Seismic Hazard and Screening Report (Example Submittal for CEUS Site)

1.0 Introduction

Comment [C1]: Our focus on developing this example is primarily on Sections 2, 3, and 4, which describe the hazard evaluation and screening process. The example is for a plant that closely follows the SPID and the site response method described in Appendix B.

Following the accident at the Fukushima Daiichi nuclear power plant resulting from the March 11, 2011, Great Tohoku Earthquake and subsequent tsunami, the NRC Commission established a Near Term Task Force (NTTF) to conduct a systematic review of NRC processes and regulations and to determine if the agency should make additional improvements to its regulatory system. The NTTF developed a set of recommendations intended to clarify and strengthen the regulatory framework for protection against natural phenomena. Subsequently, the NRC issued a 50.54(f) letter that requests information to assure that these recommendations are addressed by all U.S. nuclear power plants. The 50.54(f) letter requests that licensees and holders of construction permits under 10 CFR Part 50 reevaluate the seismic hazards at their sites against present-day NRC requirements. Depending on the comparison between the reevaluated 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 a seismic margin assessment (SMA). Based upon this information, the NRC staff will determine whether additional regulatory actions are necessary.

This report provides the information requested in items (1) through (7) of the "Requested Information" section and Attachment 1 of the 50.54(f) letter pertaining to NTTF Recommendation 2.1 for the *Plant*, located in *County, State*. In providing this information, *Licensee* followed the guidance provided in the *Seismic Evaluation Guidance: Screening, Prioritization, and Implementation Details (SPID) for the Resolution of Fukushima Near-Term Task Force Recommendation 2.1: Seismic* (EPRI 1025287, 2012). The Augmented Approach, Seismic Evaluation Guidance: Augmented Approach for the Resolution of Fukushima Near-Term Task Force Recommendation 2.1: Seismic (EPRI 3002000704, 2013), has been developed as the process for evaluating critical plant equipment prior to performing the complete plant seismic risk evaluations.

The original geologic and seismic siting investigations for *Plant* were performed in accordance with Appendix A to 10 CFR Part 100 and meet General Design Criterion 2 in Appendix A to 10 CFR Part 50. The Safe Shutdown Earthquake Ground Motion (SSE) was developed in accordance with Appendix A to 10 CFR Part 100 and used for the design of seismic Category I systems, structures and components.

In response to the 50.54(f) letter and following the guidance provided in the SPID (EPRI 1025287, 2012), a seismic hazard reevaluation for *Plant* was performed to develop a Ground Motion Response Spectrum (GMRS) for comparison with the SSE. In the 1 to 10 Hz part of the response spectrum, the GMRS exceeds the SSE and above 10 Hz the GMRS also exceeds the SSE. As a result, a seismic risk evaluation for *Plant* will be performed in accordance with the schedule for central and eastern United States (CEUS) nuclear plants provided in the April 9, 2013, letter from industry to the NRC.

2.0 Seismic Hazard Reevaluation

Plant is located approximately *X* miles *southeast* of *City*, *State*, adjacent to the *water source*. *Plant* is in the Triassic Lowland section of the Piedmont physiographic province. The area is within the Newark-Gettysburg Basin, which is underlain by red sandstones, shales and siltstones of the Triassic Newark Group. These sedimentary basin deposits are gently tilted and warped, and are cut by diabase dikes and sills and by minor faulting. Some minor Triassic faults occur near the site; however, detailed studies carried out during the siting investigation for *Plant* show that they are not capable faults. The principle plant structures are founded on competent bedrock, about 100 feet above the river. Bedrock at the site, which consists of Triassic siltstone, sandstone, and shale, is moderately to closely jointed.

Earthquake activity in historic time within 200 miles of the plant site has been moderate. Sources of major earthquakes in the central and eastern United States (CEUS) are distant, and have not had an appreciable effect at the site. The original investigation of historical seismic activity in the region indicated that a design intensity of VII (Modified Mercalli Scale) is adequately conservative for the site. *Licensee* determined that Intensity VII corresponds to a peak ground acceleration of 0.13 g, which was increased to 0.15 g for the SSE.

2.1 Regional and Local Geology

The site is located in the Triassic Lowland section of the Piedmont physiographic province. The northeast-southwest trending Piedmont province is an eroded plateau of low relief and rolling topography. The lowland section of the Piedmont province, in which *Plant* is located, is north and west of the Piedmont uplands and is formed largely on shales and sandstones of Triassic-age. The dominant structural feature in the region surrounding the site is the Appalachian Orogenic Belt. The part of the Appalachian Piedmont in Pennsylvania, New Jersey, and Maryland is typified by the presence of several Triassic basins such as the Culpeper, Gettysburg, and Newark Basins.

