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

Draft
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1.0 Introduction

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 was performed. For screening purposes, a Ground Motion Response Spectrum (GMRS) was developed.

[Optional text (for plants that screen in):
Based on the results of the screening evaluation, *a Seismic PRA [or an NRC SMA (JLD-ISG-2012-04)], a Spent Fuel Pool evaluation, and/or a High Frequency Confirmation will be performed.*]

[Optional text (for plants that screen out):
Based on the results of the screening evaluation, no further evaluations will be performed.]

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

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. 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.

Table 2.3.1-1 Geologic profile and estimated layer thicknesses for *EXAMPLE NPP*.

Depth Range (feet)	Soil/Rock Description	Density (pcf)	Shear Wave Velocity (fps)	Compressional Wave Velocity (fps)	Poisson's Ratio
0	SSE control point (at surface)	---	---	---	---
0-40	Pleistocene Silt, Sand, Clay and Gravel	120	600	2200 ± 200 2000[b]	0.32
40-240	Miocene Sand, Silt and Clay (Mostly dense to very dense sandy silts, silty sands, and slightly clayey sands)	120-135	1600	5900 ± 300 5500[b]	0.46
240-690	Oligocene medium dense Silty Sand and Clay	120-135	1800	6000±300	0.45
690-1040	Cretaceous Sand, Silt, and Clay	130-135	3400	7000 ± 400	0.35
1040-1890	Paleozoic Siltstone, Shale, and dense Shale	150-155	4400	8000 ± 500	0.28
1890+	Precambrian Igneous and Metamorphic Rock	170	10000	17,000 ± 1000	0.25

NOTES:

[a] Two sets of modulus and damping curves are used for soils: the EPRI (1993) curves (Ref. 1) and the Peninsular Range curves (Ref. 2, 3). These are weighted equally, per the SPID Appendix B. For soft or firm rock, two sets of assumptions are used: the EPRI (1993) rock curves (Ref. 1), and the assumption of linear behavior. These assumptions are weighted equally, per the SPID Appendix B.

[b] Uphole Seismic Survey

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 *Example NPP* 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. Geologic profile and estimated layer thicknesses for *EXAMPLE NPP*.

Profile 1			Profile 2			Profile 3		
thickness(ft)	depth (ft)	Vs(ft/s)	thickness(ft)	depth (ft)	Vs(ft/s)	thickness(ft)	depth (ft)	Vs(ft/s)
	0	1297		0	830		0	2036
4.0	4.0	1297	4.0	4.0	830	4.0	4.0	2036
5.0	9.0	1297	5.0	9.0	830	5.0	9.0	2036
5.0	14.0	1297	5.0	14.0	830	5.0	14.0	2036
5.0	19.0	1297	5.0	19.0	830	5.0	19.0	2036
1.0	20.0	1297	1.0	20.0	830	1.0	20.0	2036
4.0	24.0	1297	4.0	24.0	830	4.0	24.0	2036
5.0	29.0	1297	5.0	29.0	830	5.0	29.0	2036
5.0	34.0	1297	5.0	34.0	830	5.0	34.0	2036
4.4	38.4	990	4.4	38.4	634	4.4	38.4	1555

Profile 1			Profile 2			Profile 3		
thickness(ft)	depth (ft)	Vs(ft/s)	thickness(ft)	depth (ft)	Vs(ft/s)	thickness(ft)	depth (ft)	Vs(ft/s)
4.4	42.8	990	4.4	42.8	634	4.4	42.8	1555
4.4	47.2	990	4.4	47.2	634	4.4	47.2	1555
2.8	50.0	990	2.8	50.0	634	2.8	50.0	1555
1.6	51.6	990	1.6	51.6	634	1.6	51.6	1555
4.4	56.0	990	4.4	56.0	634	4.4	56.0	1555
10.3	66.3	1740	10.3	66.3	1114	10.3	66.3	2732
10.3	76.6	1740	10.3	76.6	1114	10.3	76.6	2732
10.3	86.9	1740	10.3	86.9	1114	10.3	86.9	2732
10.3	97.2	1740	10.3	97.2	1114	10.3	97.2	2732
10.3	107.5	1740	10.3	107.5	1114	10.3	107.5	2732
10.3	117.8	1740	10.3	117.8	1114	10.3	117.8	2732
2.2	120.0	1740	2.2	120.0	1114	2.2	120.0	2732
8.1	128.1	1740	8.1	128.1	1114	8.1	128.1	2732
10.3	138.4	1740	10.3	138.4	1114	10.3	138.4	2732
10.3	148.7	1740	10.3	148.7	1114	10.3	148.7	2732
10.3	159.0	1740	10.3	159.0	1114	10.3	159.0	2732
8.5	167.5	1740	8.5	167.5	1114	8.5	167.5	2732
8.5	176.0	1740	8.5	176.0	1114	8.5	176.0	2732
10.0	186.0	1744	10.0	186.0	1116	10.0	186.0	2738
10.0	196.0	1744	10.0	196.0	1116	10.0	196.0	2738
10.0	206.0	1744	10.0	206.0	1116	10.0	206.0	2738
10.0	216.0	1744	10.0	216.0	1116	10.0	216.0	2738
10.0	226.0	1744	10.0	226.0	1116	10.0	226.0	2738
10.0	236.0	1800	10.0	236.0	1152	10.0	236.0	2826
14.0	250.0	4027	14.0	250.0	2577	14.0	250.0	6323
25.0	275.0	4027	25.0	275.0	2577	25.0	275.0	6323
25.0	300.0	4027	25.0	300.0	2577	25.0	300.0	6323
25.0	325.0	4027	25.0	325.0	2577	25.0	325.0	6323
25.0	350.0	4027	25.0	350.0	2577	25.0	350.0	6323
25.0	375.0	4027	25.0	375.0	2577	25.0	375.0	6323
25.0	400.0	4027	25.0	400.0	2577	25.0	400.0	6323
25.0	425.0	4027	25.0	425.0	2577	25.0	425.0	6323
25.0	450.0	4027	25.0	450.0	2577	25.0	450.0	6323
25.0	475.0	4027	25.0	475.0	2577	25.0	475.0	6323
25.0	500.0	4027	25.0	500.0	2577	25.0	500.0	6323
118.0	618.0	4027	118.0	618.0	2577	118.0	618.0	6323
118.0	736.0	4027	118.0	736.0	2577	118.0	736.0	6323
250.0	985.9	4950	250.0	985.9	3168	250.0	985.9	7771
250.0	1235.9	4950	250.0	1235.9	3168	250.0	1235.9	7771

