From: <eddie.grant@exeloncorp.com>
To: <nvg@nrc.gov>
Date: 11/19/04 3:20PM
Subject: Seismic Primer

Attached is your copy of the letter transmitting the requested seismic approach primer.
This letter is being mailed out today.
Thanks,

Exelon
Early Site Permit Project
Eddie R. Grant
610.765.5001 voice
610.765.5755 fax
850.598.9801 cell

CC: <thomas.mundy@exeloncorp.com>

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52.17

November 19, 2004

U.S. Nuclear Regulatory Commission
ATTN: Document Control Desk
Washington, DC 20555

Subject: Seismic Risk (Performance Goal) Based Approach Primer – Exelon Early Site Permit (ESP) Application for the Clinton ESP Site (TAC No. MC1122)

Re: ASCE Standard 43-05, *Seismic Design Criteria for Structures, Systems, and Components in Nuclear Facilities,* American Society of Civil Engineers, 2005 (in publication)

The subject application presents Exelon Generation Company, LLC's (EGC) seismic information pursuant to 10 CFR § 100.23 in terms of a risk-based approach premised on the referenced industry standard (the "ASCE Method" or "Standard"). While EGC believes that the subject application provides sufficient information in the form of discussion or reference to support the Standard's use, at the request of the Nuclear Regulatory Commission (NRC) staff, during the May 18th and 19th seismic site visit, EGC presented a detailed explanation of the ASCE Method. In addition, at that time, EGC provided the NRC staff with copies of the following three technical publications.

a) Excerpts from the utilized ASCE Standard and Commentary entitled: "Seismic Design Criteria for Structures, Systems, and Components in Nuclear Facilities," dated March 22, 2004,

b) *"Establishing Seismic Design Criteria to Achieve an Acceptable Seismic Margin,"* by Dr. R.P. Kennedy, from Plenary 5, Transaction of the 14th International Conference (SMiRT), Lyon, France, August 1997, and

c) *"Development of Risk-Based Seismic Design Criterion,"* by Dr. R.P. Kennedy, from the Proceedings of the OECD-NEA Workshop on Seismic Risk, Tokyo, Japan, August 1999.

These technical publications provide further explanation of the ASCE Method.

Specifically, the first paper includes Sections 1.2, 1.3, 2.1, and 2.2 and their corresponding Commentary Sections of the Standard. The specific equations used to develop the Exelon ESP SSE response spectrum are given in Section 2.2.1. The basis for these equations is given in Commentary Section C2.2.1. The derivation of the Design Factor is given in Commentary Section C2.2.1.2. The demonstration that the Design Factor approach reasonably achieves the target performance goal is given in Commentary Section C2.2.1.3. Sections 1.2 and 1.3 and their Commentary Sections also provide useful background information.

The second paper provides background information on the general approach used to develop risk-based seismic design criteria. Finally, the third paper provides an example application of this risk-based approach. These latter two papers led to the development of the ASCE Method.

At our meeting of September 16, 2004, the NRC staff commented that while the previous information provided in May is instructive, it would be of greater benefit to the staff if this material were presented in a single compilation. The enclosed paper entitled, "*Risk (Performance-Goal) Based Approach for Establishing the SSE Design Response Spectrum Used in Exelon Generation Company Early Site Permit Application,*" satisfies the NRC staff's request.

We are hopeful that with the presentation of the enclosed paper, the NRC staff is positioned with the information it needs to timely complete its review of the seismic portions of EGC's ESP application.

Please contact Eddie Grant of my staff at 610-765-5001 if you have any questions regarding this submittal.

Sincerely yours,

Marilyn C Kray

Marilyn C. Kray
Vice President, Project Development

TPM/ERG

cc: U.S. NRC Regional Office (w/ enclosures)
Ms. Nanette V. Gilles (w/ enclosures)

Enclosure

AFFIDAVIT OF MARILYN C. KRAY

State of Pennsylvania

County of Chester

The foregoing document was acknowledged before me, in and for the County and State aforesaid, by Marilyn C. Kray, who is Vice President, Project Development, of Exelon Generation Company, LLC. She has affirmed before me that she is duly authorized to execute and file the foregoing document on behalf of Exelon Generation Company, LLC, and that the statements in the document are true to the best of her knowledge and belief.

Acknowledged and affirmed before me this 19th day of November, 2004.

My commission expires 10-6-07.

