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).

ELECTRIC POWER RESEARCH INSTITUTE

3420 Hillview Avenue
Palo Alto, CA 94304-1338
USA

PO Box 10412
Palo Alto, CA 94303-0813
USA

800.313.3774
650.855.2121

askepri@epri.com

www.epri.com

XXXXXX

Draft Report, August 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.

ELECTRIC POWER RESEARCH INSTITUTE

3420 Hillview Avenue
Palo Alto, CA 94304-1338
USA

PO Box 10412
Palo Alto, CA 94303-0813
USA

800.313.3774
650.855.2121

askepri@epri.com

www.epri.com

XXXXXX

Draft Report, August 2012

Table of Contents

List of Figures

List of Tables

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 guidance could be useful in streamlining elements of any seismic risk evaluation.

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 1.5 years following the issuance of the 50.54(f) letter. 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. 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 definition of Generic Issue 199 [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 identified some higher seismic hazard estimates than previously assumed. 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/COL submittals, along with 2008 U.S. Geological Survey seismic hazard estimates. 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 on seismic core damage frequency (SCDF) estimates. The changes in SCDF estimate in the safety/risk assessment 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 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 include a seismic probabilistic risk assessment (SPRA) or an "NRC"-type of SMA that was described in NUREG-1407 [11] for IPEEEs.

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.

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).

Step 2. Provide the new seismic hazard curves, the GMRS, and the safe shutdown earthquake (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. Step 3 characterizes the 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. It 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 risk-based SMA approach that will be acceptable for this 2.1 application (*Reference to be added when ISG published*).

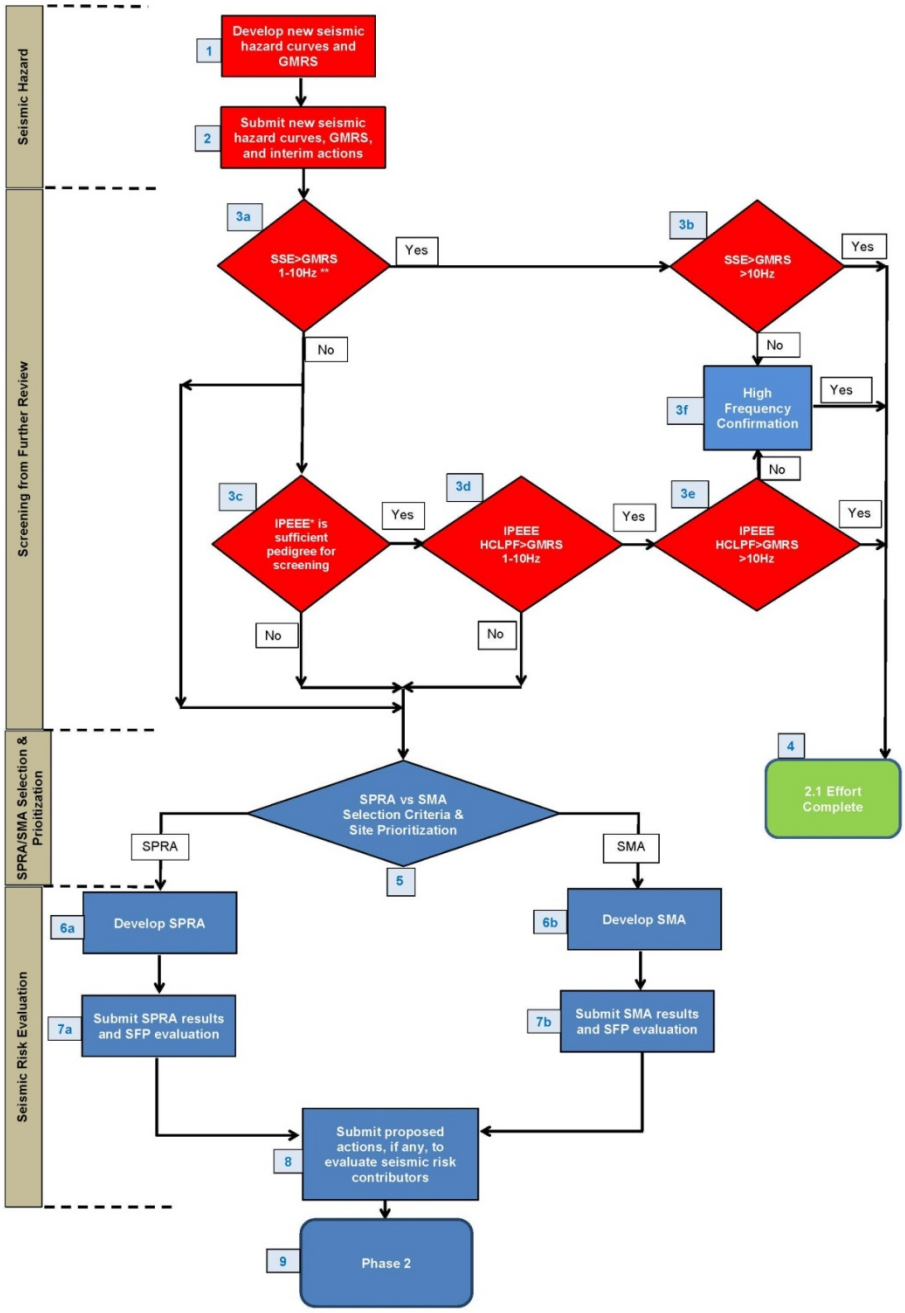
Step 7a. Document and submit the results of the SPRA to the NRC for review. The "Requested Information" Section in the main body of

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
** Narrow frequency exceedances addressed only for those SSCs affected by that frequency 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 design spectra can lead to informed evaluations on priorities to mitigate seismic risk, when compared to existing plants' seismic design spectra and seismic qualification spectra. Such comparisons should account for both relative and absolute differences between up-to-date seismic design spectra and existing plants' seismic ruggedness. The major part of the analysis is to calculate seismic hazard at existing plant sites, calculate uniform hazard response spectra (UHRS), and from those calculate ground motion response spectra (GMRS), using up-to-date models representing seismic sources, ground motion equations, and site amplification. 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.

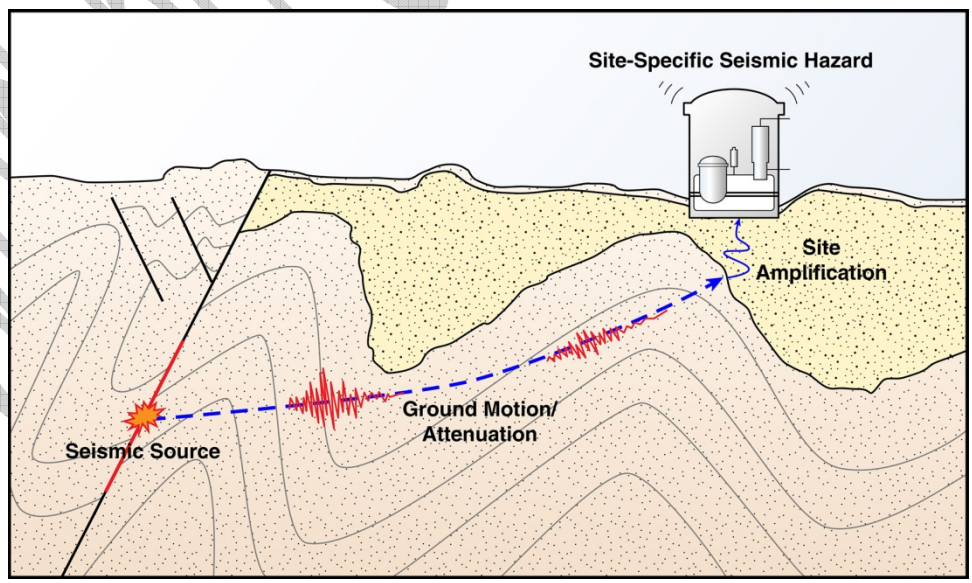


Figure 2-1
Steps to Obtain Site-Specific Seismic Hazard

Seismic sources – 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. This study was conducted as a 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 considered and given appropriate weights. The results were published in 2012 [14], were reviewed by the USNRC [15], and were anticipated to be an acceptable set of seismic sources to use for seismic hazard studies [23, p. 115]. 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. For a screening-level study of multiple plants for the purpose of setting priorities, the use of these seismic sources as published is appropriate.

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.

Ground motion estimates – For the CEUS, a general set of ground motion prediction equations (GMPEs) was published by EPRI in 2004 [16] , with aleatory uncertainties updated by an EPRI study in 2006 [17]. 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. (See Section 2.3 for studies relating to updates of these GMPEs).

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 solid, 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 must make site-specific decisions on which equations are appropriate for their sites because of the aforementioned differing conditions on faults and subduction zones that affect each site.

Site-specific ground motion amplification – Every site that does not consist of hard rock must have an evaluation made of the site amplification that will occur as a result of bedrock ground motions

traveling upward through the soil 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 that account for uncertainties in soils and layer properties, and this often results in significant uncertainties in site amplification.

2.2 Seismic Source Characterization

A 3-year project was conducted to characterize seismic sources in the CEUS. Designated the "CEUS Seismic Source Characterization" project [14], this study followed a SSHAC Level 3 procedure as defined in [13]. 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. Such evaluations would be appropriate in a licensing application, where focus could be made on site-specific applications. However, for a general comparison of hazards across a range of site locations, application of the CEUS Seismic Source Characterization seismic sources is appropriate.

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 evidence.

The second category of seismic sources were background sources, which are large regions within which earthquakes occur 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 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 seismic zone and the Charleston seismic zone 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 engineered facilities. For other RLME sources, 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.

2.3 Ground Motion Attenuation

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.

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 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.

The 2004-2006 EPRI GMPEs are a valid set of ground motion models that have been accepted by the NRC. It is acceptable to calculate seismic

hazard in the CEUS using these models (see Attachment 1 to Seismic Enclosure 1 of [1]). As indicated above, there are two sets of GMPEs, one for the mid-continent region and a second for the Gulf of Mexico region. 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 the EPRI 2012 ground motion update project is proceeding with updating the EPRI 2004-2006 GMPEs, those updated equations are also an acceptable method to calculate ground motions for seismic hazard calculations. In this case it is anticipated that, as in EPRI 2004-2006, multiple models with weights will be determined for the GMPEs and for the aleatory uncertainties. It is also anticipated that equations will be recommended for the two regions (mid-continent and Gulf of Mexico). If a more sophisticated technique is not developed for cases in which the travel path of seismic waves crosses from one region to the other, the method of weighting regional equations described above is acceptable.

2.4 Site Seismic Response

2.4.1 Horizons and SSE Control Point

This Section provides a rationale 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 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).

The seismic analysis and design of existing plants varied based on their vintage. NPPs used current state-of-the-art for seismic analysis at the

time the plants were analyzed, designed, constructed, and licensed. This means the SSE ground motion for input to these seismic analyses was treated differently depending on the seismic analysis methodologies currently accepted at the time. For example, most earlier plants simply applied the SSE at the foundation of simplified stick models of the seismic category I structures without considering embedment, depth of foundation, and the soil profile characteristics between plant grade down to the elevation of the bottom of the foundation. Later plants used more sophisticated soil structure interaction (SSI) methodologies and models that explicitly accounted for embedment where the SSE control point was at plant grade or top of the highest competent soil layer. Licensees using these more sophisticated methodologies were required to check that the de-convolved motion at the elevation of the bottom of foundation did not produce a free-field ground response spectrum at that elevation less than 60% of the input SSE ground response spectra at plant grade (control point). Otherwise, the input motion had to be increased accordingly, as described in Appendix A of a later version of NUREG 0800 [18]. The recommended approach in this document is outlined in Figure 1-1.

Conclusions

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. 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).

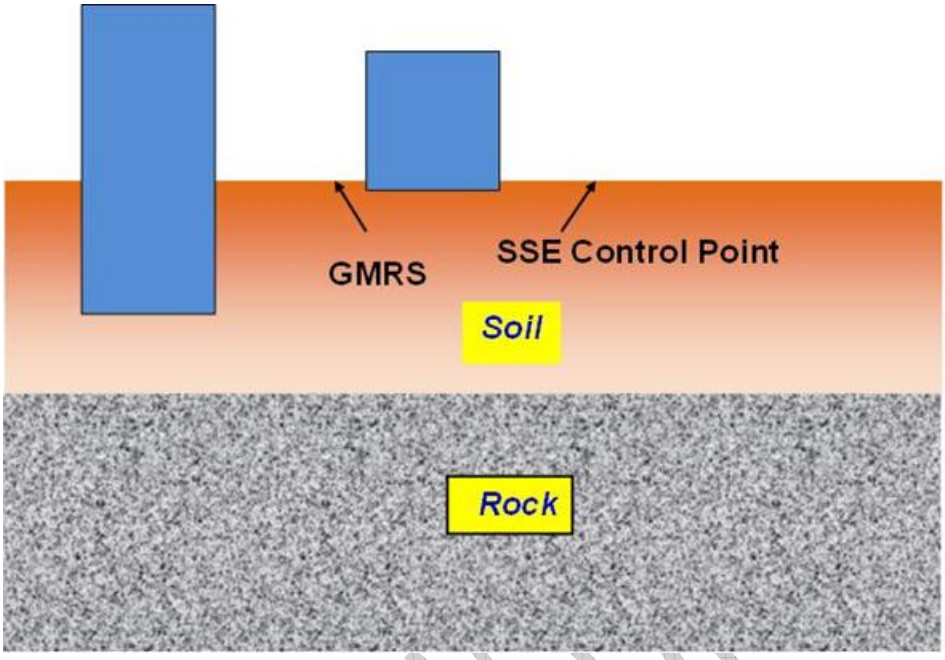


Figure 2-2
Soil Site Example

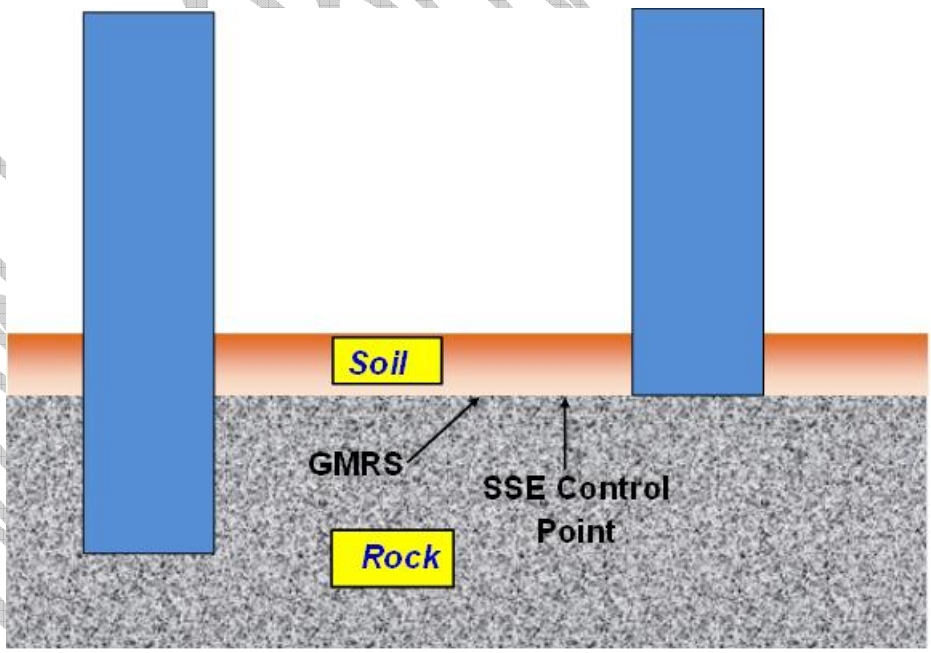


Figure 2-3
Rock Site Example

2.4.2 Addressing 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 (Vs). For soil sites where Vs was 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 Vs in the upper 30 m of soil, called Vs30, 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 Vs data, a Vs 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 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, indirect data on Vs, uncertainty in Vs will typically be characterized with a logarithmic standard deviation (σ_μ) of 0.35, except for sites with particularly sparse or indirect data. For those sites, σ_μ will be increased to 0.50. At sites where Vs has been measured directly and accurately, σ_μ will be taken to be zero. At sites with some direct measurements over limited depths, σ_μ will be taken as 0.35/2. Upper- and lower-range profiles will be developed using the assigned value of σ_μ.

Dynamic soil and soft-rock properties. Other soil and soft-rock properties such as dynamic moduli, hysteretic damping, and kappa (a measure of inherent 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.

2.5 Hazard Calculations and Documentation

2.5.1 PSHA and Hazard Calculations

The PSHA will proceed with the CEUS Seismic Source Characterization seismic sources (Section 2.2 above), with the GMPEs for the CEUS (Section 2.3 above), and with the site seismic response (quantified as described in Section 2.4). Several assumptions are appropriate regarding the PSHA calculations as follows:

Seismic sources should be included for the range of distances indicated in Section 2.2.

Either the EPRI 2004-2006 [16, 17] or updated EPRI GMPEs may be used, as indicated in Section 2.3. 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.

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 the August 23, 2011 Mineral, Virginia, earthquake are not required.

The CAV (Cumulative Absolute Velocity) model developed by EPRI [19] may be applied to account for the damageability of ground motions from small magnitude earthquakes. If applied, the lower-bound magnitude in calculations should be **M** 4.0, and the CAV model should not be applied for **M**>5.5 (see Attachment 1 to Seismic Enclosure 1 of Reference [1]).

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 overall log-mean and log-standard deviations of site amplification for each spectral frequency. For each spectral frequency and input rock motion (i.e., input rock amplitude) the total log-mean m_T and log-standard deviation σ_T of site amplification are calculated as:

$$\sigma_T = \sqrt{[\sum w_i((M_i - M_T)^2 + \sigma_i^2)]} \qquad \text{Equation 2-1}$$

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

The soil uncertainties should be incorporated into the seismic hazard calculations using a formulation similar to Eq. (6-5) in Reference 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 ground motion response spectrum (GMRS) for the site, using the method of Reference 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. 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 seven spectral frequencies at which GMPEs are defined. Results should include strain-compatible modulus and damping values for 10^{-4} and 10^{-5} input rock motions.

2.5.3 Seismic Hazard Data at Control Points and Base-Rock

Hazard data at control points 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 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 (100 Hz = Peak Ground Acceleration (PGA), 25 Hz, 10 Hz, 5 Hz, 2.5 Hz, 1 Hz, and 0.5 Hz). Hazard curves should be represented for annual exceedance frequencies from 10^{-3} to 10^{-6}. Hazard curves should be provided in graphical and tabular format.

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 GMRS calculation discussed/defined in Section 2 is being taken as a target for characterizing the amplitude of the new seismic hazard at each NPP site, as defined by the NRC [1]. The GMRS should be compared to the SSE as shown in diamonds 3a and 3b of Figure 1-1. If the SSE is exceeded, then utilities have the option to perform the screening of the GMRS to the response spectrum corresponding to the HCLPF documented from the seismic IPEEE program as shown in diamonds 3c through 3e in Figure 1-1. 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.