Profile 1			Profile 2			Profile 3		
thickness(ft)	depth (ft)	Vs(ft/s)	thickness(ft)	depth (ft)	Vs(ft/s)	thickness(ft)	depth (ft)	Vs(ft/s)
233.3	1469.3	7150	233.3	1469.3	4576	233.3	1469.3	9285
233.3	1702.6	7150	233.3	1702.6	4576	233.3	1702.6	9285
233.3	1935.9	7150	233.3	1935.9	4576	233.3	1935.9	9285
275.0	2210.9	8500	275.0	2210.9	5440	275.0	2210.9	9285
275.0	2485.9	8500	275.0	2485.9	5440	275.0	2485.9	9285
275.0	2760.9	8500	275.0	2760.9	5440	275.0	2760.9	9285
275.0	3035.8	8500	275.0	3035.8	5440	275.0	3035.8	9285
300.0	3335.8	8500	300.0	3335.8	5440	300.0	3335.8	9285
300.0	3635.8	8500	300.0	3635.8	5440	300.0	3635.8	9285
300.0	3935.8	8500	300.0	3935.8	5440	300.0	3935.8	9285
300.0	4235.8	8500	300.0	4235.8	5440	300.0	4235.8	9285
300.0	4535.8	8500	300.0	4535.8	5440	300.0	4535.8	9285
300.0	4835.8	8500	300.0	4835.8	5440	300.0	4835.8	9285
300.0	5135.7	8500	300.0	5135.7	5440	300.0	5135.7	9285
300.0	5435.7	8500	300.0	5435.7	5440	300.0	5435.7	9285
300.0	5735.7	8500	300.0	5735.7	5440	300.0	5735.7	9285
300.0	6035.7	8500	300.0	6035.7	5440	300.0	6035.7	9285
3280.8	9316.5	9285	3280.8	9316.5	9285	3280.8	9316.5	9285

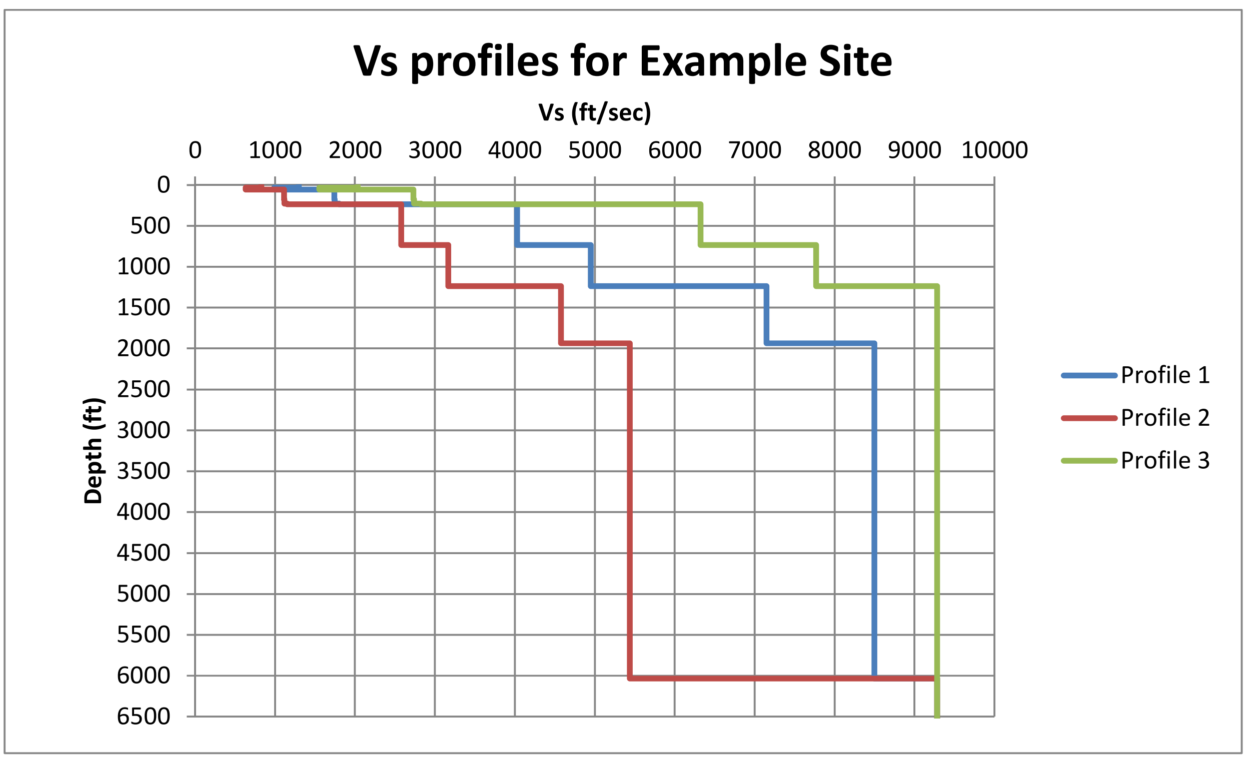

Figure 2.3.2-1. Shear wave velocity profile used in site response calculations for *Example NPP*

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.

2.3.2.1 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.

2.3.2.2 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 Profile*s

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. Consistent with the discussion in Appendix B of the SPID (EPRI 1025287, 2012), the velocity randomization procedure made use of random field models which describe the statistical correlation between layering and shear wave velocity. The default randomization parameters developed in Toro (1997) for USGS A site conditions were used for this site. 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 factors (5% damped pseudo absolute response spectra) which describe the amplification (or de-amplification) of hard reference rock motion as a function of frequency and input reference rock amplitude. The amplification factors are represented in terms of a median amplification value and an associated standard deviation (sigma) for each oscillator frequency and input rock amplitude. Consistent with the SPID (EPRI, 2013a) a minimum median amplification value of 0.5 was employed in the present analysis. Figure 2.3.6-1 illustrates the median and +/- 1 standard deviation in the predicted amplification factors developed for the eleven loading levels parameterized by the median reference (hard rock) peak acceleration (0.01g to 1.50g) for profile P1 and EPRI (1993) rock G/G_{max} and hysteretic damping curves. The variability in the amplification factors results from variability in shear-wave velocity, depth to hard rock, and modulus reduction and hysteretic damping curves. To illustrate the effects of nonlinearity at the *Example* firm rock site, Figure 2.3.6-2 shows the corresponding amplification factors developed with linear site response analyses (model M2).