Plant is located approximately *X* miles *southeast* of *City*, *State*, adjacent to the *water source*. The principal plant structures are located in a broad ridge, approximately 100 feet above the river. Bedrock, encountered at shallow depths, consists predominantly of red siltstone, sandstone, and shale of late Triassic age. The soils are residual, derived from the weathering of the underlying bedrock. Minor Triassic-age faults, inactive since Middle Mesozoic time, occur to the west and south of the construction area. Fracture zones with a few inches of offset were encountered in the excavation. However, they are not significant to the plant structures.

2.2 Probabilistic Seismic Hazard Analysis

2.2.1 Probabilistic Seismic Hazard Analysis Results

In accordance with the 50.54(f) letter and following the guidance in the SPID (EPRI 1025287, 2012), a probabilistic seismic hazard analysis (PSHA) was completed using the recently developed Central and Eastern United States Seismic Source Characterization (CEUS-SSC) for Nuclear Facilities (EPRI 1021097 and NUREG-2115, 2012) together with the updated EPRI Ground-Motion Model (GMM) for the CEUS (EPRI 3002000717, 2004, 2006, 2013). For the PSHA, a minimum moment magnitude cutoff of 5.0 was used, as specified in the 50.54(f) letter.

For the PSHA, the CEUS-SSC background seismic source zones out to a distance of 200 miles (320 km) around the site were included. For the large magnitude sources (Repeated Large Magnitude Earthquake or RLME) modeled for the CEUS-SSC, the Charlevoix and Charleston sources, as they lie within 1,000 km of the site, were included in the PSHA. For each of the CEUS-SSC sources, the mid-continent version of the updated CEUS EPRI GMM was used.

2.2.2 Base Rock Seismic Hazard Curves

Consistent with the SPID (EPRI 1025287, 2012), base rock seismic hazard curves are not provided as the site amplification approach referred to as Method 3 has been used. Seismic hazard curves are shown below in Section 3 at the SSE control point elevation.

2.3 Site Response Evaluation

Following the guidance contained in Seismic Enclosure 1 of the 3/12/2012 50.54(f) Request for Information and in the SPID (EPRI 1025287, 2012) for nuclear power plant sites that are not sited on hard rock (defined as 2.83 km/sec), a site response analysis was performed for *Plant*.

2.3.1 Description of Subsurface Material

Comment [C2]: Sections 2.3.1 and 2.3.2 (with the exception of Section 2.3.2.3 on kappa) are to be provided in September.

Bedrock at the site consists of well-indurated Triassic sandstones, siltstones, and shales that extend to a depth of several thousand feet. Bedrock is overlain by from 0-40 feet of residual soil, developed in situ by weathering and decomposition of the parent rock. The soil grades into weathered rock, then into fresh, unweathered rock; no clearly defined boundary exists between soil and weathered rock and between weathered and un-weathered rock. ~~The seismic Category I reactor and diesel generator buildings, as well as the turbine and radwaste buildings, are founded on bedrock.~~ Bedrock strata of the Brunswick lithofacies underlie most of the site and consist of siltstone, sandstone, and shale. Table 2.3.1-1 provides a brief description of the subsurface material in terms of the geologic units and layer thicknesses.

Comment [C3]: Locations of Cat 1structure foundations are not needed here. Covered as part of control point elevation discussion.

Table 2.3.1-1. Regional geologic profile and estimated layer thicknesses for the *site*.

Comment [C4]: Robin to provide Example Table that combines Tables 2.3.1-1 and Table 2.3.2-2.

Period	Formation	Material	Thickness ft	Thickness m
Triassic	Brunswick	Soft Red Shale, Siltstones & Sandstones	7000	2134
	Lockatong	Shale	1500	457
	Hammer Creek	Conglomerates & Sandstone	6500	1981

	Stockton	Sandstone, Conglomerate, Red shale, and Red Siltstone	3000	914
Ordovician	Connestorga	Limestone	1000	305
	Beekmantown	Limestone	2000	610
Cambrian	Conococheague	Limestone	900	274
	Elbrook	Limestone	500	152
	Ledger	Dolomite	800	244
	Kinzers	Limestone?	200	61
	Vintage	Dolomite	450	137
	Antietam	Quartzite	150	46
	Harpers	Phyllite	1250	381
	Chickies	Quartzite	270	82
	Hardyston	Quartzite	150	46
	Wissahickon	Gneiss	9000	2743

2.3.2 Development of Base Case Profiles and Nonlinear Material Properties

Seismic refraction surveys ranging in length from about 400 to 700 ft (120 to 215 m) were performed during the original site investigation to determine the compressional wave (P-wave) and shear wave (S-wave) velocities for the site foundation materials. P-wave velocities in the rock range from about 7700 ft/sec to 20,000 ft/sec, with an average of about 12,500 ft/sec. S-wave velocities range from 5800 ft/sec to 6100 ft/sec, with an average of about 5950 ft/sec. Table 2.3.2-1 provides the S-wave velocities determined from the seismic refraction survey. The depths indicated in the table are depths below original site grade.

