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MAR 26 2014

10 CFR 50.54 (f)

U. S. Nuclear Regulatory Commission
Attn: Document Control Desk
Washington, DC 20555-0001

SUSQUEHANNA STEAM ELECTRIC STATION SEISMIC HAZARD AND SCREENING REPORT (CEUS SITES), RESPONSE TO NRC REQUEST FOR INFORMATION PURSUANT TO 10 CFR 50.54(F) REGARDING RECOMMENDATION 2.1 OF THE NEAR-TERM TASK FORCE REVIEW OF INSIGHTS FROM THE FUKUSHIMA DAI-ICHI ACCIDENT PLA-7145

Docket No. 50-387 and No. 50-388

References:

1. *NRC Letter, 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, dated March 12, 2012, ADAMS Accession No. ML12073A202*
2. *NEI Letter, Proposed Path Forward for NTTF Recommendation 2.1: Seismic Reevaluations, dated April 9, 2013, ADAMS Accession No. ML13101A379*
3. *NRC Letter, Electric Power Research Institute Final Draft Report XXXXXX, "Seismic Evaluation Guidance: Augmented Approach for the Resolution of Fukushima Near-Term Task Force Recommendation 2.1: Seismic," as an Acceptable Alternative to the March 12, 2012, Information Request for Seismic Reevaluations, dated May 7, 2013, ADAMS Accession No. ML13106A331*
4. *EPRI Report 1025287, Seismic Evaluation Guidance, Screening, Prioritization and Implementation Details (SPID) for the Resolution of Fukushima Near-Term Task Force Recommendation 2.1: Seismic, dated November 2012, ADAMS Accession No. ML12333A170*
5. *NRC Letter, Endorsement of EPRI Final Draft Report 1025287, "Seismic Evaluation Guidance," dated February 15, 2013, ADAMS Accession No. ML12319A074*

The purpose of this letter is to provide PPL Susquehanna, LLC's (PPL) Seismic Hazard Evaluation and Screening Report.

On March 12, 2012, the Nuclear Regulatory Commission (NRC) issued Reference 1 to all power reactor licensees and holders of construction permits in active or deferred status. Enclosure 1 of Reference 1 requested each addressee located in the Central and Eastern United States (CEUS) to submit a Seismic Hazard Evaluation and Screening Report within 1.5 years from the date of Reference 1.

In Reference 2, the Nuclear Energy Institute (NEI) requested NRC agreement to delay submittal of the final CEUS Seismic Hazard Evaluation and Screening Reports so that an update to the Electric Power Research Institute (EPRI) ground motion attenuation model could be completed and used to develop that information. NEI proposed that descriptions of subsurface materials and properties and base case velocity profiles be submitted to the NRC by September 12, 2013, with the remaining seismic hazard and screening information submitted by March 31, 2014. NRC agreed with that proposed path forward in Reference 3.

Reference 4 contains industry guidance and detailed information to be included in the Seismic Hazard Evaluation and Screening Report submittals. NRC endorsed this industry guidance in Reference 5.

The attached Seismic Hazard Evaluation and Screening Report for Susquehanna SES provides the information described in Section 4 of Reference 4 in accordance with the schedule identified in Reference 2.

This letter contains no new or revised regulatory commitments.

If you have any questions regarding this report, please contact
Mr. John L. Tripoli at 570-542-3100.

I declare under penalty of perjury that the foregoing is true and correct.

Executed on: 3/26/14

Sincerely,

T. S. Rausch

Enclosure: Seismic Hazard and Screening Report for Susquehanna Steam Electric Station Units 1 and 2

Copy: NRC Region I
Mr. J. E. Greives, NRC Sr. Resident Inspector
Mr. J. A. Whited, NRC Project Manager
Mr. L. J. Winker, PA DEP/BRP

Enclosure to PLA-7145

Seismic Hazard and Screening Report

For

Susquehanna Steam Electric Station

Units 1 & 2

(March 2014)

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 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 10 CFR 50.54(f) letter (USNRC, 2012) that requests information to assure that these recommendations are addressed by all U.S. nuclear power plants. The 10 CFR 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 10 CFR 50.54(f) letter pertaining to NTTF Recommendation 2.1 for the Susquehanna Steam Electric Station (SSES), located in Luzerne County, PA. In providing this information, PPL Susquehanna, LLC 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, 2013a). The Augmented Approach, Seismic Evaluation Guidance: Augmented Approach for the Resolution of Fukushima Near-Term Task Force Recommendation 2.1: Seismic (EPRI, 2013c), 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 SSES 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 was used for the design of seismic Category I systems, structures and components.

In response to the 10 CFR 50.54(f) letter and following the guidance provided in the SPID (EPRI, 2013a), a seismic hazard reevaluation was performed. For screening purposes, a Ground Motion Response Spectrum (GMRS) was developed. For frequencies greater than 7 Hz, the GMRS exceeds the SSE. A seismic risk evaluation for

SSES will not be performed since SSES screens out using the results of the Individual Plant Examination for External Events (IPEEE) (PPL, 1994). Based on the results of the screening evaluation, a Spent Fuel Pool evaluation and a High Frequency Confirmation will be performed.

2.0 Seismic Hazard Reevaluation

SSES is located approximately 7 miles east of Berwick, PA adjacent to the Susquehanna River.

Earthquake activity in historic time within 200 miles of the plant site has been low to 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 VI (Modified Mercalli Scale) is adequately conservative for the site. PPL conservatively used a horizontal peak ground acceleration of 0.10g for the SSE to comply with the minimum design requirement of the regulatory agencies. This section summarizes the regional and local geologic conditions.

2.1 Regional and Local Geology

Regional Geology

The SSES site is located within the Susquehanna Lowland Section of the Ridge and Valley Physiographic Province, which is situated within the Appalachian Basin in eastern Pennsylvania. The Province is characterized by folded Paleozoic sedimentary rocks that form a series of accordant ridges and intervening valleys with a general northeast to southwest trend.

The Paleozoic strata include Middle and Lower Devonian and Silurian shale, limestone and sandstone; and a thick sequence of Cambrian-Ordovician clastic and carbonate rock. In Pennsylvania, within the Ridge and Valley Province, the Paleozoic strata is underlain by a crystalline basement rock estimated to be at a depth of approximately 33,000 ft. (10,058 m) below ground surface. Based on data from several deep exploratory wells located in western Pennsylvania, it was inferred that the age of the Precambrian basement rock is approximately 1 billion years old, and it is composed of metamorphosed green schist and amphibolite. It was also inferred that this rock has a regular, gently sloping surface, dipping eastward and forming the western margin of the Appalachian miogeosyncline (UniStar, 2013).

Local Geology

The local geologic formations have been subjected to a series of mountain-building episodes including the Grenville, Taconic and Alleghenian orogenies. The local structure of the Ridge and Valley Province was imparted to the area during the Alleghenian Orogeny at the end of the Permian Period, nearly 250 million year ago. The site geologic history has been quiet since the end of the Permian Period (UniStar, 2013). Lithology and structure control the drainage pattern, the principal direction of which is to the southeast (PPL, 2011).

The Berwick Anticlinorium, an east-northeast striking, gently northeast plunging anticline trends through the site area. This anticlinorium is a symmetrical structure with the north-northwest and south-southeast limbs dipping with an average 35 degree NNW and SSE, respectively. This structure imparted by the Alleghenian Orogeny is significant to the topography, drainage, and seismicity of the site area, defining the major landforms (elongated ridges and valleys), drainage patterns, and structural discontinuities within the Paleozoic strata (UniStar, 2013).

The soils at the SSES sites are characterized by glacio-fluvial deposits, and were subjected to both glacial and periglacial events during the Quaternary period. Underneath this glacio-fluvial overburden lies the Middle Devonian (~400 million years) bedrock denominated the Mahantango Formation, part of the Hamilton Group. Past reports indicate that the total thickness of the Mahantango Formation exceeds 1,500 ft. (457 m), and is described as a complex series of interbedded shales, siltstones and sandstones (UniStar, 2013). The site presents generally gentle to moderately sloping hills and well developed drainage patterns.