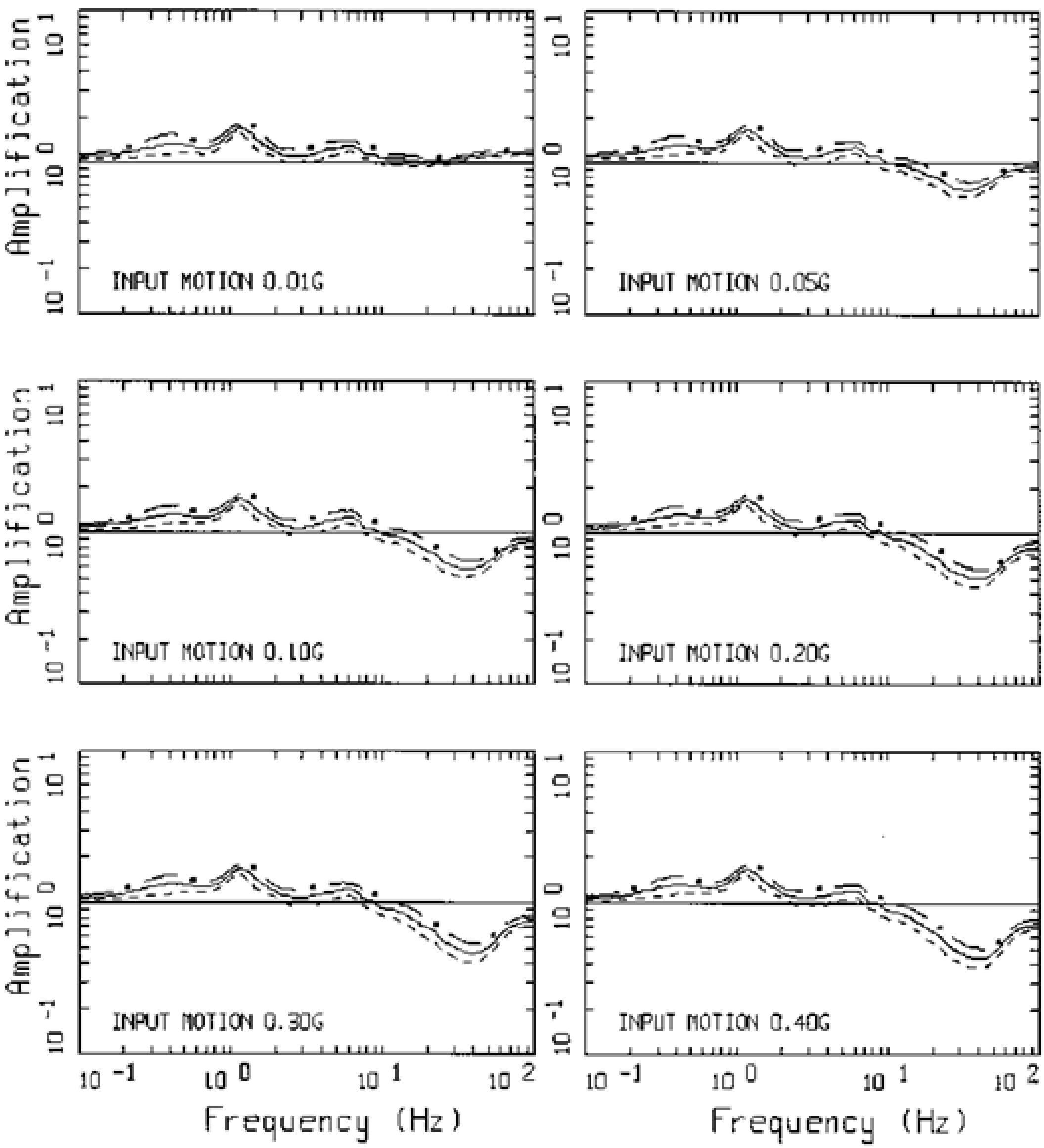

Figure 2.3.6-1. Example suite of amplification factors (5% damping pseudo absolute acceleration spectra) developed for the mean base-case profile (P1), EPRI rock modulus reduction and hysteretic damping curves (model M1), and base-case kappa (K1) at eleven loading levels of hard rock median peak acceleration values from 0.01g to 1.50g. **M** 6.5 and single-corner source model (SPID, EPRI, 2013a).

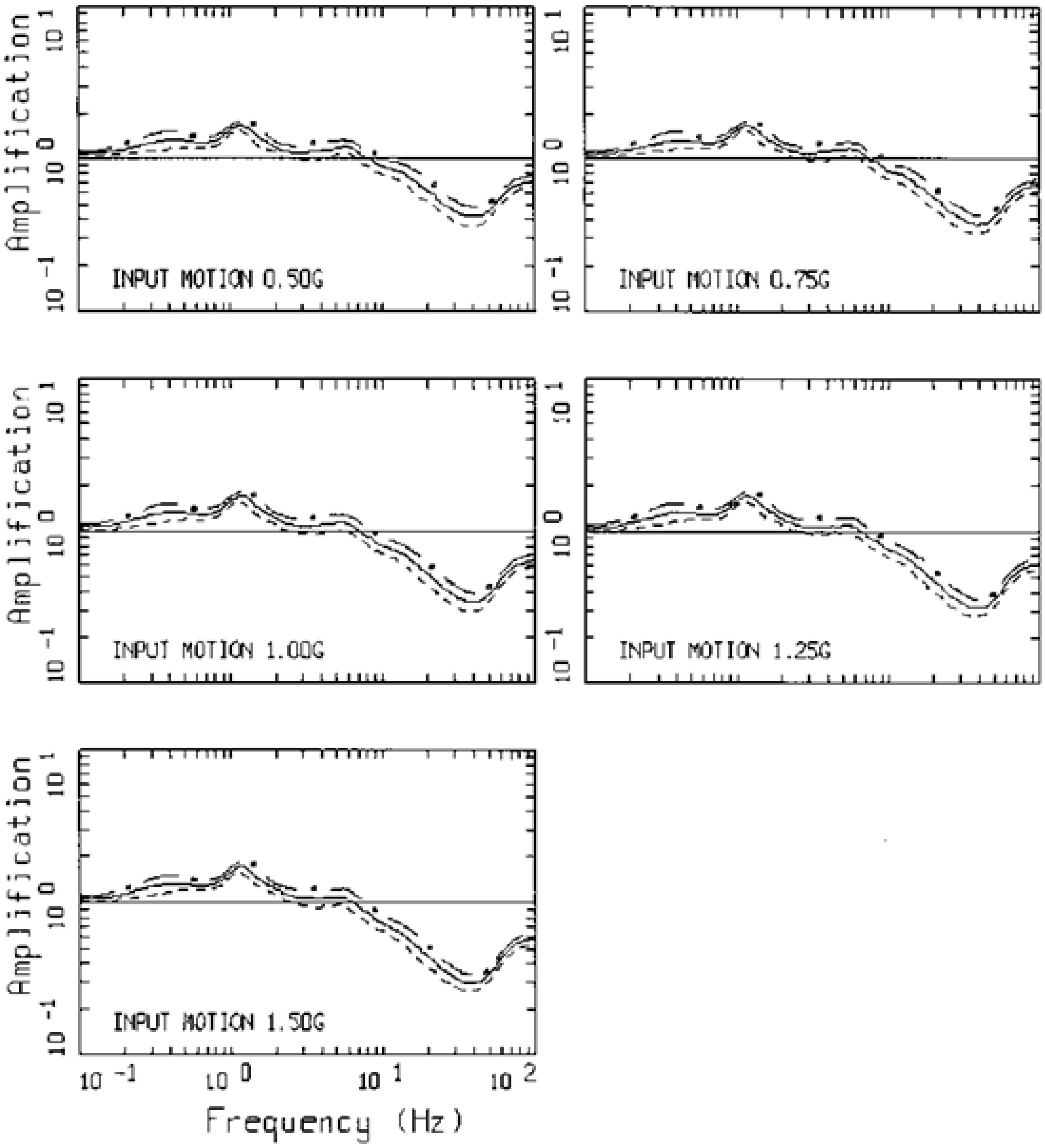

Figure 2.3.6-1. (Continued)