Table 2.3.2-1. Shear wave velocities determined for the *Plant site* from refraction survey

Depth ft	Shear Wave Velocity ft/sec
20 - 110	5800 – 6100
110 - 130	3150
130 - 200	5800 - 6100

Using these measured S-wave velocities, the information on the regional geologic profile (Table 2.3.1-1) and the SPID guidance (EPRI 1025287, 2012), three base-case S-wave velocity profiles were developed for the *Plant* site. The first, best-estimate case is based explicitly on the measured near-surface shear-wave velocities. Since the *Plant* site lacks detailed velocity measurements over the necessary depth range, alternative profiles were developed to represent the uncertainty in velocity with depth.

In developing the initial base case shear-wave velocity profile, the foundation level shear-wave velocity (elevation of +200 ft above msl) was assumed to be equal to the average velocity measured in the upper depth interval (20-110 ft) of 5950 ft/sec (1815 m/sec) within the Brunswick Formation. Consistent with the SPID guidance (EPRI 1025287, 2012), the shear-

wave velocity was assumed to increase linearly through the sedimentary rock materials at a rate of 0.5 ft/sec/ft (0.5 m/sec/m). Because the shear-wave velocity of 5950 ft/sec is at the higher end of the range of typical shear wave velocities for these types of sedimentary rock, this base case velocity profile is used for both the median and upper range base cases. Due to limited shear wave velocity data, a standard deviation of 0.35 is used to estimate the lower range shear wave velocity profile in the sedimentary rock. The lower range shear wave velocity at the foundation level was set equal to 3775 ft/sec and was then increased at a rate of 0.5 ft/sec/ft.

Although the velocity increases linearly with depth, a constant average shear wave velocity for each layer is used in the analyses. The shear wave velocity profile for the base cases is provided in Table 2.3.2-2 and shown in Figure 2.3.2-1.

Table 2.3.2-2. Base-case shear-wave velocity profiles used for site response analyses

Formation	Material	Lower Range (ft/sec)	Median (ft/sec)	Upper Range (ft/sec)	Layer Thickness (ft)	Top Elevation (ft msl)
Brunswick	Shale, Siltstone & Sandstone	3775	5950	5950	90	200
		2000	3150	3150	20	110
		3775	5950	5950	70	90
		3855	6030	6030	320	20
		4685	6860	6860	3000	-300

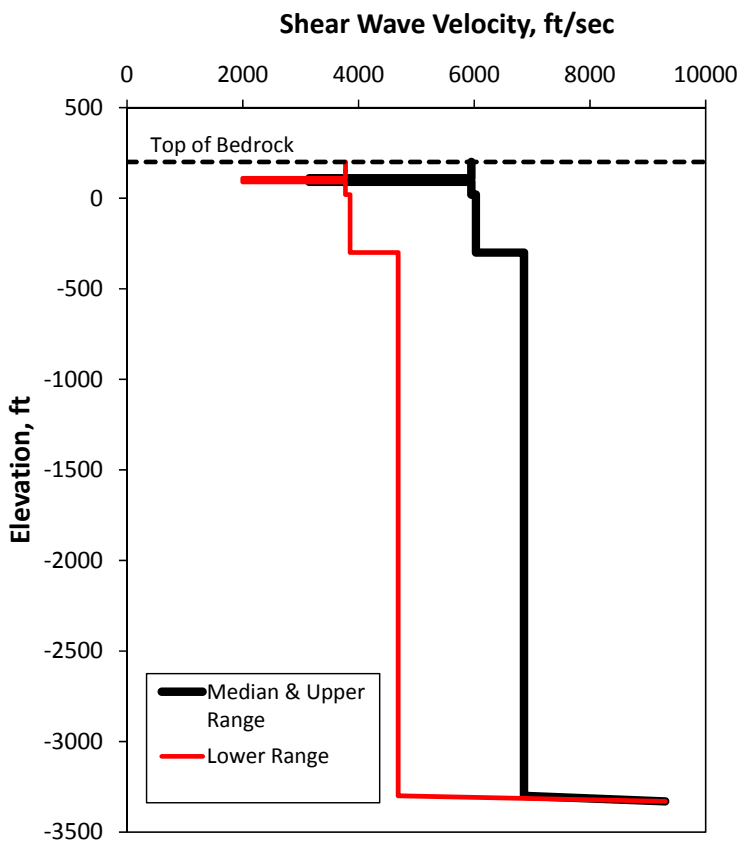

Figure 2.3.2-1. Shear wave velocity profile used in site response calculations for *Plant*

The depth to hard rock for both base cases is defined as the depth where the shear wave velocity reaches a value of 9300 ft/sec (2830 m/s). Assuming a linear increase in shear-wave velocity with depth of 0.5 ft/sec/ft, the estimated depth to the 9300 ft/sec hard rock interface is estimated to lie at a depth of ~7000 ft (~2135 m) for the median & upper range profile and 11000 ft (~3350 m) for the lower range profile. Consistent with the guidance in the SPID (EPRI 1025287), the depth to hard rock can be modeled at a shallower depth provided reasonable site amplification values can be obtained for spectral frequencies of 0.5 Hz and higher. Hence, the depth to the 9300 ft/sec hard rock interface was defined as 3500 ft (1067 m) for the purposes of estimating site response at the *Plant* site. Because the depth to hard rock is very large (>7000 ft) at this site, no additional uncertainty in this parameter was incorporated in the analyses.