Vivia V. Gallimore

Notary Public

COMMONWEALTH OF PENNSYLVANIA

Notarial Seal
Vivia V. Gallimore, Notary Public
Kennett Square Boro, Chester County
My Commission Expires Oct. 6, 2007

Member, Pennsylvania Association Of Notaries

October 27, 2004

Risk (Performance- Goal) Based Approach Used in Exelon Generation Company Early Site Permit Application for Establishing the SSE Design Response Spectrum[1]

Prepared
by
R.P. Kennedy[2]

1. Introduction

The Exelon Generation Company, LLC (EGC) Early Site Permit (ESP) application (EGC ESP) for the Clinton site established the Safe Shutdown Earthquake (SSE) Design Response Spectrum (DRS) following the Risk (Performance-Goal) Based Approach defined in ASCE Standard 43-05 (Ref. 1). The standard is a professional consensus committee developed standard. This standard is formally constructed to produce designs that achieve 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 DRS. When these URRSs are used as the DRSs, plants at different sites (all designed to the same design criteria, such as NUREG 0800 for their particular site-specific DRSs) should have consistent seismic risks. In contrast, this risk-consistency goal is not achieved when, as now, a Uniform Hazard Response Spectrum (UHRS) is used as the DRS; 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 to define the DRS, which is used in the EGC ESP and defined in Ref. 1, was first recommended in 1994 in the Commentary of DOE-STD-1020-94 (Ref. 2) for risk-consistent seismic design of High Consequence (PC4) DOE facilities. The detailed basis was given in Ref. 3. Therefore, this approach has been in existence and has been used for about 10 years. Very similar risk-consistent approaches for defining the DRS are presented in Refs. 4 and 5. A more liberal risk-consistent approach for defining the DRS was proposed and studied in NUREG/CR-6728 (Ref. 6). The EGC ESP has chosen to use the ASCE Standard (Ref. 1) approach instead of that in NUREG/CR-6728 because the ASCE Standard definition of the DRS is more conservative and because this Standard is a professional consensus standard.

---

[1] This document was prepared Dr. Robert Kennedy on behalf of the Exelon Generation Company (EGC) as part of the EGC ESP Application. Contents of this report have been reviewed by Dr. Carl Stepp, Earthquake Hazards Solutions, and Dr. Allin Cornell, Stanford University.

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The purpose of this paper is to amplify upon the Commentary of Ref. 1 in explaining the basis and assumptions behind the ASCE Standard approach for defining the risk-consistent DRS used in the EGC ESP. To do so this paper has extracted extensive material from Refs. 1 through 6.

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

Issue #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.

Issue #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.

Issue #3: To maintain the convention of using a UHRS, the DRS will be calculated by:

$$DRS = DF * UHRS \tag{1.1}$$

where UHRS is a "reference" Uniform Hazard Response Spectrum and DF is the Design (Scale) Factor used to define the DRS 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 DRS at a site that will provide risk consistency. But there are clearly many pairs of UHRS levels and DF factors that will produce the same DRS. 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 Eqn. (1.1) will be a function of the Probability Ratio R_P defined by:

$$R_P = \frac{H}{P_{FT}} \tag{1.2}$$

Clearly the larger the value of H the lower the UHRS and the larger DF needs to be to give the unique DRS. 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.

Issue #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 DRS defined by Eqn. (1.1) 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 Standard (Ref. 1) equation for DF is derived and discussed in Section 7 for R_P=10 as used in the EGC ESP.

Lastly, some core damage frequency results are presented in Sections 8.

Before launching into a discussion of the four issues, the ASCE Standard (Ref. 1) criteria used to define the DRS for the EGC ESP will be summarized briefly in Section 2.

2. Summary of ASCE Standard 43-05 Used in EGC ESP for Defining Risk Based Design Response Spectrum DRS

A fundamental assumption in the EGC ESP is that Seismic Category 1 Structures, Systems, and Components (SSCs) will be designed for the DRS utilizing the seismic capacity, seismic demand, and seismic design criteria laid out by the U.S. NRC for nuclear power plants in NUREG-0800 (Ref. 10), 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 Standard (Ref. 1) for the most stringent Seismic Design Category SDC-5D. Therefore, the criteria specified in the ASCE Standard for SDC-5D are used in the EGC ESP to define the DRS.

For SDC-5D, the quantitative target acceptable annual probability of unacceptable performance P_{FT} is[3]:

$$P_{FT} = mean\ 1x10^{-5} / yr \qquad (2.1)$$

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

Thus, the DRS 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[4] 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

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[3] 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.

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

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. 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 defines the DRS by Eqn. (1.1) where the reference UHRS is defined at a reference seismic hazard exceedance frequency H of:

$$H = \text{mean } 1x10^{-4}/\text{yr} \tag{2.2}$$

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:

$$A_R = \frac{SA_{0.1H}}{SA_H} \tag{2.3}$$

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:

$$DF = \text{Maximum } (DF_1 , DF_2) \tag{2.4}$$

where $DF_1 = 1.0$

and $DF_2 = 0.6(A_R)^{0.80}$

which correspond to the appropriate DF_1 and DF_2 from Table 2.2-1 of the ASCE Standard (Ref. 1) for $R_P = 10$ from Eqn. (1.2).

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[5]").