2.2 Probabilistic Seismic Hazard Analysis

2.2.1 Probabilistic Seismic Hazard Analysis Results

In accordance with the 10 50.54(f) letter and following the guidance in the SPID (EPRI, 2013a), 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 (CEUS-SSC, 2012) together with the updated EPRI Ground-Motion Model (GMM) for the CEUS (EPRI, 2013b). For the PSHA, a lower-bound moment magnitude of 5.0 was used, as specified in the 10 CFR 50.54(f) letter.

For the PSHA, the CEUS-SSC background seismic sources out to a distance of 400 miles (640 km) around Susquehanna were included. This distance exceeds the 200 mile

(320 km) recommendation contained in USNRC (2007) and was chosen for completeness. Background sources included in this site analysis are the following:

1. Atlantic Highly Extended Crust (AHEX)
2. Extended Continental Crust—Atlantic Margin (ECC_AM)
3. Great Meteor Hotspot (GMH)
4. Mesozoic and younger extended prior – narrow (MESE-N)
5. Mesozoic and younger extended prior – wide (MESE-W)
6. Midcontinent-Craton alternative A (MIDC_A)
7. Midcontinent-Craton alternative B (MIDC_B)
8. Midcontinent-Craton alternative C (MIDC_C)
9. Midcontinent-Craton alternative D (MIDC_D)
10. Northern Appalachians (NAP)
11. Non-Mesozoic and younger extended prior – narrow (NMESE-N)
12. Non-Mesozoic and younger extended prior – wide (NMESE-W)
13. Paleozoic Extended Crust narrow (PEZ_N)
14. Paleozoic Extended Crust wide (PEZ_W)
15. St. Lawrence Rift, including the Ottawa and Saguenay grabens (SLR)
16. Study region (STUDY_R)

For sources of large magnitude earthquakes, designated Repeated Large Magnitude Earthquake (RLME) sources in CEUS-SSC (2012), the following sources lie within 1,000 km of the site and were included in the analysis:

1. Charleston
2. Charlevoix
3. Wabash Valley

For each of the above background and RLME 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, 2013a), 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 10 CFR 50.54(f) Request for Information and in the SPID (EPRI, 2013a) for nuclear power plant sites that are not founded on hard rock (defined as 2.83 km/sec), a site response analysis was performed for Susquehanna.

2.3.1 Description of Subsurface Material

Site specific information on the stratigraphy of geologic materials underlying and directly adjacent to the SSES Site is based on the geologic/geotechnical investigations performed at the SSES site and the more recent investigations performed at the nearby Bell Bend Nuclear Power Plant (BBNPP) site. The conclusions and recommendations of this Report are based on information obtained from FSAR documents (SSES and BBNPP), boring logs, and drawings as referenced throughout the text.

The subsurface of SSES site can be divided into the following stratigraphic units:

- Glacial overburden soils, characterized by silty sand and gravel with varying amounts of clay and silt;
- Mahantango Formation (Mahantango Shale), which is further subdivided into two units as follows:
    - Weathered Rock (intensely weathered yellowish gray shale),
    - Sound (Competent Rock) – massive medium to dark gray shale.

The geologic conditions at SSES and the BBNPP sites are very similar. Therefore, the data from the BBNPP more recent and in-depth investigation is used to supplement the SSES information and enhance the characterization of the subsurface conditions at the SSES site. Note that for the SSES Site, explorations did not extend below a depth of 130 ft. (40 m). However, site explorations extended to a depth of 420 ft. (128 m) at the BBNPP site.

The SSES and BBNPP site geotechnical and geophysical investigations provide a detailed description of the subsurface stratigraphy of the area. Based on the available geotechnical and geophysical information for the SSES and on the findings from the BBNPP investigations, there is evidence that the Mahantango formation is wide spread throughout the area. Furthermore, descriptions from extracted cores, laboratory tests, geophysical tests, and foundation mappings indicate that the physical attributes of the Mahantango shale are equivalent for both sites. The elevation of the hard rock horizon ($V_s \geq 9{,}200$ fps) at the SSES site is extrapolated from the BBNPP site.

The general site conditions consist of about 15 ft. (4.6 m) of glacial sand and gravel overlying about 15 ft. (4.6 m) of weathered rock with Paleozoic sedimentary rocks to Precambrian basement at a depth of about 33,000 ft. (10 km). The SSE is located at the top of the Paleozoic sequence at elevation of 640 ft. (FZ Ares, 2013). Table 2.3.1-1 shows the geotechnical profile for the site.

Table 2.3.1-1 (FZ Ares, 2013)
Summary of Geotechnical Profile Data for Susquehanna Steam Electric Station

SSES RECOMMENDED	EL.[1] [ft]	DEPTH[2] [ft]	γ[3] [pcf]	V_P[3] [fps]	V_S[3] [fps]	ν[3]
Glacial overburden soils	670.0	0.0	130	1500	700	0.35
Weathered rock	655.0	15.0	130	7600	3600	0.35
Mahantango Formation 1	640.0	30.0	170	16000	7500	0.30
Mahantango Formation 2	540.0	130.0	170	15600	7905	0.32
Mahantango Formation 3	445.8	224.2	170	15705	8415	0.30
Mahantango Formation 4	383.3	286.7	170	15755	9050	0.28
Mahantango Formation 5	248.7	421.3	170	16850	9600	0.26

NOTES:
[1] Actual ground surface elevation
[2] No comment
[3] SSES FSAR for depth less than 130 ft; BBNPP interpolated for depth greater than 130 ft

2.3.2 Development of Base Case Profiles and Nonlinear Material Properties

Table 2.3.1-1 shows the material descriptions and recommended shear-wave velocities as well as unit weights versus depth for the best estimate profile (P1). The location of the SSE control point at an elevation of 640 ft. in at a nominal depth of 30 ft. (9 m) at the top of the Mahantango Formation (Table 2.3.1-1). Site-specific measured shear-wave velocities were based on down-hole and cross-hole surveys and extend only to 130 ft. (40 m), shallow depths into the Mahantango (Table 2.3.1-1). The resulting shear-wave velocities were consistent with more recent measurements at the nearby Bell Bend (BBNPP) Combined Operating License Application (COLA) site with similar geology (Table 2.3.1-1). As a result of the similarities in geology between the Susquehanna and Bell Bend sites, the tabulated shear-wave velocities and unit weights for the Mahantango Formation below a depth of 130 ft. (40 m) were taken from measurements at the BBNPP, which extended to depths of about 420 ft. (128 m). Depth to hard rock conditions (shear-wave velocity at least 9,300 ft./s, 2.83 km/s) was estimated to occur at a depth of about 300 ft. (91 m) at the BBNPP and estimated to be at a depth of 421 ft. (128 m), 391 ft.

(119 m) below the SSE control point elevation (Table 2.3.1-1), at the Susquehanna NPP site.

Based on the similarity of geology and measured shear-wave velocities in the shallow portion of the Mahantango Formation between the Susquehanna NPP site and the nearby BBNPP, a scale factor of 1.25 for developing upper and lower base-cases was judged to reflect an appropriate expression of epistemic uncertainty in shear-wave velocities at the site. The scale factor of 1.25 reflects a σ_{ln} of about 0.2 based on the SPID (EPRI, 2013a) 10th and 90th fractiles, which implies a 1.28 scale factor on σ_{μ}.

Using the shear-wave velocities specified in Table 2.3.1-1, three base-case profiles were developed using the scale factor of 1.25. The specified shear-wave velocities were taken as the mean or best estimate base-case profile (P1) with lower and upper range base-case profiles P2 and P3. Profile P1 extended to hard reference rock at a depth below the SSE control point at 391 ft. (119 m), randomized ± 117 ft. (36 m). To accommodate epistemic uncertainty in depth to hard reference rock, which was not encountered over the depths sampled by measurements, the softest profile (P2) was extended to a depth of 5,000 ft. (1,524 m), randomized ± 1,500 ft. (457 m). For the stiffest profile (P3), upper-range shear-wave velocities exceeded the hard rock value of 9,300 ft./s (2,830 m/s), resulting in adopting P3 as reflecting reference site conditions.