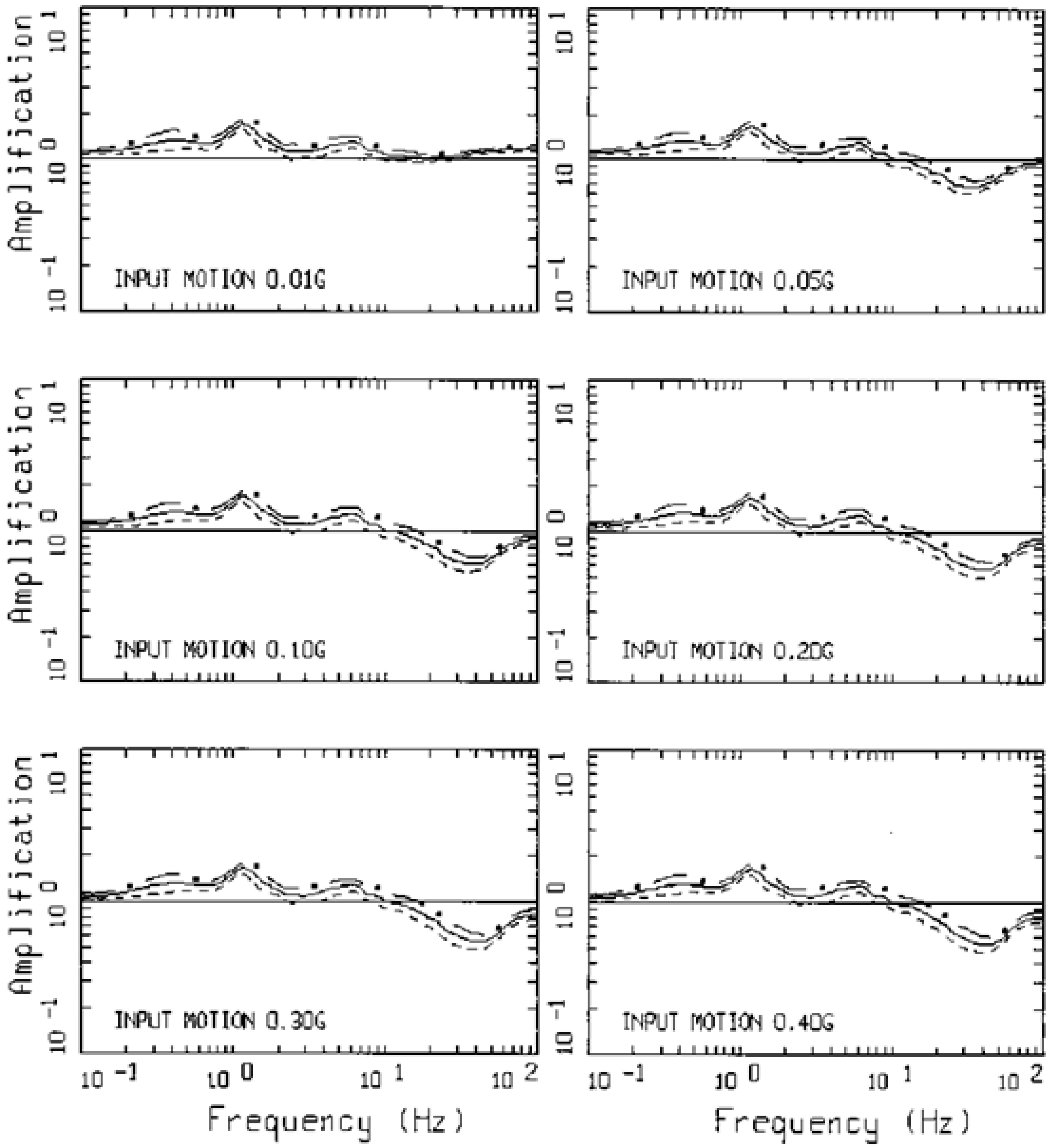

Figure 2.3.6-2. Example suite of amplification factors (5% damping pseudo absolute acceleration spectra) developed for the mean base-case profile (P1), linear site response (model M2), and base-case kappa (K1) at eleven loading levels of hard rock median peak acceleration values from 0.01g to 1.50g. **M** 6.5 and single-corner source model (SPID, EPRI, 2013a).