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:

$$P_F = -\int_0^{+\infty} P_F(a)\left(\frac{dH(a)}{da}\right) da \tag{3.1a}$$

[5] For use in the EGC ESP, failure consists of unacceptable FOSID

$$P_F = \int_0^{+\infty} H(a)\left(\frac{dP_F(a)}{da}\right)da \tag{3.1b}$$

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 a. 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 a. (The minus sign is a 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 Fig. 3.1). Thus over any (at least) ten-fold difference in exceedance frequencies such hazard curves may be approximated by a power law:

$$H(a) = K_I a^{-K_H} \tag{3.2}$$

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:

$$K_H = \frac{1}{\log(A_R)} \tag{3.3}$$

in which A_R is the ratio of ground motions corresponding to a ten-fold reduction in exceedance frequency, Eq. 2.3.

So long as the fragility curve $P_F(a)$ is lognormally distributed and the hazard curve is defined by Eqn. (3.2), a rigorous closed-form solution exists for the seismic risk Eqn. (3.1). This closed-form solution is derived in Appendix A as:

$$P_F = H \; M_{50\%}^{-K_H} \; e^{\alpha} \tag{3.4}$$

in which $$M_{50\%} = \frac{C_{50\%}}{C_H} \tag{3.4a}$$

and $$\alpha = \tfrac{1}{2}(K_H\beta)^2 \tag{3.4b}$$

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, and β is the logarithmic standard deviation of the fragility.

Eqn. (3.4) is referred to here as the simplified seismic risk equation. The only approximations in its derivation are that the hazard curve is approximated by Eqn. (3.2) over the exceedance frequency range of interest and the fragility curve is lognormally distributed.

3.3 Design Factor Equation

With the Probability Ratio R_P defined by Eqn. (1.2), Eqn. (3.4) can be rearranged to define the median fragility capacity $C_{50\%}$ required to achieve a desired Probability Ratio R_P:

$$C_{50\%} = C_H \left[R_p e^{-\alpha} \right]^{1/K_H} \quad (3.5)$$

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

$$F_P = \frac{C_P}{DRS} \quad (3.6)$$

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 design ground motion level DRS, those criteria will insure that this C_P fractile is F_P times larger than DRS. Next, defining the DRS by Eqn. (1.1) and recognizing that C_H=UHRS, then:

$$F_P = \frac{C_P}{DF * C_H} \quad (3.7)$$

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

$$C_P = C_{50\%} e^{X_p \beta} \quad (3.8)$$

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.

Combining Eqns. (3.5), (3.7) and 3.8):

$$DF = \frac{\left[R_P e^{-f}\right]^{1/K_H}}{F_P} \tag{3.9}$$

$$\text{in which} \quad f = X_P\left(K_H\beta\right) - \tfrac{1}{2}\left(K_H\beta\right)^2 \tag{3.9a}$$

Eqn. (3.9) 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 RP and DF must be to compensate. But how strongly it depends on R_P depends on K_H, the hazard curve slope (Eqn. (3.3).

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 43-05, 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:

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:

$$DF = \frac{\left[R_P e^{-f}\right]^{1/K_H}}{F_{1\%}} \tag{3.10}$$

$$f = 2.326 K_H\beta - \tfrac{1}{2}(K_H\beta)^2 \tag{3.10a}$$

Eqn. (3.10) will be used in Section 7 to develop the simplified equation for the ASCE Standard Design Factor in Eqn. (2.4) 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 Standard (Ref. 1) SDC-5D SSCs, which was adopted for the EGS ESP, is a mean annual 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 4.1 shows the mean seismic CDF for 25 plants which performed SPRAs using EPRI-type hazard curves as reported in NUREG 1742 (Ref. 12). 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.

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 $4x10^{-6}$/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 in the EGC ESP is that Seismic Category 1 SSCs will be designed for the DRS utilizing the seismic capacity, seismic demand, and seismic design criteria laid out by the U.S. NRC for nuclear power plants in NUREG-0800 (Ref. 10), 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 Standard 43-05 (Ref. 1) for SDC-5D SSCs. This ASCE Standard 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 Standard 43-05 which is reproduced herein in Attachment I.

In computing the required DF for determining the DRS, 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} (Eqn. (3.6)), and an estimate of the capacity variability (i.e., the logarithmic standard deviation β). The two ASCE target levels of conservatism defined above result in the following seismic margin factors $F_{1\%}$, $F_{5\%}$,, $F_{10\%}$, and $F_{50\%}$, corresponding to a 1%, 5%, 10%, and 50% conditional probability of unacceptable behavior, respectively:

β	$F_{1\%}$	$F_{5\%}$	$F_{10\%}$	$F_{50\%}$
0.30	1.10	1.35	1.50	2.20
0.40	1.00	1.31	1.52	2.54
0.50	1.00	1.41	1.69	3.20
0.60	1.00	1.50	1.87	4.04

(5.1)

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:

$F_{1\%} \geq 1.0$

$F_{5\%} \geq 1.3$

$F_{10\%} \geq 1.5$

$F_{50\%}$ 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 Ref. 8 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 has required that a study be performed to show that the Core Damage HCLPF[6] margin factor is at least 1.67 times the DRS. The HCLPF point on the fragility curve computed in accordance with Ref. 9 corresponds to the mean 1% conditional probability of failure point on the Core Damage fragility curve. Thus, for Core Damage:

$$F_{1\%} = 1.67 \tag{5.2}$$

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

Section 8 computes the mean Core Damage Frequency (CDF) when the DRS is defined by the ASCE Standard 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 Standard 43-05 defines the reference mean hazard exceedance frequency H to be:

$$H = \text{mean } 1 \times 10^{-4}/\text{yr} \tag{6.1}$$

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 $1\text{x}10^{-5}$.

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 $2\text{x}10^{-4}$/yr and $2\text{x}10^{-5}$/yr could have been used to achieve P_{FT}= mean $1\text{x}10^{-5}$/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 Design Response Spectrum DRS for any H and R_P

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

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 Eqn. (2.3) 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 Eqn. (2.4)) so that the DRS 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 DRS less than the mean $1x10^{-4}$ UHRS nor one likely to be larger than 1.8 times the mean $1x10^{-4}$ UHRS.

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

The ASCE Standard DF is computed by Eqn. (2.4) 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, Eqn. (3.10), and by comparing how close the failure probabilities implied by use of Eqn. (2.4) are to the target acceptable failure probability. The latter computation will be done two ways, using the analytical approximation (Eqn. (3.4)) and by numerical integration of the exact integrals.

7.1 Computation of Required DF for Comparison with ASCE Standard DF

The required Design Factors DF computed using Eqn. (3.10) 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 7.1 for an Amplitude Ratio A_R range from 1.5 to 6.0. These required DF factors are compared with ASCE Standard DF given by Eqn. (2.4). The ASCE Standard DF Eqn. (2.4) was empirically developed to closely fit these required DF values.

It can be seen that the ASCE Standard DFs given by Eqn. (2.4) are conservatively biased on average. For the practical A_R range from 1.5 to 4.0, these Eqn. (2.4) DF values range between 93% for β=0.3 and 136% for β=0.6 of the required DF. This shows that there is only a moderate sensitivity of DF to the logarithmic standard deviation of the fragility curve. Hence it could, for practical purposes, be dropped from appearing in the ASCE Standard definition.

7.2 Comparison of the Target Risk Goal, P_{FT}, with the Computed Risk, P_{FC}, Using the DF Defined by Eqn. (2.4).

7.2.1 Using the Simplified Risk Equation.

The Simplified Risk Equation, Eqn. (3.4), was derived assuming the hazard curve can be approximated by Eqns. 3.2 and 3.3. From Eqn. (3.4), the computed mean unacceptable performance annual probability P_{FC} can be obtained by recasting Eqn. (3.10) to:

$$\left(P_{FC}/H\right) = e^{-f}\left[DF * F_{1\%}\right]^{-K_H} \tag{7.1}$$

where f is obtained from Eqn. (3.10a).

Table 7.2 presents P_{FC} results computed from Eqn. (7.1) with the ASCE Standard DF defined by Eqn. (2.4) and $F_{1\%}$ defined in Section 5.1 for various logarithmic standard deviations β. The conclusion is that with the ASCE Standard DRS 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:

$$\text{FOSID} = \text{mean } 1.2\text{x}10^{-5}/\text{yr to } 0.5\text{x}10^{-5}/\text{yr} \tag{7.2}$$

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

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).

7.2.2 Using Rigorous Numerical Convolution of Fragility and Actual Hazard Curves

Fig. 3.1 shows some representative normalized hazard curves taken from Figs 7.7 and 7.8 of NUREG-6728 (Ref. 6). These hazard curves are all normalized to unity spectral acceleration at the reference hazard exceedance frequency H = mean $1\text{x}10^{-4}$/yr for ease of visualizing the differences in hazard curve slopes. Table 7.3 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.

The approximate hazard curves used in the simplified risk analysis of Section 7.2.1 are defined by Eqns. (3.2) and (3.3) with A_R defined by Eqn. (2.3). These approximate hazard curves would appear as a straight line on the log-log plots of Fig. 3.1 with the amplitude and slope defined by the spectral accelerations at $1\text{x}10^{-4}$/yr and $1\text{x}10^{-5}$/yr hazard exceedance frequencies. However, all actual seismic hazard curves have

a downward curvature similar to those shown in Fig. 3.1 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 Tables 7.3, Table 7.4 shows the Amplitude Factor A_R computed by Eqn. (2.3), the ASCE Standard Design Factor DF computed by Eqn. (2.4), and the resulting DRS spectral accelerations computed by Eqn. (1.1). The SSC fragility curves are defined by conservatism factors given in Section 5.1 times the normalized DRS for each case considered. The actually achieved P_{FC} values are also shown in Table 7.4.