The three base-case profiles are shown in Figure 2.3.2-1 and listed in Table 2.3.2-1.

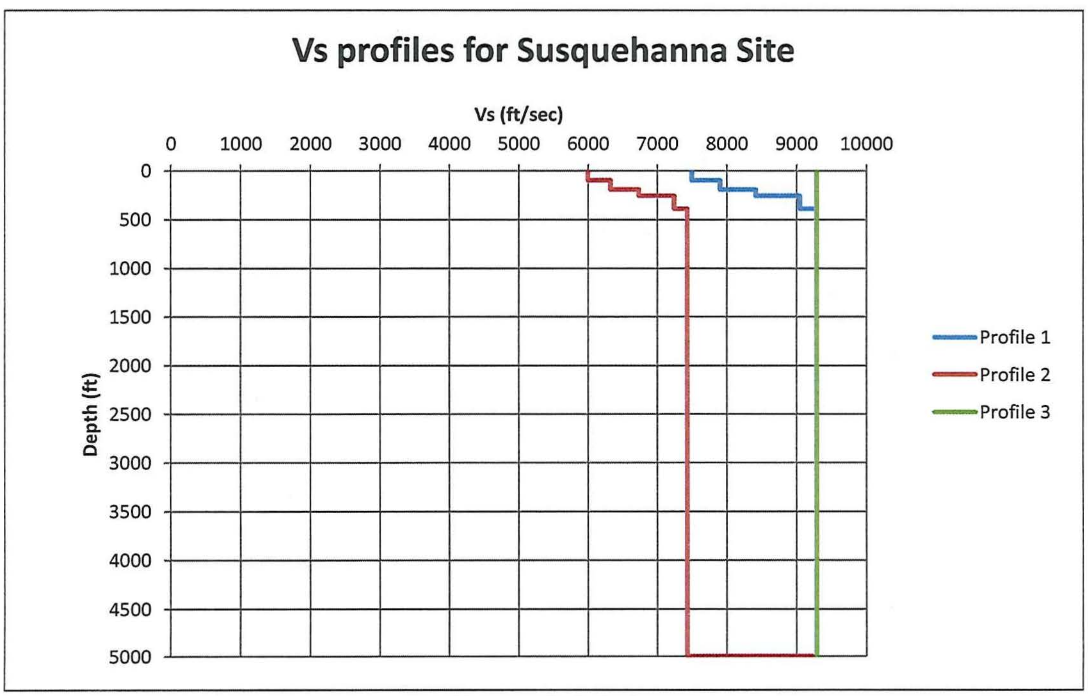

Figure 2.3.2-1
Shear-Wave Velocity Profiles for the Susquehanna NPP Site

Table 2.3.2-1
Layer Thicknesses, Depths, and Shear-Wave Velocities
(Vs) for 3 Profiles at the Susquehanna NPP Site

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	7497		0	5997		0	9285
10.0	10.0	7497	10.0	10.0	5997	5.0	5.0	9285
10.0	20.0	7497	10.0	20.0	5997	5.0	10.0	9285
10.0	30.0	7497	10.0	30.0	5997	10.0	20.0	9285
10.0	40.0	7497	10.0	40.0	5997	10.0	30.0	9285
10.0	50.0	7497	10.0	50.0	5997	5.0	35.0	9285
10.0	60.0	7497	10.0	60.0	5997	10.0	45.0	9285
10.0	70.0	7497	10.0	70.0	5997	5.0	50.0	9285
10.0	80.1	7497	10.0	80.1	5997	10.0	60.0	9285
10.0	90.1	7497	10.0	90.1	5997	10.0	70.0	9285
10.0	100.1	7497	10.0	100.1	5997	5.0	75.0	9285
9.4	109.5	7905	9.4	109.5	6324	10.0	85.0	9285
9.4	118.9	7905	9.4	118.9	6324	10.0	95.0	9285
9.4	128.3	7905	9.4	128.3	6324	10.0	105.0	9285
9.4	137.7	7905	9.4	137.7	6324	10.0	115.0	9285
9.4	147.1	7905	9.4	147.1	6324	5.0	120.0	9285
9.4	156.6	7905	9.4	156.6	6324	10.0	130.0	9285
9.4	166.0	7905	9.4	166.0	6324	5.0	135.0	9285
9.4	175.4	7905	9.4	175.4	6324	10.0	145.0	9285
9.4	184.8	7905	9.4	184.8	6324	10.0	155.0	9285
9.4	194.2	7905	9.4	194.2	6324	10.0	165.0	9285
12.5	206.8	8415	12.5	206.8	6732	10.0	175.0	9285
12.5	219.3	8415	12.5	219.3	6732	10.0	185.0	9285
12.5	231.8	8415	12.5	231.8	6732	10.0	195.0	9285
12.5	244.4	8415	12.5	244.4	6732	10.0	205.0	9285
5.6	250.0	8415	5.6	250.0	6732	10.0	215.0	9285
6.9	256.9	8415	6.9	256.9	6732	10.0	225.0	9285
67.3	324.1	9050	67.3	324.1	7240	10.0	235.0	9285
67.3	391.4	9050	67.3	391.4	7240	10.0	245.0	9285

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)
100.0	491.4	9285	100.0	491.4	7428	100.0	345.0	9285
100.0	591.4	9285	100.0	591.4	7428	100.0	445.0	9285
100.0	691.4	9285	100.0	691.4	7428	100.0	545.0	9285
100.0	791.4	9285	100.0	791.4	7428	100.0	645.0	9285
100.0	891.4	9285	100.0	891.4	7428	100.0	745.0	9285
100.0	991.4	9285	100.0	991.4	7428	100.0	845.0	9285
100.0	1091.4	9285	100.0	1091.4	7428	100.0	945.0	9285
100.0	1191.4	9285	100.0	1191.4	7428	100.0	1045.0	9285
100.0	1291.4	9285	100.0	1291.4	7428	100.0	1145.0	9285
100.0	1391.4	9285	100.0	1391.4	7428	100.0	1245.0	9285
100.0	1491.3	9285	100.0	1491.4	7428	100.0	1345.0	9285
100.0	1591.3	9285	100.0	1591.4	7428	100.0	1445.0	9285
100.0	1691.3	9285	100.0	1691.4	7428	100.0	1545.0	9285
100.0	1791.3	9285	100.0	1791.4	7428	100.0	1645.0	9285
100.0	1891.3	9285	100.0	1891.4	7428	100.0	1744.9	9285
100.0	1991.3	9285	100.0	1991.3	7428	100.0	1844.9	9285
100.0	2091.3	9285	100.0	2091.3	7428	100.0	1944.9	9285
100.0	2191.3	9285	100.0	2191.3	7428	100.0	2044.9	9285
100.0	2291.3	9285	100.0	2291.3	7428	100.0	2144.9	9285
100.0	2391.3	9285	100.0	2391.3	7428	100.0	2244.9	9285
100.0	2491.3	9285	100.0	2491.3	7428	100.0	2344.9	9285
100.0	2591.3	9285	100.0	2591.3	7428	100.0	2444.9	9285
100.0	2691.3	9285	100.0	2691.3	7428	100.0	2544.9	9285
100.0	2791.3	9285	100.0	2791.3	7428	100.0	2644.9	9285
100.0	2891.3	9285	100.0	2891.3	7428	100.0	2744.9	9285
100.0	2991.3	9285	100.0	2991.3	7428	100.0	2844.9	9285
100.0	3091.3	9285	100.0	3091.3	7428	100.0	2944.9	9285
100.0	3191.3	9285	100.0	3191.3	7428	100.0	3044.9	9285
100.0	3291.3	9285	100.0	3291.3	7428	100.0	3144.9	9285
100.0	3391.2	9285	100.0	3391.3	7428	100.0	3244.9	9285
100.0	3491.2	9285	100.0	3491.3	7428	100.0	3344.9	9285
100.0	3591.2	9285	100.0	3591.3	7428	100.0	3444.9	9285
100.0	3691.2	9285	100.0	3691.3	7428	100.0	3544.9	9285
100.0	3791.2	9285	100.0	3791.3	7428	100.0	3644.9	9285
100.0	3891.2	9285	100.0	3891.3	7428	100.0	3744.9	9285

