October 18, 2012

Seismic Evaluation Guidance

Screening, Prioritization and Implementation Details (SPID) for the Resolution of Fukushima Near-Term Task Force Recommendation 2.1: Seismic

This document does **NOT** meet the requirements of 10CFR50 Appendix B, 10CFR Part 21, ANSI N45.2-1977 and/or the intent of ISO-9001 (1994).

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Draft Report, October 2012

Executive Summary

(*This section to be expanded*) This document will serve as guidance for responding to the March 12, 2012 U.S. Nuclear Regulatory Commission (NRC) Request for Information 50.54(f) Letter regarding Recommendation 2.1 for Seismic.

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List of Acronyms

AEF	annual exceedance frequency
ANS	American Nuclear Society
AOV	air-operated valve
ASME	American Society of Mechanical Engineers
BWR	boiling water reactor
CAV	Cumulative Absolute Velocity
CDF	core damage frequency
CDFM	Conservative Deterministic Failure Margin
CENA	Central and Eastern North America
CEUS	Central and Eastern United States
COLA	Combined Operating License Application
COV	coefficient of variation
ESP	early site permit
FSAR	Final Safety Analysis Report
GI	Generic Issue
GIP	Generic Issues Program
GL	Generic Letter
GMPE	ground motion prediction equations
GMRS	ground motion response spectra
HCLPF	High Confidence of Low Probability of Failure
IEEE	Institute of Electrical and Electronics Engineers
IHS	IPEEE HCLPF Spectrum
IPEEE	Individual Pant Examination of Individual Events
ISG	Interim Safety Guide
ISRS	in-structure response spectra
LERF	large-safety release frequency
LHT	Low Hazard Threshold
LMSM	Lumped-mass stick models
MCC	motor control center

MCCB	Molded case circuit breakers
MOV	motor-operated valve
MSVH	Main Steam Valve House
NGA	Next Generation Attenuation
NPP	nuclear power plant
NRC	Nuclear Regulatory Commission
NTTF	Near Term Task Force
PGA	Peak Ground Acceleration
PSD	power spectral density
PSHA	probabilistic seismic hazard analysis
RLE	review level earthquake
RLME	Repeated Large Magnitude Earthquake
RMF	Random Multi-Frequency
RVT	Random Variation Theory
SAMG	Severe Accident Management Guidance
SCDF	seismic core damage frequency
SER	Safety Evaluation Report
SFP	spent fuel pool
SMA	seismic margin assessment
SPID	Screening, Prioritization, and Implementation Details
SPRA	seismic probability risk assessment
SQUG	Seismic Qualification Utilities Group
SSC	structures, systems, and components
SSE	safe-shutdown earthquake
SSHAC	Senior Seismic Hazard Analysis Committee
SSI	soil-structure interaction
TS	Time Series
UHRS	uniform hazard response spectrum
UHS	uniform hazard spectrum
USNRC	United States Nuclear Regulatory Commission
WNA	Western North America
WUS	Western United States
ZPA	zero period acceleration

Section 1: Purpose and Approach

Following the accident at the Fukushima Daiichi nuclear power plant resulting from the March 11, 2011 Great Tohoku Earthquake and subsequent tsunami, the United States Nuclear Regulatory Commission (NRC) established the Near Term Task Force (NTTF) in response to Commission direction. The NTTF issued a report that made a series of recommendations, some of which were to be acted upon “without unnecessary delay.” Subsequently, the NRC issued a 50.54(f) letter that requests information to ensure that these recommendations are addressed by all U.S. nuclear power plants (NPPs). The principal purpose of this report is to provide guidance for responding to the request for information in the 50.54(f) Letter, Enclosure 1, Recommendation 2.1: Seismic [1].

Although the guidance in this document is specifically directed at supporting responses to the 50.54(f) letter, much of the guidance is appropriate for elements of any seismic risk evaluation.

Comment [C1]: May want to update this paragraph to be consistent with Section 6.6.

Comment [GSH2]: Agree, will update one Section 6.6 finalized.

Section 1 of this report provides the background on two past seismic programs (IPEEE and GI 199) that are particularly relevant to the 2.1 seismic assessment, and summarizes both the NTTF recommendations and the technical approach intended to support the response to the 2.1 seismic requests. Section 2 characterizes the seismic hazard elements of the response to the information requests. Section 3 contains the ground motion response spectra (GMRS) screening criteria associated with the resolution of the 2.1 seismic issue. Section 4 describes the elements of the recommended seismic hazard and screening report to be submitted to the NRC. Section 5 describes the schedule prioritization for completion of the seismic risk part of the 2.1 seismic program. Section 6 contains the seismic risk evaluation methods for those plants required to conduct these assessments. Finally, Section 7 documents an approach to the evaluation of the seismic integrity of spent fuel pool integrity assessment.

1.1 Background on Seismic Risk Evaluations in the U.S.

The risk posed by seismic events to plants operating in the United States was previously assessed in the mid-1990s as part of the response to the request for an Individual Plant Examination of External Events [2].

Further efforts to understand seismic risks, particularly in light of increased estimates of seismic hazard for some sites, led to the initiation of the Generic Issue 199 program [6]. An understanding of these two programs provides valuable background for the discussion of seismic evaluations related to the current 50.54(f) letter.

1.1.1 Individual Plant Examination of External Events – Seismic

On June 28, 1991, the NRC issued Supplement 4 to Generic Letter (GL) 88-20, "Individual Plant Examination of External Events (IPEEE) for Severe Accident Vulnerabilities," [2]. This supplement to GL 88-20, referred to as the IPEEE program, requested that each licensee identify and report to the NRC all plant-specific vulnerabilities to severe accidents caused by external events. The IPEEE program included the following four supporting objectives:

1. Develop an appreciation of severe accident behavior.
2. Understand the most likely severe accident sequences that could occur at the licensee's plant under full-power operating conditions.
3. Gain a qualitative understanding of the overall likelihood of core damage and fission product releases.
4. Reduce, if necessary, the overall likelihood of core damage and radioactive material releases by modifying, where appropriate, hardware and procedures that would help prevent or mitigate severe accidents.

The following external events were to be considered in the IPEEE: seismic events; internal fires; high winds; floods; and other external initiating events, including accidents related to transportation or nearby facilities and plant-unique hazards. The IPEEE program represents the last comprehensive seismic risk/margin assessment for the U.S. fleet of NPPs and, as such, represents a valuable resource for future seismic risk assessments.

EPRI conducted a research project to study the insights gained from the seismic portion of the IPEEE program [3]. The scope of that EPRI study was to review the vast amounts of both NRC and licensee documentation from the IPEEE program and to summarize the resulting seismic IPEEE insights, including the following:

- Results from the Seismic IPEEE submittals
- Plant improvements/modifications as a result of the Seismic IPEEE Program
- NRC responses to the Seismic IPEEE submittals

The seismic IPEEE review results for 110 units are summarized in the EPRI Report [3]. Out of the 75 submittals reviewed, 28 submittals (41 units) used seismic probabilistic risk assessment (PRA) methodology; 42 submittals (62 units) performed seismic margin assessments (SMAs) using a methodology developed by EPRI [39]; three submittals (three units) performed SMAs using an NRC developed methodology; and two submittals (four units) used site-specific seismic programs for IPEEE submittals.

In addition to the EPRI review of seismic IPEEE insights, the NRC conducted a parallel study. NUREG-1742, "Perspectives Gained from the Individual Plant Examination of External Events (IPEEE) Program," issued April 2002 [4], provides insights gained by the NRC from the seismic part of the IPEEE program. Almost all licensees reported in their IPEEE submittals that no plant vulnerabilities were identified with respect to seismic risk (the use of the term "vulnerability" varied widely among the IPEEE submittals). However, most licensees did report at least some seismic "anomalies," "outliers," or other concerns. In the few submittals that did identify a seismic vulnerability, the findings were comparable to those identified as outliers or anomalies in other IPEEE submittals. Seventy percent of the plants proposed improvements as a result of their seismic IPEEE analyses.

1.1.2 Generic Issue 199

In support of early site permits (ESPs) and combined operating license applications (COLAs) for new reactors, the NRC staff reviewed updates to the seismic source and ground motion models provided by applicants. These seismic updates included new EPRI models to estimate earthquake ground motion and updated models for earthquake sources in the Central and Eastern United States (CEUS), such as those around Charleston, South Carolina, and New Madrid, Missouri. These reviews produced some higher seismic hazard estimates than previously calculated. This raised a concern about an increased likelihood of exceeding the safe-shutdown earthquake (SSE) at operating facilities in the CEUS. The NRC staff determined that, based on the evaluations of the IPEEE program, seismic designs of operating plants in the CEUS do not pose an imminent safety concern. At the same time, the NRC staff also recognized that because the probability of exceeding the SSE at some currently operating sites in the CEUS is higher than previously understood, further study was warranted. As a result, the NRC staff concluded on May 26, 2005 [5] that the issue of increased seismic hazard estimates in the CEUS should be examined under the Generic Issues Program (GIP).

Generic Issue (GI)-199 was established on June 9, 2005 [6]. The initial screening analysis for GI-199 suggested that estimates of the seismic hazard for some currently operating plants in the CEUS have increased. The NRC staff completed the initial screening analysis of GI-199 and held a public meeting in February 2008 [7], concluding that GI-199 should proceed to the safety/risk assessment stage of the GIP.

Subsequently, during the safety/risk assessment stage of the GIP, the NRC staff reviewed and evaluated the new information received with the ESP/COLA submittals, along with NRC staff estimates of seismic hazard produced using the 2008 U.S. Geological Survey seismic hazard model. The NRC staff compared the new seismic hazard data with the earlier seismic hazard evaluations conducted as part of the IPEEE program. NRC staff completed the safety/risk assessment stage of GI-199 on September 2, 2010 [8], concluding that GI-199 should transition to the regulatory assessment stage of the GIP. The safety/risk assessment also concluded that (1) an immediate safety concern did not exist, and (2) adequate protection of public health and safety was not challenged as a result of the new information. NRC staff presented this conclusion at a public meeting held on October 6, 2010 (ADAMS Accession No. ML102950263). Information Notice 2010-018, "Generic Issue 199, Implications of Updated Probabilistic Seismic Hazard Estimates in Central and Eastern United States on Existing Plants," dated September 2, 2010 [9], summarizes the results of the GI-199 safety/risk assessment.

For the GI-199 safety/risk assessment, the NRC staff evaluated the potential risk significance of the updated seismic hazards using the risk information from the IPEEE program to calculate new seismic core damage frequency (SCDF) estimates. The changes in SCDF estimate calculated through the safety/risk assessment performed for some plants lie in the range of 10^{-4} per year to 10^{-5} per year, which meet the numerical risk criteria for an issue to continue to the regulatory assessment stage of the GIP. However, as described in NUREG-1742 [4], there are limitations associated with utilizing the inherently qualitative insights from the IPEEE submittals in a quantitative assessment. In particular, the NRC staff's assessment did not provide insight into which structures, systems, and components (SSCs) are important to seismic risk. Such knowledge is necessary for NRC staff to determine, in light of the new understanding of seismic hazards, whether additional regulatory action is warranted. The GI 199 issue has been subsumed into Fukushima NTTF recommendation 2.1 as described in subsequent sections.

1.2 NRC NTTF Recommendations

The NRC issued an information request on March 12, 2012 related to the Fukushima NTTF recommendations 2.1, 2.3, and 9.3 [1]. The requested seismic information associated with Recommendation 2.1 is stated to reflect:

- Information related to the updated seismic hazards at operating NPPs
- Information based on a seismic risk evaluation (SMA or seismic probability risk assessment (SPRA)), as applicable
- Information that would be obtained from an evaluation of the spent fuel pool (SFP)

The basic seismic information requested by the NRC is similar to that developed for GI-199 as presented in the draft GL for GI-199 [10]. The NRC has identified an acceptable process for responding to the 2.1 seismic requests, which is documented in Attachment 1 to the March 12, 2012 10CFR 50.54f letter [1]. The NRC asks each addressee to provide information about the current hazard and potential risk posed by seismic events using a progressive screening/evaluation approach. Depending on the comparison between the re-evaluated seismic hazard and the current design basis, the result is either no further risk evaluation or the performance of a seismic risk assessment. Risk assessment approaches acceptable to the staff, depending on the new hazard estimates, include a SPRA or an "NRC"-type of SMA that was described in NUREG-1407 [11] for IPEEEs, with enhancements.

1.3 Approach to Responding to Information Request for NTTF Recommendation 2.1

The approach described in this report has been developed by EPRI, working with experts from within the nuclear industry, with the intent of identifying reasonable measures that can be employed to reduce the resources that might be required to complete an effective seismic evaluation. More specifically, the approach was designed to constitute a specific path to developing a response to the request for information made in connection with NTTF Recommendation 2.1. This approach reflects careful consideration of the NRC's description of an acceptable approach for the seismic elements of Recommendation 2.1 (documented in Attachment 1 to Seismic Enclosure 1 of the March 12, 2012 Request for Information [1]). In general, the approach described in this report is intended to conform to the structure and philosophy of the nine steps suggested by the NRC and outlined in that attachment. Key elements of the approach are designed to streamline several of these nine steps (summarized below) while still yielding an appropriate characterization of the impact of any change in hazard for the plant being evaluated. Figure 1-1 illustrates the process for employing this approach; it is based on a progressive screening approach and is broken down into four major task areas:

- Seismic Hazard and Site Response Characterization
- GMRS Comparisons and Plant Screening
- Prioritization of Risk Assessments
- Seismic Risk Evaluation

The following paragraphs provide a brief discussion about each individual step in Figure 1-1. The subsequent sections of this guide contain the detailed descriptions of the methods and the documentation associated with this approach.

Comment [C3]: Consistency with Section 6.6? Highlighted portion limits applicability of SPID Guidance.

Comment [GSH4]: Should not limit applicability. Need careful wording in Section 6.6 to ensure not limiting.

Step 1. Develop site-specific ~~base rock and~~ control point elevation hazard curves over a range of spectral frequencies and annual exceedance frequencies determined from a probabilistic seismic hazard analysis (PSHA).

Comment [C5]: Since using Approach 3 (within integral) not producing base rock hazard curves

Step 2. Provide the new seismic hazard curves, the GMRS, and the SSE in graphical and tabular format. Provide soil profiles used in the site response analysis, as well as the resulting soil amplification functions.

Step 3. Utilize a screening process to eliminate certain plants from further review. If the SSE is greater than or equal to the GMRS at all frequencies between 1 and 10 Hz, then addressees may terminate the evaluation (Step 4) after providing a confirmation, if necessary, that SSCs which may be affected by high-frequency ground motion, will maintain their functions important to safety. A similar screening review based on the IPEEE High Confidence of Low Probability of Failure (HCLPF) Spectrum comparison to the GMRS can also be conducted. Diamonds 3a thru 3f outline the overall screening process, and Section 3 provides additional guidance.

Step 4. This step demonstrates termination of the process for resolution of NTTF Recommendation 2.1 for plants whose SSE is greater than the calculated GMRS.

Step 5. Based on criteria described in Section 6.2, perform a SPRA (steps 6a and 7a) or a SMA (steps 6b and 7b). Step 5 also describes the prioritization process for determining completion schedules for the seismic risk assessments.

Step 6a. If a SPRA is performed, it needs to be technically adequate for regulatory decision making and to include an evaluation of containment performance and integrity. This guide is intended to provide an acceptable approach for determining the technical adequacy of a SPRA used to respond to this information request.

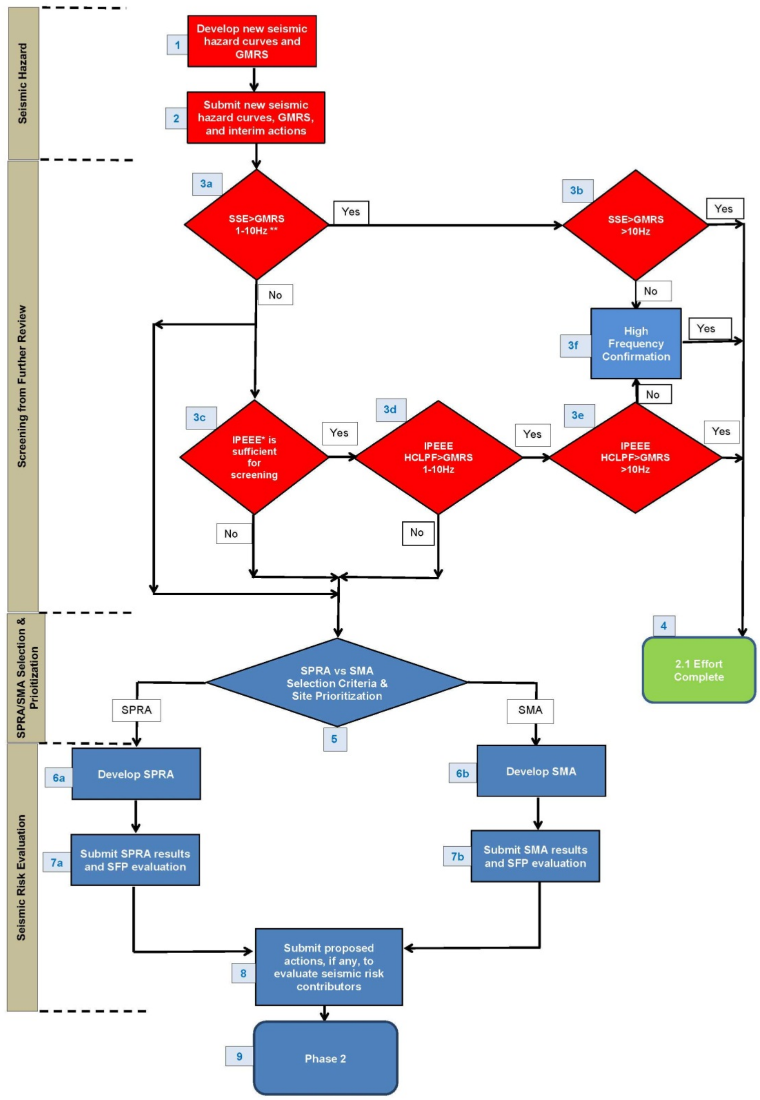
Step 6b. If a SMA is performed, it should use a composite spectrum review level earthquake (RLE), defined as the maximum of the GMRS and SSE at each spectral frequency. The SMA should also include an evaluation of containment performance and integrity. The American Society of Mechanical Engineers/American Nuclear Society (ASME/ANS) RA-Sa-2009 [12] provides an acceptable approach for determining the technical adequacy of a SMA used to respond to this information request. In addition, the NRC is generating an Interim Safety Guide (ISG) on the NRC SMA approach that will be acceptable for this 2.1 application (*Reference to be added when ISG published*).

Step 7a. Document and submit the results of the SPRA to the NRC for review. The "Requested Information" Section in the main body of Enclosure 1 [1] identifies the specific information that is requested. In addition, addressees are requested to submit an evaluation of the SFP integrity.

Step 7b. Document and submit the results of the SMA to the NRC for review. The "Requested Information" Section in the main body of Enclosure 1 [1] identifies the specific information that is requested. In addition, addressees should submit an evaluation of the SFP integrity.

Step 8. Submit plans for actions that evaluate seismic risk contributors. NRC staff, EPRI, industry, and other stakeholders will continue to interact to develop acceptance criteria in order to identify potential vulnerabilities.

Step 9. The information provided in Steps 6 through 8 will be evaluated in Phase 2 to consider any additional regulatory actions. *(Note – Phase 2 placeholder, further description to be provided)*



*Optional - Not Applicable to Reduced Scope plants; Focused Scope plants must upgrade to Full Scope before using this path
**Two special screening considerations are described in Section 3.2.1:

- GMRS Comparisons & Screening of Plants at Low Seismic Hazard Sites, and
- Narrow Band Exceedances in the 1 to 10 Hz Range.

KEY
Red boxes/diamonds are to be completed within 18 months of the issuance of the 50.54f Post-Fukushima letter.
Blue boxes/diamonds indicate completion beyond the 18 month period.

Figure 1-1
Recommended Approach to Respond to Information Request 2.1

Section 2: Seismic Hazard Development

2.1 Introduction and Background

Seismic hazard analysis and the calculation of up-to-date seismic response spectra is the first step to informed evaluations on priorities to mitigate seismic risk. To determine if a reevaluation of seismic risk for a nuclear power plant is appropriate, the comparison of the up-to-date seismic response spectra with the existing plants' seismic design spectra is the next step. Such a comparison should account for both relative and absolute differences between up-to-date seismic response spectra and the existing plants' seismic ruggedness, as characterized by the seismic design spectra.

The first major part of the Seismic Enclosure 1 of the March 12, 2012 Request for Information [1] is to calculate seismic hazard at existing plant sites by first calculating uniform hazard response spectra (UHRS), using up-to-date models representing seismic sources, ground motion equations, and site amplification. From the UHRS results, GMRS are calculated. Figure 2-1 depicts (for illustrative purposes only) the three basic elements of the seismic hazard analysis (seismic source characterization, ground motion attenuation, and site amplification), which will be described in more detail in the sections below.

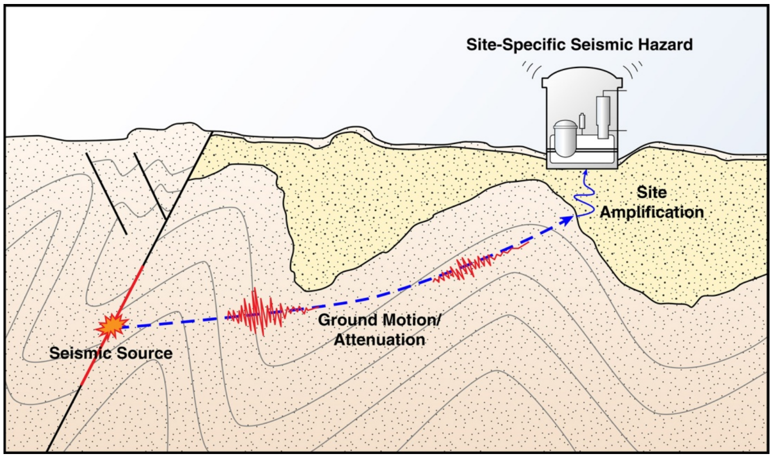


Figure 2-1
Steps to Obtain Site-Specific Seismic Hazard

2.2 Seismic Source Characterization

Seismic Sources for the CEUS – For the region designated the CEUS (United States east of the Rocky Mountains), a general study was conducted during the period 2009-2011 to develop a comprehensive representation of seismic sources for nuclear plant seismic evaluation purposes. The results were published in 2012 [14], were reviewed by the USNRC [15], and are an acceptable set of seismic sources to use for seismic hazard studies [23, p. 115]. This study was conducted as a Senior Seismic Hazard Analysis Committee (SSHAC) Level 3 study [13], meaning that a detailed step-by-step process was used to evaluate data and theories on earthquake occurrences, their potential locations and sizes, and the rates with which they might occur, and that process was documented and reviewed in a structured way. This ensured that all credible data and theories were appropriately considered. Specifically, detailed workshops were held that addressed the fundamental technical bases upon which models of seismic sources could be developed, and alternative models, with their technical bases, were defined. This applied to the geometries of seismic sources, as well as to the parameters of the sources (earthquake magnitude distributions, rates of activity, maximum magnitudes, and characteristics of faulting within the earth's crust). Alternative models and parameters were quantitatively weighted to express the credibility of each alternative. A Technical Integration team conducted these analyses and documented the derivation of weights so that a logic-tree approach (alternatives with weights) could be used to characterize the interpretations and their uncertainties. This set of interpretations forms the basis for characterizing the distribution of future earthquake occurrences in the CEUS. Because of the large regional study area of the CEUS Seismic Source Characterization project, detailed evaluations of geology, topography, and other data in the vicinity of NPPs was not undertaken.

Seismic sources were defined in the CEUS Seismic Source Characterization project in two categories. First were Repeated Large Magnitude Earthquake (RLME) sources, which represent sources where there is evidence of repeated, large-magnitude earthquakes. The two major RLME sources in the CEUS are the New Madrid seismic zone and the Charleston seismic zone. However, the CEUS Seismic Source Characterization project identified additional RLME sources on the basis of paleo-earthquake and other evidence.

The second category of seismic sources were background sources, which are large regions within which earthquakes are modeled as occurring according to an exponential magnitude distribution but where specific faults or causative structures have not been identified. Two sets of background sources were identified based on alternative methods to estimate maximum magnitude, and each set of background sources covers the entire CEUS (and surrounding territory). An updated earthquake catalog was created and used to estimate rates of activity within the sources, the rate of activity varying spatially to reflect the historical occurrences of small and moderate earthquakes. Thus, for example, sub-regions of the CEUS that have experienced relatively many historical earthquakes would have a higher rate of activity than sub-regions that have experienced relatively few historical earthquakes.

For site-specific licensing applications or site-specific safety decisions, these seismic sources would be reviewed on a site-specific basis to determine if they need to be updated. Such evaluations would be appropriate in a licensing application, where focus could be made on site-specific applications. However, for a screening-level study of multiple plants for the purpose of setting priorities, the use of these seismic sources as published is appropriate.

In addition, for applications in a regional study, it is sufficient to include background sources within 320 km (200 miles) of a site, and specifically to include only parts of those background sources that lie within 320 km of the site. This follows the guidance in [18] regarding examination of sources within the “site region” defined as the surrounding 320 km. For RLME sources, it is sufficient to include the New Madrid, Charlevoix, and the Charleston seismic zones if they lie within 1,000 km of a site. Beyond 1,000 km, ground motion equations have not been well-studied, and such distant earthquakes do not generally cause damage to modern engineered facilities. For other RLME sources and sub-regions of background sources with higher rates of activity, it is sufficient to include them in the analysis if they lie within 500 km of a site, based on test hazard results published in the CEUS Seismic Source Characterization project.

Seismic Sources for the WUS– For Western United States (WUS) plants, designation of seismic sources is much more site-specific. These sites are Diablo Canyon and San Onofre in California, Palo Verde in Arizona, and Columbia in Washington. For the California sites, local faults dominate the seismic hazard; for the Columbia site, local faults, background sources, and subduction zone earthquakes are a consideration. For the Arizona site, background sources and distant faults (including the San Andreas Fault) are important. The development of seismic sources should be made on a site-specific basis for these four sites by conducting a SSHAC Level 3 study [13].

2.3 Ground Motion Attenuation

Ground Motion Estimates for the CEUS – In 2004, EPRI [16] published a set of ground motion prediction equations (GMPEs) for the CEUS, which included both aleatory and epistemic uncertainties. In 2006, EPRI [17] published an updated set of aleatory uncertainties to use with the 2004 equations. These GMPEs estimate the aleatory and epistemic uncertainty in ground motion for the mid-continent region of the CEUS and for the Gulf of Mexico region~~, and are an appropriate starting point for seismic hazard studies~~. ~~There are some CEUS NPPs that are currently developing SPRAs. Consistent with the current SPRA standard requirement of using the most recent seismic hazard information, they are using the EPRI 2004-2006 ground motion attenuation model with the CEUS Seismic Source Characterization model for the seismic hazard portion of their SPRAs.~~

Beginning in 2012, EPRI has been evaluating the 2004-2006 GMPEs in light of new ground motion models published in the technical literature and in light of recorded ground motion data obtained during earthquakes in the CEUS and south-eastern Canada. The overall goals of the project are to determine (a) if the 2004-2006 GMPEs should be updated in light of the new models and data, and (b) if so, how to quantitatively update those GMPEs so they reflect the new information. A decision to update the 2004-2006 GMPEs was confirmed on August 14, 2012, and the updated models are expected in mid-February 2013.

It is anticipated that, as in EPRI 2004-2006, multiple models with weights will be determined for the 2013 updated GMPEs and for the aleatory uncertainties. It is also anticipated that equations will be developed for the two regions (mid-continent and Gulf of Mexico). In cases where the travel path of seismic waves between a potential earthquake source and a site is predominantly in one region, equations for that region should be used. In cases where the travel path crosses from one region to the other, with a substantial fraction of the total travel path of seismic waves in each region, hazard calculations can be made using either the more conservative equations, or using a weighted average of hazard results based on the approximate fraction that seismic waves travel through each region.

~~Since~~ Because the EPRI 2012 ground motion update project is proceeding with updating the EPRI 2004-2006 GMPEs, those updated equations, if approved by the NRC, should be used to calculate ground motions for seismic hazard calculations for all CEUS sites for Step 2 "Submit new seismic hazard curves, GMRS, and interim actions." Otherwise the EPRI 2004-2006 GMPEs should be used.

Currently some CEUS NPPs are developing SPRAs. Consistent with the current SPRA standard requirement of using the most recent seismic hazard information, they are using the EPRI 2004-2006 ground motion attenuation model with the CEUS Seismic Source Characterization model for the seismic hazard portion of their SPRAs.

~~Subsequent SPRAs which will be developed under Step 5 should use the most recent seismic hazard models (e.g., updated EPRI 2004-2006 GMPEs) if approved by the NRC.~~ Th~~o~~ese CEUS NPPs ~~that are currently completing SPRAs that have used the EPRI 2004-2006 ground motion attenuation model with the CEUS Seismic Source Characterization model for the seismic hazard portion of their SPRAs~~ should, in Step 7a, address the effect of the new site hazard based on the updated EPRI 2004-2006 GMPEs.

Ground Motion Estimates for the WUS – In the WUS, earthquake ground motions can be estimated using recorded motions, and the seismic hazard is often dominated by the possible occurrence of a moderate-to-large earthquake at close distances. There are published GMPEs available, the "Next Generation Attenuation," or NGA, equations, but these will be updated in the next several years by the NGA-2 equations. Nuclear plant sites in the WUS should perform a SSHAC Level 3 study [13] in order to make site-specific decisions on which equations are appropriate for their sites or to develop site-specific relationships.

2.4 Site Seismic Response

Every site that does not consist of hard rock should conduct an evaluation of the site amplification that will occur as a result of bedrock ground motions traveling upward through the soil/rock column to the surface. Critical parameters that determine which frequencies of ground motion might experience significant amplification (or de-amplification) are the layering of soil and/or soft rock, the thicknesses of these layers, the initial shear modulus and damping of these layers, their densities, and the degree to which the shear modulus and damping change with increasing ground motion. The methods to calculate possible site amplification are well-established, but at some sites the characterization of the profile and layering is limited. For these sites, analyses must be conducted, as described below, that account for uncertainties in soils and layer properties, and this often results in significant uncertainties in site amplification. This Section also provides a method for defining the elevation(s) for the SSE to GMRS comparison for use in the 2.1 seismic screening.

2.4.1 Site Response for Sites with Limited Data

Many sites, particularly those licensed in the early 1970s, do not have detailed, measured soil and soft-rock parameters to extensive depths. These sites will be handled using the following guidelines (see Appendix B for a more detailed discussion).

Shear-wave Velocity (V_s) – For soil sites where V_s is estimated from compression-wave measurements, or was measured only at shallow depths, template profiles will be used based on experience with other, well-documented sites. The template profiles will be adjusted and/or truncated to be consistent with measured or estimated V_s in the upper 30 m of soil, called V_{s30}, to obtain a reasonable profile to use for analysis that includes the potential effects on ground motion of soils at large depths.

For firm rock sites (typically underlain by sedimentary rocks) that have little measured V_s data, a V_s profile will be adopted that is consistent with shallow estimates or measurements and that increases with depth using a gradient typical of sedimentary rocks. A consistent gradient has been documented for sedimentary rock sites in various locations around the world, and a profile developed in this way will give reasonable results for the potential effects on ground motion of sedimentary rock at large depths.

For sites with limited, or indirect data on V_s, multiple profiles or base cases should be developed to account for the epistemic uncertainty. Typically three base cases should be developed. To account for the variability in V_s over the scale of the footprint of a NPP, which is treated as an aleatory uncertainty, randomization about the base cases should be implemented. Additional discussion regarding the methodology to incorporate the various types of uncertainty is provided in Appendix B.

Dynamic Soil and Soft-rock Properties – Other soil and soft-rock properties such as dynamic moduli, hysteretic damping, and kappa (a measure of inherent near surface site damping) will be adopted using published models. The same will be done for soil and soft-rock densities, if they have not been measured and reported.