2.3.2.2 Shear Modulus and Damping Curves

No site-specific dynamic material properties were determined in the initial siting of *Plant*. Rock Quality Designation (RQD) values from cores in the foundation area reported in the *Plant* FSAR are generally in the range of 90 to 100% as demonstrated in Boring 166. One boring (Boring 229) showed significantly lower RQD values, indicating poor rock quality. Based on these RQD values, the rock material over the upper 500 ft (150 m) was assumed to have behavior that could be modeled as either linear or non-linear.

To represent this potential for either linear or non-linear behavior in the upper 500 ft of sedimentary rock present at the *Plant* site, two sets of shear modulus degradation and damping curves were used in the present analyses. Consistent with the SPID (EPRI 1025287, 2012), the non-linear Peninsular curves are considered to be appropriate to represent the softest (i.e., most non-linear) response likely in the materials at this site and linear curves represent the stiffest response. When linear curves are used, the low strain damping from the Peninsular curves (1.06% for 0 – 50 ft and 0.6% for 50 – 500 ft) are used as the constant damping values in the upper 500 ft.

Linear curves with 0% damping are used below a depth of 500 ft and kappa is used to account for damping below 500 ft.

2.3.2.3 Kappa

Because two base case profiles (median & upper range and the lower range) have been defined for the *Plant* site, two sets of kappa values are required for the site response analyses. The kappa estimate is based on the material below a depth of 500 ft (Elevation -329 ft) below which the site response is considered to be linear for all analyses. Damping above a depth of 500 ft is accounted for explicitly in the damping curves as discussed above in Section 2.3.2.1.

Kappa was determined using Section B-5.1.3.1 of the SPID (EPRI 1025287, 2012) for a firm CEUS rock site. Kappa for a firm rock site is estimated from the average S-wave velocity over the upper 100 ft (V_{s100}) of the subsurface profile. For *Plant* site, the median & upper range V_{s100} (over the depth range of 500 – 600 ft) is 6135 ft/sec and the lower range V_{s100} is 3935 ft/sec. Using these two average velocities, the kappa for the sedimentary rock is 0.0120 sec for the median/upper range profile and 0.0193 sec for the lower range profile. As specified in Section B-5.1.3.2 of the SPID (EPRI 1025287, 2012), a natural log standard deviation of 0.4 was used to estimate the upper and lower range values of kappa. Table 2.3.2-3 summarizes the kappa values used for the site response analysis.

Table 2.3.2-3. Kappa Values Used for Site Response Analyses

Velocity Profile	Lower (sec)	Median (sec)	Upper (sec)
Lower Range	0.0115	0.0193	0.0325
Median and Upper Range	0.0071	0.0120	0.0202

2.3.3 Randomization of Base Case Profiles

Comment [C5]: Subsection on kappa was originally to be placed in this Section but it has been moved up to Section 2.3.2 as it is part of the subsurface profile.

To account for the aleatory variability in material properties that is expected to occur across a site at the scale of a typical nuclear facility, variability in the assumed shear-wave velocity profiles has been incorporated in the site response calculations. For the *Plant* site, random shear wave velocity profiles were developed from the base case profiles as shown in Figure 2.3.2-1. Thirty random velocity profiles were generated for each base case profile. These random velocity profiles were generated using a natural log standard deviation of 0.25 over the upper 50 ft and 0.15 below that depth. As specified in the SPID (EPRI 1025287, 2012), correlation of shear wave velocity between layers was modeled using the USGS A correlation model. In the correlation model, a limit of +/- 2 standard deviations about the median value in each layer was assumed for the limits on random velocity fluctuations. All random velocities were limited to be less than or equal to 9830 ft/sec.

2.3.4 Input Spectra

Consistent with the guidance in Appendix B of the SPID (EPRI 1025287, 2012), input Fourier amplitude spectra were defined for a single representative earthquake magnitude using two different assumptions regarding the shape of the seismic source spectrum (single-corner and double-corner). A range of 11 different input amplitudes (peak ground accelerations (PGA) ranging from 0.01 to 1.5 g) were used in the site response analyses. The characteristics of the seismic source and upper crustal attenuation properties assumed for the analysis of the *Plant* site were the same as those identified in Tables B-4, B-5, B-6 and B-7of the SPID (EPRI 1025287, 2012) as appropriate for typical CEUS sites.