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)
100.0	3991.2	9285	100.0	3991.3	7428	100.0	3844.9	9285
100.0	4091.2	9285	100.0	4091.3	7428	100.0	3944.9	9285
100.0	4191.2	9285	100.0	4191.3	7428	100.0	4044.9	9285
100.0	4291.2	9285	100.0	4291.3	7428	100.0	4144.9	9285
100.0	4391.2	9285	100.0	4391.3	7428	100.0	4244.9	9285
100.0	4491.2	9285	100.0	4491.3	7428	100.0	4344.9	9285
100.0	4591.2	9285	100.0	4591.3	7428	100.0	4444.9	9285
100.0	4691.2	9285	100.0	4691.3	7428	100.0	4544.8	9285
100.0	4791.2	9285	100.0	4791.3	7428	100.0	4644.8	9285
100.0	4891.2	9285	100.0	4891.2	7428	100.0	4744.8	9285
100.0	4991.2	9285	100.0	4991.2	7428	100.0	4844.8	9285
8.5	4999.7	9285	8.5	4999.7	7428	155.0	4999.8	9285
3280.8	8280.5	9285	3280.8	8280.6	9285	3280.8	8280.7	9285

2.3.2.1 Shear Modulus and Damping Curves

No site-specific nonlinear dynamic material properties were determined for the firm rock materials in the initial siting of the Susquehanna NPP. The rock material over the upper 500 ft. (152 m) was assumed to have behavior that could be modeled as either linear or non-linear. To represent this potential for either case in the upper 500 ft. of firm rock at the Susquehanna NPP site, two sets of shear modulus reduction and hysteretic damping curves were used. Consistent with the SPID (EPRI, 2013a), the EPRI rock curves (model M1) were considered to be appropriate to represent the upper range nonlinearity likely in the materials at this site and linear analyses (model M2) was assumed to represent an equally plausible alternative rock response across loading level. For the linear analyses, the low strain damping from the EPRI rock curves were used as the constant damping values in the upper 500 ft.

2.3.2.2 Kappa

Base-case kappa estimates were determined using Section B-5.1.3.1 of the SPID (EPRI, 2013a) for a firm CEUS rock site. Kappa for a firm rock site with at least 3,000 ft. (1 km) of sedimentary rock may be estimated from the average S-wave velocity over the upper 100 ft. (V_{s100}) of the subsurface profile while for a site with less than 3,000 ft. (1 km) of firm rock, kappa may be estimated with a Q_s of 40 below 500 ft. combined with the low strain damping from the EPRI rock and or soil curves and an additional kappa of 0.006 s for the underlying hard rock.

For the Susquehanna profile P1, with about 400 ft. (122 m) of firm rock over hard reference rock, the kappa value of 0.006 s for hard rock (EPRI, 2013a) was combined with the low strain damping in the hysteretic damping curves to give a value of 0.009 s, listed in Table 2.3.2-2. For profile P2, with about 5,000 ft. (1,524 m) of firm rock, the kappa may be estimated from the average shear-wave velocity over the top 100 ft. (30 m) of the profile resulting in a value of 0.012 s for a $\overline{V_s}$ (100 ft.) of 6,000 ft./s (1,829 m/s). For the stiffest profile (P3) taken as reference rock conditions the kappa estimate is 0.006 s (Table 2.3.2-2). The low strain kappa values range from 0.006 s for the stiffest profile (P3) to 0.012 s for the softest profile (P2). The full epistemic uncertainty in overall profile damping has contributions from kappa at low strain in the firm rock but also the wide range in hysteretic damping curves at higher loading levels of significance to design. Additionally, since the profile is relatively stiff, approaching hard reference rock for the mean base-case, kappa will be relatively small. For such cases, epistemic uncertainty in kappa is assumed to have a contribution from the epistemic uncertainty included in the hard rock GMPEs.

Table 2.3.2-2
Kappa Values and Weights Used for Site Response Analyses

Velocity Profile	Kappa(s)
P1	0.009
P2	0.012
P3	0.006
Velocity Profile	Weights
P1	0.4
P2	0.3
P3	0.3
G/G_{max} and Hysteretic Damping Curves	
M1	0.5
M2	0.5

2.3.3 Randomization of Base Case Profiles

To account for the aleatory variability in dynamic 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 Susquehanna NPP site, random shear wave velocity profiles were developed from the base case profiles shown in Figure 2.3.2-1. Consistent with the discussion in Appendix B of the SPID (EPRI, 2013a), 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 United States Geological Survey (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, 2013a), correlation of shear wave velocity between layers was modeled using the footprint 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.

2.3.4 Input Spectra

Consistent with the guidance in Appendix B of the SPID (EPRI, 2013a), input Fourier amplitude spectra were defined for a single representative earthquake magnitude (**M** 6.5) using two different assumptions regarding the shape of the seismic source spectrum (single-corner and double-corner). A range of 11 different input amplitudes (median peak ground accelerations (PGA) ranging from 0.01 to 1.5g) were used in the site response analyses. The characteristics of the seismic source and upper crustal attenuation properties assumed for the analysis of the Susquehanna NPP site were the same as those identified in Tables B-4, B-5, B-6 and B-7 of the SPID (EPRI, 2013a) as appropriate for typical CEUS sites.

2.3.5 Methodology

To perform the site response analyses for the Susquehanna NPP 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, 2013a). The guidance contained in Appendix B of the SPID (EPRI, 2013a) 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 Susquehanna NPP 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 spectral frequency and input rock amplitude. Consistent with the SPID 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, 2013a) 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 Susquehanna NPP firm rock site, Figure 2.3.6-2 shows the corresponding amplification factors developed with linear site response analyses (model M2). Between the linear and nonlinear (equivalent-linear) analyses, Figures 2.3.6-1 and Figure 2.3.6-2 respectively show relatively minor differences across structural frequency as well as loading level. Tabulated values of the amplification factors are provided in Appendix A.

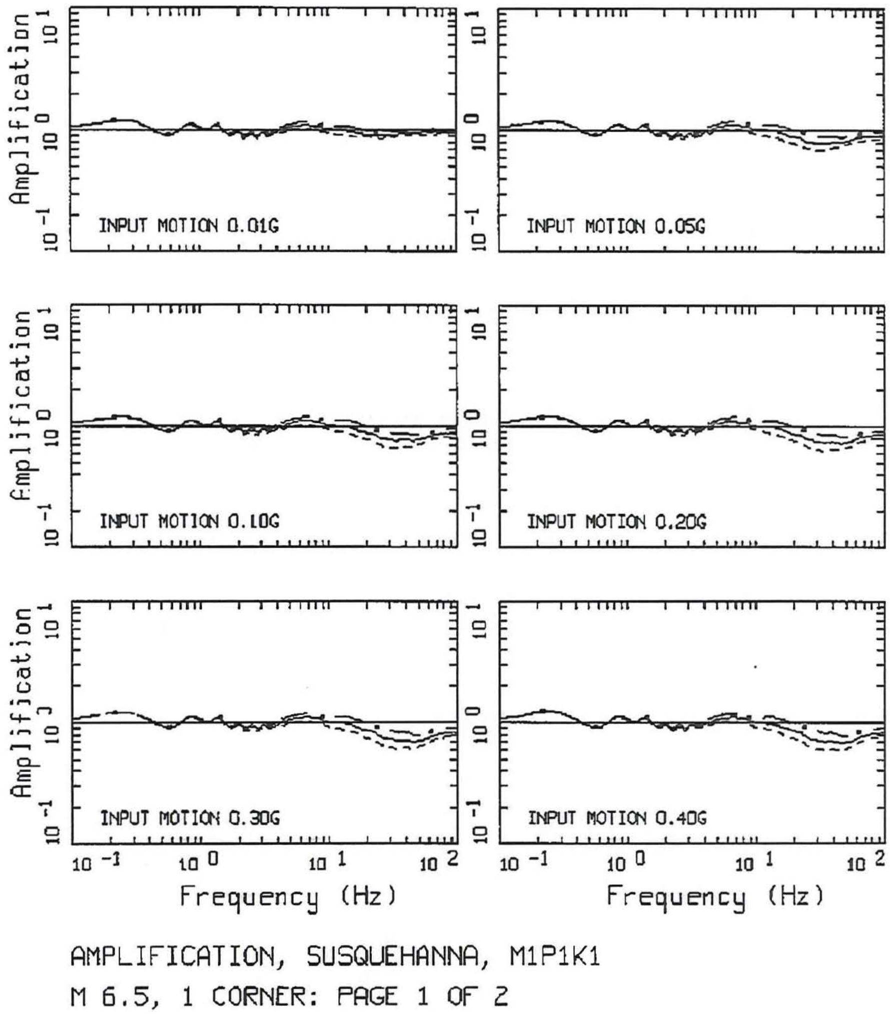

Figure 2.3.6-1

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 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 (EPRI, 2013a).