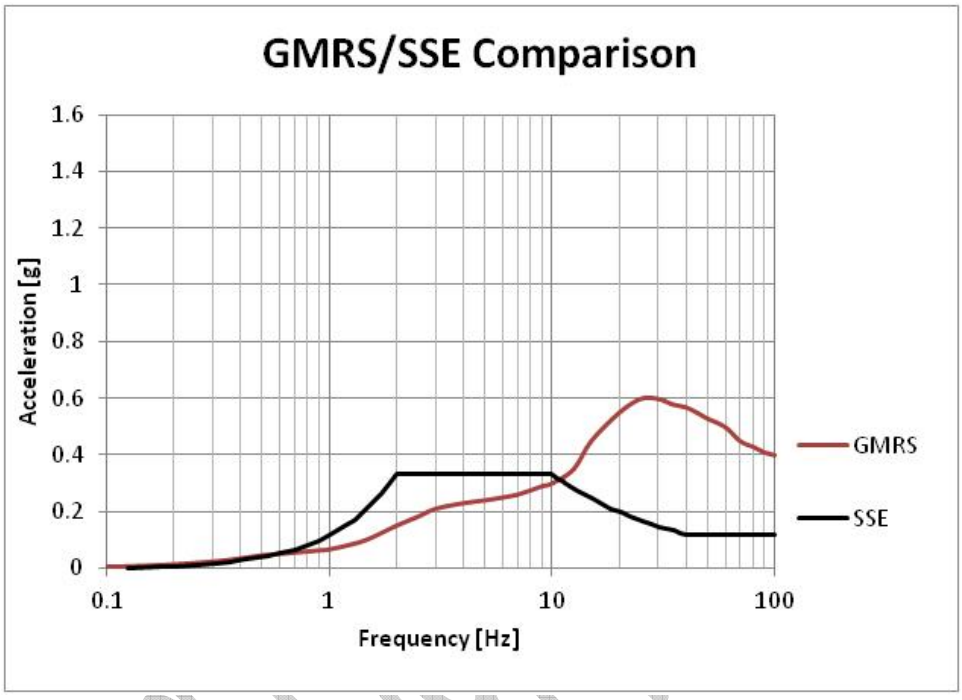


Figure 3-1
Example Comparison of GMRS to SSE

If the initial review of the SSE to GMRS (Diamond 3a in Figure 1-1) does not demonstrate that the SSE envelopes the GMRS in the 1 to 10 Hz region, then the licensees have the option of either conducting:

1) A screening evaluation for narrow band exceedances as described in Section 3.2.1, or
2) A screening evaluation using the IPEEE HCLPF described in Section 3.3, or
3) Bypass the screening evaluations and perform the seismic risk evaluation using either an SPRA or SMA approach described in Section 6 of this report.

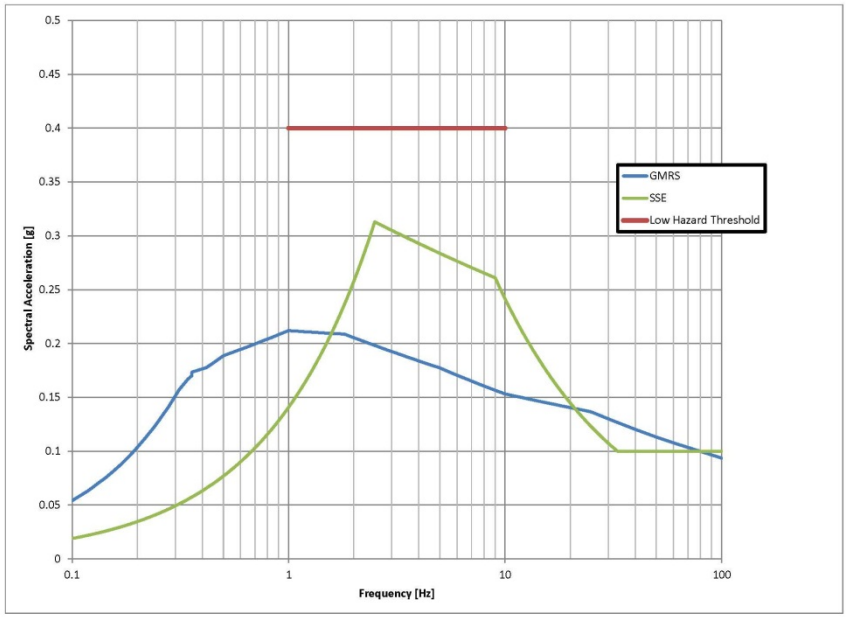


Figure 3-2
Example Comparison of GMRS to SSE and LHT

3.2.1 Narrow Band Exceedance Treatment

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 unlikelihood 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 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 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 reactor vessel

The industry is also currently researching the subject of soil liquefaction to assess whether it is also potentially a realistic failure mode at these acceleration and frequency ranges. For each of the identified potentially low-frequency susceptible SSCs, 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.

If the IPEEE HCLPF capacity evaluations are considered to be of sufficient pedigree for screening, the IPEEE HCLPF response spectral accelerations may be used for this HCLPF/GMRS comparison for screening potentially low-frequency susceptible SSCs. 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 (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.

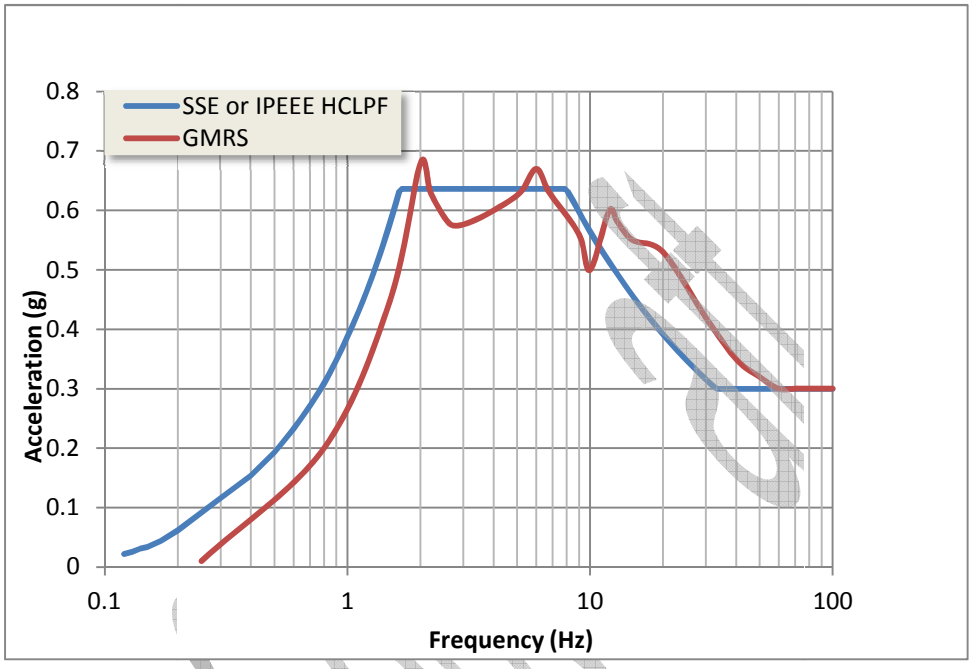


Figure 3-3
Screening – Example Narrow Exceedances at 2 Hz and 6 Hz

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 pedigree, 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 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

All plants choosing to employ the HCLPF screen will complete an IPEEE adequacy review. IPEEE submittals that used either SPRA or SMA analyses can be considered for screening but in either case the analysis must have certain attributes to be considered acceptable to the NRC staff.

Use of IPEEE Results for Screening

The necessary criteria for use of the IPEEE results for screening purposes are categorized 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 and screening submittal to the NRC. The staff will review each submittal and determine whether the provided evaluation demonstrates the adequacy of the IPEEE analysis and risk insights for the purpose of screening an individual plant from the need to perform a further risk evaluation. In addition to reviewing the documentation provided in the submittal to the NRC, the staff will also review the SERs and TERs that the staff wrote about the IPEEE submittals to assess the strengths and weaknesses of those IPEEE submittals. Each of the four categories is discussed below.

General Considerations

IPEEE reduced scope margin assessments cannot be used for screening. Focused scope margin submittals may be used after performing updates necessary to meet the full scope criteria. These updates include (1) addressing soil failure evaluations and (2) providing justification that the low ruggedness relay review performed for IPEEE is sufficient.

The spectrum to be compared to the GMRS for screening purposes should be based on the plant-level HCLPF capacity actually determined by the IPEEE and reported to NRC. If this is less than the review level earthquake (RLE) spectrum, then the RLE must be shifted appropriately to reflect the actual HCLPF capacity. In cases where modifications were required to achieve the HCLPF capacity that was in the plant's IPEEE report as submitted to the NRC, the licensee must 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) that was used for IPEEE seismic probabilistic risk analyses (SPRA) should be anchored at the plant-level HCLPF capacity.

Prerequisites

Responses to the following items must 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. Verify that commitments made under the IPEEE have been met. If not, address and close those commitments.
2. Verify whether all of the modifications and other changes credited for the plant capacity (HCLPF or SCDF) in the IPEEE analysis are in place such that the IPEEE analysis is still valid for those modifications or changes.
3. Verify that any identified deficiencies or weaknesses to NUREG-1407 in the plant specific NRC SER and related TER are properly dispositioned and provide justification that the IPEEE conclusions remain valid.
4. Verify 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 verified and documented in the submittal to the NRC, then the IPEEE results will not be considered by the staff to be adequate for screening purposes even if responses are provided to the adequacy criteria provided below.

IPEEE Adequacy Criteria

The following items, and the information that should be provided, reflect the major technical considerations that the staff will take into account in determining whether the IPEEE analysis, documentation, and peer review are considered adequate to support use of the IPEEE results for screening purposes. The staff will consider the description of each of the criteria below in its integrated totality rather than using a pass/fail approach.

With respect to each of the criteria listed below, the submittal should describe the key elements of (1) the methodology used and (2) whether the analysis was conducted in accordance with the guidance in NUREG-1407 and other applicable guidance.

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 in-structure response spectra (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 and completeness 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 was 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 a conclusion statement stating that the IPEEE results are adequate for screening and that the risk insights from the IPEEE are still valid under current plant configurations. The staff will review each submittal and determine whether the provided information demonstrates the adequacy of the IPEEE results for the purpose of screening out from the need to perform a further risk evaluation. 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 HCLPF, which was typically reported to the NRC as part of the IPEEE closure process. The IPEEE reported HCLPF are typically documented in the report sent to the NRC. 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 shape of the IHS is consistent with the RLE used for the SMA (typically the NUREG 0098 shape). For the case of those plants that conducted an SPRA as part of the IPEEE program, the shape of the IHS should correspond to the Uniform Hazard Spectrum (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 core damage frequency (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), has been met. However, for this example, the higher frequency criteria (Diamond 3e in Figure 1-1), has 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.

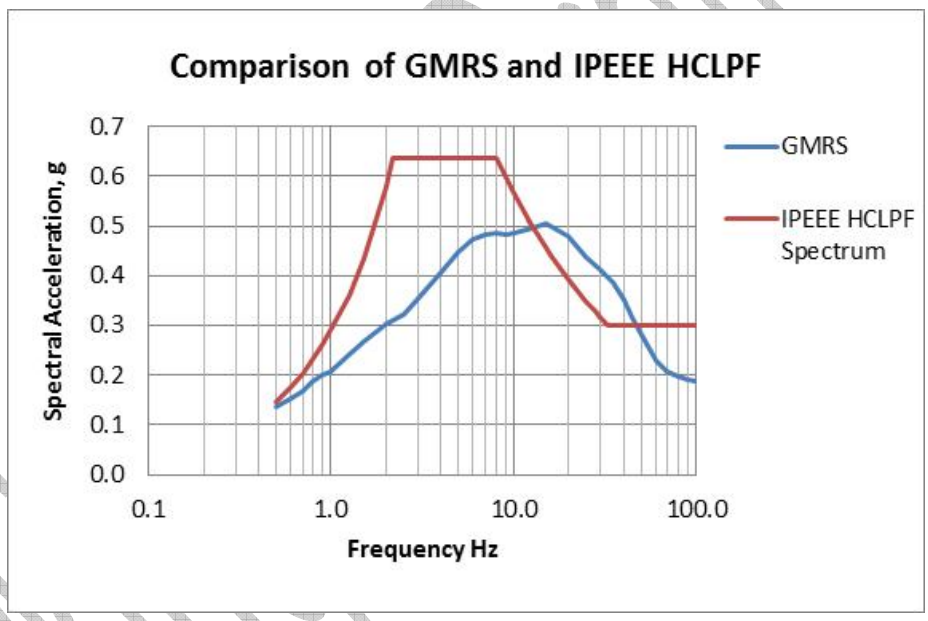


Figure 3-3
Example Comparison of GMRS to IHS

3.4 Treatment of High-Frequency Exceedances (Confirmation)

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 beyond the SSE 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 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.

EPRI has established a test program to develop fragility data 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 would not be necessary for those few 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 were 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 Report 1015109

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 in accordance with IEEE Standard 344 [25]. Since 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 components in new plants was 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 New Plant Licensing Reviews

During licensing reviews for the AP1000, Westinghouse and the NRC developed 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 includes potentially sensitive subcomponents, and the AP1000 list includes 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 (MOVs, AOVs). 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, although there is no indication that cell electrical degradation is associated with influences the high-frequency vs. low-frequency support motion sensitivity. 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.

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

3.4.1.3 Final Component Type List

The final list of component types to be evaluated in the Figure 1-1 high-frequency exceedances confirmation is provided in Table 3-3.

3.4.2 Phase 1 Testing

The high-frequency test program consists of two phases. The first phase pilot effort will focus 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.

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)

3.4.2.1 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 final list of Phase 1 components is provided in Table 3-3.

Table 3-4
Phase 1 Test Samples

▪ Electro-mechanical relay (600V industrial control relay) ▪ Electro-mechanical relay (pneumatic timing relay) ▪ Electro-mechanical contactor and Auxiliary contact ▪ Electro-mechanical relay (lockout relay) ▪ 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 are being 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] recommend a response spectra with peak 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 3-4 shows an example ground motion where high-frequency motions may be significant.

Since 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 will conservatively address a broader frequency range of 16 to 64 Hz.

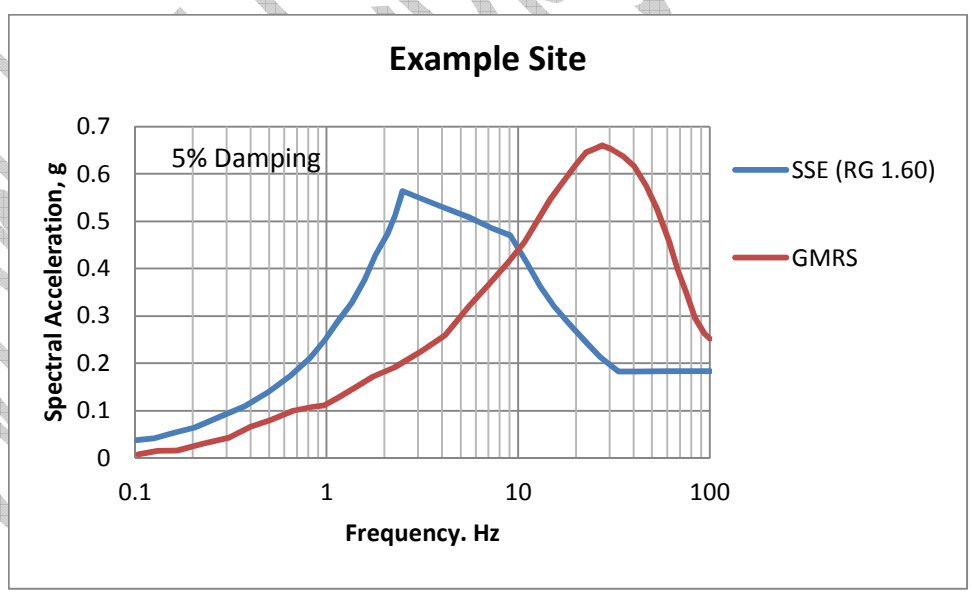


Figure 3-4
Example High Frequency Ground Motion Response Spectrum

Test Input Motions

Three types of test input motions will be investigated in Phase 1: sine sweeps, sine beats, and random multi-frequency (RMF) motions. In each case, the input motions will be increased in amplitude until the components fail the acceptance criteria (typically 2 ms contact chatter) or until the test machine limits are reached.

Sine sweep input motions: This test series will use single-axis sine sweep inputs with constant acceleration levels over the 16 to 64 Hz range. The components will be tested in multiple directions (e.g., front-to-back, side-to-side) in both the de-energized (non-operate) and energized (operate) states. The objective of this test series is to develop a plot of input acceleration amplitude vs. input motion frequency as a means of displaying the regions of high-frequency sensitivity for each component.

Sine beat input motions: This test series will use a wide-band multi-frequency tri-axial independent random input motion with an additional narrow-band single axis sine beat motion superimposed as depicted in Figure 3-5. The narrow-band motions will be applied one at a time at 1/6 octave frequencies between 17.8 and 44.9 Hz. The sine beat tests will be performed separately in the component front-to-back direction and the side-to-side direction.

RMF input motions: This test series will use wide-band multi-frequency tri-axial independent random motions covering three separate amplified frequency ranges as shown in Figure 3-6. The three frequency ranges are 16 to 32 Hz, 24 to 48 Hz, and 20 to 40 Hz. The general shape of the amplified spectral region will follow the normalized test shape from IEEE C37.98 [31] with the peak acceleration 2.5 times the ZPA, but with the frequency ranges adjusted as shown in Figure 3-6.