2.4.2 Horizons and SSE Control Point

This Section provides a method for defining the elevation(s) for the SSE to GMRS comparison for use in the 2.1 seismic screening. The SSE to GMRS comparison for 2.1 screening per the 50.54(f) letter are recommended to be applied using the licensing basis definition of SSE control point. The SSE is part of the plant licensing basis which is typically documented in the FSAR. Three specific elements are required to fully characterize the SSE:

- Peak Ground Acceleration
- Response Spectral Shape
- Control Point where the SSE is defined

The first two elements of the SSE characterization are normally available in the part of the Final Safety Analysis Report (FSAR) that describes the site seismicity (typically Section 2.5). The control point for the SSE is not always specifically defined in the FSAR and, as such, guidance is required to ensure that a consistent set of comparisons are made. Most plants have a single SSE, but several plants have two SSEs identified in their licensing basis (e.g., one at rock and one at top of a soil layer).

For purposes of the SSE-to-GMRS comparisons as part of the 50.54(f) 2.1 seismic evaluations, the following criteria are recommended to establish a logical comparison location:

1. If the SSE control point(s) is defined in the FSAR, use as defined.
2. If the SSE control point is not defined in the FSAR then the following criteria should be used:
   a. For sites classified as soil sites with generally uniform, horizontally layered stratigraphy and where the key structures are soil-founded (Figure 2-2), the control point is defined as the highest point in the material where a safety-related structure is founded, regardless of the shear wave velocity.
   b. For sites classified as a rock site or where the key safety-related structures are rock-founded (Figure 2-3), then the control point is located at the top of the rock.
   c. The SSE control point definition is applied to the main power block area at a site even where soil/rock horizons could vary for some smaller structures located away from the main power block (e.g., an intake structure located away from the main power block area where the soil/rock horizons are different).

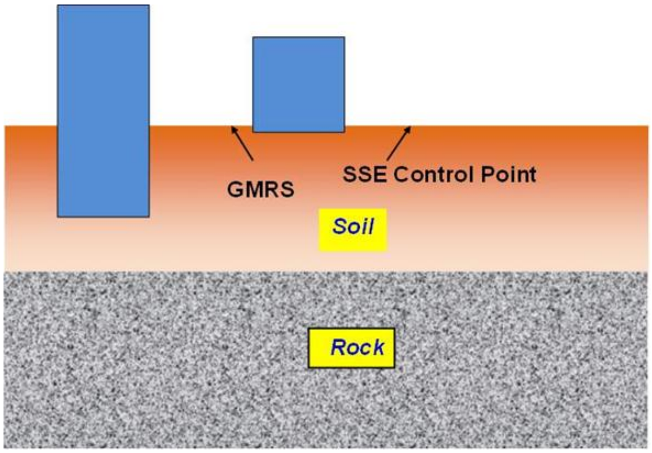


Figure 2-2
Soil Site Example

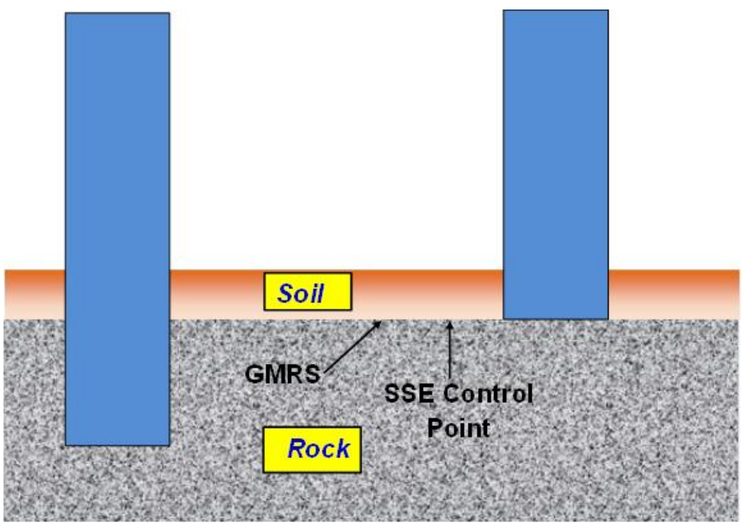


Figure 2-3
Rock Site Example

The basis for the selected control point elevation should be described in the submittal to the NRC. Deviations from the recommendations described below should also be documented.

2.5 Hazard Calculations and Documentation

2.5.1 PSHA and Hazard Calculations

The PSHA will proceed with (1) the CEUS Seismic Source Characterization models [14] or a regional WUS seismic source characterization (Section 2.2 above), with (2) GMPEs for the CEUS or the WUS (Section 2.3 above), and with (3) a site seismic response analysis (quantified as described in Section 2.4 and Appendix B). Several assumptions are appropriate regarding the PSHA calculations as follows:

For CEUS sites, seismic sources should be included for the range of distances indicated in Section 2.2. For WUS sites, the Technical Integration team for the SSHAC Level 3 study with input from the Participatory Peer Review Panel should determine which seismic sources should be included in the PSHA.

As indicated in Section 2.3, for the CEUS the updated EPRI GMPEs should be used for purposes of the 50.54(f) 2.1 seismic evaluations, if approved by the NRC, otherwise, the EPRI 2004-2006 ground motion models [16, 17] should be used. In addition, estimates of ground motion for source-site configurations with seismic wave travel paths across both the mid-continent and Gulf of Mexico regions should be handled as described in Section 2.3. For the WUS, a SSHAC Level 3 study should be performed to select or develop appropriate GMPEs.

For the purposes of responding to the Seismic Enclosure 1 of the March 12, 2012 Request for Information [1], updates to seismic sources to account for historical seismicity since 2008 (the last year of the earthquake catalog in the CEUS Seismic Source Characterization study) are not required. Similarly, updates to seismic sources to account for more recent earthquakes are not necessary.

The CAV (Cumulative Absolute Velocity) filter developed by EPRI [19] may be applied to account for the damageability of ground motions from small magnitude earthquakes. However, if the CAV filter is applied, the lower-bound magnitude for the PSHA should be set at **M** 4.0, and the CAV model should not be applied for **M** greater than 5.5 (see Attachment 1 to Seismic Enclosure 1 of Reference [1]). In place of the CAV filter a minimum magnitude of **M** 5.0 may be used.

Site amplification factors should be calculated as described in Section 2.4. As discussed in that section, multiple models of site amplification factors (and associated uncertainties) should be developed, indicating the log-mean and log-standard deviation of control-point motion divided by input rock motion, for various spectral frequencies. For input to site hazard calculations, these multiple models should be combined, with weights, to derive the overall log-mean and log-standard deviations of site amplification for each spectral frequency, as described in Appendix B.

The soil uncertainties should be incorporated into the seismic hazard calculations using a formulation similar to Eq. (6-5) in [24], wherein the site amplifications (with uncertainties) are incorporated into the hazard integral to estimate the distribution of site amplitudes given earthquake magnitude and distance. The implementation should estimate the distribution of rock amplitude as a function of M and R, and the site amplification (given the rock amplitude) for the value of M at which site amplifications were calculated. This is sufficiently accurate since site amplifications are not highly dependent on M and R.

The control-point elevation seismic hazard curves should be used to calculate a GMRS for the site, using the method of [21]. The GMRS depends, in this calculation, on the 10^{-4} and 10^{-5} spectral accelerations at each spectral frequency. The control point should be defined at the same elevation as the design basis SSE. Given that the site amplification factors are calculated assuming free-surface conditions above the control point, the GMRS will be consistent with that assumption.

2.5.2 Seismic Hazard Data Deliverables

Soil Profile and Properties – A description of the development of the base case profile as it relates to the local geology should be described. In addition, for each base case, the soil profile used to calculate site amplification factors should be described, including layer boundaries, properties (Vs and density), modulus and damping curves used for each layer, and uncertainties in these properties.

Site Amplification Factors – Site amplification factors should be documented as log-mean amplification factors and log-standard deviations of amplification factors as a function of input rock acceleration, for the spectral frequencies at which GMPEs are defined.

2.5.3 Seismic Hazard Data at Control Points and Base-Rock

Hazard Data at Control Point~~s and Base-rock~~ – Seismic hazard curves should be documented for ~~base-rock conditions and for~~ the control-point elevation corresponding to the mean hazard and common fractiles ~~and to fractiles of 0.05, 0.16, 0.5, 0.84, and 0.95~~. These curves should represent seismic hazard at the spectral frequencies for which GMPEs are available. ~~Both the base-rock and~~The control-point elevation hazard curves should be represented for annual exceedance frequencies from 10^{-3} to 10^{-7}. Hazard curves should be provided in graphical and tabular format along with the site response amplification function, SSE and GMRS.

Comment [E6]: Hard rock hazard curves only for Method 2. Method 3 will produce control pt hazard curves

Comment [C7]: Modifications to paragraph reflect use of Method 3 (within hazard integral)

Section 3: GMRS Comparisons and Screening of Plants

3.1 Background on Screening

Following completion of the updated seismic hazard as described in Section 2, a screening process is needed to determine which plants are required to perform new seismic risk evaluations. The horizontal GMRS calculation discussed/defined in Section 2 is being used to characterize the amplitude of the new seismic hazard at each NPP site, as defined by the NRC [1]. The GMRS should be compared to the horizontal 5% critical damped SSE as shown in Diamonds 3a and 3b of Figure 1-1. If the SSE is exceeded, then licensees may have the option to perform the screening of the GMRS to the IPEEE HCLPF spectrum (IHS). The IHS is the response spectrum corresponding to the HCLPF level documented from the seismic IPEEE program, as shown in Diamonds 3c through 3e in Figure 1-1. The use of the IHS for screening is contingent upon satisfying specific adequacy criteria, as described in Section 3.3. This screening process, along with examples, is described in more detail in the Sections below.

3.2 SSE Screening Task (SSE-to-GMRS Comparison)

The SSE is the plant licensing basis earthquake and is uniquely defined for each NPP site. The SSE consists of:

- A PGA value which anchors the response spectra at high frequencies (typically 33 Hz for the existing fleet of NPPs),
- A response spectrum shape which depicts the amplified response at all frequencies below the PGA (typically plotted at 5% damping), and
- The control point applicable to the SSE (described in Section 2 of this report). It is essential to ensure that the control point for both the SSE and for the GMRS is the same.

The first step in the SSE screening process is to compare the SSE to the GMRS in the 1 to 10 Hz part of the response spectrum (see Diamond 3a in Figure 1-1). If the SSE exceeds the GMRS in the 1 to 10 Hz region, then a check of the greater than 10 Hz part of the spectrum is performed as shown in Diamond 3b. If the SSE exceeds the GMRS in the greater than 10 Hz region, then no further action is required for NTTF Recommendation 2.1 seismic review (Box 4 in Figure 1-1). If there are exceedances in the greater than 10 Hz region, then a high-frequency confirmation should be performed (Box 3f in Figure 1-1) as described in Section 3.4.

An example comparison of an SSE with a GMRS is shown in Figure 3-1. In this example, only a high frequency confirmation is needed.

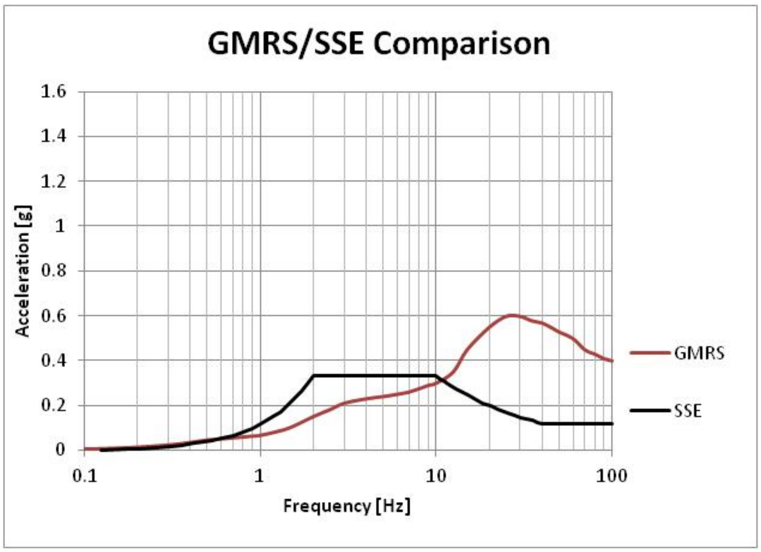


Figure 3-1
Example Comparison of GMRS to SSE (5% Damping)

If the initial review of the SSE to GMRS (Diamond 3a in Figure 1-1) does not demonstrate that the SSE envelops the GMRS in the 1 to 10 Hz region, then, depending upon the nature of the exceedance, the licensees have the option of:

1) Conducting a screening evaluation for narrow band exceedances as described in Section 3.2.1, or

2) Conducting a screening evaluation using the IPEEE HCLPF capacity as described in Section 3.3, or

3) Bypassing the screening evaluations and performing the seismic risk evaluation using either an SPRA or SMA approach, as appropriate, as described in Section 6 of this report.

3.2.1 Special Screening Considerations

There are two special screening considerations:

- GMRS Comparisons and Screening of Plants at Low Seismic Hazard Sites
- Narrow Band Exceedances in the 1 to 10 Hz Range

3.2.1.1 GMRS Comparisons and Screening of Plants at Low Seismic Hazard Sites

A low seismic hazard site is defined herein to be a site where the GMRS peak 5% damped spectral acceleration (SA_p) at frequencies between 1 and 10 Hz do not exceed 0.4g, which is shown in Figure 3-2 as the Low Hazard Threshold (LHT). Because of the low likelihood of any seismically designed SSC being damaged by ground motion with an SA_p less than this LHT, the following relief from having to perform a full SMA or SPRA is considered to be warranted for plants at sites where the GMRS is less than this LHT in the 1 to 10 Hz range.

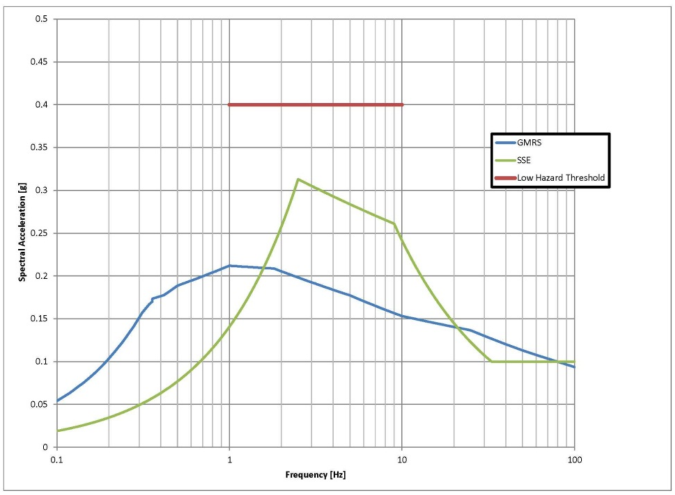


Figure 3-2
Example Comparison of GMRS to SSE and LHT (5% Damping)

Figure 3-2 shows an example where the SSE spectral accelerations exceed the GMRS spectral accelerations at frequencies below 10 Hz except for low frequencies. Because the SSE response spectral accelerations reduce rapidly as frequencies reduce below 2.5 Hz, the situation shown in

Figure 3-2 can occur at low seismic hazard sites. For most SSCs, such exceedance below 2.5 Hz is non-consequential because the fundamental frequency of these SSCs exceeds 2.5 Hz.

Low-frequency exceedances (below 2.5 Hz) at low seismic hazard sites (SA_p less than LHT) do not require a plant to perform a full SMA or SPRA. Instead, it is sufficient to first identify all safety-significant SSCs that are potentially susceptible to damage from spectral accelerations at frequencies below which the highest frequency f_L (f_L < 2.5 Hz) acceleration exceeds the SSE spectral acceleration. Examples of SSCs and failure modes potentially susceptible to damage from spectral accelerations at low frequencies are:

1) Liquid sloshing in atmospheric pressure storage tanks
2) Very flexible distribution systems with frequencies less than f_L
3) Sliding and rocking of unanchored components
4) Fuel assemblies inside the reactor vessel
5) Soil liquefaction

After identifying all safety-significant SSCs that are potentially susceptible to lower frequency accelerations, the HCLPF to GMRS seismic margin needs to be computed and reported. As long as the HCLPF/GMRS ratio for all of these potentially low-frequency susceptible SSCs exceeds unity, the plant is screened out from having to perform additional seismic evaluations.

Comment [C8]: This isn't clear. PGAs?

Comment [GSH9]: PGA comparison should not be required. This is a 1 to 10 hz assessment only.

Comment [E10]: Re-write in terms of IHS

If the IPEEE HCLPF capacity evaluations are considered to be sufficient for screening (as described in Section 3.3.1), the IPEEE HCLPF response spectral accelerations may be used for this HCLPF/GMRS comparison for screening potentially low-frequency susceptible SSCs at low seismic hazard sites. The IPEEE HCLPF response spectral accelerations also reduce rapidly as frequencies reduce below 2.5 Hz so that the GMRS spectral accelerations might also exceed the HCLPF spectral accelerations at low frequencies. In this case, new HCLPF capacities can be computed for these potentially low-frequency susceptible SSCs using the GMRS response spectrum shape instead of the IPEEE response spectrum.

3.2.1.2 Narrow Band Exceedances in the 1 to 10 Hz Range

If the GMRS exceeds the SSE in narrow frequency bands anywhere in the 1 to 10 Hz range, the screening criterion is as follows: In the 1 to 10 Hz range, a point on the GMRS may fall above the SSE by up to 10% provided the average ratio of GMRS to SSE in the adjacent 1/3 octave bandwidth (1/6 on either side) is less than unity. There may be more than one such exceedance point above the SSE in the 1 to 10 Hz range provided they are at least one octave apart. Figure 3-3 shows an example of this narrow-band criterion. If the GMRS meets the criteria, no SMA or SPRA is required for the NTTF Recommendation 2.1 seismic review.

If the IPEEE HCLPF capacity evaluations are considered to be of sufficient quality for screening, the IPEEE HCLPF response spectral accelerations may be used for a HCLPF/GMRS comparison in narrow frequency bands. In this case, the SSE is replaced by the IPEEE-HCLPF spectrum to determine if a plant can be screened-out from further seismic review.

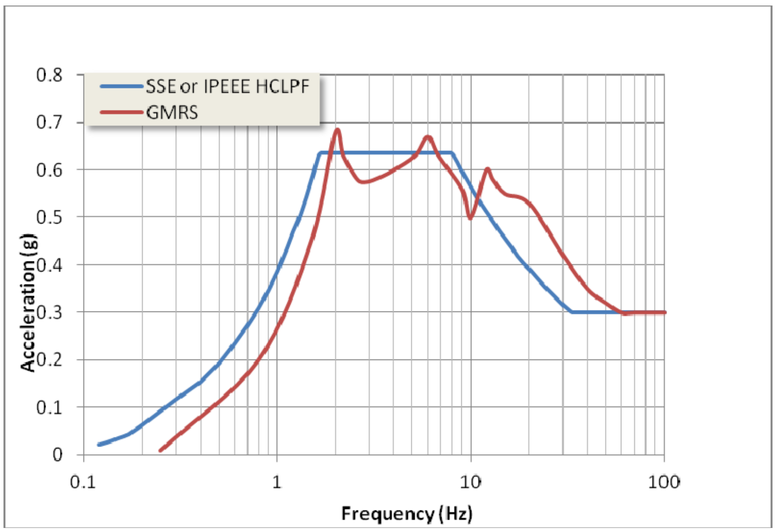


Figure 3-3
Screening – Example Narrow Exceedances at 2 Hz and 6 Hz (5% Damping)

3.3 IPEEE Screening Task

The second method to demonstrate plant seismic adequacy based on screening from further review consists of a comparison of the GMRS to the IPEEE HCLPF spectrum, which is described in Section 3.3.2 below. The use of the IPEEE HCLPF spectrum in the screening process is depicted in Boxes 3c, 3d, and 3e in Figure 1-1.

For plants that conducted an SPRA, focused scope SMA, or full scope SMA during the IPEEE, the screening is an optional approach that consists of the comparison of the ~~IPEEE HCLPF spectrum (~~IHS~~)~~ to the new GMRS. If the IPEEE HCLPF is used for screening, the IPEEE will be required to pass an adequacy review (Diamond 3c in Figure 1-1). If the IPEEE demonstrates sufficient quality, the next step in this screening process is to compare the IHS to the GMRS in the 1 to 10 Hz part of the response spectrum (see Diamond 3d in Figure 1-1). If the IHS exceeds the

Comment [C11]: May want to define and use this acronym in preceeding sections.

Comment [GSH12]: Agree, defined at the start of section 3.1

GMRS in the 1 to 10 Hz region, then a check of the greater than 10 Hz part of the spectrum is performed, as shown in Diamond 3e. If the IHS exceeds the GMRS in the greater than 10 Hz region, then no further action is required for the NTTF 2.1 seismic review (Box 4 in Figure 1-1). If there are exceedances in the greater than 10 Hz region, then a high-frequency confirmation should be performed (Box 3f in Figure 1-1) as described in Section 3.4.

3.3.1 IPEEE Adequacy

Background

Seismic risk assessments performed as part of the Individual Plant Examination of External Events (IPEEE) for Severe Accident Vulnerabilities (Generic Letter 88-20, Supplement 4) [2] that demonstrate plant capacity to levels higher than the new GMRS can be used to "screen out" plants, provided they meet certain criteria, in which case these plants would not need to perform new seismic risk analyses. IPEEE submittals using either SPRA or SMA analyses can be considered for screening, but in either case the analysis must have certain attributes to be considered for review by the NRC staff.

Use of IPEEE Results for Screening

Certain criteria are necessary if licensees choose to screen a facility based on IPEEE results. The criteria for screening have been grouped into four categories:

- General Considerations
- Prerequisites
- Adequacy Demonstration
- Documentation

Responses to the items in the Prerequisite and Adequacy Demonstration categories should be provided in the hazard submittal to the NRC.

General Considerations

IPEEE reduced scope margin assessments cannot be used for screening. Focused scope margin submittals may be used after having been enhanced to bring the assessment in line with full scope assessments. The enhancements include (1) a full scope detailed review of relay chatter for components such as electric relays and switches, and (2) a full evaluation of soil failures, such as liquefaction, slope stability, and settlement.

The spectrum to be compared to the GMRS for screening purposes should be based on the plant-level HCLPF actually determined by the IPEEE and reported to the NRC. If this is less than the review level earthquake (RLE) spectrum, then the RLE must be shifted appropriately to reflect the actual

HCLPF. In cases where modifications were required to achieve HCLPF submitted in the IPEEE, verify the changes (and describe the current status) in the submittal. This information is also required as part of the Recommendation 2.3 seismic walkdown. Similarly, the uniform hazard spectrum (UHS) for IPEEE SPRA should be anchored at the plant-level HCLPF.

Prerequisites

Responses to the following items should be provided with the hazard evaluation. In order to use the IPEEE analysis for screening purposes and to demonstrate that the IPEEE results can be used for comparison with the GMRS:

1) Confirm that commitments made under the IPEEE have been met. If not, address and close those commitments.
2) Confirm whether all of the modifications and other changes credited in the IPEEE analysis are in place.
3) Confirm that any identified deficiencies or weaknesses to NUREG-1407 [11] in the plant specific NRC Safety Evaluation Report (SER) are properly justified to ensure that the IPEEE conclusions remain valid.
4) Confirm that major plant modifications since the completion of the IPEEE have not degraded/impacted the conclusions reached in the IPEEE.

If any of the four above items are not confirmed and documented in the hazard submittal to the NRC, then the IPEEE results may not be adequate for screening purposes even if responses are provided to the adequacy criteria provided below.

Adequacy Demonstration

The following items, and the information that should be provided, reflect the major technical considerations that will determine whether the IPEEE analysis, documentation, and peer review are considered adequate to support use of the IPEEE results for screening purposes.

With respect to each of the criteria below, the submittal should describe the key elements of (1) the methodology used, (2) whether the analysis was conducted in accordance with the guidance in NUREG-1407 [11] and other applicable guidance, and (3) a statement, if applicable, as to whether the methodology and results are adequate for screening purposes. Each of the following should be addressed in the submittal to the NRC.

1) Structural models and structural response analysis (use of existing or new models, how soil conditions including variability were accounted for)

2) In-structure demands and ISRS (scaling approach or new analysis)

3) Selection of seismic equipment list or safe shutdown equipment list

4) Screening of components

5) Walkdowns

6) Fragility evaluations (generic, plant-specific analysis, testing, documentation of results)

7) System modeling (diversity of success paths, development of event and fault trees, treatment of non-seismic failures, human actions)

8) Containment performance

9) Peer review (how peer review conducted, conformance to guidance, peer review membership, peer review findings and their disposition)

Documentation

Licensees that choose to implement the use of the IPEEE results for screening purposes should provide a response for each of the criteria in the Prerequisite and Adequacy Demonstration categories in their hazard submittal to the NRC. Licensees should also provide an overall conclusion statement asserting that the IPEEE results are adequate for screening and that the risk insights from the IPEEE are still valid under current plant configurations. The information used by each licensee to demonstrate the adequacy of the IPEEE results for screening purposes should be made available at the site for potential staff audit.

3.3.2 Development of HCLPF Spectrum

The IHS is developed directly from the plant HCLPF capacity established in the IPEEE program. The IPEEE-reported HCLPF values were typically calculated by each plant during the 1990s and documented in the IPEEE submittal reports sent to the NRC by the licensees. These HCLPF values for many of the plants are also documented in NUREG-1742, "Perspectives Gained from the Individual Plant Examination of External Events (IPEEE) Program," April 2002 [4]. For those plants that performed an SMA, the IHS is anchored to the lowest calculated HCLPF of any SSC, and the shape of the IHS is consistent with the RLE used for the SMA (typically the NUREG/CR- 0098 shape). For those plants that conducted an SPRA as part of the IPEEE program, a plant HCLPF value was typically calculated (or can be calculated) from the plant core damage frequency (CDF) and the IHS should be anchored at that value. The shape of the IHS should correspond to the UHS associated with the seismic hazard utilized within the SPRA. Typically, the shapes of the UHS are similar between the 10^{-4} and the 10^{-5} return period UHS and, thus, either shape could be used for the purpose of generating the IHS. These two return periods are considered to be the appropriate ones for use in the generation of the IHS since the cumulative distribution of the contribution to the CDF has typically been shown to be centered in this return period range.

3.3.3 *Comparison of IPEEE HCLPF Spectrum to GMRS*

An example of the comparison of a GMRS to the IHS is shown in Figure 3-4. The IHS exceeds the GMRS in the 1 to 10 Hz range, and thus the lower frequency criteria (Diamond 3d of Figure 1-1) have been met. However, for this example, the higher frequency criteria (Diamond 3e in Figure 1-1) have not been met since the GMRS exceeds the IHS in this range. It is noted that (a) the control point for the IHS will typically be defined in a similar way as for the SSE, which is described in Section 2.4.1, and (b) the treatment of Narrow Band Exceedance is the same as discussed in Section 3.2.1 for SSE.

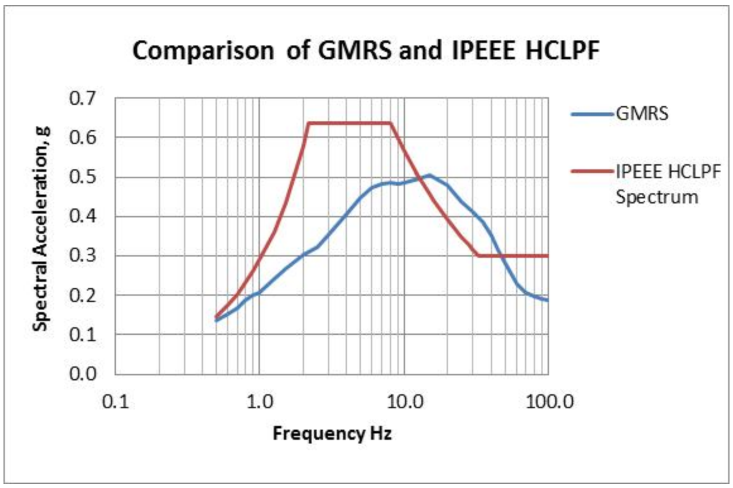


Figure 3-4
Example Comparison of GMRS to IHS (5% Damping)

3.4 Treatment of High-Frequency Exceedances

Equipment important to safety within operating NPPs has been seismically qualified for the SSE defined for each plant. The equipment has also been evaluated, in general, for a RLE under each plant's IPEEE program. The SSE and RLE ground motions, however, do not typically include significant frequency content above 10 Hz. Seismic hazard studies conducted in the late 1990s developed UHS that had spectral peaks occurring in the 20 to 30 Hz range. EPRI Report NP-7498, "Industry Approach to Severe Accident Policy Implementation," November 1991 [26], included an appendix titled "Recommended Procedures to Address High-Frequency Ground Motions in Seismic Margin Assessment for Severe Accident Policy Resolution." This appendix reviewed the bases for concluding that high-frequency motions were, in general, non-damaging to components and structures that have strain- or stress-based potential failures modes. It concluded that components, such as relays and other devices subject to electrical functionality failure modes, have unknown acceleration sensitivity for frequencies greater than 16 Hz. Thus, the evaluation of high-frequency vulnerability was limited to components that are subject to intermittent states.

In the IPEEE program, the consideration of high-frequency vulnerability of components was focused on a list of "bad actor" relays mutually agreed to by the industry and the NRC, with known earthquake or shock sensitivity. These specific model relays, designated as low ruggedness relays were identified in EPRI Report 7148, "Procedure for Evaluating Nuclear Power Plant Relay Seismic Functionality," December 1990 [27]. Rather than considering high-frequency capacity vs. demand screening, relays on this list were considered program outliers and were evaluated using circuit analysis, operator actions, or component replacements.

EPRI published the following reports during initial new plant licensing activities to provide additional information regarding the potential high-frequency vulnerability of NPP SSCs:

- EPRI Report 1015108, "Program on Technology Innovation: The Effects of High-Frequency Ground Motion on Structures, Components, and Equipment in Nuclear Power Plants," June 2007 [28].
- EPRI Report 1015109, "Program on Technology Innovation: Seismic Screening of Components Sensitive to High-Frequency Vibratory Motions," October 2007 [29].

Report 1015108 [28] summarized a significant amount of empirical and theoretical evidence, as well as regulatory precedents, that support the conclusion that high-frequency vibratory motions above about 10 Hz are not damaging to the large majority of NPP structures, components, and equipment. An exception to this is the functional performance of vibration sensitive components, such as relays and other electrical and instrumentation devices whose output signals could be affected by high-frequency excitation. Report 1015109 [29] provided guidance for identifying and evaluating potentially high-frequency sensitive components for plant applications that may be subject to possible high-frequency motions.

In response to the current NTTF activities, EPRI has established a test program to develop data to support the high frequency confirmation in Step 3f of Figure 1-1 as well as fragility data for a SPRA (Step 6a) or SMA (Step 6b) of Figure 1-1 for potential high-frequency sensitive components. The test program will use accelerations or spectral levels that are sufficiently high to address the anticipated high-frequency in-structure and in-cabinet responses of various plants. Therefore, it will not be necessary for those plants where GMRS > SSE or IHS only above 10 Hz to perform dynamic analysis of structures to develop ISRS.

3.4.1 Scope of High-Frequency Sensitive Components

The following types of failure modes of potentially high-frequency sensitive components and assemblies have been observed in practice:

- Inadvertent change of state
- Contact chatter
- Change in output signal or set-point
- Electrical connection discontinuity or intermittency (e.g., insufficient contact pressure)
- Mechanical connection loosening
- Mechanical misalignment/binding (e.g., latches, plungers)
- Cyclic strain effects (e.g., cracks in solder joints)
- Wiring not properly restrained
- Inadequately secured mechanical fasteners and thumb screw connections

These failure modes are considered below to determine the appropriate scope of potentially high-frequency sensitive components requiring additional information to perform the NTTF 2.1 seismic screening in Figure 1-1, Step 3f.

