) should have consistent seismic risks. In contrast, this risk-consistency goal is not achieved when a Uniform Hazard Response Spectrum (UHRS) is used; the UHRS fails to reflect the fact that the seismic hazard curves at different sites have substantially different slopes, and consideration of these slopes is critical to obtaining risk-consistent seismic designs. As described below, the URRS does depend on both the UHRS and these slopes.

The risk-consistent approach presented in ASCE (2005) to define the SSRS was first adopted in 1994 in the Commentary of DOE-STD-1020-94 (USDOE, 1994) for risk-consistent seismic design of High Consequence (PC4) DOE facilities. The detailed basis was given in Kennedy and Short (1994). Therefore, this approach has been in existence and has been used for over 10 years. Very similar risk-consistent approaches for defining the SSRS are presented in Kennedy (1997) and Kennedy (1999). A more liberal risk-consistent approach for defining the SSRS was proposed and studied in NUREG/CR-6728 (REI, 2001). The ASCE (2005) Standard 43-05 approach instead of that in NUREG/CR-6728 is recommended for nuclear power plant application because the ASCE Standard 43-05 definition of the SSRS is more conservative and because this Standard is a professional consensus standard.

[1] This paper has benefited from review comments provided by Dr. Carl Stepp, Earthquake Hazards Solutions, and Dr. Allin Cornell, Stanford University.

The purpose of this paper is to amplify upon the Commentary of ASCE (2005) in explaining the basis and assumptions behind the ASCE Standard 43-05 approach for defining the risk-consistent SSRS. To do so this paper has extracted extensive material from ASCE (2005), USDOE (1994), Kennedy and Short (1994), Kennedy (1997), Kennedy (1999), Kennedy (1999a), and REI (2001).

Four issues must be addressed in order to establish the criteria for computing the risk-consistent SSRS. These issues are:

1. What is the target seismic risk goal P_{FT} that is to be aimed at by the specified seismic criteria? This goal needs to be defined in terms of both a quantitative target acceptable annual probability of unacceptable performance P_{FT}, and a qualitative description as to what constitutes unacceptable performance. This issue is further discussed in Section 4.

2. What is the level of conservatism implied by use of the specified seismic design criteria? In particular, to what degree does NUREG-0800 provide seismic margin in the structures, systems and components designed to its criteria? And how is this represented? This issue will be discussed in Section 5.

3. To maintain the convention of using a UHRS, the SSRS will be calculated by

   $$SSRS = DF * UHRS \qquad \textbf{Equation 1}$$

   where UHRS is a "reference" Uniform Hazard Response Spectrum and DF is the Design (Scale) Factor used to define the SSRS relative to the UHRS. Given this basis, at what reference seismic hazard exceedance frequency H should the reference UHRS be defined? As discussed above there is a unique SSRS at a site that will provide risk consistency for any specified performance-goal. But there are clearly many pairs of UHRS levels and DF factors that will produce the same SSRS. Therefore there is some latitude in the selection of the value of H to be used. For practical reasons it should be within the bounds of 2 to 20 times P_{FT}, as described in Section 6. However, once the value of H is chosen the required DF to be used in Equation 1 will be a function of the Probability Ratio R_P defined by:

   $$R_P = \frac{H}{P_{FT}} \qquad \textbf{Equation 2}$$

   Clearly the larger the value of H the lower the UHRS and the larger DF needs to be to give the unique SSRS. Therefore DF is an increasing function of R_P. In addition, DF is a decreasing function of the conservatism of the seismic design criteria (Issue #2) and a decreasing function of the amplitude of the (negative) slope of the seismic hazard curve. This issue of selecting the value of H is discussed in Section 6.

4. Having defined P_{FT} (Issue #1), conservatism of seismic design criteria (Issue #2), and H (Issue #3), the equation for DF needs to be developed which insures that the performance goal P_{FT} is achieved with the SSRS defined by Equation 1 when UHRS is defined at the exceedance frequency H. This step involves first using a basic probabilistic analysis to find an analytical equation for the P_{FT} as a function of a seismic hazard curve and a fragility curve of a typical component, and then re-arranging and empirically simplifying this result to form the equation for DF for use in application. Section 3 will present the derivation of the underlying theoretical equations used to develop the equation for the Design Factor DF. The

ASCE (2005) Standard 43-05 equation for DF is derived and discussed in Section 7 for R_P=10 which is proposed herein.

2. Summary of ASCE (2005) Standard 43-05 Approach for Defining Performance-Goal Based Site Specific Response Spectrum (SSRS)

A fundamental assumption is that Seismic Category 1 Structures, Systems, and Components (SSCs) in a nuclear power plant will be designed for the SSRS utilizing the seismic capacity, seismic demand, and seismic design criteria laid out by the U.S. NRC for nuclear power plants in NUREG-0800 (USNRC, No Date), Regulatory Guides, and professional design codes and standards referenced therein. The U.S. NRC criteria are very similar to the criteria presented in the ASCE (2005) Standard 43-05 for the most stringent Seismic Design Category SDC-5D. Therefore, the criteria specified in the ASCE Standard 43-05 for SDC-5D are used to define the SSRS for nuclear power plants.

For SDC-5D, the quantitative target acceptable annual probability of unacceptable performance P_{FT} is[2]:

$$P_{FT} = mean\, 1x10^{-5} / yr$$ **Equation 3**

The qualitative description of acceptable performance for SDC-5D is to not exceed Limit State D which is defined in the ASCE Standard 43-05 as “Essentially Elastic Behavior.” Thus, the definition of unacceptable performance for SDC-5D is the “onset of significant inelastic deformation.”

Thus, the SSRS is established at a level such that SSCs designed to meet U.S. NRC criteria for nuclear power plants will have a target mean annual frequency[3] of $1x10^{-5}$/yr for seismic-induced onset of significant inelastic deformation (FOSID).

It should be noted that Limit State D is well short of damage that might interfere with functionality, which generally corresponds to Limit States B or C. Furthermore, the onset of significant cyclic strength reduction in structures also corresponds to Limit States B or C, and the onset of collapse corresponds to beyond Limit State A defined in the ASCE Standard 43-05. The mean annual frequency of exceeding Limit States C, B, or A which might lead to core damage are less than $1x10^{-5}$ by increasingly larger factors.

