Guidance for Determination of Performance-Based (Risk-Informed) Site Specific Safe Shutdown Earthquake Response Spectra for Future Nuclear Plants

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PRODUCT DESCRIPTION

This report describes guidance for determination of performance goal-based (risk-informed) site-specific safe shutdown earthquake response spectra (SSRS) for future nuclear plant sites. The guidance includes recommendations for generic updating of the earthquake recurrence and ground motion elements of the CEUS probabilistic seismic hazard model. The generic update of the earthquake recurrence model implements the CAV filter to eliminate small earthquakes with negligible potential for damage to nuclear facilities from the hazard analysis. The generic update of the ground motion model element implements the EPRI 04 Ground Motion Model for the CEUS. Guidance for determining SSRS implements ASCE Standard 43-5 methodology; a tutorial for implementation of the methodology is provided as Appendix A to the report. Guidance for determining the response of local site geology to seismic waves is provided, implementing Approach 2A/3 recommended in NUREG/CR-6769. Guidance is provided for determining site-specific risk-informed design response spectra (DRS) consistent with the SSRS and for determining the control point location for defining the SSRS and DRS based on site-specific rock and soil properties.

Results and Findings
Implementation of the performance goal-based (risk-informed) method for determining SSRS is intended to implement the Nuclear Regulatory Commission's Rick-Informed Regulation Policy for seismic regulation. The method replaces the "reference probability" criterion for determining site specific SSE ground motion contained in NRC Reg Guide 1.165. Application of the performance goal-based method will insure essentially seismic risk across future nuclear unites that is lower than the mean risk for the population of existing nuclear plants.

Challenges and Objectives
This report will be of interest to nuclear utilities that plan to obtain an Early Site Permit (ESP) or a Combined Operating License (COL) for new nuclear units located in the central and eastern United States. The objectives of the report are to provide methods and procedures: 1) for updating the CEUS probabilistic seismic hazard model with current technologies, 2) for updating certain methods and procedures for determining SSE Ground Motion that are contained in USNRC Reg. Guide 1.165 and in SRP Section 2.5.2, and 3) for updating methods and procedures for evaluating site response to seismic waves based on recent research results to support updating of seismic regulatory guidance contained in SRP Sections 3.7.1 and 3.7.2. There are no significant technical challenges to using these methods.

Applications, Values, and Use

The methods and procedures described in this report should be used for determination of performance goal-based (risk-informed) site-specific safe shutdown earthquake response spectra (SSRS) and design response spectra (DRS) to support utility's ESP or COL applications. The main value of the methods and procedures is that their use implements performance-goal based, risk-informed seismic designs that assure a uniform level of seismic safety across nuclear unites and plant site locations.

EPRI Perspective

EPRI has an industry-wide perspective and a mandate to address broad issues related to safe design and safe and efficient operation of nuclear facilities. The methods and procedures described in this report contribute to stabilizing seismic safety review and licensing procedures for new nuclear units and to implementation of the U. S. Nuclear Regulatory Commission's Risk-Informed Regulation Policy.

Approach

The goal of this report is to provide up-to-date technical guidance for utilities to use for preparing the seismic component of ESP or COL applications and for interactions between the nuclear utility industry aimed at updating the current seismic regulatory guidance with state-of-practice technologies.

Keywords

Risk-informed seismic evaluation

Probabilistic seismic hazard analysis

Performance-based seismic analysis

Seismic risk assessment.

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1 INTRODUCTION

Reg. Guide 1.165 (USNRC 1997a) contains guidance and describes technical approaches and procedures acceptable to the USNRC for satisfying the requirements of the geologic and seismic siting regulation, 10 CFR Part 100.23 (USNRC 1997b). The guidance accepts either the EPRI (EPRI 1989a, 1989b) or the LLNL (Bernreuter, et al. 1989) CEUS probabilistic seismic hazard model as the starting basis for determining site-specific SSE Ground Motion for new nuclear units at a site, provided that the probabilistic seismic hazard model used is shown to remain valid considering updated basic information/data for the site region. The Reg. Guide provides specific guidance for: 1) the scope of geological, geophysical, and seismological data compilation required to update the basic data for a site region, 2) the required geotechnical investigations for a site, 3) the procedures for assessing whether or not new information results in a significant increase in the hazard for a site, and 4) the procedures for determination of the required site-specific SSE Ground Motion.

Although the technologies upon which the Reg. Guide 1.165 guidance is based date from the late 1980s and early 1990s, much of the guidance remains current, requiring updating only in detail. There are two main exceptions. Significant advances have been made since the early 1990s in the area of modeling ground motion for the CEUS (EPRI 1993, McGuire, et al. 2002a and 2002b, McCann, et al. 2004, Abrahamson and Bommer 2005) and in development of performance goal-based (risk-informed) procedures for determining site-specific response spectra (ASCE 2005, USNRC 2001, McGuire, et al. 2002a and 2002b; McGuire 2005a and 2005b). Technical tasks performed as part of the New Plant Seismic Issues Resolution Program (NPSIRP) have demonstrated the importance of updating Reg. Guide 1.165 with performance-based (risk-informed) procedures in order to achieve risk consistency of new nuclear units with existing operating nuclear units and, importantly, to achieve regulatory stability and the assurance of consistent seismic safety across future nuclear units (McGuire 2005a and 2005b). Other technical advances demonstrate the importance of updating the probabilistic seismic hazard model for the CEUS with new lower bound magnitude guidance and with an updated CEUS ground motion model (Abrahamson, et al. 2005, McCann, et al. 2004). The importance of implementing performance-based (risk-informed) procedures is supported by the results of individual plant examination of external events (USNRC 2001). The results of comprehensive research supported by the USNRC have demonstrated the importance of providing updated guidance for determination of site-specific seismic design response spectra (McGuire 2002a and 2002b).

An important advancement in nuclear plant regulation was initiated with the formalized NRC commitment to implement risk-informed, performance-based regulation. The Commission's PRA Policy Statement states, in part, ***"The use of PRA technology should be increased in all regulatory matters to the extent supported by the state of the art in PRA methods and data, …."*** (USNRC 1995). SECY-98-144 provides a framework for implementation of the

Commission's policy by elaborating the concepts involved, by defining terms, and by describing the NRC's expectations for risk-informed, performance-based regulation implementation (USNRC 1998). Among the terms defined in SECY-98-144, "Risk-informed, Performance-based", is most directly relevant for updating the seismic regulatory guidance.

> ***"A risk-informed, performance-based approach to regulatory decision-making combines the "risk-informed" and "performance-based" elements …, and applies these concepts to NRC rulemaking, licensing, inspection, assessment, enforcement, and other decision-making. Stated succinctly, risk-informed, performance-based regulation is an approach in which risk insights, engineering analysis and judgment, and performance history are used, to (1) focus attention on the most important activities, (2) establish objective criteria based upon risk insights for evaluating performance, (3) develop measurable or calculable parameters for monitoring system and licensee performance, and (4) focus on the results as the primary basis of regulatory decision-making."***

The realization of this important advancement for seismic regulation requires updating of the seismic regulatory guidance.

This document contains guidance for implementing the ASCE Standard 43-05 Method to determine performance goal-based (risk-informed) site-specific response spectra. Details of the method are described in Appendix A. The performance goal-based (risk-informed) procedures directly respond to the Commission's expectations for risk-informed, performance-based regulation implementation. We adopt the terminology performance goal-based (risk-informed) site-specific response spectrum (SSRS) in order to emphasize that the ASCE Standard 43-05 method is performance-goal based. It is risk-informed by comparison with the results for the subset of 28 plants that have the most modern seismic designs and with core damage frequency results for the 25 nuclear plants for which seismic PRAs have been performed and reported in NUREG-1742 (See Appendix A, McGuire 2005a, USNRC 2001).

The purpose of this document then is to provide guidance for the determination of site-specific performance goal-based (risk-informed) SSRS starting with the site-specific SSE ground motion spectrum and implementing current technologies. Section 4 of the document describes the interface between the site-specific performance goal-based (risk-informed) SSRS and the development of the site-specific seismic design response spectrum (SDRS).

2

GENERIC UPDATING OF COMPONENTS OF THE CENTRAL AND EASTERN UNITED STATES (CEUS) PROBABILISTIC SEISMIC HAZARD MODEL

Technical tasks performed as part of the New Plant Seismic Issues Resolution Program (NPSIRP) developed technical bases for generic updating of two components of the CEUS probabilistic seismic hazard model: the lower bound magnitude for the earthquake recurrence model component and the ground motion model component (McCann et al. 2004; Abrahamson, et al. 2005; Abrahamson, et al. 2006). This section draws heavily from these reports and from McGuire, 2006, and references the reports for additional guidance.