3.4.1.1 EPRI 1015109 Potentially High Frequency Sensitive Components

EPRI Report 1015109 [29] reviewed potentially high-frequency sensitive components and recommended change of state, contact chatter, signal change/drift, and intermittent electrical connections as the most likely failure modes. These are the first four failure modes highlighted in the above list.

Failures resulting from improper mounting design, inadequate design connections and fasteners, mechanical misalignment/binding of parts, and the rare case of subcomponent mechanical failure, are associated with the same structural failure modes as those experienced during licensing basis qualification low frequency testing conducted in accordance with the Institute of Electrical and Electronics Engineers (IEEE) Standard 344 [25]. Because the equipment experiences higher stresses and deformations when subjected to low-frequency excitation, these failure modes are more likely to occur under the low-frequency qualification testing.

The evaluation of potentially high-frequency sensitive components in new plants was therefore directed to mechanically actuated bi-stable devices, such as relays, contactors, switches, potentiometers and similar devices, and those components whose output signal or settings (set-points) could be changed by high-frequency vibratory motion. Table 3-1 shows the components identified in EPRI Report 1015109 [29] as being potentially sensitive to high-frequency motion.

3.4.1.2 AP1000 Potentially High Frequency Sensitive Equipment

During licensing reviews for the AP1000, Westinghouse and the NRC identified a broader list of potentially high-frequency sensitive components and assemblies (Table 3-2) to be evaluated in the AP1000 Design Control Document [30].

Table 3-1
EPRI 1015109 Potentially High Frequency Sensitive Items

▪ Electro-mechanical relays (e.g., control relays, time delay relays, protective relays) ▪ Circuit breakers (e.g., molded case and power breakers – low and medium voltage) ▪ Control switches (e.g., benchboard, panel, operator switches) ▪ Process switches and sensors (e.g., pressure, temperature, flow, limit/position)	▪ Electro-mechanical contactors (e.g., MCC starters) ▪ Auxiliary contacts (e.g., for MCCBs, fused disconnects, contactors/starters) ▪ Transfer switches (e.g., low and medium voltage switches with instrumentation) ▪ Potentiometers (without locking devices) ▪ Digital/solid state devices (mounting and connections only)

The primary difference between the list of components in EPRI 1015109 [29] and the AP1000 list [30] is that the EPRI 1015109 list is focused on potentially sensitive subcomponents, and the AP1000 list is focused on assemblies that would include those subcomponents. For example, the potentially sensitive parts of a Battery Charger or a 250 Vdc Motor Control Center are the relays, switches, and contactors noted in the EPRI 1015109 component list [29]. Therefore, evaluating the potential sensitivity of the items in the EPRI 1015109 list would also address the items in the AP1000 list.

Three key exceptions on the AP1000 list [30] are transformers, batteries, and valves (motor-operated valves (MOVs), air-operated valves (AOVs), solenoid valves (SVs). Transformers are primarily passive systems with strain- or stress-based potential failures modes. Some transformers may include subcomponents on the EPRI 1015109 list [29], but they would be addressed as noted above.

Battery cells have a material aging phenomenon that occurs over time. There is no indication that cell electrical degradation is influenced by the frequency content of the cell support motion being either high-frequency or low-frequency. Batteries do not fail during support motion, but rather fail to produce the rated amp-hour capacity following the support motion. It is judged that the post-earthquake electrical capacity is a function of cell age and the RMS acceleration level of the input motion rather than the frequency content of the motion. Batteries that are less than ten years in age would not experience post-earthquake degradation due to cell shaking.

Valves have been subjected to significant high-frequency test motions due to Boiling Water Reactor (BWR) hydrodynamic loads and have not demonstrated high frequency unique sensitivities. EPRI Report 1015108 [29] provides an example of previous MOV operator combined seismic and BWR hydrodynamic qualification testing with inputs up to 100 Hz. This example valve operator is the same as used in other plant designs. These types of tests also show that additional high frequency content does not affect equipment function. In addition, line mounted valves and operators are subjected to 5-100 Hz sine sweep vibration testing as part of normal valve qualification to simulate normal plant induced vibration environments.

Table 3-2
AP1000 Potentially High Frequency Sensitive Items

▪ 125V Batteries ▪ 250Vdc Distribution Panels ▪ Fuse Panels ▪ Battery Disconnect Switches ▪ 250Vdc Motor Control Centers ▪ Regulating Transformers ▪ 6.9KV Switchgear ▪ Level Switches (Core Makeup Tank, Containment Flood) ▪ Radiation Monitors (Containment High Range Area, Control Room Supply Air) ▪ Transmitters (Flow, Level, Pressure, Differential	▪ Battery Chargers ▪ 120Vdc Distribution Panels ▪ Fused Transfer Switches ▪ Termination Boxes ▪ 250Vdc Switchboard ▪ Inverters ▪ Reactor Trip Switchgear ▪ Neutron Detectors (Source Range, Intermediate Range, Power range) ▪ Speed Sensors (Reactor Coolant Pump) ▪ Protection and Safety Monitoring Systems (System Cabinets, Transfer Switches, Neutron Flux Preamplifiers,

Pressure) ▪ Control Room (Workstations, Switch Station, Display Units) ▪ Motor Operated Valves (Motor Operators, Limit Switches) ▪ Air Operated Valves (Solenoid Valves, Limit Switches)	High Voltage Distribution Boxes) ▪ Other Valves (Squib [Explosive Opening] Operators, Limit Switches)

3.4.1.3 Component Types to be Evaluated

The list of component types to be evaluated in the above noted high frequency test program was developed based on the reviews described in Sections 3.4.1.1 and 3.4.1.2 and is provided in Table 3-3. A subset of these component types are used in in the Phase 1 testing effort described below. The complete list of component types in the table will be considered in a follow-on Phase 2 testing effort.

Test samples will be selected from the component types in Table 3-3 to represent the components installed in operating nuclear power plants. A review of selected utility components lists will be used to inform the test sample selection.

Table 3-3
High Frequency Confirmation Component Types

▪ Electro-mechanical relays (e.g., control relays, time delay relays, protective relays) ▪ Circuit breakers (e.g., molded case and power breakers – low and medium voltage) ▪ Control switches (e.g., benchboard, panel, operator switches) ▪ Process switches and sensors (e.g., pressure, temperature, flow, limit/position)	▪ Electro-mechanical contactors (e.g., MCC starters) ▪ Auxiliary contacts (e.g., for MCCBs, fused disconnects, contactors/starters) ▪ Transfer switches (e.g., low and medium voltage switches with instrumentation) ▪ Potentiometers (without locking devices)

3.4.2 Phase 1 Testing

The high-frequency test program consists of two phases. The first phase pilot effort has focused on (1) developing a recommended high-frequency test protocol to be used in the full test program, and (2) acquiring sufficient data to allow development of criteria for comparison of fragility levels obtained using high-frequency wide-band and narrow-band motions.

3.4.2.1 Phase 1 Test Samples

The components included in the Phase 1 test program were selected to provide a representative sample of the types of components listed in Section 3.4.1.3, as well as a variety of expected seismic capacity levels. The list of components used for Phase 1 testing is provided in Table 3-4.

Table 3-4
Phase 1 Test Samples

▪ Electro-mechanical relay (600V industrial control relay) ▪ Electro-mechanical relay (pneumatic timing relay) ▪ Electro-mechanical contactor (with auxiliary and overload contacts) ▪ Electro-mechanical relay (lockout relays, two configurations considered) ▪ Electro-mechanical relay (auxiliary relay - hinged armature)	▪ Electro-mechanical relay (socket mounted control relay) ▪ Electro-mechanical relay (300V industrial control relay) ▪ Electro-mechanical relay (600V control relay – prior HF testing history) ▪ Electro-mechanical relay (induction disk protective relay) ▪ Process switch (pressure switch)

3.4.2.2 Phase 1 Testing Protocol

A number of test parameters were investigated in Phase 1, as described below.

Primary Frequency Range of Interest

For the component types listed in Section 3.4.1.3, licensing basis seismic qualification testing is typically performed over a frequency range up to 33 Hz. For floor- or wall-mounted components, in-structure response spectra typically peak in the 4 to 10 Hz range and reach the Zero Period Acceleration (ZPA) in the 15 to 20 Hz range. For in-cabinet mounted components, IEEE C37.98 [31] recommends a response spectrum shape with peak spectral accelerations in the 4 to 16 Hz range and a ZPA at 33 Hz.

Some of the new ground motion estimates have peak accelerations in the 25 to 30 Hz range, which may produce significant in-structure or in-cabinet motions in the 20 to 40 Hz range. Figure 3-5 shows an example ground motion where in-structure and in-cabinet high-frequency motions may be significant.

Because licensing basis seismic qualification testing adequately addresses the lower frequency range, the high-frequency test program will focus on this higher frequency range. The primary focus of the high-frequency testing program is the 20 to 40 Hz frequency range. Phase 1 testing initially considered a broader frequency range of 16 to 64 Hz to insure that the focus on the 20-40 Hz range is sufficient.

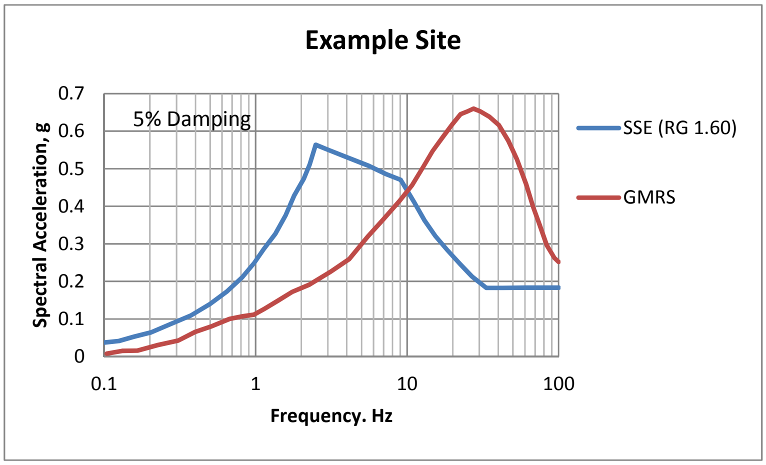


Figure 3-5
Example High Frequency Ground Motion Response Spectrum

Test Input Motions

Three types of test input motions were investigated in Phase 1: sine sweeps, random multi-frequency (RMF) motions and filtered RMF motions. In each case, the input motions were increased in amplitude until either the components failed the acceptance criteria (typically 2 ms contact chatter per ANSI C37.98 [31]), or had anomalous behavior, or the test machine limits were reached.

<u>Sine Sweep Input Motions</u> – This test series used single-axis sine sweep inputs with constant acceleration levels over the 16 to 64 Hz range. The components were tested in each primary direction (e.g., front-to-back, side-to-side, vertical) in the de-energized (non-operate) state with subsequent tests in the energized (operate) state. The objective of this test series was to develop a plot of chatter threshold frequency vs. peak input motion acceleration as a means of displaying the regions of high-frequency sensitivity for each component.

RMF Input Motions – This test series used wide-band multi-frequency tri-axial independent random motions with response spectra covering three separate amplified frequency ranges as shown in Figure 3-6. The three frequency ranges were 16 to 32 Hz, 24 to 48 Hz, and 20 to 40 Hz. The general shape of the amplified spectral region was patterned after the normalized test shape from IEEE C37.98 [31] with the peak acceleration region being 2.5 times the ZPA, but with the frequency ranges shifted as shown in Figure 3-6. A set of three motions was generated for each frequency range. Each axis of motion of each set was independent but had the same general response spectrum shape and amplitude. The purpose of these tests was to determine the fragility level of each device associated with each set of RMF motions for a given frequency range.

Filtered Random Multi-Frequency (FRMF) Input Motions – This test series used wide-band multi-frequency independent random input motions along two primary axes with a set of narrow-band filtered motions along the third axis as depicted in Figure 3-7. The narrow-band motions were applied along the third axis, one at a time, at the indicated 1/6 octave frequencies between 17.8 and 44.9 Hz (21.2 Hz, 23.8 Hz, 26.7 Hz, 30.0 Hz, 33.6 Hz, 37.8 Hz). The RMF motions applied to the other two axes had strong motion frequency range of 17.8 Hz to 44.9 Hz. The FRMF motions were applied separately in the component front-to-back direction and the side-to-side direction. Note that the FRMF testing was intended to simulate either in-structure response or high frequency local panel in-cabinet response, which is expected to be dominated by front-to-back or side-to-side responses; therefore, the filtered motions were limited to those two directions. Each filtered motion had the appearance of multiple sine-beat motions superimposed on a wide band random backbone motion. The purpose of these tests was to determine the fragility level of each device associated a given FRMF motion. The comparison of the fragility response spectra for both the FRMF and RMF motion allows a 'clipping factor' to be defined that can be used to convert an in-structure or in-cabinet demand (response spectrum) to an effective wide band motion for comparison to a RMF fragility test spectrum.

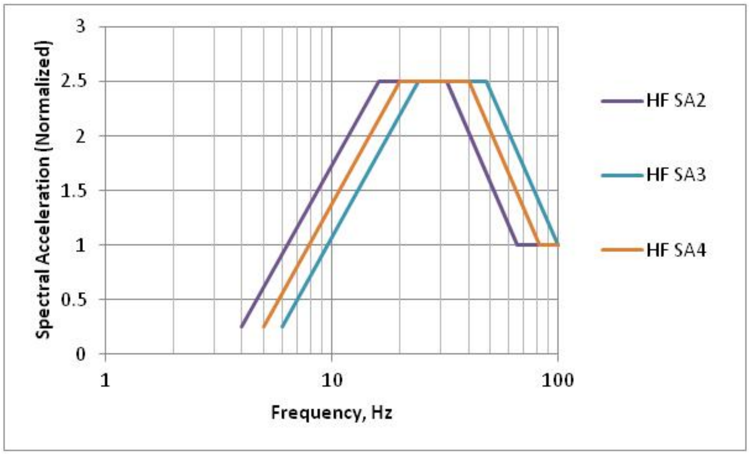


Figure 3-6
Random Multi-Frequency Test Input Motions

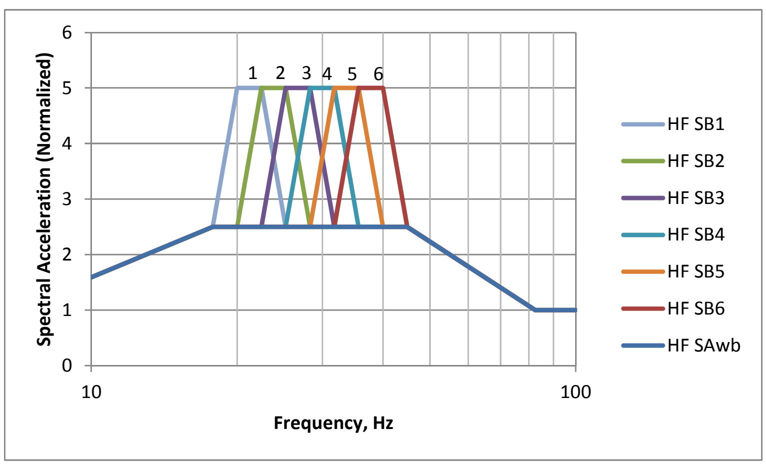


Figure 3-7
Filtered Random Multi-Frequency Test Input Motions

3.4.2.3 Phase 1 Test Results

Component High Frequency Sensitivity

No chatter or change of state occurred for any Phase 1 relay in the energized state for any input motion, thus a relay in the energized state is considered to not be frequency sensitive

Four devices did not have any chatter in the de-energized state for the highest input levels tested. These devices may be considered as not frequency sensitive. Two devices were expected to not have any high-frequency sensitivity based on the high demonstrated low frequency ruggedness (12.5 g spectral), however, these models had anomalous behavior in the greater than 30 Hz range. The remaining 5 relay models demonstrated various high-frequency sensitivities for the de-energized state.

Test Input Motion Results

The general conclusions from each of the three test input motion types are described below.

Sine Sweep Input Motions – The sine sweep tests were primarily exploratory tests but they did not appropriately simulate earthquake motions at the component mounting locations and at high amplitudes, they over predicted component sensitivity.

RMF Input Motions – The RMF tests over the three peak frequency ranges (16 to 32 Hz, 20 to 40 Hz, and 24 to 48 Hz) proved to be efficient to perform, and effective at identifying high frequency component sensitivity. Some of the Phase 1 test components were a little more sensitive to the test motions from 16 to 32 Hz than the other two test ranges; however, this was due to the lower frequency energy included in those tests. That lower frequency energy is already included standard relay testing performed using the IEEE relay qualification standard [31]; therefore, it is more indicative of lower frequency sensitivities than high frequency sensitivities. Between the remaining two RMF frequency ranges, the 20 to 40 Hz range is more consistent with the expected high frequency ground motions shown in Figure 3-5. The Phase 1 components were also slightly more sensitive to that frequency range input than the 24 to 48 Hz RMF motion.

FRMF Input Motions – The FRMF tests may be the most accurate at simulating the kinds of earthquake motions at the component mounting locations, but they were very time consuming to perform. Comparisons of the FRMF peak spectral accelerations that produced component chatter with the RMF peak spectral accelerations confirmed that previous narrow band clipping factors (e.g. [39], Appendix Q) are also generally applicable to high frequency motions.

Phase 1 Overall Conclusions

The Phase 1 study indicates that the high-frequency sensitivity of contact devices is generally device specific. Thus, the best means to identify such frequency sensitivity is to test the devices. Additional testing in Phase 2 may facilitate more general conclusions for some categories of potentially sensitive devices (e.g. miniature relays, potentiometers).

The use of the 20-40 Hz multi-frequency random input motion appears to be the best compromise for determining frequency sensitivity. The use of other input motions requires considerable effort and do not appear to provide any better resolution for determining high-frequency sensitivity.

Filtered multi-frequency narrow-band inputs resulted in peak spectral fragility values that were 2-3 times the spectral fragility values obtained using the wide-band multi-frequency inputs. Thus, it appears that the clipping factors used for low frequency fragility are valid for high-frequency fragility. (This is still under study)

3.4.3 Phase 2 Testing

Phase 2 testing will be performed to address the component types identified in Section 3.4.1.3. The complete test results will be compiled as appropriate to support utility high-frequency confirmation screening in Figure 1-1, Step 3f, as well as SPRA or SMA evaluations in Figure 1-1, Steps 6a and 6b.

3.4.3.1 Phase 2 Test Protocol

Base on the Phase 1 ~~testing ,~~testing, the 20 to 40 Hz RMF response spectrum shape will be used to develop the test motions for the Phase 2 test protocol. These motions will be used to determine the fragility spectra for each component.

3.4.3.2 Expanded Sample

The test sample list for Phase 2 testing will be selected to address the range of component types identified in Section 3.4.1.3. Components will be selected to represent a distribution of manufacturers and specific model numbers. Components will also be selected to address a variety of contact mechanical motions (e.g., plunger- and clapper-type relays) and physical forms (e.g., socket and bolted mounting configurations). The number of components in any component type category may be adjusted depending on the expected degree of high-frequency sensitivity. In addition, the specific model numbers selected may be adjusted depending on the component availability. To the extent practical, the distribution of test samples will be selected to achieve the broadest possible conclusions.

Section 4: Seismic Hazard and Screening Report

The NRC 50.54(f) information request associated with NTTF Recommendation 2.1 seismic is delineated in [1]. Within 1.5 years of the March 12, 2012 date of the information request, each CEUS addressee is requested to submit information related to the seismic hazard and the screening portions of the program (see Boxes 1, 2, and 3a-3e of Figure 1-1). An example of the type of information that has been requested, which could form the table of contents for that report, is listed below.

- Introduction
    1. Responding to 50.54(f) letter
    2. Brief history of seismic licensing basis (summary of SSE and which codes, standards, and methods were used in the design of Seismic Category I SSCs)
    3. Brief description of method used to develop GMRS and outcome of screening comparisons
- Seismic Hazard Results: GMRS
    1. Regional and Local Geology
        a. Regional Geology
            i. 1-2 paragraphs describing tectonic setting and history
        b. Local Geology
            i. 1-2 paragraphs described any prominent geologic features, complexity of geologic features (folding and faulting)
    2. Probabilistic Seismic Hazard Analysis
        a. Probabilistic Seismic Hazard Analysis
            i. Summary of sources used (sub-set of CEUS Seismic Source Characterization sources or site-specific for WUS sites)
            ii. Ground Motion Prediction Equations used or developed (WUS sites)

   b. Base Rock Seismic Hazard Curves (if hard rock site)
      i. Common fractiles and mean for spectral frequencies for which GMPEs are available

3. Site Response Evaluation (if not a hard rock site)
   a. Description of Subsurface Materials and Properties
      i. Soil/rock types, layering, and properties
   b. Development of Base Case Profiles and Nonlinear Material Properties
      i. Resources used and basis for base case profiles
         1. Base case shear wave velocity profiles
         2. Selected Shear Modulus and Damping curves
   c. Randomization of Profiles
      i. Randomization method and parameters
      ii. Constraints applied on layer thicknesses and velocities
      iii. Kappa values
   d. Input Spectra
      i. Fourier amplitude spectra and response spectra including input elevation
      ii. Any modifications to input spectra (kappa correction)
   e. Methodology
      i. Brief description of Random Vibration Theory (RVT) or time series approach
      ii. Parameters used in RVT or time histories used
   f. Amplification Functions
      i. Amplification functions
      ii. Amplification versus Input Amplitude including uncertainty bands for each of the spectral frequencies
   g. Control Point Seismic Hazard Curves
      i. Common fractiles and mean for spectral frequencies for which GMPEs are available

4. Ground Motion Response Spectrum
   a. Uniform Hazard Response Spectra
      i. 10^{-4} and 10^{-5} UHRS

b. GMRS

i. Table of 10^{-4} and 10^{-5} UHS, Design Factor values, and GMRS

- Safe Shutdown Earthquake Response Spectra
  1. Spectral Shape and Anchor Point PGA for 5% critical damping
     a. Brief description from FSAR
  2. Control Point Elevation(s)
     a. Description from FSAR or assumptions used to determine control point elevation
- Special Screening Considerations
  1. GMRS and SSE Comparison
     a. Discussion of results
     b. High-frequency, Narrow Band Exceedance (if applicable)
  2. Evaluation of IPEEE Submittal
     a. see Section 3.3-1
  3. GMRS and IHS Comparison
     a. If applicable, discussion of results (narrow-band exceedance if applicable)
  4. Screening for Risk Evaluation (SPRA or SMA see Section 6.2)
     a. If applicable, discussion of results
- Interim Actions*
  1. Any interim actions taken or planned while risk evaluation is being performed
- Conclusions
  1. Summary of results
  2. Path forward based on Screening Evaluations

*The NRC has requested that each addressee provide information on "any actions taken or planned to address the higher seismic hazard relative to the design basis, as appropriate, prior to the risk evaluation." Examples of the types of information which could be included in this response are:

- Modifications or upgrades that the addressee decides to undertake prior to the seismic risk evaluation (Section 6 of this report).
- Addressee intentions relative to conducting an SPRA or SMA.
- Description of the types of exceedances (low-frequency range, high-frequency range, narrow-frequency band, etc.) and the types of SSCs which may be affected by that exceedance (e.g., high-frequency exceedance could affect chatter sensitive devices which are going to be

addressed by the EPRI testing program described in Section 3.4 of this report).

Section 5: Prioritization (Schedule)

The resolution for the 2.1 seismic information requests [1] consists of first generating the new seismic hazard information for all sites, followed by the screening assessments described in the previous sections. Those plants required to perform the seismic risk evaluation are then required to be prioritized in terms of their schedule for submittals (Diamond 5, Figure 1-1). This prioritization occurs after seismic hazard and screening submittals described in Section 4 of this report are submitted to the NRC. That report is scheduled to be completed by Fall 2013 for CEUS sites and Spring 2015 for the WUS sites.

For seismic hazard evaluations that are demonstrated to need a higher priority, addressees are requested to complete the risk evaluation (SPRA ~~or SMA~~) over a period not to exceed three years from the date of the prioritization (Fall 2016 for CEUS and Fall 2018 for WUS). In accordance with the March 12, 2012 50.54(f) information request, for seismic hazard evaluations that do not demonstrate the need for a higher priority, addressees are requested to complete the risk evaluation (SPRA or SMA) over a period not to exceed four years from the date of the prioritization (Fall 2017 for CEUS and Fall 2019 for WUS).

Comment [C13]: Consider rewriting in terms of augmented approach.

The intent of the prioritization is to take into account:

- the amount of the seismic hazard exceedance,

  GMRS to SSE Ratios

  GMRS to IPEEE HCLPF

- the available resources (industry-wide and individual utility multi-unit fleets), and
- plant vintage.

Comment [C14]: How so?

The exact criteria/methods to be used for this prioritization are being discussed between the NRC and the nuclear utility industry as part of ongoing discussions on the resolution of the 2.1 seismic program.

Section 6: Seismic Risk Evaluation

6.1 Background on SPRA and SMA

SPRA and SMA studies have been conducted for many of the U.S. NPPs over the last twenty years. Initially they were conducted to answer safety concerns in heavily populated areas. The next widespread application was for satisfaction of the USNRC request for information regarding severe accident vulnerabilities in Generic Letter 88-20, Supplement 4 [2]. The USNRC is currently encouraging the use of PRA for making risk-informed decisions and has developed a Risk-Informed Regulation Implementation Plan [32] and associated regulatory guides. The Licensees in turn are using PRAs for Changes to Licensing Basis, Changes to Technical Specifications, Graded Quality Assurance, Significance Determination Processes, etc. Seismic PRAs are also required for each new NPP one year prior to fuel load. SPRAs and SMAs are now also being recommended as paths to conduct the seismic risk evaluations within Tasks 6 and 7 of Figure 1-1.

6.1.1 SPRA Methods and Procedures

Current U.S. NPPs were designed to withstand a conservatively selected large earthquake ground motion (the SSE) with adequate margins introduced at different stages of design, analysis, qualification, and construction. However, it is understood that larger earthquake ground motions (although rare) could occur. The basic objective of the SPRA is to estimate the probability of occurrence of different levels of earthquake ground shaking that may affect the plant, and to assess the plant response to such ground motions. Following the historical PRA practice, the results of this plant seismic assessment are presented in terms of seismically induced CDF and large-early release frequency (LERF). SPRAs completed to date, have shown that the seismic contribution to the overall CDF and LERF at some NPPs could be significant and occasionally can even be dominant. Therefore, a quantitative assessment of the seismic risk (e.g., SPRA) can be an important component of the overall risk-informed decision making process.

The key elements of a SPRA can be identified as:

- **Seismic Hazard Analysis:** Used to assess the seismic hazard in terms of the frequency of exceedance for selected ground motion parameters during a specified time interval. The analysis involves the characterization of earthquake sources, evaluation of the regional earthquake history, and an estimation of the intensity of the earthquake-induced ground motion at the site (Figure 6-1).

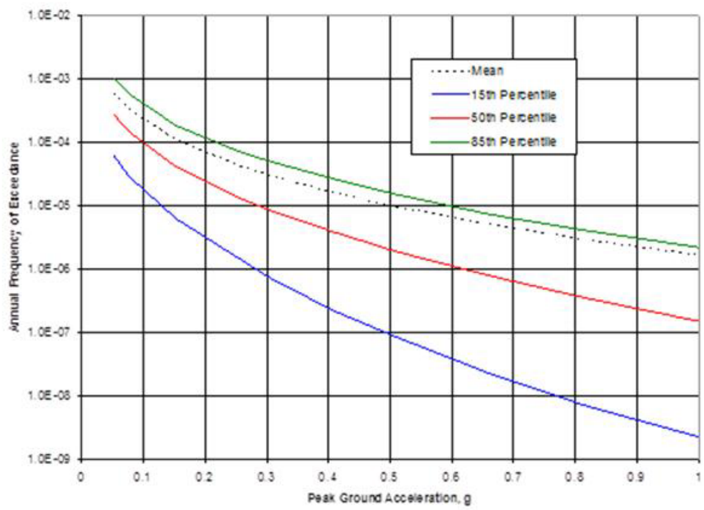


Figure 6-1
Example Seismic Hazard Curve

- **Seismic Fragility Analysis**: Estimates the conditional probability of SSC failures at a given value of a seismic motion parameter, such as PGA, peak spectral acceleration, floor spectral acceleration, etc. Seismic fragilities used in a seismic PRA are realistic and plant-specific based on actual current conditions of the SSCs in the plant, as confirmed through a detailed walkdown of the plant. The fragilities of all the systems that participate in the accident sequences are included (Figure 6-2).

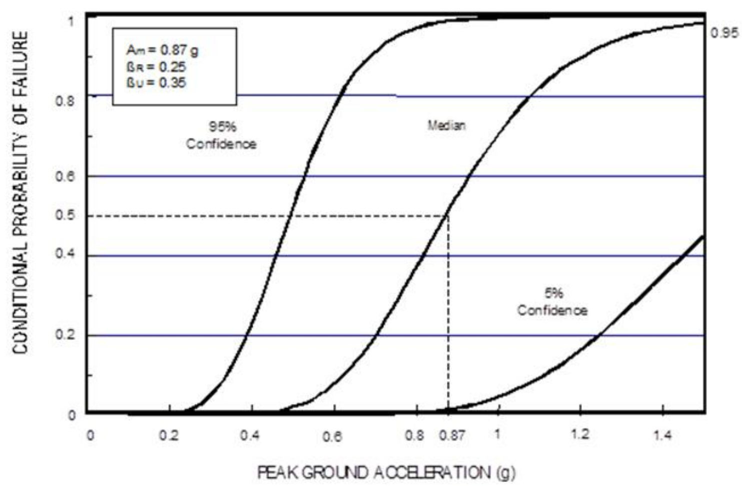


Figure 6-2
Example Seismic Fragility Curve

- **Systems/Accident Sequence Analysis**: Modeling of the various combinations of structural and equipment failures that could initiate and propagate a seismic core damage sequence.
- **Risk Quantification**: Calculates the frequencies of severe core damage and radioactive release to the environment by using the plant logic model and accident sequences for which the SSC fragilities are integrated with the seismic hazard. The analysis is usually carried out by adding some earthquake-related basic events to the PRA internal events model, as well as eliminating some parts of the internal events model that do not apply or that can be screened-out.

The overall SPRA process is characterized in Figure 6-3.

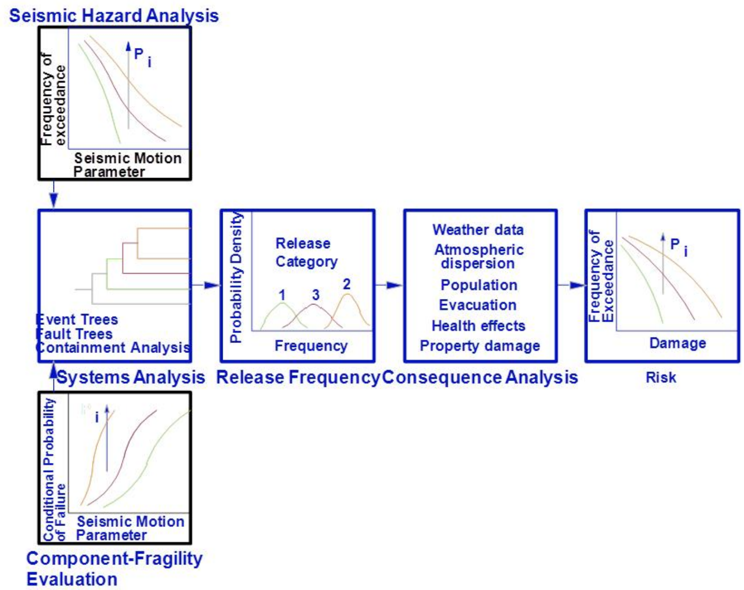


Figure 6-3
Overview of the SPRA Methodology

The detailed methods and criteria to develop the seismic fragility, seismic hazard, and seismic plant logic models are well beyond the scope of this guide. Fortunately, there are many technical references which document these methods. Table 6-1 is intended to provide a good list of references on these topics, while there are obviously many more in the literature.