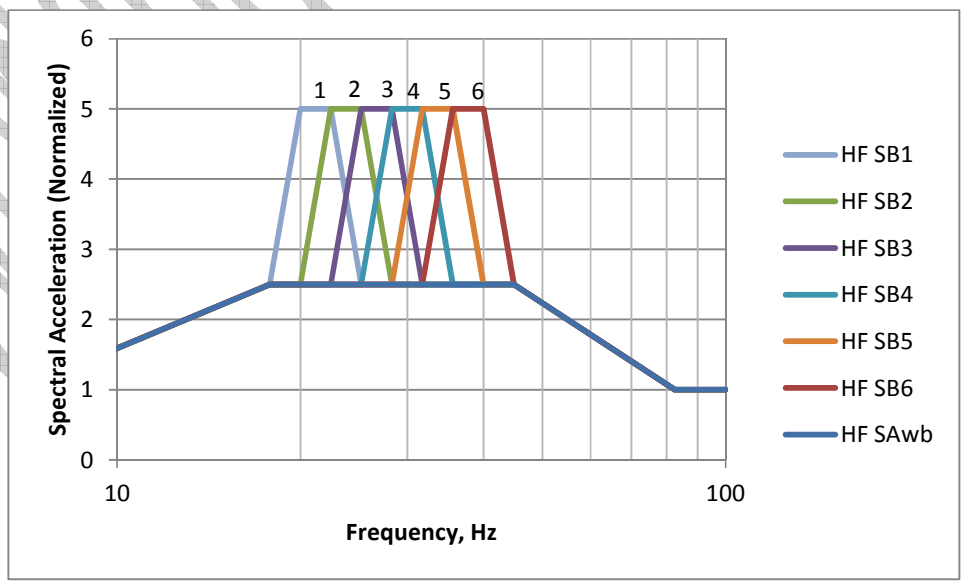


Figure 3-5
Sine Beat Test Input Motions

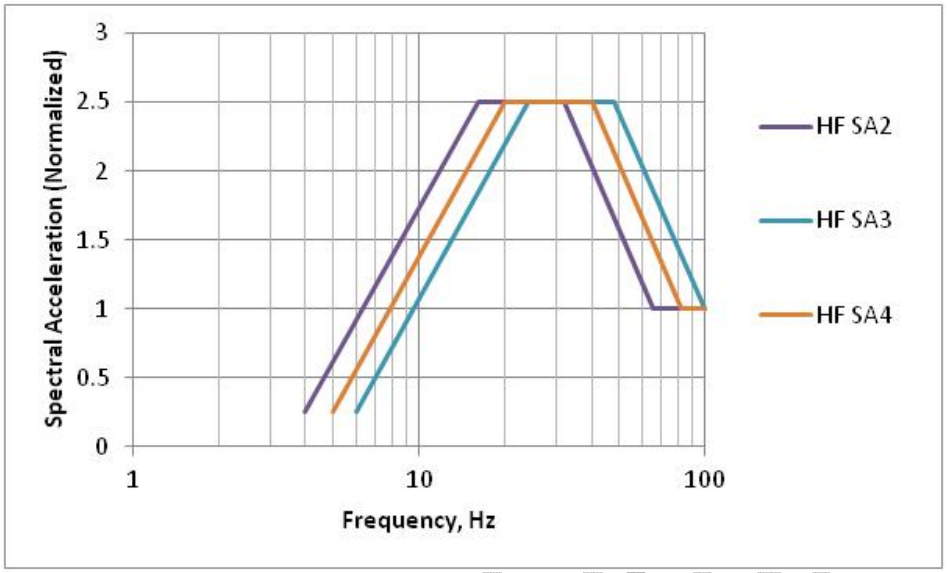


Figure 3-6
Random Multi-Frequency Test Input Motions

3.4.2.3 Conduct Sample Testing and Review Results

The Phase 1 test results will be evaluated to determine the optimal test protocol for Phase 2 testing, including the primary frequency range and the input motions necessary to develop the appropriate high-frequency sensitivity data. Phase 1 results will also be reviewed to consider criteria for comparison of fragility levels obtained using high-frequency wide-band and narrow-band motions.

3.4.3 Phase 2 Testing [This Section will be updated following Phase 1 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.

3.4.3.1 Phase 2 Test Protocol

Phase 1 testing will be used to develop the Phase 2 test protocol. The key parameters expected to be derived from the Phase 1 results are the high-frequency range of interest and the optimal test input method to develop the necessary component performance data.

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 Reference [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 types 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 CEUS SSC sources used
        ii. Ground Motion Prediction Equations used
     b. Deaggregation Results: Controlling Earthquakes
        i. Brief discussion of method

ii. Note that deaggregation is done for site-licensing studies when detailed site-specific amplification factors must be developed

c. Hard Rock Seismic Hazard Curves

i. Common fractiles and mean for 0.5, 1.0, 2.5, 5.0, 10.0, 25.0, and 100.0 Hz

3. Site Response Evaluation

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 RVT or time series approach

ii. Parameters used in RVT or time histories used

f. Amplification Functions

i. Amplification versus Input Amplitude including uncertainty bands for each of the seven spectral frequencies

g. Control Point Seismic Hazard Curves

i. Common fractiles and mean for 0.5, 1.0, 2.5, 5.0, 10.0, 25.0, and 100.0 Hz

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
- Screening Evaluation
    1. GMRS and SSE Comparison
        a. Discussion of results
    2. Evaluation of IPEEE Submittal
        a. TBD
    3. GMRS and IPEEE HCLPF Spectrum
        a. If applicable, discussion of results
    4. Screening for Risk Evaluation
        a. If applicable, discussion of results
- Interim Actions*
    1. Any interim actions taken 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 on 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).

The intent of the prioritization is to take into account:

- the amount of the seismic hazard exceedance,
- the overall seismic hazard level, and
- the available resources.

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 U.S. NRC 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 have been designed to withstand a conservatively selected large earthquake (the SSE) with adequate margins introduced at different stages of design, analysis, qualification, and construction. However, it is understood that larger earthquakes (although rare) could occur. The basic objective of the SPRA is to estimate the probability of occurrence of different sizes of earthquakes that may affect the plant, and to assess the plant response to such earthquakes. 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 aid in the overall risk-informed decision making process.

The key elements of a SPRA can be identified as:

- **Seismic Hazard Analysis:** used to express the seismic hazard in terms of the frequency of exceedance for selected ground motion

parameters during a specified time interval. The analysis involves identification of earthquake sources, evaluation of the regional earthquake history, and an estimate of the intensity of the earthquake-induced ground motion at the site (Figure 6-1).

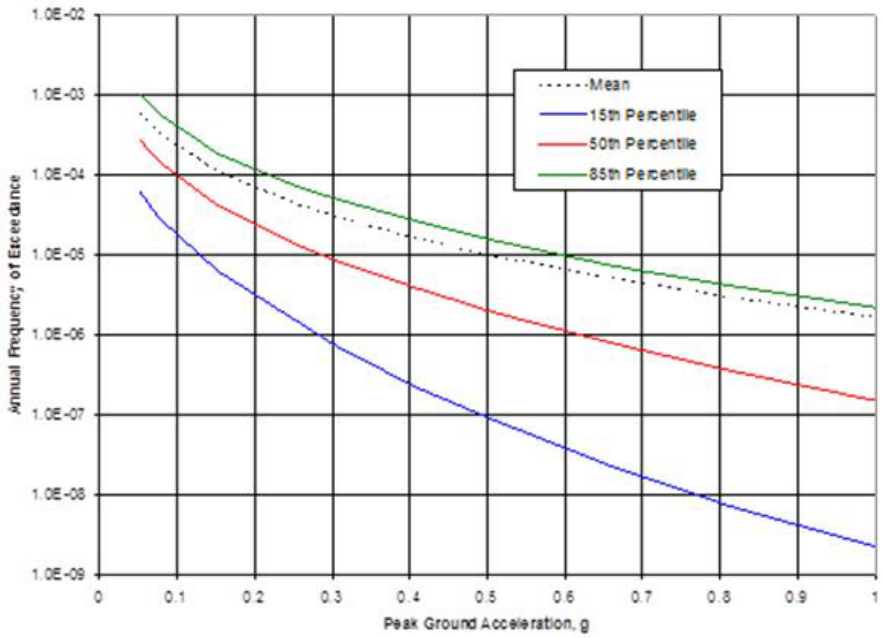


Figure 6-1
Example Seismic Hazard Curve

- **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).

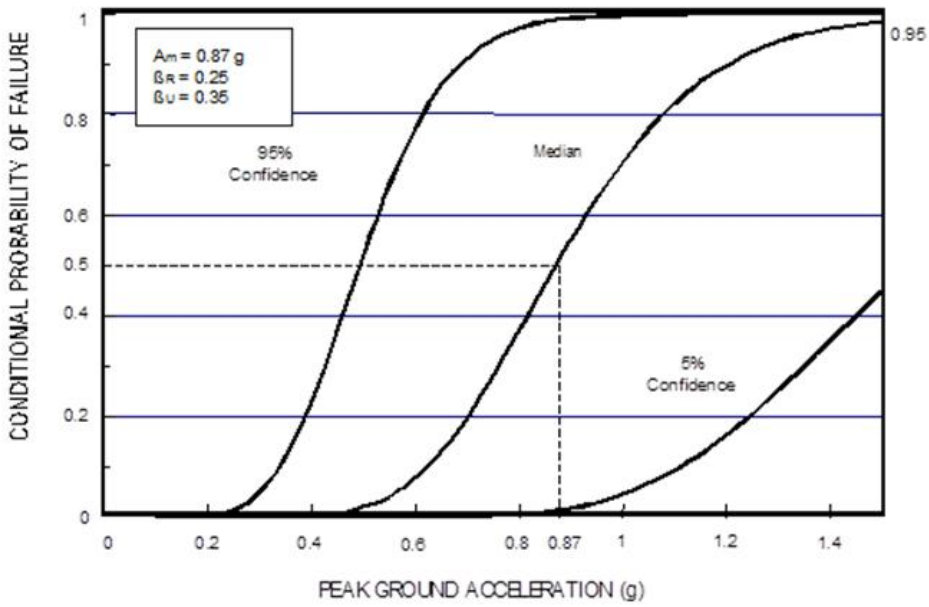


Figure 6-2
Example Seismic Fragility Curve

- **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.

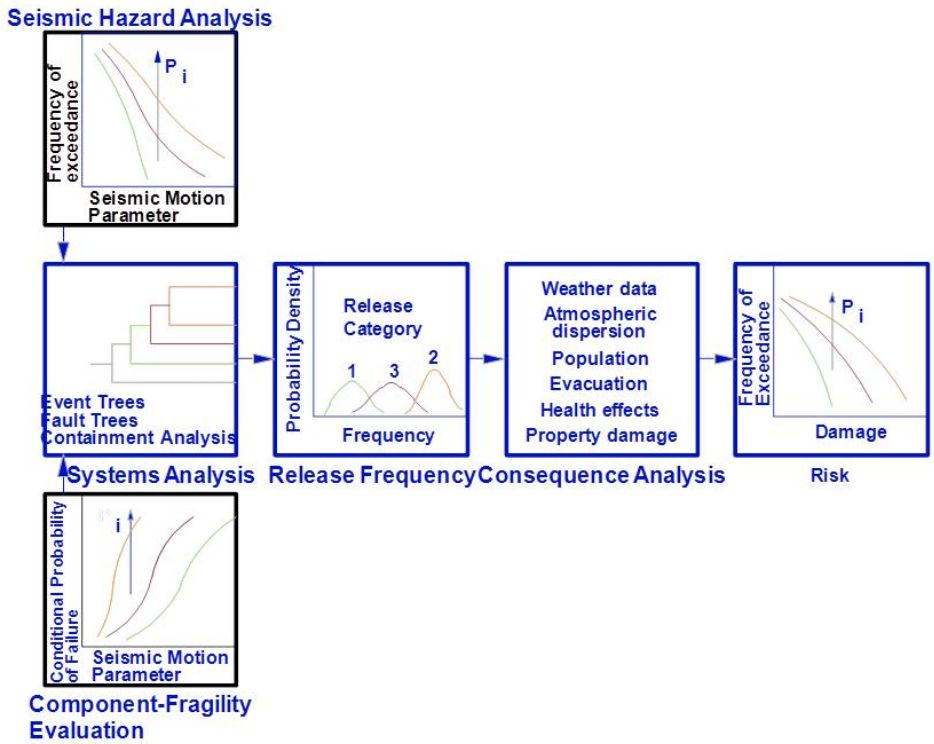


Figure 6-3
Overview of the SPRA Methodology

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 (October 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 (October 2001) [24]

6.1.2 Risk-Based SMA Methods and Procedures

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 reevaluated 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 (LHT) 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.

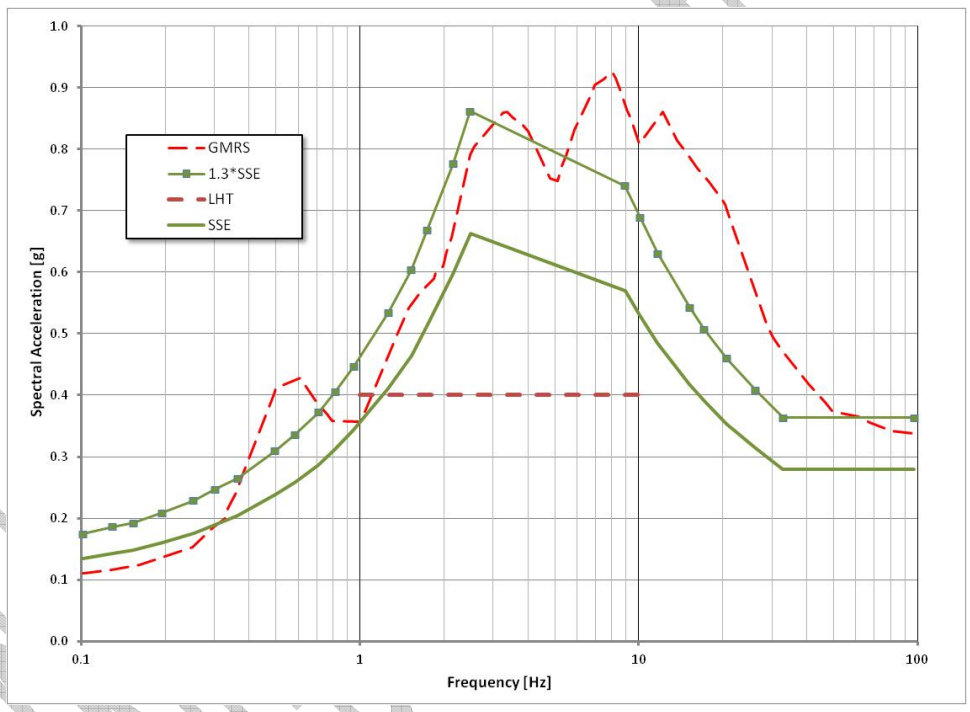


Figure 6-4
Example for Selection of SPRA vs. SMA

6.3 Key Elements of Seismic Structural and SSI Response

6.3.1 Control Point for FIRS

(Currently being developed)

6.3.2 Structure Modeling

Existing structural models (i.e., those used for design basis or in USI-A-46 / 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. 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. If necessary, the existing structural models can be enhanced using the structural modeling criteria discussed below.

The existing structural models that have been used, such as lumped-mass stick models (LMSM) in dynamic analyses to develop seismic responses for the design, licensing and qualification of plant SSCs, are reasonably complex for their original intended purpose and 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 within the simpler model is decreased as modal mass is shifted to other modes, resulting in lower spectral peaks for the significant modes of the structure.

Using the existing structural models will facilitate the timely completion of the SPRA/SMA effort within 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 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 review if one stick model sufficiently represents the structure. For example, two stick models could be 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.

6.3.3 Seismic Response Scaling

Scaling of ISRS is considered a technically sound approach and has been used in previous SPRAs and SMAs. Scaling will reduce the effort involved in performing detailed soil-structure interaction (SSI) analyses for the new hazard/UHS, facilitating the timely completion of the SPRA 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, 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 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.

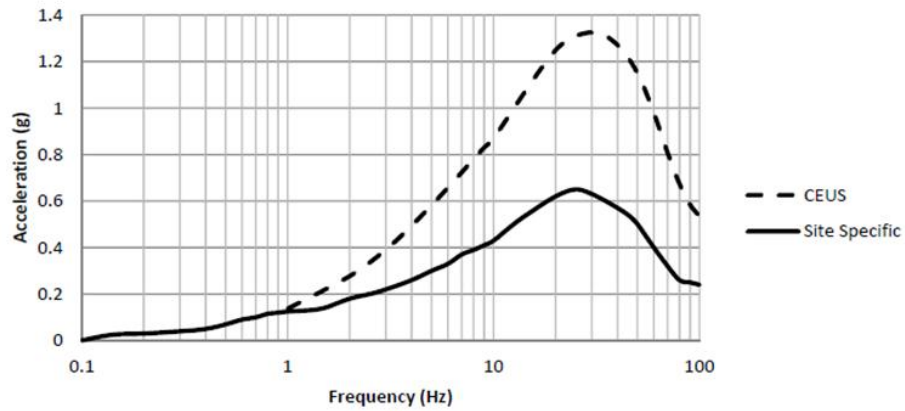


Figure 6-5
Example of Ground Response Spectra that are Similar

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.

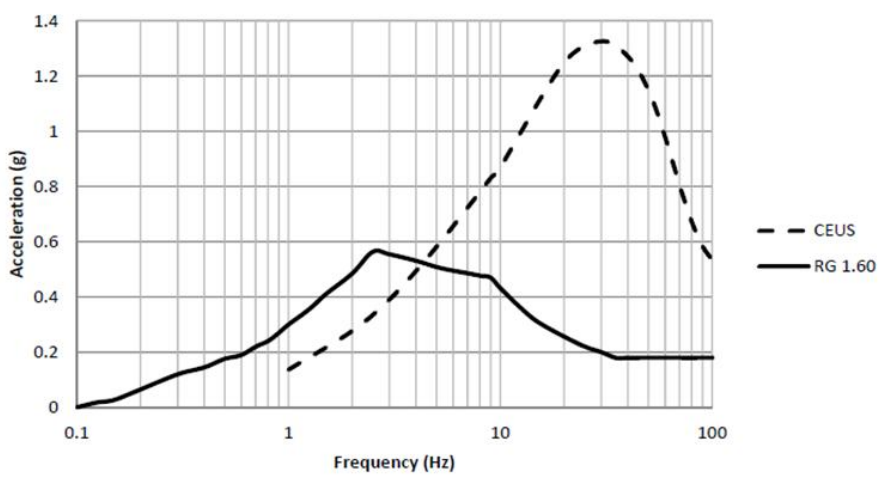


Figure 6-6
Example of Ground Response Spectra that are not Similar

A summary of an example for the scaling of "non-similar" shapes conducted for the Surry Pilot SPRA project is discussed below:

(Example will be provided here)

6.3.4 Fixed-Based Analysis of Structures for Sites Previously Defined as "rock"

For nuclear safety-related structures founded on the commonly used definition of rock for many operating plants (i.e., shear wave velocity > about 3500 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. In general, SSI analyses tend to reduce the structural responses. Therefore, it is conservative 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. The original definition of rock (> 3,500 ft/sec) that was used by some plants in the past can still be used as the criterion (with a potential caveat as discussed below) 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 an NPP. The results from the first example lead to a caveat on the use of the above approach. These examples are discussed below.

The first example was the analysis of a typical containment structure that has a fixed-base fundamental mode of about 5 Hz. This structure was analyzed [42] as fixed-base and then with SSI with three sets of shear wave velocities (SWV) for this site: lower bound (about 3,400 ft/sec), best estimate (about 5,200 ft/sec) and an upper bound value (about 7,900 ft/sec). The ISRS at the operating deck (about 75 ft higher than the basemat) of the containment structure from the SSI analyses (lower bound and best estimate values of the SWV, comparison of upper bound is closer to the fixed base and not presented here) were compared to the ISRS from a fixed-base analysis. Figures 6-7, 6-8, and 6-9 show these comparisons for the east-west, north-south, and vertical directions, respectively. From these figures, it can be seen that for the east-west and north-south directions, the lower bound SWV case resulted in a slight shift of frequency of the spectral peak as compared to the hard rock (about 1 Hz lower with SSI) because of the rotational effects. For the best estimate SWV case, 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 no frequency shift compared to the fixed-base case. In all three directions, the spectral peaks from the fixed-base analysis were higher in comparison to the SSI analyses, and the shape of the ISRS in the entire frequency range remained about the same. Therefore, for a tall structure like the Reactor Containment, if one can accept a slight potential frequency shift of the spectral peak, then a fixed-base analysis can be used for sites with rock SWV>3,500 ft/sec. If not, then for a tall structure such as the Reactor Containment structure, a fixed-base analysis is only recommended for rock SWV > 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.