By comparing the P_{FC} values presented in Table 7.4 with those presented in Table 7.2 for the same A_R and β cases, one can see that the simplified risk equation approach used in Section 7.2.1 for Table 7.2 introduces a slight, but negligible, conservative bias for the computed P_{FC} so long as A_R is defined by Eqn. (2.3) and the extrapolation beyond the range where A_R is defined is not large.

Therefore, the FOSID conclusion reached in Section 7.2.1 and presented in Eqn. (7.2) remains valid.

8. Estimation of Core Damage Frequency (CDF) When DRS is Defined by ASCE Standard Method

Section 5.2 indicates that for new Standard Plant designs the Core Damage HCLPF seismic margin factor $F_{1\%}$ is at least 1.67. With the DRS defined by the ASCE Standard for SDC-5D SSCs, it was shown in Section 7 that the FOSID will lie within the range of 0.5×10^{-5}/yr and 1.2×10^{-5}/yr. The Core Damage Frequency (CDF) will be much less assuming a HCLPF seismic margin $F_{1\%}=1.67$. Table 8.1 shows the CDF 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 7.3.

The CDF values are in the range of 3.5×10^{-6}/yr to 0.6×10^{-6}/yr. These CDF values are in the low range of CDF values shown in Table 4.1 for existing plants.

References

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2. *Natural Phenomena Hazards Design and Evaluation Criteria for Department of Energy Facilities,* DOE-STD-1020-94, U.S. Dept. of Energy, April 1994

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4. Kennedy, R.P., *Establishing Seismic Design Criteria To Achieve an Acceptable Seismic Margin,* Plenary/5, Transactions of the 14th International Conference on Structural Mechanics in Reactor Technology, Lyon, France, August, 1997

5. Kennedy, R.P., *Development of Risk-Based Seismic Design Criterion,* Proceedings of the OECD-NEA Workshop on Seismic Risk, Tokyo, Japan, August 1999

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7. Kennedy, R.P., *Overview of Methods for Seismic PRA and Margin Assessments Including Recent Innovations,* Proceedings of the OECD-NEA Workshop on Seismic Risk, Tokyo, Japan, August 1999

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10. *Standard Review Plan,* NUREG-0800, U.S. NRC

11. ASCE Standard 4-86, *Seismic Analysis of Safety-Related Nuclear Structures and Commentary,* American Society of Civil Engineers, 1986

12. *Perspective Gained From the Individual Plant Examination of External Events (IPEEE) Program,* NUREG-1742, Volumes 1 and 2, U.S. NRC, Sept. 2001

Table 4.1: 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
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

Table 7.1
Design Factor DF Values Required To Achieve A Probability Ratio $R_P = 10$

A_R	DF				DF
	$F_{1\%}$=1.1 $\beta = .3$	$F_{1\%}$=1.0 $\beta = .4$	$F_{1\%}$=1.0 $\beta = .5$	$F_{1\%}$=1.0 $\beta = .6$	Eqn (2.4)
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

- **Recommended Eqn. (2.4) DF Factors Are Conservatively Biased on Average**

Table 7.2:
Individual SSC Seismic Risk P_{FC} (FOSID) Obtained Using Eqn. (2.4) Design Factors
(P_{FC} values shown should be multiplied times $0.1 * H_D$)

A_R	P_{FC}			
	$F_{1\%}=1.1$ $\beta = .3$	$F_{1\%}=1.0$ $\beta = .4$	$F_{1\%}=1.0$ $\beta =.5$	$F_{1\%}=1.0$ $\beta = .6$
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

- **For $H = 1x10^{-4}$ and $R_P=10$, then $P_{FT}=1x10^{-5}$/yr**

  P_{FC} = 1.20 to 0.47 x 10^{-5}/yr

Table 7.3: Typical Normalized Spectral Acceleration Hazard Curve Values

Hazard Exceedance Frequency $H_{(SA)}$	Eastern U.S. 1 Hz SA	Eastern U.S. 10 Hz SA	California 1Hz SA	California 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

Table 7.4: Individual SSC Seismic Risks P_{FC} (FOSID) Achieved for Representative Hazard Curves

Hazard Curve	UHRS SA_{UHRS}	A_R	DF	DRS SA_{DRS}	SSC Seismic Risk P_{FC} ($*10^{-5}$) $F_{1\%}=1.1$ $\beta = 0.30$	$F_{1\%}=1.0$ $\beta = 0.40$	$F_{1\%}=1.0$ $\beta = 0.50$	$F_{1\%}=1.0$ $\beta = 0.60$
EUS 1Hz	1.00	3.27	1.68	1.68	1.09	0.93	0.69	0.52
EUS 10 Hz	1.00	2.88	1.52	1.52	1.03	0.87	0.62	0.46
Calif 1 Hz	1.00	1.89	1.08	1.08	1.04	0.96	0.73	0.61
Calif 10 Hz	1.00	1.76	1.02	1.02	0.84	0.78	0.58	0.48