2.1 Lower Bound Earthquake Magnitude for the Earthquake Recurrence Component of the CEUS Probabilistic Seismic Hazard Model: The CAV Filter

2.1.1 Motivation for Developing the CAV Filter

Probabilistic seismic hazard analysis (PSHA) for a site integrates over all components of the probabilistic seismic hazard model for the site region to obtain the site-specific ground motion hazard due to potentially damaging earthquakes occurrences throughout the site region. Recent PSHA practice has considered potentially damaging earthquakes to be those that have magnitudes larger than a conservatively determined lower bound earthquake magnitude. Earthquakes larger than the lower bound magnitude are considered to be potentially damaging and earthquakes smaller than the lower bound magnitude are considered to have no potential for causing damage. The threshold magnitude between non-damaging and potentially damaging earthquakes was conservatively established in the mid 1980s as a body wave magnitude value of 5.0 (approximate moment magnitude of 4.6). This lower bound magnitude cut-off level is a conservatively defined value based on several EPRI studies whose objective was to estimate the damage potential of small earthquakes and on consensus of the earthquake engineering community at the time.

Subsequent experience with PSHA application has shown that a single magnitude value between potentially damaging and non-damaging earthquakes is not realistic or appropriate for PSHA calculations. Since the mid 1980s the large increase in the number of earthquake recordings has lead to the understanding that there is significant variability of strong-ground motion for a given earthquake magnitude value and therefore, in the potential for an earthquake of a given magnitude to cause damage. Abrahamson, et al. 2005 and 2006 have developed an approach that uses CAV (Abrahamson, et al. 2006, eq. 1-2) to filter earthquakes that have negligible potential for causing damage and demonstrate how the method can be applied in the PSHA computation integral.

The CAV was established as a measure of the potential for a ground motion recording to cause damage by EPRI studies conducted in the late 1980s and early 1990s (Reed and Kennedy, 1988; EPRI 1991). These studies established the threshold value for potential damage to be 0.16 g-sec and this value was adopted as the threshold for OBE exceedance, for the potential to cause damage to nuclear facilities, for the purpose of implementing Reg. Guide 1.166. Abrahamson, et al. 2006, Appendix A contains a summary of these studies supporting the technical basis for establishing the CAV damage threshold at 0.16 g-sec.

2.1.2 Methodology for Application of Minimum CAV in Seismic Hazard Analysis (Abrahamson, et al. 2006, Chapter 4)

The study performed by Abrahamson, et al. 2006 developed the technical basis (the CAV filter) for establishing the appropriate distribution of low magnitude earthquakes for use in PSHAs for nuclear power plants based on the established CAV threshold damage value of 0.16 g-sec. The study developed two alternative approaches for implementing the CAV filter as part of the hazard computation. The most direct approach is to apply the CAV filter directly within the hazard calculation (i.e., inside the hazard integration) by adding an integral over the PGA aleatory variability. This application is implemented by Equations 4-1, 4-2, and 4-3, from Abrahamson, et al.2006. This most direct approach is recommended for performing PSHAs for future nuclear plant sites, as it is implemented by an additional integral in the hazard computation.

For large hazard calculations the additional integral in the hazard integration may add significantly to the computation time. A more computationally efficient method, which does not require the additional integral inside the hazard computation, is to apply the CAV filtering as part of the post processing of the hazard calculation. Implementation of this approach is described in Abrahamson, et, al. 2006, Chapter 4, and is implemented by Equations 4-4 through 4-6.

2.1.3 Sensitivity of Ground Motion Hazard to the CAV Filter

As stated above, the most direct, and therefore the preferred, approach for applying the CAV filter to filter contributions of small earthquakes that have negligible potential to cause damage form the hazard results is directly within the hazard calculation by adding an integral over the PGA aleatory variability. McGuire 2006 applied this approach and re-computed hazard results reported in McGuire 2005a and 2005b for 28 nuclear plant sites in the CEUS. These 28 sites participated in the EPRI-SOG CEUS PSHA study (EPRI 1989a); the sites represent all except one of the 29 nuclear plant sites upon which the Reg. Guide 1.165 reference hazard probability criterion was established. These sites are the locations of nuclear units that have the most modern seismic design (Reg. Guide 1.60 spectrum shape, and modern NUREG-0800 seismic design criteria and procedures). They are geographically distributed over the region of the

CEUS such that in composite, the sites represent the variation in the seismic hazard environment for the CEUS (see McGuire 2006, Figure 1-1). The results of the sensitivity study are reported in McGuire 2006, Chapter 3.

The sensitivity results show that the effect of applying the CAV filter ranges from negligible to major depending on structural frequency, the seismic design response spectrum level, and the seismic hazard environment of the site. The effects for the 28 sites are categorized as small (less than about 5% reduction in spectral amplitudes over all structural frequencies), moderate (between 5% and 15% reduction), and major (> 15%). Sites where the CAV filter has small effect are those where the ground motion hazard is dominated by seismic source characterizations for either the Charleston Seismic Zone or the New Madrid Seismic Zone, both of which are the locations of frequent large magnitude earthquakes. Sites where application of the CAV filter has moderate effect are those located at distances from either the Charleston or New Madrid seismic sources such that these sources contribute to the ground motion hazard, but are not the dominant contributors. Sites where the CAV filter has a major effect are those at locations where neither the Charleston nor the New Madrid seismic sources significantly contribute to the ground motion hazard. Ground motion hazard at these locations is generally dominated by moderate magnitude earthquakes and for many, low rates of earthquake occurrence.

McGuire (2006) Table 3-1 summarizes the effect of applying the CAV filter for the range of nuclear plant structural frequencies. Figures 2-1 and 2-2 compare spectra for representative sites where the effect of the CAV filter is small and major, respectively. (Note that the term ASCE-DRS in these figures corresponds to the term performance goal-based (risk-informed) site-specific response spectrum (SSRS) that is that is preferred and used throughout this report). These figures show: 1) Reg. Guide 1.60 scaled to 0.3g at 33 Hz, the standard seismic design response spectrum adopted for seismic design of advanced reactors, the standard plant seismic design response spectrum (SDRS), 2) the SSRS for the site determined using the EPRI 04 Ground Motion Model (base case), 3) the SSRS for the site determined using the EPRI 04 Ground Motion Model modified with the updated sigma from Abrahamson and Bommer (2006) (labeled "revised sigmas"), which is discussed in Section 2.2.5 of this report, and 4) the SSRS for the site determined using the revised sigmas and the CAV filter (labeled "revised sigmas and CAV").

Incorporation of the CAV filter to eliminate small earthquakes that have negligible damage potential from the hazard calculation has little effect on the SSRS for sites where the hazard is

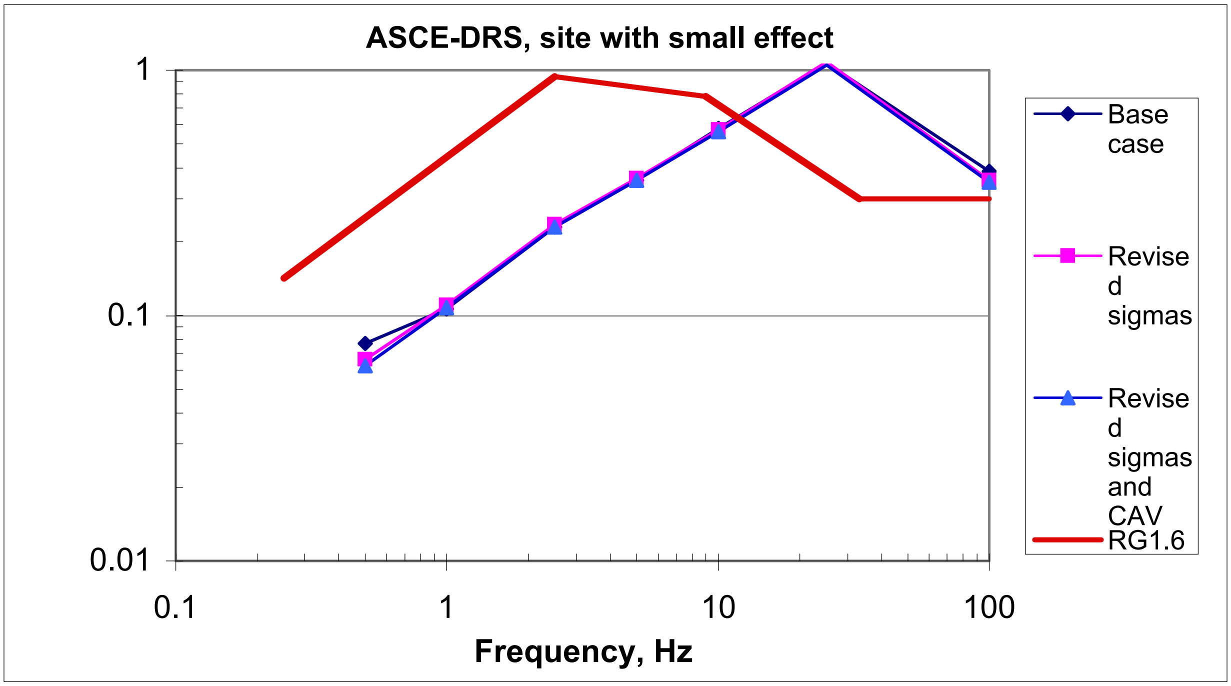

Figure 2-1 Representative site where application of the CAV filter has small effect (McGuire 2006, Figure 4-1)