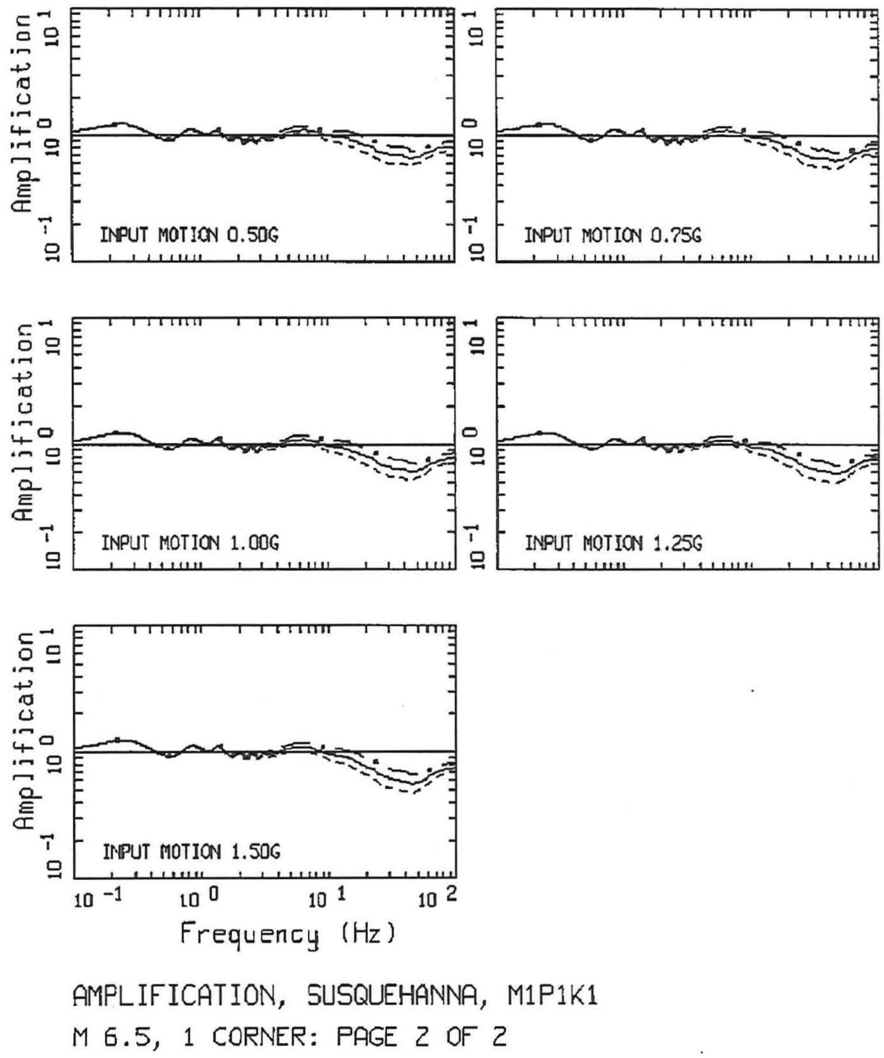

Figure 2.3.6-1 (cont.)

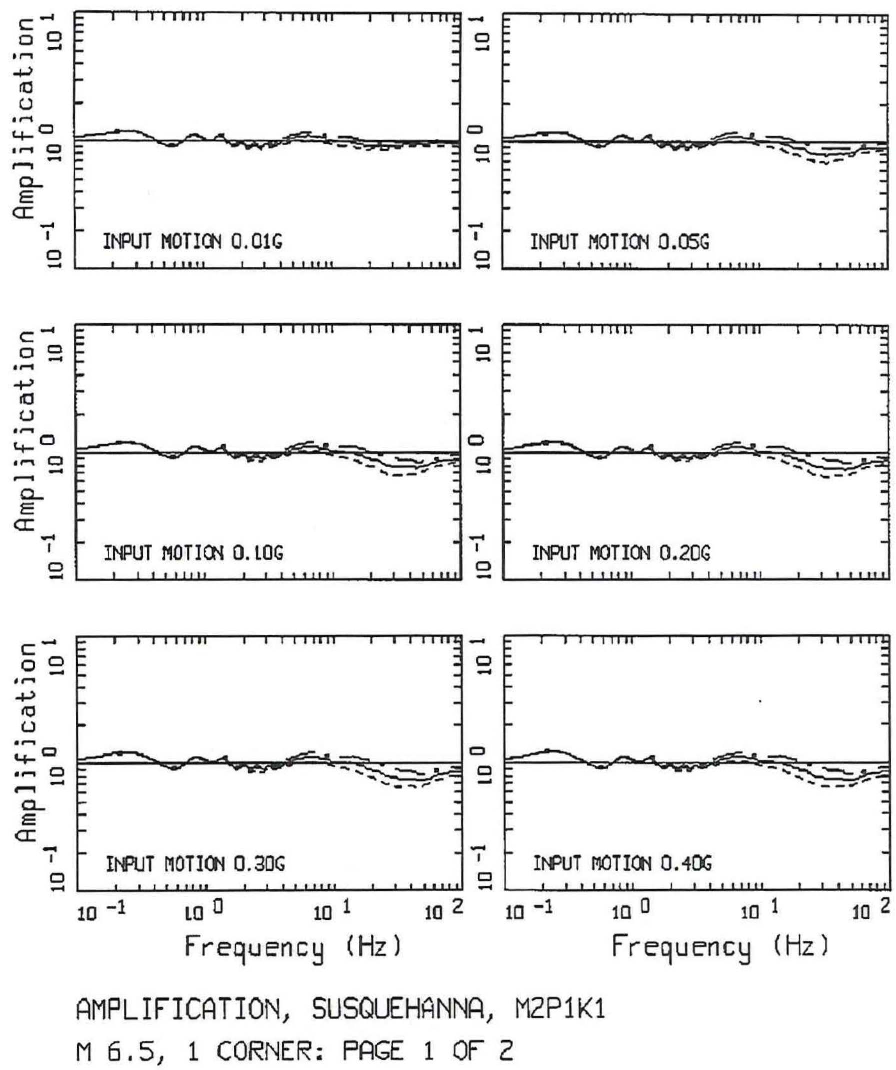

Figure 2.3.6-2

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 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 (EPRI, 2013a).