In order to achieve the above defined target performance goal for SDC-5D, the ASCE Standard 43-05 defines the SSRS by Equation 1, where the reference UHRS is defined at a reference seismic hazard exceedance frequency H of:

$$H = \text{mean } 1x10^{-4}/yr$$ **Equation 4**

Next, the required Design Factor DF is computed as follows. First, at each spectral frequency at which the UHRS is defined, an Amplitude Ratio A_R is computed from:

[2] The term “mean” in front of the probability here and elsewhere means that the *mean* estimate of this probability should be used, in contrast to, for example, Reg Guide 1.165, which calls for the median estimate.

[3] The terms “annual frequency” and “annual probability”, while not strictly equivalent, are used interchangeably here as they are numerically equivalent at these low levels.

$$A_R = \frac{SA_{0.1H}}{SA_H}$$ **Equation 5**

where SA_H is the spectral acceleration at the mean exceedance frequency H and $SA_{0.1H}$ is the spectral acceleration at 0.1H (i.e., the spectral accelerations at $1x10^{-4}$, and $1x10^{-5}$/yr). Then the Design Factor, DF, at each spectral frequency is given by

$$DF = \text{Maximum}\ (DF_1\ , DF_2)$$ **Equation 6**

where

$$DF_1 = 1.0$$ **Equation 7**

and

$$DF_2 = 0.6(A_R)^{0.80}$$ **Equation 8**

which correspond to the appropriate DF_1 and DF_2 from Table 2.2-1 of the ASCE (2005) Standard 43-05 for $R_P = 10$ from Equation 2.

Furthermore, for SDC-5D, the ASCE Standard 43-05 specifies a lower bound on the SSRS peak ground acceleration (PGA) of 0.10g. For nuclear power plant applications, the lower bound on the SSRS is recommended to be a Reg. Guide 1.60 response spectrum anchored to a PGA of 0.10g.

3. Theoretical Derivation of Design Factor DF

This section develops an equation for the DF from an analytical result for the risk, that is, the probability of unacceptable performance (or "failure[4]").

3.1 Rigorous Seismic Risk Equation

Given a mean seismic hazard curve and a mean fragility curve, 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 of two analytically equivalent equations:

$$P_F = -\int_0^{+\infty} P_F(a)\left(\frac{dH(a)}{da}\right) da$$ **Equation 9**

$$P_F = \int_0^{+\infty} H(a)\left(\frac{dP_F(a)}{da}\right) da$$ **Equation 10**

where $P_F(a)$ is the conditional probability of failure given the ground motion level a , which, by definition, is the mean fragility curve, and $H(a)$ is the mean hazard exceedance frequency corresponding to ground motion level For example, in words, the first says loosely that the probability of failure is the probability that the ground motion has value *a* times the probability of component failure given that level, integrated over all possible levels of (The minus sign is a

[4] As used herein, "failure" consists of unacceptable FOSID

result of "correcting" for the derivative of H(*a*) being negative. Recall the H(*a*) is the probability of exceeding *a* so it decreases as *a* increases.)

The mean fragility curves used can be that for failure (i.e., unacceptable performance) of an individual SSC or for a plant damage state such as core damage.

3.2 Simplified Seismic Risk Equation

Typical seismic hazard curves are close to linear when plotted on a log-log scale (for example see Figure 1). Thus over any (at least) ten-fold difference in exceedance frequencies such hazard curves may be approximated by a power law:

$$H(a) = K_I a^{-K_H} \qquad \textbf{Equation 11}$$

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:

$$K_H = \frac{1}{\log(A_R)} \qquad \textbf{Equation 12}$$

in which A_R is the ratio of ground motions corresponding to a ten-fold reduction in exceedance frequency, Equation 5.

So long as the fragility curve $P_F(a)$ is lognormally distributed and the hazard curve is defined by Equation 11, a rigorous closed-form solution exists for the seismic risk (Equations 9 or 10). This closed-form solution is derived in Appendix A as:

$$P_F = H \; F_{50\%}^{-K_H} \; e^{\alpha} \qquad \textbf{Equation 13}$$

in which

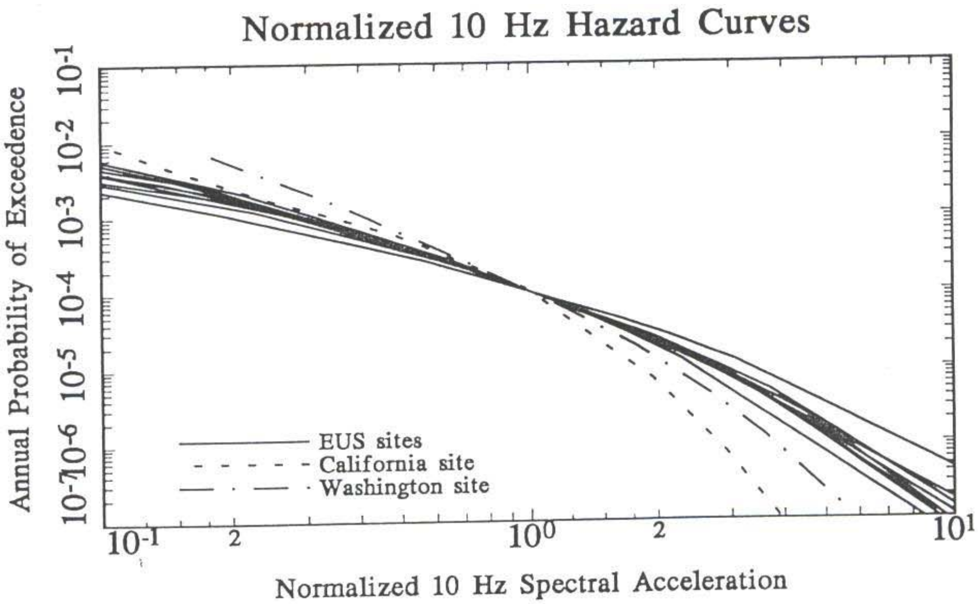
$$F_{50\%} = \frac{C_{50\%}}{C_H} \qquad \textbf{Equation 14}$$

and

$$\alpha = \tfrac{1}{2}(K_H \beta)^2 \qquad \textbf{Equation 15}$$

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

Equation 13 is referred to here as the simplified seismic risk equation. The only approximations in its derivation are that the hazard curve is approximated by Equation 11 over the exceedance frequency range of interest and the fragility curve is lognormally distributed.