Table 8.1:
Core Damage Frequency (CDF) for DRS Defined by ASCE Standard 43-5 Method and HCLPF Seismic Margin of 1.67

Hazard Curve	DRS	CDF ($*10^{-6}$)			
	SA_{DRS}	β=0.30	β=0.40	β=0.50	β=0.60
EUS 1Hz	1.68	3.5	2.4	1.7	1.3
EUS 10 Hz	1.52	2.4	1.5	1.0	0.8
Calif 1 Hz	1.08	1.3	0.9	0.7	0.6
Calif 10 Hz	1.02	1.0	0.7	0.6	0.6

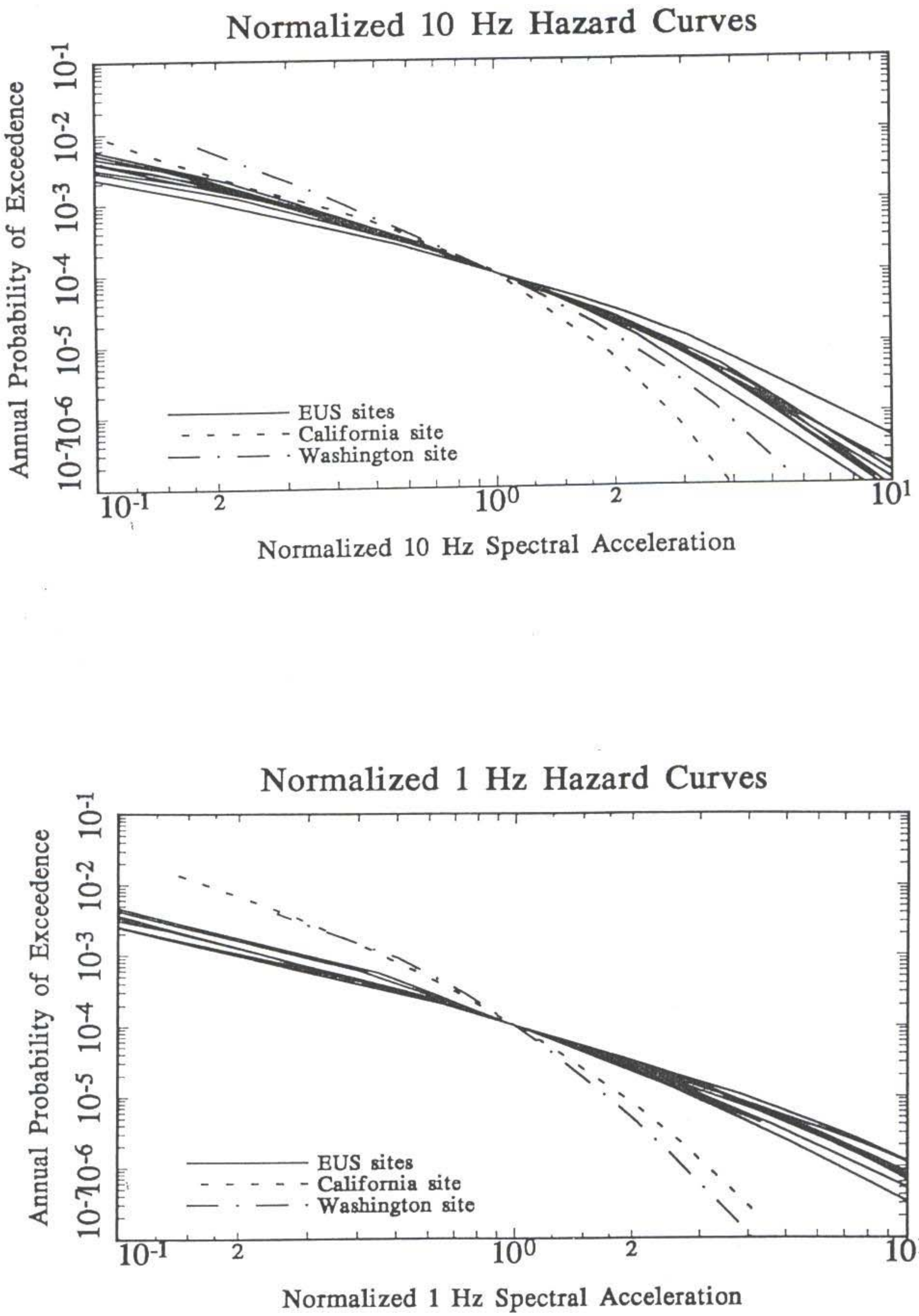

Figure 3.1: SA (10 Hz) and SA (1 Hz) hazard curves for the eleven sites normalized by the acceleration value corresponding to mean 10^{-4} annual probability. (From Figs. 7.7 and 7.8 of Ref. 6)