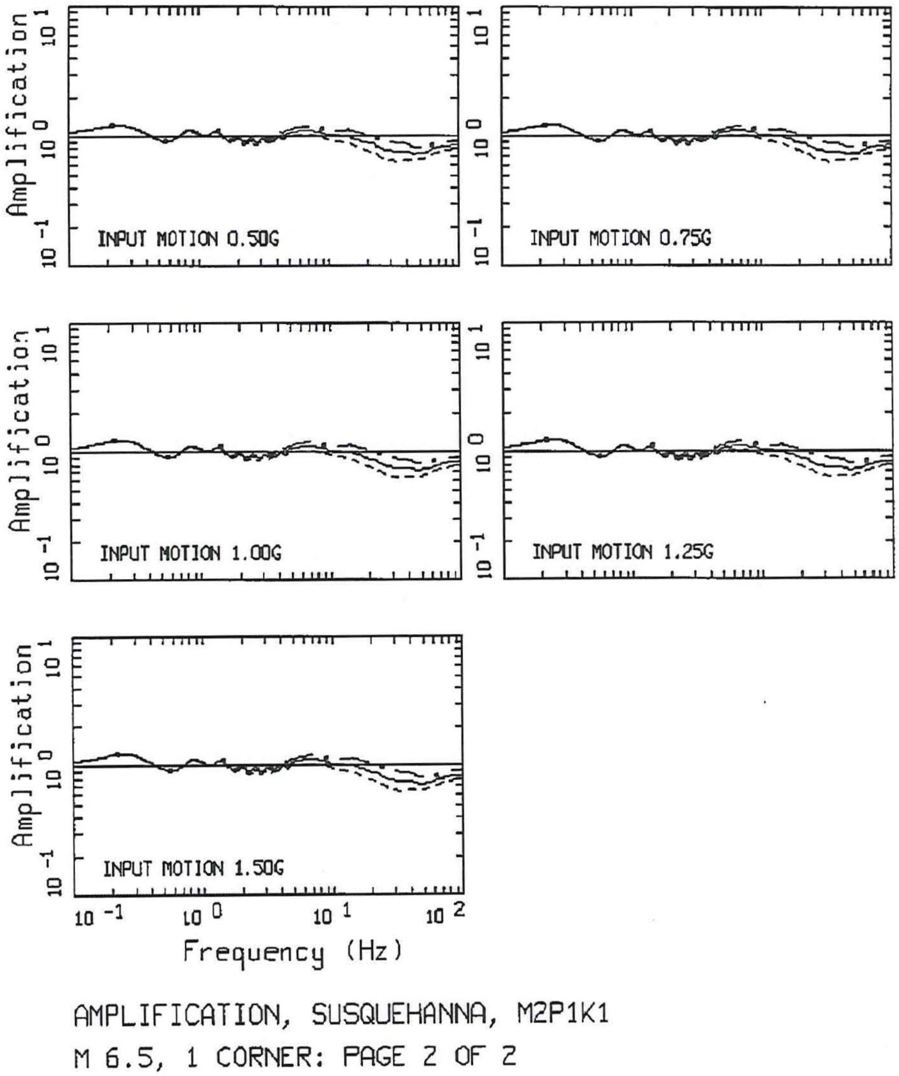

Figure 2.3.6-2 (cont.)

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, 2013a). 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 spectral frequencies for which ground motion equations are available. 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 Susquehanna are shown in Figure 2.3.7-1 for the seven spectral frequencies for which ground motion equations are defined. Tabulated values of mean and fractile seismic hazard curves are provided in Appendix A.

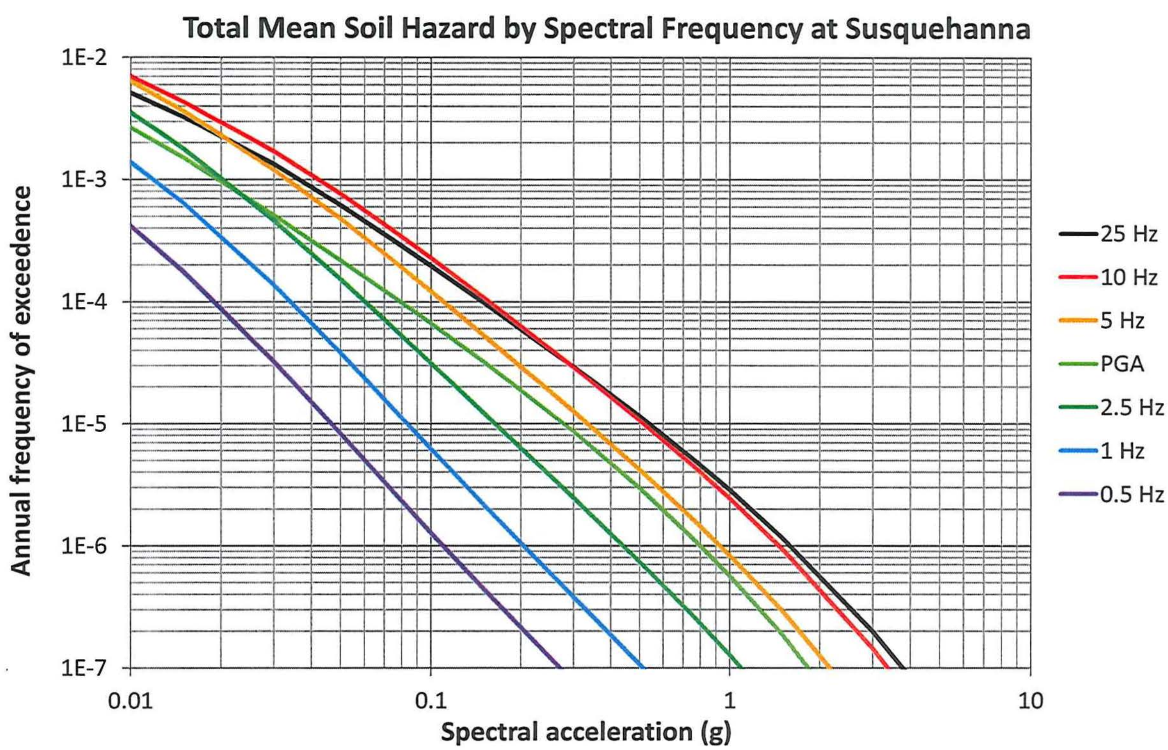

Figure 2.3.7-1
Control Point Mean Hazard Curves for Spectral Frequencies of 0.5, 1, 2.5, 5, 10, 25 and 100 Hz at Susquehanna

2.4 Ground Motion Response Spectrum

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 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 accelerations for each of the seven frequencies.

Table 2.4-1
UHRS and GMRS for Susquehanna

Freq. (Hz)	10^{-4} UHRS (g)	10^{-5} UHRS (g)	GMRS (g)
100	7.87E-02	2.77E-01	1.29E-01
90	7.89E-02	2.78E-01	1.30E-01
80	7.96E-02	2.82E-01	1.31E-01
70	8.18E-02	2.93E-01	1.36E-01
60	8.80E-02	3.24E-01	1.50E-01
50	1.03E-01	3.95E-01	1.81E-01
40	1.26E-01	4.85E-01	2.22E-01
35	1.36E-01	5.16E-01	2.37E-01
30	1.42E-01	5.28E-01	2.44E-01
25	1.48E-01	5.37E-01	2.49E-01
20	1.56E-01	5.48E-01	2.56E-01
15	1.58E-01	5.40E-01	2.53E-01
12.5	1.58E-01	5.30E-01	2.49E-01
10	1.56E-01	5.14E-01	2.43E-01
9	1.50E-01	4.88E-01	2.31E-01
8	1.42E-01	4.55E-01	2.17E-01
7	1.32E-01	4.17E-01	1.99E-01
6	1.22E-01	3.76E-01	1.80E-01
5	1.10E-01	3.32E-01	1.60E-01
4	9.22E-02	2.70E-01	1.31E-01
3.5	8.26E-02	2.37E-01	1.15E-01
3	7.18E-02	2.00E-01	9.79E-02
2.5	6.01E-02	1.63E-01	8.02E-02
2	5.41E-02	1.43E-01	7.08E-02
1.5	4.51E-02	1.16E-01	5.76E-02
1.25	3.90E-02	9.81E-02	4.89E-02
1	3.39E-02	8.32E-02	4.17E-02
0.9	3.23E-02	7.92E-02	3.97E-02

Freq. (Hz)	10^{-4} UHRS (g)	10^{-5} UHRS (g)	GMRS (g)
0.8	2.98E-02	7.32E-02	3.67E-02
0.7	2.62E-02	6.45E-02	3.23E-02
0.6	2.24E-02	5.50E-02	2.76E-02
0.5	1.89E-02	4.65E-02	2.33E-02
0.4	1.51E-02	3.72E-02	1.86E-02
0.35	1.32E-02	3.25E-02	1.63E-02
0.3	1.13E-02	2.79E-02	1.40E-02
0.25	9.44E-03	2.32E-02	1.16E-02
0.2	7.55E-03	1.86E-02	9.31E-03
0.15	5.66E-03	1.39E-02	6.98E-03
0.125	4.72E-03	1.16E-02	5.82E-03
0.1	3.78E-03	9.29E-03	4.66E-03

The 1E-4 and 1E-5 UHRS are used to compute the GMRS at the control point and are shown in Figure 2.4-1.