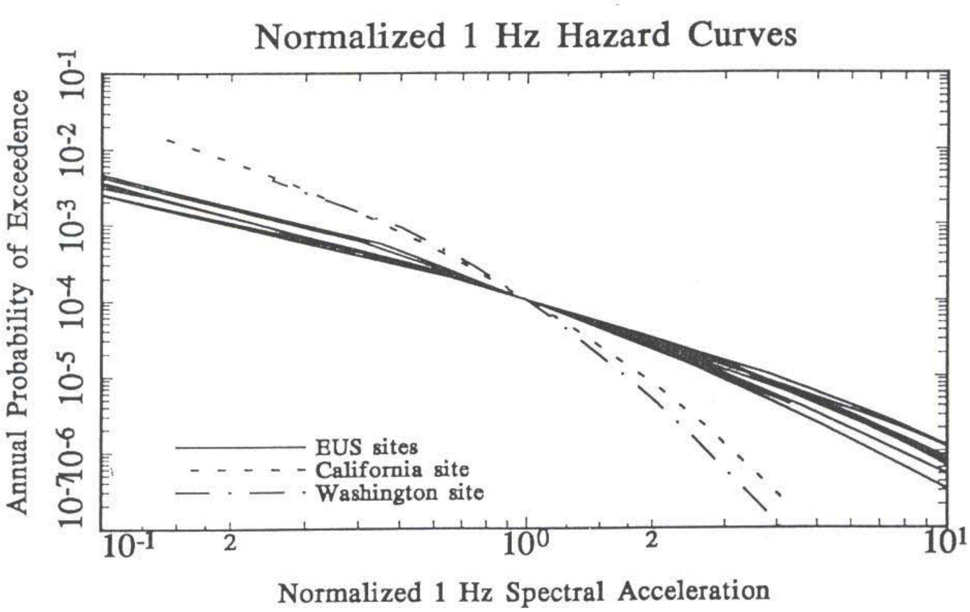


Figure 1
SA (10 Hz) and SA (1 Hz) hazard curves for the eleven sites normalized by the acceleration value corresponding to mean 10^{-4} annual probability. (From Figures. 7.7 and 7.8 of REI, 2001)

3.3 Design Factor Equation

With the Probability Ratio R_P defined by Equation 2, Equation 13 can be rearranged to define the median fragility capacity $C_{50\%}$ required to achieve a desired Probability Ratio R_P:

$$C_{50\%} = C_H \left[R_p e^{\alpha}\right]^{1/K_H} \quad \textbf{Equation 16}$$

The conservatism introduced by the seismic design criteria such as NUREG-0800 can be defined by a seismic margin factor F_P given by:

$$F_P = \frac{C_P}{SSRS} \quad \textbf{Equation 17}$$

where C_P, defined more formally below, is a value on the fragility curve corresponding to a conditional failure probability, P, i.e., C_p is a fractile of the fragility curve. In words, if one designs a component by some set of seismic criteria (e.g., NUREG-0800) for a ground motion level SSRS, those criteria will insure that this C_P fractile is F_P times larger than SSRS. Next, defining the SSRS by Equation 1 and recognizing that C_H=UHRS, then:

$$F_P = \frac{C_P}{DF * C_H} \quad \textbf{Equation 18}$$

Lastly, the C_P fractile or "seismic capacity point" on a lognormal fragility curve can be defined in terms of the median fragility capacity $C_{50\%}$ and logarithmic standard deviation β by:

$$C_P = C_{50\%} e^{X_p \beta} \quad \textbf{Equation 19}$$

where X_P is the standard normal variable associated with P percent non-exceedance probability (NEP). For example, $C_{1\%}$, is factor $e^{-2.326\,\beta}$ times the median capacity so that X_p is -2.326.

Combining Equations. 16, 18 and 19:

$$DF = \frac{\left[R_P e^{-f}\right]^{1/K_H}}{F_P} \quad \textbf{Equation 20}$$

in which

$$f = -X_P (K_H \beta) - \tfrac{1}{2}(K_H \beta)^2 \quad \textbf{Equation 21}$$

Equation 20 defines the required Design Factor DF to achieve any desired Probability Ratio R_P. As anticipated above, DF is an increasing function of R_P. For a given target P_{FT} the larger you set H (i.e., the lower you make the UHRS), the larger R_P and DF must be to compensate for this higher H. But how strongly it depends on R_P depends on K_H, the hazard curve slope defined in Equation 12.

Note, too, that the required DF is a complicated but generally decreasing function of the slope parameter K_H and a simple inverse function of the seismic conservatism factor F_P of the seismic design criteria. Again there is latitude in that the factor F_P can be defined in terms of any conditional failure probability P point on the fragility curve. The value chosen has practical implications, however. If P is defined in the 1% to 20% failure probability range, DF is only moderately sensitive to β. This insensitivity is exploited in practical seismic guidelines, such as ASCE (2005), as it permits DF to be defined effectively independently of β. The X_P values corresponding to various failure probability P levels at which F_P is to be defined are:

Table 1
X_p Values for Different Failure Probabilities

P	X_P
1%	-2.326
5%	-1.645
10%	-1.282
20%	-0.842

As an example, if the seismic conservatism factor is defined at the 1% probability of failure level $F_{1\%}$, then:

$$DF = \frac{\left[R_P e^{-f}\right]^{1/K_H}}{F_{1\%}}$$ **Equation 22**

$$f = 2.326K_H\beta - \frac{1}{2}(K_H\beta)^2$$ **Equation 23**

Equation 22 will be used in Section 7 to develop the simplified equation for the ASCE Standard 43-05 Design Factor in Equation 6 given in Section 2 for R_P=10.

4. Basis for Target Performance Goal

As discussed in Section 2, the target performance goal for the ASCE (2005) Standard 43-05 SDC-5D SSCs, which was adopted herein for nuclear power plant application, is a mean frequency of $1x10^{-5}$/yr for seismic induced onset of significant inelastic deformation (FOSID).