Appendix A

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 (3.2), then from Equation (3.1b) one obtains:

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

in which $M = \ell n \; C_{50}$

Defining $X = \ell n\, a$, Equation (A.1) becomes:

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

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

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

or from the previous definition of M:

$$P_F = K_I C_{50}^{-K_H} e^{\frac{1}{2}\left(K_H \beta\right)^2} \qquad \text{(A.4)}$$

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. (3.2):

$$K_I = H\left[C_H\right]^{K_H} \qquad \text{(A.5)}$$

from which:

$$P_F = H F_{50\%}^{-K_H} e^{\alpha} \qquad \text{(A.6)}$$

$$F_{50\%} = \frac{C_{50\%}}{C_H} \qquad \text{(A.6a)}$$

$$\alpha = \tfrac{1}{2}\left(K_H \beta\right)^2 \qquad \text{(A.6b)}$$

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

Attachment I

Commentary Section C1.3 of ASCE Standard 43-05 (Ref. 1) (Prepublication)

C1.3 Alternate Methods to Meet Intent of This Standard

The Design Basis Earthquake Ground Motion is defined in terms of a Design Response Spectrum (DRS) defined by Eq. (2.2-1). The Design Factor (DF) used in Eq. (2.2-1) to define the DRS is aimed at achieving the target performance goal annual frequencies defined in Table 2.2-1 so long as the seismic demand and structural capacity evaluations have sufficient conservatism to 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.

Therefore, alternate methods that are aimed at achieving the above specified level of conservatism are acceptable.

The Standard is based on achieving both probability goals, which represent two points on the underlying fragility curve. Having these two probability goals allows the target probabilities to be achieved with less possibility of unconservatism. The work required to demonstrate that both goals are achieved when alternate methods are used is only slightly greater than showing that one of the two goals is achieved.

Seismic fragility (conditional probabilities of failure versus ground motion levels) is typically defined as being lognormally distributed so that it can be fully described in terms of a seismic margin factor F_{PF} (factor applied to the DBE ground motion) corresponding to a conditional failure probability P_{FC}, and an estimate of the failure variability (logarithmic standard deviation β). The two target levels of conservatism defined above result in the following seismic margin factors $F_{1\%}$, $F_{5\%}$,, $F_{10\%}$, and $F_{50\%}$, corresponding to a 1%, 5%, 10%, and 50% conditional probability of failure (unacceptable behavior), respectively:

β	$F_{1\%}$	$F_{5\%}$	$F_{10\%}$	$F_{50\%}$
0.30	1.10	1.35	1.50	2.20
0.40	1.00	1.31	1.52	2.54
0.50	1.00	1.41	1.69	3.20
0.60	1.00	1.50	1.87	4.04

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

$$F_{1\%} \geq 1.0$$

$$F_{5\%} \geq 1.3$$

$F_{10\%}$ $\geq$ 1.5

$F_{50\%}$ increases with increasing β

These minimum margins are sufficient to reasonably achieve the target performance goals as is shown below.

C1.3.1 Expected Factors of Safety Achieved by Seismic Acceptance Criteria

C1.3.1.1 Introduction

In this standard, strengths are specified in terms of the ACI code ultimate strengths, the AISC code LRFD limit state strengths including the code specified strength reduction factors (ϕ), and the ASME code service level D strengths. The seismic demand is specified in terms of ASCE 4 requirements. For ductile failure modes, appropriately conservative inelastic energy absorption factors F_μ are specified within this standard in Table 5.1-1.

In this section, the resulting strength, seismic demand, and nonlinear factors of conservatism are first estimated and then combined to obtain an overall estimate of the factor of safety achieved by the seismic acceptance criteria specified in this standard.

C1.3.1.2 Estimation of Median Conservatism Introduced By Standard Seismic Acceptance Criteria

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

$$C_{50\%} = \frac{S_{50\%}}{D_{50\%}} F_{\mu_{50\%}} DBE \qquad \text{(Eq. C1.3-1)}$$

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

$$C_{STD.} = \frac{S_{Std.}}{D_{Std.}} F_{\mu Std} DBE \qquad \text{(Eq. C1.3-2)}$$

where $S_{STD.}$ $D_{STD.}$, and $F_{\mu_{Std.}}$ are the deterministic strength, demand, and nonlinear factors defined in accordance with this standard. Defining R_S, R_D, and R_N as the median conservatism ratios associated with this standard, then:

$$S_{50\%} = R_S S_{STD}$$

$$D_{50\%} = D_{STD}/R_D \qquad \text{(Eq. C1.3-3)}$$

$$F_{\mu_{50\%}} = R_N F_{\mu Std}$$

and

$$C_{50\%} = R_C C_{STD.} \qquad \text{(Eq. C1.3-4)}$$

$$R_C = R_S R_D R_N \qquad \text{(Eq. C1.3-5)}$$

where R_C is the overall median conservatism ratio associated with this standard's acceptance criteria. The ratios R_S, R_D, and R_N will be estimated in the following three subsections.