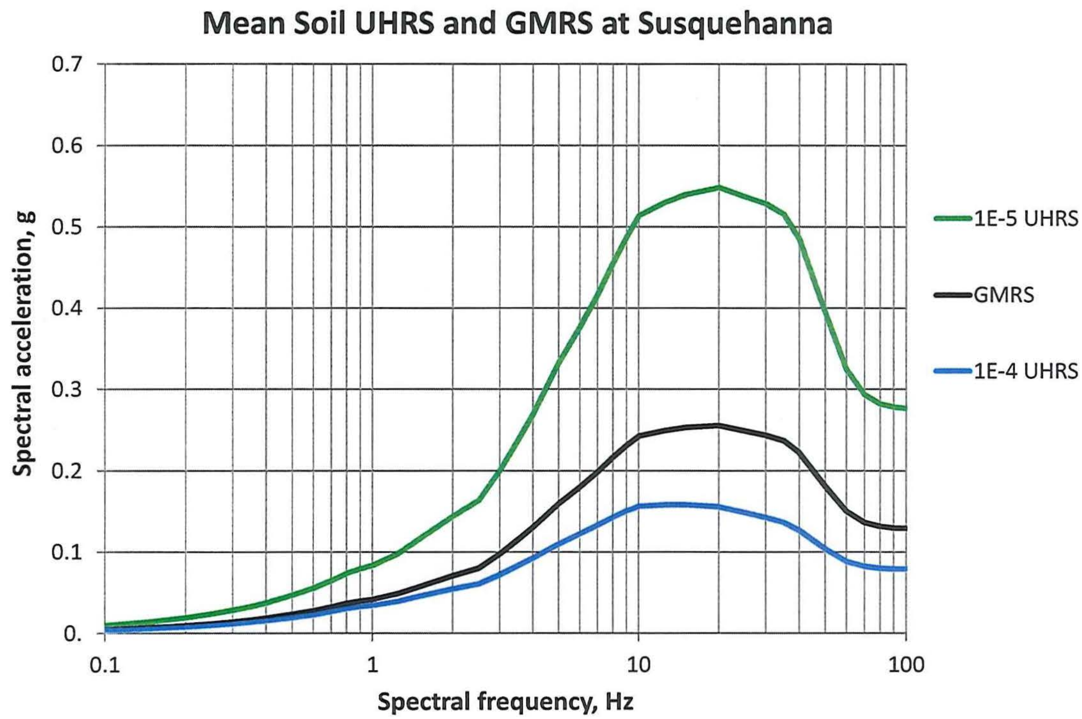

Figure 2.4-1
Plots of 1E-4 and 1E-5 Uniform Hazard Spectra and GMRS
at Control Point for Susquehanna (5%-Damped Response Spectra)

3.0 Plant Design Basis and Beyond Design Basis Evaluation Ground Motion

The design basis for SSES is identified in the Updated Final Safely Evaluation Report (PPL, 2011).

An evaluation for beyond design basis (BDB) ground motions was performed in the IPEEE (PPL, 1994) The IPEEE capacity response spectrum is included below for screening purposes.

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, PPL determined that a design intensity of VI (Modified Mercalli Scale) is adequately conservative for the site. PPL conservatively used a horizontal peak ground acceleration of 0.10g for the SSE to comply with the minimum design requirement of the regulatory agencies. The SSE is defined in terms of a PGA and a design response spectrum. A horizontal peak ground acceleration of 0.10g was used as the anchor point for the SSE. The spectral shape was defined prior to the release of Regulatory Guide 1.60 Rev. 1 (USNRC, 1973). The spectral shape is based on work performed by Nathan Newmark. The design spectral shape is composed of regions that have constant displacement, constant velocity, and constant acceleration. The Zero Period Acceleration starts at a frequency of 33 Hz. Table 3.1-1 shows the control point frequency and spectral acceleration values for the 5% damped horizontal (except Emergency Diesel Generator 'E') SSE while Table 3.1-2 shows the control point frequency and spectral acceleration values for the 5% damped horizontal Emergency Diesel Generator 'E' SSE. (PPL, 2011)

Table 3.1-1
Control Points for Susquehanna (except EDG 'E')
SSE Horizontal Design Response Spectrum

(Hz) Freq	SSE (g)
0.233	0.025
2.0	0.21
6.67	0.21
33.00	0.10
100.00	0.10

Table 3.1-2
Control Points for Susquehanna EDG 'E'
SSE Horizontal Design Response Spectrum

Freq (Hz)	SSE (g)
0.25	0.047
2.5	0.313
9.0	0.261
33.00	0.10
100.00	0.10

3.2 Control Point Elevation

The SSE and the IPEEE high confidence of low probability of failure (HCLPF) Spectrum control point elevation is defined at the top of bedrock (the Mahantango Formation 1) at an elevation of 640 feet above mean sea level (msl).

3.3 IPEEE Description and Capacity Response Spectrum

The IPEEE was performed as a focused scope SMA using the EPRI approach. (PPL, 1994) The IPEEE Adequacy Determination according to SPID Section 3.3.1 is included as Appendix B.

SSES performed a focused scope Seismic Margins Assessment (SMA) utilizing a NUREG/CR0098 median spectral shape for a rock site anchored to a 0.3g PGA. The IPEEE SMA concluded that SSES Units 1 and 2 could be safely shutdown for a Review Level Earthquake (RLE) anchored at 0.3g PGA using the Structures, Systems, and Components associated with the two safe shutdown paths. However, the SMA identified 4 components to have HCLPF values less than 0.3g with the lowest HCLPF value determined to be 0.21 g. For screening purposes, the plant has conservatively used an IHS of 0.21 g.

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

Table 3.3-1
IHS for Susquehanna SES

Freq (Hz)	IHS@0.21g	IHS@0.30g
0.10	0.006	0.0086
0.15	0.014	0.0193
0.20	0.024	0.0344
0.30	0.054	0.0773
0.37	0.075	0.107
0.70	0.142	0.203
1.00	0.203	0.290
1.25	0.253	0.362
1.50	0.304	0.435
1.80	0..365	0.521
2.00	0.406	0.579
2.50	0.445	0.636
3.33	0.445	0.636
4.00	0.445	0.636
5.00	0.445	0.636
5.60	0.445	0.636
6.67	0.445	0.636
8.00	0.445	0.636
10.00	0.396	0.565
13.50	0.337	0.482
20.00	0.274	0.391
33.00	0.210	0.300
100.00	0.210	0.300