The basis for selecting a quantitative target performance goal P_T of mean $1x10^{-5}$/yr is that mean $1x10^{-5}$/yr represents approximately the average seismic-induced Core Damage Frequency (CDF) reported for those nuclear power plants which have performed seismic probabilistic risk assessments (SPRAs) and presented their results to the U.S. NRC. For example, Table 2 shows the mean seismic CDF for 25 plants which performed SPRAs using EPRI-type hazard curves as reported in NUREG 1742 (USNRC, 2001). The reported mean seismic CDFs range from approximately $2x10^{-7}$/yr to $2x10^{-4}$/yr with a median value of $1.2x10^{-5}$/yr and a mean value of $2.5x10^{-5}$/yr. For these 25 plants, 7 plants report mean seismic CDF values significantly less than $1x10^{-5}$/yr and 7 plants report values significantly higher than $1x10^{-5}$/yr. The mean seismic CDF values for the remaining 11 plants are all close to $1x10^{-5}$/yr.

Table 2
Mean Seismic CDF for Plants Performing Seismic PRA
from Table 2.2 from NUREG 1742, Vol. 2

Plant	Mean Seismic CDF (EPRI)*
South Texas Project 1 & 2	1.90E-07
Nine Mile Point 2	2.50E-07
La Salle 1 & 2	7.60E-07
Hope Creek	1.06E-06
D.C. Cook 1 & 2	3.20E-06
Salem 1 & 2	4.70E-06
Oyster Creek	4.74E-06
Surry 1 & 2	8.20E-06
Millstone 3	9.10E-06
Beaver Valley 2	1.03E-05
Kewaunee	1.10E-05
McGuire 1 & 2	1.10E-05

Plant	Mean Seismic CDF (EPRI)*
Seabrook	1.20E-05
Beaver Valley 1	1.29E-05
Indian Point 2	1.30E-05
Point Beach 1 & 2	1.40E-05
Catawba 1 & 2	1.60E-05
San Onofre 2 & 3	1.70E-05
Columbia (Washington Nuclear Project No. 2)	2.10E-05
TMI 1	3.21E-05
Oconee 1, 2, and 3	3.47E-05
Diablo Canyon 1 & 2	4.20E-05
Pilgrim 1	5.80E-05
Indian Point 3	5.90E-05
Haddam Neck	2.30E-04

Median of Mean Seismic CDF Value (EPRI Results)	1.20E-05
Mean of Mean Seismic CDF Value (EPRI Results)	2.50E-05
* CDF Values reported are for EPRI hazard curves. LLNL hazard curves produced substantially higher CDF results	

Additionally, a conservative bias is introduced by choosing the onset of significant inelastic deformation as the qualitative performance goal. This performance goal corresponds to significantly less damage than would be required to reach core damage. Therefore, holding the FOSID to a target of mean $1x10^{-5}$ /yr insures that the CDF will be significantly below mean $1x10^{-5}$/yr. It is expected that the CDF will be between $6x10^{-6}$/yr and $0.6x10^{-6}$/yr. The basis for this expectation is presented in Section 8.

5. Level of Conservatism of Specified Seismic Design Criteria

5.1 Factor of Conservatism for the Onset of Significant Inelastic Deformation

As noted in Section 2, a fundamental assumption is that Seismic Category 1 SSCs will be designed for the SSRS utilizing the seismic capacity, seismic demand, and seismic design criteria laid out by the U.S. NRC for nuclear power plants in NUREG-0800 (USNRC, No Date), Regulatory Guides, and professional design codes and standards referenced therein. It was also noted that these U.S. NRC criteria are very similar to the criteria presented in the ASCE (2005) Standard 43-05 for SDC-5D SSCs. Thus ASCE Standard 43-05 states that the seismic demand and structural capacity evaluation criteria presented therein are aimed at having sufficient conservatism to reasonably achieve *both* of the following:

1. Less Than About a 1% Probability of Unacceptable Performance for the Design Basis Earthquake Ground Motion, and
2. Less than About a 10% Probability of Unacceptable Performance for a Ground Motion Equal to 150% of the Design Basis Earthquake Ground Motion

The basis for these estimated factors of Conservatism is presented in the Commentary Section C1.3 of ASCE (2005) Standard 43-05.

In computing the required DF for determining the SSRS, these same factors of conservatism against the onset of significant inelastic deformation will be used for nuclear power plant Seismic Category I SSCs designed to meet NRC criteria. Even for the onset of significant inelastic deformation, the above factors of conservatism are expected to be conservatively underestimated because designers do not typically design an SSC to just barely satisfy the acceptance criteria. Additional margin or conservatism is generally included. However, no credit is taken for this added margin when determining the required DF.

Seismic fragility (i.e., the conditional probability of failure versus ground motion levels, $P_F(a)$) is typically defined as being lognormally distributed so that it can be fully described by two parameters, such as a seismic margin factor F_P corresponding to a conditional failure probability P_{FC} (Equation 17), and an estimate of the capacity variability (i.e., the logarithmic standard deviation β). The two ASCE Standard 43-05 target levels of conservatism defined above result in the following seismic margin factors $F_{1\%}$, $F_{5\%}$,, $F_{10\%}$, $F_{50\%}$, and $F_{70\%}$ corresponding to a 1%, 5%, 10%, 50%, and 70% conditional probability of unacceptable behavior, respectively:

Table 3
Seismic Margin Factors for Different β Values

β	$F_{1\%}$	$F_{5\%}$	$F_{10\%}$	$F_{50\%}$	$F_{70\%}$
.30	1.10	1.35	1.5	2.2	2.58
.4	1	1.31	1.52	2.54	3.13
.5	1	1.41	1.69	3.2	4.16
.6	1	1.5	1.87	4.04	5.53

Note that for a logarithmic standard deviation less than 0.39, the second of the two conditional probability goals controls the fragility. For β greater than 0.39, the first goal controls. By specifying both goals, the following margins are achieved:

- $F_{1\%}$ ≥ 1.0
- $F_{5\%}$ ≥ 1.3
- $F_{10\%}$ ≥ 1.5
- $F_{50\%}$ increases with increasing β

The required Design Factor DF will be computed in Section 7 for the above values of β which range from 0.3 to 0.6, and the corresponding seismic factors of conservatism F_P.