2.3.5 Methodology

To perform the site response analyses for the *Plant* site, a random vibration theory (RVT) approach was employed. This process utilizes a simple, efficient approach for computing site-specific amplification functions and is consistent with existing NRC guidance and the SPID (EPRI 10252872012, 2012). The guidance contained in Appendix B of the SPID (EPRI 10252872012, 2012) on incorporating epistemic uncertainty in shear-wave velocities, kappa, non-linear dynamic properties and source spectra for plants with limited at-site information was followed for the *Plant* site.

2.3.6 Amplification Functions

The results of the site response analysis consist of amplification functions which describe the amplification (or de-amplification) of rock motions as a function of frequency and input bedrock amplitude. The amplification functions are represented in terms of a median amplification value and an associated standard deviation (sigma) for each response spectral frequency and input rock amplitude. Seven spectral frequencies and 11 input rock amplitudes were used in the present analysis. Consistent with the SPID (EPRI 1025287 2012) a minimum median amplification value of 0.5 was employed in the present analysis. Figures 2.3.5-1 illustrates the

median amplification results for three of the 11 input bedrock amplitude levels. Figure 2.3.5-2 illustrates the median and +/- 1 standard deviation in the predicted amplification function for an input bedrock acceleration of 0.03g as well as the variability that arises due to the randomization in velocity profiles.

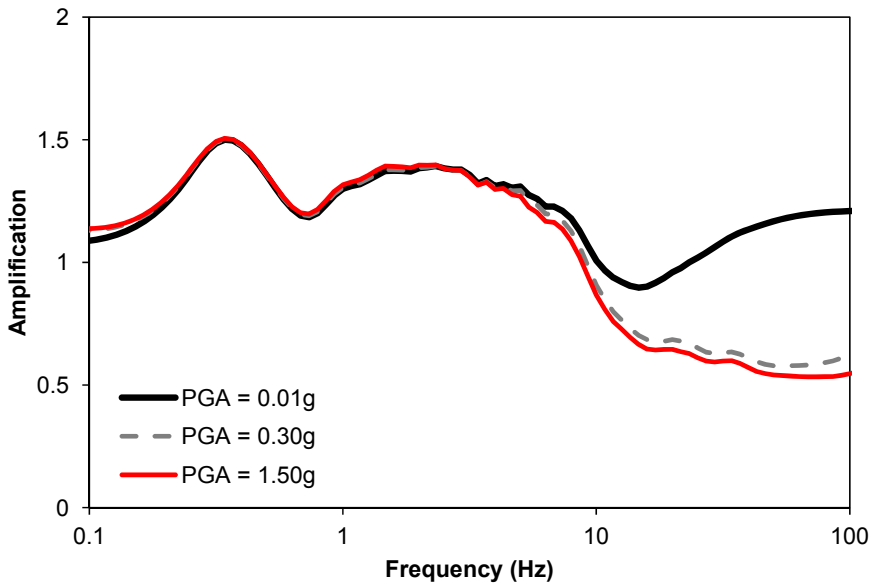

Figure 2.3.5-1. Median amplification as a function of frequency for the *Plant* site. Results for input bedrock amplitudes of 0.01, 0.3 and 1.5g are shown.

Comment [C6]: Robin to provide example figures for Amp functions. Staff expressed concern with not having Amp functions for entire frequency range instead of just seven used for GMM.