The second example structure, which has a fundamental mode of about 10 Hz in one horizontal direction, was also analyzed as fixed-base and then with SSI analysis. For this structure (which is not as tall as the Reactor Containment structure), the results show that..... Therefore, ...(analysis is in progress).

The determination whether a fixed-base model for rock sites with shear wave velocity above 3,500 ft/sec is appropriate should be made by an experienced structural engineer and peer reviewed and justified in the submittal report to NRC.

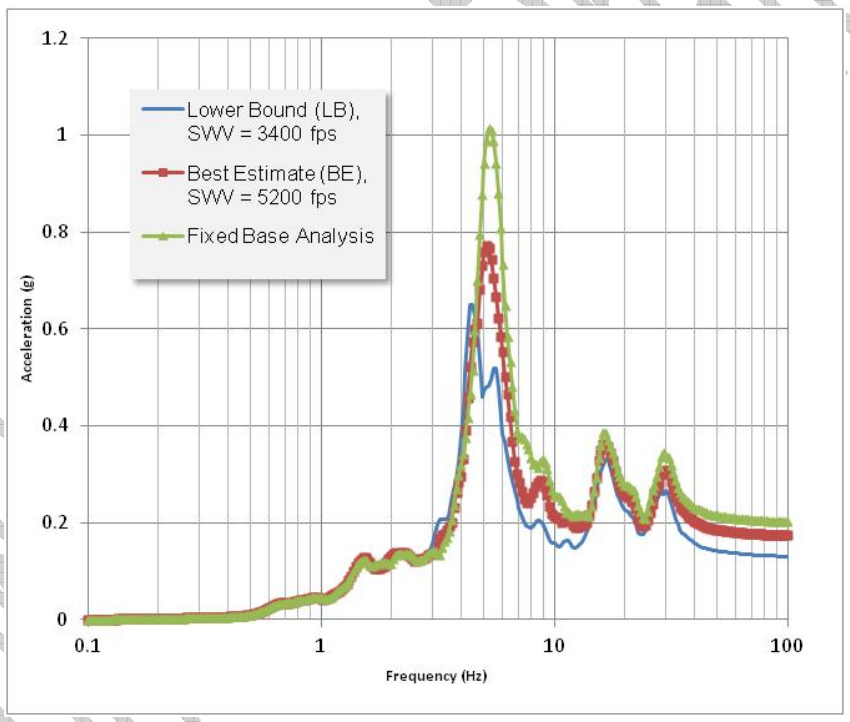


Figure 6-7
Reactor Containment Internal Structure, Operating Floor East-West Direction, 5% damping

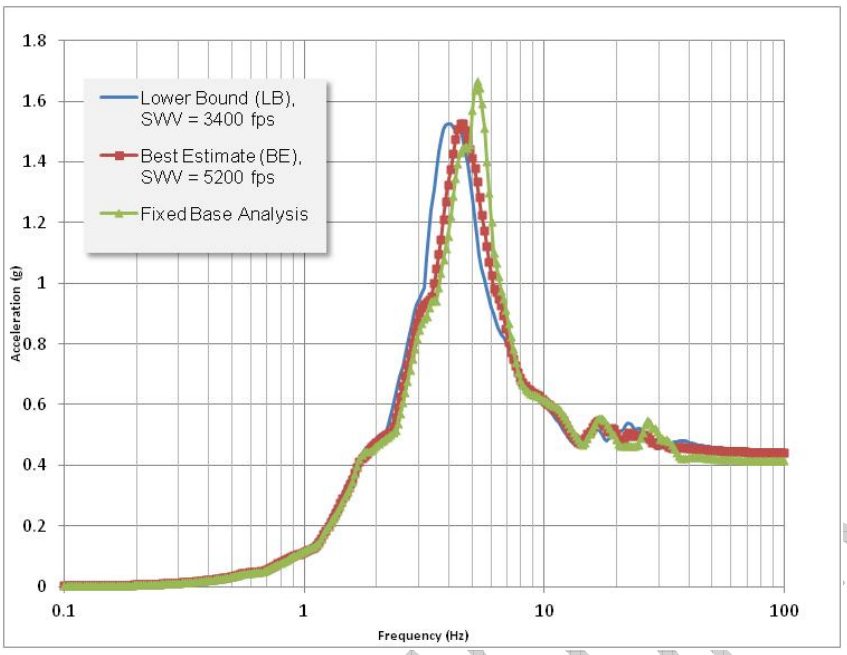


Figure 6-8
Reactor Containment Internal Structure, Operating Floor North-South Direction, 5% damping

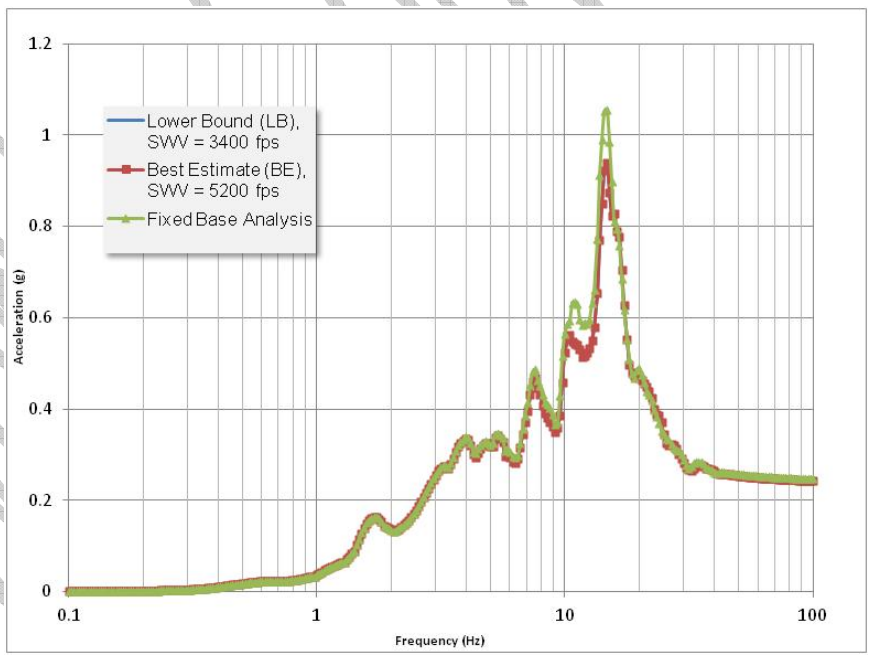


Figure 6-9
Reactor Containment Internal Structure, Operating Floor Vertical Direction, 5% damping

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 [39, 36, 37, 38]. 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, well known to many more analysts who perform seismic PRAs, and is acceptable for generating fragilities within an SPRA. Therefore, the CDFM approach can be used to calculate fragilities of SSCs in seismic PRAs for NTTF Recommendation 2.1.

In the CDFM fragility approach (also referred to as 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 β_C and its subdivision into random variability β_R and uncertainty β_U are used to estimate the corresponding fragility curve. As noted in Ref. [51], typically β_C lies within the range of 0.3 to 0.6. In fact, if all of the sources of variability discussed in Ref. [38] are appropriately considered, it is not possible to obtain an estimated β_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 β_C. This annual probability (seismic risk) can be computed with adequate precision from the CDFM Capacity C_{CDFM} and an estimate of β_C. It is shown in Ref. [51] that the computed seismic risk at $\beta = 0.3$ is approximately 1.5 times that at $\beta = 0.4$, while at $\beta = 0.6$ the computed seismic risk is approximately 60% of that at $\beta = 0.4$.

Table 6-2 provides recommended values for β_C, β_R, β_U, and the ratio of the median capacity $C_{50\%}$ to the $C_{1\%}$ capacity computed by the CDFM Method. The recommended β_C values are based on Ref. [51] recommendations and on average are biased slightly conservative (i.e., slightly low β_C on average). Since random variability β_R is primarily due to ground motion variability, a constant β_R value of 0.24 is recommended irrespective of the SSC being considered. The recommended β_U values are back-computed from the recommended β_C and β_R values.

Table 6-2
Recommended βC, βR, βU, and C50%/C1% Values to Use in Hybrid Method for Various Types of SSCs

Type SSC	Composite B_C	Random B_R	Uncertainty B_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 select SSCs for which fragility analyses should be done are 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 establish which SSCs will require explicit calculation of fragility parameters for inclusion in the plant logic models. SSCs with capacities above the level based on the criteria will not have any significant impact on the seismic PRA analyses, ranking of accident sequences, or the seismic CDFs of accident sequences or the plant.

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. 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 is available for identifying seismically rugged SSCs [39, 41].

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 6-10 illustrates the use of screening level, which is applicable to the SSCs in the middle box.

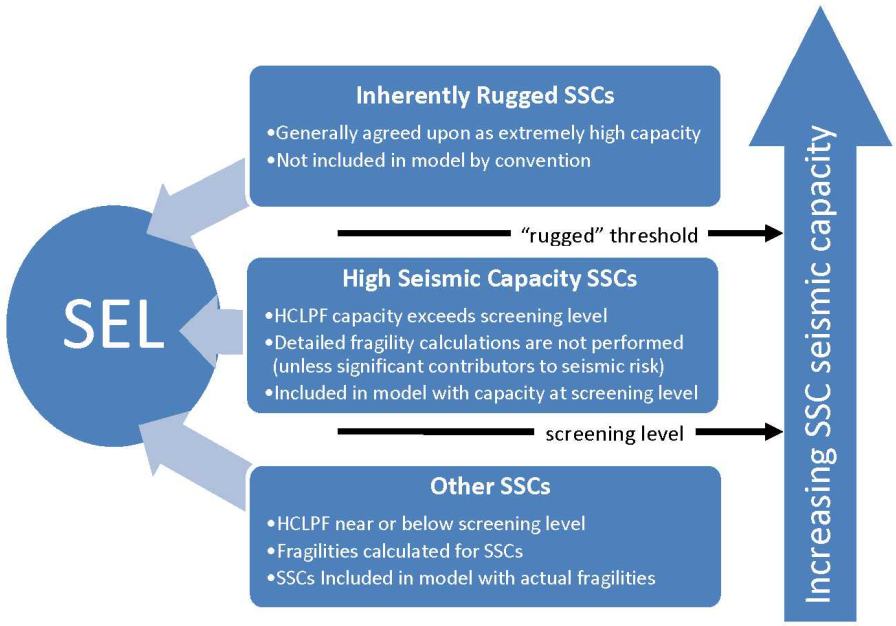


Figure 6-10
Capacity-based Criteria for Fragility Analyses

The criteria were developed using a parametric sensitivity analysis with several different hazard curves (PGA, as well as 5 Hz and 10 Hz) for soil and rock sites and a typical plant logic model from a recent seismic PRA. A large number of parametric analyses were performed. Cumulative effects of screening out several SSCs in a logic model were considered in two ways: first about one-fifth of the SSCs in the logic model that were known to be risk-significant were modeled at the screening level

(conservative) and next, one-fifth of somewhat randomly chosen SSCs were modeled (more realistic). These analyses were done by convolving the fragilities with the hazard curves, one at a time, for the selected logic model. The results were tabulated in this white paper, and sensitivity of various parameters was reviewed to develop the screening criteria. These parametric/sensitivity analyses are documented in a white paper [42]. The criteria are as follows:

The screening HCLPF value of SSCs should be calculated by convolving the fragility of a single element with the site-specific hazard curve such that the seismic CDF is 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 optional criterion, equivalent to the above CDF-based HCLPF, is to screen SSCs above about 2.5 x GMRS HCLPF value. There appears to be no significant distinction for the screening criteria between the hazard curves for a soil site and a rock site.

The sensitivity analyses 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 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. Retention of such failures will ensure that future changes or sensitivities that could increase their importance are not overlooked.

SSCs modeled at the screening level should also be assessed if they are significant contributors to CDF or to LERF sequences. If they are, then detailed fragility calculations should be performed for those SSCs. One approach to demonstrate that is to model the screening level SSCs at three times that level in the logic model, quantify the seismic CDF/LERF and ensure that the CDF/LERF estimates are not reduced by more than 20%. 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 greater than about 2%, the criterion may need to be adjusted.

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.

In conclusion, the above screening criteria will ensure that the time and effort to develop detailed fragilities is focused on the most risk-significant SSCs. It is expected that 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

6.5.1 LERF Considerations

(Additional text to be inserted)

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

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 account for plant-specific configuration and design, and reflect plant-specific reliability data where it could affect the important risk contributors.

For this application, which is aimed at developing an improved understanding of the impact of new 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 timely 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.

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 SPRA/SMA technical requirements may be used if they provide results that are equivalent or superior to the methods usually used.

The Peer Review Team should consist of a minimum of two individuals, one of whom has seismic engineering experience as it applies to nuclear power plants. 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] or equivalent, or shall have demonstrated equivalent experience in seismic walkdowns.

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
- Review a sample of 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 elements of the peer review process is characterized in the most recent SPRA Standard [53] Section 5.3, titled "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 fragilities.

It is also recommended that these peer review activities be performed during implementation of this program, to the extent practicable, rather than waiting until all the work is complete.

6.8 Evaluation Criteria/Significance for Risk Contributors and Risk Contribution

(Additional text to be inserted)

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 spent fuel pool (SFP). More specifically, plants were asked to consider "…all seismically induced failures that can lead to draining of the SFP."

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.

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.

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 inch 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.

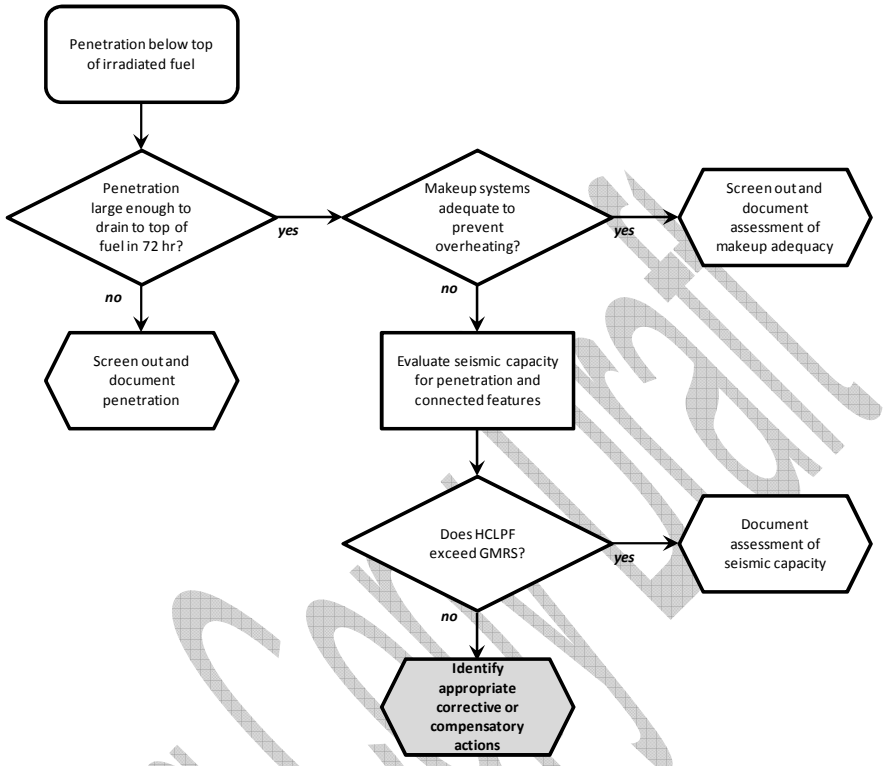


Figure 7-1
Basic Process for Evaluation of Potential Failures for Penetrations below Top of Irradiated Fuel

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].

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 a 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.

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.

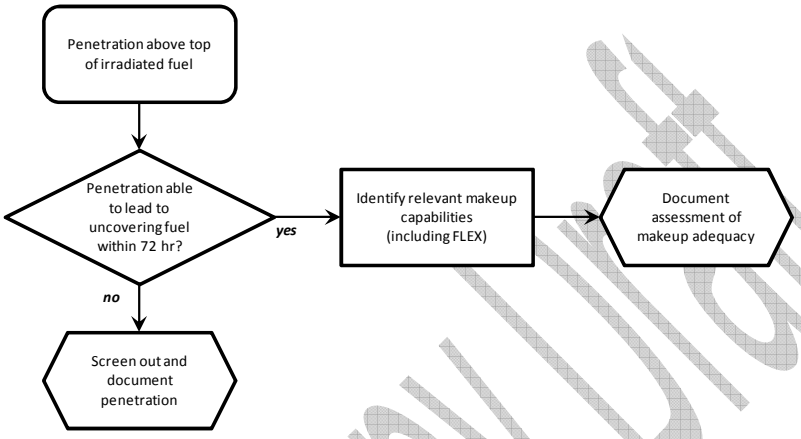


Figure 7-2
Basic Process for Evaluation of Potential Failures for Penetrations above Top of Irradiated Fuel

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.

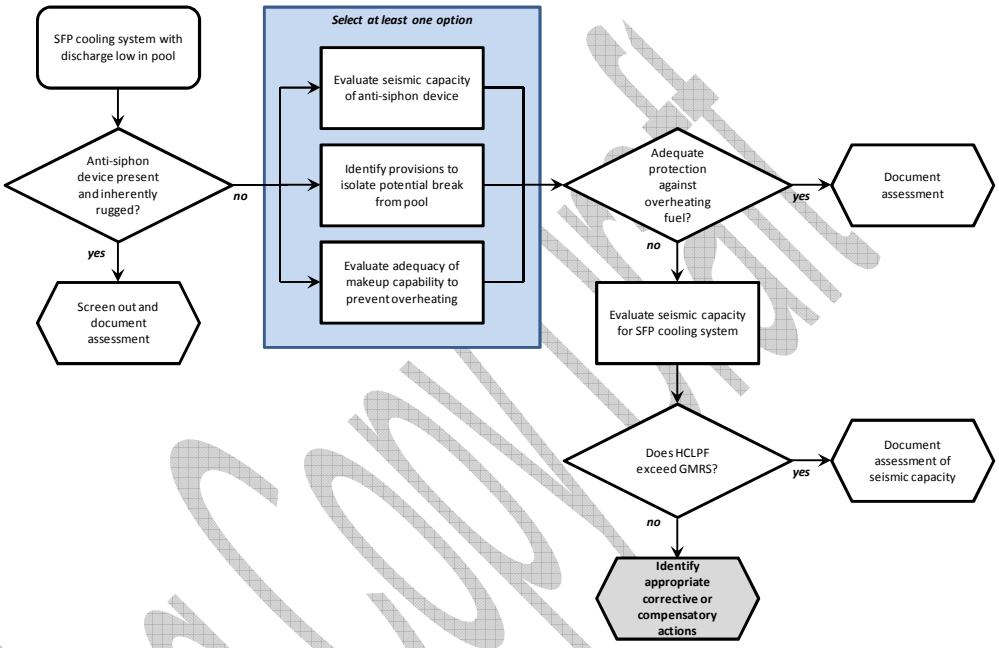


Figure 7-3
Basic Process for Evaluation of Potential Siphoning of SFP Inventory

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:

$$f_{c1} = (\tfrac{1}{2}\pi)[3.16g / L)\ \tanh(3.16h / L)]^{0.5} \qquad \text{Equation 7-1}$$

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:

$$h_{s1} = \tfrac{1}{2}L(SA_{c1} / g) \qquad \text{Equation 7-2}$$

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 peak ground acceleration 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.