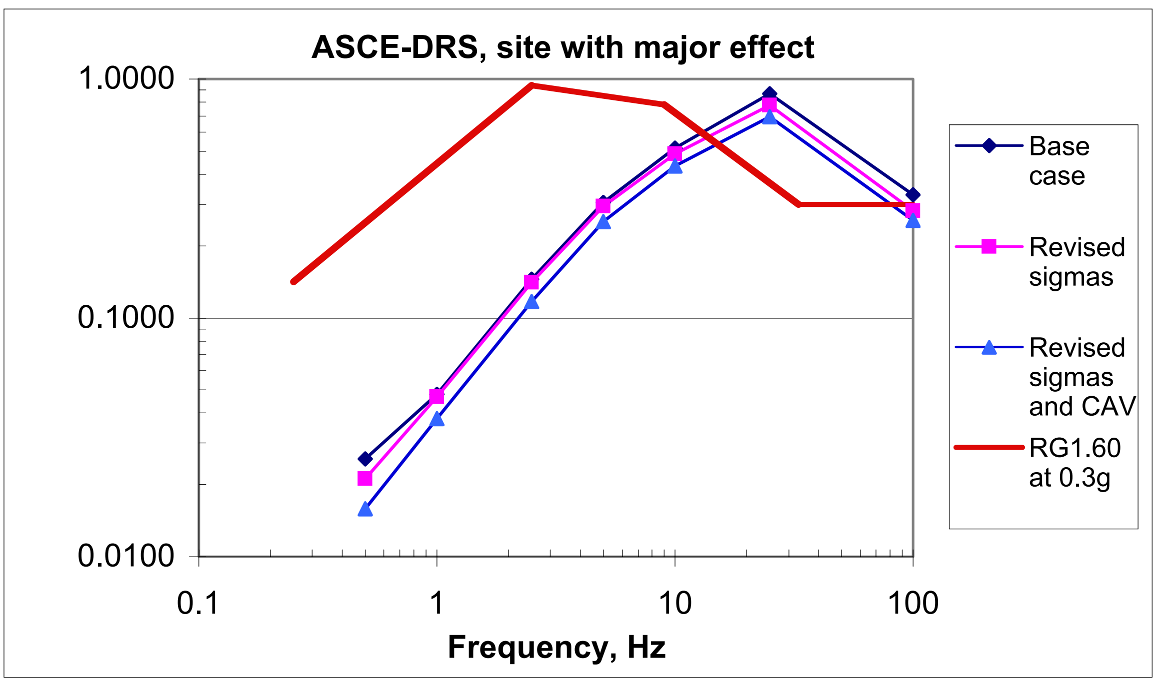

Figure 2-2 Representative site where application of the CAV filter has major effect (McGuire 2006, Figure 4-3)

dominated by frequent large earthquakes, i.e., the Charleston or New Madrid seismic source characterizations. The dominant contribution of large earthquakes to the hazard at these sites means that the CAV filter has little effect. For sites located in regions of the CEUS (generally the Southwest, upper Midwest, central Atlantic states, or New England) implementation of the CAV has major effect across important structural frequencies, as shown in Figure 2-2.

2.2 Updated CEUS Ground Motion Model

The CEUS Ground Motion Project conducted by EPRI for the nuclear utility industry (McCann, et al. 2004) developed a contemporary composite ground motion model for the CEUS, the EPRI 04 Ground Motion Model. The model development was accomplished by implementation of a SSHAC Level 3 expert elicitation process (Budnitz, et al. 1997). The SSHAC Level 3 process and details of its implementation for development of the EPRI 04 Ground Motion model are well described in Sections 1.2 and 2.2 of McCann, et al, 2004. The EPRI 04 Ground Motion model was developed to support early site permit applications for sites located in the CEUS. This model is recommended for generic updating of the ground motion modeling element of the CEUS probabilistic seismic hazard model for licensing of future nuclear facilities located in the CEUS.

2.2.1 EPRI 04 Ground Motion Model: Motivation and Scope (after McCann, et al. 2004)

PSHA results show that uncertainty in ground motion prediction is the largest contributor to the total uncertainty in ground motion hazard. Ground motion models by necessity represent the complex physical phenomena of ground motion generation and propagation from source to a recording location with a few variables. The consequence is a high level of uncertainty about the value of ground motion predicted using any single proponent ground motion prediction model and different proponent models give different results. Thus it is necessary to develop a composite ground motion prediction model that incorporates the uncertainty in estimation of median ground motion and the variability about the median in order to provide a stable basis for nuclear plant seismic regulation for a period of time into the future. The EPRI 04 Ground Motion Model was developed to meet this critical need.

Because ground motion prediction models continue to evolve as new data become available, ground motion modeling is a particularly active area of research. Extensive earthquake ground motion modeling developments supporting the need to develop a composite model for the region of the CEUS have occurred since 1989. These developments include:

• publication of the SSHAC study which provides guidance for the assessment of epistemic and aleatory uncertainty in ground motion modeling and a trial application of the guidance (Budnitz, et al 1997),

• limited additional strong-ground motion data recorded in the eastern North America,

• development of a number of additional proponent ground motion models incorporating more recent representations of earthquake source models and models of the propagation of seismic waves in the CEUS (see McCann, et al. 2004 for a description of proponent models), and

• expanded modeling of aleatory and epistemic uncertainties in ground motion modeling based on first principles (EPRI 1993) and on application of the SSHAC assessment procedures (Stepp, et al. 1998).

The scope of these developments motivated the development of a composite CEUS ground motion model using formal SSHAC assessment procedures to perform comprehensive assessments of both epistemic and aleatory uncertainties. The assessment was based on existing proponent ground motion prediction models as opposed to the development of a new model based on first principles (e.g., EPRI 1993) or on weighted assessments of multiple ground motion experts (e.g., Stepp, et al. 1998).

2.2.2 Features of the EPRI 04 Ground Motion Model

The EPRI 04 Ground Motion model is an engineering model for estimating earthquake ground motions for use in PSHA calculations for sites located in the CEUS. The composite model is based on existing proponent ground motion prediction models. It includes an assessment of epistemic uncertainties in the median ground motion determined using evaluation criteria that placed strong emphasis on the consistency of proponent models with available strong-motion data and model parameterization, e.g., consistency with seismological principles and degree of consideration of uncertainty in the proponent model. The SSHAC Level 3 evaluation process grouped the proponent models by model type into four groups, called Clusters, based on how seismic source and details of wave propagation are modeled: 1) spectral single corner source (six proponent models), 2) spectral double corner source (three proponent models), 3) hybrid (three proponent models), and 4) finite source/Greens function (one proponent model) (see McCann, et al. 2004, Section 3.4.1 for a complete discussion of the proponent model Clusters). The evaluation processes developed assessment criteria and assessed weights for each proponent model and for each proponent model Cluster, and assessed epistemic uncertainty in each Cluster median (McCann, et al. 2004, Figure 3-7). Aleatory variability was assessed using four proponent models that had included assessments of uncertainty (McCann et al. 2004, Figure 3-7).

Basic features and attributes of the EPRI 04 Ground Motion Model are described in Section 5.1 of McCann, et al. 2004. The features and attributes are summarized in the following table (Table 6-1 from McCann, et al. 2004, with modification).