Table 6-1
Partial List of SPRA Technical References

SPRA Topic	Recommended Document Title	Reference
SPRA	Seismic Probabilistic Risk Assessment Implementation Guide	EPRI 1002989 (Dec 2003) [33]
	Seismic Evaluation of Existing Nuclear Power Plants	Safety Report Series No. 28 [34]
	Probabilistic Safety Assessment for Seismic Events	IAEA Tecdoc-724 (Oct 1993) [35]
Seismic Fragility	Seismic Fragility Applications Guide Update	EPRI Report 1019200 (Dec 2009) [36]
	Seismic Fragility Application Guide	EPRI 1002988 (Dec 2002) [37]
	Methodology for Developing Seismic Fragilities	EPRI TR-103959 (June 1994) [38]
	A Methodology for Assessment of Nuclear Plant Seismic Margin	EPRI NP 6041 (Oct 1988) [39]
Seismic Hazard	PRA Procedures Guide: A Guide to the Performance of Probabilistic Risk Assessments for Nuclear Power Plants	NUREG/CR-2300 (1983) [54]
	Recommendations for Probabilistic Seismic Hazard Analysis: Guidance on Uncertainty and Use of Experts	NUREG/CR-6372 (1997) [13]
	Practical Implementation Guidelines for SSHAC Level 3 and 4 Hazard Studies	NUREG-2117 (2012) [23]
	Technical Basis for Revision of Regulatory Guidance on Design Ground Motions: Hazard- and Risk-Consistent Ground Motion Spectra Guidelines	NUREG/CR-6728 (Oct 2001) [24]

6.1.2 NRC SMA Methods and Procedures

The Nuclear Regulatory Commission (NRC) staff developed interim staff guidance (ISG) [100] on an acceptable method for performing a Seismic Margin Assessment (SMA) as referred to in the March 12, 2012 NRC letter [1]. This SMA method includes enhancements to the NRC SMA method originally described in NUREG/CR-4334 that the NRC deemed necessary to meet the objectives of the 50.54(f) letter. The NRC ISG approach for SMA is specifically intended be used to respond to the 50.54f letter. The level of effort to perform a SMA to meet this ISG is nearly equal to that required for a SPRA. The primary difference is

Comment [C15]: unbold

that the SMA reports results in terms of HCLPF values, rather than risk metrics such as CDF or LERF.

A list of the high level features and enhancements to an SMA that are documented in the draft ISG are listed below. Some of these topics are similar to staff positions taken during the IPEEE program, and others are additional enhancements.

- The SMA should use a systems-analysis approach that begins by following the NRC SMA methodology, using event trees and fault trees, with enhancements; an EPRI SMA approach using success-path systems logic is not acceptable.

- The SMA should be a full-scope SMA, not a focused-scope or reduced-scope SMA (as described in NUREG-1407).

- The systems model should be enhanced over what was contained in either the original NRC SMA guidance (in NUREG/CR-4334 and NURE/CR-5076) or the NRC's IPEEE guidance (in NUREG-1407).

- The scope should include certain containment and containment systems, so as to enable analysis of the plant-level HCLPF for large early release.

- The "mission time" should extend to when the plant reaches a stable state.

- The use of the so-called "Min-Max" method must be justified and, if used, should follow certain guidance provided in [100]. The Convolution Method is stated to be the NRC's preferred method.

- When developing sequence-level and plant-level HCLPF capacities, the analysis should differentiate between those sequences that lead to core damage and those that lead to a large early release.

- Separately report HCLPF capacities for those sequences with non-seismic failures and human actions and HCLPF capacities for those sequences without them.

- A formal peer review of the SMA is required. The ISG peer review requirements are not consistent with the peer review requirements of RG 1.200 or the ASME/ANS PRA Standard.

The details for each of these features and enhancements are described in detail within the draft ISG [56].

Licensees may propose other methods for satisfying SMA requirements of the 50.54f NRC letter. The NRC staff will review such methods and determine their acceptability on a case-by-case basis.

6.2 Criteria for Selection of Risk Evaluation Method (SPRA vs. SMA)

As shown in Figure 1-1, plants that do not meet the screening criteria outlined in Section 3 of this report need to proceed to a seismic risk evaluation. Reference [1] describes two different approaches for performing the seismic risk evaluation, an SPRA, or an NRC SMA. The NRC SMA is appropriate for sites where the re-evaluated seismic hazard is not considerably higher than the design basis seismic hazard or for sites that have a relatively low seismic hazard level. The SPRA could be used for any of the plants proceeding to the seismic risk evaluation phase.

The NRC criteria for requiring the use of the SPRA consists of the following:

- If the GMRS exceeds the response spectra between 1 and 10 Hz represented by the higher of the following two spectra, then an SPRA should be conducted:
    1. 1.3 times the SSE
    2. Low Hazard Threshold of 0.4g

Figure 6-4 shows an example of a GMRS exceeding the 1.3 SSE and the LHT spectra in the 1 to 10 Hz range.

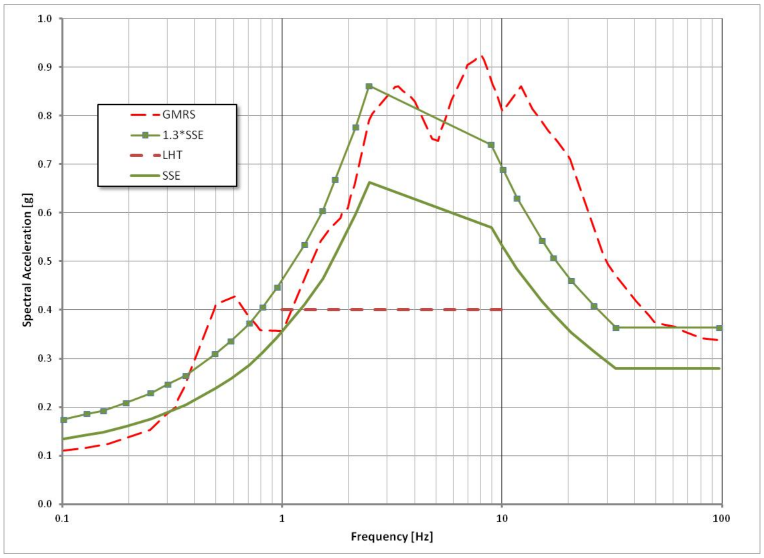


Figure 6-4
Example for Selection of SPRA vs. SMA

6.3 Key Elements of Seismic Structural and SSI Response

Comment [C16]: no guidance on development of FIRS to be consistent with ISG-17?

6.3.1 Structure Modeling

Many existing structural models (i.e., those used for design basis, USI-A-46 or IPEEE studies) could be used in structural dynamic analyses that are performed to support SPRAs or SMAs required as part of the response to the 50.54(f) letter on 2.1, provided that their adequacy is demonstrated for this purpose. This requires a review of the existing models to be performed by an experienced structural engineer(s) (and a peer reviewer) to determine the adequacy of the models for dynamic analysis for application in risk assessments for 2.1 using the criteria provided below. If necessary, the existing structural models can be enhanced to meet the structural modeling criteria.

The existing structural models that have been used in dynamic analyses to develop seismic responses for the design, licensing and qualification of plant SSCs (e.g., lumped-mass stick models (LMSM)), were reasonably complex for their original intended purpose at the time they were developed. These models were used to capture the overall structural frequencies, mode shapes, and seismic responses. Typically, if a model complexity is increased, the contribution of the modes identified within the simpler model is decreased as modal mass is shifted to other modes, often resulting in lower spectral peaks for the significant modes of the structure. However, more recent experience has shown that, for some structures, additional complexity of the numerical model may lead to the identification of important higher modes that may be important for some systems and components.

Using the existing structural models, in either their current or enhanced state, will facilitate the completion of the SPRA/SMA effort with the desired accuracy required as part of the response to the 50.54(f) letter on 2.1.

The criteria against which structural engineer(s) and peer reviewer(s) should review the existing models are listed below.

1. The structural models should be capable of capturing the overall structural responses for both the horizontal and vertical components of ground motion.
2. If there is significant coupling between the horizontal and the vertical responses, one combined structural model should be used for analyzing all three directions of the earthquake. See ASCE 4-98 Section 3.1.1.1 "Models for Horizontal and Vertical Motions" [40].
3. Structural mass (total structural, major components, and appropriate portion of live load) should be lumped so that the total mass, as well as the center of gravity, is preserved. Rotational inertia should be included if it affects response in the frequency range of interest. See ASCE 4-98 Section 3.1.4.1 "Discretization of Mass" Part (b) 1 [40].
4. The number of nodal or dynamic degrees of freedom should be sufficient to represent significant structural modes. All modes up to structural natural frequencies of about 20 Hz in all directions should be included (vertical floor slab flexibility will generally not be considered because it is expected to have frequencies above 15 Hz). This will ensure that the seismic responses and in-structure response spectra (ISRS) developed in the 1 to 10 Hz frequency range are reasonably accurate. See ASCE 4-98 Section 3.1.4.1 "Discretization of Mass" Part (b) 2 [40].
5. Torsional effects resulting from eccentricities between the center of mass and the center of rigidity should be included. The center of mass and the center of rigidity may not be coincident at all levels, and the torsional rigidity should be computed. See ASCE 4-98 Section 3.1.8.1.3 "Requirements for Lumped-mass Stick Models" Parts (b) and (c) [40]. Alternatively, a multiple LMSM may be used if the stiffness elements are located at the centers of rigidity of the respective groups of element and the individual models are properly interconnected.
6. The analyst should assess whether or not a "one-stick" model sufficiently represents the structure. For example, two-stick models could be more appropriate for the analysis of internal and external structures of the containment founded on a common mat.
7. The structural analyst should review whether in-plane floor flexibility (and subsequent amplified seismic response) has been captured

appropriately for the purposes of developing accurate seismic response up to the 15 Hz frequency. Experience has shown that, for nuclear structures with floor diaphragms that have length to width ratios greater than about 1.5, the in-plane diaphragm flexibility may need to be included in the LMSM. As with all these recommendations, alternate approaches can be used when justified.

The use of existing models must also be justified in the submission to the NRC using the above criteria.

6.3.2 Seismic Response Scaling

Scaling of ISRS to account for higher ground motions levels is considered a technically sound approach and has been used in previous SPRAs and SMAs. Using scaling approaches, where appropriate, will reduce the effort involved in performing detailed soil-structure interaction (SSI) analyses for the new hazard/UHS, facilitating the completion of the SPRA or SMA effort for those plants that are screened-in.

Scaling of responses will be based on

- previously developed ISRS,
- shapes of the previous UHS/RLE,
- shapes of the new UHS/RLE, and
- structural natural frequencies, mode shapes, and participation factors.

Example guidance on scaling methods is provided in EPRI documents EPRI NP-6041-SL Rev. 1 [39] and EPRI 103959 [38].

Scaling can be used in developing ISRS for those cases where the new UHS or RLE shape is approximately similar to the spectral shape previously used to generate the ISRS. An example of two response spectra with similar shapes is shown below in Figure 6-5.

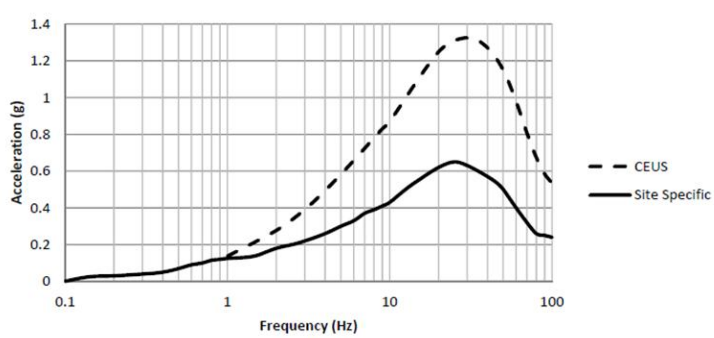


Figure 6-5
Example of Ground Response Spectra that are Similar

Scaling of rock or soil sites where the shape of the new hazard spectrum is not similar to the previous spectrum will require a rigorous justification that demonstrates the validity of the scaling approach. An example of spectra that are not similar is shown in Figure 6-5 below. The peak response of these two spectra is significantly shifted in frequency.

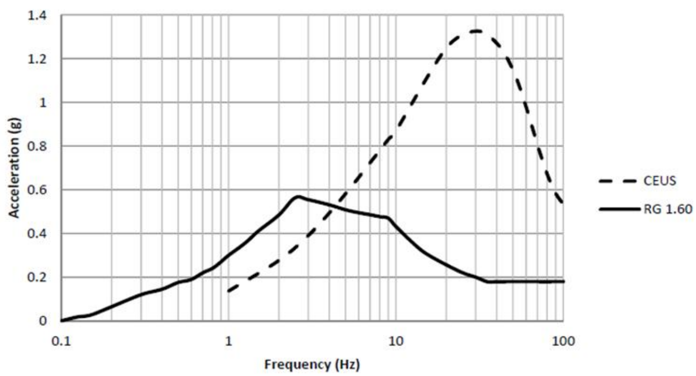


Figure 6-6
Example of Ground Response Spectra that are not Similar

Scaling of "non-similar" shapes would need to be technically justified on a case specific basis and would need to appropriately consider any non-linear effects to the structure or to the soil/rock profile resulting from the new response spectra shape and amplitude.

6.3.3 Fixed-Based Analysis of Structures for Sites Previously Defined as "Rock"

For nuclear safety-related structures founded on the commonly used definition of rock as defined in the design documentation for many operating plants, i.e., shear wave velocity (V_s) > about 5000 ft/sec, past experience has shown that the amplified response spectra in the 1 to 10 Hz are generally about the same from a fixed-based analysis of the structure as from a model that uses SSI analysis. Therefore, it is reasonable to use fixed-base dynamic analyses for rock-founded structures even when the rock shear wave velocities are not as high as 9200 ft/sec, which is the definition of hard rock for new reactors licensed by the NRC. The original definition of rock (V_s > about 5000 ft/sec) that was used by some plants in the past can still be used as the criterion for performing a fixed-base dynamic analysis to develop ISRS that are needed to perform fragility or HCLPF calculations.

The validity of the above criterion was reviewed using two examples of existing structures at a nuclear power plant [55]. The first example describes the analysis of a structure with a fundamental frequency of about 5 Hz, and the second example used a structure with a fundamental frequency of about 10 Hz in one horizontal direction. These examples considered fixed-base analysis and SSI analyses with different V_s values and are discussed in Appendix C. The results from this study show that there is a slight shift of frequency to the left, and some changes in spectral peak amplitudes occur when the fixed base is compared to an SSI analysis with V_s of about 3500 ft/second; however, the comparison of fixed-base analysis is much better with an SSI analysis using V_s of about 5000 ft/sec or higher.

Therefore, it is appropriate to model a rock-founded structure as fixed base if the best estimate of V_s is greater than about 5000 ft/sec. For structures founded on rock with V_s between 3500 ft/sec and 5000 ft/sec, peak-broadening or peak-shifting of the ISRS in fragility analyses can potentially alleviate the effect of a slight frequency/amplitude shift between the SSI and fixed-base analyses. The determination whether a fixed-base model can be used for rock sites with V_s values in this range should be made by an experienced structural engineer and justified in the submittal report to NRC. This assessment should also be peer reviewed by an experienced structural engineer as part of the peer review process.

Comment [DB17]: The deleted paragraph is redundant.

6.4 Key Elements of Fragility/Capacity for the Resolution of NTTF Recommendation 2.1

6.4.1 Hybrid Approach for Fragilities

There are two well-known methods to calculate fragilities of SSCs for use in a seismic PRA model [36, 37, 38, 39]. These are: (a) the Conservative Deterministic Failure Margin (CDFM) approach [39] where the HCLPF capacity is calculated first and the median capacity with an assumed (experience-based) composite variability (typically in the 0.35 to 0.45 range) is then calculated from the HCLPF; and (b) the fragility or separation of variables approach [36, 37, 38] where the median capacity is calculated, and the randomness and uncertainty variabilities (logarithmic standard deviations) are then calculated in a detailed manner for various parameters.

The CDFM approach for developing fragilities is a simpler method that can be performed consistently by more analysts and is an acceptable approach for generating fragilities within an SPRA for the majority of components for which a less detailed assessment is necessary. Because only a handful of components are risk-significant enough to justify the additional effort required by the separation of variables method, the CDFM method can provide efficiencies in the overall effort. Therefore, use of the CDFM approach is useful and beneficial for calculating fragilities of SSCs for use in seismic PRAs conducted to address the 50.54(f) letter.

In the CDFM fragility approach (also referred to as the Hybrid Method), the 1% failure probability capacity $C_{1\%}$ is computed along with an estimate of the composite logarithmic standard deviation $\hat{a}_C$ and its subdivision into random variability $\hat{a}_R$ and uncertainty $\hat{a}_U$, which, are used to estimate the corresponding fragility curve. As noted in [51], typically $\hat{a}_C$ lies within the range of 0.3 to 0.6. In fact, if all of the sources of variability discussed in [38] are appropriately considered, it is not possible to obtain an estimated $\hat{a}_C$ less than approximately 0.3.

The Hybrid Method is based on the observation that the annual probability of unacceptable performance (P_F) for any SSC is relatively insensitive to $\hat{a}_C$. This annual probability (seismic risk) can be computed with adequate precision from the CDFM Capacity (C_{CDFM}) and an estimate of $\hat{a}_C$. It is shown in [51] that the computed seismic risk at $\hat{a} = 0.3$ is approximately 1.5 times that at $\hat{a} = 0.4$, while at $\hat{a} = 0.6$ the computed seismic risk is approximately 60% of that at $\hat{a} = 0.4$.

Table 6-2 provides recommended values for $\hat{a}_C$, $\hat{a}_R$, $\hat{a}_U$, and the ratio of the median capacity $C_{50\%}$ to the $C_{1\%}$ capacity computed by the CDFM Method. The recommended $\hat{a}_C$ values are based on Ref. [51] recommendations and on average are biased slightly conservative (i.e., slightly low $\hat{a}_C$ on average). Because random variability $\hat{a}_R$ is primarily due to ground motion variability, a constant $\hat{a}_R$ value of 0.24 is recommended irrespective of the SSC being considered. The recommended $\hat{a}_U$ values are back-computed from the recommended $\hat{a}_C$ and $\hat{a}_R$ values. The Beta values for Table 6-2 apply to fragilities tied to ground motion parameters (e.g., PGA or Peak Spectral Acceleration at 5 Hz). Appendix D contains a sensitivity study on the computed probability of failure, P_F, to the logarithmic standard deviation used in the hybrid method. The results of the study in Appendix D demonstrates a lack of sensitivity of the computed seismic risk exists over the full practical range of seismic hazard curve slopes.

Table 6-2
Recommended β_C, β_R, β_U, and $C_{50\%}/C_{1\%}$ Values to Use in Hybrid Method for Various Types of SSCs

Type SSC	Composite β_C	Random β_R	Uncertainty β_U	$C_{50\%}/C_{1\%}$
Structures & Major Passive Mechanical Components Mounted on Ground or at Low Elevation Within Structures	0.35	0.24	0.26	2.26
Active Components Mounted at High Elevation in Structures	0.45	0.24	0.38	2.85
Other SSCs	0.40	0.24	0.32	2.54

Following the generation of the fragilities using the hybrid approach, the fragility parameters are then used in the systems model to convolve with the hazard. For those SSCs that are determined to be the dominant risk contributors or are risk-significant in the seismic accident sequences, estimates of median capacity ($C_{50\%}$) and variabilities (β_u and β_r) should be done using the fragility or separation of variables approach and then used in the integration.

6.4.2 High-Frequency Capacities

(This section to be completed following the Phase 1 high frequency testing project.)

6.4.3 Capacity-based Selection of SSCs for Performing Fragility Analyses

Capacity-based criteria to determine the SSCs for which fragility analyses should be conducted have been developed to provide uniform guidance to analysts performing seismic PRAs (or margin analyses) and to ensure that proper focus is given to those SSCs that have the potential to be risk-significant. These criteria were developed as a result of a parametric/sensitivity study [42] that was undertaken for that purpose. These criteria establish which SSCs will require explicit calculation of fragility parameters for inclusion in the plant logic models. SSCs with capacities above the screening level calculated using the criteria are not expected to have significant impact on the result of the seismic PRA analyses, the ranking of accident sequences, or the calculated sequence- or plant-level seismic CDF or LERF values.

It is noted that a standard practice for seismic PRA practitioners has been to use insights from logic models to determine the need for fragility calculations and to prioritize SSCs. A preliminary SPRA plant logic model is developed even before the fragility calculation effort begins. Screening or ranking of SSCs from this preliminary SPRA logic model can be done by performing parametric sensitivity analyses with assumed initial fragilities and ranges of fragility values. Those SSCs that do not contribute significantly to the SCDF of an accident sequence may not need detailed fragility calculations. These SSCs may be retained in the model with a screening level capacity value which is described below.

Certain SSCs are inherently seismically rugged and consequently have a very low probability of failing as a result of a seismic event, as shown in Figure 6-7. Consistent with long-standing practice in seismic PRAs, seismic failure of such SSCs need not be included in the PRA logic models. Exclusion of such SSCs from the logic models does not affect the seismic CDF or the insights derived from the seismic PRA. Guidance in industry documents [39, 41] is available for identifying seismically rugged SSCs.

Other SSCs may be less rugged but would still have sufficient capacity such that their failures would be unlikely to contribute significantly to the SCDF in a seismic PRA. Screening criteria discussed below are developed for these SSCs. Detailed fragility calculations are not warranted for SSCs that meet these criteria. Figure 6-7 illustrates the use of screening level, which is applicable to the SSCs in the middle box.

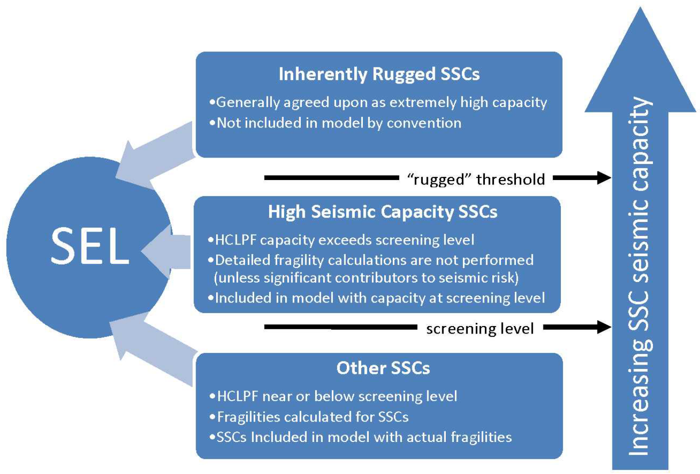


Figure 6-7
Capacity-based Criteria for Fragility Analyses

Based on the results of the sensitivity study conducted to develop this guidance [ref?], the screening HCLPF value of SSCs for a site should be calculated by convolving the fragility of a single element with the site-specific hazard curve such that the seismic CDF is at most about 5E-7 per year. This can be done with trial and error runs using a quantification code or with a spreadsheet with an assumed composite variability (e.g., β_c= 0.4). Because each site will have a different hazard curve, the screening HCLPF value for each seismic PRA needs to be separately derived. An alternative criterion, equivalent to the above CDF-based HCLPF, is to screen SSCs that have a HCLPF capacity above about 2.5 x GMRS. The results of the sensitivity study do not indicate that the screening criteria would be different for soil and rock sites.

The results of the sensitivity analyses performed indicate that the recommended screening HCLPF capacity derived from a CDF of 5E-7 is conservative for some hazard curves; a more liberal criterion may be appropriate for some sites and can be developed from an initial quantification of the logic model. Even though certain SSCs can be screened-out from having to perform detailed fragility calculations using the above criteria, their failure should be retained in the seismic PRA logic model with their capacity equal to the screening level or at a higher capacity level, if calculated, to allow for a more efficient ranking of accident sequences. Retention of such failures will ensure that future changes or sensitivities that could increase their importance are not overlooked and also addresses the problem of a potential cumulative effect of screened out components.

The results of the SPRA should be reviewed to determine whether or not an SSC modeled at the screening level could be identified as a significant contributor to CDF or to LERF sequences. If such an SSC is identified, then detailed fragility calculations should be performed for that SSC using the separation of variables method, and the quantification analysis should be rerun with the new fragility values.

If a component modeled at the screening level is risk significant, then the screening level has been set too low. One approach to assess whether the screening level was set at a sufficiently high level is to use a screening level set to three times the screening level originally used in the logic model for that component, quantify the seismic CDF/LERF and ensure that the CDF/LERF estimates are not reduced by more than 20%. It is likely that there will be several SSCs initially modeled at the screening capacity level, and the above approach, if performed for one component at a time, may be cumbersome. The procedure can also be done for multiple SSCs modeled at the screening capacity level simultaneously (i.e., set the screening level for all such SSCs to three times the original level and ensure that the CDF/LERF estimates do not reduce by more than 20%). If either of the estimates change by more than 20%, it may take some effort to pinpoint which component(s) modeled at the screening level is/are risk-significant. The sensitivity analyses in [42] showed that if the ratio of the screening level CDF (i.e., 5E-7 per year) to the plant's seismic CDF is not much greater than about 3% to 4%, the cumulative impact of SSCs modeled at the screening level is not expected to be significant (i.e., none of the screening level SSCs is likely to be risk-significant). This 3% to 4% ratio can be used as a guide by the PRA analyst to determine if the initial screening criterion for SSCs was appropriate or the screening capacity level needs to be adjusted (up or down).

To implement the capacity-based screening criteria, engineers can review previous calculations and reports (e.g., design basis, IPEEE, USI A-46 analyses, shake-table tests, etc.) to determine and judge if the seismic capacity of a component or structure for the new seismic hazard is such that no further calculation of fragility parameters is warranted.

It is expected that the use of the above screening methods will reduce the scope of the fragility or margin calculations required in the SPRA or SMA, and still meet the objective of identifying and ranking safety-significant SSCs. It is noted that, while the use of the above criteria is optional, engineers should not select a low screening HCLPF level (such as 0.3g) that was used by some plants during the IPEEEs. The above criteria are expected to result in sufficiently high screening levels to minimize the surrogate SCDF contribution (modeled at the screening level). Once the screening level is selected, the list of SSCs can be ordered so that the ones with the highest SCDF impact are calculated first.

6.5 Key Elements of SPRA/SMA Scope and Plant Logic Modeling

Comment [C18]: Sections 6.5, 6.6, 6.7 to be discussed Nov 5 meeting.

6.5.1 LERF Considerations

(Additional text to be inserted.)

6.6 Comparison to ASME/ANS SPRA Standard and RG1.200

6.6.1 Background

The PRA Standard is intended to identify the degree of detail and plant specificity in a risk assessment that reflects the nature of the application for which the risk assessment is being used. In any PRA performed for a risk-informed application, the intent is that the analyses meet at least the minimum requirements that could be relevant for the application, at the capability category corresponding to the nature of the application. The application in this case is to gain an updated understanding of the risk of seismic events at NPPs in light of new information about seismic hazard. This includes developing a new or changed understanding of risk outliers due to seismic events.

Because of the significance of this application, an attempt will be made to meet the requirements for Capability Category II, wherever feasible. To meet Capability Category II, the PRA must generally account for plant-specific configuration and design, and reflect plant-specific data where doing so could affect the important risk contributors.

For this application, which is aimed at developing an improved understanding of the impact of new seismic hazard estimates, screening approaches will be used to limit the scope of detailed analyses for some elements of the seismic PRA. Where more detailed analyses are essential to achieve an adequate level of understanding (e.g., with respect to "realism"), these analyses will be performed or alternative measures will be taken (such as making plant changes to address the impacts).

Applying the approach that has been specified for the seismic hazard is expected to satisfy Capability Category II in most respects. Some limitations in the approach may be employed to support completion of the required SPRAs. The Supporting Requirements will be examined in light of these limitations to ensure that the limitations do not affect the usefulness of the results or insights from the seismic PRA.

6.6.2 Comparison of 2.1 Seismic Approach to the SPRA Standard

For this application, the requirements corresponding to Capability Category II of the ASME/ANS PRA Standard [12] will generally be applied in the performance of elements of an SPRA. In limited cases, exceptions to the Standard requirements may be taken. The intent of the Standard will be met. Each Supporting Requirement will be reviewed against the 2.1 recommended technical approach to assess the Capability Category that applies. This review will be performed for all of the following:

- Regulatory Guide 1.200, Rev. 2 (the currently approved version of the Regulatory Guide that endorses the ASME/ANS PRA Standard) [43];
- ASME/ANS RA-Sa–2009, the currently approved version of the ASME/ANS PRA Standard [12];
- Addenda B to ASME/ANS RA-Sa-2009, the version of the Standard that is currently undergoing balloting [44].

In some cases, Regulatory Guide 1.200 [43] provides further clarification or specification beyond the details in the Standard. Because the newest version of the Standard may be approved by the time these seismic PRAs are performed, it is valuable to examine the specific implications for both the current Standard and the newer version.

6.7 Peer Review

This section describes the peer review requirements for the activities performed to meet the 50.54(f) letter [1] relative to the seismic 2.1 requests for information. The peer review need not assess all aspects of the SPRA or SMA against all technical requirements; however, enough aspects of the PRA shall be reviewed for the reviewers to achieve consensus on the adequacy of methodologies and their implementation for each PRA or SMA element. Alternative methods and approaches to meet the intent of SPRA/SMA technical requirements may be used if they provide results that are equivalent or superior to the methods usually used, and it is expected that the peer review team should concentrate on reviewing such alternate methods and approaches if they are used.

The peer review team shall have combined experience in the areas of systems engineering, seismic hazard, seismic capability engineering, and seismic PRAs, or seismic margin methodologies. The reviewer(s) focusing on the seismic fragility work shall have successfully completed the SQUG Walkdown Screening and Seismic Evaluation Training Course [52] and have experience with seismic fragilities.

One of the peer reviewers should be designated as the overall Team Leader. The peer review Team Leader is responsible for the entire peer review process, including completion of the final peer review documentation. The Team Leader is expected to provide oversight related to both the process and technical aspects of the peer review. The Team Leader should also pay attention to potential issues that could occur at the interface between various activities.

The peer review process includes a review of the following SPRA activities:

- Selection of the SSCs included on the SEL
- ~~Seismic hazard assessment~~
- The documentation from the Seismic Walkdowns
- Seismic response analyses
- Seismic fragility assessments
- Seismic risk quantification
- Final report

The results of the peer review should be documented in a separate report. Specific guidance on the key elements of the peer review process is found in Section 5.3 of the SPRA part of the ASME-ANS PRA Standard [12] entitled “Peer Review for Seismic Events At-Power.” This guidance is felt to be appropriate for this peer review, with the recommended exception that independent seismic fragility analyses are not required to be performed by the peer reviewers. Adequate peer review of the seismic fragilities can be accomplished (as in past SPRAs and SMAs) based on a review of a sample of the fragility calculations.

For the NTF 2.1 Seismic resolution, it may be preferable to conduct more focused peer review activities for individual SPRA elements during implementation of this program, to the extent practicable, rather than waiting until all the work is complete.

Comment [C19]: Is there a complication here since it would have been submitted earlier, except for looking at how the PSHA results were implemented in to the modeling?

Comment [GSH20]: If we are restricting this guidance to 2.1 application at CEUS plants , then we may be able to limit/remove this hazard part of the peer review.

Section 7: Spent Fuel Pool Integrity Evaluation

The 50.54(f) letter also requested that, in conjunction with the response to NTTF Recommendation 2.1, a seismic evaluation be made of the SFP. More specifically, plants were asked to consider "…all seismically induced failures that can lead to draining of the SFP."

Comment [E21]: Need to specify conditions under which SFP evaluation to be performed. Plants screening in to performing SFP evaluations – 50.54f letter

This section provides guidance that may be employed in addressing this consideration for plant-specific evaluations.

7.1 Scope of the Seismic Evaluation for the SFP

The focus of the evaluation process described in this report is on elements of the SFP that might fail due to a seismic event such that draining of the SFP could result, and the measures available to respond to such failures. This approach is intended to ensure that efforts to gain an understanding of potential seismic risks needed to respond to the 50.54(f) letter make the best possible use of available resources.