Section 8: References

1. NRC (E Leeds and M Johnson) Letter to All Power Reactor Licensees et al., “Request for Information Pursuant to Title 10 of the Code of Federal Regulations 50.54(f) Regarding Recommendations 2.1, 2.3 and 9.3 of the Near-Term Task Force Review of Insights from the Fukushima Dai-Ichi Accident”, March 12, 2012, Nuclear Regulatory Commission.
2. NRC Supplement 4 to Generic Letter (GL) 88-20, "Individual Plant Examination of External Events (IPEEE) for Severe Accident Vulnerabilities," (ADAMS Accession No. ML031150485).
3. EPRI TR-1000895, “Individual Plant Examination for External Events (IPEEE) Seismic Insights”, Revision to EPRI Report TR-112932, Electric Power Research Institute, December 2000.
4. NUREG-1742, "Perspectives Gained from the Individual Plant Examination of External Events (IPEEE) Program," issued April 2002 (ADAMS Accession Nos. ML021270070 and ML021270674).
5. NRC letter, May 26, 2005, recommendation to create GI-199, ML051450456
6. NRC, June 9, 2005, GI-199 established, ML051600272
7. NRC meeting, February 2008, ML073400477 and ML080350189
8. NRC Safety/Risk Analysis for GI-299, September 2, 2010, ML100270582
9. NRC Information Notice 2010-018, "Generic Issue 199, Implications of Updated Probabilistic Seismic Hazard Estimates in Central and Eastern United States on Existing Plants,” September 2, 2010, ML101970221
10. NRC, Draft Generic Letter 2011-XX: Seismic Risk Evaluations for Operating Reactors, July 26, 2011, ML11710783
11. NRC NUREG-1407, Procedural and Submittal Guidance for the Individual Plant Examination of External Events (IPEEE) for Severe Accident Vulnerabilities, June 1991
12. American Society of Mechanical Engineers/American Nuclear Society (ASME/ANS) RA-Sa-2009
13. Budnitz, et al (1997). Recommendations for Probabilistic Seismic Hazard Analysis: Guidance on Uncertainty and Use of Experts.

Report NUREG/CR-6372, Lawrence Livermore National Laboratory. Sponsored by the U.S. Nuclear Regulatory Commission, U.S. Department of Energy, and the Electric Power Research Institute

14. NUREG-2115, DOE/NE-0140, EPRI 1021097. Central and Eastern United States Seismic Source Characterization for Nuclear Facilities, 6 Volumes, January 2012.
15. NRC endorsement of the 2012 CEUS SSC
16. EPRI 1009684, CEUS Ground Motion Project Final Report, Electric Power Research Institute, December 2004.
17. EPRI 1014381, Program on Technology Innovation: Truncation of the Lognormal Distribution and Value of the Standard Deviation for Ground Motion Models in the Central and Eastern United States, Electric Power Research Institute, August 2006.
18. Regulatory Guide 1.208, "A Performance-Based Approach to Define the Site-Specific Earthquake Ground Motion," U.S. Nuclear Regulatory Commission, 2007.
19. EPRI 1014099, Program on Technology Innovation: Use of Cumulative Absolute Velocity (CAV) in Determining Effects of Small Magnitude Earthquakes on Seismic Hazard Analyses, August 2006
20. Reference for equation 6-5 in Section 2.5.1
21. EPRI report justifying the 43-05 method.
22. (not used)
23. NUREG -2117, Practical Implementation Guidelines for SSHAC Level 3 and 4 Hazard Studies: Rev. 1, Office of Nuclear Regulatory Research, 2012, ML12118A445.
24. NUREG/CR-6728, Technical Basis for Revision of Regulatory Guidance on Design Ground Motions: Hazard- and Risk-Consistent Ground Motion Spectra Guidelines, Risk Engineering, Inc., October 2001.
25. IEEE Standard 344 2004, "IEEE Recommended Practice for Seismic Qualification of Class I E Equipment for Nuclear Power Generating Stations".
26. EPRI NP-7498, "Industry Approach to Severe Accident Policy Implementation," November 1991
27. EPRI 7148, "Procedure for Evaluating Nuclear Power Plant Relay Seismic Functionality," December 1990
28. EPRI 1015108, "Program on Technology Innovation: The Effects of High-Frequency Ground Motion on Structures, Components, and Equipment in Nuclear Power Plants," June 2007.

29. EPRI 1015109, "Program on Technology Innovation: Seismic Screening of Components Sensitive to High-Frequency Vibratory Motions," October 2007, (Also available under ML072600202)

30. Westinghouse AP1000 Design Control Document, Revision 17, September 2008

31. IEEE C37.98-1987, IEEE Standard Seismic Testing of Relays

32. NRC, Risk-Informed Regulation Implementation Plan, 2000

33. EPRI 1002989, Seismic Probabilistic Risk Assessment Implementation Guide, December 2003

34. IAEA Safety Report Series No. 28, Seismic Evaluation of Existing Nuclear Power Plants, International Atomic Energy Agency, 2003

35. IAEA TECDOC-724, Probabilistic Safety Assessment for Seismic Events. International Atomic Energy Agency 1993

36. EPRI 1019200, Seismic Fragility Applications Guide Update, December 2009

37. EPRI 1002988, Seismic Fragility Application Guide, December 2002

38. EPRI TR-103959, Methodology for Developing Seismic Fragilities, July 1994

39. EPRI NP-6041-SL, "A Methodology for Assessment of Nuclear Plant Seismic Margin, Revision 1", Electric Power Research Institute, August 1991.

40. ASCE 4-98, Seismic Analysis of Safety-Related Nuclear Structures, American Society of Civil Engineers

41. EPRI TR-104871 "Generic Seismic Technical Evaluations of Replacement Items (GSTERI) for Nuclear Power Plants", Electric Power Research Institute, May 1995.

42. "Development of Capacity-Based Screening Criteria for Structures, Systems and Components for Performing a Seismic Probabilistic Risk Analysis", Technical Report (in progress), Dominion Generation, September 2012.

43. NRC Regulatory Guide 1.200, An Approach For Determining The Technical Adequacy Of Probabilistic Risk Assessment Results For Risk-Informed Activities, Revision 2, March 2009

44. Addenda B to ASME/ANS RA-Sa-2009, the version of the Standard that is currently undergoing balloting

45. Dominion/Bechtel Calculation 25161-G-606, August 2012

46. EPRI 1025286, Seismic Walkdown Guidance for Resolution of Fukushima Near Term Task Force Recommendation 2.3: Seismic, June 2012.

47. NRC NUREG-1738, *Technical Study of Spent Fuel Pool Accident Risk at Decommissioning Nuclear Power Plants*. February 2001

48. NRC NUREG/CR-5176, *Seismic Failure and Cask Drop Analyses of the Spent Fuel Pools at Two Representative Nuclear Power Plants*. January 1989.

49. EPRI 1025295*, Update of the Technical Bases for Severe Accident Management Guidance*. Electric Power Research Institute, August 2012.

50. NEI-12-06*, Diverse and Flexible Coping Strategies (FLEX) Implementation Guide*. Nuclear Energy Institute, Revision B1, May 2012.

51. 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.

52. "Walkdown Screening and Seismic Evaluation Training Course and Add-On SMA Training Course," Seismic Qualification Utility Group (1993); available from Electric Power Research Institute.

53. “Addenda to ASME/ANS RA-S-2008 Standard for Level 1/Large Early Release Frequency Probabilistic Risk Assessment for Nuclear Power Plant Applications”, American Society of Mechanical Engineers and American Nuclear Society, 2009.

54. NRC NUREG/CR-2300, PRA Procedures Guide: A Guide to the Performance of Probabilistic Risk Assessments for Nuclear Power Plants, January 1983.

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

1.0 Introduction

Site-specific amplification factors (5% damped pseudo absolute acceleration, Sa) for soil and firm rock profiles were developed for each site using equivalent-linear analyses for cases which had surficial materials with shear-wave velocity less than the hard rock reference value of 2.83 km/sec (9,285 ft/sec) and of sufficient thickness to impact design motions at frequencies below about 50 Hz. The conservative criteria used to qualify a site for development of site-specific amplification, defined as Sa of the site divided by Sa of reference rock, was more than 25 ft of material over hard rock with an average shear-wave velocity less than 7,500 ft/s. Site-specific amplification was developed for all sites not meeting the combined stiffness and velocity criteria. Epistemic uncertainty in dynamic material properties; shear-wave velocity profile, G/G_{max} and hysteretic damping, and total profile low strain damping (kappa; [3]) was accommodated by development of multiple base-case models. For each base-case set of dynamic material properties, median and $\pm 1\sigma$ amplification factors were computed over a range in reference site loading levels. The aleatory variability about each base-case set of dynamic material properties was developed by randomizing (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 rock extended to depths exceeding 500 ft, linear response was assumed [28, 27, 26, 25].

In all site response analyses motions were generated with the point-source model. The effects of control motion spectral shape on site amplification were examined conditional on reference site peak acceleration. The potential effects of both magnitude as well as single- verses double-corner [4] source spectra at large magnitude (**M** 6.5) were considered for a stiff soil profile. The results suggested the difference in amplification between single- vs. double-corner source models were significant enough to consider the implied epistemic uncertainty in

central and eastern North America (CENA) source processes at large magnitude (Section 3.1.2).

Considering magnitude, since the dominant contributions to the hazard across the central and eastern US. (CEUS) and oscillator frequency reflect magnitudes between about **M** 6.0 and **M** 7.5, the analyses suggested the use of a single magnitude of **M** 6.5 resulted in slightly conservative amplification (about 5%) for **M** 7.5 and a slight unconservative amplification (about 5 to 10%) for **M** 5.5 (Section 3.1.1). The difference between amplification computed for **M** 6.5 and both **M** 5.5 and **M** 7.5 were in the 5 to 10 Hz frequency range and only at high loading levels of hard rock peak accelerations of 0.50g and above. In view of the little contribution of **M** 5.5 to the CEUS hazard, being dominated by about **M** 6.0 and larger, the use of a single magnitude of **M** 6.5 was adopted in lieu of two magnitudes, **M** 6.0 and **M** 7.5.

2.0 Development of Base-Case Profiles and Assessment of Epistemic Uncertainty in Dynamic Material Properties

In general a mean based approach was taken to accommodate epistemic uncertainty in dynamic material properties: shear-wave velocity profile, site material damping at low strain (parameterized through kappa), and modulus reduction and hysteretic damping curves. In this context, for poorly characterized sites with few if any measured dynamic material properties, three profile estimates combined with three kappa estimates and two sets of modulus reduction and hysteretic damping curves results in eighteen sets of amplification factors for each magnitude. The three estimates for the shear-wave velocity profile and kappa were intended to capture uncertainty of the mean or best-estimate profile and kappa value and reflect the mean base-case as well as upper and lower range base-cases. A general set of guidelines was employed to develop base-cases for dynamic material properties as well as associated weights as given below.

2.1 Shear-Wave Velocity Profiles

For sites with sparse information regarding dynamic material properties (e.g., shear-wave velocity profile was unavailable), typically an estimate based on limited surveys (e.g., compressional-wave refraction) was available over some shallow 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 templates was adopted, parameterized with $\mathbf{V_s}$ (30m) ranging from 190m/s to 2,032m/s. The suite of profile templates is shown in Figure xx.1 to a depth of 1,000 ft. The templates were taken from [32] supplemented for the current application with 190m/s, 1,364m/s, and 2,032m/s. The latter two profiles were added to accommodate residual soils (saprolite) overlying hard rock. For both soil and soft rock sites, the profile with the closest velocities over the appropriate depth range was 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 closest template profile was used at the appropriate depth. The template was used to provide a rational basis to extrapolate the profile to the required depths.

For firm rock sites, which are typically Paleozoic sedimentary rocks such as shales, sandstones, siltstones, etc., the shear-wave velocity gradient of 0.5m/s/m from deep measurements in similar rocks in Japan [19] was adopted as a template and adjusted to the velocities provided over the appropriate depth range. The gradient of 0.5m/s/m 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. 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 amplification.

2.1.1 Epistemic Uncertainty, σ_μ

Profile epistemic uncertainty was taken as 0.35 (σ_μ ln) throughout the profile and was based on the estimates of epistemic uncertainty in V_s (30m) [9] for stiff profiles. The logarithmic factor assumes shear-wave velocities are lognormally distributed and was originally developed to characterize the epistemic uncertainty in measured V_s (30m) at ground motion recording sites where measurements were taken at a maximum distance of 300 m from the actual site. The uncertainty accommodates spatial variability over maximum distances of 300 m, and was adopted here as a reasonable and realistic uncertainty assessment reflecting a combination of few velocity measurements over varying depth ranges and shear-wave velocities inferred from compressional-wave measurements, in addition to the spatial variability associated with the velocities provided by the utilities. The application of the uncertainty estimate over the entire profile that is based largely on V_s (30 m) implicitly assumes perfect correlation that is independent of depth. While velocities are correlated with depth beyond 30 m, which forms the basis for the use of V_s (30 m) 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].

The value of 0.35 adopted for profile epistemic uncertainty is also similar to the COV of 0.25 for shear-wave velocities over the top 50 to 75 ft found by Toro [28] for spatial variability over structural footprint dimensions of several hundred feet. In this case, based on measured profiles over a

footprint, the COV decreased for depths beyond about 75 ft indicating less spatial variability with increasing depth. For the current application, the value of 0.35 is retained with depth, as the actual epistemic uncertainty is more appropriately considered to be depth independent as the uncertainty was intended to capture reasonable ranges in gradients.

More direct support for the assumption of a σ_μ of 0.35 comes from the measured range in (V_s (30 m) conditional on proxy inferences. For the four currently employed (V_s (30 m) proxies surficial geology (age; [34, 33]) Geomatrix site category [9], topographic slope [31], and terrain [35], the overall within class or category uncertainty is about 0.35. This uncertainty reflects the variability of measured (V_s (30 m) about the predicted value and is relatively constant across proxies [23]. The proxy uncertainty of about 0.35 is a direct measure of the uncertainty of the predicted value or estimate and supports the adoption of 0.35 to quantify the velocity uncertainty for cases with few or absent direct measurements. For the application to site characterization the σ_μ of 0.35 has been extended to the entire profile.

For cases where site-specific velocity measurements are particularly sparse (e.g., based on static tests for shear- and compressional-moduli), the uncertainty was increased to 0.50. The uncertainty was also increased to 0.50 in the absence of site-specific velocity measurements. For these cases the mean base-case profile was based on analogue materials from soil and rock descriptions provided by the utility. For the current application, where the desire was to implement a consistent approach to develop representative profiles and associated uncertainties to achieve mean based estimates of hazard, the assumption of a depth independent correlation is considered to reflect a sufficiently accurate approximation to the actual epistemic uncertainty.

2.1.2 Weights

To provide an expression of the epistemic uncertainty of a σ_μ (ln) of 0.35 in the final site-specific hazard, median amplification factors over thirty realizations expressing aleatory variability were computed for the base-case profile (mean base-case) as well as upper range and lower range base-cases. For scale factors and associated weights, to reflect a realistic uncertainty in the profile, the 90% level was adopted with a corresponding profile scale factor of 1.3 σ_μ or 0.45. This represents an absolute factor of 1.57 on the mean base-case profile.

To accommodate the range in velocity profiles in terms of relative weights, the lower- and upper-range profiles were assumed to reflect 10th and 90th fractiles. The corresponding weights reflecting an accurate three point approximation of a normal distribution which preserves the mean are 0.30 for the 10th and 90th percentiles and 0.40 for the median [21] and summarized in Table xx.2. In the sections for each site, relative weights

are tabulated, including any departures based on available site-specific information.

As an illustration, Figure xx.2 shows the 560m/s base-case profile template (Figure xx.1) along with the expression of epistemic uncertainty as upper and lower ranges based on the scale factor of 0.45 (1.3 σ_μ). Also shown in Figure xx.2 is the firm rock gradient adopted from deep measurements of sandstones and siltstones in Japan of 0.5m/s/m [19]. The assumed gradient is also consistent with that reflected in the Tertiary claystones, siltstones, sandstones, and conglomerates for measured shear-wave velocities at the Varian Well in Parkfield, California [20].

For the illustration the firm rock was taken to have an initial shear-wave velocity of 5,000 ft/s and the gradient was considered to reflect sedimentary rocks such as sandstones, siltstones, mudstone, and shales. In application the empirical gradient was simply scaled to reflect the shear-wave velocity and depth range specified for the particular site. If velocity information was lacking with only rock type specified, 5,000 ft/s was assumed at the surface and σ_μ taken as 0.5.

It is important to note for deep profiles, while the range in relative velocities is depth independent, the range in absolute velocity is large, accommodating a range in potential gradients as Figure xx.2 illustrates. For shallow profiles, at a shear-wave velocity of 1,000 ft/s, the lowest stiffness considered for siting a NPP, an uncertainty of 0.35 (scale factors of 1.3σ_μ) results in a considerable range in velocity from 633 ft/s to 1,576 ft/s. This velocity range reflects a range in elastic high-frequency amplification, relative to hard rock at 2.83 km/s of 3.8 to 2.4.

2.1.1.1 Sites with Intermediate Velocity Information

The profile σ_μ of 0.35 was adopted for cases of few if any measurements and reflects a maximum uncertainty. At the other extreme, for cases with complete and relatively recent (< 30 years) shear-wave velocity measurements throughout the entire profile, the epistemic uncertainty in shear-wave velocity was taken as zero.

For cases with intermediate or incomplete shear-wave velocity information, which reflects the majority of sites, the assessment of profile epistemic uncertainty was necessarily dependent on the mix of available profile information and based solely on judgment.

For the intermediate cases which, for example, shear-wave velocity inferred from in-situ compressional-wave measurements over extended portions of the profile (e.g., several hundred feet), σ_μ was divided by 2 with the corresponding scale factor of 1.29 σ_μ (1.26) to generate upper- and lower-range base-cases. For deeper depths the uncertainty was

increased to 0.35 with offsets in the upper- and lower-range base-cases tapered to reduce the development of resonances.

2.2 G/Gmax and Hysteretic Damping Curves

To characterize the epistemic uncertainty in nonlinear dynamic material properties for both soil and firm rock sites, two sets of modulus reduction and hysteretic damping curves were used.

For soils, the two sets of curves used were EPRI (1993) and Peninsular Range [28, 32]). 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 xx.3, 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 xx.3.

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 EPRI (1993) curves applied to 0 to 50 ft and EPRI (1993) 501 to 1,000 ft curve applied to 51 to 500 ft.

The two sets of soil curves were 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 cobles. 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 two sets of soil curves were given equal weights (Table xx.2) and are considered to represent a reasonable accommodation of epistemic uncertainty in nonlinear dynamic material properties for the generic types of soils in CEUS:

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.