Table 2-1 Basic features and attributes of the EPRI 04 Ground Motion

Model (Table 6-1 from McCann, et al. 2004, with modification)

Features	Attributes
Model General Form: $\ln(y) = \mu(\mathbf{M}, r) + \varepsilon_r$	y = Ground Motion Measure $\mu(\mathbf{M}, r)$ = function describing variation of median ground motion with magnitude and site-to-source distance ε = standard normal random variable with zero mean and logarithmic standard deviation, σ, that quantifies aleatory variability in ground motion
Ground Motion Measures	Peak Ground Acceleration Spectral Acceleration (S_a) at frequencies of 0.5, 1, 2.5, 5, 10, and 25 Hz
CEUS Generic Rock Conditions	CEUS generic rock, i.e., CEUS crustal model rock: defined by Vs = 2.8km/sec and shallow crustal damping parameter, kappa, equal to 0.006 sec.
Earthquake Magnitude Scale	Moment magnitude, **M**
Proponent Ground Motion Model Types included in the assessment of epistemic uncertainty in median estimated ground motion (see McCann, et al. 2004, Section 3.4.1)	• Single-corner source spectral models • Double-corner source spectral models • Hybrid • Elasto-dynamic dislocation and wave propagation model
Aleatory Variability based on four alternative aleatory models	Includes: magnitude, distance and frequency dependence of the aleatory variability parameter; sigma and epistemic uncertainty in sigma
Site-to-Source Distance Measures	• Joyner-Boore distance • Closest distance to fault rupture • Point source (McCann, et al. 2004, Chapter 5 and Appendix F)
CEUS Sub-regions	Mid-Continent; Gulf Coast (Figure 3-2, McCann, et al. 2004)

2.2.3 EPRI 04 Ground Motion Model Implementation

The EPRI 04 Ground Motion Model is proposed as the ground motion model element of the CEUS probabilistic seismic hazard model for future PSHAs for nuclear plant sites located in the CEUS. The implementation should follow the detailed guidance contained in Chapter 5, Sections 5.2 and 5.3 of McCann, et al. 2004, which provide detailed step-by-step guidance for implementing the model for given site-seismic source conditions. The model implementation summarized below draws heavily on the model implementation described in Exelon 2004, Appendix D, Section 4.1.4.

Figure 2-3 shows the logic tree structure defined by McCann et al. 2004 to characterize (i.e., represent the uncertainty in the median ground motion relationship and in modeling the aleatory variability about the median (standard deviation in the log of ground motion amplitude)). As discussed above, the EPRI 04 ground motion model defines four groupings of proponent median ground motion models, called Clusters, to represent the alternative modeling types/forms. Three of the Clusters are appropriate for use in assessing the ground motion from earthquakes occurring in area seismic sources in which earthquakes are model as occurring randomly within the source and all four Clusters are appropriate for use to assess ground motion from earthquake occurrences in sources capable of earthquakes large enough to be modeled as extended ruptures with known spatial orientation.

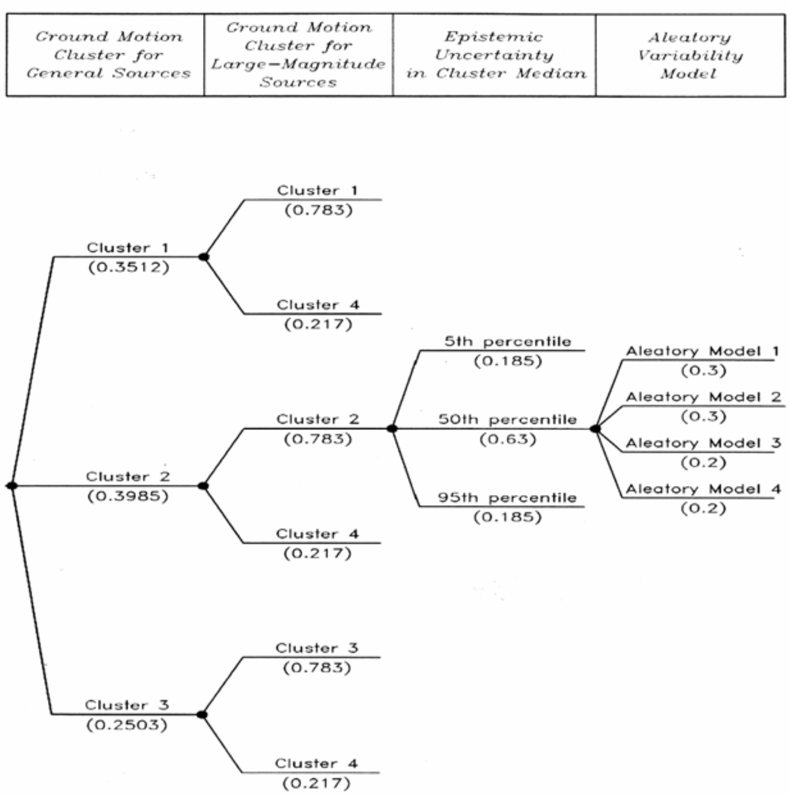

Figure 2-3 EPRI 04 Ground Motion Model Implementation Logic Tree (Robert R. Youngs, personal communication)

The first level of the logic tree shows the weights assigned to the three proponent ground motion model Cluster medians that are appropriate to model ground motion from earthquakes occurring randomly in seismic sources. The second level weights the appropriate proponent ground motion model Cluster median to use for seismic sources that have the potential for large extended rupture earthquakes. Examples of these sources are the Charleston zone seismic sources, the Wabash Valley-southern Illinois zone seismic sources and the New Madrid zone seismic sources. For these sources, two alternatives are given, either use of the Cluster model used for the local sources or use of the Cluster 4 model. The effect of this logic structure on the PSHA is as follows. Following the branch for Cluster 1 at the first node, two options are available. The first is to also use the Cluster 1 model for the large magnitude extended rupture sources. The second option is to use Cluster 1 for only the local sources and use Cluster 4 for the large magnitude extended rupture sources. This same logic is repeated for the branches for Clusters 2 and 3. The non-rift Cluster 4 model should be used for the Wabash Valley-southern Illinois zone extended source and the rift Cluster 4 model should be used for the New Madrid zone extended sources.

The third level of the logic tree addresses the uncertainty in the median attenuation relationship for each ground motion model Cluster. This uncertainty is modeled by a three-point discrete distribution with ground motion relationships for the 5th, 50th, and 95th percentiles of the epistemic uncertainty in the median relationship for each ground motion model Cluster.

The fourth level of the logic tree assesses the uncertainty in the model for the aleatory variability in ground motions about the median ground motion relationship.

The proponent ground motion relationships assessed for development of the EPRI 04 Ground Motion model define distance to the earthquake source in terms of either closest distance to the rupture plane or closest distance to the surface projection of the rupture plane (Joyner-Boore distance). Consistent with practice in the mid 1980s, the EPRI-SOG seismic source models model earthquake occurrences as point sources. EPRI 04 provides a set of relationships to convert point-source distance to equivalent Joyner-Boore or rupture distance under the assumption that the orientation of the earthquake rupture (the strike of the fault) is uniformly distributed in azimuth between 0 and 360 degrees. These distance adjustments must be used in the updated PSHA for the EPRI-SOG sources.

2.2.4 Truncation of Ground Motion Variability

As stated above and illustrated by Abrahamson and Bommer 2006, the variability in ground motion has a large effect on computed ground motion hazard. And the effect is increasingly dominant with lower annual hazard frequency. A ground motion amplitude value relative to the estimated median value predicted by a ground motion model is fundamentally controlled by the value of the standard deviation of the ground motion variability, sigma, which is the fundamentally important parameter describing ground motion variability. But for PSHA the number of standard deviations, epsilon, that the motion is above or below the estimated median value has an important effect on the ground motion hazard. For increasingly lower annual hazard frequencies ground motion amplitudes that may be several standard deviations above the median value increasingly dominate the hazard. Consequently, for some applications of PSHA it has become common practice to truncate the lognormal distribution at some number of epsilons, usually between 2 and 3. An adequately strong technical basis to support this practice for nuclear plant regulation has not been established, however, and the practice has therefore not been adopted for probabilistic seismic hazard analysis for nuclear plant sites.

As part of the NPSIRP, EPRI implemented a scope of work to determining whether or not a technical basis could be established for truncating the ground motion distribution at some maximum value of epsilon the significantly expanded data set that is now available. As described in Abrahamson and Bommer 2006, the study included statistical analysis of empirical data and theoretical simulations as well as detailed evaluations to determine whether recorded large positive ground motion amplitude values can be considered outliers or contaminants (data following other than a lognormal distribution) or whether they can be attributed to some anomalous physical feature of the earthquake source or propagation path. The results of this study, which are summarized in Abrahamson and Bommer 2006, Section 4, found:

- no systematic physical feature associated with recorded large positive epsilon ground motions that would exclude them from the empirical data set;
- residuals from empirical ground motion prediction models using large data sets have positive epsilon values ranging to 4; and
- large positive epsilon values are present in ground motion amplitudes from kinematic numerical simulations.