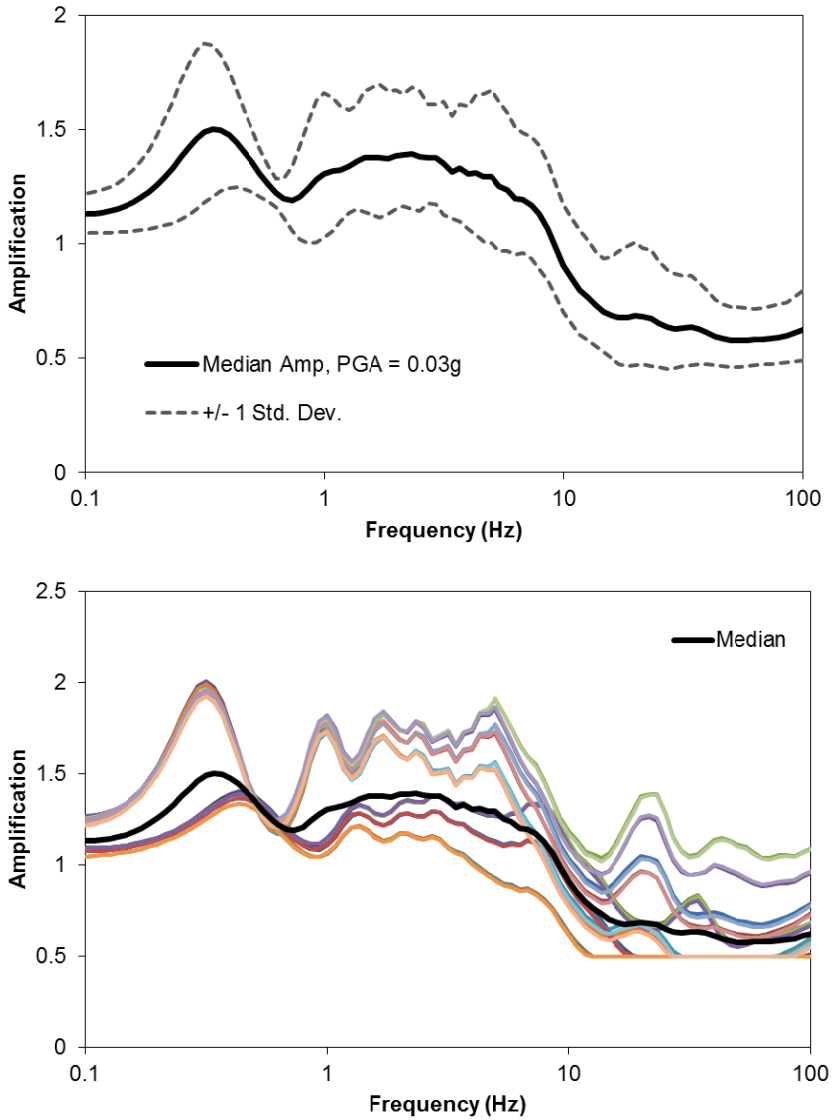

Figure 2.3.5-2. Amplification functions for the *Plant* site for an input amplitude of 0.03g (PGA). Top: Solid thick black line is median amplification and dashed black lines are +/- 1 sigma. Bottom: Colored curves are for various realizations, solid black line is median.

2.3.7 Control Point Seismic Hazard Curves

The procedure to develop probabilistic site-specific control point hazard curves used in the present analysis follows the methodology described in Section B-6.0 of the SPID (EPRI 1025287, 2012). This procedure (referred to as Method 3) computes a site-specific control point hazard curve for a broad range of spectral accelerations given the site-specific bedrock hazard curve and site-specific estimates of soil or soft-rock response and associated uncertainties. This process is repeated for each of the seven specified oscillator frequencies. The dynamic response of the materials below the control point was represented by the frequency and amplitude-dependent amplification functions (median values and standard deviations) developed and described in the previous section. The resulting control point mean hazard curves for the *Plant* site are shown in Figure 2.3.7-1 for the seven oscillator frequencies for which the GMM is defined. Tabulated values of the site response amplification functions and control point hazard curves are provided in the attached Appendix.

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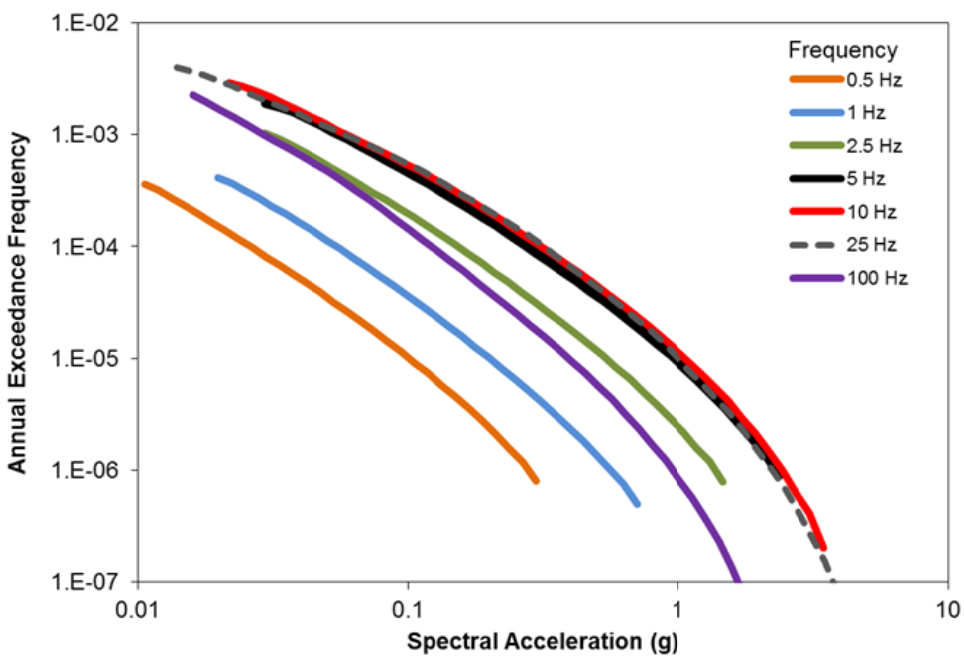

Figure 2.3.7-1. Control point mean hazard curves for oscillator frequencies of 0.5, 1, 2.5, 5, 10, 25 and 100 Hz at the *Plant* site.

2.4 Ground Motion Response Spectrum

The control point hazard curves described above have been used to develop uniform hazard spectra and the GMRS. The uniform hazard spectra were obtained through linear interpolation in log-log space to estimate the spectral acceleration at each oscillator frequency for the 1E-4 and 1E-5 per year hazard levels. Table 2.4-1 shows the uniform hazard spectra and GMRS spectral accelerations for each of the seven frequencies.