From EPRI (1994) and past SPRA studies, 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. Therefore, the range 0.3 to 0.6 covers the practical range for β.

5.2 Expected Factor of Conservatism for Core Damage Fragility

The seismic design criteria factors of conservatism defined in Section 5.1 are for the unacceptable performance defined as the onset of significant inelastic deformation. These margin factors are substantially too low for a Core Damage definition of unacceptable performance.

For the new Standard Plant designs, the U.S. NRC staff (SECY-93-087) has required that a study be performed to show that the Core Damage HCLPF[5] margin factor is at least 1.67 times the SSRS. The HCLPF point on the fragility curve computed in accordance with EPRI (1991) corresponds to the mean 1% conditional probability of failure point on the Core Damage fragility curve. Thus, for Core Damage:

$$F_{1\%} = 1.67 \qquad \textbf{Equation 24}$$

For the above reason, NUREG/CR-6728 used the more liberal $F_{1\%}$=1.67 HCLPF margin when computing risk-consistent SSRS.

Section 8 computes the mean Core Damage Frequency (CDF) when the SSRS is defined by the ASCE Standard 43-05 method described in Section 2 and a Core Damage $F_{1\%}$=1.67 is used.

6. Reference Mean Hazard Exceedance Frequency H Used to Define the Reference UHRS

For SDC-5D SSCs, the ASCE (2005) Standard 43-05 defines the reference mean hazard exceedance frequency H to be:

$$H = \text{mean } 1 \times 10^{-4}/\text{yr} \qquad \textbf{Equation 25}$$

and defines the Design Factor DF so as to achieve a Probability Ratio R_P of 10; together these two values achieve the target FOSID Performance Goal of P_{FT}= mean $1\text{x}10^{-5}$/yr.

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[5] HCLPF is short for "High Confidence of a Low Probability of Failure".

While the ratio of H/R_p is important to obtaining the final Performance Goal, this particular choice of H and R_p values is, as discussed above, rather arbitrary. Any hazard exceedance frequency H between mean $2x10^{-4}$/yr and $2x10^{-5}$/yr could have been used to achieve P_{FT}= mean $1x10^{-5}$/yr, but for a different H value the value of R_P would have to change correspondingly. That would be done by changing the value of DF. The result would be essentially the same SSE Site Specific Response Spectrum SSRS for any H and R_P pair. Therefore the reasons for a particular choice of H (and hence R_P) is practical convenience.

The primary reason for choosing R_P=10 is to insure that the DF is never less than unity, which would be an unfamiliar value for a structural load factor. For Western U.S. sites near major tectonic plate boundaries, the mean hazard curve has a steep slope so that the Amplitude Ratio A_R defined by Equation 5 is less than 1.9 implying the slope K_H is greater than 3.6. For these Western U.S. sites DF=1.0 (as given by Equation 6) so that the SSRS simply equals the mean $1x10^{-4}$ UHRS. For Central and Eastern U.S. (CEUS) sites the mean hazard curve slope is shallower so that A_R typically lies in the range of 1.9 to 4.0 so that the DF ranges from 1.0 to 1.8. For these CEUS sites the DF is always equal to or greater than 1.0, but never excessively large. Thus, the proposed method never ends up with a SSRS less than the mean $1x10^{-4}$ UHRS nor likely to be larger than 1.8 times the mean $1x10^{-4}$ UHRS.

7. Assessment of ASCE Standard 43-05 Design Factor DF for Probability Ratio R_P of 10

The ASCE Standard 43-05 DF is computed by Equation 6 which was obtained by an empirical fit. In this section we assess how well the simplified formula works by comparing these DFs with those obtained from the more precise formula, Equation 22, and by comparing how close the failure probabilities implied by use of Equation 6 are to the target acceptable failure probability. The latter computation will be done two ways, using the analytical approximation (Equation 13) and by numerical integration of the exact integrals.

7.1 Computation of Required DF for Comparison with ASCE Standard 43-05 DF

The required Design Factors DF computed using Equation 22 to achieve R_P=10 for the onset of significant inelastic deformation $F_{1\%}$ and β combinations defined in Section 5.1 are shown in Table 4 for an Amplitude Ratio A_R range from 1.5 to 6.0. These required DF factors are compared with ASCE Standard 43-05 DF given by Equation 6. The ASCE Standard 43-05 DF Equation 6 was empirically developed to closely fit these required DF values.

Table 4
Design Factor DF Values Required to Achieve A Probability Ratio R_P = 10

A_R	DF				DF
	$F_{1\%}$=1.1 β = .3	$F_{1\%}$=1.0 β = .4	$F_{1\%}$=1.0 β = .5	$F_{1\%}$=1.0 β = .6	Eqn (6)
1.5	0.88	0.93	0.95	1.03	1.0
1.75	0.96	0.96	0.91	0.91	1.0
2	1.05	1.03	0.95	0.9	1.04
2.25	1.16	1.11	1	0.93	1.15
2.5	1.27	1.21	1.07	0.97	1.25
2.75	1.38	1.3	1.14	1.03	1.35
3	1.50	1.4	1.22	1.08	1.44
3.25	1.61	1.5	1.3	1.14	1.54
3.5	1.73	1.6	1.38	1.21	1.63
3.75	1.84	1.7	1.46	1.27	1.73
4	1.96	1.8	1.54	1.34	1.82
4.25	2.07	1.9	1.62	1.4	1.91
4.5	2.19	2.01	1.7	1.47	2.0
4.75	2.30	2.11	1.79	1.54	2.09
5	2.42	2.21	1.87	1.6	2.17
5.25	2.54	2.31	1.95	1.67	2.26
5.5	2.65	2.42	2.04	1.74	2.35
5.75	2.77	2.52	2.12	1.8	2.43
6	2.88	2.62	2.2	1.87	2.52

Equation 6 was chosen to provide a generally conservatively biased DF over the range of A_R and β values considered in Table 4. The results for β of 0.4 and 0.5 were weighted more heavily than those for β of 0.3 and 0.6 because the fragility β values are most likely to lie in the 0.4 to 0.5 range and β of 0.3 and 0.6 are considered to be extreme low and high values, respectively. Even so, the entire range of β values was considered. Similarly, A_R values between 1.5 and 4.5 were considered most heavily when developing Equation 6 for DF. Hazard curves with A_R values less than 1.5 have not been seen for the $1x10^{-4}$ to $1x10^{-5}$ range. Also, over this exceedance frequency range, A_R values greater than 4.5 are very unlikely.