The principal conclusion reached from the results of the study is that there is currently no technical evidence that epsilon values greater than 3 are not possible.

Given the lack of a technical basis for truncation, a maximum value of epsilon should not be incorporated into probabilistic seismic hazard analyses for nuclear plant sites. This recommendation supports the approach taken for development of the EPRI 04 Ground Motion Model, which did not include a truncation of the ground motion distribution.

2.2.5 Recent Advances for Quantifying the Logarithmic Standard Deviation of Ground Motion Variability

Modeling the standard deviation of ground motion, sigma, in PSHA involves significant uncertainty both in the value of sigma and in whether the parameter is dependent on earthquake magnitude and distance from the earthquake source. The Next Generation of Attenuation (NGA) project has demonstrated that to a degree the large uncertainty in sigma and the way it is modeled in ground motion prediction equations is due to limited available empirical data. The NGA project, a partnered ground motion modeling effort that is being lead by the Pacific Earthquake Engineering Research (PEER) Center, the U. S. Geological Survey, and the Southern California Earthquake Center, has made significant progress toward over coming the empirical data limitation by compiling a large strong-motion recording data set consisting of about 3500 recordings from about 173 earthquakes (PEER, 2005, Power, et al. 2006). The project has been conducted over a period of three years. The project was structured to accomplish: 1) development of an updated and much expanded database of uniformly processed strong-motion recordings together with supporting metadata; 2) six working groups that provided supporting analysis and simulations addressing elements of ground motion modeling resulting in a common technical information base for use by the model developer teams; 3) a series of eight state-of-knowledge workshops and a number of working group meetings that brought together the model developer teams, members of the working groups, and outside reviewers to address technical developments and simulation methods and results; and 4) independent development of ground motion models by five developer teams. Taking into consideration the very large database of strong-motion recordings that was used for the project, the uniform high quality processing of the recordings, the level of effort devoted analyzing the data to evaluate components of ground motion modeling, and the integrated evaluations of the results of these analyses in a series of workshops, the NGA project is the most comprehensive and complete ground motion model development effort conducted to date.

Results of the NGA project support significant revisions of the value of the logarithmic standard deviation, sigma. The indicated revisions include a decrease in the value of sigma and significant revision of its dependence on earthquake magnitude distance. The additional empirical data obtained and used in the NGA project studies show that sigma is independent of earthquake magnitude. Previous ground motion prediction models consider sigma to be dependent on earthquake magnitude such that sigma decreased with increasing magnitude. The NGA project results support a value of sigma of about 0.6 natural log units for all earthquake magnitudes. This is smaller than the sigmas typically obtained for previous models for **M**<6 and larger for **M**>6.5 (Abrahamson and Bommer 2006, Figure 5-1). Abrahamson and Bommer 2006 consider these changes in the value of sigma and its dependence on earthquake magnitude to be due to the much expanded strong-motion recording data set that was developed and used for the NGA project.

Abrahamson and Bommer 2006 evaluated the applicability of the NGA results to ground motion modeling for the region of the CEUS. Their study, which implemented a SSHAC Level 2 assessment procedure (Budnitz, et al. 1997), and included evaluations of possible differences in sigma from WUS to the CEUS in terms earthquake source, wave propagation path and site contributions, a demonstration that the NGA results developed using largely WUS strong-motion recordings are appropriate for application in the CEUS, and the development of a recommended sigma model for application in the CEUS (Abrahamson and Bommer 2006, Chapter 6). The study concluded that the findings of the NGA studies with respect to the value of sigma and how sigma should be modeled are applicable for the CEUS and that the sigma values used in the EPRI 04 Ground Motion model are larger on average than is supported by the NGA findings. Thus, the ground motion hazard computed using the revised sigma model will be lower on average across sites compared to the hazard computed using the EPRI 04 Ground Motion Model.

2.2.6 Sensitivity of SSRS to Revised CEUS Sigma

Sensitivity of SSRS to the revised CEUS sigma model recommended by Abrahamson and Bommer 2006 were conducted and reported by McGuire 2006 (see Figures 2-1, 2-2, above). The sensitivity study shows that application of the revised sigma model results in lower mean SSRS amplitudes across all spectral frequencies for the 28 test sites used (see McGuire 2006, Figure 2-6). The mean reduction is approximately 9% at 100 Hz (PGA) and approximately 19% at 5 Hz. The effect varies across the 28 sites and across structural frequencies. This behavior is illustrated in McGuire 2006, Figures 2-7 and 2-9, which show that the cumulative distribution of the reduction across the 28 sites for PGA and 0.5 Hz structural frequency respectively. For PGA the reduction ranges from 2% to 15% while for 0.5 Hz structural frequency the reduction ranges from about 12% to about 28%.

2.2.7 Discussion and Recommendation

In Section 2.2.3 we recommended that the EPRI 04 Ground Motion model should be implemented for generic use to compute ground motion hazard for nuclear plant sites located in the CEUS; i.e., it should be implemented as a generic update of the ground motion component of the CEUS probabilistic seismic hazard model. However, new significantly larger data sets and evaluations performed by Abrahamson and Bommer 2006 show that modeling of sigma in the EPRI 04 model resulted in sigma values that are larger than can currently be supported. The EPRI 04 study based the assessment of sigma on four available proponent alternative models (Abrahamson and Silva 1997, Toro et al. 1997 updated with the revised modeling variability developed by Silva, et al. 1996, USNRC 2002, and Silva, et al. 2002 (McCann, et al. 2004). Abrahamson and Bommer 2006 review the basis for and features of each of these models. Of particular relevance, the Toro, et al. model is based on numerical simulations. The issue to be resolved is whether the sources of variability upon which the simulations were based are independent. The sigma model recommended by Abrahamson and Bommer 2006 is based on a large empirical data set.

Because the revised CEUS sigma model recommended by Abrahamson and Bommer 2006 is supported by a greatly expanded empirical data set and because the value of sigma can have a significant impact on the hazard at a site, it is important to update the EPRI 04 Ground Motion model. To accomplish this would involve revisiting the numerical simulation, which received high weight in the EPRI 04 model, and a review of these results together with the results of the Abrahamson and Bommer study by the expert panel that was convened for development of the EPRI 04 model. This assessment has not yet been accomplished. However, some utilities may consider using the revised sigma recommended by Abrahamson and Bommer for a specific site.

3

SITE RESPONSE ANALYSIS FOR DEVELOPMENT OF UNIFORM HAZARD RESPONSE SPECTRA AT THE GROUND SURFACE

The recommended approach for determining a UHRS at the ground surface, given a CEUS generic rock UHRS uses the combination of site response Approaches 2A and 3 described in NUREG/CR-6769, Section 6.1.2 (McGuire, et al. 2002). Approach 2A is applied to compute the mean site response and its standard deviation and Approach 3 is applied to integrate over the appropriate range of rock ground motion values to obtain the ground surface UHRS. McGuire et al. 2002 label this Approach 2A/3 and describe the steps (Steps 1 and 2) for determining ground surface UHRS in Section 6.2 of their report. Step 3 in Section 6.2 of McGuire et al. describes procedures for deriving Uniform Risk Response Spectra (URRS) starting with UHRS. This step depends on the seismic margin attained by the required seismic design and, therefore, the risk reduction factor that it is achieved the required seismic design criteria (see Appendix A, Equations A-1 and A-2). For determination of SSRS for future nuclear plants the guidance contained in Appendix A of this report should be followed as discussed in more detail in Chapter 4. Implementation of Approach 2A/3 for site response analysis involves additional steps to characterize the site properties, described below.

The EPRI 04 Ground Motion Model is defined for CEUS generic rock conditions corresponding to a shear-wave velocity of 2.83 km/sec (9,300 ft/sec) (see Table 2-1). CEUS generic rock conditions are present at the ground surface in some areas of the CEUS. However, for the locations of most nuclear plants CEUS generic rock will be at a depth, which will vary form site to site, under a cover of softer rock and near surface soil strata. For site response analysis for these sites it is therefore necessary to characterize the properties of the softer rock and soil strata. Current practice is to characterize the shear-wave velocity profile for the geologic section beneath the site to the depth where CEUS generic rock is encountered by developing a site-specific median profile and its standard deviation. The median and standard deviation of shear-wave velocity profile for the near-surface geologic strata is based on measurements in borings across the footprint of the proposed facility. The shear-wave velocity profile for strata deeper than is penetrated by the site boring program but shallower than the depth of the CEUS generic rock may be characterized based existing deep drill-holes located in the vicinity of the site that have P-wave velocity measurements. This is done by converting the P-wave velocity profiles to shear-wave velocity profiles using reasonable values of Poisson's ratio (e.g., Exelon 2004; McGuire 2005b).