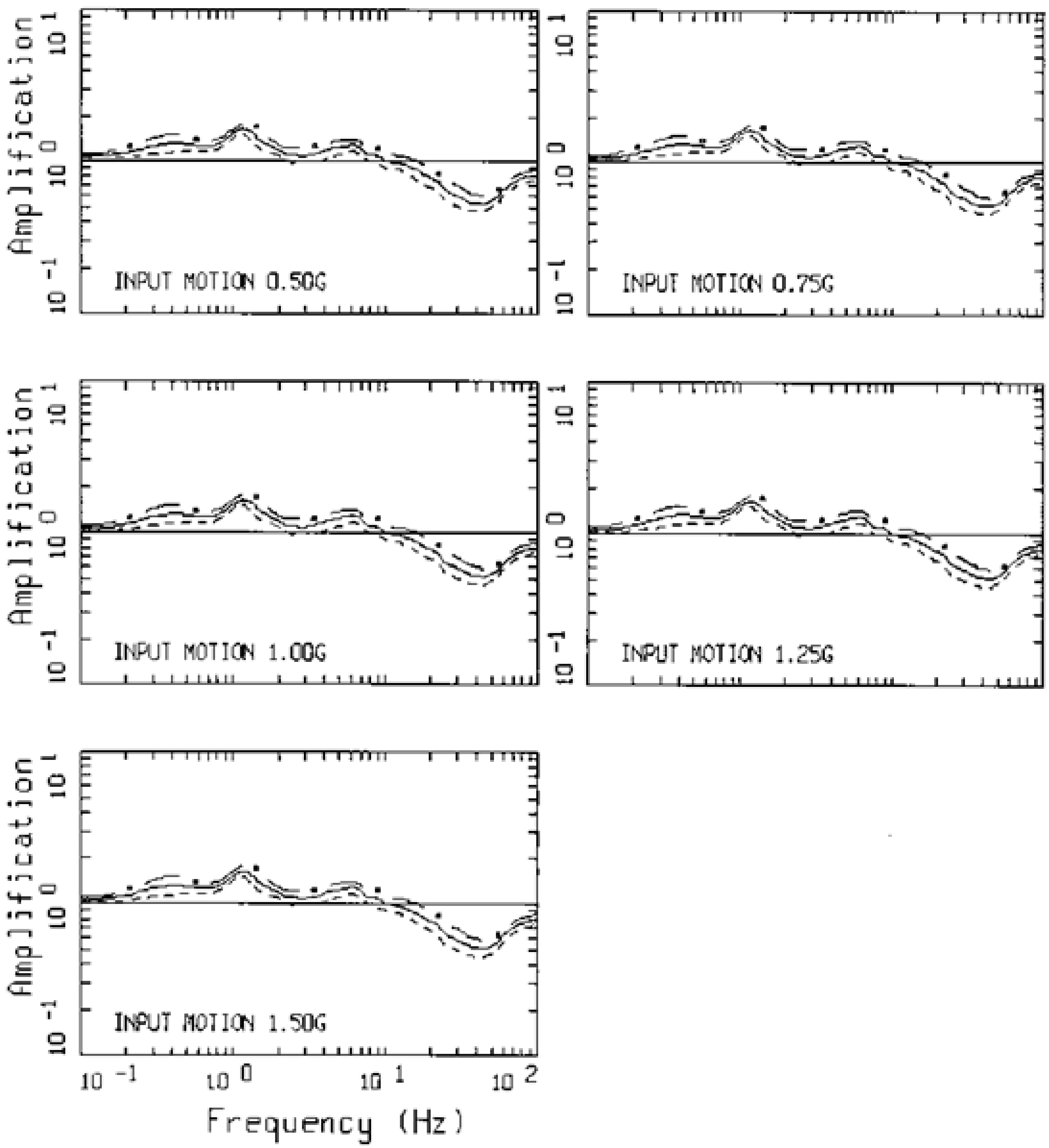

Figure 2.3.6-1. (Continued)

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.

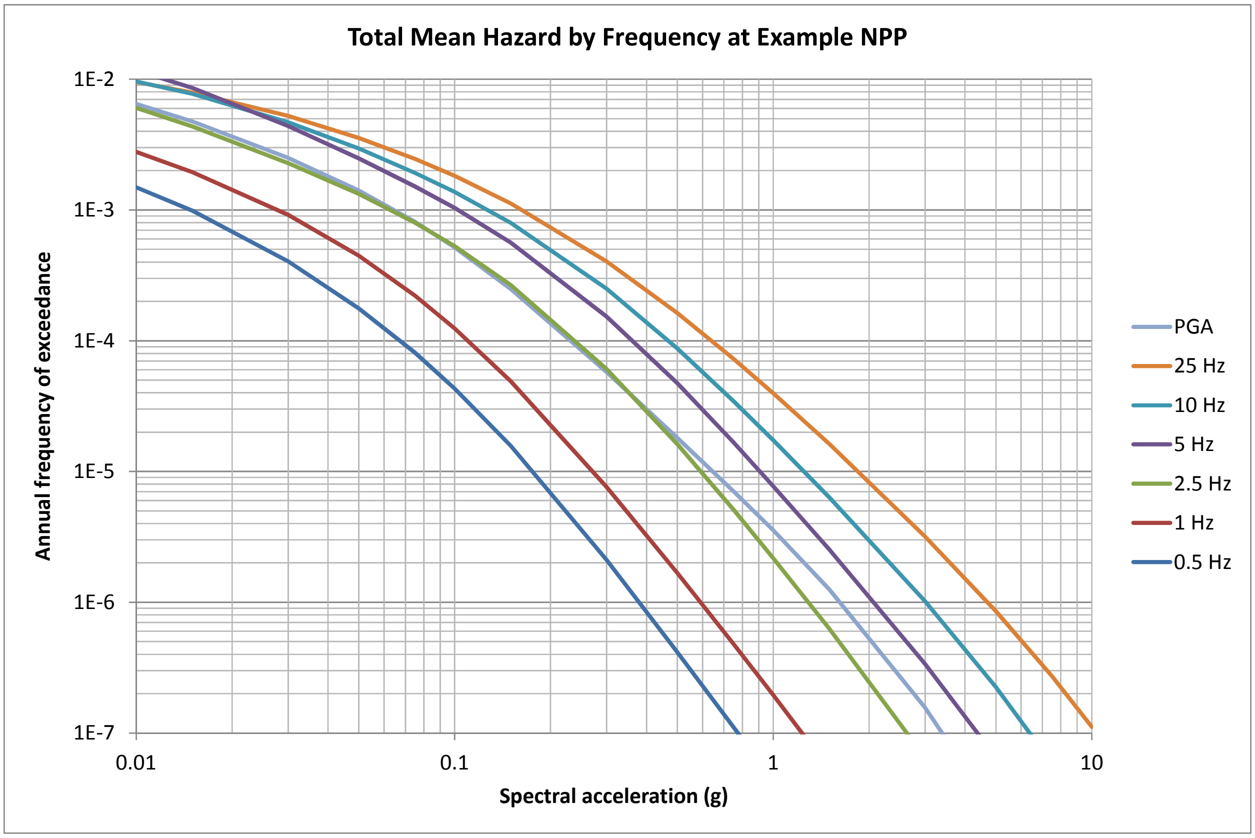

Figure 2.3.7-1. Control point mean hazard curves for spectral frequencies of 0.5, 1, 2.5, 5, 10, 25 and 100 Hz at the *Plant* site.

2.4 Control Point Response Spectra

The control point hazard curves described above have been used to develop uniform hazard response spectra (UHRS) and the ground motion response spectrum (GMRS). The UHRS 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.

The GMRS is probabilistically based ground motion with a mean annual frequency of exceedance between 1 chance in 10,000 years and 1 chance in 100,000 years. The 1E-4 and 1E-5 UHRS, along with a design factor (DF) are used to compute the GMRS at the control point using the criteria in Regulatory Guide 1.208. Table 2.4-1 shows the UHRS and GMRS spectral accelerations.

Table 2.4-1. UHRS for 10-4 and 10-5 and GMRS at control point for *Example NPP.*

Freq, Hz	1E-4 UHRS	1E-5 UHRS	GMRS
100	0.113	0.349	0.167
90	0.117	0.364	0.174
80	0.124	0.388	0.185
70	0.136	0.437	0.208
60	0.165	0.545	0.257
50	0.222	0.748	0.352
40	0.280	0.926	0.437
35	0.297	0.963	0.457
30	0.307	0.973	0.463
25	0.312	0.963	0.461
20	0.292	0.887	0.426
15	0.260	0.781	0.376
12.5	0.237	0.707	0.341
10	0.208	0.617	0.298
9	0.202	0.591	0.286
8	0.189	0.546	0.265
7	0.170	0.483	0.235
6	0.159	0.443	0.216
5	0.146	0.398	0.195
4	0.116	0.310	0.153
3.5	0.113	0.296	0.147
3	0.090	0.231	0.115
2.5	0.078	0.194	0.097
2	0.077	0.188	0.094
1.5	0.056	0.134	0.0676
1.25	0.048	0.115	0.0579
1	0.042	0.098	0.0495
0.9	0.041	0.097	0.0490
0.8	0.039	0.094	0.0475
0.7	0.036	0.088	0.0442
0.6	0.031	0.077	0.0385
0.5	0.025	0.062	0.0312
0.4	0.020	0.050	0.0250
0.35	0.018	0.044	0.0219
0.3	0.015	0.037	0.0187
0.25	0.013	0.031	0.0156
0.2	0.010	0.025	0.0125