In developing Table 4 the seismic hazard curve was approximated by a power law which results in a linear hazard curve when plotted on a log-log plot. Seismic hazard curves are close to linear when plotted on a log-log plot (for example see Figure 1). However, they are not perfectly linear. They always curve downward with decreasing hazard exceedance frequency. Thus A_R reduces as the hazard exceedance frequency is reduced. In other words, an A_R computed over the range of the hazard exceedance frequency from $1x10^{-4}$/yr to $1x10^{-5}$/yr will be larger than that computed over the $1x10^{-5}$/yr to $1x10^{-6}$/yr range. Furthermore, note in Table 4 that the required Design Factor DF increases with increasing A_R. Therefore, one must guard against selecting too low of an A_R value.

Based upon several hundred rigorous convolutions of hazard and fragility curves, it has been found that P_F is dominated by the portion of the fragility curve between about the 1% failure probability capacity $C_{1\%}$ and the 70% failure probability capacity $C_{70\%}$. The 1% failure probability capacity equals or exceeds the SSRS. In turn, the SSRS is given by Equation 1 with DF being always equal or greater than 1.0. Therefore, $C_{1\%}$ will always exceed the $1x10^{-4}$ UHRS.

Similarly, given the capacity conditions defined earlier for β=0.30, the $C_{70\%}$ will be at least:

$$C_{70\%} = 2.58(DF)(UHRS) \qquad \textbf{Equation 26}$$

where DF is given by Equation 6. For higher β, the $C_{70\%}$ will be even higher. Since the $1x10^{-5}$ ground motion is given by A_R (UHRS), it can be seen from Table 1that $C_{70\%}$ will always exceed the $1x10^{-5}$ ground motion.

Therefore, defining A_R over the range of $1x10^{-4}$ /yr to $1x10^{-5}$/yr slightly overestimates A_R for the range of ground motions that dominate P_F. Thus, establishing DF by approximating the hazard curve by a power law with A_R defined by Equation 5 introduces a slight conservative bias. This slight conservative bias will subsequently be illustrated.

7.2 Comparison of the Target Risk Goal, P_{FT}, with the Computed Risk, P_{FC}, Using the DF Defined by Equation 6

7.2.1 Using the Simplified Risk Equation.

The Simplified Risk Equation, Equation 13, was derived assuming the hazard curve can be approximated by Equations 11 and 12. From Equation 13, the computed mean unacceptable performance annual probability P_{FC} can be obtained by recasting Equation 22 to:

$$\left(P_{FC}/H\right) = e^{-f}\left[DF * F_{1\%}\right]^{-K_H} \qquad \textbf{Equation 27}$$

where f is obtained from Equation 23.

Table 5 presents P_{FC} results computed from Equation 27 with the ASCE Standard 43-05 DF defined by Equation 6 and $F_{1\%}$ defined in Section 5.1 for various logarithmic standard deviations β. The conclusion is that with the ASCE Standard 43-05 SSRS defined as described in Section 2 the annual frequency of onset of significant inelastic deformation (FOSID) for an SSC that barely meets the acceptance criteria with no additional margin lies in the range of:

$$\text{FOSID} = \text{mean } 1.2x10^{-5}/yr \text{ to } 0.5x10^{-5}/yr \qquad \textbf{Equation 28}$$

which on average is safely less than the target performance goal and never is higher than 120% of the target goal.

Table 5
Individual SSC Seismic Risk P_{FC} (FOSID) Obtained Using Equation 6 Design Factors

(P_{FC} values shown should be multiplied times $0.1*H_D$)

A_R	P_{FC}			
	$F_{1\%}$=1.1 β = .3	$F_{1\%}$=1.0 β = .4	$F_{1\%}$=1.0 β =.5	$F_{1\%}$=1.0 β = .6
1.5	0.47	0.67	0.76	1.2
1.75	0.82	0.84	0.69	0.68
2	1.03	0.95	0.72	0.61
2.25	1.03	0.92	0.68	0.55
2.5	1.04	0.92	0.68	0.53
2.75	1.06	0.92	0.69	0.54
3	1.08	0.93	0.7	0.55
3.25	1.09	0.95	0.71	0.56
3.5	1.1	0.96	0.73	0.57
3.75	1.12	0.97	0.74	0.59
4	1.13	0.98	0.76	0.6
4.25	1.14	1	0.77	0.61
4.5	1.15	1.01	0.78	0.62
4.75	1.16	1.02	0.79	0.64
5	1.17	1.02	0.81	0.65
5.25	1.17	1.03	0.82	0.66
5.5	1.18	1.04	0.83	0.67
5.75	1.19	1.05	0.83	0.68
6	1.19	1.05	0.84	0.68

This degree of variability in achieved P_{FC} cannot be avoided for any simple criteria that are independent of β because P_{FC} varies by about a factor of two as a function of β. The goal has been to specify DF values that accurately achieve the target performance goal for low variability failure modes (β between 0.3 and 0.4) while accepting increased conservatism for larger variability failure modes (β larger than 0.4) for A_R of 2.0 and greater. For A_R between 1.5 to 2.0, generally conservative bias is introduced.

7.2.2 Using Rigorous Numerical Convolution of Fragility and Actual Hazard Curves

Figure 1 shows some representative normalized hazard curves taken from Figures 7.7 and 7.8 of NUREG/CR-6728 (REI, 2001). These hazard curves are all normalized to unity spectral acceleration at the reference hazard exceedance frequency H = mean $1x10^{-4}$/yr for ease of visualizing the differences in hazard curve slopes. Table 6 presents the tabulated normalized spectral acceleration values SA at 1 Hz and 10 Hz for one Eastern U.S. hazard curve and for the California hazard curve.