In developing guidance for the walkdowns associated with NTTF Recommendation 2.3 [46], the emphasis was on SFP connections whose failure could result in "rapid draindown." The definition of "rapid draindown" encompassed failures that could lead to uncovering of irradiated fuel stored in the SFP within 72 hours of the earthquake [46]. This criterion is used for the evaluations under NTTF Recommendation 2.1 as well; that is, the evaluations consider possible failures that could lead to uncovering fuel stored in the SFP within 72 hours.

Failures that could conceivably lead to uncovering of irradiated fuel stored in the SFP would include the following:

- A significant failure of the steel-lined, reinforced concrete structure of the SFP, causing inventory in the pool to drain out.
- Failure of a connection penetrating the SFP structure (drain line, cooling-water line, etc.) below the top of the stored fuel.
- Failure of a connection penetrating the SFP structure above the fuel sufficient to drain significant inventory from the pool and interrupt SFP cooling, such that (in the absence of adequate makeup)

evaporation and boil-off could cause fuel to be uncovered within 72 hours.

- Extensive sloshing such that sufficient water could be lost from the pool to interrupt SFP cooling and, as in the previous item, lead to uncovering of the fuel within 72 hours.
- Failure of a cooling-water line or other connection that could siphon water out of the pool sufficient to lead to uncovering of the fuel within 72 hours.
- Tearing of the steel liner due to movement of fuel assemblies as a result of the earthquake.

Comment [E22]: Consider drainage due to failure of reactor cavity seal

With regard to these possibilities, the evaluation may generally be focused on connected structures and systems that penetrate the SFP structure, rather than the basic structure of the SFP itself. The rationale for focusing the scope of the evaluation in this manner accounts for the following:

- Detailed assessments have been made of SFP integrity, including by the NRC, and these have found SFP structures to be reasonably rugged; and
- Even if the SFP were to experience a structural failure that led to draining of its inventory, systems (including those associated with the FLEX capability) should be able to prevent serious damage to the stored fuel.

With regard to previous evaluations, NUREG-1738 [47] characterized the robust nature of the design of SFPs currently in use, and identified inspection criteria that could be used to evaluate whether a SFP should be expected to retain its integrity to a peak spectral acceleration of at least 1.2g. Moreover, evaluations reported in NUREG/CR-5176 [48] for two older plants concluded that "…seismic risk contribution from spent fuel pool structural failures is negligibly small."

Tearing of the stainless-steel liner due to sliding or other movement of the fuel assemblies in the pool is considered to be very unlikely. Even if the liner should tear, the result would not be a direct breach of the integrity of the SFP but rather seepage through the reinforced concrete structure. Therefore, this possibility can also be excluded from more detailed evaluation for this purpose.

Comment [E23]: Provide more justification

While some sloshing has been observed during, for example, the 2007 earthquake at Kashiwazaki-Kariwa in Japan, it may be necessary to evaluate sloshing for larger earthquakes. Guidance related to this aspect of the earthquake response is provided in Section 7.3.2.

Beyond the impact of possible failures on the cooling of the fuel stored in the SFP, for some plants the loss of inventory from the pool could cause flooding that could affect other systems. The assessment of flooding will be evaluated separately, as part of the response to a NTTF Tier 3 recommendation.

The remainder of this section outlines a process for identifying and evaluating features that could lead to draining of the SFP.

7.2 Evaluation Process for the SFP

The process for evaluating the SFP begins with the identification of any penetrations that should be considered. All penetrations should be identified and placed into one of the following three categories:

1. Those that are at a level below the top of the fuel in the SFP;
2. Those that are above the level of the fuel in the SFP; and
3. Those that may have the potential to siphon water from the SFP (most typically, the discharge line from the SFP cooling system).

The sections that follow provide guidance for addressing each of these categories.

7.2.1 Evaluation of Penetrations below Top of Fuel

The SFPs for plants operating in the United States are generally configured so that they do not have penetrations below the top of the stored fuel. The absence of penetrations lower in the pool inherently limits the potential to drain inventory sufficiently to begin uncovering fuel. It is possible, however, that some SFPs may have penetrations (e.g., drain lines) below the top of the stored fuel assemblies. Moreover, while the transfer gates opened when moving fuel into and out of the SFP typically extend down to relatively near the top of the stored fuel, there may be some SFPs for which the bottom of the gates is below the top of the fuel. A failure associated with such a penetration could drain the pool level below the top of the fuel if there were inadequate makeup flow to the pool. A process for evaluating connections to the SFP with penetrations below the top of the fuel is outlined in Figure 7-1.

The first step is to determine whether a failure of system connected through the penetration in question could drain water from the pool at a rate sufficient to lead to uncovering of the fuel within 72 hours. Note that, for a typical SFP, even an opening with an effective diameter of 1 in. at or near the bottom of the SFP could result in a drainage flow rate, at least initially, on the order of 100 gpm. For a nominal SFP containing approximately 400,000 gal, such a flow rate would be sufficient to lower level to below the top of the fuel within about 50 hours. Therefore, only very small penetrations could be eliminated based solely on the length of time it would take to lower the level in the pool below the top of the fuel.

If the failure of interest is that of a fuel transfer gate, the time it would take to drain down to the top of the fuel could be substantially less than 72 hours.

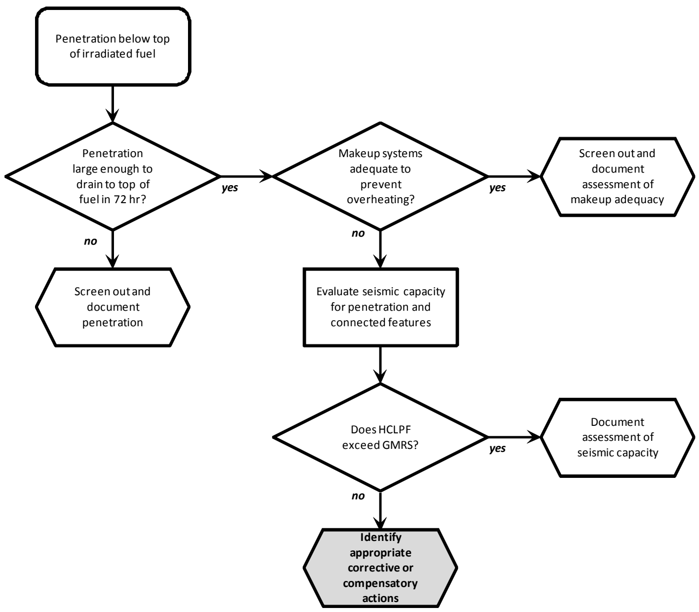


Figure 7-1
Basic Process for Evaluation of Potential Failures for Penetrations below Top of Irradiated Fuel

Comment [E24]: Change bottom outcome to PHASE2

Thus, the first step may be irrelevant for all but the smallest potential penetration failures. These very small penetrations should be screened from further analysis on the basis of the very long time it would take to drain the SFP to the top of the irradiated fuel. The evaluation of the penetrations should be documented for future reference.

If the penetration cannot be screened out, the next step is to determine whether provisions to make up to the SFP following an earthquake are sufficient to keep the fuel cooled. Two primary elements need to be considered together in performing this part of the evaluation:

- The location of the penetration relative to the top of the fuel, and
- The potential size of the failure.

One consideration is that a significant failure low in the pool has the potential to drain water from the pool at a rate in excess of readily available makeup provisions. If the penetration is above about two-thirds of the height of the fuel assemblies in the pool, however, maintaining the water level at that point should prevent overheating the fuel [49]. So, for example, if the transfer gate extends down to 2 ft below the top of the fuel, its failure may be acceptable, even though it may not be possible to restore water level to above the top of the fuel.

Makeup capabilities that might be considered would include normal plant systems (if they are seismically rugged) and capabilities installed as part of the FLEX initiative (which are intended to be rugged). Current requirements for FLEX capabilities call for being able to make up at a rate of at least 100 gpm, or to spray the fuel at a rate of at least 200 gpm for larger failures [50].

Comment [E25]: Will FLEX equipment be available? Given that we have a GMRS that exceeds the SSE.

Lowering the level in the pool will also interrupt normal SFP cooling (even if the SFP cooling system itself is not affected by the earthquake). Therefore, makeup sufficient to match boil-off from the SFP must be provided. The makeup required to match decay heat if the SFP does not have fuel assemblies freshly removed from the reactor may be as low as 20 to 30 gpm. For an SFP that contains freshly offloaded fuel, the decay heat load may be three times as high. Plants routinely maintain information needed to calculate the heat load in the SFP. Straightforward guidance for calculating the required makeup rates can be found in Appendix EE of the report documenting the technical bases for severe accident management guidance (SAMG) [49].

Finally, timing comes into consideration. Actions to restore level and ensure continued cooling of the fuel need to be accomplished before level decreases to two-thirds of the height of the fuel assembly. If actions need to be taken in the vicinity of the SFP, however, the time may be much shorter. As the level in the SFP decreases, the shielding normally provided by the water also decreases. The time available before the SFP area would no longer be habitable may be much shorter than the time it would take to uncover the fuel.

Comment [E26]: Include consideration for sloshing

Therefore, this portion of the analysis requires evaluating the following:

- The rate at which makeup is needed to prevent draining water below the acceptable level (about two-thirds of the height of the fuel assemblies) and to match boil-off due to decay heat in the pool;
- The capacity of makeup systems that would remain available following the earthquake;
- The ability of the FLEX spray function to prevent damage to the fuel if level cannot be maintained;
- The time available to effect these makeup provisions[1]; and

[1] Note that the estimation of time available should account for the possibility that a portion of the inventory in the SFP will be lost at the outset due to sloshing. This is addressed in Section 7.3.2.

- The feasibility of performing any manual actions required for establishing makeup, including the time available and the implications of reduced shielding in the SFP area.

If the evaluation of these aspects of responding to a failure concludes that makeup may not be sufficient or that it may not be possible to implement it in time, an assessment of the seismic capacity of the feature potentially subject to failure is needed. This assessment should establish the HCLPF value for the affected portion of the system and should compare the estimated HCLPF to the GMRS.

7.2.2 Evaluation of Penetrations above Top of Fuel

In most cases, penetrations in the SFP will be located above the top of the irradiated fuel. Assessment of these penetrations does not need to account for the potential that a failure would, in and of itself, result in draining the pool level below the fuel. Failures of these penetrations could, however, still affect SFP cooling. If the level in the pool could be lowered sufficiently due to a failure associated with a connection via such a penetration, SFP cooling could be interrupted, and the volume in the pool serving as a heat sink for the residual decay heat in the fuel assemblies could be reduced. This, in turn, would decrease the amount of time available to take corrective action and could ultimately lead to boil-off sufficient to uncover the fuel.

The process for evaluating this type of penetration is shown in Figure 7-2. In this case, the evaluation should first determine whether the potential failure could lead to uncovering the fuel within 72 hours. This time would reflect the following components:

- The rate at which leakage through the failed connection caused level in the SFP to drop (until the bottom of the penetration was reached). For a relatively large potential failure (such as that of the fuel transfer gate), the analysis should begin with an assumption that the level in the SFP drops to the bottom of the penetration at essentially the same time as when the failure occurred. For smaller failures, the time required to lower pool level to the bottom of the penetration may be significant.
- The amount of water lost due to sloshing (refer to Section 7.3.2 for guidance in addressing this consideration).
- The amount of time it would take to heat up the pool and boil off sufficient inventory after SFP cooling was lost. As addressed in Section 7.2.1, guidance for performing such calculations can be found in Appendix EE of the SAMG Technical Basis Report [49].

For a failure associated with a penetration above the top of the fuel, the loss of inventory through the break will be limited to the level of the penetration. Therefore, the makeup requirements are only those associated with matching decay heat. Even if normal makeup systems are not immediately available, FLEX capabilities should provide ample makeup (either directly or via the spray function) to assure that fuel remains adequately cooled.

Thus, the evaluation should document the assessment of the penetration, including the provisions for makeup to prevent overheating the stored fuel.

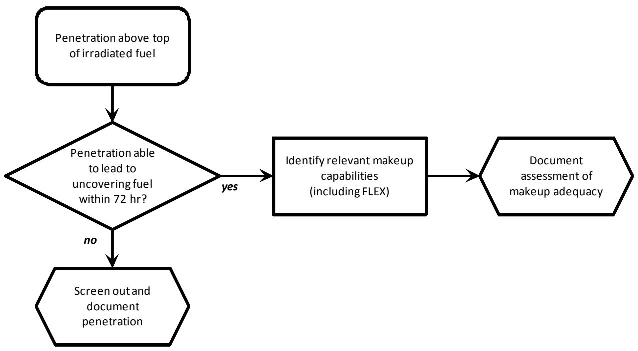


Figure 7-2
Basic Process for Evaluation of Potential Failures for Penetrations above Top of Irradiated Fuel

Comment [E27]: Consider whether should follow Figure 7-1 logic

7.2.3 Evaluation of Potential for Siphoning Inventory

Although designs differ from plant to plant, for some SFPs the discharge line from the SFP cooling system extends down into the pool. Cool water is introduced low in the pool, and the suction line takes warm water from closer to the top of the pool. If the SFP cooling system were to experience a failure, it is possible that water could be siphoned back through the discharge line and out the break. To prevent such an occurrence, SFP cooling systems with this configuration are typically equipped with anti-siphon devices. If the anti-siphon device were to function improperly, the effect would essentially be equivalent to a break below the top of the fuel, as addressed in Section 7.2.1. Thus, the process for evaluating failures in the SFP cooling system that might lead to siphoning inventory from the pool is outlined in Figure 7-3.

The anti-siphon devices are expected to be relatively rugged; for purposes of this evaluation, an evaluation should be made to confirm that, if such a feature is needed to prevent siphoning water from the pool. If there are questions about the ruggedness of the feature, the evaluation may follow one of three paths, depending on what information is most readily available:

- The capacity of the anti-siphon feature can be assessed and the resulting HCLPF compared to the GMRS;

- The SFP cooling system can be examined to determine if there are effective isolation features that could be used to terminate the loss of inventory; or
- An evaluation of makeup capabilities could be made, as for other breaks below the level of the fuel.

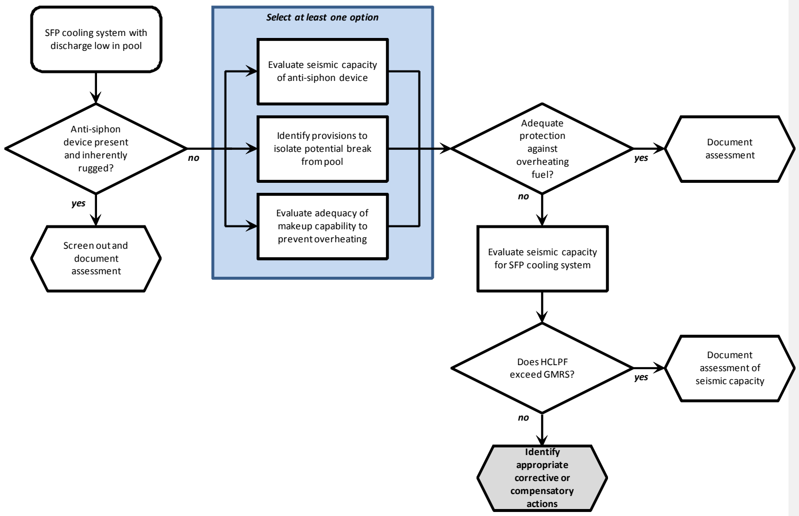


Figure 7-3
Basic Process for Evaluation of Potential Siphoning of SFP Inventory

Note that, even if the break in the SFP cooling system could stop the loss of inventory from the SFP, it would still be necessary eventually to establish makeup to the SFP because of the interruption of the cooling function.

In the very unlikely event that none of these options is viable, an evaluation can be made of the seismic capacity of the SFP system (analogous to the assessment called for in Section 7.2.1 for penetrations below the top of the stored fuel).

7.3 Guidance for Additional Evaluations

To accomplish the tasks outlined in the preceding section, additional evaluations may be required. This section provides guidance for the assessment of the timing of uncovering fuel and for addressing the effects of sloshing.

7.3.1 Draindown and Evaporative Losses

The evaluation of whether fuel could be uncovered in the event of a failure of an interconnection at a level above the fuel can be accomplished in a relatively straightforward manner.

For failures of piping systems connected above the top of the fuel, a flow rate can be approximated using standard correlations, and assuming a driving head equivalent to the initial height of water above the top of the connection. This flow rate can be used to bound the time it would take lower level to that of the connection.

For larger connections (such as the gate used for transferring fuel during refueling), the level can be assumed to drop to the bottom of the connection nearly instantaneously.

Once level drops to the connection, a calculation can be made to determine the time it would take to boil off inventory sufficient to begin uncovering fuel in the absence of makeup flow. This time can be determined using the correlations provided in Appendix EE of the report documenting the technical bases for severe accident management guidance [49].

These times can then be used to determine (a) whether the top of the fuel could begin to be uncovered within 72 hours, and (b) if so, how much time would be available for the operators to effect adequate makeup to the SFP.

7.3.2 Assessment of the Potential for Sloshing

To support the timing assessments described in Section 7.2, an estimate is needed of the amount of water lost from the SFP due to sloshing. An initial, bounding assessment can be made using the approach described in this section.

The natural frequency (f_{c1}) for the fundamental convective (sloshing) mode of vertical oscillation of the water surface in a rectangular pool due to shaking input in either horizontal direction can be expressed as follows:

$$f_{c1} = (½\ \pi)[3.16g / L) \tanh(3.16h / L)]^{0.5} \qquad \text{Equation 7-1}$$

Comment [E28]: fix

where: L = pool length in the direction of shaking
h = water depth
g = gravity

Next, the slosh height (h_{s1}) for the fundamental convective mode can be estimated from:

$$h_{s1} = ½L(SA_{c1} / g) \qquad \text{Equation 7-2}$$

where: SA_{c1} = ½% damped horizontal spectral acceleration at the top of the pool wall at the frequency f_{c1} in the direction of motion

In order to account for higher convective modes of sloshing and nonlinear sloshing effects (more upward splash than downward movement) observed during stronger shaking, the theoretical slosh height predicted by Equation 7-2 may be increased by 20%. Thus, the total estimated slosh height becomes:

$$h_s = 0.6L(SA_{c1} / g) \qquad \text{Equation 7-3}$$

For a rectangular pool of length a in the x-direction, and width b in the y-direction, the slosh height due to x-direction shaking, and y-direction shaking can be computed independently by substituting a and b, respectively, into Equations 7-1 and 7-3. Next, the total slosh height (h_{st}) can be estimated from:

$$h_{st} = [h_{sx}^2 + h_{sy}^2]^{0.5} \qquad \text{Equation 7-4}$$

where: h_{sx} = slosh height due to x shaking
h_{sy} = slosh height due to y shaking

An upper bound estimate of the total volume V of water that might splash out of the pool can be estimated from:

$$V = (h_{st} - h_f)ab \qquad \text{Equation 7-5}$$

where: h_f = freeboard height of the wall above the top of the water

Note that this approach reflects that sloshing in a pool is a very low frequency phenomenon governed by either the peak ground displacement or the peak ground velocity of the ground motion. It is independent of the PGA of the ground motion.

While this approach is expected to produce a reasonable estimate of the slosh height, it is expected to produce a very conservative estimate of the volume of water displaced from the pool. It effectively assumes that a solid mass of water equivalent to the product of the splash height above the side of the pool and the pool area is lost from the pool.

This relatively simple calculation may be adequate for purposes of estimating the timing associated with pool draindown. For most possible penetrations, it is judged that the conservative estimate of the volume lost due to sloshing will not have a significant effect on the estimate of the time it takes to drain the pool and to boil off inventory to the top of the stored fuel.

If for a penetration into the SFP of a particular size and at a particular depth below the water the volume lost due to sloshing has a significant impact on the timing of scenarios involving uncovering and overheating of stored fuel, a more careful calculation may be required. Such a calculation would need to account for the time histories of a range of earthquakes, and is likely to require significant resources. These more extensive calculations may also be needed to support later evaluations of flooding induced by an earthquake. Such would be the case if differences in the volume of water lost due to sloshing could affect which equipment could be subjected to flooding.

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Comment [JMR29]: Shouldn't this be a real reference? Is there a letter or something else we can reference?

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Appendix A: Control Point Discussion from Standard Review Plan

NUREG-0800 USNRC Standard Review Plan Rev. 2 1989:

> The FSAR 2.5.2 Vibratory Ground Motion described the development of the site SSE. Typically a peak ground acceleration (PGA) is determined and a generic spectral shape was defined; e.g., Housner spectra, Modified Newmark spectra, RG 1.60 spectra.
>
> In FSAR 3.7.1 the implementation of the SSE ground motion for seismic analysis and design is described. As discussed above the methodologies for seismic analysis and design varied depending on the vintage of the Plant.

NUREG-0800 Rev. 2 August 1989 provides the acceptance criteria for the later set of existing NNP designs:

> 3.7.1 Seismic Design Parameters states under 1. Design Ground Motion the following:
>
> > "The control motion should be defined to be a free ground surface...Two cases are identified depending on the soil characteristics at the site...uniform sites of soil or rock with smooth variation of properties with depth, the control point (location at which the control motion is applied) should be specified on the soil surface at the top of finished grade...for sites composed of one or more thin soil layers overlaying a competent material...the control point is specified on an outcrop or a hypothetical outcrop at a location on the top of the competent material..."
>
> 3.7.2 Seismic System Analysis states under II Acceptance Criteria the following:

"Specific criteria necessary to meet the relevant requirements of GDC 2 and Appendix A to Part 100 are as follows:

4. Soil-Structure Interaction...

C. Generation of Excitation System...

The control point...for profile consisting of component soil or rock, with relatively uniform variations of properties with depth the control motion should be located...at top of the finish grade...For profiles consisting of one or more thin soil layers overlaying component material, the control motion should be located at an outcrop (real or hypothetical) at top of the competent material...

...The spectral amplitude of the acceleration response spectra (horizontal component of motion) in the free field at the foundation depth shall be not less than 60 percent of the corresponding design response spectra at the finish grade in the free field..."

Appendix B: Development of Site-Specific Amplification Factors

B1.0 Introduction

It has long been recognized that the amplitude and frequency content of ground motions at a site are strongly influenced by the characteristics of the near-surface materials. For most sites, however, the properties of the near-surface materials and the parameters that control the dynamic response are not known with certainty. The uncertainty in these parameters needs to be accounted for when developing site-specific hazard curves. Ultimately, the goal or objective of the site response analysis is to produce site-specific hazard curves and response spectra which reflect the desired exceedance frequencies, that is, preserve the reference site annual exceedance frequency (AEF) thereby maintaining hazard consistency for results produced at any elevation in the profile. However, the uncertainty in characterizing the soil profile and dynamic properties of the near-surface materials presents a challenge to preserving hazard levels for sites that differ from some specified reference condition.

Previously, the state of practice in calculating a site-specific ground motion has been to calculate probabilistic reference rock ground motions and then multiply them by deterministic site-amplification factors [13, 14]. However, as stated above, there is uncertainty in the layering, spatial distribution, and dynamic properties of near-surface materials. This leads to uncertainty in the estimation of site amplification functions. To alleviate this problem it is necessary to calculate the effects of uncertainty on the estimate of the site-amplification functions and use the resulting site-amplification distribution within a probabilistic methodology [15, 16, 17].

The first step in developing site-specific seismic hazard curves and response spectra consists of performing a PSHA that reflects an outcropping reference site condition. The reference site condition is usually hard or firm rock and is consistent with assumptions made in the development of the most recent ground motion prediction equations. For central and eastern North America (CENA) this represents a site with a theoretical shear wave velocity over the top 1 km of the crust of 2.83 km/sec with a specified shallow crustal damping parameter [12]. The shear-wave velocity is based on the empirical Mid-continent compressional-wave velocity model of [18], taken by EPRI (1993) to represent the CENA, and an assumed Poisson ratio. For western U.S. (WUS) sites an appropriate reference condition should be selected that is well-constrained by observational data in the GMPEs. Site-specific amplification functions are then developed relative to the reference site condition.

After completing PSHA calculations for reference rock site conditions, the development of consistent, site-specific horizontal seismic hazard results may be considered as involving two independent analyses. The first is the development of frequency- and amplitude-dependent relative amplification factors (for 5% damped response spectra, Sa) between the site of interest and the reference site ($Sa_{SITE}(f)/Sa_{REFERENCE}(f)$) that accommodates potential linear or nonlinear site response. Currently the state-of-practice approach involves vertically propagating shear-waves and approximations using equivalent-linear analysis using either a time domain method (e.g., SHAKE) or a more computationally efficient frequency domain random vibration theory (RVT) method [28].

Subsequent to the development of the amplification factors, site-specific motions are computed by scaling the reference site motions with the transfer functions. As suggested above, probabilistic methods have been developed [15, 17] that accurately preserve the reference site hazard level and result in full site-specific hazard curves. These fully probabilistic approaches represent a viable and preferred mechanism to properly incorporate the site-specific aleatory (randomness) and epistemic (uncertainty) variabilities of the dynamic properties and achieve desired hazard levels and performance goals. The following sections describe the specific steps in the development of the site-specific amplification functions.

B2.0 Description of Sites Requiring Response Analysis and Basis for Alternative Models

The level of detail and scope of the geological and geotechnical investigations conducted during the licensing of the currently operating NPPS was consistent with the state of the practice at the time of the plants design and licensing. However, the state of the practice in earthquake engineering has evolved over the last several decades. As a result, some of the detailed information required to perform modern site response analyses (consistent with the request in the March 12, 2012 50.54(f) letter [24] are lacking for some of the older plants. This lack of information results in increased levels of uncertainty in the site response analyses. The following sections describe how this uncertainty will be accommodated in the site response analyses. The amount, quality and applicability of the available data will determine the analysis procedures.

The information available to develop estimates of site properties and characteristics will be primarily based on readily available sources (FSAR and other regional data) for most locations. However, for sites with recent COL and ESP submittals, the co-located operating plants would be expected to utilize any applicable information developed in the ESP and COL site characterization process to the maximum extent practicable.

Site response analyses will be required for sites in CENA (i.e., those sites located east of the Rocky Mountains) when available information suggests surficial materials will impact design motions at frequencies below about 50 Hz. The conservative criteria used to determine if site-specific amplification functions are required is more than 25 ft (7.5 m) of material with an average shear-wave velocity less than 7,500 ft/s (2286 m/s) over hard rock. Site-specific response analyses will be required for all sites in the western United States (sites west of the Rocky Mountain front).

Based on the need to determine if a facility requires detailed site response analyses (the combined stiffness and velocity criteria described above), the first step in the process is the compilation and evaluation of site geotechnical and geophysical characteristics. This information should be summarized consistent with the documentation described in Section 4 of the main report. The available site-specific information will be highly variable in terms quantity and applicability, and this range in available site-characterization data and information will necessitate several different approaches to developing site amplification functions. The different approaches are described more fully below.

B2.1 Background on the Treatment of Uncertainties

There are two different types of uncertainty in the development of site-specific amplification functions (AF(*f*)). First, at any given site, at the spatial dimensions of typical nuclear facilities (100-200 m (~325-650 ft) scale dimensions), there is expected to be some variability in shear-wave velocities and dynamic properties at any depth across the footprint of the facility. It is important to attempt to capture this uncertainty in the final AF(*f*) estimates. This is treated as an aleatory (randomness) type of variability. Current practice represents this variability by developing a candidate shear-wave velocity profile and associated dynamic properties (shear modulus reduction and damping curves). This is referred to as a "base-case" model. Subsequently, potential variations in shear-wave velocity and layer thickness are represented by correlated random perturbations to the base-case values. This is frequently referred to as a randomization process. A sufficient number of realizations (thirty or more) are used to develop statistical estimates (log median and log standard deviation) of the amplification functions.

The second type of uncertainty is epistemic or lack of knowledge uncertainty. This represents the uncertainty in the development of the base-case models for site profile, dynamic properties, and seismological parameters. For well characterized sites with abundant high quality data, this uncertainty would be assumed to be quite low or zero. This epistemic uncertainty would increase with decreasing confidence in the available data and information. This uncertainty is evaluated through the development of alternative base-case models. The approach applied for the development of alternative base-case models (epistemic uncertainty) is discussed in more detail in the following sections.

The following information is required to perform the site-specific response analyses: site shear-wave velocity profiles, dynamic material properties, estimates of low-strain site damping (parameterized through the parameter, kappa), and input or control motions (including relevant seismological parameters). These various factors are discussed individually in the following sections.

B3.0 Development of Base-Case Profiles and Assessment of Epistemic Uncertainty in Profiles and Dynamic Material Properties

Epistemic uncertainty in dynamic material properties, which includes shear-wave velocity profiles, site material damping at low strain (parameterized through kappa), and modulus reduction and hysteretic damping curves, should be accommodated through the development of alternative mean cases. The specific methodology utilized to develop the alternative cases will depend on the amount of information available at a given site. Conceptually in this context, for poorly characterized sites with few if any measured dynamic material properties, multiple cases should be developed with broad ranges of epistemic uncertainty applied in the development of the parameters of the alternative cases. For sites that have more complete site characterization information available, smaller epistemic uncertainty factors can be employed in the development of the alternative cases.

For those cases where limited or no at-site information is available, three profile estimates combined with three kappa estimates and two sets of modulus reduction and hysteretic damping curves should be developed. The three cases for shear-wave velocity profiles and kappa are referred to hereafter as base-case, and upper-range and lower-range models. A general set of guidelines should be employed to develop these cases for dynamic material properties and associated weights and is described more fully below. The general computational framework for developing the mean site amplification functions and associated standard deviations is illustrated in Figure B-1.

B3.1 Development Process for Base-Case Shear Wave Velocity Profiles

In order to predict site response as accurately as possible, and ultimately prevent error from propagating into other engineering calculations, it is important to define a detailed shear wave velocity profile that represents the known or inferred in-situ velocity structure as realistically as possible. The following discussion describes the development of the mean or base-case V_S profile. The alternative (upper-range and lower-range) models are derived from the base-case model utilizing an information-informed epistemic factor. The development of the upper-range and lower-range models is discussed in Section B-3.2 after the base-case development.

For sites with sparse or very limited information regarding dynamic material properties (e.g., a measured shear-wave velocity profile was unavailable), typically an estimate based on limited surveys (e.g., compressional-wave refraction) is available over some shallow, limited depth range. For such cases, as well as to provide a basis for extrapolating profiles specified over shallow depths to hard rock basement material, a suite of profile velocity templates has been developed, parameterized with V_{S30} (time averaged shear-wave velocity over upper 30 m of the profile) ranging from 190 m/s to 2,032 m/s (620 ft/s to 6,670 ft/s). The suite of profile templates is shown in Figure B-2 to a depth of 305 m (1,000 ft). The templates are from [32] supplemented for the current application with profiles for V_{S30} values of 190 m/s, 1,364 m/s, and 2,032 m/s. The latter two profiles were added to accommodate cases where residual soils (saprolite) are present and overly hard rock. For both soil and soft rock sites, the profile with the closest velocities over the appropriate depth range should be adopted from the suite of profile templates and adjusted by increasing or decreasing the template velocities or, in some cases, stripping off material to match the velocity estimates provided.

For intermediate cases, such as when only the upper portion of a deep soil profile is constrained with measured velocities, the V_S template profile with velocities closest to the observed velocity at the appropriate depth should be identified. This template can then be used to provide a rational basis to extrapolate the profile to the required depth.

For soft or firm rock sites, which are often composed of Cenozoic or Paleozoic sedimentary rocks such as shales, sandstones, siltstones, or similar rock types, a constant shear-wave velocity gradient of 0.5 m/s/m (0.5 ft/s/ft) should be used as a template and used to estimate the velocities over the appropriate depth range. This gradient is based on deep measurements in similar rock types in Japan [19] The 0.5 m/s/m velocity gradient is also consistent with sedimentary rocks of similar type at the Varian well in Parkfield, California [20]. It is recognized that the soil or firm rock gradients in the original profiles are primarily driven by confining pressure and may not be strictly correct for each adjusted profile template at each site. However, any shortcoming in the assumed gradient is not expected to be significant as the range in multiple base-case profiles accommodates the effects of epistemic uncertainty in the profile gradient on the resulting amplification functions.