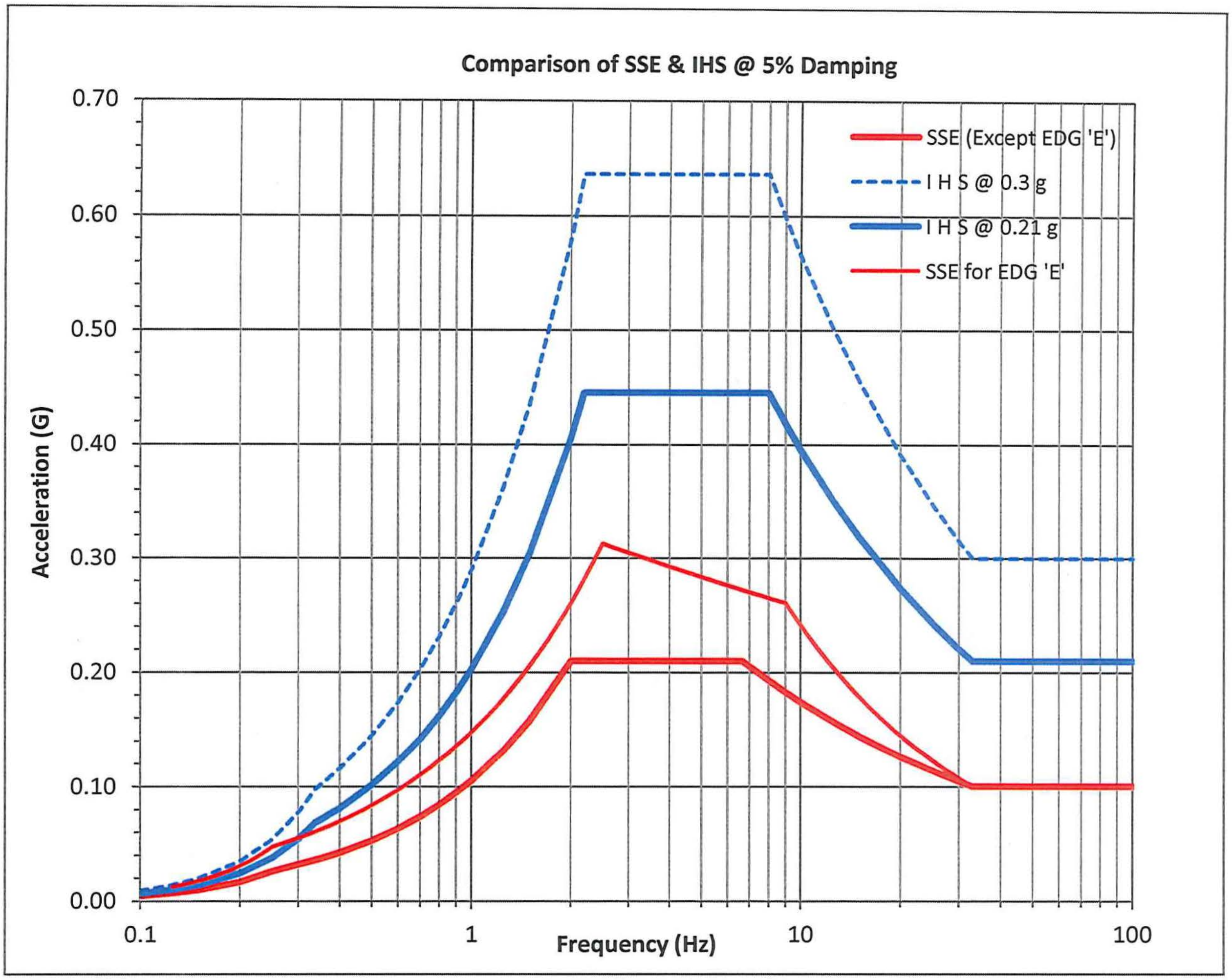

Figure 3.3-1
SSE and IHS Response Spectra for Susquehanna SES

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)

In the 1 to 10 Hz part of the response spectrum, the GMRS exceeds the SSE. However, in the 1 to 10 Hz part of the response spectrum, the IHS anchored at 0.21g exceeds the GMRS. Based on this comparison, a risk evaluation will not be performed. The documentation for fulfilling the IPEEE Adequacy requirements (General Considerations, Prerequisites, and Adequacy Demonstration) as outlined in Section 3.3.1 of the SPID is presented in Appendix B.

The GMRS is enveloped by the SSE response spectrum used for the Emergency Diesel Generator 'E' Building (RG 1.60 Rev. 1 anchored at 0.10g) for frequencies below 10 Hz.

4.2 High Frequency Screening (> 10 Hz)

For a portion of the range above 10 Hz, the GMRS exceeds the IHS @ 0.21 g. Therefore, a high frequency confirmation will be performed.

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

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.

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 (ML13101A379) and agreed to by NRC in a letter dated May 7, 2013 (ML13106A331).

Consistent with NRC letter dated February 20, 2014, (ML14030A046) the seismic hazard reevaluations presented herein are distinct from the current design and licensing bases of Susquehanna SES. Therefore, the results do not call into question the operability or functionality of SSCs and are not reportable pursuant to 10 CFR 50.72, "Immediate notification requirements for operating nuclear power reactors," and 10 CFR 50.73, "Licensee event report system."

The NRC letter also requests that licensees provide an interim evaluation or actions to demonstrate that the plant can cope with the reevaluated hazard while the expedited approach and risk evaluations are conducted. In response to that request, NEI letter dated March 12, 2014, provides seismic core damage risk estimates using the updated seismic hazards for the operating nuclear plants in the Central and Eastern United States. These risk estimates continue to support the following conclusions of the NRC GI-199 Safety/Risk Assessment:

> Overall seismic core damage risk estimates are consistent with the Commission's Safety Goal Policy Statement because they are within the subsidiary objective of

10^{-4}/year for core damage frequency. The GI-199 Safety/Risk Assessment, based in part on information from the U.S. Nuclear Regulatory Commission's (NRC's) Individual Plant Examination of External Events (IPEEE) program, indicates that no concern exists regarding adequate protection and that the current seismic design of operating reactors provides a safety margin to withstand potential earthquakes exceeding the original design basis.

Susquehanna SES is included in the March 12, 2014 risk estimates. Using the methodology described in the NEI letter, all plants were shown to be below 10^{-4}/year; thus, the above conclusions apply.

In response to NRC Information Notice 2010-18 (USNRC, 2010) , PPL performed the following actions to increase the HCLPF values of the 4 items that were reported to have HCLPF values less than 0.3g in the original IPEEE submittal:

1) Removed excessive conservatism associated with the computation of the RLE displacements.
2) Determined that impact during a RLE would not prevent the SSEL item from performing its safety related function.
3) Increased the clear space to the adjacent item in the plant to prevent impact from occurring during a RLE.

These completed actions provided additional assurance that SSES can be safely shutdown using the two IPEEE success paths subsequent to a RLE anchored at 0.3g.

The seismic IPEEE improvements made in combination with the Expedited Seismic Evaluation is noted to be commensurate with the seismic hazard risk. The seismic hazard risk for Susquehanna is quite low as noted by the low spectral acceleration levels of the Ground Motion Response Spectrum.

The Seismic Walkdowns performed for NTTF 2.3 for Unit 1 & 2, did not identify any issues that would impact the ability of any equipment item from performing its safety related function during or after a Design Basis Earthquake. In addition, no open issues were identified with respect to IPEEE commitments. (A small remaining walkdown scope remains for Unit 1's inaccessible items, which will be performed in the Spring of 2014 during a refueling outage.) (PPL, 2012b & 2013)

6.0 Conclusions

In accordance with the 50.54(f) request for information letter a seismic hazard and screening evaluation was performed for SSES. This reevaluation followed the guidance provided in the SPID (EPRI, 2013a) in order to develop a GMRS for the site. The screening evaluation comparison demonstrates that the GMRS exceeds the SSE for frequencies greater than 7 Hz. Based on the screening criteria presented in Figure 1-1 of the SPID, SSES screens out from performing a seismic risk evaluation based on a comparison of the IPEEE HCLPF anchored at 0.21g to the GMRS. The documentation for fulfilling the IPEEE Adequacy requirements (General Considerations, Prerequisites, and Adequacy Demonstration) as outlined in Section 3.3.1 of the SPID is presented in Appendix B. Per the guidance presented in the SPID, a Spent Fuel Pool Integrity Evaluation and a High Frequency Confirmation are required. The results of these evaluations will be transmitted to the NRC at a later date.

Appendix A

Table A-1a. Mean and Fractile Seismic Hazard Curves for 100 Hz at Susquehanna

Table A-1b. Mean and Fractile Seismic Hazard Curves for 25 Hz at Susquehanna

Table A-1c. Mean and Fractile Seismic Hazard Curves for 10 Hz at Susquehanna

Table A-1d. Mean and Fractile Seismic Hazard Curves for 5 Hz at Susquehanna

Table A-1e Mean and Fractile Seismic Hazard Curves for 2.5 Hz at Susquehanna

Table A-1f. Mean and Fractile Seismic Hazard Curves for 1 Hz at Susquehanna

Table A-1 Mean and Fractile Seismic Hazard Curves for 0.5 Hz at Susquehanna

Table A-2 Amplification Functions for Susquehanna

7.0 References

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