C1.3.1.2.1 Median Strength Conservatism Ratio

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 low ductility failure modes, an additional factor of conservatism of about 1.33 is typically introduced.

Thus:

$$\text{(Ductile)} \qquad R_S = e^{2.054\beta_S}$$

$$\text{(Low Ductility)} \qquad R_S = 1.33e^{2.054\beta_S} \qquad \text{(Eq. C1.3-6)}$$

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.

C1.3.1.2.2 Median Demand Conservatism Ratio

Seismic demands are computed in accordance with the requirements of ASCE 4 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 preface, ASCE 4 is aimed at achieving about a 10% probability of the actual seismic response exceeding the computed

response, given the occurrence of the DBE. Thus the median demand ratio R_D can be estimated from:

$$R_D = \frac{e^{1.282\beta_D}}{1.22} \qquad \text{(Eq. C1.3-7)}$$

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

C1.3.1.2.3 Median Nonlinear Conservatism Ratio

In this standard, the nonlinear factor is aimed at about the 5% NEP level. Thus for ductile failure modes, the median nonlinear factor ratio R_N should be:

Ductile $$R_N = e^{1.645\beta_N} \qquad \text{(Eq. C1.3-8a))}$$

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.:

Brittle $$F_{\mu 50\%} = 1.0$$ $$R_N \approx 1.0 \qquad \text{(Eq. C1.3-8b)}$$

C1.3.1.3 Resulting Capacity Conservatism

Combining Eqs. (C1.3-5) through (C1.3-8) the median capacity ratio R_C is estimated to be:

(Ductile Failures) $$R_C = 0.82e^{2.054\beta_S+1.282\beta_D+1.645\beta_N}$$
(Low Ductility) $$R_C = 1.09e^{2.054\beta_S+1.282\beta_D} \qquad \text{(Eq. C1.3-9)}$$

and:

$$C_{1\%} = R_C\, C_{STD.}\, e^{-2.326\beta} \qquad \text{(Eq. C1.3-10)}$$

$$\beta = \left[\beta_S^2 + \beta_D^2 + \beta_N^2\right]^{1/2} \qquad \text{(Eq. C1.3-11)}$$

The resulting nominal factor of safety $F_{N1\%}$ against a 1% conditional probability of failure is then given by:

$$F_{N1\%} = \frac{C_{1\%}}{C_{Std.}} = R_C e^{-2.326\beta} \qquad \text{(Eq. C1.3-12a)}$$

Similarly, the nominal factor of safety $F_{N10\%}$ against a 10% conditional probability of failure is given by:

$$F_{N10\%} = \frac{C_{10\%}}{C_{Std.}} = R_C e^{-1.282\beta} \qquad \text{(Eq. C1.3-12b)}$$

Table C1.3-1 presents $F_{N1\%}$ for typical values of β_S, β_D, and β_N. It can be seen that over this entire range of β values:

$$F_{N1\%} \approx 1.0 \qquad \text{(Eq. C1.3-13)}$$

with $F_{N1\%}$ ranging from 0.93 to 1.20 with a median value of 1.07. Table C1.3-2 presents $F_{N10\%}$ for typical values of β_S, β_D, and β_N. It can be seen also that:

$$F_{N10\%} \approx 1.5 \qquad \text{(Eq. C1.3-14)}$$

Thus, both Eqs. (C1.3-13) and (C1.3-14) are satisfied by the seismic acceptance criteria presented in this standard.

Table C1.3-1 Nominal Factor of Safety $F_{N1\%}$

Strength Variability β_S	Demand Variability β_D	Low Ductility Failure Modes	Ductile Failure Modes	
			β_N=0.2	β_N=0.4
0.2	0.2	1.10	0.99	0.99
	0.3	1.04	0.97	1.00
	0.4	0.97	0.92	0.99
0.3	0.2	1.12	1.04	1.08
	0.3	1.11	1.04	1.11
	0.4	1.05	1.00	1.10
0.4	0.2	1.13	1.07	1.15
	0.3	1.13	1.08	1.19
	0.4	1.11	1.07	1.20

Table C1.3-2 Nominal Factor of Safety $F_{N10\%}$

Strength Variability β_S	Demand Variability β_D	Low Ductility Failure Modes	Ductile Failure Modes	
			β_N=0.2	β_N=0.4
0.2	0.2	1.48	1.42	1.64
	0.3	1.52	1.49	1.76
	0.4	1.54	1.53	1.84
0.3	0.2	1.64	1.60	1.89
	0.3	1.72	1.69	2.03
	0.4	1.77	1.76	2.15
0.4	0.2	1.80	1.78	2.15
	0.3	1.91	1.90	2.32
	0.4	2.00	2.00	2.46