For all sites where limited data exists, or only exists for very shallow depths, it is necessary to fully evaluate and integrate all existing geological information into the development of the base-case profile. For sites with soil or soft rock at the surface and much stiffer materials at relatively shallow depths (less than approximately 200 ft (60m)) the potential for strong resonance in the frequency range of engineering interest exists. All relevant geological information should be assessed to ensure this condition is identified.

An example is provided in Figure B-3 to schematically illustrate how a combination of geological information and geophysical measurements may be used to develop a base-case profile. The data available at this hypothetical site consists of shallow shear-wave velocity measurements (a single S-wave refraction profile) over only about the upper 100 ft (31 m) of the profile with a V_{S30} of approximately 450 m/s. There are also geologic profiles and regional data available in the FSAR that indicate firm rock is present at a depth about 45 m (150 ft) beneath the site. A shear-wave velocity of approximately1,525 m/s (5,000 ft/s) is inferred for the firm rock based on velocity measurements on comparable material elsewhere. Regional data indicates the firm, sedimentary rocks extend to a depth of at least a kilometer before crystalline basement rock is encountered. The information is combined in the following manner to construct a base-case profile. The closest template profile to the 450 m/s V_{S30} estimate is the 400 m/s profile. The velocities in the 400 m/s template are scaled by a factor of 1.125 (450/400) to adjust to the desired V_{S30} value. At the 45 m (150 ft) depth, a velocity discontinuity is inserted with a velocity of 1,525 m/s (5,000 ft/s). Below this depth, the firm rock gradient model of 0.5 m/s/m is used to estimate velocities. This gradient is extended to a sufficient depth such that 2,830 m/sec is reached, or the depth is greater than the criteria for no influence on response for frequencies greater than 0.5 Hz. The uncertainty in the depth to the soil-firm rock interface is incorporated in the treatment of epistemic uncertainty as discussed below.

B3.2 Capturing Epistemic Uncertainty in Velocity Profiles

There are basically two approaches for constructing shear wave velocity profiles, either through inference from geotechnical/geologic information or through the use of geophysical measurements. Each approach will inherently have some level of uncertainty associated with its ability to accurately represent the in-situ velocity structure. The level of uncertainty will depend on the amount of information available along with how well the information is correlated with shear-wave velocity. By adopting the general mean based approach outlined in Section B-3.1, a level of uncertainty can be assigned to a template velocity profile, commensurate with the available information, in order to account for the epistemic uncertainty associated with the in-situ velocity structure.

For sites where geophysical information such as limited shear-wave velocity data exists or compressional-wave velocities are used to infer shear-wave velocities, the estimate for uncertainty is to be taken as:

$$\sigma_{\mu} \ln = 0.35$$

This value is similar to a Coefficient of Variation (COV) of 0.25, which is consistent with [36] for observed spatial variability over a structural footprint of several hundred meters. The profile epistemic uncertainty factor of 0.35 (σ_{μ} ln) is to be applied throughout the profile and is based on the estimates of epistemic uncertainty in V_{S30} developed for stiff profiles [9]. The logarithmic factor assumes shear-wave velocities are lognormally distributed and was originally developed to characterize the epistemic uncertainty in measured V_{S30} at ground motion recording sites where measurements were taken within 300 m (900 ft) from the actual site. The uncertainty accommodates spatial variability over maximum distances of 300 m, and is adopted here as a reasonable and realistic uncertainty assessment reflecting a combination of (1) few velocity measurements over varying depth ranges, (2) shear-wave velocities inferred from compressional-wave measurements, and (3) the spatial variability associated with observed velocities. While velocities are undoubtedly correlated with depth beyond 30 m, which forms the basis for the use of V_{S30} as an indicator of relative site amplification over a wide frequency range, clearly the correlation is neither perfect nor remains high over unlimited depths [6]. An example of the resulting mean $\pm\sigma_{\mu}$ ln shear-wave profiles for the 760 m/s template is shown in Figure B-4.

For sites where site-specific velocity measurements are particularly sparse (e.g., based on inference from geotechnical/geologic information rather than geophysical measurements) a conservative estimate of the uncertainty associated with the template velocity is to be taken as:

$$\sigma_{\mu} \ln = 0.5$$

For sites were detailed shear-wave velocity profiles are available, the level of uncertainty may be taken as zero if justified.

B3.2.1 Epistemic Uncertainty in Final Hazard Calculations

It is necessary to represent the epistemic uncertainty in the distribution of potential shear-wave velocity profiles (mean base-case and a σ_{μ} (ln) of 0.35, for example) in the final site-specific hazard results. Practicality requires this be accomplished with the minimum number of cases. The recommended approach for this application is to utilize three cases, the mean base-case and upper range and lower range base-cases with relative weights applied. An accurate three point approximation of a normal distribution which preserves the mean utilizes the 50th percentile (median) and 10th and 90th percentiles, with relative weights of 0.30 for the 10th and 90th percentiles and 0.40 for the median applied [21]. These values are summarized in Table B-1. The 10th and 90th fractiles correspond to a profile scale factor of 1.28 σμ. When $\sigma_\mu ln = 0.35$ the 10th or 90th percentiles are obtained by subtracting or adding 0.45 in natural log units to the shear wave velocity. For $\sigma_\mu ln = 0.5$, a value of 0.64 is subtracted or added to the natural log of the shear wave velocity for the 10th and 90th percentile values. This is equivalent to an absolute factors of 1.57 or 1.90 applied to the mean base-case profile for $\sigma_\mu ln = 0.35$ or $\sigma_\mu ln = 0.5$, respectively. Figure B-5 illustrates application of these two factors applied to the 760 m/s (1525 ft/s) V_{S30} template. Figure B-6 illustrates the same type of curves for the firm rock template derived using the empirical gradient of [19]. For some individual sites it may be necessary to deviate from these standard weights if application of the standard factors results in velocities that are not deemed credible.

Figure B-7 illustrates the development of Upper Range and Lower Range profiles to accommodate epistemic uncertainties for the hypothetical example shown in Figure B-3. A σ_μ ln = 0.35 has been used to develop the 10th and 90th-percentile curves in the upper portion of the profile where sparse V_s measurements were available. A σ_μ ln = 0.50 was applied to the lower portion of the profile where the V_s of the Base-Case was inferred from geological information. The 90th-percentile curve was capped at a value equal to the 2,830 m/s V_s value assumed for the hard rock basement. This example illustrates the broad range of velocities encompassed by the Upper Range, Mean, and Lower Range profiles for sites lacking in good data.

B3.3 Nonlinear Dynamic Material Properties

The potential nonlinear response of near-surface materials to input ground motions is an important element of the site that needs to be characterized in a proper site response analysis. To characterize the epistemic uncertainty in nonlinear dynamic material properties for both soil, and soft/firm rock sites, the use of two sets of modulus reduction and hysteretic damping curves is suggested.

For soils, the two sets of proposed curves are the EPRI (1993) and Peninsular Range [28, 32]) results. The two sets of generic curves are appropriate for cohesionless soils comprised of sands, gravels, silts, and low plasticity clays. The EPRI (1993) curves, illustrated in Figure B-7, were developed for application to CENA sites and display a moderate degree of nonlinearity. The EPRI (1993) curves are depth (confining pressure) dependent as shown in Figure B-7.

The Peninsular Range curves reflect more linear cyclic shear strain dependencies than the EPRI (1993) curves [32] and were developed by modeling recorded motions as well as empirical soil amplification in the Abrahamson and Silva WNA (Western North America) GMPE [28, 1]. The Peninsular Range curves reflect a subset of the EPRI (1993) soil curves with the 51 to 120 ft (15 to 37 m) EPRI (1993) curves applied to the 0 to 50 ft (0 to 15 m) depth range and the EPRI (1993) 501 to 1,000 ft (153 to 305 m) curve applied to the 51 to 500 ft (15 to 152 m) depth range, below which linear behavior is assumed.

The two sets of soil curves are considered to reflect a realistic range in nonlinear dynamic material properties for cohesionless soils. The use of these two sets of cohesionless soil curves implicitly assumes the soils considered do not have response dominated by soft and highly plastic clays or coarse gravels or cobbles. The presence of relatively thin layers of hard plastic clays are considered to be accommodated with the more linear Peninsular Range curves while the presence of gravely layers are accommodated with the more nonlinear EPRI (1993) soil curves, all on a generic basis. The potential impact on the amplification functions of the use of these two sets of nonlinear dynamic property curves was evaluated and is shown in Figure B-8. The results indicate that above 1 Hz the difference can be significant and the resulting epistemic uncertainty needs to be included in the development of the final amplification functions.

The two sets of soil curves are given equal weights (Table B-1 and Figure B-1) and are considered to represent a reasonable accommodation of epistemic uncertainty in nonlinear dynamic material properties for the generic types of soils found at most in CEUS sites which include:

1. Glaciated regions which consist of both very shallow Holocene soils overlying tills as well as deep soils such as the Illinois and Michigan basins, all with underlying either firm rock (e.g., Devonian Shales) and then hard basement rock or simply hard basement rock outside the region of Devonian Shales,

2. Mississippi embayment soils including loess,

3. Atlantic and Gulf coastal plain soils which may include stiff hard clays such as the Cooper Marl,

4. Residual soils (saprolite) overlying hard metamorphic rock along the Piedmont and Blue Ridge physiographic regions.

For soft or firm rock site conditions, taken generally as Paleocene sedimentary rocks, such as shale, sandstones, or siltstones, two alternative expressions of nonlinear dynamic material behavior are proposed: the EPRI "rock curves" (Figure B-9) and linear response. The EPRI rock curves were developed during the EPRI (1993) project by assuming firm rock, with nominal shear-wave velocities in the range of about 914 m/s to 2,134 m/s (3,000 ft/s to about 7,000 ft/s, about 5,000 ft/s on average), behaves in a manner similar to gravels [12] being significantly more nonlinear with higher damping than more fine grained sandy soils. The rock curves were not included in the EPRI report as the final suite of amplification factors was based on soil profiles intended to capture the behavior of soils ranging from gravels to low plasticity sandy clays at CEUS nuclear power plants. With the stiffness typically associated with consolidated sedimentary rocks, cyclic shear strains remain relatively low compared to soils. Significant nonlinearity in the soft-to-firm rock materials largely confined to the very high loading levels (e.g., ≥ 0.75g).

As an alternative to the EPRI rock curves, linear response should be assumed. Implicit in this model is purely elastic response accompanied with damping that remains constant with cyclic shear strain at input loading levels up to and beyond 1.5g (reference site). Similar to the two sets of curves for soils, equal weights were given to the two sets of nonlinear properties for soft/firm rock sites as summarized in Table B-1.

B3.4 Densities

Because relative (soil surface/reference site) densities play a minor role in site-specific amplification, a simple model based on the shear-wave velocity of the mean base-case profile is proposed for those sites where a profile density is not available. This model relating estimated shear-wave velocity and density is summarized in Table B-2.

Due to the square root dependence of amplification on the relative density, a 20% change in soil density results in only a 10% change in amplification and only for frequencies at and above the column resonant frequency. As a result, only an approximate estimate of profile density is considered necessary with the densities of the mean base-case profile held constant for the upper and lower range base-case profiles. This approach provides a means of accommodating epistemic uncertainty in both density as well as shear-wave velocity (Section B3.1) in the suite of analyses over velocity uncertainty.

B4.0 Representation of Aleatory Variability in Site Response

To accommodate the aleatory variability in dynamic material properties that is expected to occur across each site (at the scale of the footprint of a typical nuclear facility), shear-wave velocity profiles as well as G/G_{MAX} and hysteretic damping curves should be randomized. The aleatory variability about each base-case set of dynamic material properties should be developed by randomizing (a minimum of thirty realizations) shear-wave velocities, layer thickness, depth to reference rock, and modulus reduction and hysteretic damping curves. For all the sites considered, where soil and soft rock extended to depths exceeding 150 m (500 ft), linear response can be assumed in the deep portions of the profile [28, 27, 26, 25].

B4.1 Randomization of Shear-Wave Velocities

The velocity randomization procedure makes use of random field models [36, 28] to generate V_S profiles. These models assume that the shear-wave velocity at any depth is lognormally distributed and correlated between adjacent layers. The layer thickness model also replicates the overall observed decrease in velocity fluctuations as depth increases. This realistic trend is accommodated through increasing layer thicknesses with increasing depth. The statistical parameters required for generation of the velocity profiles are the standard deviation of the natural log of the shear-wave velocity (σ_{lnVs}) and the interlayer correlation (ρ_{IL}). For the footprint correlation model, the empirical σ_{lnVs} is about 0.25 and decreases with depth to about 0.15 below about 15 m (50 ft) [28]. To prevent unrealistic velocity realizations, a bound of $\pm 2\sigma_{lnVs}$ should be imposed throughout the profile. In addition, randomly generated velocity should be limited to 2.83 km/s (9,200 ft/s).

B4.2 Aleatory Variability of Dynamic Material Properties

The aleatory variability about each base-case set of dynamic material properties (e.g., EPRI depth dependent vs. Peninsular) will be developed by randomizing modulus reduction and hysteretic damping curves for each of the thirty realizations. A log normal distribution may be assumed with a σ ln of 0.15 and 0.30 at a cyclic shear strain of 3 x 10^{-2}% for modulus reduction and hysteretic damping respectively [28]. Upper and lower bounds of $\pm 2\sigma$ should be applied. The truncation is necessary to prevent modulus reduction or damping models that are not physically realizable. The distribution is based on an analysis of variance of measured G/G_{MAX} and hysteretic damping curves and is considered appropriate for applications to generic (material type specific) nonlinear properties [28]. The random curves are generated by sampling the transformed normal distribution with a σ ln of 0.15 and 0.30 as appropriate, computing the change in normalized modulus reduction or percent damping at 3 x 10^{-2}% cyclic shear strain, and applying this factor at all strains. The random perturbation factor is then reduced or tapered near the ends of the strain range to preserve the general shape of the base-case curves [12, 28]. Damping should be limited to a maximum value of 15% in this application.

B5.0 Development of Input Motions

The ground motion used as input to site response analyses is commonly referred to as the "control motion." This can be reflected in time histories matched or scaled to a response spectrum or, in the case of Random

Vibration Theory, a power spectral density (PSD). Because of the very large number of cases that will need to evaluated to capture the range of epistemic uncertainty and aleatory variability in this application (Figure B-1 and Table B-3) the following discussion will assume that the much more efficient random vibration theory (RVT) approach to performing site response analyses will be utilized as opposed to a time series (TS)-based technique. Recent studies [37] have confirmed that the two approaches yield similar results. The following sections describe the model used in the development of the control motions and the parameters of that model that require an assessment of uncertainty.

B5.1 Simple Seismological Model to Develop Control Motions

The methodology suggested for developing the input or control motions relies on a widely used, simple seismological model to represent earthquake source, propagation path and site characteristics ([38] and references therein). The ground motions recorded at a given site from an earthquake can be represented in the frequency domain as:

$$Y(M_0, R, f) = E(M_0, f)\cdot P(R, f)\cdot G(f).$$

Where $Y(M_0,R,f)$ is the recorded ground motion Fourier amplitude spectrum, $E(M_0,f)$ is the seismic source spectrum, $P(R,f)$ represents the propagation path effects, and $G(f)$ represents the modification due to site effects. In this equation M_0 is the seismic moment of the earthquake, R is the distance from the source to the site, and f is frequency. The seismic moment and the earthquake magnitude are related through the definition of the moment magnitude, M [39]:

$$\text{Log } M_0 = 1.5M + 16.05.$$

The $P(R, f)$ term accounts for path effects, geometrical spreading and anelastic attenuation and can be expressed as:

$$P(R,f) = S(R)\ exp((-\pi f R)/(Q(f)V_S)).$$

Where $S(R)$ is the geometrical spreading function, $Q(f)$ is the seismic quality factor, and V_S is the shear-wave velocity in the upper crust.

The $G(f)$ term accounts for upper crustal amplification and diminution:

$$G(f) = A(f)\cdot D(f).$$

Where $A(f)$ is the amplification function relative to source depth velocity conditions ($A(f) = (Zsource/Zavg)^{0.5}$), where Z is the product of density and velocity (ρV_S) and D(f) represents the frequency-dependent diminution term ($D(f) = exp(-\pi\kappa_0 f)$).

Kappa (κ_0) is an upper crustal site ground motion attenuation parameter that accounts for the overall damping in the basement rock immediately beneath a site. The properties and behavior of the upper few hundreds of meters of the crust has been shown to produce as much as 50% or more of the total attenuation of the high-frequency portion of the ground motion spectrum [40; 3]. The value of kappa influences the shape of the ground motion spectrum observed at a given site. High values of kappa result in enhanced attenuation of the high-frequency portion of the spectrum.

Comment [CC30]: add to ref list

The factors in the simple seismological model that affect the spectral shape of the input motions are kappa, magnitude, attenuation model, and source model. These factors are discussed below. An example of the potential effect of these parameters on the spectral shape of the input ground motions (Fourier amplitude spectra) is shown in Figure B-10.

B5.1.1 Magnitude

Conditional on reference site peak acceleration, amplification factors depend, to some extent, upon control motion spectral shape due to the potential nonlinear response of the near-surface materials. For the same reference site peak acceleration, amplification factors developed with control motions reflecting M 5.5 will differ somewhat with those developed using a larger or smaller magnitude, for example.

Figure B-11 shows amplification factors developed for the 400 m/s V_{S30} template profile (Figure B-2) using the single-corner source model for magnitudes M 5.5, 6.5, and 7.5. For this sensitivity analysis the more nonlinear EPRI G/G_{MAX} and hysteretic damping curves (Figure B-7) were used. The dependence on control motion spectral shape is observed to decrease with degree of nonlinearity becoming independent for linear analyses. As Figure B-11 illustrates, the largest amplification reflects the lowest magnitude (M 5.5). Over the frequency range of about 5 to 10 Hz, and the ground motion amplitude range of most engineering interest (between 0.1g and 0.75g), the difference in the derived amplification functions between the magnitudes is minor. The largest difference in amplification is about 20% and at the highest loading levels (≥ 0.75g). The largest difference in amplification is between M 5.5 and M 6.5 with little difference (< 10%) between M 6.5 and M 7.5. With the current source characterization in CENA [NRC/EPRI CEUS Seismic Source Characterization report] and the distribution of existing NPP sites, the dominant contribution for the AEF of 10^{-4} and below are from magnitudes in the range of about M 6 to M 7+. Given these factors, and the large number of analyses required (Table B-3) a single magnitude (M 6.5) is proposed for development of the control motions. This is felt to adequately characterize the amplification, with tacit acceptance of slight conservatism for magnitude contributions above about M 7.

Comment [CC31]: add to ref list

B5.1.2 Attenuation Model

As illustrated in Figure B-10, major differences in the assumed crustal attenuation model will influence the spectral shape of the control motions. However, within a given tectonic region (e.g., CENA or WUS) changes in the crustal attenuation model do not contribute significantly to changes in the derived amplification functions. Appropriate, widely referenced crustal attenuation models are proposed for the CEUS and WUS sites (Table B-4).

B5.1.3 Kappa

In the context of this discussion, the kappa referred to here is the profile damping contributed by both intrinsic hysteretic damping as well as scattering due to wave propagation in heterogeneous material. Both the hysteretic intrinsic damping and the scattering damping within the near-surface profile and apart from the crust are assumed to be frequency independent, at least over the frequency range of interest for Fourier amplitude spectra (0.33 to about 25.0 Hz). As a result, the kappa estimates reflect values that would be expected to be measured based on empirical analyses of wavefields propagating throughout the profiles at low loading levels and reflect the effective damping or "effective" Qs within the profile [7]. Changes in kappa can exert a strong impact on derived control motion spectra and as a result are an important part of the input model for development of control motions. Hence, similar to the treatment of uncertainties in shear-wave velocity profiles, multiple base-cases (mean and upper and lower ranges) are developed for kappa.

B5.1.3.1 Development of Base Case kappa Models

Mean base-case kappa values were developed differently for soil and firm rock sites.

Rock Sites: For rock sites with at least 3,000 ft (1,000 m) of firm sedimentary rock (V_{S30} > 500 m/s) overlying hard rock, the kappa-V_{S100} (average shear-wave velocity over the upper 100ft of the profile) relationship of

$$\log(\kappa) = 2.2189 - 1.0930*\log(V_{S100}),$$

Where V_{S100} is in ft/s, is proposed to assign a mean base-case estimate for kappa [25, 22]. The requirement of a 3,000 ft (1,000 m) thickness of firm materials reflects the assumption that the majority of damping contributing to kappa occurs over the upper km of the crust with a minor contribution from deeper materials (e.g., 0.006s for hard rock basement material). As an example, for a firm sedimentary rock with a shear-wave velocity of 5,000 ft/s (1,525 m/s), this relationship produces a kappa estimate of about 0.02s. The assumption that is typically used implies a kappa of 0.014s is contributed by the sedimentary rock column and 0.006s from the underlying reference rock (Table B-4), and reflects an average Qs of about 40 over the 3,000 ft depth interval. The Q_s value of 40 for sedimentary rocks is consistent with the average value of 37 observed (measured) over the depth range of 0 to 298 m in Tertiary claystones, siltstones, sandstones, and conglomerates at a deep borehole in Parkfield, California [20].

For soft/firm rock cases with low estimated velocity values, an upper bound kappa value of 0.04s should be imposed. The maximum kappa value of about 0.04s reflects a conservative average for soft rock conditions [29, 28].

For cases where the thickness of firm rock is less than about 3,000 ft (1000 m) and the relationship cited above is not applicable, the kappa contributed by the firm rock profile can be computed assuming a Q_s of 40 plus the contribution of the reference rock profile of 0.006s (Table B-4). For the three base-cases firm rock template profiles shown in Figure B-5, the total kappa values assuming a Q_s of 40 are 0.019s, 0.025s, and 0.015s for the mean, lower range, and upper range base-cases respectively.

Soil Sites: For soil sites (either in the WUS or CEUS) with depths exceeding 3,000 ft (1,000 m) to hard rock, a mean base-case kappa of 0.04s should be assumed based upon observed average values for deep soil sites and low loading levels. The mean base-case kappa of 0.04s adopted for deep firm soils is lower than the value of approximately 0.06s based on recordings at alluvium sites located in Southern California [3, 28]. For soil sites, due to nonlinear effects, low strain kappa may be overestimated depending upon loading level and the nonlinear dynamic material properties. To avoid potential bias in the deep, firm soil, low strain kappa, the value of 0.04s is based on inversions of the Abrahamson and Silva [2] soil site GMPE [28]. In that inversion, a range of rock site loading levels was used with the soil value of 0.04s based upon a rock site peak acceleration of 5% g or less, clearly a low strain estimate. The deep soil mean base-case kappa of 0.04s is adopted for both the upper and lower range profiles with the assumption that the suite of profiles reflect deep firm soils. The assumed kappa of 0.04s for deep (≥ 3,000 ft) firm soils in the CEUS is somewhat less than the 0.054s inferred by Campbell [7] based on Cramer et al. [10] analyses for effective Qs within the 960 m deep sedimentary column in the Mississippi embayment near Memphis, Tennessee. The deep firm soil kappa of 0.04s is in fair agreement with 0.052s found by Chapman, et al. [8] for the 775m thick sedimentary column near Summerville, South Carolina.

In summary, for deep firm soil sites (≥ 3,000 ft (1,000 m) to basement rock) in the CEUS, a nominal kappa value of 0.04s based on an average of many empirical estimates predominately in the WNA tectonic regime is proposed. Sparse analyses for deep soil sites in the CEUS suggest 0.04s reflects some conservatism. However it should be noted the small strain total kappa is rapidly exceeded (i.e., becomes less important) as loading level increases due to nonlinear response. The initial low strain kappa serves primarily as a means of adjusting (lowering) kappa to accommodate the scattering component due to the profile randomization.

Hence, no significant bias in the final amplification functions at loading levels of engineering interest is anticipated.

For cases of shallower soils, less than 3,000 ft (1,000 m) to hard rock basement material, the empirical relation of Campbell [7] should be used for the contribution to kappa from the thickness of the sediment column (H):

$$\kappa \text{ (ms)} = 0.0605 * H \text{ (thickness in meters).}$$

The assumed basement kappa value of 0.006s (Table B-4) is used in lieu of Campbell's [7] estimate of 0.005s to estimate the sediment contribution to total kappa. For 3,000 ft (1 km) of soil, Campbell's [7] relation predicts a total kappa of 0.0665s (0.0605 contribution from soil and 0.006 contribution from basement rock), considerably larger than the mean base-case value of 0.04s, suggesting a degree of conservatism at low loading levels for CENA firm soils. For continuity, in the implementation of Campbell's equation, a maximum kappa of 0.04s was implemented for sites with less than 3,000 ft (1,000 m) of firm soils.

B5.1.3.2 Representation of Epistemic Uncertainty in Kappa

The parameter kappa is difficult to measure directly. Since no measurements of the type required exist at the sites of interest, a large uncertainty is applied in the site response analyses. Epistemic uncertainty in kappa is taken as 50% ($\sigma_{\mu Ln}$ = 0.40, Table B-1; [12]) about the mean base-case estimate for this assessment. The uncertainty is based on the variability in kappa determined for rock sites which recorded the 1989 M 6.9 Loma Prieta Earthquake [12], and adopted here as a reasonable expression of epistemic uncertainty at a given site. As with the shear-wave velocity profiles (Section 3.2.1), the ±0.51 natural log units (1.28 σ_{μ}) variation is considered to reflect 10% and 90% fractiles with weights of 0.30 and a weight of 0.40 for the mean base-case estimate. The models for epistemic uncertainty are summarized in Table B-1.

B5.1.4 Source Model

Alternative conceptual models to represent the earthquake source spectral shape exist in the literature. A single corner frequency model of the earthquake source spectrum has been widely used in the simple seismological model described above [41, 38, 18]. However, based on the limited ground motion data in CENA, as well as inferences from intensity observations, an alternative empirical two-corner source model for CENA earthquakes has been developed [42]. The two-corner source model addresses the potential for CENA source processes to reflect a significant spectral sag at large magnitude (M ≥ 6) and intermediate frequency [4], compared to source processes of tectonically active regions. Such a trend was suggested by the 1988 M 5.9 Saguenay, Canada, and 1985 M 6.8 Nahanni, Canada, earthquakes. The two-corner source model for CENA

[4] incorporates the spectral sag between two empirical corner frequencies, which are dependent on magnitude. The two-corner model merges to the single-corner model for M less than about M 5. Interestingly, the two-corner model has been implemented for tectonically active regions and shown to be more representative of WNA source processes than the single-corner model [43], albeit with a much less pronounced spectral sag than the CENA model.

The two-corner source model may be the more appropriate model for CENA strong ground motions. However, debate regarding the applicability of these two source models continues. The lack of relevant observations for M >6 in CENA precludes identifying either model as a preferred model. As a result, in the interest of representing the epistemic uncertainty in this element of the control motions, both single- and two-corner [4] source models were used with M 6.5 to develop control motions. The two models were considered to reflect a reasonable range in spectral composition for large magnitude CENA sources. As a result, equal weights were selected as shown in Table B-1 to develop amplification factors using each source model.

Additionally, for moderately stiff soils, typical for NPP siting, the difference in amplification between single- and double-corner source models becomes significant only at the higher loading levels as Figure B-12 illustrates. Figure B-12 compares the amplification computed for both the single- and double-corner source models using the EPRI modulus reduction and hysteretic damping curves (Figure B-7), the most nonlinear set of curves for soils. These results suggest the difference in amplification between single- vs. double-corner source models are significant enough to consider the implied epistemic uncertainty in CENA source processes at large magnitude (M >6).

B5.1.4.1 Development of Input Motions

It is necessary to define the site response over a broad range of input amplitudes to develop amplification functions. For sites in the CEUS, the mid-continent crustal model [12] (Table B-5) with a shear-wave velocity of 2830 m/s, a defined shallow crustal damping parameter (kappa; [3]) of 0.006 s, and a frequency dependent deep crustal damping Q model of 670 f0.33 [12] is used to compute reference motions (5% damped pseudo absolute acceleration spectra). The selected Q(f), kappa, and reference site shear-wave velocities are consistent with the EPRI GMPEs [11]. The site-specific profiles are simply placed on top of this defined crustal model which has a reference shear-wave velocity of 2,830 m/s (≈ 9,300 ft/s) and a reference kappa value of 0.006 s. Distances are then determined to generate a suite of reference site motions with expected peak acceleration values which cover the range of spectral accelerations (at frequencies of 0.5, 1.0, 2.5, 5.0, 10.0, 25.0, 100.0 Hz) anticipated at the sites analyzed. To cover the range in loading levels, eleven expected (median) peak acceleration values at reference rock are needed to span from 0.01g to 1.50g. Table B-4 lists the suite of distances for the single-corner source model, and Table B-6 lists the corresponding distances for the double-corner model.

Amplification factors (5% damping response spectra) are then developed by placing the site profile on the mid-continent crustal model at each distance with the input motion being equal to the reference rock motion convolved with a diminution function which implements the site specific kappa (e.g., kappa from the equations in Section 5.1.3 and a 0.006s contribution from basement rock) , generating soil motions, and taking the ratios of site-specific response spectra (5% damped) to hard rock reference site response spectra. For the higher levels of rock motions, above about 1 to 1.5g for the softer profiles, the high frequency amplification factors may be significantly less than 1, which may be exaggerated. To adjust the factors for these cases an empirical lower bound of 0.5 is to be implemented [12, 2].

The general framework for the site response calculations are summarized in Figure B-1 and Tables B-1 and B-3.

B6.0 Development of Probabilistic Hazard Curves

The procedure to develop probabilistic site-specific soil hazard curves was described by McGuire et al. [15] and Bazzurro and Cornell [17]. That procedure (referred to as Approach 3) computes a site-specific soil hazard curve for the spectral acceleration at a selected spectral frequency (or period) given the site-specific hazard curve for the bedrock spectral acceleration at the same oscillator period and site-specific estimates of soil response. The soil response is quantified through the period/frequency-dependent amplification factor, AF(f). The function AF(f) is given by:

$$AF(f) = Sa_{SOIL}(f)/Sa_{ROCK}(f),$$

where f is frequency, and $Sa_{SOIL}(f)$ and $Sa_{ROCK}(f)$, are the 5% damped spectral accelerations at the soil surface and bedrock, respectively. Since the near-surface materials frequently exhibit nonlinear behavior, the variation of $AF(f)$ with input intensity needs to be captured. Most commonly the input intensity is quantified by Sa_{ROCK} at the frequency of interest.

In the fully probabilistic approach, the annual probability of exceedance of soil ground motion level z (GZ(z)) at spectral frequency f is computed as:

$$G_Z(z) = \int_0^{\infty} P\left(AF \geq \frac{z}{x} \middle| x\right) f_X(x) dx$$

Where $P\left(AF \geq \frac{z}{x} \middle| x\right)$ is the probability that AF is greater than the quantity

$\frac{z}{x}$,

given a bedrock amplitude of x, and $f_X(x)$ is the probability density function of Sa_{ROCK}.

In discretized form, the above equation can be expressed as:

$$G_Z(z) = \sum_{all\ x_j} P\left[\left(AF \geq \frac{z}{x} \middle| x_j\right)\right] p_x(x_j)$$

Where $p_x(x_j)$ is the annual probability of occurrence for Sa_{ROCK} equal to x_j. This probability is obtained by differentiating the appropriate rock hazard curve.