Freq, Hz	1E-4 UHRS	1E-5 UHRS	GMRS
0.15	0.008	0.019	0.0094
0.125	0.006	0.016	0.0078
0.1	0.005	0.012	0.00625

Figure 2.4-1 shows the control point UHRS and GMRS

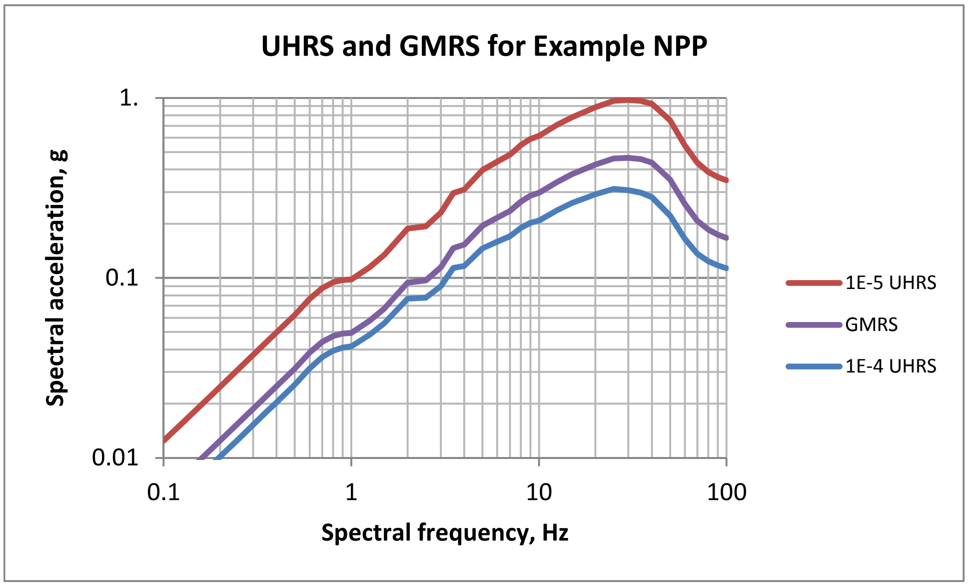

Figure 2.4-1. UHRS for 1E-4 and 1E-5 and GMRS at control point for *Example NPP*.

3.0 Plant Design Basis and Beyond Design Basis Evaluation Ground Motion

The design basis for *Plant* is identified in the Updated Final Safely Evaluation Report (*reference*).

[Optional text (for plants that use IPEEE HCLPF for screening):
An evaluation for beyond design basis (BDB) ground motions was performed in the Individual Plant Examination of External Events (IPEEE). The IPEEE capacity response spectrum is included below for screening purposes.]

[Optional text (for plants that provide IPEEE HCLPF for information only):
An evaluation for beyond design basis (BDB) ground motions was performed in the Individual Plant Examination of External Events (IPEEE). The IPEEE capacity response spectrum is included below to provide additional information about previously reported BDB evaluations.]

3.1 SSE Description of Spectral Shape

The SSE was developed in accordance with 10 CFR Part 100, Appendix A through an evaluation of the maximum earthquake potential for the region surrounding the site. Considering the historic seismicity of the site region, 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.

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 0.16 g as the anchor point for the SSE. The 5% damped horizontal SSE is shown in Table 3.1-1.

Table 3.1-1. SSE for *Example NPP.*

Freq (Hz)	SSE (g)
0.33	0.055
1.60	0.275
8.00	0.250
33.00	0.160
100.00	0.160

3.2 Control Point Elevation

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

3.3 IPEEE Description and Capacity Response Spectrum

The Individual Plant Examination of External Events (IPEEE) was performed as a focused scope SMA using the EPRI approach.

[Optional text (if the IHS is used for screening):
Include text explaining the basis for the IHS, including applicable references. Information should be included describing the IHS response spectrum shape and the PGA level. The IPEEE Adequacy Determination according to SPID Section 3.3.1 is included as Appendix B.]

[Optional text (if the IHS is provided for information:
Include text explaining the basis for the IHS, including applicable references and noting that it will not be used for screening. Information should be included describing the IHS response spectrum shape and the PGA level.]

The 5% damped horizontal IHS spectral acceleration is provided in Table 3.3-1. The SSE and IHS are shown in Figure 3.3-1.

Table 3.3-1. IHS for *Example NPP.*

Freq (Hz)	IHS
0.10	0.004
0.15	0.010
0.20	0.017
0.30	0.039
0.37	0.059
0.70	0.222
1.00	0.318
1.25	0.396
1.50	0.476
1.80	0.570
2.00	0.637
2.50	0.637
3.33	0.637
4.00	0.637
5.00	0.637
5.60	0.637
6.67	0.637
8.00	0.637
10.00	0.576
13.50	0.510