In order to appropriately characterize the uncertainty in the shear-wave velocity profile beneath a site the shear-wave velocity profiles across the site are randomized. This is accomplished using the shear-wave velocity correlation model develop by Toro (1996). The converted P-wave velocity data can be used for computing the median and standard deviation for deep velocity profiles. The commonly accepted practice for modern site response analysis is to generate sixty randomized shear-wave velocity profiles to represent the uncertainty and variability in shear-wave velocity beneath a site.

Site response has been shown to be relatively insensitive to reasonable variation in velocity at depth provided that the site-specific total low strain damping value, kappa, remains fixed (McGuire, 2005b, McGuire et al. 2002, Section 6.2.2). Kappa varies among sites depending on the properties of local geology to a depth of 1 to 2 km (Silva and Darragh 1995). Silva and Darragh (1995) and Silva et al. (1996) have compiled values of total kappa appropriate for typical CEUS rock shear-wave velocities. An acceptable approach for using these data to characterize kappa for a site is described in Appendix B, Sections 4.2.1 and 4.2.2 of Exelon 2004. For sites where the depth to CEUS generic rock is poorly known or very deep (e.g., the Gulf Coast region) it is appropriate to assume a depth to CEUS generic rock that ensures that the site response for the lowest structural frequency of interest, 0.5 Hz, is properly captured (McGuire, et al. 2002; McGuire 2005b). For these cases the depth to the CEUS generic rock should be randomized over a large enough range to smooth the low frequency resonance (McGuire 2005b).

The uncertainty in the dynamic properties, strain-dependent modulus reduction and strain-dependent damping ratio, for the geologic section beneath a site is characterized for site response analysis. For near-surface strata that are penetrated by the site boring program, the characterizations uses the shear modulus and damping tests performed on samples taken as part of the site boring and testing program. It may be appropriate for some sites to compare the modulus reduction and damping curves developed from the site boring and testing program to generic CEUS curves (EPRI 1993). Where found appropriate generic curves may be used.

The base-case shear modulus reduction and damping relationships are randomized to characterize the uncertainty and variability in these properties. Silva, et al. 1996 present data on the variability in modulus reduction and damping ratio based on testing of rock and soil samples. These data can be used to establish the standard deviation of shear modulus reduction and damping ratio about the base case curves for a site. An acceptable approach for accomplishing this analysis is described in Appendix B, Section 4.2.2 of Exelon 2004. Linear elastic behavior should be assumed for the shear-wave velocities larger than about 4,000 ft/sec.

For the purpose of implementing site response Approach 2A/3 it is necessary to de-aggregate hazard for at least 10^{-4}, 10^{-5}, and 10^{-6} annual hazard frequencies. For this step generic rock UHRS is de-aggregated for 10 Hz (HF) and 1 Hz (LF) to determine the mean magnitudes and distances of earthquakes that control the hazard for these mean annual frequencies and spectral frequencies – HF and LF reference earthquakes (McGuire, et al. 2002b). The resulting set of reference earthquakes is used together with the randomized shear-wave velocity profiles and randomized modulus reduction and damping curves to compute median site response and the standard deviation of site response (McGuire, et al. 2002b). Acceptable approaches for accomplishing this step are described in Exelon 2004, Appendix B, Section 4 and in McGuire 2005b, Chapter 5. The median site response factors are then used to obtain UHRS at the ground surface as described in McGuire, et al. 2002b, Section 6.2 and in Appendix A, using Equation A-16. Equation 3-1, which is Equation A-16 from McGuire, et al. 2002, corrected for a typo, should be used for this step (G. Toro, personal communication).

$$P[A_S > z] = P[A > \frac{z}{AF}]\exp\{\frac{1}{2}k^2\sigma_\delta^2 / d_3^2\}$$ **Equation 3-1**

4 DETERMINATION OF SSRS AND SITE-SPECIFIC RISK-INFORMED DESIGN RESPONSE SPECTRA (DRS)

It is proposed that performance-based (risk-informed) site-specific safe shutdown earthquake response spectra (SSRS) for future nuclear plants be determined using the performance-goal based (risk-informed) approach defined in the ASCE 43-05 Standard (ASCE 2005). Appendix A of this report describes the ASCE 43-05 Standard approach, amplifies upon the Commentary by explaining the bases and assumptions underpinning the approach for defining the risk-informed SSRS, and demonstrates the level of safety that is achieved when the approach is used together with NRC's seismic design criteria and procedures (USNRC 1996).

The SSRS implements the U. S. Nuclear Regulatory Commission's Risk-Informed regulation policy with respect to determination of risk-informed SSE Ground Motion and thereby satisfies the requirements of 10 CFR Part 100.23 and the requirements of Appendix S to 10 CFR Part 50 for definition of SSE Ground Motion to be used for development of the appropriate design response spectra consistent with the guidance contained in NUREG-0800, Section 3.7.1(I) and Section 3.7.1(I)(1)(a).

The principal steps involved in determination of a site-specific SSRS are illustrated in Figure 4-1. Figure 4-1 also shows the additional step (discussed below) required to determine the appropriate site-specific risk-informed DRS that will be used as the control motion for analysis of the soil-structure. The two remaining steps shown in Figure 4-1 are to compare the site-specific risk-informed DRS with the minimum required DRS and with the appropriate standard plant DRS. These steps are elaborated below.

The determination of the SSRS for a site begins with a site-specific PSHA based on and updated probabilistic seismic hazard model for the site region. A properly updated probabilistic seismic hazard model includes: 1) updated characterization of seismic sources, accomplished following the guidance contained in Reg. Guide 1.165, 2) updated characterization of earthquake recurrence for seismic sources following the guidance contained in Reg. Guide 1.165 and including generic implementation of the CAV filter described in Section 2.1 and in detail in Abrahamson et al. 2006, and 3) generic implementation of the EPRI 04 Ground Motion Model for the CEUS (see Section 2.2 and McCann, et al. 2004). The PSHA computation results obtained at a site using the updated probabilistic seismic hazard model are for CEUS generic rock (see Table 2-1 for definition). For defining the UHRS, as a minimum ground motion hazard should be computed for response spectra frequencies of 0.5, 1.0, 2.5, 10, and 25 Hz and for PGA (assumed to be 100 Hz for sites located in the CEUS). Hazard curves should extend to at least 10^{-6} annual hazard frequency.

CEUS generic rock is present at the ground surface for some sites located in the CEUS. For these sites the ASCE 43-05 Standard approach is applied as described in Appendix A using the sit-specific CEUS generic rock UHRS. For these sites no site response analysis is required; the generic rock UHRS is used as the "reference" UHRS that is scaled by the

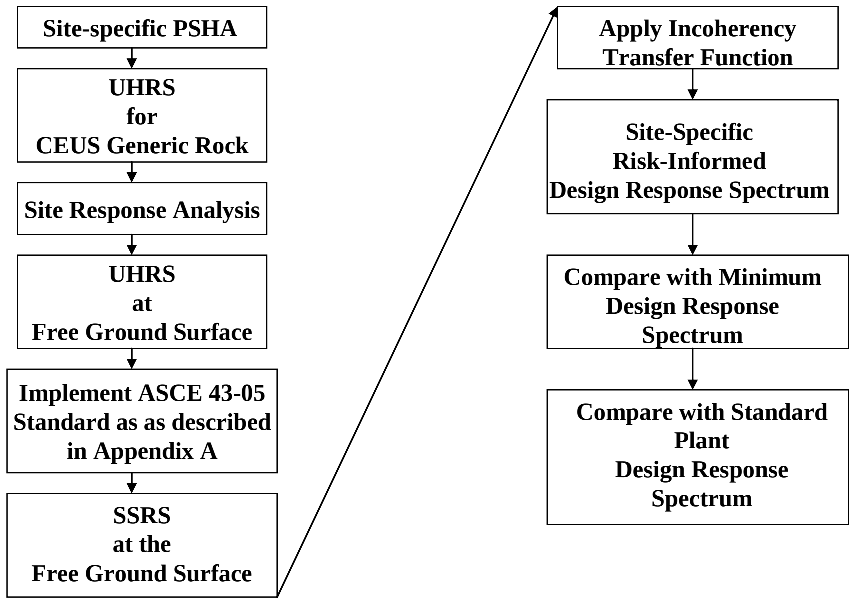

Figure 4-1
Principal Steps for Determining SSRS and Site-Specific Risk-Informed DRS

appropriate design factor (DF) using Equation A-1 as described in Appendix A in order to obtain the SSRS.