Then, $P\left[\left(AF \geq \frac{z}{x} \middle| x_j\right)\right]$

can be computed by assuming *AF* is lognormally distributed and a function of *x,* since

$$P\left[\left(AF \geq \frac{z}{x} \middle| x\right)\right] = \varphi\left(\frac{\ln\frac{z}{x} - \mu_{(\ln AF|x)}}{\sigma_{(\ln AF|x)}}\right)$$

Where $\mu_{\ln AF|x}$ is the mean value of *ln AF* given $Sa_{ROCK} = x$, and $\sigma_{\ln AF|x}$ is the standard deviation of *ln AF* given $Sa_{ROCK} = x$. The term for $\varphi(*)$ is simply the standard Gaussian cumulative distribution function. The parameters $\mu_{\ln AF|x}$ and $\sigma_{\ln AF|x}$ are obtained from the distribution for AF derived from the site response analyses described above, and are a function of bedrock amplitude x.

The site amplification functions are to be developed as described in Sections B-1 through B-5. As discussed in those sections, multiple models of site amplification functions are derived. To compute site-specific hazard results using the equations above, these multiple models are to be combined, with associated weights (See Figure B-1 and Table B-1), to derive overall log-mean and log-standard deviation values for each spectral frequency. For each spectral frequency and input rock amplitude, the total log-mean, μ_T ($\mu_{\ln AF|x}$ in the equation above), and log-standard deviation, σ_T ($\sigma_{\ln AF|x}$ in the equation above), are calculated as:

$$\sum_i w_i \mu_i$$

$$\sigma_T = \sqrt{\sum_i w_i \left((\mu_i - \mu_T)^2 + \sigma_i^2\right)}$$

Where i indicates individual site amplification models, w_i is the weight on each model, and μ_i and σ_i are the log-mean and log-standard deviation of each site amplification model, *i.*

7.0 References

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Tables

Table B-1.
Site Independent Relative Weights and Epistemic Uncertainty

Parameter	Relative Weight	σ_{μ}
Mean Base-Case Profile	0.40	0.35
Lower-Range	0.30	
Upper-Range	0.30	
Mean Base-Case Kappa	0.40	0.40
Lower-Range	0.30	
Upper-Range	0.30	
G/Gmax and Hysteretic Damping Curves		0.15*, 0.30**
Soil		
EPRI Cohesionless Soil	0.5	
Peninsular Range	0.5	
Firm Rock		
EPRI Rock	0.5	
Linear	0.5	

* Modulus variability at cyclic shear strain 3 x 10^{-2}%

** Shear-wave damping variability at cyclic shear strain 3 x 10^{-2}%

Table B-2.
Model to Estimate Density from Shear-Wave Velocity

Shear-Wave Velocity (m/s)	Density (g/cm^3)
<500	1.84
500 to 700	1.92
700 to 1,500	2.10
1,500 to 2,500	2.20
>2,500	2.52

Table B-3.
Maximum Number of Models to Characterize Epistemic Uncertainty

Parameter	Maximum	Soil	Firm Rock	Soil/Firm Rock
	N	N	N	N
Profile	3	3	3	3
Curves	2	2	2	2
Kappa	3	1	3	3
Magnitude	2	1	1	1
1,2-Corner	2	2	1	2
Total Models	**72**	**12**	**18**	**36**

Table B-4
Suite of Hard Rock Peak Accelerations, Source Epicentral Distances, and Depths (M 6.5; 1-corner source model)

Expected Peak Acceleration (%g)	Distance (km)	Depth (km)
1	230.00	8.0
5	74.00	8.0
10	45.00	8.0
20	26.65	8.0
30	18.61	8.0
40	13.83	8.0
50	10.45	8.0
75	4.59	8.0
100	0.0	7.0
125	0.0	5.6
150	0.0	4.7

Additional parameters used in the point-source model are:

$\Delta\sigma$ (1-corner) = 110 bars
ρ = 2.71 cgs
β = 3.52 km/s
R_C = 60 km, crossover hypocentral distance to $R^{-0.5}$ geometrical attenuation
T = $1/f_c$ + 0.05 R, RVT duration, R = hypocentral distance (km)
Q_o = 670
η = 0.33
kappa(s) = 0.006

Table B-5
Generic CEUS Hard Rock Crustal Model

Thickness (km)	**V_s (km/sec)**	**ρ (cgs)**
1	2.83	2.52
11	3.52	2.71
28	3.75	2.78
--	4.62	3.35

Table B-6
Suite of Hard Rock Peak Accelerations, Source Epicentral Distances, and Depths (M 6.5; 2-corner source model)

Expected Peak Acceleration (%g)	Distance (km)	Depth (km)
1	230.00	8.0
5	81.00	8.0
10	48.00	8.0
20	28.67	8.0
30	20.50	8.0
40	15.60	8.0
50	12.10	8.0
75	6.30	8.0
100	0.0	7.9
125	0.0	6.4
150	0.0	5.4

Table B-7
Geometrical spreading and attenuation models for the CEUS and WUS

Region	**Geometric Spreading**	**Anelastic Attenuation**
CEUS	1/R for R ≤ 60km 1/60 for 60km < R ≤ 130km $(1/60)(60/R)^{0.5}$ for R > 130 km	$Q(f) = 670f^{0.33}$
WUS	1/R	$Q(f) = 180f^{0.45}$

Figures

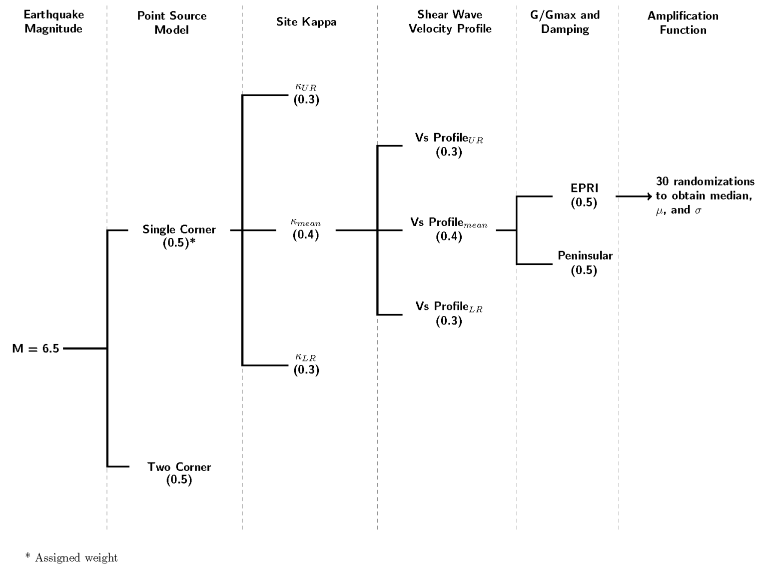


Figure B-1
Logic tree illustrating the process for capturing uncertainty in the development of site-specific amplification functions. This illustration is for a site with limited at-site geophysical and geotechnical data available. UR and LR indicate Upper-Range and Lower-Range about the mean Base-Case model.

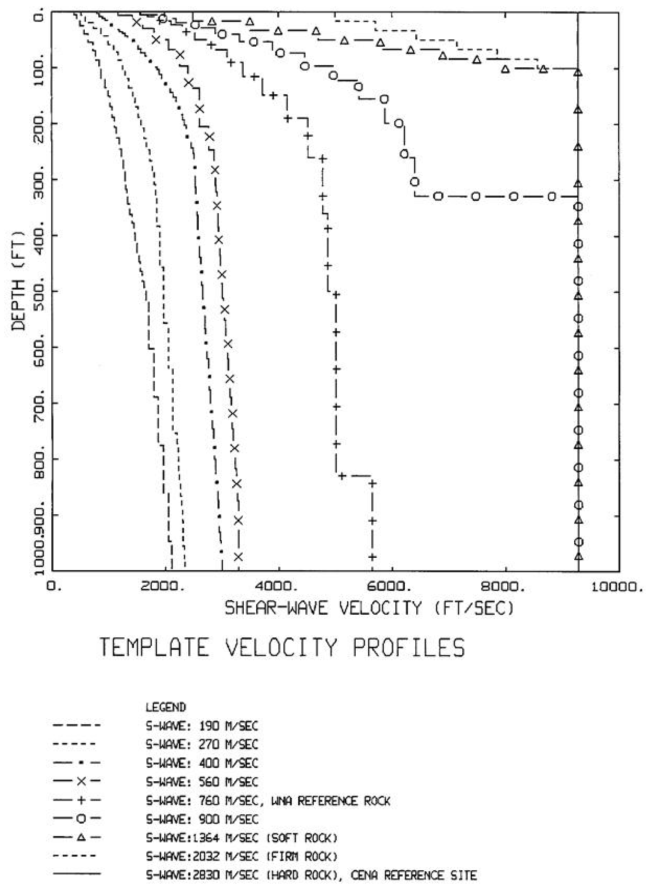


Figure B-2
Template Shear Wave Velocity Profiles for Soils, Soft Rock, and Firm Rock. Rock Profiles Include Shallow Weathered Zone. Indicated velocities are for V_{S30}.

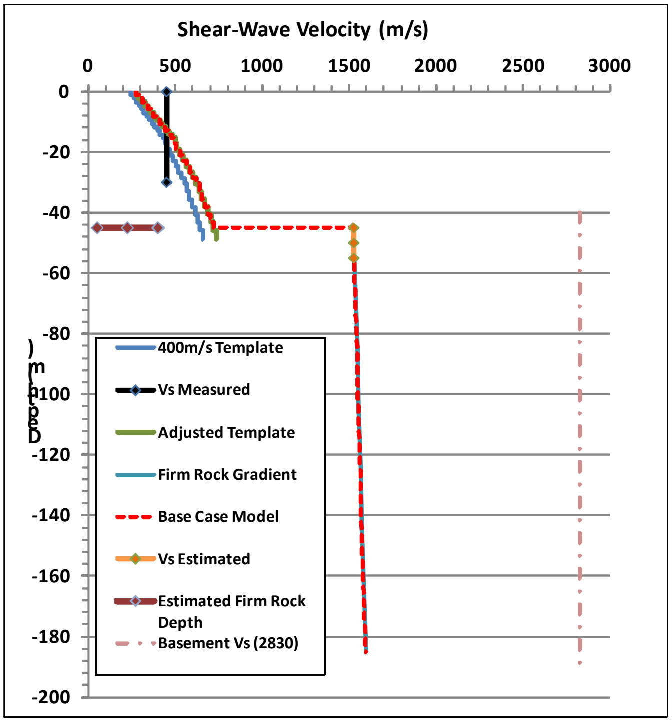


Figure B-3
Illustration of how available information is used to develop a mean base-case profile. The information available is represented by the measured near-surface soil V_{S30} (solid black line), estimated depth to firm rock (solid brown line) and estimated firm rock V_S (solid orange line). Proposed mean base-case V_S profile is indicated by dashed red line.

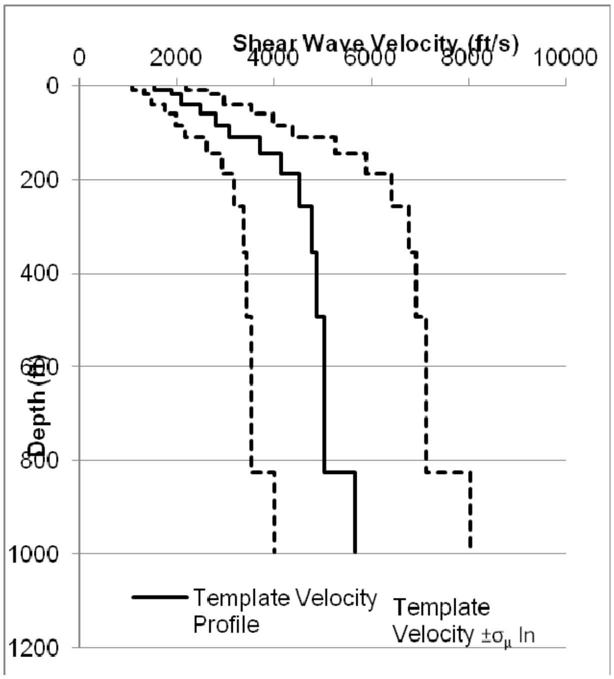


Figure B-4
This figure illustrates the range of velocity implied by the method used to account for epistemic uncertainty in site specific shear wave velocity profiling where sparse or limited information is available. Displayed is the 760 m/s WNA reference rock template velocity (solid curve) with dashed curves representing ± σμ ln = 0.35.

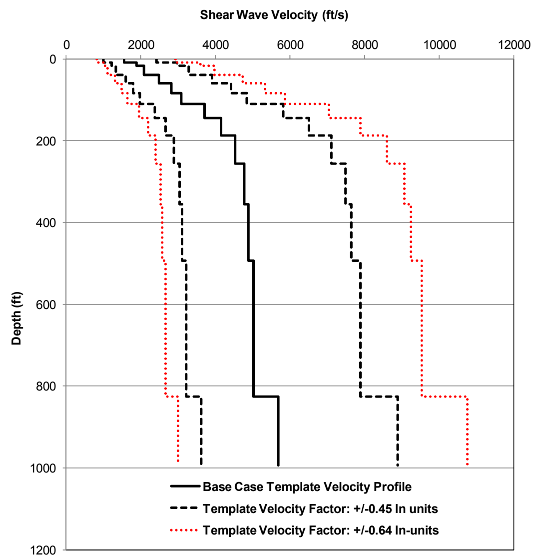


Figure B-5
This figure displays the method used to account for epistemic uncertainty in site specific shear wave velocity profiling where very limited or no information is available. Displayed is the 760m/s reference template velocity (solid black curve) with dashed curves representing 10th and 90th-percentile values (±0.45 natural log units which corresponds to a σμ ln = 0.35). Dotted red curves are for ±0.64 natural log units which corresponds to a σμ ln = 0.5.

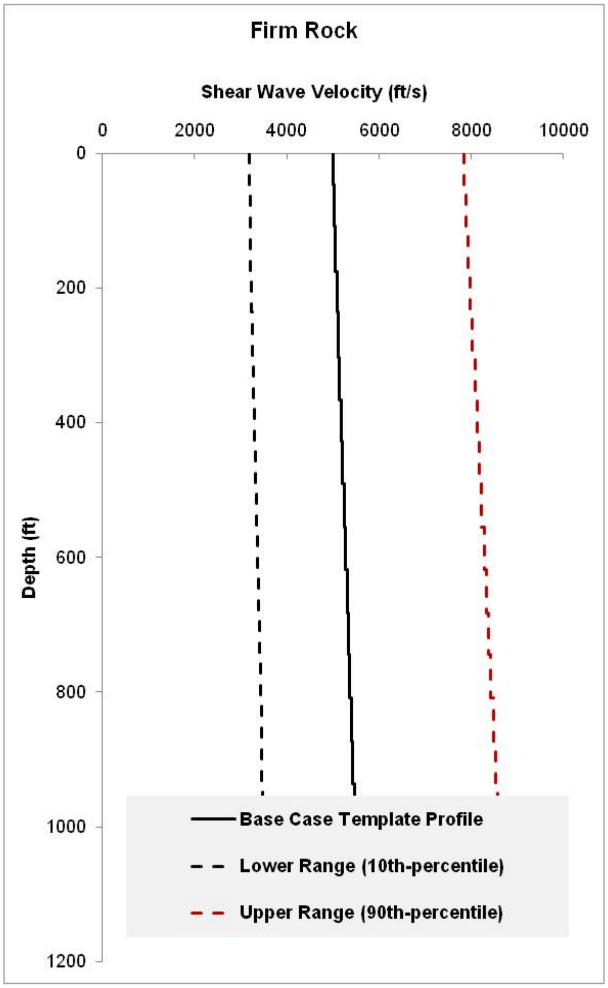


Figure B-6
Illustration of Upper Range and Lower Range Base-Case profiles (10th and 90th percentiles) developed to represent the epistemic uncertainty in the Mean Base-Case for firm rock conditions. A mean surface velocity of 5000ft/s (1525m/s) was assumed for the Base Case and the empirical gradient of Fukishima et al. (1995) [19] was applied. A σμ ln = 0.35 was used.

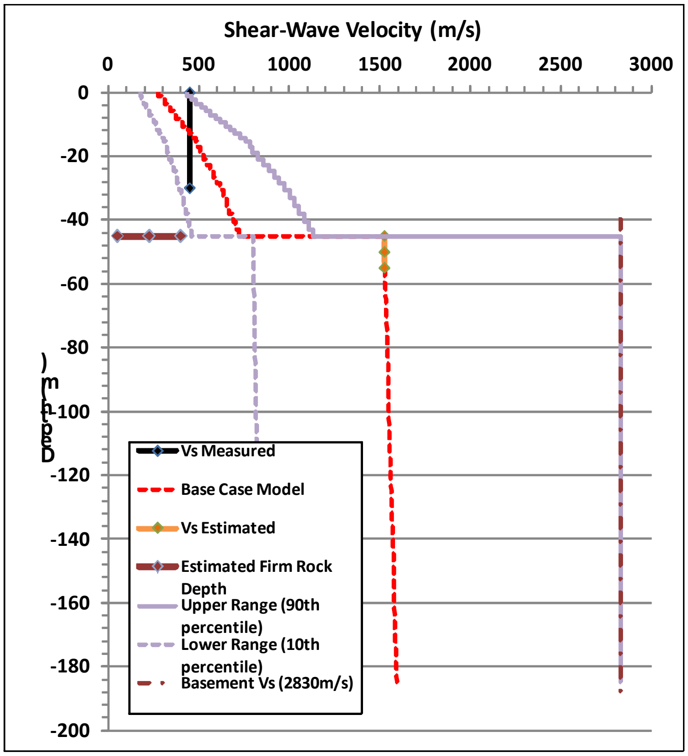
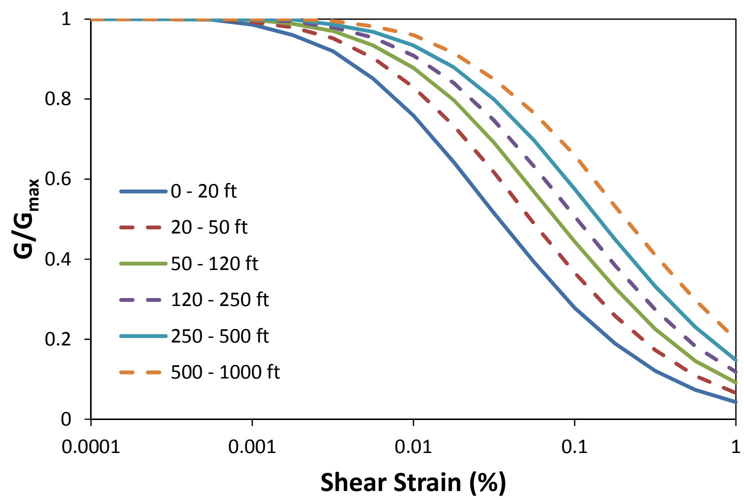


Figure B-7
This figure illustrates the development of Upper Range and Lower Range profiles to accommodate epistemic uncertainties for the hypothetical example shown in Figure B-3. A σμ ln = 0.35 has been used to develop the 10th and 90th-percentile curves in the upper portion of the profile where sparse Vs measurements were available. A σμ ln = 0.50 was applied to the lower portion of the profile where the Vs of the Base Case was inferred from geological information. The 90th-percentile curve was capped at a value equal to the 2830m/s Vs value assumed for the hard rock basement.



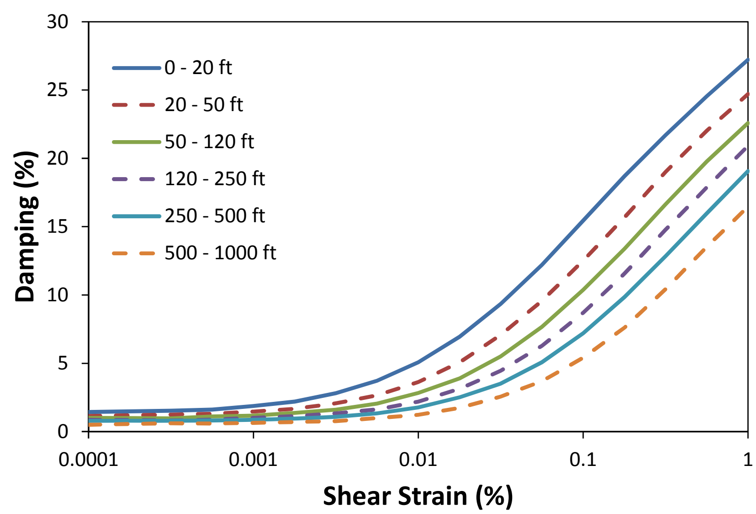


Figure B-8
Generic G/GMAX and hysteretic damping curves for cohesionless soil [12]. Note that damping will be limited to 15% for this application.

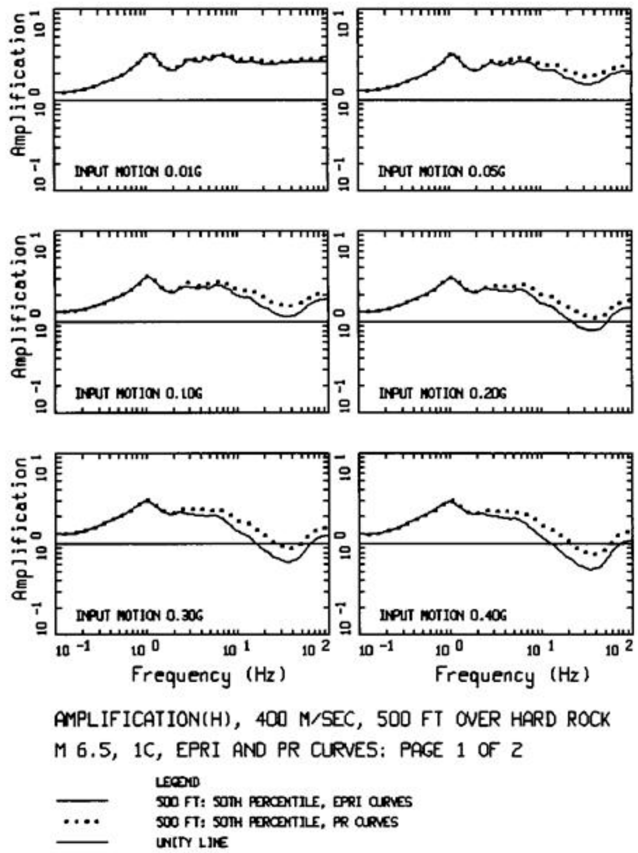


Figure B-8a. Comparison of median amplification functions (5%-damped PSa) derived using the EPRI (1993) [12] (see Figure B-7) and Peninsular Range [28]) G/GMAX and hysteretic damping curves. The results are for the 400 m/sec VS30 template profile and a single-corner source model for reference rock loading levels of 0.01 to 1.50g.

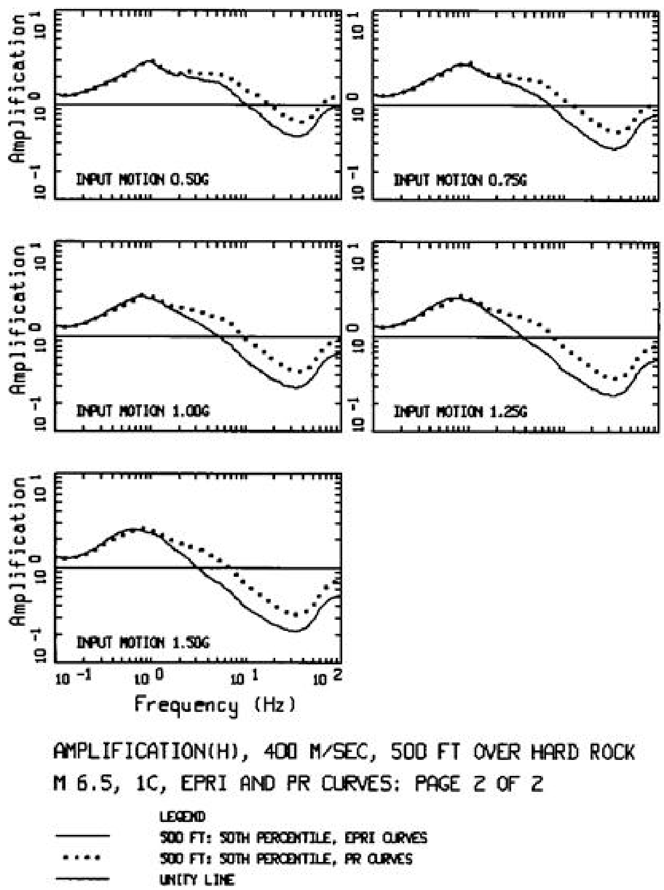


Figure B-8b. Comparison of median amplification functions (5%-damped PSa) derived using the EPRI (1993) [12] (see Figure B-7) and Peninsular Range [28] G/GMAX and hysteretic damping curves. The results are for the 400 m/sec VS30 template profile and a single-corner source model for reference rock loading levels of 0.01 to 1.50g.

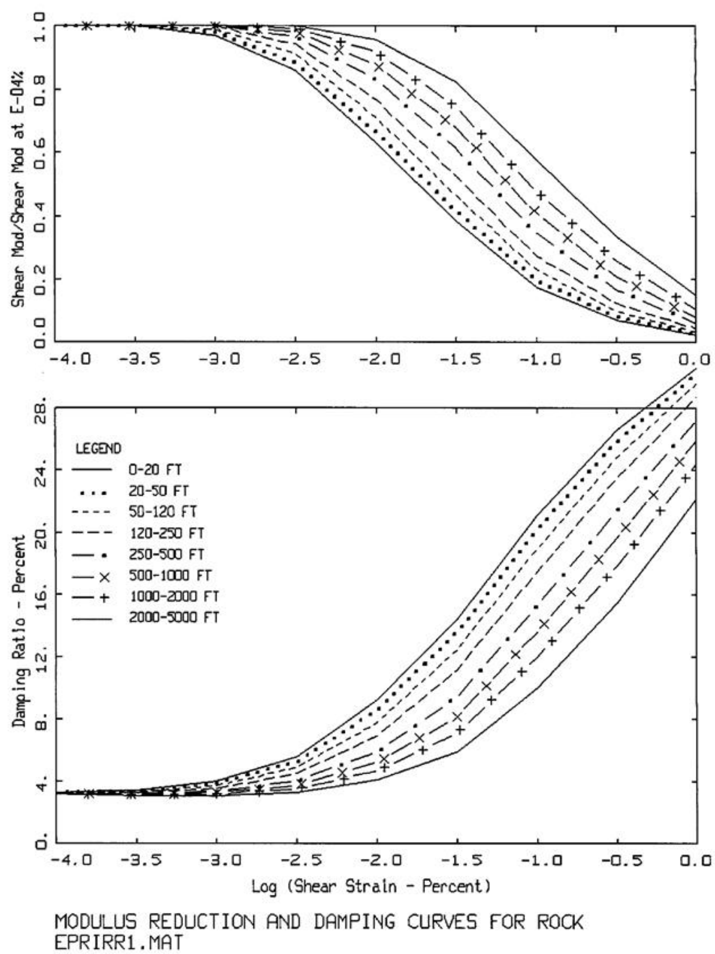


Figure B-9. Generic G/GMAX and hysteretic damping curves developed for firm rock in the EPRI (1993) study [12] (from Dr. Robert Pyke). Note that damping is limited to 15% in this application.

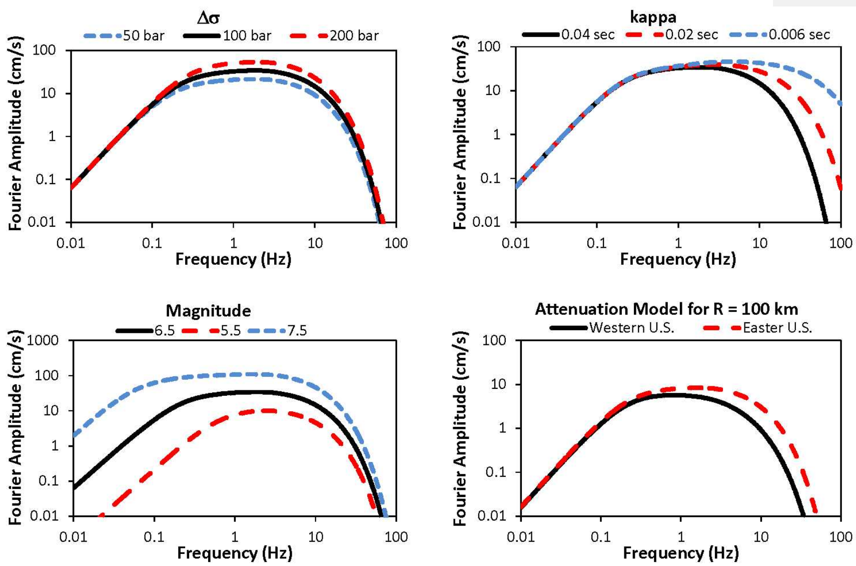


Figure B-10. Illustration of effect of various factors in the simple seismological model on Fourier spectral shape.

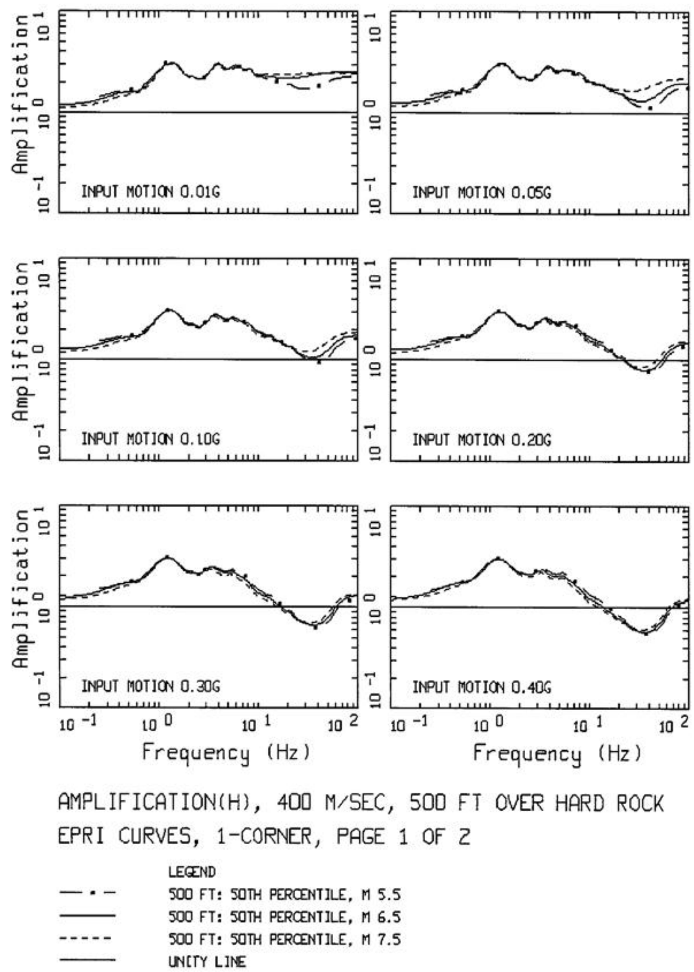


Figure B-11a. Comparison of amplification functions (5%-damped PSa) computed for magnitudes of M 5.5, 6.5, and 7.5, using the single-corner source model and the 400 m/sec VS30 stiff-soil template profile (Figure B-2) with the EPRI (1993) [12] G/GMAX and hysteretic damping curves (Figure B-7). The input reference rock loading levels varied from 0.01 to 1.50 g (Table B-4).