Freq (Hz)	IHS
20.00	0.430
33.00	0.300
100.00	0.300

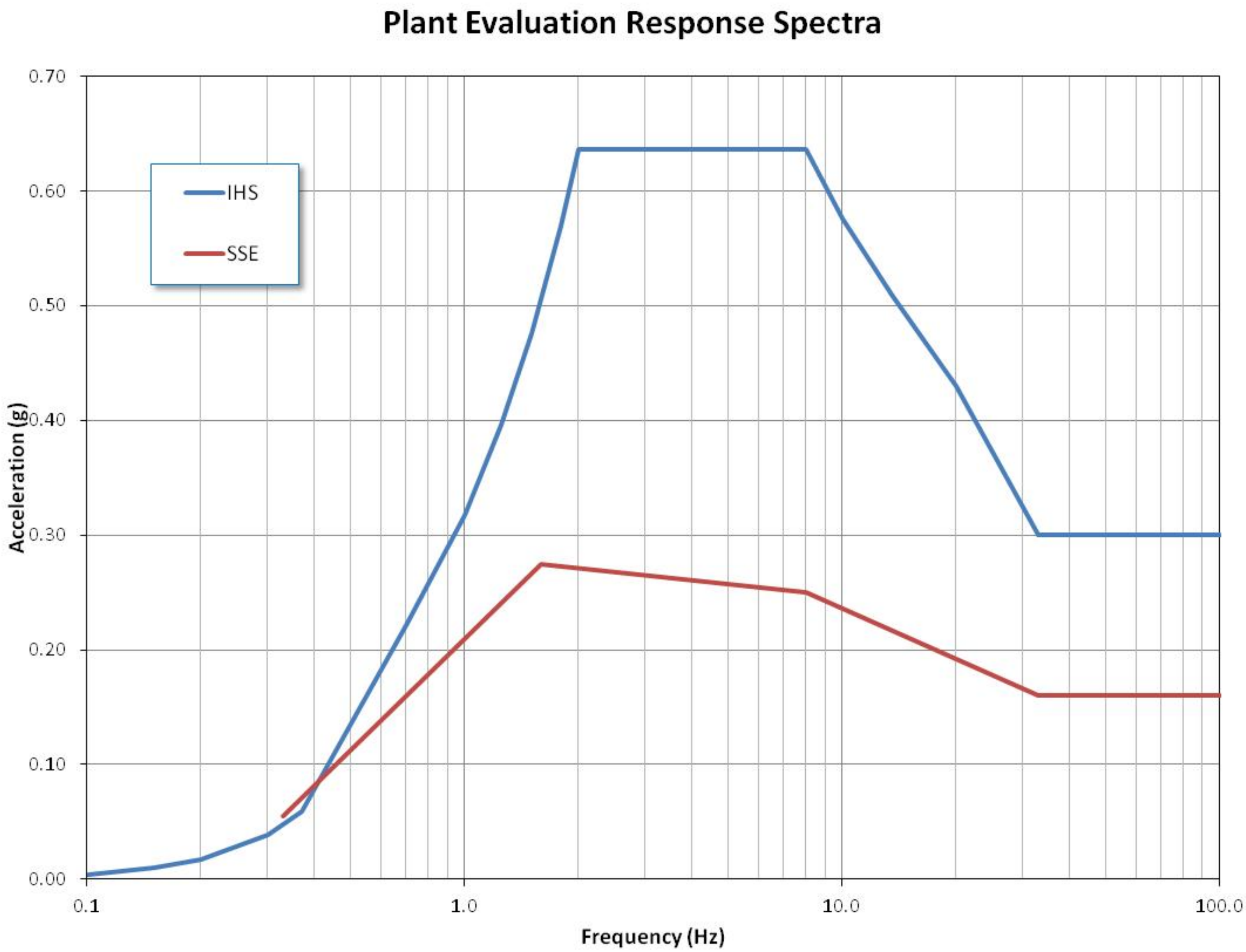

Figure 3.3-1. SSE and IHS Response Spectra for *Example NPP*.

4.0 Screening Evaluation

In accordance with SPID Section 3, a screening evaluation was performed as described below.

4.1 Risk Evaluation Screening (1 to 10 Hz)

[Optional text (for plant that screens in based on SSE):
In the 1 to 10 Hz part of the response spectrum, the GMRS exceeds the SSE. Therefore, a risk evaluation will be performed following the guidance in Section 6.2 of the SPID.]

[Optional text (for plant that screens out based on SSE):
In the 1 to 10 Hz part of the response spectrum, the SSE exceeds the GMRS. Therefore, a risk evaluation will not be performed.]

[Optional text (for plant that screens out based on IHS):
In the 1 to 10 Hz part of the response spectrum, the IHS exceeds the GMRS. Based on this comparison, a risk evaluation will not be performed.]

4.2 High Frequency Screening (> 10 Hz)

[Optional text (for plants that screened out for risk evaluation but have high frequency exceedances:
For a portion of the range above 10 Hz, the GMRS exceeds the SSE. Therefore, a high frequency confirmation will be performed following the guidance in Section 6.4.2 of the SPID.]

[Optional text (for plants that have high frequency exceedances and screened in for a risk evaluation:
For a portion of the range above 10 Hz, the GMRS exceeds the SSE. The high frequency exceedances will be addressed in the risk evaluation discussed in 4.1 above.].

[Optional text (if SSE > GMRS above 10 Hz):
Above 10 Hz, the SSE exceeds the GMRS. Therefore, the high frequency confirmation will not be performed.]

[Optional text (if IHS > GMRS above 10 Hz):
Above 10 Hz, the IHS exceeds the GMRS. Therefore, the high frequency confirmation will not be performed.]

4.3 Spent Fuel Pool Evaluation Screening (1 to 10 Hz)

[Optional text (for plant that screens in):
In the 1 to 10 Hz part of the response spectrum, the GMRS exceeds the SSE. Therefore, a spent fuel pool evaluation will be performed following the guidance in Section 7 of the SPID.]

[Optional text (for plant that screens out):
In the 1 to 10 Hz part of the response spectrum, the SSE exceeds the GMRS. Therefore, a spent fuel pool evaluation will not be performed.]

5.0 Interim Actions

Based on the screening evaluation, the expedited seismic evaluation described in EPRI 3002000704 will be performed as proposed in a letter to NRC dated April 9, 2013 (ML131 01A379) and agreed to by NRC in a letter dated May 7, 2013 (ML13106A331).

[This is where licenses would reference the NRC letter (to be issued) addressing beyond licensing basis seismic considerations.]

NEI letter dated xx provides overall seismic core damage risk estimates using the updated site seismic hazard for the operating fleet of nuclear plants, including *Plant.* Risk estimates for the operating fleet of nuclear plants are comparable to the risk estimates in the NRC's GI-199 Safety/Risk Assessment.

6.0 Conclusions

In accordance with the 50.54(f) request for information, a seismic hazard and screening evaluation was performed for *Plant*. A GMRS was developed solely for purpose of screening for additional evaluations in accordance with the SPID.

[Optional text (if no further evaluations will be performed):
Based on the results of the screening evaluation, no further evaluations will be performed.]

[Optional text (if additional evaluations will be performed):
Based on the results of the screening evaluation, *a Seismic PRA [or an NRC SMA (JLD-ISG-2012-04)], a Spent Fuel Pool evaluation, and/or a High Frequency Confirmation will be performed*.]