To illustrate the difference in amplification between the EPRI (1993) and Peninsular Range cohesionless soil curves, Figure xx.4 shows estimates of median amplification factors (5% damped Sa) computed for template profile 400m/s (Figure xx.1) and a depth of 500 ft to basement rock using both sets of curves. In Figure xx.4 hard reference rock loading levels range from 0.01g to 1.50g (Table xx.1) and shows significantly greater nonlinear effects for the EPRI (1993) compared to the Peninsular Range curves at high frequency (≥ 2 Hz).

For firm rock, taken generally as Paleocene sedimentary rocks, such as shales, sandstones, siltstones, etc., two expressions of nonlinear dynamic material properties were assumed: EPRI rock curves (Figure xx.5) 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 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 with significant nonlinearity confined largely to the very high loading levels (e.g. ≥ 0.75g).

As an alternative to the EPRI rock curves, linear response was assumed. Implicit in this model is purely elastic response accompanied with damping that remains constant with cyclic shear strain at loading levels up to and beyond 1.5g (reference site). Note in all cases the amplification at 1.5g was applied at higher loading levels.

Similar to the two sets of curves for soils, equal weights were given to the two sets of nonlinear properties for firm rock as summarized in Table xx.2.

2.3 Kappa

In the context of this report the kappa referred to is the profile damping contributed to 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 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" Q_s within the profile [7].

2.3.1 Low Strain Kappa for Rock Sites

Mean base-case kappa values were developed differently for soil and firm rock sites. For rock sites with at least 3,000 ft of firm sedimentary rock ($\overline{V}_s$ (30m) > 500m/s) overlying hard rock, the kappa $\overline{V}_s$ (100 ft) relation of

$$\kappa = 2.2189 - 1.0930 \log (\overline{V}_s (100\ ft)) \qquad \text{Equation B-1}$$

was used to assign a mean base-case estimate for kappa [25, 22]. The requirement of 3,000 ft reflects the assumption that the majority of damping contributing to kappa occurs over the top 3,000 ft 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, Equation 1 gives a kappa estimate of about 0.02s. The assumption implies a kappa of 0.014s is contributed by the sedimentary rock column, 0.006s from the underlying reference rock (Table xx.1), and reflects an average Q_s of about 40 over 3,000 ft. The Q value of 40 for sedimentary rocks is consistent with the average value of 37 over the depth range of 0m to 298m in Tertiary claystones, siltstones, sandstones, and conglomerates at a deep borehole in Parkfield, California [20]. The average shear-wave velocity of about 4,800 ft/s [7] is somewhat below the assumed base-case average of about 5,600 ft/s but well within the epistemic uncertainty (Figure xx.2).

For low firm rock velocities an upper bound kappa value of 0.04s was 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 was less than about 3,000 ft, kappa contributed by the firm rock profile was computed assuming a Q_s of 40 plus the contribution of the reference rock profile of 0.006s (Table xx.1a). For the three base-cases firm rock profiles shown in Figure xx.2, 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.

2.3.2 Low Strain Kappa for Soil Sites

For soil sites with depths exceeding 3,000 ft to hard rock, a mean base-case kappa of 0.04s was assumed based upon 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 was 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 was adopted for both the upper and lower range profiles with the assumption 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 Q_s within the 960m 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) in the CEUS, a nominal kappa value of 0.04s based on an average of many empirical estimates predominately in the WNA tectonic regime was adopted. 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 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.

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

$$\kappa(ms) = 0.0605 * H(m) \qquad \text{Equation B-2}$$

The basement kappa value of 0.006s (Table xx.1a) in lieu of Campbell's [7] estimate of 0.005s was added to the sediment contribution to estimate the total kappa. For 3,000 ft (1 km) of soil, Campbell's [7] relation predicts a total kappa of 0.066s, 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 Equation 2, a maximum kappa of 0.04s was implemented for sites with less than 3,000 ft of firm soils.

The final class of soils considered comprises soils over firm rock with a total (firm rock plus soil) thickness of less than 3,000 ft. For these cases the appropriate soil or firm rock approaches outlined above were applied.

Additionally, for these relatively shallow soil/shallow rock sites, a global maximum kappa for the mean base-case was taken as 0.04s, slightly lower than the average low strain value based on a large sample of WNA analyses [3, 29].

2.3.1 Epistemic Uncertainty

Epistemic uncertainty in kappa was taken as 50% (σ_μ = 0.40, Table xx.2; [12]) about the mean base-case estimate. 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 profiles (Section 2.1), the 1.68 (1.3 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 xx.2.

2.4 Densities

Because relative (soil surface/reference site) densities play a minor role in site-specific amplification, a simple model based on shear-wave velocity of the mean base-case profile was implemented for cases where profile density was not specified:

Table B-1

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

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 was considered necessary with the densities of the mean base-case profile held constant for the upper and lower range base-case profiles. This approach was a means of accommodating epistemic uncertainty in both density as well as shear-wave velocity (Section 2.1.1) in the suite of analyses over velocity uncertainty.

3.0 DEVELOPMENT OF AMPLIFICATION FACTORS

To develop amplification factors, the Mid-continent crustal model [12] with a shear-wave velocity of 2.83 km/sec, a defined shallow crustal damping parameter (kappa; [3]) of 0.006 sec, and a frequency dependent deep crustal damping Q model of 670 $f^{0.33}$ [12] (Table xx.1a) was used to compute reference motions (5% damped pseudo absolute acceleration spectra). The Q(f) kappa, and reference site shear-wave velocities are consistent with the EPRI GMPEs (Ground Motion Prediction Equations) [11] and the site-specific profiles were simply placed on top of this defined

crustal model which has a reference shear-wave velocity of 2.83 km/sec (≈ 9,300 ft/sec) and a reference kappa value of 0.006 sec. Distances were 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 (shear-wave velocity of 2.83 km/s, kappa = 0.006s; Table xx.1) were run from 0.01g to 1.50g (Table xx.1).

Amplification factors (5% damping response spectra) were then developed by placing the site profile on the Mid-continent crustal model at each distance, 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 were significantly less than 1, which may be exaggerated. To adjust the factors for these cases an empirical lower bound of 0.5 was implemented [12, 2].

3.1 Effects of Control Motion Spectral Shape on Amplification

Conditional on reference site peak acceleration, amplification factors depend, to some extent, upon control motion spectral shape due to nonlinear response. 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.

The other potential issue regarding control motion spectral shape that may affect amplification is 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. As a result, a simple source model was developed to characterize the potential for CENA large magnitude source spectra to reflect an intermediate frequency departure from the single-corner frequency point-source model [5]. The two-corner source model for CENA [4] manifests 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 with the depth and width of the spectral sag increasing with magnitude. 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 (Atkinson and Silva, 2000), albeit with a much less pronounced spectral sag than the CENA model.

3.1.1 Effects of Magnitude on Amplification

Figure xx.6 shows amplification factors developed for profile 400m/s (Figure xx.1) 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 xx.3) were used as dependence on control motion spectral shape decreases with degree of nonlinearity becoming independent for linear analyses. As Figure xx.6 illustrates, the largest amplification reflects the lowest magnitude. Over the frequency range of about 5 to 10 Hz, the largest range in amplification is about 20% and at the higher 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 and distribution of sites, the dominant contribution for the annual exceedance frequencies (AEF) of 10^{-4} and below are from magnitudes in the range of about **M** 6 to **M** 7+. As a result, to reduce the analyses to a manageable level, a single magnitude **M** 6.5 was selected to adequately characterize the amplification, with tacit acceptance of slight conservatism for magnitude contributions above about **M** 7.

3.1.2 Effects of Single-Verses Double-Corner Control Motion Spectral Shape on Amplification

One- and two-corner [4] source models were also used for **M** 6.5. While neither the single- or two-corner source models alone are considered appropriate for CENA sources due to a lack of observations for **M** > 6, 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 xx.2 for amplification factors developed 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 xx.7 illustrates as an example. For profile 400m/s Figure xx.6 compares the amplification computed for both as single- and double-corner source models and EPRI modulus reduction and hysteretic damping curves (Figure xx.3), the most nonlinear set of curves for soils. As Figure xx.7 shows, for loading levels up to about 0.5g (reference rock median peak acceleration), the difference in amplification is maximum around 10 Hz and in the 10% to 15% range, at higher loading levels the differences increase and become more significant.

Considering three profiles, three low-strain kappa values, two sets of G/G_{max} and hysteretic and hysteretic damping curves along with two magnitudes using both single- and double-corner source models results in a maximum of seventy-two sets of amplification factors for poorly characterized sites. Accommodation of epistemic uncertainty in dynamic material properties reflects a computationally intense process unless consideration is given to a statically based selection approach such as Latin Hypercubes. Alternatively the number of models to characterize the

epistemic uncertainty in dynamic material properties and source processes can be reduced. Reducing the number of magnitudes from two to one reduces the suite of amplification factors from 72 to 36 (Table xx.3). Further consideration of the sensitivities of source and site uncertainties on amplification can substantially reduce the number of models. For soils, since low strain kappa is only operational at the lowest loading levels as nonlinearity controls the effective damping throughout the nonlinear section (maximum of 500 ft) of the profile, multiple kappa values are redundant for practical applications. For soil sites overlying hard rock, the practical number of models required to characterize source and site epistemic uncertainty is reduced to 12 as shown in Table xx.3.

For firm rock sites, kappa represents a controlling parameter even for the nonlinear case using rock G/G_{max} and hysteretic damping curves (Figure xx.5) as the increase in kappa only occurs at very high loading levels due to the relatively high shear-wave velocities (Figure xx.2). For firm rock site conditions, since significant nonlinearity is expected only at very high loading levels, the effects of single-verses double corner source models are small, only the single-corner model was run, reducing the number of models to 18 (Table xx.3). For cases where firm rock shear-wave velocities were high, exceeding about 7,000 ft/s, only linear analyses were run further reducing the model count to 9.

Finally, the most computationally intense case was for soil overlying firm rock. To fully capture the range in epistemic uncertainty in both the soil as well as firm rock, the maximum model count was 36 (Table xx.3).

In all the cases for site type, soil, firm rock, and soil plus firm rock, available site-specific information and judgment conditioned the actual number of cases analyzed. In general emphasis was placed on adequately characterizing the epistemic uncertainty on a case-by-case basis.

4.0 ALEATORY VARIABILITY IN DYNAMIC MATERIAL PROPERTIES

To accommodate aleatory variability in dynamic material properties expected to occur across each site (footprint), shear-wave velocity profiles as well as G/Gmax and hysteretic damping curves were randomized. Since depth to hard rock material (defined as shear-wave velocity of 2.83 km/sec (9,285 ft/sec)) is poorly known at many deep soil and firm rock sites, it was taken at a large enough depth to accommodate maximum amplification to the lowest frequency of interest, 0.33 Hz [26, 25]. For these cases, basement depth was randomized over a range of ± 30% of the best-estimate depth to accommodate aleatory variability about the median amplification and to smooth over potential low-frequency resonances. For sites where depth to basement was relatively well known, perhaps from nearby well logs, a more restrictive range of ± 20% was

used. In all cases, the basement depth randomization assumed a uniform distribution [12].

The profile randomization scheme, which varies both layer velocity and thickness, is based on a correlation model developed from an analysis of variance of about 500 measured shear-wave velocity profiles [12, 28]. This model uses variability in velocity that is appropriate for a large structural footprint. The parametric variation includes profile velocity and layer thickness variation as well as depth to hard rock material (2.83 km/sec). To prevent unrealistic velocity realizations, a bound of $\pm 2\sigma$ was placed throughout the profile. For the footprint correlation model the empirical σ is about 0.25 and decreases with depth to about 0.15 below about 50 ft [28].

To accommodate aleatory variability in the modulus reduction and hysteretic damping curves on a generic basis, the curves were independently randomized about the base case values (Section 2.0). A log normal distribution was assumed with a σ_{ln} of 0.15 and 0.30 at a cyclic shear strain of 3×10^{-2}% for modulus reduction and hysteretic damping respectively [28] with upper and lower bounds of 2σ. 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 were 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 3×10^{-2}% cyclic shear strain, and applying this factor at all strains. The random perturbation factor was reduced or tapered near the ends of the strain range to preserve the general shape of the base-case curves [12, 28]. Also, damping was limited to a maximum value of 15% in this application for NPPs.

To accommodate epistemic uncertainty in dynamic material properties (Section 2.0), multiple base-case (mean) models were considered, each with associated aleatory variability captured by the randomization process. Amplification factors for each case of epistemic uncertainty in dynamic material properties were then expressed as median and $\pm 1\sigma$ estimates based on thirty realizations at each distance or reference site loading level [25, 26]. Final hazard was developed by weighting over exceedance frequencies computed for each base-case model.

5.0 REFERENCES

1. Abrahamson, N.A and K.M. Shedlock (1997). "Overview." Seis. Research Lett., 68(1), 9-23.
2. Abrahamson, N.A. and Silva, W.J. (1997). "Empirical response spectral attenuation relations for shallow crustal earthquakes." Seism. Res. Lett., 68(1), 94-127.

3. Anderson, J. G. and S. E. Hough (1984). "A Model for the Shape of the Fourier Amplitude Spectrum of Acceleration at High Frequencies." Bull. Seism. Soc. Am., 74(5), 1969-1993.

4. Atkinson, G. M., and D. M. Boore (1995). “New ground motion relations for eastern North America.” Bull. Seism. Soc. Am. 85, 17–30. 12.

5. Boore, D.M. (1983). "Stochastic simulation of high-frequency ground motions based on seismological models of the radiated spectra." Bull. Seism. Soc. Am., 73(6), 1865-1894.

6. Boore, D.M., E.M. Thompson, and H. Cadet (2011). “Regional correlations of VS30 and velocities averaged over depths less than and greater than 30m.” Bull. Seism. Soc. Am., in-press.

7. Campbell, K. W. (2009). “Estimates of shear-wave Q and κo for unconsolidated and semiconsolidated sediments in Eastern North America.” Bull. Seism. Soc. Am, 99(4), 2365-2392.

8. Chapman, M. C., P. Talwani, and R. C. Cannon (2003). Ground-motion attenuation in the Atlantic Coastal Plain near Charleston, South Carolina, Bull. Seism. Soc. Am., 93, 998–1011.

9. Chiou, B., R. Darragh, N. Gregor, and W. Silva (2008). “NGA project strong-motion database.” Earthquake Spectra, 24: 23-44.

10. Cramer, C. H., J. S. Gomberg, E. S. Schweig, B. A. Waldron, and K. Tucker (2004). The Memphis, Shelby Country, Tennessee, seismic hazard maps, U.S. Geol. Surv. Open-File Rept. 04-1294.

11. Electric Power Research Institute (2004). “CEUS Ground Motion Project” Palo Alto, Calif: Electric Power Research Institute, Final Report.

12. Electric Power Research Institute (1993). "Guidelines for determining design basis ground motions." Palo Alto, Calif: Electric Power Research Institute, vol. 1-5, EPRI TR-102293.

13. Not used.

14. Not used.

15. Not used.

16. Not used.

17. Not used.

18. Not used.

19. Fukushima, Y., J.C. Gariel, and R. Tanaka (1995). "Site-Dependent Attenuation Relations of Seismic Motion Parameters at Depth Using Borehole Data." Bull. Seism. Soc. Am., 85(6), 1790-1804.

20. Jongmans, D. and P.E. Malin (1995). "Microearthquake S-wave Observations from 0 To 1 km in the Varian Well at Parkfield, California." BSSA, 85(6), 1805-1820.

21. Keefer D. I. and Bodily, S. E. (1983). "Three-point approximations for continuous random variables" Management Science, 2(5), 595-695.

22. Houtte, C.V., S. Drouet, and F. Cotton (2011). "Analysis of the origins of κ (kappa) to compute hard rock to rock adjustment factors for GMPEs." Bull. Seism. Soc. Am., v.101, p. 2926-2941.

23. Silva, W. J., E. Thompson, H. Magistrale, C. Wills (2012). "Development of a V_s (30m) map for central and eastern north america (cena): median estimates and uncertainties." FM Global final report.

24. Not used.

25. Silva, W.J., R. Darragh, N. Gregor, G. Martin, C. Kircher, N. Abrahamson (2000). "Reassessment of site coefficients and near-fault factors for building code provisions." Final Report USGS Grant award #98-HQ-GR-1010.

26. Silva, W. J.,S. Li, B. Darragh, and N. Gregor (1999). "Surface geology based strong motion amplification factors for the San Francisco Bay and Los Angeles Areas."A PEARL report to PG&E/CEC/Caltrans, Award No. SA2120-59652.

27. Silva, W.J. Costantino, C. Li, Sylvia (1998). "Quantification of nonlinear soil response for the Loma Prieta, Northridge, and Imperial Valley California earthquakes." Proceedings of The Second International Symposium on The effects of Surface Geology on Seismic Motion Seismic Motion/Yokohama/Japan/1-3 December 1998, Irikura, Kudo, Okada & Sasatani (eds.), 1137—1143.

28. Silva, W.J., N. Abrahamson, G. Toro and C. Costantino. (1996). "Description and validation of the stochastic ground motion model." Report Submitted to Brookhaven National Laboratory, Associated Universities, Inc. Upton, New York 11973, Contract No. 770573.

29. Silva, W.J., and R. Darragh, (1995). "Engineering characterization of earthquake strong ground motion recorded at rock sites." Palo Alto, Calif.: Electric Power Research Institute, Final Report RP 2556-48.

30. Silva, W.J. (1992). "Factors controlling strong ground motions and their associated uncertainties." Seismic and Dynamic Analysis and Design Considerations for High Level Nuclear Waste Repositories, ASCE 132-161.

31. Wald, D.J. and T. I. Allen (2007). "Topographic slope as a proxy for seismic site conditions and amplification." Bull. Seism. Soc. Am., 97(5), 1379-1395.

32. Walling, M., W. Silva and N. Abrahamson (2008). “Nonlinear site amplification factors for constraining the NGA models.” Earthquake Spectra, 24(1), 243-255.

33. Wills, C. J. and K. B. Clahan (2006). “Developing a map of geologically defined site-condition categories for California.” Bull. Seism. Soc. Am., 96(4A), 1483-1501.

34. Wills, C.J. and W.J. Silva (1998). "Shear-wave velocity characteristics of geologic units in California." Earthquake Spectra, 14(3), 533-556.

35. Yong, A., S.E. Hough, A. Braverman, and J. Iwahashi (2010). “A terrain-based Vs30 estimation map of the contiguous United States.” Seism. Res. Letters, 81(2), 294.