Table 2.4-1. Uniform Hazard Spectra and GMRS for *Plant*

Freq (Hz)	UHS 10^{-4} (g)	UHS 10^{-5} (g)	GMRS (g)
0.5	0.026	0.101	0.046
1.0	0.055	0.200	0.093
2.5	0.153	0.540	0.252
5.0	0.277	0.970	0.453
10.0	0.309	1.067	0.500
25.0	0.319	0.999	0.477
100.0	0.122	0.393	0.187

Comment [C7]: For this example, GMRS determined at only 7 frequencies. Submittals should provide UHS and GMRS for at least 20 frequencies.

The 1E-4 and 1E-5 uniform hazard spectra along with the design factor (DF) are used to compute the GMRS at the control point and are shown in Figure 2.4-1.

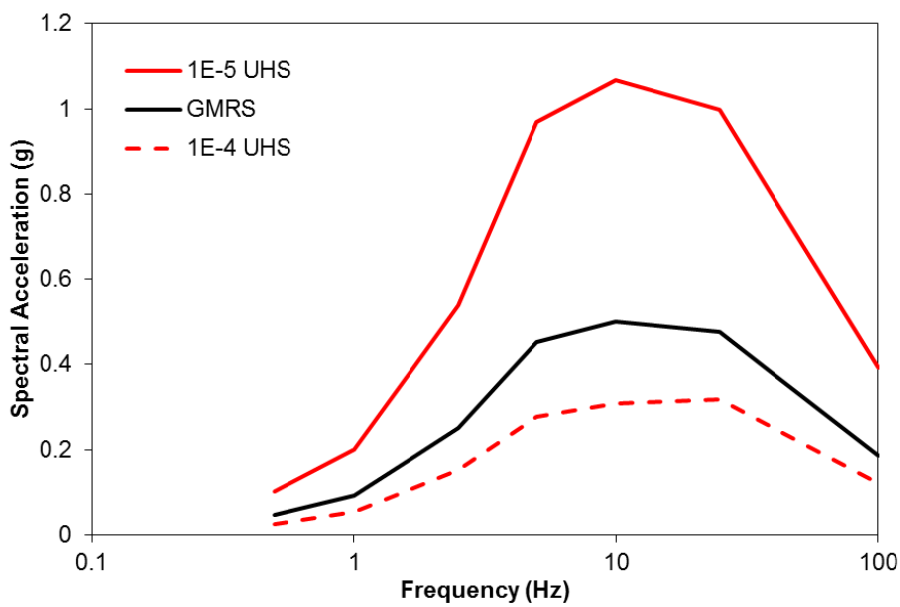

Figure 2.4-1. Plots of 1E-4 and 1E-5 uniform hazard spectra and GMRS at control point for *Plant* (5%-damped response spectra).

3.0 Safe Shutdown Earthquake Ground Motion

The SSE for *Plant* was developed through an evaluation of the maximum earthquake potential for the region surrounding the site. Considering the historic seismicity of the site region, *Licensee* determined that the maximum potential earthquake might be either an intensity VII (Modified Mercalli Scale) event along the Fall Zone at its closest approach to the site or an intensity VI event very near the site. Because of the uncertainties involved in associating regional activity with specific geologic structures, the maximum potential earthquake was specified as being equivalent to the intensity VII 1871 Wilmington, Delaware earthquake occurring near the site.

3.1 Description of Spectral Shape and Anchor Point

The SSE is defined in terms of a PGA and a design response spectrum. Considering a site intensity of VII, a PGA of 0.13 g was estimated. For additional conservatism this peak ground acceleration was increased to 0.15 g as the anchor point for the SSE. The 5% damped horizontal SSE for *Plant* is shown in comparison with the GMRS in the screening evaluation, provided below in the Section 4. Table 3.1-1 shows the spectral acceleration values as a function of frequency for the 5% damped horizontal SSE.

Table 3.1-1. SSE for *Plant*

Freq(Hz)	100/PGA	33	10	2	0.5
SA(g)	0.15	0.15	0.42	0.42	0.11

Comment [C8]: Newmark and RG 1.60 spectra can be defined at only necessary frequencies. Need more frequencies for Housner spectra.

3.2 Control Point Elevation

The SSE control point elevation is defined at the top of bedrock (the Brunswick Formation) at an elevation of 200 feet above msl.

Comment [C9]: More detail likely to be needed to justify control point elevation. See SPID Section 2.4.2

4.0 Screening Evaluation

Following completion of the seismic hazard reevaluation, as requested in the 50.54(f) letter, a screening process is needed to determine if a seismic risk evaluation is needed. The horizontal GMRS determined from the hazard reevaluation is used to characterize the amplitude of the new seismic hazard at each of the nuclear power plant sites. The screening evaluation is based upon a comparison of the GMRS with the 5% damped horizontal SSE.