For most sites located in the CEUS, however, CEUS generic rock is at depth below softer rock and/or soil strata. For these sites it is necessary to perform the additional step of a site response analysis in order to obtain hazard curves at the ground surface and to determine a site-specific UHRS at the ground surface. Approach 2A/3 outlined in Chapter 3 of this report and described in detail in McGuire, et al. 2002, Section 6.2 is the recommended method for performing the site response analysis and obtaining hazard curves at the ground surface for development of the ground surface UHRS. The ground surface UHRS is the "reference" UHRS that is scaled by the appropriate design factor DF using Equation A-1 as described in Appendix A in order to obtain the SSRS.

The SSRS determined following the above steps and defined at the free ground surface meets the requirements of the seismic regulations (USNRC 1997b, USNRC 1997c) using performance-goal based (risk-informed) procedures described in Appendix A. Appendix S to Part 50 (USNRC, 1997C) provides that appropriate design response spectra consistent with the SSRS and appropriate for the site-specific material properties should be determined. Standard Review Plan (SRP) Section 3.7.1, which provides qualitative guidance for acceptable procedures for meeting the requirements of Appendix S to Part 50, provides guidance that the appropriate design response spectra are to be used as input control motion for analyses of the soil-structure interaction system. Current SRP (Rev. 2, 1989, which was not significantly revised for Rev. 3, 1997) guidance is to define the control motion on a free ground surface determined based on the site-specific soil-rock profile properties of the site. For rock sites or soil sites with smooth variation of properties current SRP guidance is to define the control motion at the top of the finished grade. For sites where one or more thin soil layers overlie competent material the current SRP guidance is to define the control motion at the top of competent material treating this location as an outcrop or a hypothetical outcrop.

Subsequent to issuance of SRP 3.7.1 Rev. 2 the NRC funded a study aimed at developing criteria for implementation of modern soil-structure interaction codes such as CLASSI and SASSI. A component of this study specifically assessed the appropriate location for control motion when these modern codes are used and developed recommended quantitative definitions for control point locations depending on soil-rock properties of a site. Based on that study the following definitions for control point location were recommended (Costantino and Miller 1992).

- "***When the site consists of relatively uniform competent materials (defined as material with a shear wave velocity equal to or exceeding 1000 fps) throughout the depth of interest, the control motion can be applied at the ground surface as currently recommended in the SRP.***
- ***If the shear wave velocity exceeds 1,500 fps, the generic broad-banded spectra can conservatively be located at a rock outcrop (either real or fictitious) and the resulting surface spectra are conservative.***
- ***If the site consists of a soft, thin layer resting atop the competent material, the ground motion defined by the broad-banded spectra can be defined at the topof the competent material. The thin, soft layer is defined as a layer with shear wave velocity equal to or less than 750 fps, with thickness up to 100 feet.***
- ***The current limitation on deamplification with depth (60%) should be maintained.***
- ***For stiffer soil layers (shear wave velocity exceeding about 1500 fps and thickness equal to or less than 300 feet), the broad-banded spectra can be located at an outcrop (real or fictitious) below the foundation depth which will yield conservative estimates of foundation level motions.***
- ***Care should be taken to ensure that the data base used to evaluate the scaling parameters for the generic spectra are conservatively estimated."***

For these definitions shear-wave velocities are low strain values as would be obtained form standard site geotechnical characterization methods. "Broad-banded spectra" means the Reg. Guide 1.60 spectrum or a similar broad-banded site independent spectrum. The above guidance assumes that SSI analysis will be performed using modern codes such as CLASSI or SASSI, a standard broad-banded spectrum such Reg. Guide 1.60 will be used as input (the DRS) for the analysis, and the analysis will involve deconvolution of the broad-banded spectrum.

Modern site response methods available today (e.g., Approach 2A/3) involve convolution upward through the geologic section at a site, overcoming constraints imposed by deconvolving a broad-banded spectrum. These methods account for uncertainties in the site shear-wave velocity profile and in the material properties, resulting in consistent motions at all levels (McGuire, et al. 2002a, 2002b). With these modern site response methods it is proposed that the control motion for soil sites be obtained on "competent material", defined as having low strain shear-wave velocity greater than 1,000 ft/sec, with further site response then performed by convolving upward through any soft materials.

Materials above the control motion location when it is defined in this way will be treated as part of the soil-structure interaction analysis using modern codes such as CLASSI or SASSI, and appropriate guidance for representing strain compatible soil properties. Guidance for this implementation will be provided in updated SRP Section 3.7.2 (C. J. Costantino personal communication).

Application of the Incoherence Transfer Function (Abrahamson, 2006; Short, et al. 2005), which is dependent on foundation size, is the final step for deriving the site-specific risk-informed DRS, which will be used as the input or control motion for engineering analysis of the soil-structure system. It is proposed that the Incoherence Transfer Function be applied to the SSRS to obtain the site-specific DRS used as input for SSI analyses (Short, et al. 2005.

Appendix S to Part 50(IV)(1)(i) requires that the minimum horizontal component of the site-specific risk-informed DRS must be an appropriate response spectrum with a peak ground acceleration of at least 0.1g. In order to insure that this requirement is satisfied it is proposed that the site-specific risk-informed DRS be compared with the appropriate minimum DRS. It is proposed that the Reg. Guide 1.60 broad-banded horizontal response spectrum scaled at 33 Hz to 0.1g be adopted as the minimum required DRS. This comparison step will insure that the site-specific risk-informed DRS has spectral amplitudes over the frequencies important for the design of nuclear plants that are as large or larger than the minimum required DRS.

The final step compares the site-specific risk-informed DRS with the appropriate DRS for the certified design plant to be placed at the site in order to insure that the site is suitable for the certified design plant.

5
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A

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

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A.1 Introduction

It is proposed that the Safe Shutdown Earthquake (SSE) Site-Specific Response Spectrum (SSRS) for future nuclear power plants 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

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

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.

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 A.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 A.5.

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

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

   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 A.6. However, once the value of H is chosen the required DF to be used in Equation A-1 will be a function of the Probability Ratio R_P defined by:

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

   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 A.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 A-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 A.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 A.7 for R_P=10 which is proposed herein.

A.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]:

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

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

[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.

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 A-1, where the reference UHRS is defined at a reference seismic hazard exceedance frequency H of:

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

$$A_R = \frac{SA_{0.1H}}{SA_H}$$ **Equation A-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 A-6**

where

$$DF_1 = 1.0$$ **Equation A-7**

and

$$DF_2 = 0.6(A_R)^{0.80}$$ **Equation A-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 A-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.

A.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]").

A.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:

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

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

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 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.

A.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 A.1). Thus over any (at least) ten-fold difference in exceedance frequencies such hazard curves may be approximated by a power law:

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

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:

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

$$K_H = \frac{1}{\log(A_R)}$$ **Equation A-12**

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

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

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

in which

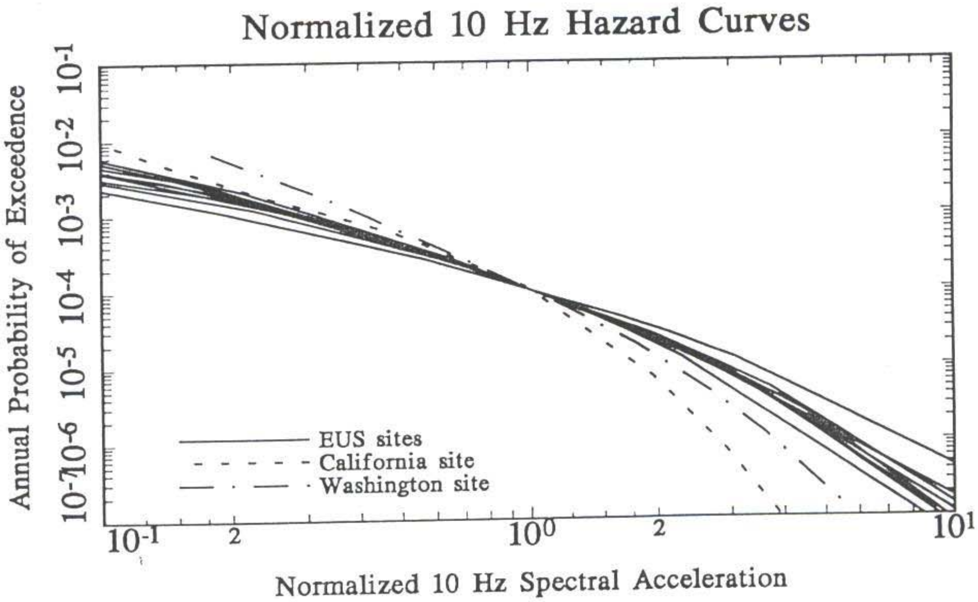
$$F_{50\%} = \frac{C_{50\%}}{C_H}$$ **Equation A-14**

and

$$\alpha = \tfrac{1}{2}(K_H \beta)^2$$ **Equation A-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 A-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 A-11 over the exceedance frequency range of interest and the fragility curve is lognormally distributed.