Table B-2
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-3
Generic 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-4
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-5
Site Independent Relative Weights and Epistemic Uncertainty

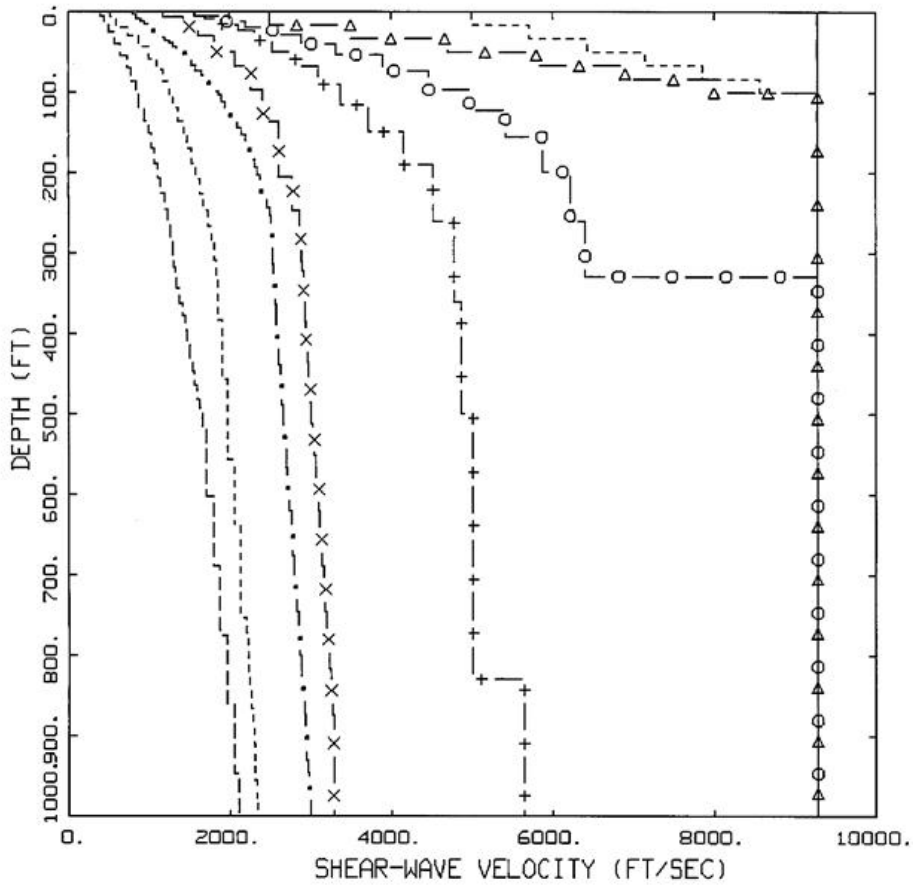
Parameter	Relative Weight	σ_μ
Mean Base-Case Profile	0.40	0.35
Lower-Range	0.30	
Upper-Range	0.30	
Mean Base-Case Kappa	0.40	0.40
Lower-Range	0.30	
Upper-Range	0.30	
G/Gmax and Hysteretic Damping Curves		0.15*, 0.30**
Soil		
EPRI Cohesionless Soil	0.5	
Peninsular Range	0.5	
Firm Rock		
EPRI Rock	0.5	
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-6
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**



TEMPLATE VELOCITY PROFILES

LEGEND

- S-WAVE: 190 M/SEC
- S-WAVE: 270 M/SEC
- S-WAVE: 400 M/SEC
- S-WAVE: 560 M/SEC
- S-WAVE: 760 M/SEC, WNA REFERENCE ROCK
- S-WAVE: 900 M/SEC
- S-WAVE:1364 M/SEC (SOFT ROCK)
- S-WAVE:2032 M/SEC (FIRM ROCK)
- S-WAVE:2830 M/SEC (HARD ROCK), CENA REFERENCE SITE

Figure B-1
Template Shear Wave Velocity Profiles for Soils, Soft Rock, and Firm Rock. Rock Profiles Include Shallow Weathered Zone.

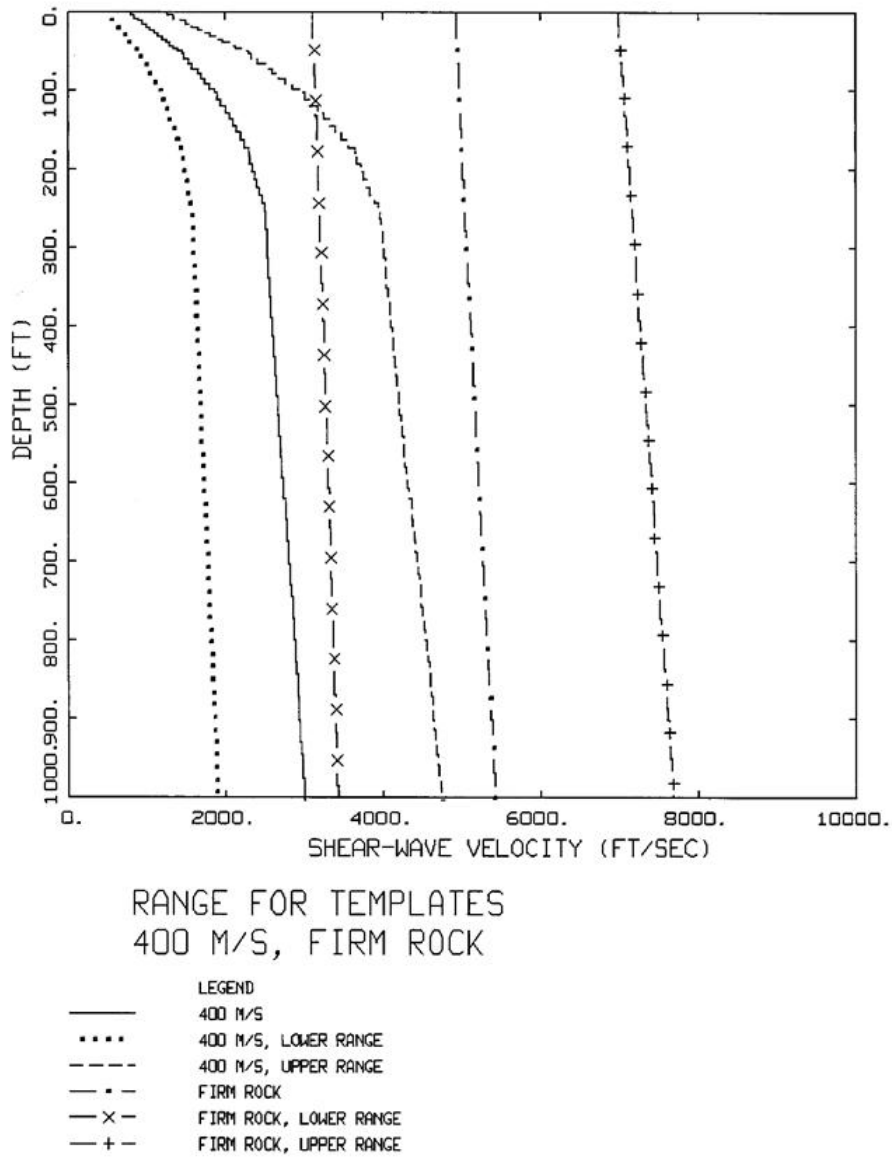


Figure B-2
Illustration of the Upper Range and Lower Range Base-Case Profiles Adopted to Accommodate Epistemic Uncertainty of the Mean Base-Case.

400m/s mean base-case (Figure xx.1) and firm rock (weathered zone removed) taken as 5,000 ft/s at the surface, empirical gradient adopted from Fukushima et al. (1995). Range is a maximum and reflects cases with few or no measured shear-wave velocities.

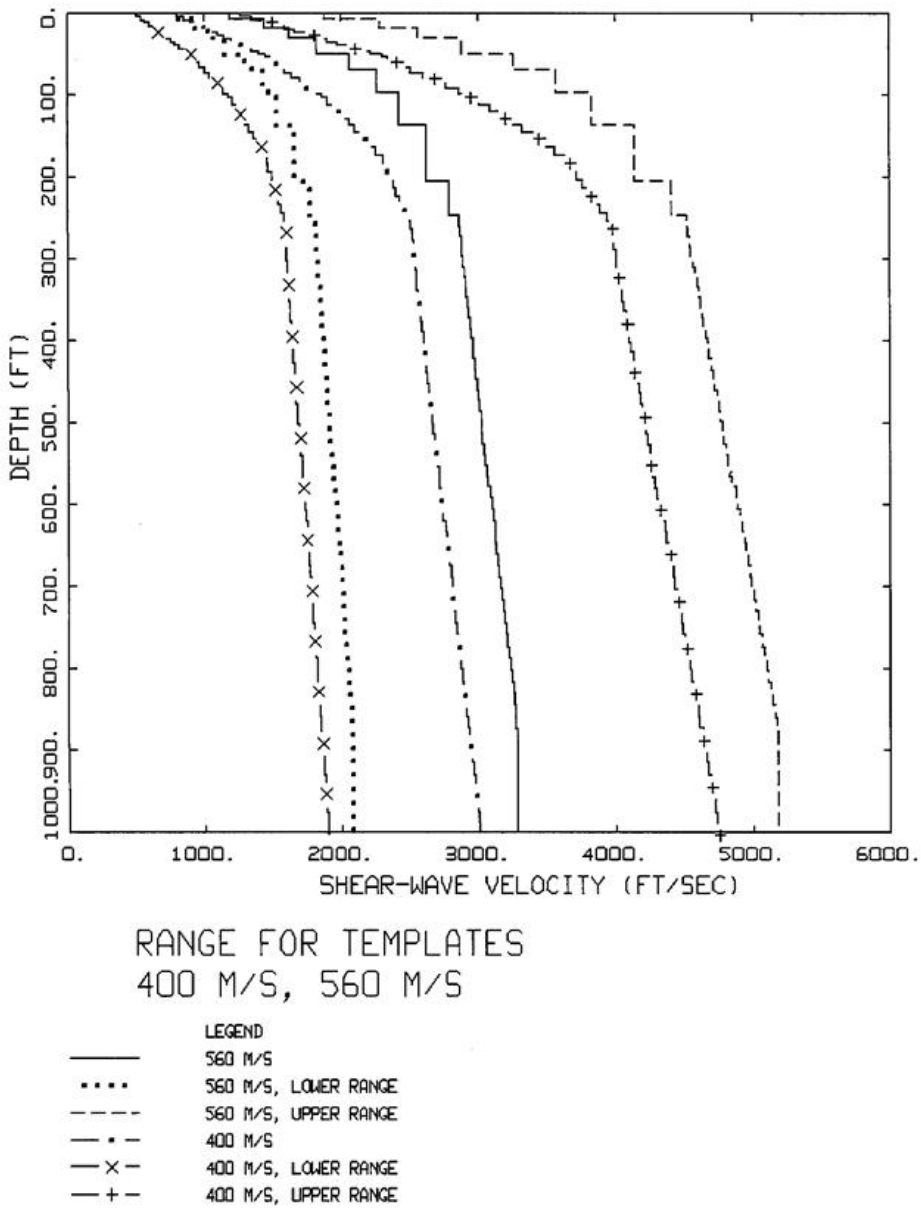


Figure B-3
Similar to Figure xx.2a, but Illustrates Maximum Ranges for Soil Templates 400m/s and 560m/s (Figure xx.1).

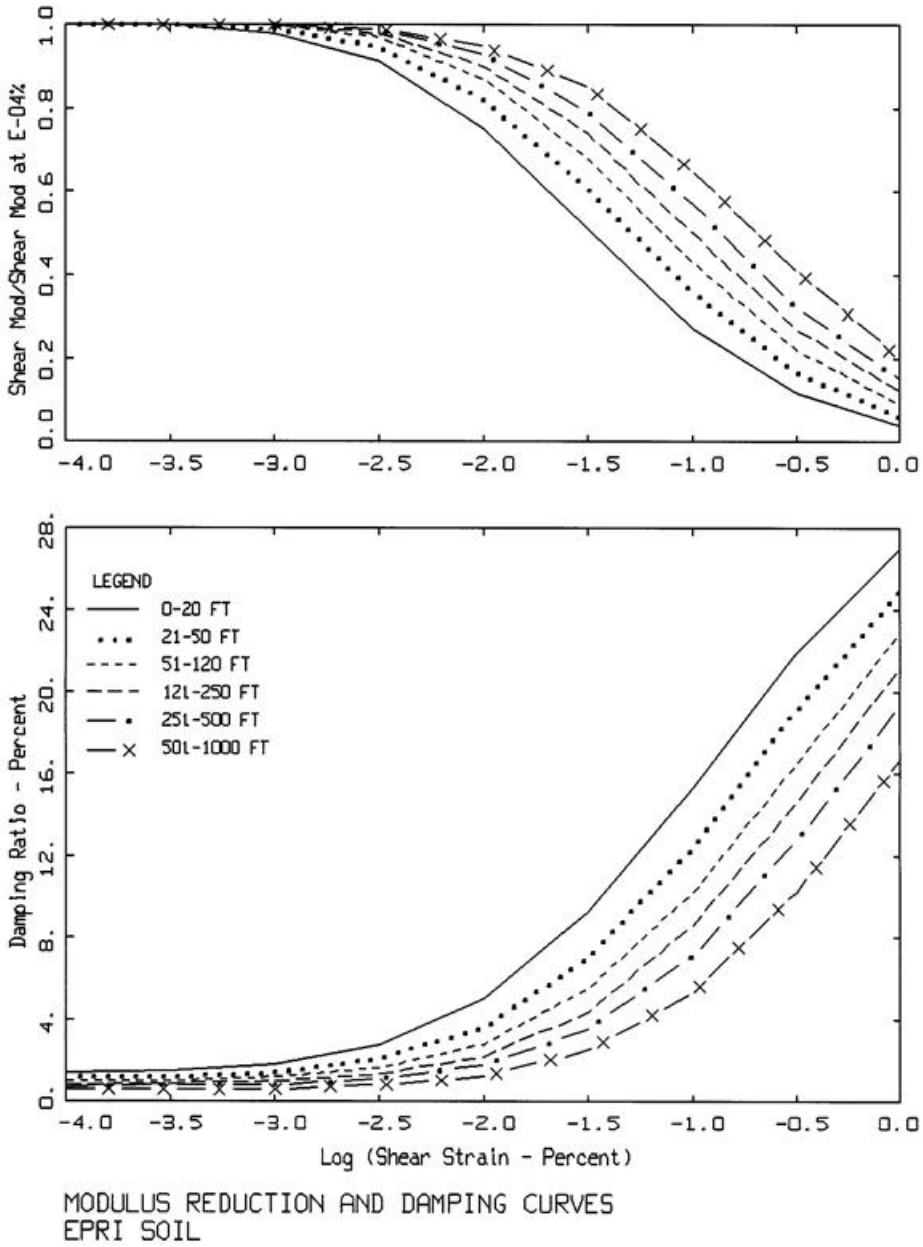


Figure B-4
Generic G/G_{max} and Hysteretic Damping Curves for Cohesionless Soil (EPRI, 1993).

Damping limited to 15% in application.

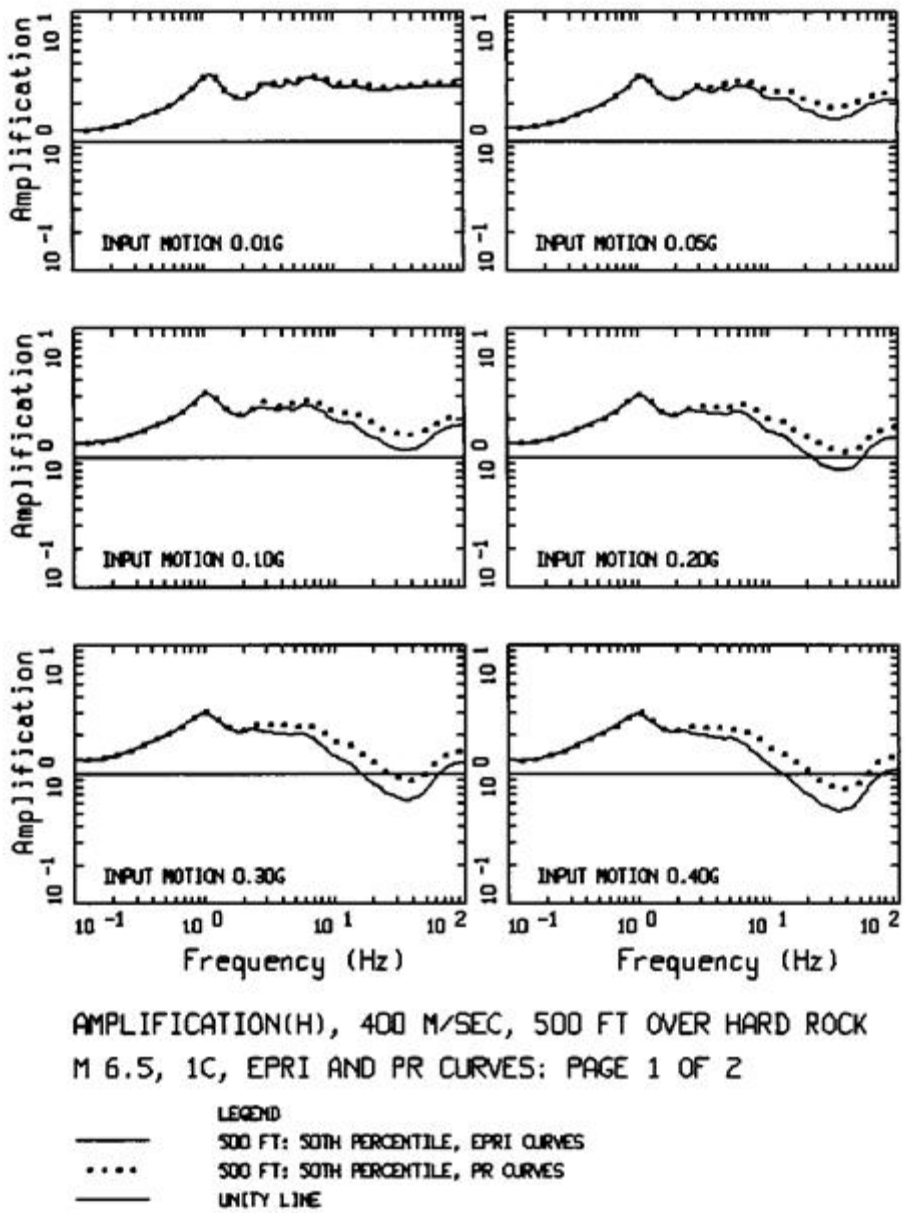
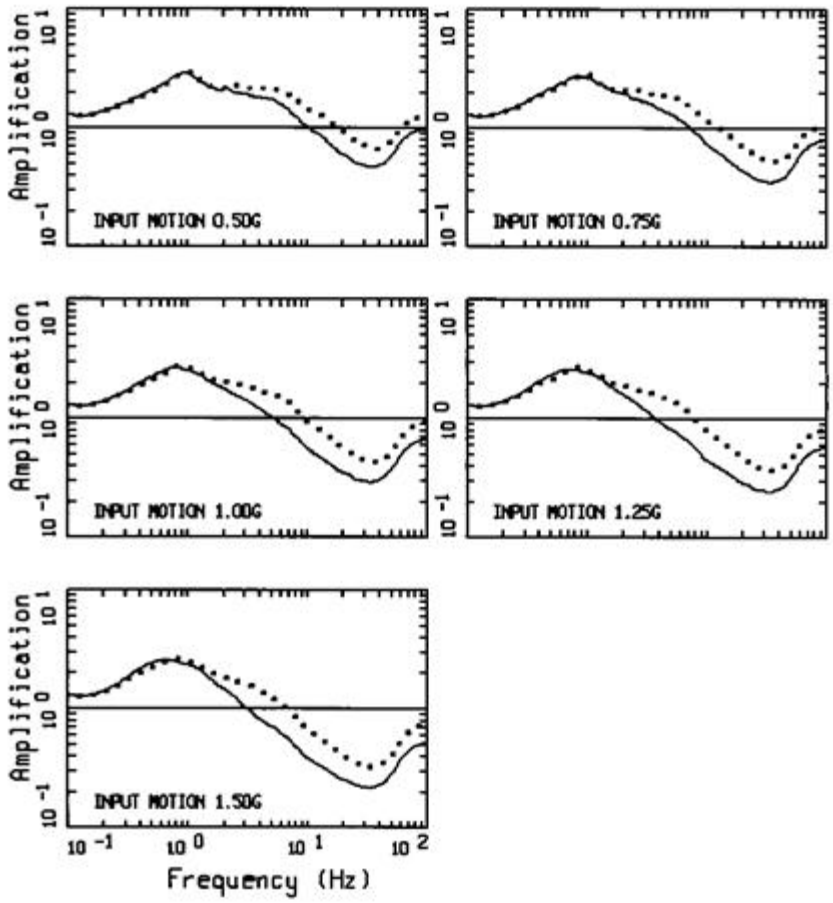


Figure B-5
Comparison of Amplification (5% damped PSa) Computed using EPRI (1993) (Figure xx.3) and Peninsular Range (Silva et al., 1996) G/Gmax and Hysteretic Damping Curves, Profile 400m/s (Figure xx.2), and the Single-Corner Source Model (Table xx.1a).