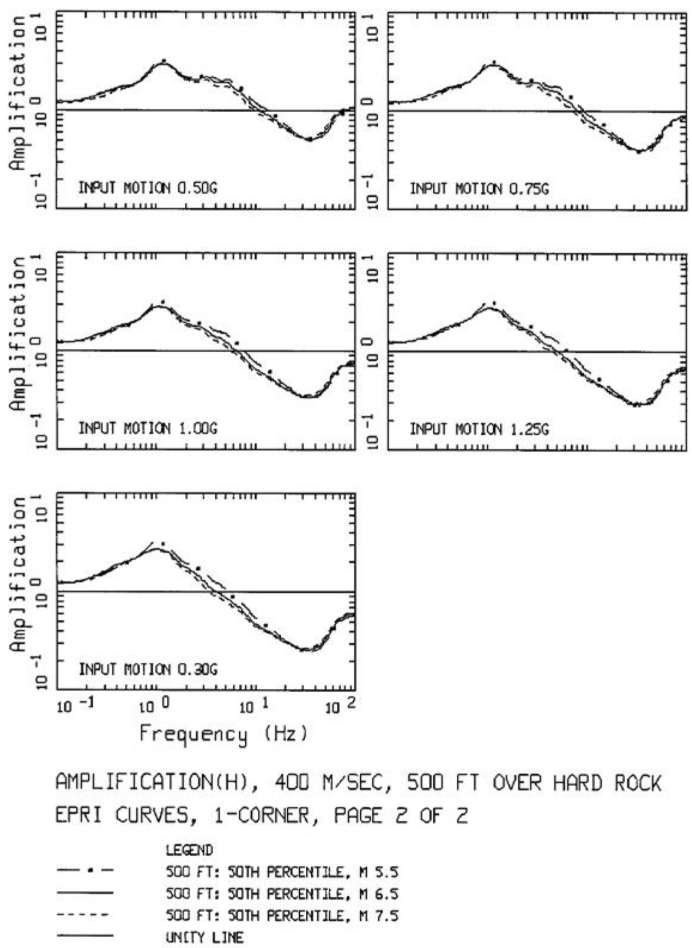


Figure B-11b. Comparison of amplification functions (5%-damped PSa) computed for magnitudes of M 5.5, 6.5, and 7.5, using the single-corner source model and the 400 m/sec VS30 stiff-soil template profile (Figure B-2) with the EPRI (1993) [12] G/GMAX and hysteretic damping curves (Figure B-7). The input reference rock loading levels varied from 0.01 to 1.50 g (Table B-4).

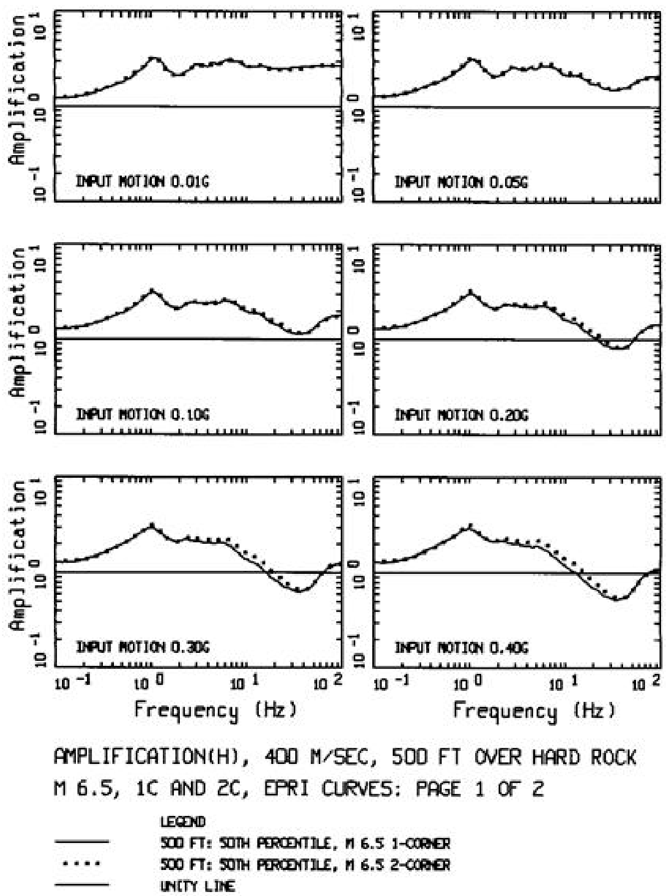


Figure B-12a. Comparison of amplification functions (5% damped PSa) computed using the Single- and Double-Corner source models (Tables B-4 and B-6) for the 400 m/sec VS30 stiff-soil template profile (Figure B-2) with the EPRI (1993) [12] G/GMAX and hysteretic damping curves (Figure B-7). The input reference rock loading levels varied from 0.01 to 1.50 g (Tables B-4 and B-6).

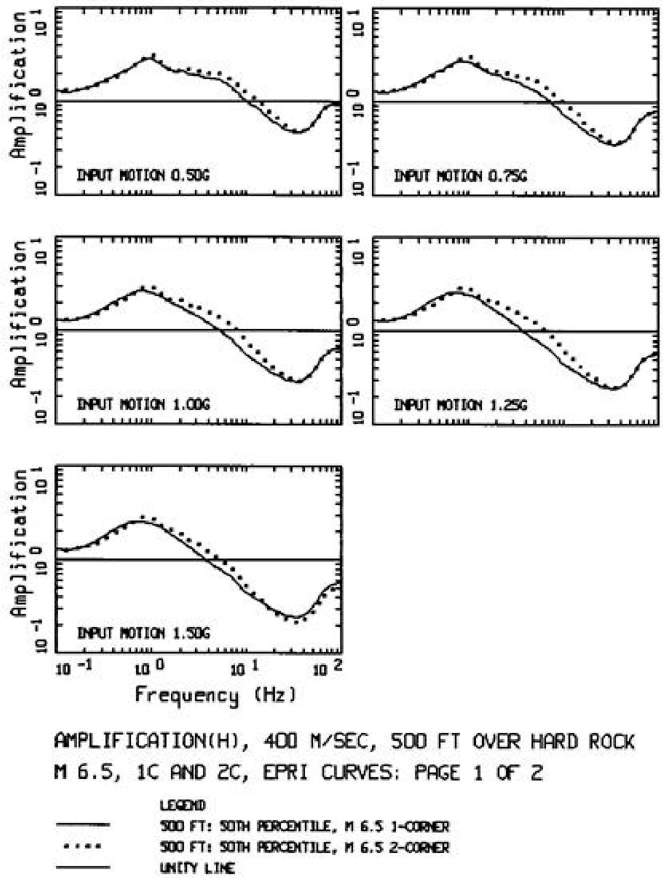


Figure B-12b. Comparison of amplification functions (5% damped PSa) computed using the Single- and Double-Corner source models (Tables B-4 and B-6) for the 400 m/sec VS30 stiff-soil template profile (Figure B-2) with the EPRI (1993) [12] G/GMAX and hysteretic damping curves (Figure B-7). The input reference rock loading levels varied from 0.01 to 1.50 g (Tables B-4 and B-6).

Appendix C: Sensitivity Studies to Develop Criteria for Analyzing Rock-Founded Structures as Fixed-Base Models

Two examples of models of existing structures at a nuclear power plant were analyzed in a study to compare the ISRS developed from a fixed-base dynamic analysis with the ISRS obtained from SSI analyses with various shear wave velocities. The first example analyzed a containment structure with a fundamental frequency of about 5 Hz, and the second example was for a Main Steam Valve House (MSVH) structure which has a fundamental frequency of about 10 Hz in one horizontal direction.

C1.0 Containment Structure

The containment structure, which has a horizontal fundamental mode of about 5 Hz, was analyzed [45] using a fixed-base model and, subsequently, with SSI analyses using three sets of shear wave velocities (Vs) for this site: lower bound (about 3,400 ft/sec), best estimate (about 5,200 ft/sec) and an upper bound (about 7,900 ft/sec). Figures C-1, C-2 and C-3 show the results of the analyses at the operating deck of the structure (about 75 ft above the basemat) in the east-west, north-south, and vertical directions, respectively. All three figures compare the results from SSI analyses using the lower and best estimate shear wave velocity values with the results of the fixed-base analysis. The results from the SSI analysis using the upper bound Vs were very close to the fixed-base results and therefore are not shown in this Appendix but are included in [45].

From these figures, it can be seen that for the east-west and north-south directions, the lower bound Vs case resulted in a slight shift of frequency of the spectral peak as compared to the fixed-base model (about 1 Hz lower with SSI) because of the rotational effects. For the best estimate case (Vs of 5200 ft/sec), the frequency shift was smaller in the north-south direction, and there was no shift in the east-west direction. In the vertical direction, the two SSI cases gave identical ISRS, and there was also no frequency shift compared to the fixed-base case. In all three directions, the spectral peaks determined from the fixed-base analysis were higher than those from the SSI analyses, and the shapes of the ISRS in the entire frequency range remained about the same. Therefore, based on the analysis of this structure, a fixed-base analysis can be used for sites with rock Vs > 3,500 ft/sec only if one can accept a slight potential frequency shift of the spectral peak. If not, a fixed-base analysis is only recommended for rock Vs > 5,000 ft/sec. It is noted that peak-broadening or peak-shifting of the ISRS in fragility analyses can alleviate the effect of a slight frequency shift between the SSI and fixed-base analyses.

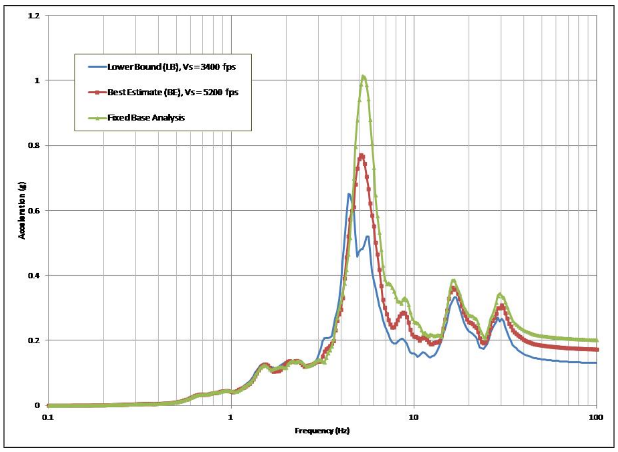


Figure C-1
Reactor Containment Internal Structure, Operating Floor
East-West Direction, 5% Damping

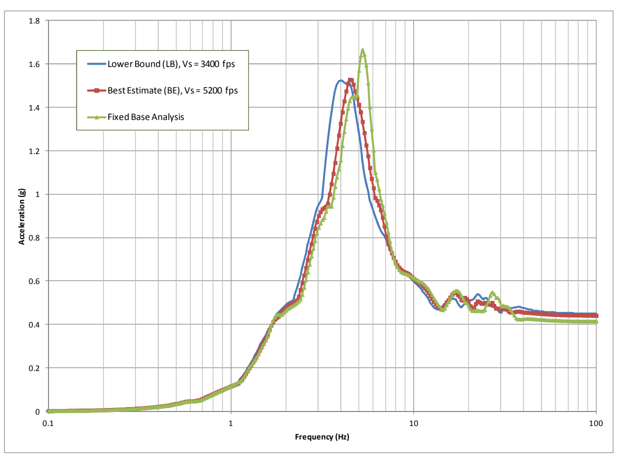


Figure C-2
Reactor Containment Internal Structure, Operating Floor
North-South Direction, 5% Damping

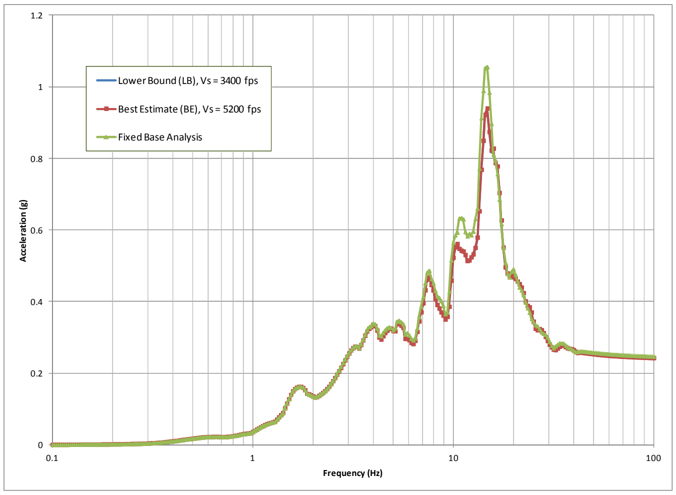


Figure C-3
Reactor Containment Internal Structure, Operating Floor
Vertical Direction, 5% Damping

C2.0 Main Steam Valve House Structure

The MSVH structure, which has a fundamental mode of about 10 Hz in one horizontal direction, was analyzed [55] using a fixed-base model and, subsequently, with SSI analyses using two sets of shear wave velocities (Vs): 3,500 ft/sec and 5,200 ft/sec (best estimate Vs for this site). Three sets of input motions were used in the analysis; however, ISRS for only two sets of representative motions are presented in this Appendix (results of the third set of input motions are about the same as the first set and are documented in [55]). The two set of results presented here are: (a) ISRS using time histories derived from the design basis earthquake (DBE) ground spectrum shape of this plant, which is similar to that of RG 1.60, and (b) ISRS using time histories derived from GMRS from a recent PSHA for a hard rock site that contains high frequency content.
Figures C-4, C-5 and C-6 show the ISRS for case (a) and Figures C-7, C-8 and C-9 show the ISRS for case (b), in the east-west, north-south, and vertical directions, respectively. These ISRS are compared at a node (Elevation 306 ft) that is about 60 ft above the basemat of the structure. These figures compare the results from the fixed-base case and the SSI cases with (Vs) = 3,500 ft/sec case and (Vs) = 5,200 ft/sec.

From these figures, it can be seen that the comparison of fixed-base ISRS with the (Vs) = 5,200 ft/sec case is reasonably good for all directions, whereas the lower bound Vs =3500 ft/sec case results in a slight shift of frequency and the spectral peaks are also slightly different in comparison to the fixed-base model. The shapes of the ISRS in the entire frequency range remain about the same from all these analyses. Therefore, based on the analysis of this structure, a fixed-base analysis is appropriate for rock sites with Vs > about 5,000 ft/sec. A fixed-base analysis can be used for sites with rock Vs > 3,500 ft/sec only if one can accept potential small frequency and amplitude shifts of the spectral peaks. Again, peak-broadening or peak-shifting of the ISRS in fragility analyses can alleviate the effect of a slight frequency shift between the SSI and fixed-base analyses.

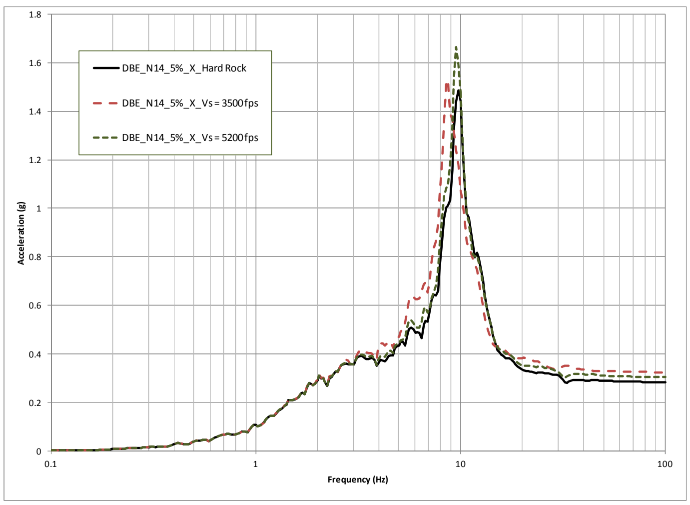


Figure C-4
Main Steam Valve House, Elevation 306 ft
East-West Direction, 5% Damping

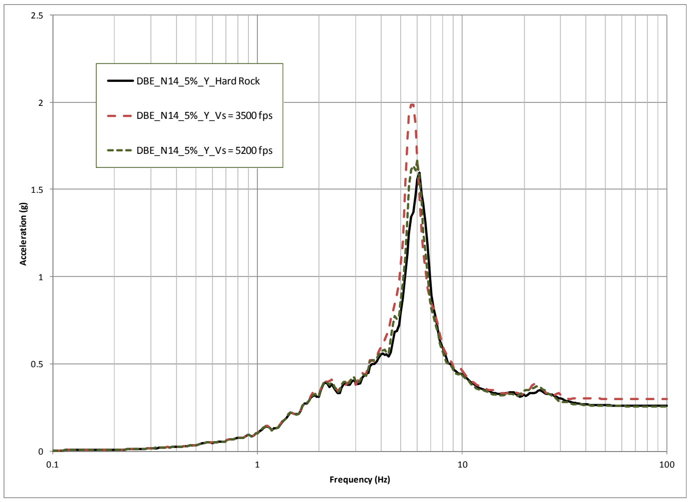


Figure C-5
Main Steam Valve House, Elevation 306 ft
North-South Direction, 5% damping

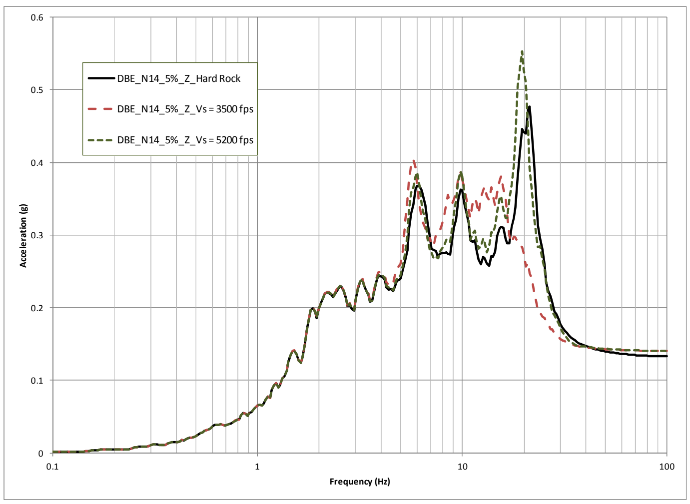


Figure C-6
Main Steam Valve House, Elevation 306 ft
Vertical Direction, 5% damping

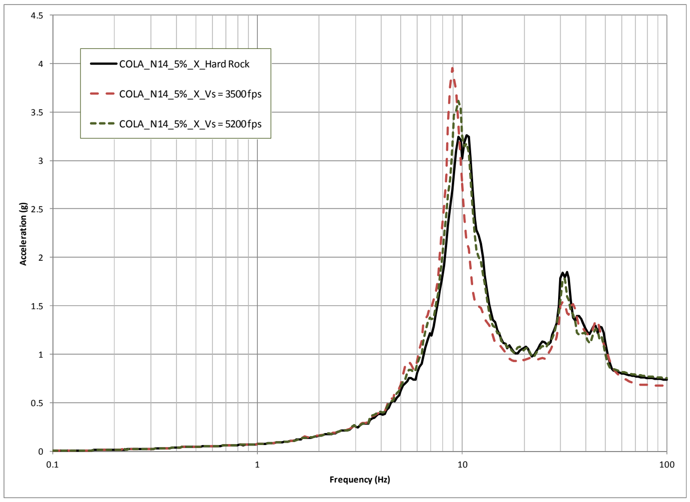


Figure C-7
Main Steam Valve House, Elevation 306 ft
East-West Direction, 5% damping

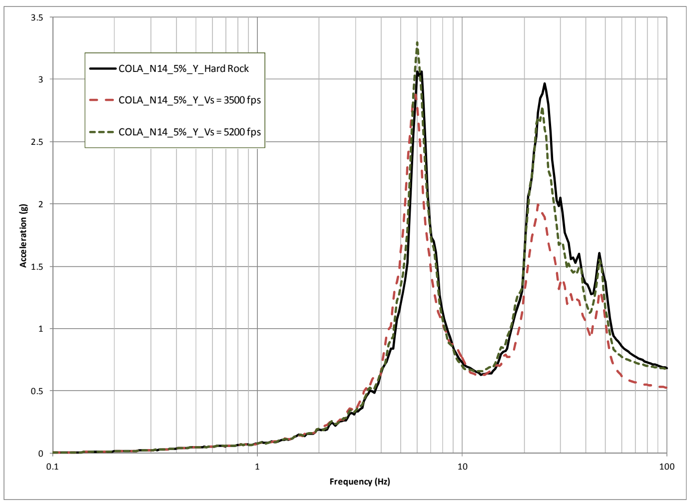


Figure C-8
Main Steam Valve House, Elevation 306 ft
North-South Direction, 5% damping

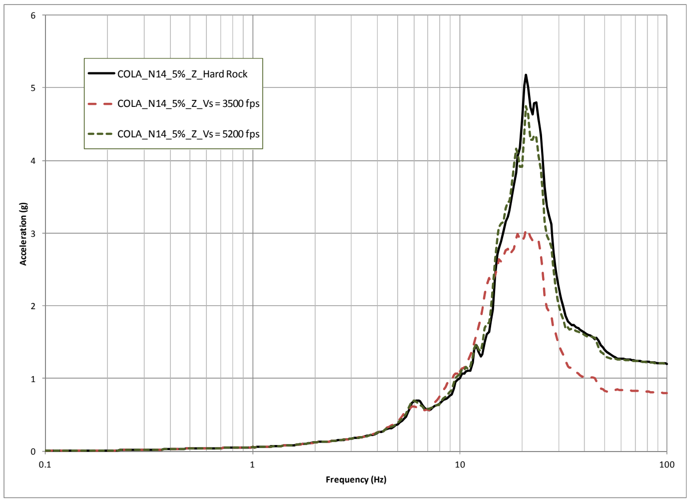


Figure C-9
Main Steam Valve House, Elevation 306 ft
Vertical Direction, 5% damping

Appendix D: Sensitivity of Computed Annual Probability of Failure P_F to Assumed Logarithmic Standard Deviation β Used in Hybrid Method with Capacities Defined by 1% Failure Probability Capacity $C_{1\%}$

D1.0 Introduction

In the Hybrid Method, the 1% failure probability capacity $C_{1\%}$ is computed by the CDFM Method. Then an estimate of the composite logarithmic standard deviation $\hat{a}_C$ and its subdivision into random variability $\hat{a}_R$ and uncertainty $\hat{a}_U$ are used to estimate the corresponding fragility curve. As noted in Ref. 1, typically $\hat{a}_C$ lies within the range of 0.3 to 0.6. In fact, if all of the sources of variability discussed in Ref. 2 are appropriately considered, it is not possible to obtain an estimated $\hat{a}_C$ less than approximately 0.3.

The Hybrid Method is based on the observation that the annual probability of unacceptable performance P_F for any Seismic Source Characterization is relatively insensitive to $\hat{a}_C$. Thus, annual probability (seismic risk) can be computed with adequate precision from the CDFM Capacity, C_{CDFM}, and an estimate of $\hat{a}_C$. It is concluded in Ref. 1 that the computed seismic risk at $\hat{a} = 0.3$ is approximately 1.5 times that at $\hat{a} = 0.4$, while at $\hat{a} = 0.6$ the computed seismic risk is approximately 60% of that at $\hat{a} = 0.4$.

In Section 3, it is demonstrated that the Ref. 1 conclusion concerning the lack of sensitivity of the computed seismic risk remains valid over the full practical range of hazard curve slopes. This demonstration uses the Simplified Seismic Risk Equation defined in Section 2.

D2.0 Simplified Seismic Risk Equation

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

$$H(a) = K_I a^{-K_H} \qquad \text{Equation 1}$$

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

$$K_H = \frac{1}{\log(A_R)} \qquad \text{Equation 2}$$

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

$$A_R = \frac{SA_{0.1H}}{SA_H} \qquad \text{Equation 3}$$

where SAH is the spectral acceleration at the mean exceedance frequency H and $SA_{0.1H}$ is the spectral acceleration at 0.1H.

So long as the fragility curve $P_{F(a)}$ is lognormally distributed and the hazard curve is defined by Equation 1, a rigorous closed-form solution exists for the seismic risk. This closed-form solution has been derived in a number of references including Appendix C of Ref. 1 and Appendix A of Ref. 3:

$$P_F = H\ F_{50\%}^{-K_H}\ e^{\alpha} \qquad \text{Equation 4}$$

in which

$$F_{50\%} = \frac{C_{50\%}}{C_H} \qquad \text{Equation 5}$$

and

$$\alpha = \tfrac{1}{2}(K_H \beta)^2 \qquad \text{Equation 6}$$

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 4 is referred to herein as the Simplified Seismic Risk Equation. The only approximations in its derivation are that the hazard curve is approximated by Equation 1 over the exceedance frequency range of interest and the fragility curve is lognormally distributed.

By defining C_H at the 1% failure probability capacity C1%, then H is replaced in Equation 4 by $H_{1\%}$ corresponding to the annual frequency of exceeding $C_{1\%}$. With these definitions for C_H and H, Equations 4 through 6 can be rewritten as:

$F_{50\%} = (C_{50\%}/C_{1\%}) = e^{2.326\hat{a}}$ Equation 7

$(P_F/H_{1\%}) = e^{\alpha_1}$ Equation 8

$\acute{a}1 = 0.5\ (KH\hat{a})^2 - 2.326\ (K_H\hat{a})$ Equation 9

From Equations 8 and 9, it can be observed that the ratio $(P_F/H_{1\%})$ is only a function of the product $(K_H\hat{a})$.

D3.0 Sensitivity of Failure Probability P_F to β

Table D-1 presents the ratio $(P_H/H_{1\%})$ computed over the full practical range of ground motion ratios A_R from 1.5 to 4.5 and â values ranging from 0.3 to 0.6 using the Simplified Seismic Risk Equation.

As shown in Table 4 of Reference 1, the failure probability P_F values computed by the Simplified Seismic Risk Equation tend to be about 10% to 20% conservatively biased when compared to P_F computed by numerical convolution of the hazard curve and fragility curve. This bias is due to the slight downward curvature of the hazard curve when plotted on a log-log plot as is shown in Figure D-1. Even with this slight conservative bias, the Simplified Seismic Risk Equation can be used to compare P_F values computed for different â values at any specified A_R ratio

AR values ranging from 2.0 to 4.5 are typical for CEUS sites over the 10^{-4} to 10^{-6} annual frequency of exceedance range. Within this A_R range, the $(P_F/H_{1\%})$ ratios shown in Table D-1 for â=0.3 range from 1.32 to 1.44 of the $(P_F/H_{1\%})$ ratios shown for â=0.4 with an average ratio of about 1.4. Many numerical convolutions of hazard and fragility curves have confirmed the conclusion that P_F computed using â=0.3 will be less than about 1.5 times the P_F computed using â=0.4 for the same $C_{1\%}$ capacity.

Over this same A_R range from 2.0 to 4.5, the $(P_F/H_{1\%})$ ratios shown in Table D-1 for â=0.6 range from 58% to 64% of the $(P_F/H_{1\%})$ ratios computed for â=0.4 with an average of about 60%. Again, many numerical convolutions of hazard and fragility curves have confirmed this value of about 60%.

For high seismic western sites, the A_R values will typically range between 1.5 and 2.25 over the 10^{-4} to 10^{-6} annual frequency range. For A_R values less than about 1.8, the lower tail of the lognormal fragility curve below the 1% failure probability capacity $C_{1\%}$ begins to significantly affect computed P_F. Many experienced seismic capacity engineers question the validity of extending the lower tail of the lognormal fragility curve substantially below the $C_{1\%}$ capacity. However, if one conservatively includes this lower tail on the lognormal fragility curve, the resulting $(P_F/H_{1\%})$ ratios computed by numerical convolution of hazard and fragility are similar to the results shown in Table D-1. As A_R is reduced below 1.75, the ratio of $(P_F/H_{1\%})$ for â=0.3 to $(P_H/H_{1\%})$ for â=0.4 will begin to rapidly reduce below 1.4. Conversely, as A_R is reduced below 2.0, the ratio of $(P_H/H_{1\%})$ for â=0.6 to $(P_H/H_{1\%})$ for â=0.4 will begin to rapidly increase. In fact at A_R = 1.5, the $(P_H/H_{1\%})$ ratio for â=0.6 will be 1.8 times that for â=0.4. This unexpected result is directly attributable to the tail of the lognormal fragility curve below the $C_{1\%}$ capacity. For sites with A_R less than 1.75:

1) one should assess whether it is appropriate to extend the lognormal fragility curve below the $C_{1\%}$ capacity, and

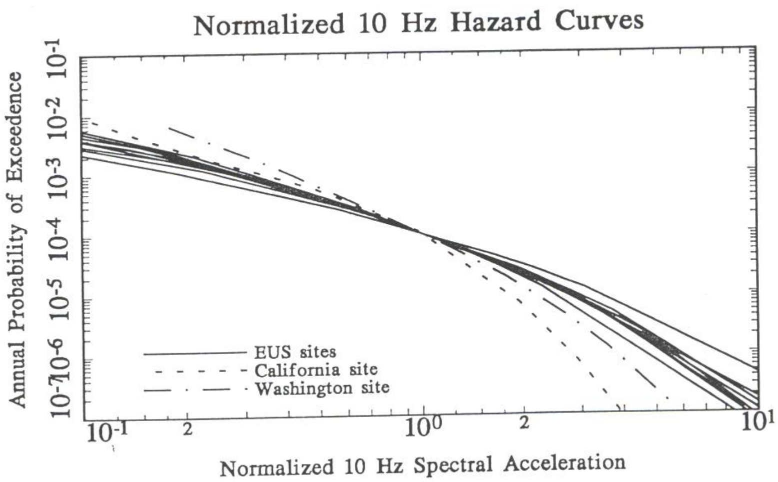
2) if the fragility curve is extended below $C_{1\%}$, one needs to carefully estimate â for those situation where â might exceed 0.5 (i.e., active components mounted at high elevation in structures).

D4.0 References

1. Kennedy, R.P. Overview of Methods for Seismic PRA and Margin Analysis Including Recent Innovations, Proceedings of the OECD-NEA Workshop on Seismic Risk, Tokyo, August 1999.
2. Methodology for Developing Seismic Fragilities, EPRI TR-103959, Electric Power Research Institute, June 1994.
3. Kennedy, R.P., Performance-Goal Based (Risk Informed) Approach for Establishing the SSE Site Specific Response Spectrum for Nuclear Power Plants. Nuclear Engineering and Design 241, pp. 648-656, 2011 (also Transactions SMiRT 19 Toronto, August, 2007).
4. Risk Engineering, Inc., REI (2001), Technical Basis for Revision of Regulatory Guidance on Design Ground Motions: Hazard- and Risk-consistent Ground Motion Spectra Guidelines, US Nuclear Regulatory Commission Rept NUREG/CR-6728, October, 2001.

Table D-1
Sensitivity of (PF/H1%) to Estimated β over Range of AR Values

A_R	K_H	$(P_F/H_{1\%})$			
		β=0.3	β=0.4	β=0.5	β=0.6
1.5	5.68	0.081	0.067	0.076	0.120
1.75	4.11	0.121	0.084	0.069	0.068
2.0	3.32	0.162	0.110	0.083	0.070
2.5	2.51	0.230	0.160	0.118	0.093
3.0	2.10	0.282	0.202	0.151	0.118
3.5	1.84	0.323	0.237	0.180	0.141
4.0	1.66	0.355	0.266	0.205	0.162
4.5	1.53	0.382	0.290	0.226	0.180



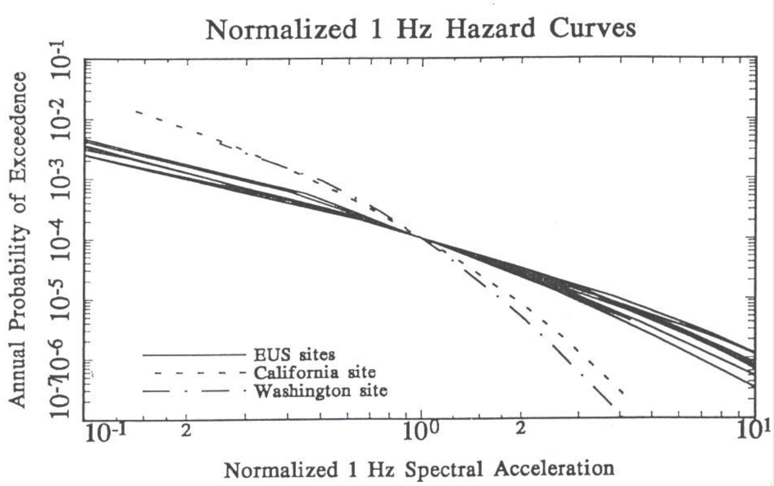


Figure D-2
SA (10 Hz) and SA (1 Hz) hazard curves for the eleven sites normalized by the acceleration value corresponding to mean 10-4 annual probability.

(From Figures. 7.7 and 7.8 of REI, 2001)