Table 6
Typical Normalized Spectral Acceleration Hazard Curve Values

Hazard Exceedance Frequency $H_{(SA)}$	Eastern U.S.		California	
	1 Hz SA	10 Hz SA	1Hz SA	10 Hz SA
5×10^{-2}	0.014	0.018	0.087	0.046
2×10^{-2}	0.027	0.034	0.13	0.072
1×10^{-2}	0.045	0.055	0.175	0.100
5×10^{-3}	0.07	0.089	0.236	0.139
2×10^{-3}	0.143	0.169	0.351	0.215
1×10^{-3}	0.235	0.275	0.474	0.334
5×10^{-4}	0.383	0.424	0.629	0.511
2×10^{-4}	0.681	0.709	0.814	0.762
1×10^{-4}	1.00	1.0	1.0	1.0
5×10^{-5}	1.46	1.41	1.23	1.22
2×10^{-5}	2.35	2.13	1.61	1.51
1×10^{-5}	3.27	2.88	1.89	1.76
5×10^{-6}	4.38	3.65	2.2	2.05
2×10^{-6}	6.44	4.62	2.68	2.42
1×10^{-6}	8.59	5.43	3.1	2.72
5×10^{-7}	10.34	6.38	3.58	3.06
2×10^{-7}	13.21	7.9	4.24	3.56
1×10^{-7}	15.9	9.28	4.67	3.84

The approximate hazard curves used in the simplified risk analysis of Section 7.2.1 are defined by Equations 11 and 12 with A_R defined by Equation 5. These approximate hazard curves would appear as a straight line on the log-log plots of Figure 1 with the amplitude and slope defined by the spectral accelerations at $1x10^{-4}$/yr and $1x10^{-5}$/yr hazard exceedance frequencies. However, all actual seismic hazard curves have a downward curvature similar to those shown in Figure 1 when plotted on log-log plots. The intent of this section is to study the effect of this downward curvature on the P_{FC} computed by rigorous numerical convolution versus the P_{FC} computed in Section 7.2.1 using the simplified risk equation method.

For each of the four normalized hazard curves tabulated in Table 6, Table 7 shows the Amplitude Factor A_R computed by Equation 5, the ASCE Standard 43-05 Design Factor DF computed by Equation 6, and the resulting SSRS spectral accelerations computed by Equation 1. The SSC fragility curves are defined by conservatism factors given in Section 5.1 times the normalized SSRS for each case considered. The actually achieved P_{FC} values computed by rigorous numerical convolution are shown in Table 7. Also shown in parenthesis are the P_{FC} computed using Equation 27 based on the power law hazard curve approximation.

Table 7
Individual SSC Seismic Risks P_{FC} (FOSID) Achieved for Representative Hazard Curves
(Power law approximation of P_{FC} shown in parenthesis)

Hazard Curve	UHRS SA_{UHRS}	A_R	DF	SSRS SA_{SSRS}	SSC Seismic Risk P_{FC} (*10^{-5})			
					$F_{1\%}$=1.1 β = 0.30	$F_{1\%}$=1.0 β = 0.40	$F_{1\%}$=1.0 β = 0.50	$F_{1\%}$=1.0 β = 0.60
EUS 1Hz	1.00	3.27	1.55	1.55	1.09 (1.09	0.93 (0.95)	0.69 (0.71)	0.52 (0.56)
EUS 10 Hz	1.00	2.88	1.40	1.40	1.03 (1.06	0.87 (0.93)	0.62 (0.69)	0.46 (0.54)
Calif 1 Hz	1.00	1.89	1.00	1.00	1.04 (1.03)	0.96 (0.98)	0.73 (0.76)	0.61 (0.68)
Calif 10 Hz	1.00	1.76	1.00	1.00	0.84 (0.84)	0.78 (0.85)	0.58 (0.70)	0.48 (0.67)

One can see that the use of the approximate power law hazard curve introduces a slight, but generally negligible, conservative bias for the computed P_{FC} so long as A_R is defined by Equation 5. Many other comparative examples using other hazard curves have shown similar results.

In summary, it has been shown that using a power law hazard curve with A_R defined by the ratio of the $1x10^{-5}$ to $1x10^{-4}$ spectral accelerations provides a very close (slightly conservative) estimate of P_{FC} as compared to rigorous numerical convolution. Therefore, the use of A_R defined by Equation 5 is justified for defining the Design Factor DF. The FOSID conclusion reached in Section 7.2.1 and presented in Equation 28 remains valid.

7.2.3 Results Obtained for 28 Central and Eastern U.S. Nuclear Power Plant Sites

EPRI (2005) has presented results obtained by the rigorous numerical convolution of fragility and hazard curves for 28 Central and Eastern US (CEUS) nuclear power plant sites. Modern Probabilistic Seismic Hazard Assessments (PSHA) were performed for each of these sites in accordance with EPRI (2004) methodology. SSE SSRS were computed for each site in accordance with the ASCE Standard 43-05 Performance Based FOSID criteria for Seismic Design Category SDC-5D as defined in Section 2. The minimum individual Structure, System or Component (SSC) fragility curves were defined using the minimum "onset of significant inelastic deformation" seismic margin factors defined in Section 5.1 and logarithmic standard deviations β of 0.3, 0.4, 0.5, and 0.6. The annual frequency P_{FC} of "onset of significant inelastic deformation" (FOSID) was computed by numerical convolution of the PSHA hazard curves and minimum fragility curves for spectral accelerations at 5 and 10 Hz. The average of the 5 and 10 Hz results for P_{FC} (FOSID) are reported in EPRI (2005). These results are summarized in Table 8.

Table 8
Individual SSC Seismic Risks P_{FC} (FOSID)
Reported in EPRI (2005) 28 CEUS Site Study

	ASCE Standard 43-05 Method FOSID $*1x10^{-5}$/yr			
β	0.3	0.4	0.5	0.6
Range	0.71-1.17	0.66-0.99	0.51-0.75	0.41-0.58
Median	1.07	0.93	0.69	0.54

All FOSID values computed by rigorous numerical convolution for the 28 sites lie within the FOSID range defined in Equation 28. The highest source of variability is due to the logarithmic standard deviation β of the fragility with results for β=0.3 and 0.4 being close to the target P_{FT}=mean $1x10^{-5}$/yr for FOSID and the β=0.6 results being between about 40 to 60% of the target. Thus, overall, a conservative bias is introduced.