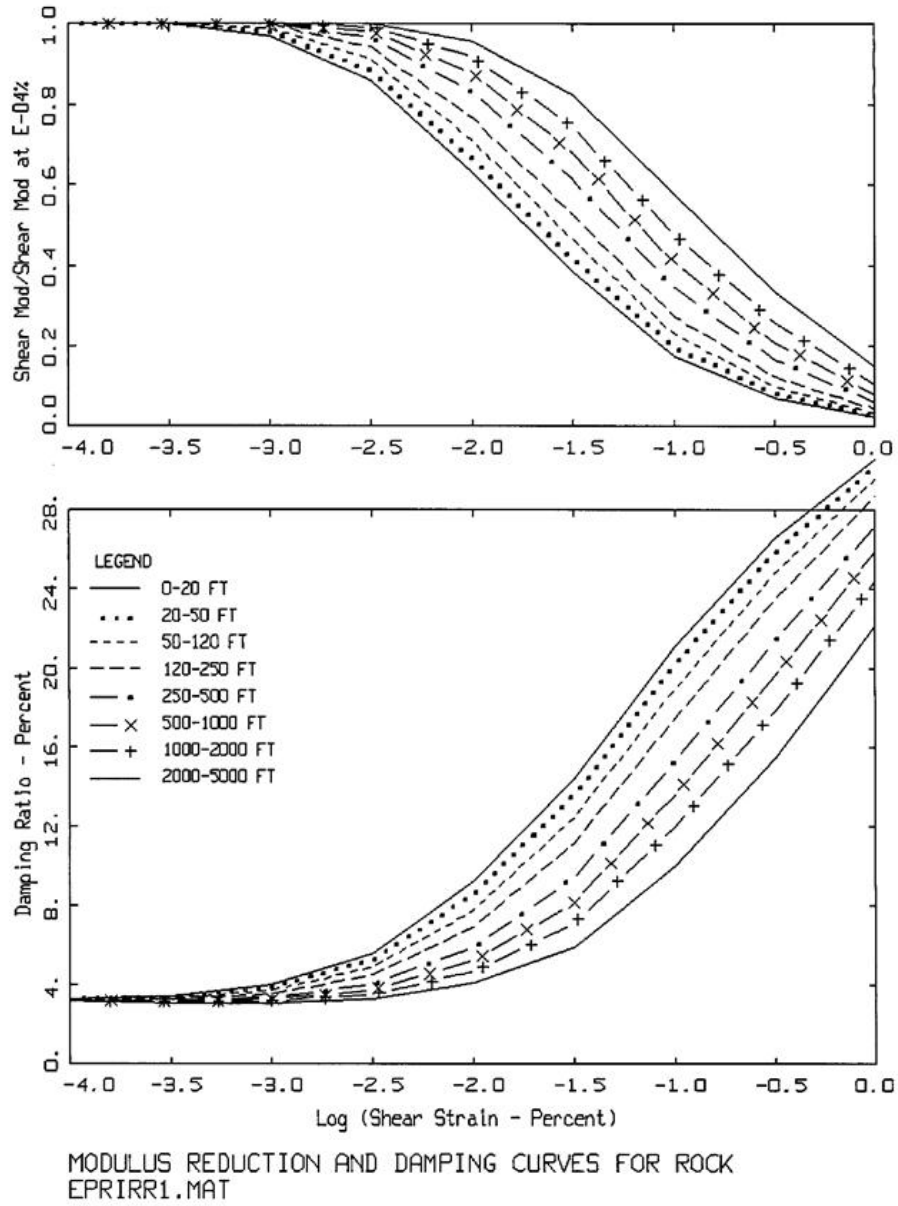
Reference rock loading levels of 0.01g to 1.50g.



AMPLIFICATION(H), 400 M/SEC, 500 FT OVER HARD ROCK
M 6.5, 1C, EPRI AND PR CURVES: PAGE 2 OF 2

LEGEND
500 FT: 50TH PERCENTILE, EPRI CURVES
500 FT: 50TH PERCENTILE, PR CURVES
UNITY LINE

Figure B-5 (continued)



MODULUS REDUCTION AND DAMPING CURVES FOR ROCK
EPRIRR1.MAT

Figure B-6
Generic G/G_{max} and Hysteretic Damping Curves for Firm Rock (developed by Dr. Robert Pyke).

Damping limited to 15% in application.

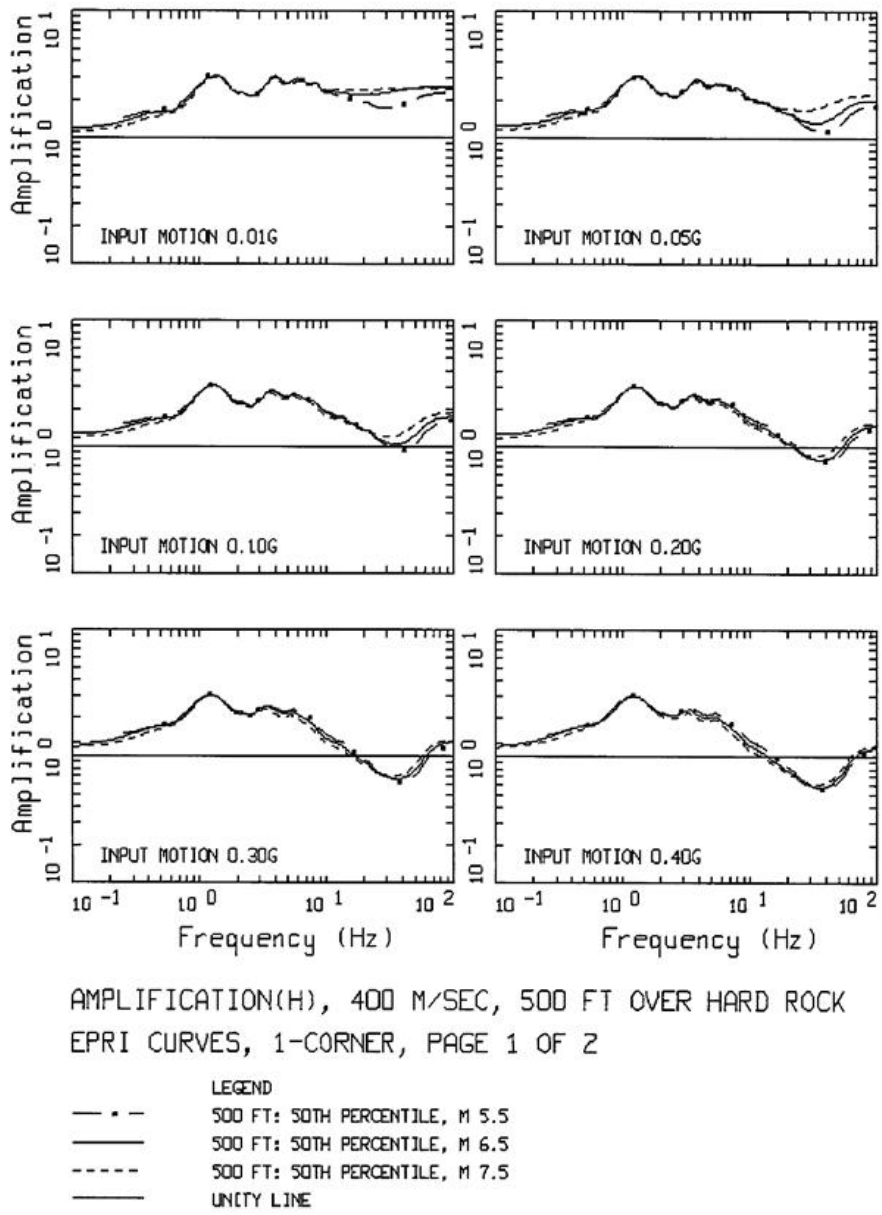
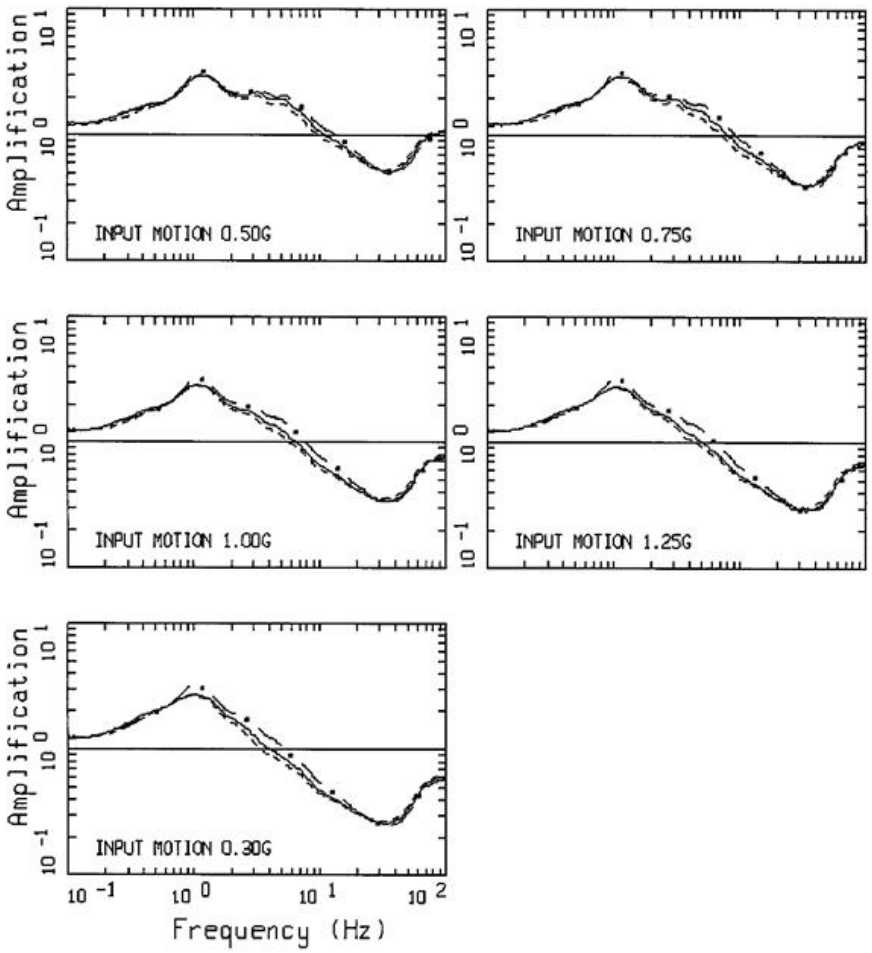


Figure B-7
Comparison of Amplification (5% damped PS_a) Computed Using the Single-Corner Source Models (Table x.1a) for Stiff Soil Profile 400m/s (Figure xx.1) and EPRI (1993) G/G_{max} and Hysteretic Damping Curves (Figure xx.4) using M 5.5, 6.5, and 7.5.

Reference rock loading levels of 0.01g to 1.50g (Table xx.1).



AMPLIFICATION(H), 400 M/SEC, 500 FT OVER HARD ROCK
EPRI CURVES, 1-CORNER, PAGE 2 OF 2

LEGEND
500 FT: 50TH PERCENTILE, M 5.5
500 FT: 50TH PERCENTILE, M 6.5
500 FT: 50TH PERCENTILE, M 7.5
UNITY LINE

Figure B-7 (Continued)

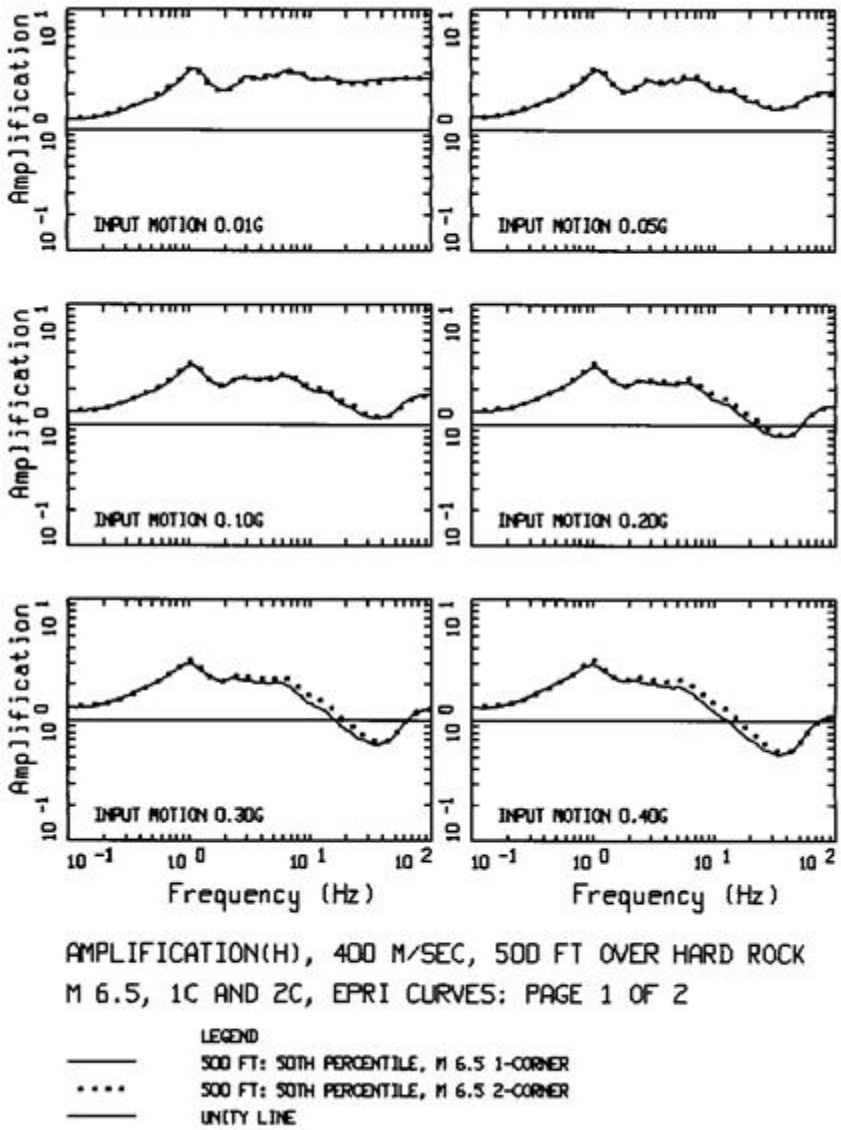
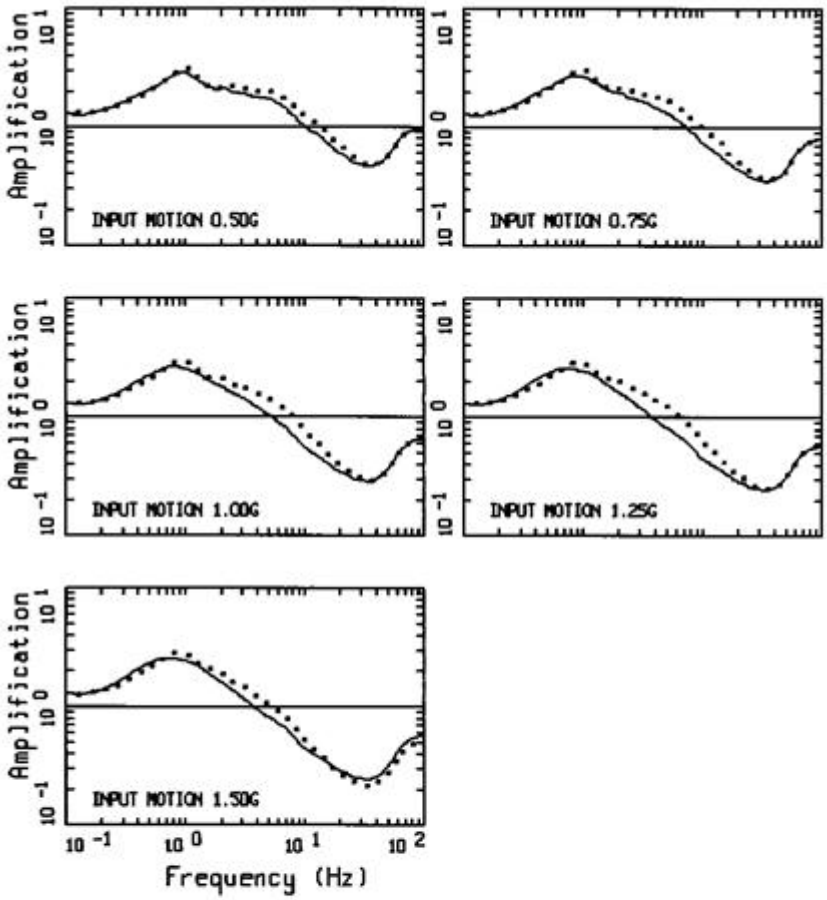


Figure B-8
Comparison of Amplification (5% damped PS_a) Computed Using the Single- and Double-Corner Source Models (Table x.1a) for Stiff Soil Profile 400m/s (Figure xx.1) and EPRI (1993) G/G_{max} and Hysteretic Damping Curves (Figure xx.4).

Reference rock loading levels of 0.01g to 1.50g (Table xx.1).



AMPLIFICATION(H), 400 M/SEC, 500 FT OVER HARD ROCK
M 6.5, 1C AND 2C, EPRI CURVES: PAGE 1 OF 2

LEGEND
500 FT: 50TH PERCENTILE, M 6.5 1-CORNER
500 FT: 50TH PERCENTILE, M 6.5 2-CORNER
UNITY LINE

Figure B-8 (Continued)