Appendix

Table A-1a. PGA Seismic Hazard Curves at *EXAMPLE NPP.*

Mean and fractiles for PGA hazard at Example NPP						
PGA (g)	MEAN	0.05	0.15	0.50	0.85	0.95
0.0005	1.15E-01	2.57E-02	6.31E-02	1.02E-01	1.66E-01	2.34E-01
0.0007	9.68E-02	2.40E-02	5.13E-02	8.32E-02	1.45E-01	2.04E-01
0.001	7.90E-02	2.24E-02	3.89E-02	6.76E-02	1.18E-01	1.84E-01
0.0015	6.10E-02	2.09E-02	2.95E-02	4.79E-02	8.91E-02	1.50E-01
0.002	4.98E-02	1.82E-02	2.40E-02	3.89E-02	7.76E-02	1.26E-01
0.003	3.67E-02	1.38E-02	1.82E-02	2.75E-02	5.50E-02	9.55E-02
0.005	2.40E-02	9.12E-03	1.20E-02	1.82E-02	3.63E-02	6.31E-02
0.007	1.77E-02	6.46E-03	8.51E-03	1.38E-02	2.75E-02	4.47E-02
0.01	1.26E-02	4.27E-03	6.03E-03	9.77E-03	1.95E-02	2.95E-02
0.015	8.29E-03	2.63E-03	3.72E-03	6.46E-03	1.29E-02	1.95E-02
0.02	6.00E-03	1.74E-03	2.46E-03	4.90E-03	9.12E-03	1.48E-02
0.03	3.64E-03	9.33E-04	1.32E-03	2.82E-03	5.82E-03	9.12E-03
0.05	1.79E-03	4.07E-04	5.75E-04	1.32E-03	2.82E-03	4.90E-03
0.07	1.09E-03	2.51E-04	3.55E-04	8.13E-04	1.74E-03	2.82E-03
0.1	6.36E-04	1.45E-04	2.04E-04	4.37E-04	1.07E-03	1.62E-03
0.15	3.44E-04	7.50E-05	1.10E-04	2.19E-04	6.17E-04	9.33E-04
0.2	2.21E-04	4.79E-05	6.76E-05	1.45E-04	4.07E-04	6.17E-04
0.3	1.17E-04	2.24E-05	3.39E-05	7.24E-05	2.19E-04	3.55E-04
0.5	4.88E-05	7.41E-06	1.20E-05	3.06E-05	8.91E-05	1.55E-04
0.7	2.58E-05	3.24E-06	5.62E-06	1.59E-05	4.79E-05	8.91E-05
1.	1.22E-05	1.15E-06	2.14E-06	6.92E-06	2.24E-05	4.17E-05
1.5	4.60E-06	3.09E-07	6.17E-07	2.29E-06	8.51E-06	1.64E-05
2.	2.10E-06	1.02E-07	2.19E-07	9.02E-07	3.72E-06	7.67E-06
3.	5.93E-07	1.76E-08	4.17E-08	2.19E-07	1.00E-06	2.29E-06
5.	8.95E-08	1.19E-09	3.72E-09	2.40E-08	1.35E-07	3.80E-07
7.	2.10E-08	1.55E-10	5.56E-10	4.57E-09	3.06E-08	9.55E-08
10.	3.75E-09	1.43E-11	5.89E-11	6.17E-10	5.25E-09	1.76E-08

Table A-1b. 0.5 Hz Seismic Hazard Curves at *EXAMPLE NPP.*
Table A-1c. 1 Hz Seismic Hazard Curves at *EXAMPLE NPP.*
Table A-1d. 2.5 Hz Seismic Hazard Curves at *EXAMPLE NPP.*
Table A-1e. 5 Hz Seismic Hazard Curves at *EXAMPLE NPP.*
Table A-1f. 10 Hz Seismic Hazard Curves at *EXAMPLE NPP.*
Table A-1g. 25 Hz Seismic Hazard Curves at *EXAMPLE NPP.*

Table A-2. Medians and logarithmic sigmas of amplification factors for *EXAMPLE NPP*.

PGA	Median AF	Sigma ln(AF)	25 Hz	Median AF	Sigma ln(AF)	10 Hz	Median AF	Sigma ln(AF)	5 Hz	Median AF	Sigma ln(AF)
1.00E-02	1.09E+00	3.90E-02	1.30E-02	9.91E-01	4.77E-02	1.90E-02	1.05E+00	8.59E-02	2.09E-02	1.17E+00	8.21E-02
4.95E-02	9.67E-01	5.22E-02	1.02E-01	8.33E-01	8.69E-02	9.99E-02	1.03E+00	1.02E-01	8.24E-02	1.16E+00	8.38E-02
9.64E-02	9.26E-01	5.64E-02	2.13E-01	8.09E-01	9.53E-02	1.85E-01	1.02E+00	1.04E-01	1.44E-01	1.16E+00	8.36E-02
1.94E-01	8.94E-01	5.92E-02	4.43E-01	7.91E-01	9.99E-02	3.56E-01	1.02E+00	1.06E-01	2.65E-01	1.16E+00	8.32E-02
2.92E-01	8.78E-01	6.06E-02	6.76E-01	7.82E-01	1.02E-01	5.23E-01	1.01E+00	1.07E-01	3.84E-01	1.14E+00	8.30E-02
3.91E-01	8.67E-01	6.14E-02	9.09E-01	7.74E-01	1.03E-01	6.90E-01	1.01E+00	1.07E-01	5.02E-01	1.14E+00	8.28E-02
4.93E-01	8.59E-01	6.21E-02	1.15E+00	7.68E-01	1.04E-01	8.61E-01	1.00E+00	1.08E-01	6.22E-01	1.14E+00	8.28E-02
7.41E-01	8.45E-01	6.31E-02	1.73E+00	7.56E-01	1.06E-01	1.27E+00	9.93E-01	1.09E-01	9.13E-01	1.13E+00	8.28E-02
1.01E+00	8.35E-01	6.37E-02	2.36E+00	7.45E-01	1.08E-01	1.72E+00	9.86E-01	1.10E-01	1.22E+00	1.13E+00	8.27E-02
1.28E+00	8.26E-01	6.38E-02	3.01E+00	7.36E-01	1.09E-01	2.17E+00	9.78E-01	1.10E-01	1.54E+00	1.13E+00	8.26E-02
1.55E+00	8.20E-01	6.38E-02	3.63E+00	7.29E-01	1.09E-01	2.61E+00	9.71E-01	1.10E-01	1.85E+00	1.12E+00	8.25E-02
2.5 Hz	**Median AF**	**Sigma ln(AF)**	**1 Hz**	**Median AF**	**Sigma ln(AF)**	**0.5 Hz**	**Median AF**	**Sigma ln(AF)**			
2.18E-02	1.12E+00	8.15E-02	1.27E-02	1.28E+00	8.47E-02	8.25E-03	1.25E+00	8.37E-02			
7.05E-02	1.11E+00	8.14E-02	3.43E-02	1.27E+00	8.26E-02	1.96E-02	1.25E+00	8.07E-02			
1.18E-01	1.11E+00	8.11E-02	5.51E-02	1.27E+00	8.19E-02	3.02E-02	1.24E+00	7.98E-02			
2.12E-01	1.11E+00	8.05E-02	9.63E-02	1.27E+00	8.13E-02	5.11E-02	1.24E+00	7.91E-02			
3.04E-01	1.11E+00	8.03E-02	1.36E-01	1.27E+00	8.09E-02	7.10E-02	1.24E+00	7.89E-02			
3.94E-01	1.11E+00	8.01E-02	1.75E-01	1.27E+00	8.06E-02	9.06E-02	1.24E+00	7.88E-02			
4.86E-01	1.11E+00	7.99E-02	2.14E-01	1.27E+00	8.05E-02	1.10E-01	1.24E+00	7.86E-02			
7.09E-01	1.11E+00	7.96E-02	3.10E-01	1.27E+00	8.03E-02	1.58E-01	1.24E+00	7.86E-02			
9.47E-01	1.11E+00	7.94E-02	4.12E-01	1.27E+00	8.02E-02	2.09E-01	1.24E+00	7.88E-02			
1.19E+00	1.11E+00	7.88E-02	5.18E-01	1.27E+00	8.01E-02	2.62E-01	1.24E+00	7.88E-02			
1.43E+00	1.11E+00	7.85E-02	6.19E-01	1.27E+00	8.01E-02	3.12E-01	1.24E+00	7.89E-02			