For a given β, very little scatter exists in the computed FOSID. For 26 of the 28 sites, the computed FOSID for a given β are within 10% of the median value. For the other 2 sites, the computed FOSID are more than 10% less than the median value for a given β. Thus, the ASCE Standard 43-05 FOSID Method SSRS achieves its goal of a nearly constant FOSID for an SSC at all sites.

8. Estimation of Seismic Core Damage Frequency (SCDF) When SSRS is Defined by ASCE Standard 43-05 Method

Section 5.2 indicates that for new Standard Plant designs the Seismic Core Damage HCLPF seismic margin factor $F_{1\%}$ is at least 1.67. With the SSRS defined by the ASCE Standard 43-05 for SDC-5D SSCs, it was shown in Section 7 that the FOSID will lie within the range of $0.5x10^{-5}$/yr and $1.2x10^{-5}$/yr. The Seismic Core Damage Frequency (SCDF) will be much less assuming a HCLPF seismic margin $F_{1\%}$=1.67. Table 9 shows the SCDF obtained from numerically convolving hazard curves and lognormal fragility curves. The fragility curves have HCLPF seismic margin $F_{1\%}$=1.67 and logarithmic standard deviations β in the range of 0.3 to 0.6. The four normalized hazard curves are defined in Table 6.

Table 9
Seismic Core Damage Frequency (SCDF) for SSRS Defined by ASCE Standard 43-05 Method and HCLPF Seismic Margin of 1.67

Hazard Curve	SSRS SA_{SSRS}	SCDF ($*10^{-6}$)			
		β=0.30	β=0.40	β=0.50	β=0.60
EUS 1Hz	1.55	4.3	2.9	2.1	1.6
EUS 10 Hz	1.40	3.1	2.0	1.4	1.1
Calif 1 Hz	1.00	1.8	1.2	1.0	0.9
Calif 10 Hz	1.00	1.1	0.8	0.7	0.6

The SCDF values are in the range of $4.3x10^{-6}$/yr to $0.6x10^{-6}$/yr. These SCDF values are in the low range of SCDF values shown in Table 2 for existing plants.

Under these same assumptions, SCDF were also computed in EPRI (2005) for the 28 CEUS sites considered therein. The SCDF results for these 28 sites are summarized in Table 10:

Table 10
Seismic Core Damage Frequency (SCDF) Results Reported in EPRI (2005) 28 CEUS Site Study

	ASCE Standard 43-05 Method SCDF $F_{1\%}$=1.67 $*1x10^{-5}$ /yr			
β	0.3	0.4	0.5	0.6
Range	0.075-0.54	0.060-0.40	0.058-0.29	0.058-0.22
Median	0.38	0.26	0.19	0.15

The ASCE Standard 43-05 FOSID method for defining the SSRS summarized in Section 2 was developed to produce a nearly constant FOSID for a given β independent of the slope of the hazard curve. This ASCE Standard 43-05 FOSID Method does not produce a SCDF that is independent of the slope of the hazard curve for plants with a Seismic Core Damage HCLPF seismic margin of 1.67. The resulting SCDF will be higher for sites with high A_R ratios than for sites with low A_R ratios. For sites with A_R ratios of about 2.0 or less, the SCDF will be in the range of $0.6x10^{-6}$/yr to $2x10^{-6}$/yr. However, for all sites considered, with a HCLPF seismic margin of 1.67 the SCDF is less than $6x10^{-6}$ which is less than 50% of the median SCDF reported for existing nuclear power plants.

The goal of a lower SCDF than the median SCDF reported for existing LWRs is achieved for advanced reactor designs with a HCLPF seismic margin of at least 1.67. On average, the reduction is at least a factor of three.

It should be further noted that a lower bound for the SSE SSRS of a Reg. Guide 1.60 response spectrum anchored to a peak ground acceleration (PGA) of 0.10g is recommended here. For the results presented herein, this lower bound requirement on the SSRS was conservatively ignored because the purpose of the study was to demonstrate the effect of the slope ratio A_R and β on the FOSID and SCDF results. For 13 of the 28 sites studied in EPRI (2005), the seismic hazard was very low so that the SSRS spectral accelerations in the 5 to 10 Hz range were less than a 0.10g Reg. Guide 1.60 spectrum would require. If the SSRS for these 13 sites had been increased to the 0.10g Reg. Guide 1.60 values, the FOSID and SCDF would have been less for these sites than reported herein. Thus, the comparisons shown are conservatively biased because this lower bound SSRS correction was not made.

9. References

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APPENDIX A
DERIVATION OF 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 11, then from Equation 10 one obtains[1]:

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

Equation A.1

in which

$$M = \ell n \quad C_{50}$$ **Equation A.2**

Defining $x = \ell n\, a$, Equation A.1 becomes:

$$P_F = \frac{K_I}{\beta\sqrt{2\pi}} \int_{-\infty}^{+\infty} \left[\exp\left\{ K_H x - \left(\frac{(x - M)^2}{2\beta^2} \right) \right\} \right] dx$$ **Equation A.3**

Many statistical textbooks[1] provide the solution to the definite integral shown in Equation A.3. The result is:

$$P_F = K_I \exp\left\{ -K_H M + \tfrac{1}{2}(K_H \beta)^2 \right\}$$ **Equation A.4**

or from the previous definition of M:

$$P_F = K_I C_{50}^{-K_H} e^{\frac{1}{2}(K_H \beta)^2}$$ **Equation A.5**

Defining H as any reference exceedance frequency, C_H is the ground motion level that corresponds to this reference exceedance frequency H, then from Equation 11:

$$K_I = H[C_H]^{K_H}$$ **Equation A.6**

from which:

$$P_F = H F_{50\%}^{-K_H} e^{\alpha}$$ **Equation A.7**

$$F_{50\%} = \frac{C_{50\%}}{C_H}$$ **Equation A.8**

$$\alpha = \tfrac{1}{2}(K_H \beta)^2$$ **Equation A.9**

[1] Elishakoff, I., Probabilistic Methods in the Theory of Structures, John Wiley & Sons, 1983