4.1 GMRS and SSE Comparison

The GMRS and 5% damped horizontal SSE are shown below in Figure 4.1-1. In the 1 to 10 Hz part of the response spectrum, the GMRS exceeds the SSE and above 10 Hz the GMRS also exceeds the SSE. However, the SSE times a factor of 1.3 exceeds the GMRS in the 1 to 10 Hz part of the response spectrum.

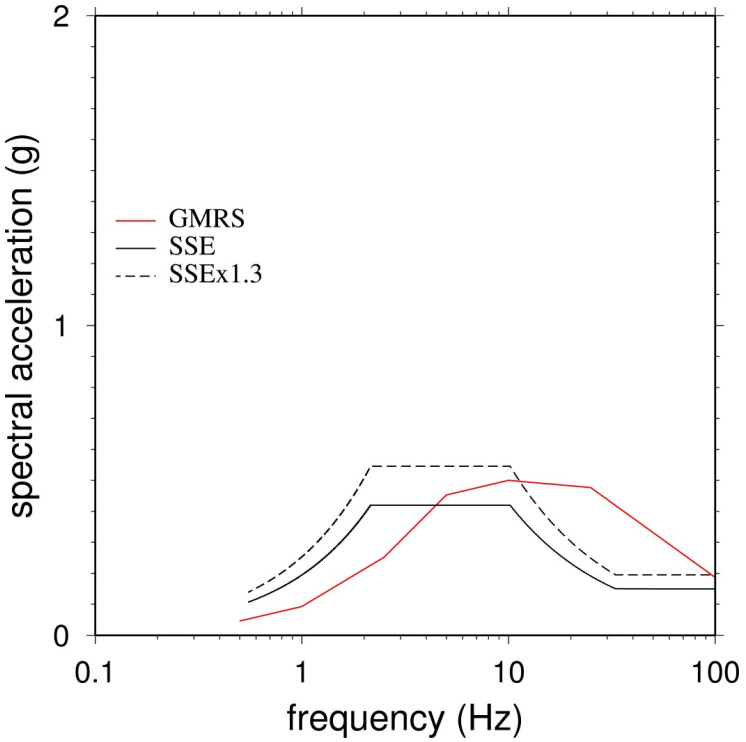

Figure 4.1-1. Comparison of the GMRS with the SSE and SSE times a factor of 1.3.

4.2 Evaluation of IPEEE Submittal

The Individual Plant Examination of External Events (IPEEE) for *Plant* was performed as a reduced-scope EPRI SMA and therefore is not eligible to be used for the screening evaluation.

4.3 GMRS and IHS Comparison

The IPEEE for *Plant* is not used for the screening evaluation.

4.4 Screening Evaluation Outcome

Based on the comparison of the SSE and GMRS, as shown and described above, a risk evaluation is needed for *Plant*. Section 6.2 of the SPID (EPRI 1025287, 2012) provides guidance as to whether an NRC SMA, as described in NRC Interim Staff Guidance (ISG 100), or an SPRA is the appropriate approach for the seismic risk evaluation.

Comment [C10]: Industry, please finish this Section as either an SMA or SPRA is appropriate.

5.0 Interim Actions

Based on the screening evaluation, the near-term evaluation, Augmented Approach (EPRI 3002000704), is needed for *Plant*. A near-term expedited seismic evaluation for *Plant* will be performed in accordance with the schedule for CEUS plants provided in the April 9, 2013, letter from industry to the NRC.

6.0 Conclusions

In accordance with the 50.54(f) request for information letter a seismic hazard and screening evaluation was performed for *Plant*. This reevaluation followed the guidance provided in the SPID (EPRI 1025287, 2012) in order to develop a GMRS for the site. The screening evaluation comparison demonstrates that the GMRS exceeds the SSE both in the 1 to 10 Hz part of the response spectrum and above 10 Hz. Based on the screening evaluation, a risk evaluation for *Plant* will be performed.

Comment [C11]: Please finish this Section describing the selected risk evaluation approach (SPRA or SMA). Add a paragraph on prioritization and subsequent schedule.

Appendix

Comment [C12]: Robin to provide description of Tables of hazard curves and amp functions.

Table A-1a. Mean Seismic Hazard Curves for Control Point Elevation at *Plant* site
Table A-1b. 5th Percentile Seismic Hazard Curves for Control Point Elevation at *Plant* site
Table A-1c. 16th Percentile Seismic Hazard Curves for Control Point Elevation at *Plant* site
Table A-1d. 84th Percentile Seismic Hazard Curves for Control Point Elevation at *Plant* site
Table A-1e. 95th Percentile Seismic Hazard Curves for Control Point Elevation at *Plant* site
Table A-2a. Amplification Functions for *Plant* Site (PGA 0.01 to 0.5 g)
Table A-2b. Amplification Functions for *Plant* Site (PGA 0.75 to 1.5 g)