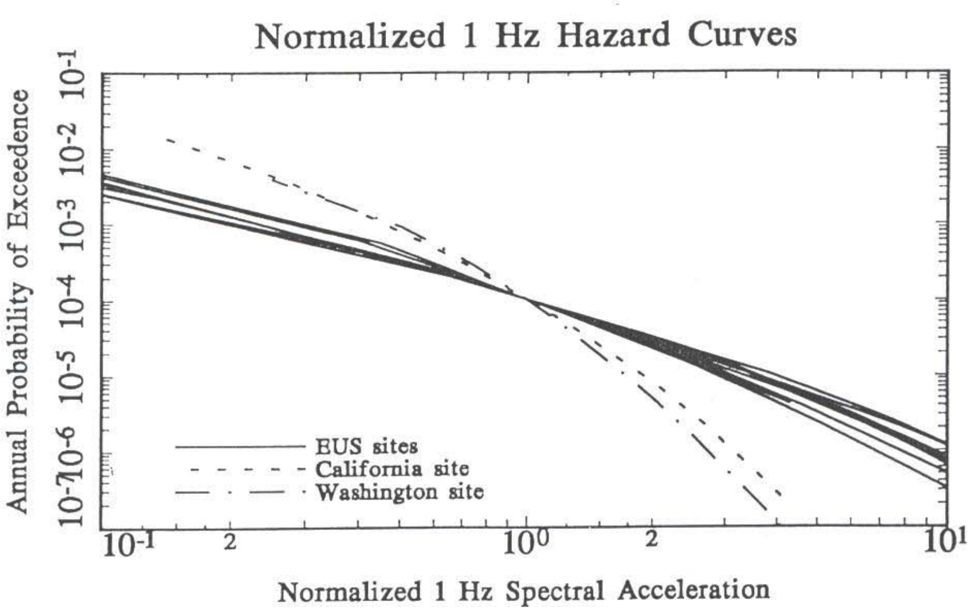


Figure A-1
SA (10 Hz) and SA (1 Hz) hazard curves for the eleven sites normalized by the acceleration value corresponding to mean 10^{-4} annual probability. (From Figures. 7.7 and 7.8 of REI, 2001)

A.3.3 Design Factor Equation

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

$$C_{50\%} = C_H \left[R_p e^{\alpha}\right]^{1/K_H}$$ **Equation A-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:

$$F_P = \frac{C_P}{SSRS}$$ **Equation A-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 A-1 and recognizing that C_H=UHRS, then:

$$F_P = \frac{C_P}{DF * C_H}$$ **Equation A-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:

$$C_P = C_{50\%} e^{X_P \beta}$$ **Equation A-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. A-16, A-18 and A-19:

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

in which

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

Equation A-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 A-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 A-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 A-22**

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

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

A.4 Basis for Target Performance Goal

As discussed in Section A.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 1×10^{-5}/yr for seismic induced onset of significant inelastic deformation (FOSID).

The basis for selecting a quantitative target performance goal P_T of mean 1×10^{-5}/yr is that mean 1×10^{-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 A-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 A-2
Mean Seismic CDF for Plants Performing Seismic PRA
from Table 2.2 from NUREG 1742, Vol. 2

Plant	Mean Seismic CDF (EPRI)*	Plant	Mean Seismic CDF (EPRI)*
South Texas Project 1 & 2	1.90E-07	Seabrook	1.20E-05
Nine Mile Point 2	2.50E-07	Beaver Valley 1	1.29E-05
La Salle 1 & 2	7.60E-07	Indian Point 2	1.30E-05
Hope Creek	1.06E-06	Point Beach 1 & 2	1.40E-05
D.C. Cook 1 & 2	3.20E-06	Catawba 1 & 2	1.60E-05
Salem 1 & 2	4.70E-06	San Onofre 2 & 3	1.70E-05
Oyster Creek	4.74E-06	Columbia (Washington Nuclear Project No. 2)	2.10E-05
Surry 1 & 2	8.20E-06	TMI 1	3.21E-05
Millstone 3	9.10E-06	Oconee 1, 2, and 3	3.47E-05
Beaver Valley 2	1.03E-05	Diablo Canyon 1 & 2	4.20E-05
Kewaunee	1.10E-05	Pilgrim 1	5.80E-05
McGuire 1 & 2	1.10E-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 A.8.

A.5 Level of Conservatism of Specified Seismic Design Criteria

A.5.1 Factor of Conservatism for the Onset of Significant Inelastic Deformation

As noted in Section A.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 A-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 A-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:

- $F_{1\%}$ $\geq$ 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 A.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.

A.5.2 Expected Factor of Conservatism for Core Damage Fragility

The seismic design criteria factors of conservatism defined in Section A.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:

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

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

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

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

A.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:

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

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 $1x10^{-5}$/yr.

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 A-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 A-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.

A.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 A-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 A-22, and by comparing how close the failure probabilities implied by use of Equation A-6 are to the target acceptable failure probability. The latter computation will be done two ways, using the analytical approximation (Equation A-13) and by numerical integration of the exact integrals.

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

The required Design Factors DF computed using Equation A-22 to achieve R_P=10 for the onset of significant inelastic deformation $F_{1\%}$ and β combinations defined in Section A.5.1 are shown in Table A-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 A-6. The ASCE Standard 43-05 DF Equation A-6 was empirically developed to closely fit these required DF values.

Table A-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 (A-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 A-6 was chosen to provide a generally conservatively biased DF over the range of A_R and β values considered in Table A-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 A-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 A-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 A.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 A-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 A-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:

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

where DF is given by Equation A-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 A-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 A-5 introduces a slight conservative bias. This slight conservative bias will subsequently be illustrated.

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

A.7.2.1 Using the Simplified Risk Equation.

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

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

where f is obtained from Equation A-23.

Table A-5 presents P_{FC} results computed from Equation A-27 with the ASCE Standard 43-05 DF defined by Equation A-6 and $F_{1\%}$ defined in Section A.5.1 for various logarithmic standard deviations β. The conclusion is that with the ASCE Standard 43-05 SSRS defined as described in Section A.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:

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

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

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

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.

A.7.2.2 Using Rigorous Numerical Convolution of Fragility and Actual Hazard Curves

Figure A-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 1×10^{-4}/yr for ease of visualizing the differences in hazard curve slopes. Table A-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 A-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 A.7.2.1 are defined by Equations A-11 and A-12 with A_R defined by Equation A-5. These approximate hazard curves would appear as a straight line on the log-log plots of Figure A-1 with the amplitude and slope defined by the spectral accelerations at 1×10^{-4}/yr and 1×10^{-5}/yr hazard exceedance frequencies. However, all actual seismic hazard curves have a downward curvature similar to those shown in Figure A-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 A.7.2.1 using the simplified risk equation method.

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

Table A-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 A-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 A-5 is justified for defining the Design Factor DF. The FOSID conclusion reached in Section A.7.2.1 and presented in Equation A-28 remains valid.

A.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 A.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 A.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 A-8.

Table A-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 A-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.

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

Section A.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 A.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 A-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 A-6.

Table A-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**	**SCDF ($*10^{-6}$)**			
	SA_{SSRS}	**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 A-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 A-10:

Table A-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 A-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.

A.9 References

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B
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 A-11, then from Equation A-10 one obtains[6]:

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

Equation B-1

in which

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

Equation B-2

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

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

Equation B-3

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

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

Equation B-4

or from the previous definition of M:

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

Equation B-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 A-11:

$$K_I = H\left[C_H \right]^{K_H}$$

Equation B-6

from which:

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

$$P_F = HF_{50\%}^{-K_H} e^{\alpha}$$ **Equation B-7**

$$F_{50\%} = \frac{C_{50\%}}{C_H}$$ **Equation B-8**

$$\alpha = \frac{1}{2}(K_H \beta)^2$$ **Equation B-9**